

APPENDIX T1

Hydrology and Flooding Technical Report—Volume 1

BORDER TO GOWRIE REVISED DRAFT ENVIRONMENTAL IMPACT STATEMENT

Contents

1	Introduction.....	1
1.1	Inland Rail Program.....	1
1.2	Border to Gowrie alignment.....	1
1.3	Objectives of this report.....	3
1.4	International Independent Panel of Flood Experts	3
1.5	Key Project impact assessment terminology.....	4
1.6	Overlap between flooding and geomorphology.....	8
2	Assessment methodology.....	9
2.1	Pre-construction, early works and construction works stage	9
2.2	Commissioning and operations stage	9
3	Existing environment.....	12
3.1	Waterways.....	12
3.2	Floodplain infrastructure.....	14
3.3	Catchment classification.....	15
3.3.1	Major catchments	15
3.3.2	Moderate catchments	15
3.3.3	Minor catchments	15
3.4	Catchment classification and model summary.....	16
3.4.1	Regional catchments (major and moderate catchments).....	16
3.4.2	Local catchments (minor catchments).....	18
4	Design requirements, standards and guidelines	20
4.1	Hydraulic design criteria	20
4.2	Flood Impact Objectives	20
4.3	Relevant standards and guidelines	23
5	Gowrie Creek.....	24
5.1	Data collection and review—Gowrie Creek.....	24
5.1.1	Previous studies	24
5.1.2	Survey.....	25
5.1.3	Aerial imagery.....	26
5.1.4	Existing drainage structure data	26
5.1.5	Stream gauge data	26
5.1.6	Rainfall data.....	27
5.1.7	Anecdotal and observed flood data.....	28
5.1.8	Site inspection	28
5.2	Development of hydrologic model—Gowrie Creek	28
5.2.1	Model setup	28
5.2.2	Model calibration	30
5.2.3	Review of stream gauges used for model calibration	37
5.2.4	Design event parameters	44
5.2.5	Design hydrograph selection	45
5.3	Development of hydraulic model—Gowrie Creek	47
5.3.1	Model setup	47
5.3.2	Model topography and grid size	48
5.3.3	Sub-grid sampling and Quadtree mesh refinement.....	48

5.3.4	Roughness.....	48
5.3.5	Hydraulic structures.....	49
5.3.6	Boundary conditions.....	53
5.3.7	Critical duration.....	54
5.3.8	Validation against observed flood markers.....	55
5.4	Existing Case modelling results—Gowrie Creek.....	56
5.4.1	Existing Case flood maps.....	56
5.4.2	Existing Case flood inundation extent and flood levels.....	56
5.4.3	Flood immunity of existing infrastructure.....	57
5.4.4	Existing Case velocities.....	57
5.4.5	Existing Case flood hazard.....	57
5.4.6	Existing Case time of inundation.....	57
5.5	Developed Case modelling results—Gowrie Creek.....	58
5.5.1	Drainage structures.....	58
5.5.2	Hydraulic design criteria outcomes.....	58
5.5.3	Flood Impact Objectives outcomes—Gowrie Creek.....	58
5.5.4	Extreme events.....	69
5.5.5	Sensitivity analysis—Gowrie Creek.....	71
5.6	Additional modelling items to be considered in detailed design.....	71
5.7	Planned and future roads.....	73
6	Westbrook and Dry creeks.....	74
6.1	Data collection and review—Westbrook and Dry creeks.....	74
6.1.1	Previous studies.....	74
6.1.2	Survey.....	75
6.1.3	Aerial imagery.....	75
6.1.4	Existing drainage structure data.....	76
6.1.5	Streamflow gauges.....	76
6.1.6	Rainfall data.....	76
6.1.7	Anecdotal and observed flood data.....	76
6.1.8	Site inspection.....	76
6.2	Hydrologic model development—Westbrook and Dry creeks.....	76
6.2.1	Model setup.....	76
6.2.2	Sub-catchments.....	77
6.2.3	Fraction impervious and roughness.....	77
6.2.4	Routing parameters.....	77
6.2.5	Loss model.....	77
6.2.6	Hydrologic modelling—Westbrook and Dry creeks.....	78
6.3	Hydraulic model development—Westbrook and Dry Creeks.....	88
6.3.1	Model setup.....	88
6.3.2	Model topography and grid size.....	88
6.3.3	Roughness.....	89
6.3.4	Hydraulic structures.....	89
6.3.5	Boundary conditions.....	90
6.4	Existing Case modelling results—Westbrook and Dry Creeks.....	90
6.4.1	Hydrologic model peak flow assessment.....	90
6.4.2	Existing Case flood maps.....	92
6.4.3	Existing Case flood inundation extent and flood levels.....	92
6.4.4	Flood immunity of existing infrastructure.....	93
6.4.5	Existing Case velocities.....	93
6.4.6	Existing Case flood hazard.....	93
6.4.7	Existing Case flood time of inundation.....	93

6.5	Developed Case modelling results—Westbrook and Dry creeks.....	94
6.5.1	Drainage structures	94
6.5.2	Hydraulic design criteria outcomes.....	94
6.5.3	Flood Impact Objectives outcomes—Westbrook and Dry Creeks	95
6.5.4	Extreme events.....	103
6.5.5	Sensitivity analysis—Westbrook and Dry creeks	104
6.6	Additional modelling items to be considered in detailed design.....	106
6.7	Planned and future roads	107
7	Condamine River	108
7.1	Data collection and review—Condamine River	108
7.1.1	Previous studies	108
7.1.2	Survey.....	110
7.1.3	Aerial imagery.....	111
7.1.4	Existing drainage structure data	111
7.1.5	Stream gauge data	111
7.1.6	Historical rainfall data	115
7.1.7	Anecdotal and observed flood data.....	117
7.1.8	Site inspection	118
7.2	Hydrologic model development—Condamine River.....	118
7.2.1	Model setup	118
7.2.2	Event selection for calibration.....	119
7.2.3	Model calibration	126
7.2.4	Model validation.....	132
7.2.5	Design event parameters	137
7.2.6	Design hydrograph selection	140
7.2.7	Flow distribution between Condamine River and North Branch.....	144
7.3	Hydraulic model development—Condamine River.....	149
7.3.1	Model setup	149
7.3.2	Model topography and grid size	151
7.3.3	Roughness.....	151
7.3.4	Hydraulic structures.....	152
7.3.5	Boundary conditions	153
7.3.6	Calibration.....	153
7.3.7	Validation	162
7.3.8	Verification	172
7.4	Existing Case modelling results—Condamine River	193
7.4.1	Existing Case flood maps	193
7.4.2	Existing Case flood inundation extent and flood levels	193
7.4.3	Existing Case flood immunity of existing infrastructure	194
7.4.4	Existing Case velocities.....	194
7.4.5	Existing Case hazard.....	194
7.4.6	Existing Case time of inundation	195
7.5	Developed Case modelling results—Condamine River	195
7.5.1	Drainage structures	195
7.5.2	Hydraulic design criteria outcomes.....	196
7.5.3	Flood impact objectives outcomes—Condamine River.....	196
7.5.4	Extreme events.....	216
7.5.5	Sensitivity analysis—Condamine River.....	225
7.6	Additional modelling items to be considered in detailed design.....	244
7.7	Planned and future roads	245

8	Back Creek	246
8.1	Data collection and review—Back Creek	246
8.1.1	Previous studies	246
8.1.2	Survey.....	246
8.1.3	Aerial imagery.....	247
8.1.4	Existing drainage structure data	247
8.1.5	Stream gauge data	247
8.1.6	Rainfall data.....	247
8.1.7	Anecdotal and observed flood data.....	247
8.1.8	Site inspection	247
8.2	Hydrologic model development—Back Creek.....	247
8.2.1	Model setup	247
8.2.2	Sub-catchments.....	248
8.2.3	Design event parameters	248
8.2.4	Model calibration	252
8.2.5	Hydrologic model validation.....	252
8.3	Hydraulic model development—Back Creek.....	253
8.3.1	Model setup	254
8.3.2	Model topography and grid size	254
8.3.3	Roughness.....	255
8.3.4	Hydraulic structures.....	255
8.3.5	Boundary conditions	255
8.3.6	Model calibration	256
8.3.7	Model validation.....	256
8.4	Existing Case modelling results—Back Creek	260
8.4.1	Hydrologic model peak flow assessment	260
8.4.2	Existing Case flood maps	261
8.4.3	Existing Case flood inundation extent and flood levels	261
8.4.4	Flood immunity of existing infrastructure.....	262
8.4.5	Existing Case velocities.....	262
8.4.6	Existing Case flood hazard.....	262
8.4.7	Existing Case flood duration of inundation	262
8.5	Developed Case modelling results—Back Creek.....	263
8.5.1	Drainage structures	263
8.5.2	Hydraulic design criteria outcomes.....	263
8.5.3	Flood Impact Objectives outcomes—Back Creek	263
8.5.4	Extreme events.....	275
8.5.5	Sensitivity analysis—Back Creek	279
8.6	Additional modelling items to be considered in detailed design.....	281
8.7	Planned and future roads	281
9	Nicol Creek	282
9.1	Data collection and review—Nicol Creek	282
9.1.1	Previous studies	282
9.1.2	Survey.....	282
9.1.3	Aerial imagery.....	283
9.1.4	Existing drainage structure data	283
9.1.5	Stream gauge data	283
9.1.6	Rainfall data.....	283
9.1.7	Anecdotal and observed flood data.....	283
9.1.8	Site inspection	283

9.2	Hydrologic model development—Nicol Creek	283
9.2.1	Unified Canning Creek model setup	283
9.2.2	Nicol Creek hydrologic model	288
9.3	Hydraulic model development—Nicol Creek	293
9.3.1	Model setup	294
9.3.2	Model topography and grid size	294
9.3.3	Roughness	295
9.3.4	Hydraulic structures	295
9.3.5	Boundary conditions	295
9.4	Existing Case modelling results—Nicol Creek	296
9.4.1	Hydrologic model peak flow assessment	296
9.4.2	Existing Case flood maps	297
9.4.3	Existing Case flood inundation extent and flood levels	297
9.4.4	Existing Case flood immunity of existing infrastructure	297
9.4.5	Existing Case flood velocities	298
9.4.6	Existing Case flood hazard	298
9.4.7	Existing Case flood duration of inundation	298
9.5	Developed Case modelling results—Nicol Creek	298
9.5.1	Drainage structures	298
9.5.2	Hydraulic design criteria outcomes	299
9.5.3	Flood Impact Objectives outcomes—Nicol Creek	299
9.5.4	Extreme events	302
9.5.5	Sensitivity analysis—Nicol Creek	304
9.6	Additional modelling items to be considered in detailed design	305
9.7	Planned and future roads	306
10	Bringalily Creek	307
10.1	Data collation and review—Bringalily Creek	307
10.1.1	Previous studies	307
10.1.2	Survey	307
10.1.3	Aerial imagery	307
10.1.4	Existing drainage structure	308
10.1.5	Stream gauge data	308
10.1.6	Rainfall data	308
10.1.7	Anecdotal and observed flood data	308
10.1.8	Site inspection	308
10.2	Hydrologic model development—Bringalily Creek	308
10.2.1	Model setup	308
10.2.2	Sub-catchments	308
10.2.3	Design model parameters	309
10.2.4	Event selection for calibration	312
10.2.5	Model calibration	312
10.2.6	Model validation	312
10.3	Hydraulic model development—Bringalily Creek	313
10.3.1	Model setup	314
10.3.2	Model topography and grid size	314
10.3.3	Roughness	315
10.3.4	Hydraulic structures	315
10.4	Existing Case modelling results—Bringalily Creek	316
10.4.1	Hydrological peak flow assessment	316
10.4.2	Existing Case flood maps	317

10.4.3	Existing Case flood inundation extent and flood levels	317
10.4.4	Flood immunity of existing infrastructure	317
10.4.5	Existing Case flood velocities	318
10.4.6	Existing Case flood hazard.....	318
10.4.7	Existing Case flood duration of inundation	318
10.5	Developed Case modelling results—Bringalily Creek	318
10.5.1	Drainage structures	318
10.5.2	Hydraulic design criteria outcomes.....	319
10.5.3	Flood Impact Objectives outcomes—Bringalily Creek	319
10.5.4	Extreme events.....	325
10.5.5	Sensitivity analysis—Bringalily Creek	327
10.6	Additional modelling items to be considered in detailed design.....	329
10.7	Planned and future roads	329
11	Native Dog Creek.....	330
11.1	Data collation and review—Native Dog Creek	330
11.1.1	Previous studies	330
11.1.2	Survey.....	330
11.1.3	Aerial imagery.....	330
11.1.4	Existing drainage structure	331
11.1.5	Stream gauge data	331
11.1.6	Rainfall data.....	331
11.1.7	Anecdotal and observed flood data.....	331
11.1.8	Site inspection	331
11.2	Hydrologic model development—Native Dog Creek	331
11.2.1	Model setup	331
11.2.2	Design model parameters	331
11.2.3	Spatial distribution	332
11.2.4	Rainfall depth estimation	332
11.2.5	Areal reduction factor	332
11.2.6	Temporal patterns	333
11.2.7	Pre-burst.....	333
11.2.8	Design rainfall losses.....	334
11.2.9	Extreme event rainfall.....	334
11.2.10	Event selection for calibration.....	335
11.2.11	Model calibration	335
11.2.12	Model validation.....	335
11.3	Hydraulic model development—Native Dog Creek	336
11.3.1	Model setup	337
11.3.2	Model topography and grid size	337
11.3.3	Roughness.....	338
11.3.4	Hydraulic structures.....	338
11.3.5	Boundary conditions	338
11.4	Existing Case modelling results—Native Dog Creek	339
11.4.1	Hydrologic model peak flow assessment	339
11.4.2	Existing Case flood maps	339
11.4.3	Existing Case flood inundation extent and flood levels	340
11.4.4	Flood immunity of existing infrastructure.....	340
11.4.5	Existing Case velocities.....	340
11.4.6	Existing Case flood hazard.....	340
11.4.7	Existing Case flood time of inundation	340

11.5	Developed Case modelling results—Native Dog Creek.....	341
11.5.1	Drainage structures	341
11.5.2	Hydraulic design criteria outcomes.....	341
11.5.3	Flood Impact Objectives outcomes—Native Dog Creek	341
11.5.4	Extreme events.....	343
11.5.5	Sensitivity analysis—Native Dog Creek	344
11.6	Additional modelling items to be considered in detailed design.....	345
11.7	Planned and future roads	346
12	Cattle Creek.....	347
12.1	Data collection and review—Cattle Creek	347
12.1.1	Previous studies	347
12.1.2	Survey.....	347
12.1.3	Aerial imagery.....	348
12.1.4	Existing drainage structure data	348
12.1.5	Stream gauge data	348
12.1.6	Rainfall data.....	348
12.1.7	Anecdotal and observed flood data.....	348
12.1.8	Site inspection	348
12.2	Hydrologic model development—Cattle Creek	348
12.2.1	Model setup	348
12.2.2	Design model parameters	348
12.2.3	Event selection for calibration.....	352
12.2.4	Model calibration	352
12.2.5	Model validation.....	352
12.3	Hydraulic model development—Cattle Creek	354
12.3.1	Model setup	354
12.3.2	Model topography and grid size	354
12.3.3	Roughness.....	355
12.3.4	Hydraulic structures.....	355
12.3.5	Boundary conditions	356
12.4	Existing Case modelling results—Cattle Creek.....	356
12.4.1	Hydrologic model peak flow assessment	356
12.4.2	Existing Case flood maps	357
12.4.3	Existing Case flood inundation extent and flood levels	357
12.4.4	Flood immunity of existing infrastructure	357
12.4.5	Existing Case velocities.....	358
12.4.6	Existing Case flood hazard.....	358
12.4.7	Existing Case flood time of inundation	358
12.5	Developed Case modelling results—Cattle Creek	358
12.5.1	Drainage structures	358
12.5.2	Hydraulic design criteria outcomes.....	359
12.5.3	Flood Impact Objectives outcomes—Cattle Creek.....	359
12.5.4	Extreme events.....	362
12.5.5	Sensitivity analysis—Cattle Creek.....	363
12.6	Additional modelling items to be considered in detailed design.....	364
12.7	Planned and future roads	365
13	Pariagara Creek	366
13.1	Data collection and review—Pariagara Creek.....	366
13.1.1	Previous studies	366
13.1.2	Survey.....	366

13.1.3	Aerial imagery.....	367
13.1.4	Existing drainage structure data	367
13.1.5	Stream gauge data	367
13.1.6	Rainfall data.....	367
13.1.7	Anecdotal and observed flood data.....	367
13.1.8	Site inspection	367
13.2	Hydrologic model development—Pariagara Creek	367
13.2.1	Model setup	367
13.2.2	Sub-catchments.....	368
13.2.3	Design event parameters	369
13.2.4	Model calibration	372
13.2.5	Hydrologic model validation.....	372
13.3	Hydraulic model development—Pariagara Creek	374
13.3.1	Model setup	375
13.3.2	Model topography and grid size	375
13.3.3	Roughness.....	376
13.3.4	Hydraulic structures.....	376
13.3.5	Boundary conditions	376
13.4	Existing Case modelling results—Pariagara Creek.....	377
13.4.1	Hydrologic model peak flow assessment	377
13.4.2	Existing Case flood maps.....	382
13.4.3	Existing Case flood inundation extent and flood levels	383
13.4.4	Flood immunity of existing infrastructure.....	383
13.4.5	Existing Case velocities.....	383
13.4.6	Existing Case flood hazard.....	383
13.4.7	Existing Case flood duration of inundation	383
13.5	Developed Case modelling results—Pariagara Creek.....	384
13.5.1	Drainage structures	384
13.5.2	Hydraulic design criteria outcomes.....	384
13.5.3	Flood Impact Objectives outcomes—Pariagara Creek	384
13.5.4	Extreme events.....	396
13.5.5	Sensitivity analysis—Pariagara Creek	398
13.6	Additional modelling items to be considered in detailed design.....	400
13.7	Planned and future roads	401
14	Macintyre Brook—Yelarbon to Inglewood.....	402
14.1	Data collection and review: Macintyre Brook—Yelarbon to Inglewood.....	402
14.1.1	Previous studies	402
14.1.2	Survey.....	406
14.1.3	Aerial imagery.....	407
14.1.4	Reservoirs.....	407
14.1.5	Existing drainage structure data	407
14.1.6	Stream gauge data	408
14.1.7	Rainfall data.....	409
14.1.8	Anecdotal and observed flood data.....	409
14.1.9	Site inspection	409
14.2	Hydrologic model development: Macintyre Brook— Yelarbon to Inglewood	409
14.2.1	Hydrologic modelling	410
14.2.2	Model validation.....	411
14.2.3	Flood frequency analysis.....	411
14.2.4	Data extraction for 2021 event	411
14.2.5	Validation model parameters.....	411

14.2.6	Validation event model results.....	412
14.3	Hydraulic model development: Macintyre Brook—Yelarbon to Inglewood	415
14.3.1	Model setup	415
14.3.2	Model topography and grid size	416
14.3.3	Sub-grid sampling and Quadtree mesh refinement.....	416
14.3.4	Existing hydraulic structures.....	416
14.3.5	Roughness.....	417
14.3.6	Boundary conditions	418
14.3.7	Calibration.....	418
14.3.8	Validation	424
14.3.9	Calibration summary.....	426
14.4	Existing Case modelling results: Macintyre Brook—Yelarbon to Inglewood.....	427
14.4.1	Hydrologic model peak flow assessment	427
14.4.2	Existing Case flood maps	428
14.4.3	Existing Case flood inundation extent and flood levels	429
14.4.4	Existing Case flood immunity of existing infrastructure	429
14.4.5	Existing Case velocities.....	429
14.4.6	Existing Case hazard.....	430
14.4.7	Existing Case time of inundation	430
14.5	Developed Case modelling results: Macintyre Brook—Yelarbon to Inglewood	430
14.5.1	Drainage structures	430
14.5.2	Proposed levee at Yelarbon	431
14.5.3	Hydraulic design criteria outcomes.....	432
14.5.4	Flood Impact Objectives outcomes—Macintyre Brook Yelarbon to Inglewood	432
14.5.5	Extreme events.....	462
14.5.6	Sensitivity analysis—Macintyre Brook.....	464
14.6	Additional modelling items to be considered in detailed design.....	468
14.7	Planned and future roads	468
15	Macintyre Brook at Bybera Road	469
15.1	Data collection and review—Macintyre Brook at Bybera Road.....	469
15.1.1	Previous studies	469
15.1.2	Survey.....	469
15.1.3	Aerial imagery.....	470
15.1.4	Existing drainage structure data	470
15.1.5	Stream gauge data	470
15.1.6	Rainfall data.....	470
15.1.7	Anecdotal and observed flood data	470
15.1.8	Site inspection	470
15.2	Hydrologic model development—Macintyre Brook at Bybera Road	470
15.2.1	Model setup	470
15.2.2	Sub-catchments.....	471
15.2.3	Design event parameters	471
15.2.4	Event selection for calibration.....	474
15.2.5	Model calibration	474
15.2.6	Model validation.....	474
15.3	Hydraulic model development—Macintyre Brook at Bybera Road	476
15.3.1	Model setup	476
15.3.2	Model topography and grid size	476
15.3.3	Roughness.....	477
15.3.4	Hydraulic structures.....	477

15.3.5	Boundary conditions	477
15.4	Existing Case modelling results—Macintyre Brook at Bybera Road	478
15.4.1	Hydrologic model peak flow assessment	478
15.4.2	Existing Case flood maps	479
15.4.3	Existing Case flood inundation extent and flood levels	479
15.4.4	Existing Case flood immunity of existing infrastructure	479
15.4.5	Existing Case velocities	479
15.4.6	Existing Case flood hazard	480
15.4.7	Existing Case flood duration of inundation	480
15.5	Developed Case modelling results—Macintyre Brook at Bybera Road	480
15.5.1	Drainage structures	480
15.5.2	Hydraulic design criteria outcomes	480
15.5.3	Flood Impact Objectives outcomes—Macintyre Brook at Bybera Road	481
15.5.4	Extreme events	486
15.5.5	Sensitivity analysis—Macintyre Brook at Bybera Road	487
15.6	Additional modelling items to be considered in detailed design	488
15.7	Planned and future roads	489
16	Macintyre Brook at Cremascos Road	490
16.1	Data collection and review—Macintyre Brook at Cremascos Road	490
16.1.1	Previous studies	490
16.1.2	Survey	490
16.1.3	Aerial imagery	491
16.1.4	Existing drainage structure data	491
16.1.5	Stream gauge data	491
16.1.6	Rainfall data	491
16.1.7	Anecdotal and observed flood data	491
16.1.8	Site inspection	491
16.2	Hydrologic model development—Macintyre Brook at Cremascos Road	491
16.2.1	Model setup	491
16.2.2	Sub-catchments	492
16.2.3	Design event parameters	492
16.2.4	Event selection for calibration	495
16.2.5	Model calibration	495
16.2.6	Hydrologic model validation	495
16.3	Hydraulic model development—Macintyre Brook at Cremascos Road	496
16.3.1	Model setup	497
16.3.2	Model topography and grid size	497
16.3.3	Roughness	498
16.3.4	Hydraulic structures	498
16.3.5	Boundary conditions	498
16.4	Existing Case modelling results—Macintyre Brook at Cremascos Road	499
16.4.1	Hydrologic model peak flow assessment	499
16.4.2	Existing Case flood maps	499
16.4.3	Existing Case flood inundation extent and flood levels	500
16.4.4	Existing Case flood immunity of existing infrastructure	500
16.4.5	Existing Case velocities	500
16.4.6	Existing Case flood hazard	500
16.4.7	Existing Case flood duration of inundation	501
16.5	Developed Case modelling results—Macintyre Brook at Cremascos Road	501
16.5.1	Drainage structures	501
16.5.2	Hydraulic design criteria outcomes	501

16.5.3	Flood Impact Objectives outcomes—Macintyre Brook at Cremascos Road	502
16.5.4	Extreme events.....	507
16.5.5	Sensitivity analysis—Macintyre Brook at Cremascos Road.....	509
16.6	Additional modelling items to be considered in detailed design.....	510
16.7	Planned and future roads	511
17	Macintyre River	512
17.1	Data collection and review.....	512
17.1.1	Previous studies	512
17.1.2	Survey.....	516
17.1.3	Planning information.....	517
17.1.4	Reservoirs.....	517
17.1.5	Aerial imagery.....	517
17.1.6	Existing drainage structure data	517
17.1.7	Stream gauge data	518
17.1.8	Rainfall data.....	520
17.1.9	Anecdotal flood data.....	521
17.1.10	Site inspection	521
17.2	Hydrologic model development—Macintyre River	521
17.2.1	Model setup	521
17.2.2	Event selection for calibration.....	522
17.2.3	Model calibration	523
17.2.4	Model validation.....	536
17.2.5	Design model parameters	536
17.2.6	Baseflow	542
17.2.7	Hydrologic model validation.....	543
17.3	Hydraulic model development—Macintyre River	554
17.3.1	Model setup	554
17.3.2	Model topography and grid size	555
17.3.3	Existing hydraulic structures.....	555
17.3.4	Roughness.....	556
17.3.5	Boundary conditions	556
17.3.6	Calibration.....	556
17.3.7	Validation	570
17.4	Existing Case modelling results—Macintyre River.....	571
17.4.1	Hydrologic model peak flow assessment	571
17.4.2	Existing Case flood maps	572
17.4.3	Flood inundation extent and flood levels	572
17.4.4	Flood immunity of existing infrastructure.....	572
17.4.5	Existing Case velocities.....	573
17.4.6	Existing Case hazard.....	573
17.4.7	Existing Case time of inundation	573
17.5	Developed Case modelling results—Macintyre River	573
17.5.1	Drainage structures	573
17.5.2	Hydraulic design criteria outcomes.....	574
17.5.3	Flood Impact Objectives outcomes—Macintyre River	574
17.5.4	Extreme events.....	579
17.5.5	Sensitivity analysis—Macintyre River.....	581
17.6	Additional modelling items to be considered in detailed design.....	583
17.7	Planned and future roads	583

18	Local catchment assessment.....	584
18.1	Local catchment hydrology.....	584
18.1.1	Local cross-drainage	584
18.1.2	Longitudinal drainage	585
18.2	Local catchment hydraulic design	585
18.2.1	Local cross-drainage	585
18.2.2	Longitudinal drainage hydraulic design	586
18.2.3	Diversion drains	586
18.3	Fish passage	586
18.4	Stormwater quality management.....	587
18.5	Existing Case modelling results – Local catchments	588
18.5.1	Existing Case flood inundation extent and flood levels	588
18.5.2	Existing Case flood velocities	588
18.5.3	Existing Case flood hazard.....	588
18.5.4	Existing Case time of inundation	589
18.6	Developed Case modelling results—Local catchments	589
18.6.1	Drainage structures	589
18.6.2	Hydraulic design criteria outcomes.....	590
18.6.3	Flood Impact Objectives outcomes—Local catchments	590
18.7	Additional modelling items to be considered in detailed design.....	619
18.8	Planned and future roads	619
19	Emergency access route assessment	620
19.1	Toowoomba Regional Council.....	620
19.1.1	Millmerran and Pampas.....	620
19.1.2	Brookstead.....	624
19.1.3	Southbrook	626
19.1.4	Wellcamp	628
19.1.5	Gowrie Mountain	630
19.1.6	Gowrie Junction.....	632
19.2	Goondiwindi Regional Council.....	632
19.2.1	Goondiwindi and Yelarbon	632
19.2.2	Inglewood	638
19.3	Ongoing consultation.....	640
20	Temporary footprint assessment.....	641
20.1	Site-selection criteria	641
20.2	Whetstone Material Distribution Centre.....	641
20.2.1	Site location	641
20.2.2	Results.....	643
20.2.3	Conclusion and mitigation measures	655
20.3	Non-resident workforce accommodations	655
20.3.1	Yelarbon workforce accommodation	656
20.3.2	Inglewood workforce accommodation	664
20.4	Turallin facility	676
20.4.1	Site location	676
20.4.2	Hydrology.....	679
20.4.3	Hydraulics	682
20.4.4	Results.....	686
20.4.5	Conclusion and mitigation measures	692

20.5	Laydown areas and other temporary Project facilities.....	692
20.5.1	Laydown areas	692
20.5.2	Temporary wastewater treatment plants and chemical storage.....	697
20.6	Pre-construction, early works and construction works flood immunity.....	698
21	Embankment overtopping failure risk assessment	700
21.1	Risk-to-life assessment	701
21.1.1	Methodology	701
21.1.2	Assessment results	706
21.1.3	Conclusions and mitigations.....	708
21.2	Design measures to mitigate overtopping failure	709
21.2.1	Formation, earthworks and cross-drainage design measures	709
21.2.2	Recommendations for risk mitigation in future design stages.....	711
21.3	Flood emergency response measures	711
21.4	Conclusion	712
22	Mitigation measures and strategies	713
22.1	Mitigation—Revised reference design stage.....	713
22.2	Overlap between geomorphology and flooding.....	714
22.3	Proposed mitigation framework—Future Project stages.....	714
22.4	Mitigation strategy for Flood Impact Objective exceedances.....	719
22.4.1	Design development, refinement and treatment	719
22.4.2	At-property treatments.....	735
22.4.3	Monitor.....	736
23	Limitations.....	742
24	Conclusions	743
25	References	748

Appendices

Appendix A

Independent International Panel of Experts for Flood Studies—Issues Management Register

Appendix B

Cross-Drainage Register

Appendix C

Macintyre River Hydrologic Model Update Mapping

Appendix D

SGM Environmental—Erosion Threshold Velocity Technical Note

Figures

Figure 1.1	Project alignment
Figure 3.1	Creeks and river systems
Figure 3.2	Regional model extents
Figure 3.3	Local drainage model extents
Figure 5.1	InterlinkSQ development terrain representation
Figure 5.2	Stream gauge locations: Cranley and Oakey

Figure 5.3	Comparison of XP-RAFTS and gauge results at Gowrie Creek at Cranley (422326A)—December 2010 event
Figure 5.4	Comparison of XP-RAFTS and gauge results at Gowrie Creek at Oakey (422332B)—December 2010 event
Figure 5.5	Comparison of XP-RAFTS and gauge results at Gowrie Creek at Cranley (422326A)—January 2011 event
Figure 5.6	Comparison of XP-RAFTS and gauge results at Gowrie Creek at Oakey (422332B)—January 2011 event
Figure 5.7	Comparison of XP-RAFTS and gauge results at Gowrie Creek at Cranley (422326A)—January 2013 event
Figure 5.8	Comparison of XP-RAFTS and gauge results at the Gowrie Creek at Oakey (422332B)—January 2013 event
Figure 5.9	Comparison of XP-RAFTS and gauge results at Gowrie Creek at Cranley (422326A)—February 2013 event
Figure 5.10	Comparison of XP-RAFTS and gauge results at the Gowrie Creek at Oakey (422332B)—February 2013 event
Figure 5.11	Cranley gauge cross-section (source: DRDMW) and reliable range based on visual inspection (blue)
Figure 5.12	Cranley—Comparison between DRDMW and TUFLOW rating curves
Figure 5.13	Cranley—Adopted annual maxima series
Figure 5.14	Revised flood frequency analysis at Cranley gauge based on LP3 model
Figure 5.15	Oakey gauge cross-section
Figure 5.16	Oakey—Comparison between DRDMW and TUFLOW rating curves
Figure 5.17	Oakey—Adopted annual maxima series
Figure 5.18	Revised flood frequency analysis at Oakey gauge based on LP3 model
Figure 5.19	Correlation of annual peak streamflow data between Cranley and Oakey—Black dashed line: 1:1 relationship; Green shaded area: in-channel events at Oakey
Figure 5.20	Gowrie Creek—Example of local sub-catchment inflow proportions
Figure 5.21	Gowrie Creek—5% AEP critical duration assessment—Existing Case
Figure 5.22	Gowrie Creek—1% AEP critical duration assessment—Existing Case
Figure 5.23	Gowrie Creek—Potentially impacted State-controlled roads
Figure 5.24	Gowrie Creek—Potentially impacted local public roads
Figure 5.25	Gowrie Creek—Potentially impacted existing rail infrastructure
Figure 6.1	Westbrook and Dry creeks catchment—Comparison of sub-catchment S1.08 modelled hydrographs in TUFLOW and RAFTS models—storm 4.5 hrs burst 9
Figure 6.2	Westbrook and Dry creeks catchment—Comparison of sub-catchment S1.10 modelled hydrographs in TUFLOW and RAFTS models— storm 4.5 hrs burst 9
Figure 6.3	Westbrook and Dry creeks catchment—Comparison of sub-catchment S3.04 modelled hydrographs in TUFLOW and RAFTS models— storm 4.5 hrs burst 9
Figure 6.4	Westbrook and Dry creeks catchment—Comparison of sub-catchment S1.05 modelled hydrographs in TUFLOW and RAFTS models—storm 4.5 hrs burst 9
Figure 6.5	Westbrook and Dry creeks—Impacted State-controlled roads
Figure 6.6	Westbrook and Dry creeks—Impacted local public roads
Figure 7.1	Talgai TW gauge location
Figure 7.2	Tummaville gauge location
Figure 7.3	Condamine River—Estimated proliferation of ring tanks within the regional flood model extent
Figure 7.4	Condamine River catchment—Gridded rainfall depths during 1991 event
Figure 7.5	Condamine River catchment—Gridded rainfall depths during 2010 event
Figure 7.6	Condamine River catchment—Gridded rainfall depths during 2013 event
Figure 7.7	Condamine River catchment—Gridded rainfall depths during 2021 event
Figure 7.8	Calibration hydrographs at Warwick 422310C—1991 event
Figure 7.9	Calibration hydrographs at Talgai TW 422355A—1991 event
Figure 7.10	Calibration hydrographs at Condamine River at Tummaville 422323A—1991 event
Figure 7.11	Calibration hydrographs at combined 'outlet'—1991 event

Figure 7.12	Calibration hydrographs at Warwick 422310C—2010 event
Figure 7.13	Calibration hydrographs at Talgai TW 422355A—2010 event
Figure 7.14	Calibration hydrographs at Condamine River at Tummaville 423323A—2010 event
Figure 7.15	Calibration hydrographs at combined ‘outlet’—2010 event
Figure 7.16	Validation hydrographs at Warwick 422310C—2013 event
Figure 7.17	Validation hydrographs at Talgai TW 422355A—2013 event
Figure 7.18	Validation hydrographs at Tummaville 423323A—2013 event
Figure 7.19	Validation hydrographs at combined ‘outlet’—2013 event
Figure 7.20	Validation hydrographs at Warwick 422310C—2021 event
Figure 7.21	Validation hydrographs at Talgai TW 422355A—2021 event
Figure 7.22	Validation hydrographs at Tummaville 423323A—2021 event
Figure 7.23	Validation hydrographs at combined ‘outlet’—2021 event
Figure 7.24	Condamine River—Adjusted design event losses
Figure 7.25	Flood frequency analysis at Warwick (422310C)
Figure 7.26	Condamine River—Combined flood frequency analysis at URBS model outlet (422316A and 422345A)
Figure 7.27	Condamine River—Critical durations for the 1% AEP event
Figure 7.28	Condamine River—Selected temporal patterns for the 1% AEP 2880-minute event (TUFLOW)
Figure 7.29	Condamine River—Flood level differences for the 1% AEP 2880-minute event: Temporal pattern 4 (selected in URBS); Temporal pattern 1 (selected in TUFLOW)
Figure 7.30	Lone Pine TUFLOW model configuration
Figure 7.31	Adopted revised rating curve at Lone Pine
Figure 7.32	Condamine River and North Branch—2010 event flood extent sensitivity to increases in hydraulic model inflows
Figure 7.33	Hydraulic model calibration hydrographs at Tummaville 422323A—1991 event
Figure 7.34	Hydraulic model calibration hydrographs at Yarramalong Weir 422353A—1991 event
Figure 7.35	Hydraulic model calibration hydrographs at Pampas 422347B—1991 event
Figure 7.36	Hydraulic model calibration hydrographs at Cecil Weir 422316A—1991 event
Figure 7.37	Hydraulic model calibration hydrographs at Tummaville 422323A—2010 event
Figure 7.38	Hydraulic model calibration hydrographs at Yarramalong Weir 422353A—2010 event
Figure 7.39	Hydraulic model calibration hydrographs at Pampas 422347B—2010 event
Figure 7.40	Hydraulic model calibration hydrographs at Cecil Weir 422316A—2010 event
Figure 7.41	Hydraulic model calibration hydrographs at Tummaville 422323A—2013 event
Figure 7.42	Hydraulic model calibration hydrographs at Yarramalong Weir 422353A—2013 event
Figure 7.43	Hydraulic model calibration hydrographs at Pampas 422347B—2013 event
Figure 7.44	Hydraulic model calibration hydrographs at Cecil Weir 422316A—2013 event
Figure 7.45	Hydraulic model calibration hydrographs at Tummaville 422323A—2021 event
Figure 7.46	Hydraulic model calibration hydrographs at Yarramalong Weir 422353A—2021 event
Figure 7.47	Hydraulic model calibration hydrographs at Pampas 422347B—2021 event
Figure 7.48	Hydraulic model calibration hydrographs at Cecil Weir 422316A—2021 event
Figure 7.49	Condamine River—Flood marker locations and relative quality
Figure 7.50	Condamine River—2010 event validation to FFJV flood markers
Figure 7.51	Condamine River—2010 event validation to Expert Flood Panel flood markers
Figure 7.52	Condamine River—Spatial distribution of flood marker validation results
Figure 7.53	Manual gauged readings at Centenary Bridge (041472) during the Dec 2010 flood event
Figure 7.54	Condamine River—Impacts to State-controlled roads
Figure 7.55	Condamine River—Impacts to local public roads
Figure 7.56	Condamine River—Impacted existing rail infrastructure
Figure 7.57	Condamine River floodplain—Evidence of historical debris deposition and scour provided by Queensland Rail (actual flood event unknown)
Figure 7.58	Condamine River—Flood level sensitivity to local inflow approach – 1% AEP: default approach minus ‘SA ALL’ approach

- Figure 7.59 Condamine River—Afflux sensitivity to local inflow approach—1% AEP: default SA approach afflux minus ‘SA ALL’ approach afflux
- Figure 7.60 Condamine River—Crop variability effects on flood level—1% AEP 2880 min: single uniform roughness: 6th lowest ranked sensitivity test result (i.e. one above median)
- Figure 7.61 Condamine River—Crop variability effects on flood extent—1% AEP 2880 min (reproduced from the draft EIS)
- Figure 7.62 Condamine River—Crop variability effects on flood depth—1% AEP event (reproduced from the draft EIS)
- Figure 7.63 Condamine River—Crop variability effects on flood sensitive receptors
- Figure 7.64 Regional flood event extents (blue hatch) without Back Creek contribution (Scenario 1) (1% AEP event)
- Figure 7.65 Influence of local Back Creek model inflows to peak flood levels—assessing the difference of Scenarios 3 and 1 and the difference of Scenarios 2 and 1 (1% AEP event)
- Figure 7.66 Impact of adopting Back Creek inflows—Frequency distribution of peak flood level changes at flood markers and sensitive receptors (1% AEP event)
- Figure 8.1 Flood markers within Back Creek catchment in 2010 flood event
- Figure 8.2 Back Creek—December 2010 flood event
- Figure 8.3 Back Creek—Potentially impacted State-controlled roads
- Figure 8.4 Back Creek—Potentially impacted local public roads
- Figure 8.5 Back Creek—Potentially impacted existing rail infrastructure
- Figure 9.1 Estimate of flows at the Canning Creek model outlet
- Figure 9.2 Estimate of flows at the Nicol Creek Project crossing
- Figure 10.1 Estimate of flows at the Bringalily Creek crossing
- Figure 10.2 Bringalily Creek—Impacted State-controlled roads
- Figure 11.1 Estimate of flows at the Native Dog Creek Project crossing
- Figure 12.1 Estimate of flows at the Cattle Creek crossing
- Figure 12.2 Cattle Creek—Impacted local public roads
- Figure 13.1 Estimate of flows at the Pariagara Creek crossing
- Figure 13.2 Critical storm durations in Pariagara Creek flood model
- Figure 13.3 Pariagara Creek—Flood level sensitivity to storm duration—1% AEP: Ensemble maximum scenario minus single critical storm scenario
- Figure 13.4 Rank 6 temporal patterns on Pariagara Creek floodplain
- Figure 13.5 Pariagara Creek—Flood level sensitivity to temporal patterns—1% AEP 720-minute: Ensemble rank 6 temporal pattern minus adopted single temporal pattern
- Figure 13.6 1% AEP afflux sensitivity at Ch 66.23 km—Comparison between draft EIS case (left) and sensitivity test case (right)
- Figure 13.7 Pariagara Creek—Potentially impacted State-controlled Roads
- Figure 13.8 Pariagara Creek—Potentially impacted local public roads
- Figure 14.1 December 2021 calculated (blue) and rated (red) hydrographs—Barongarook
- Figure 14.2 December 2021 calculated (blue) and rated (red) hydrographs—Terraine
- Figure 14.3 December 2021 calculated (blue) and rated (red) hydrographs—Coolmunda
- Figure 14.4 December 2021 calculated (blue) and rated (red) hydrographs—Inglewood
- Figure 14.5 December 2021 calculated (blue) and rated (red) hydrographs—Booba Sands
- Figure 14.6 Inglewood and Booba Sands—Comparison of recorded and modelled gauge ratings
- Figure 14.7 1976 Flood recorded and modelled gauge levels at Inglewood
- Figure 14.8 Inglewood and Booba Sands—1996 Flood recorded and modelled gauge levels
- Figure 14.9 Inglewood and Booba Sands—2011 Flood recorded and modelled gauge levels
- Figure 14.10 Inglewood and Booba Sands: 2021 Stream gauge—model results compared to rated data
- Figure 14.11 Inglewood and Booba Sands—2021 flood recorded and modelled gauge levels
- Figure 14.12 Yelarbon levee—Proposed raise heights
- Figure 14.13 Macintyre Brook—Impacted State-controlled roads at Yelarbon
- Figure 14.14 Macintyre Brook—Impacted State-controlled roads at Inglewood
- Figure 14.15 Macintyre Brook—Impacted local public roads at Yelarbon

Figure 14.16	Macintyre Brook—Impacted local public roads at Whetstone
Figure 14.17	Macintyre Brook—Impacted local public roads at Inglewood
Figure 14.18	Macintyre Brook—Impacted existing rail infrastructure at Yelarbon
Figure 14.19	Macintyre Brook—Impacted existing rail infrastructure
Figure 15.1	Estimate of flows at the outlet of Macintyre Brook at Bybera Road model
Figure 15.2	Macintyre Brook at Bybera Road— Impacted local public roads
Figure 16.1	Estimate of flows at the outlet of Macintyre Brook at Cremascos Road model
Figure 16.2	Macintyre Brook at Cremascos Road—Impacted local public roads
Figure 17.1	Rainfall gauge between Ashford/Wallangra and Holdfast for 1996 event
Figure 17.2	1976 flood hydrograph at Inglewood calculated using 416402B and current (416402C) ratings
Figure 17.3	Macintyre River—Storage-discharge relationships for major reservoirs
Figure 17.4	Comparison of Coolmunda storage volume against tailwater gauge flow
Figure 17.5	Flood frequency analysis fit and effect of Coolmunda VBF on the DEA flows at Inglewood
Figure 17.6	Effect of Pindari Dam on the DEA flows at Ashford
Figure 17.7	Macintyre River to Holdfast—Reconciliation
Figure 17.8	Dumaresq River to Glenarvon –Reconciliation
Figure 17.9	Macintyre Brook to Booba Sands –Reconciliation
Figure 17.10	Control weir and comparison of flow gauging and current gauge rating at Coolatai
Figure 17.11	Otteleys Creek to Goondiwindi—Reconciliation
Figure 17.12	Macintyre River catchment—Effect of losses on concentrated versus uniform rainfall
Figure 17.13	Relationship between total flow upstream of Boggabilla and Macintyre River flow at Holdfast
Figure 17.14	Relationship between total flow upstream of Boggabilla and Dumaresq River flow at Bonshaw
Figure 17.15	Relationship between total flow upstream of Boggabilla and Macintyre Brook flow at Inglewood
Figure 17.16	Boggabilla stream gauge—Model results compared to rated data
Figure 17.17	Boggabilla gauge—1976 flows recorded and modelled
Figure 17.18	Boggabilla gauge—1976 levels recorded and modelled (time series not available for 1976 recorded level)
Figure 17.19	Boggabilla gauge—1996 flows recorded and modelled
Figure 17.20	Boggabilla gauge—1996 levels recorded and modelled
Figure 17.21	Boggabilla gauge—2011 flows recorded and modelled
Figure 17.22	Boggabilla gauge—2011 levels recorded and modelled
Figure 17.23	Goondiwindi stream gauge—Model results compared to rated data
Figure 17.24	Goondiwindi gauge—1976 flows recorded and modelled
Figure 17.25	Goondiwindi gauge—1976 levels recorded and modelled
Figure 17.26	Goondiwindi gauge—1996 flows recorded and modelled
Figure 17.27	Goondiwindi gauge—1996 levels recorded and modelled
Figure 17.28	Goondiwindi gauge—2011 flows recorded and modelled
Figure 17.29	Goondiwindi gauge—2011 levels recorded and modelled
Figure 17.30	Boggabilla Gauge—Section 1 from BRVFMP
Figure 20.1	Whetstone MDC site –Location Plan
Figure 20.2	Whetstone MDC site—10% AEP peak flood depth
Figure 20.3	Whetstone MDC site—5% AEP peak flood depth
Figure 20.4	Whetstone MDC site—1% AEP peak flood depth
Figure 20.5	Whetstone MDC site—10% AEP peak flood flow velocities
Figure 20.6	Whetstone MDC site—5% AEP peak flood flow velocities
Figure 20.7	Whetstone MDC site—1% AEP peak flood flow velocities
Figure 20.8	Whetstone MDC site—10% AEP peak flow velocity × depth product
Figure 20.9	Whetstone MDC site—5% AEP peak flow velocity × depth product
Figure 20.10	Whetstone MDC site—1% AEP peak flow velocity × depth product
Figure 20.11	Yelarbon site—Locality plan

Figure 20.12	Yelarbon site—1% AEP peak flood depth
Figure 20.13	Yelarbon site –1% AEP peak flood flow velocities
Figure 20.14	Yelarbon site –1% AEP peak flow velocity × depth product
Figure 20.15	Inglewood site –Locality plan
Figure 20.16	Inglewood site –Hydraulic model schematisation
Figure 20.17	Inglewood site—Land-use classification
Figure 20.18	Inglewood site –1% AEP peak flood depth
Figure 20.19	Inglewood site –1% AEP peak flood flow velocities
Figure 20.20	Inglewood site—1% AEP peak flow velocity × depth product
Figure 20.21	Turallin facility—Locality plan
Figure 20.22	Flood risk area within and around the proposed site (TRC Flood Risk Information Portal)
Figure 20.23	Turallin facility—Catchment plan
Figure 20.24	Turallin site –Hydraulic model schematisation
Figure 20.25	Turallin site—Land-use classification
Figure 20.26	Turallin facility—1% AEP peak flood depth
Figure 20.27	Turallin facility—1% AEP peak flood flow velocities
Figure 20.28	Turallin facility—1% AEP peak flood velocity × flood depth
Figure 21.1	Overtopping failure progression for an embankment
Figure 21.2	Embankment overtopping risk categories
Figure 21.3	Typical flood zone embankment profile
Figure 21.4	Rail formation design
Figure 22.1	ARTC Flood and Geomorphic Impacts Mitigation Framework
Figure 22.2	Example mitigation measures
Figure 22.3	Site 1—Flood Impact Objective exceedances (pre-mitigation)
Figure 22.4	Site 1—Flood Impact Objective exceedance (post-mitigation)
Figure 22.5	Site 2—Flood Impact Objective exceedances (pre-mitigation)
Figure 22.6	Site 2—Flood Impact Objective exceedances (post-mitigation)
Figure 22.7	Site 3—Flood Impact Objective exceedances (pre-mitigation)
Figure 22.8	Site 3—Flood Impact Objective exceedances (post-mitigation)
Figure 22.9	Site 4—Flood Impact Objective exceedances (pre-mitigation)
Figure 22.10	Site 4—Flood Impact Objective exceedances (post-mitigation)

Tables

Table 1.1	Flood event nomenclature (taken from ARR 2019 Book 1)
Table 3.1	Drainage catchment classification
Table 3.2	Project hydrologic and hydraulic model summary—Regional catchments
Table 3.3	Project hydrologic and hydraulic model summary—Local catchments
Table 4.1	Project hydraulic design criteria
Table 4.2	Flood Impact Objectives
Table 5.1	Stream gauges within the Gowrie Creek catchment
Table 5.2	Summary of rainfall gauging stations used for calibration
Table 5.3	Comparison of IFD rainfall intensity for 1% AEP in Gowrie catchment
Table 5.4	Input data for Probable Maximum Precipitation estimation
Table 5.5	Rainfall gauges used for model calibration
Table 5.6	Summary of calibrated losses
Table 5.7	Summary of Toowoomba North-South Transport Corridor hydrologic model calibration
Table 5.8	Stream gauge details
Table 5.9	Revised flood frequency analysis at Cranley gauge based on LP3 model
Table 5.10	Flood frequency analysis results for Oakey gauge
Table 5.11	Gowrie Creek—Summary of URBS model design event inputs
Table 5.12	Gowrie Creek—Summary of selected design event hydrographs

Table 5.13	Cranley and Oakey—Comparison between peak flows from flood frequency analysis with hydrologic model flows
Table 5.14	Comparison of modelled 1% AEP peak flow in the hydrologic model with TRC (2014) study
Table 5.15	Gowrie Creek—Hydraulic model summary
Table 5.16	Gowrie Creek—Manning's n values
Table 5.17	Gowrie Creek—Adopted culvert hydraulic losses
Table 5.18	Gowrie Creek—Identified existing structures within the hydraulic model extent
Table 5.19	Gowrie Creek—Identified existing bridges within the hydraulic model extent
Table 5.20	Gowrie Creek—Comparison between observed and model flood levels for January 2011 event
Table 5.21	Gowrie Creek—Existing Case: Estimated overtopping depths of key infrastructure
Table 5.22	Gowrie Creek—Proposed rail bridge locations and details
Table 5.23	Gowrie Creek—Summary of impacts on State-controlled roads
Table 5.24	Gowrie Creek—Summary of impacts on local public roads
Table 5.25	Gowrie Creek—Summary of impacts on existing rail infrastructure
Table 5.26	Gowrie Creek—Summary of impacts on private land outside the permanent footprint
Table 5.27	Gowrie Creek—Extreme events: Estimated depth of water above formation level and/or over top of rail level
Table 5.28	Gowrie Creek—Summary of extreme event impacts at flood sensitive receptors
Table 6.1	Westbrook and Dry creeks catchment—Range of rainfall losses used in previous studies for 5% AEP to 1% AEP design storms
Table 6.2	Westbrook and Dry creeks catchment—Range of rainfall losses applicable
Table 6.3	Westbrook and Dry creeks catchment—IFD areas—total area longitude and latitude of the centroid of each area
Table 6.4	Westbrook and Dry creeks catchment—Rainfall intensity (mm/hr) used for the IFD1 area
Table 6.5	Westbrook and Dry creeks catchment—Rainfall intensity (mm/hr) used for the IFD2 area
Table 6.6	Westbrook and Dry creeks catchment—Rainfall intensity (mm/hr) used for the IFD3 area
Table 6.7	Westbrook and Dry creeks catchment—Rainfall intensity (mm/hr) used for the IFD4 area
Table 6.8	Westbrook and Dry creeks catchment—Aerial reduction factor coefficients from ARR 2019 Data Hub
Table 6.9	Westbrook and Dry creeks catchment—Location data for Probable Maximum Precipitation estimation
Table 6.10	Westbrook and Dry creeks catchment—Rainfall intensity for the 1 in 10,000 AEP and Probable Maximum Precipitation events
Table 6.11	Westbrook and Dry creeks catchment—Adopted rainfall losses
Table 6.12	Westbrook and Dry creeks sub-catchments—Model inputs for the Regional Flood Frequency Estimation
Table 6.13	Westbrook and Dry creeks sub-catchments—Regional Flood Frequency Estimation results obtained from ARR 2016 website
Table 6.14	Westbrook and Dry creeks sub-catchments—Comparison between estimated peak flows by Regional Flood Frequency Estimation with the RAFTS model
Table 6.15	Westbrook and Dry creeks catchment—Comparison between flows for 1% AEP event in previous studies with this study's hydrologic model
Table 6.16	Westbrook and Dry creeks—Hydraulic model setup overview
Table 6.17	Westbrook and Dry creeks—Adopted roughness values
Table 6.18	Westbrook and Dry creeks—Modelled critical duration and temporal patterns
Table 6.19	Westbrook and Dry creeks—1% AEP event peak flow at key locations as applied in the hydraulic model
Table 6.20	Westbrook and Dry creeks—Existing Case: Estimated overtopping depths of key infrastructure

Table 6.21	Westbrook and Dry creeks—Proposed bridge locations and details
Table 6.22	Westbrook and Dry creeks—Summary of flood impacts on State-controlled roads
Table 6.23	Westbrook and Dry creeks—Summary of flood impact on local public roads
Table 6.24	Westbrook and Dry creeks—Summary of flood impacts on private land outside the permanent footprint
Table 6.25	Westbrook and Dry creeks—Extreme events: Estimated depth of water above formation and top of rail levels
Table 6.26	Westbrook and Dry creeks catchment—Extreme events: Impacts at flood sensitive receptors
Table 6.27	Westbrook and Dry creeks—1% AEP event with Representative Concentration Pathways 8.5 conditions Climate Change assessment: flood sensitive receptor impacts
Table 6.28	Westbrook and Dry creeks—1% AEP event culvert blockage assessment
Table 7.1	Condamine River—LiDAR datasets
Table 7.2	Condamine River—Stream gauges
Table 7.3	Condamine River gauges—Pluviograph data
Table 7.4	Condamine River—Stream gauges adopted for calibration
Table 7.5	Condamine River—Summary of URBS model calibration inputs
Table 7.6	Condamine River—Historic gauged results
Table 7.7	Condamine River catchment—Summary of URBS model calibration parameters—1991 event
Table 7.8	Condamine River catchment—URBS model calibration performance—1991 event
Table 7.9	Condamine River catchment—Summary of URBS model calibration parameters—2010 event
Table 7.10	Condamine River catchment—URBS model calibration performance—2010 event
Table 7.11	Condamine River—URBS routing parameters adopted for validation and design events
Table 7.12	Condamine River—URBS model validation performance metrics—2013 event
Table 7.13	Condamine River—URBS model validation performance metrics—2021 event
Table 7.14	Condamine River—Summary of URBS model design event inputs
Table 7.15	Condamine River—Adjusted design event losses
Table 7.16	Data output from flood frequency analysis at Warwick (422310C)
Table 7.17	Condamine River—Data output from combined flood frequency analysis at URBS model outlet
Table 7.18	Condamine River—Summary of selected design event hydrographs
Table 7.19	1% AEP expected peak flow quantiles at Lone Pine
Table 7.20	Condamine River—Comparison between simulated and recorded peak flood levels in 2010 event ¹
Table 7.21	URBS sensitivity scenarios—Peak Flows at Lone Pine (m ³ /s) ¹
Table 7.22	Regional Condamine River hydraulic model summary
Table 7.23	Condamine River hydraulic model—Manning's n values
Table 7.24	Condamine River—Identified existing structures within the hydraulic model extent
Table 7.25	Condamine River—Identified existing bridges within the hydraulic model extent
Table 7.26	Condamine River—Summary of hydraulic model calibration—1991 event ¹
Table 7.27	Condamine River—Summary of hydraulic model calibration—2010 event ¹
Table 7.28	Condamine River—Summary of hydraulic model calibration—2013 event ¹
Table 7.29	Condamine River—Summary of hydraulic model calibration—2021 event ¹
Table 7.30	Flood marker grouping
Table 7.31	Condamine River—Surveyed flood marker comparison—2010 event
Table 7.32	Recorded and modelled peak flood levels for December 2010 event at Centenary Bridge (SKM observed water level)
Table 7.33	Recorded and modelled peak flood levels for December 2010 event at Centenary Bridge (BoM observed water level)
Table 7.34	Condamine River—Model verification against photographs—2010 flood event
Table 7.35	Condamine River—Model verification against photographs—2013 flood event

Table 7.36	Condamine River—Model verification against photographs—2021 flood event
Table 7.37	Condamine River—Existing Case: Estimated overtopping depths of key infrastructure
Table 7.38	Condamine River—Proposed bridge locations and details
Table 7.39	Condamine River—Flood Impact Objective exceedances at flood sensitive receptors
Table 7.40	Condamine River—Summary of flood impacts on State-controlled roads
Table 7.41	Condamine River—Summary of impacts on local public roads
Table 7.42	Condamine River—Summary of impacts on existing rail infrastructure
Table 7.43	Condamine River—Summary of flood impacts on private land outside the permanent footprint
Table 7.44	Condamine River—Extreme events: Estimated depth of water above formation and top of rail levels
Table 7.45	Condamine River—Extreme events: Estimated impacts at flood sensitive receptors
Table 7.46	Condamine River—1% AEP event for 2090 horizon under the RCP 8.5 scenario: flood sensitive receptors impacts
Table 7.47	Condamine River—1% AEP event blockage sensitivity: flood sensitive receptor impacts
Table 7.48	Condamine River—Crop Manning's n values used for spatial variability test
Table 7.49	Sensitivity tests for interactions between Back Creek and Condamine River
Table 7.50	Back Creek inflow sensitivity to areal reduction factor
Table 8.1	Summary of Back Creek URBS model details
Table 8.2	Back Creek—Summary of URBS model design event inputs
Table 8.3	Rainfall depth (mm) used for the Back Creek catchment centroid
Table 8.4	Back Creek—Areal reduction factor coefficients from ARR 2019 Data Hub
Table 8.5	Back Creek—Critical duration and temporal patterns for hydraulic modelling
Table 8.6	Back Creek—Adopted pre-burst depths
Table 8.7	Back Creek—Adopted design event losses
Table 8.8	Back Creek—Input data for Probable Maximum Precipitation estimation
Table 8.9	Back Creek—Adopted extreme event rainfall depths (mm)
Table 8.10	Back Creek—Comparison of URBS and QRT peak flows
Table 8.11	Canal Creek peak flow estimated (LP3) and scaled flow at focal point (310-BR38)
Table 8.12	Back Creek—Comparison between Regional Flood Frequency Estimation and URBS results at focal point
Table 8.13	Back Creek—Hydraulic model summary
Table 8.14	Back Creek catchment—Adopted roughness values
Table 8.15	Back Creek catchment—December 2010 flood event cumulative rainfall totals
Table 8.16	Back Creek—Flood model validation to 2010 event flood markers
Table 8.17	Back Creek—Peak flow at key locations as applied in the hydraulic model
Table 8.18	Back Creek—1% AEP event peak inflow at key locations as applied in the hydraulic model
Table 8.19	Back Creek—Existing Case: Overtopping depths of key infrastructure
Table 8.20	Back Creek—Proposed bridge location and details
Table 8.21	Back Creek—Summary of impacts on State-controlled roads
Table 8.22	Back Creek—Summary of impacts on local public roads
Table 8.23	Back Creek—Summary of flood impacts on existing rail infrastructure
Table 8.24	Back Creek—Summary of flood impacts on private land outside the permanent footprint
Table 8.25	Back Creek—Extreme events: Estimated depth of water above formation and top of rail levels
Table 8.26	Back Creek—Summary of extreme event impacts at flood sensitive receptors
Table 8.27	Back Creek—1% AEP event with Representative Concentration Pathways 8.5 condition Climate Change assessment
Table 9.1	Canning Creek—Summary of URBS model inputs
Table 9.2	Canning Creek—Adopted initial and continuing losses
Table 9.3	Canning Creek—Comparison of flows at key locations—1% AEP event
Table 9.4	Canning Creek—Summary of URBS model design event inputs

Table 9.5	Macintyre Brook at Inglewood (416402) peak flow estimated (LP3) and scaled flow at Canning Creek catchment outlet
Table 9.6	Estimate of flows at the Canning Creek model outlet
Table 9.7	Comparison between URBS and QRT flows for Canning Creek and tributaries
Table 9.8	Nicol Creek—Summary of URBS model design event inputs
Table 9.9	Rainfall depth (mm) used for the Nicol Creek catchment centroid
Table 9.10	Nicol Creek—Areal reduction factor coefficients from ARR 2019 Data Hub
Table 9.11	Nicol Creek—Adopted temporal patterns
Table 9.12	Nicol Creek—Adopted pre-burst depths
Table 9.13	Nicol Creek—Adopted design event losses
Table 9.14	Nicol Creek—Input data for Probable Maximum Precipitation estimation
Table 9.15	Nicol Creek—Adopted extreme event rainfall depths (mm)
Table 9.16	Nicol Creek—Model inputs for the Regional Flood Frequency Estimation
Table 9.17	Estimate of flows at the Nicol Creek Project crossing
Table 9.18	Nicol Creek hydraulic model summary
Table 9.19	Nicol Creek—Adopted roughness values
Table 9.20	Nicol Creek—Peak flow at key locations as applied in the hydraulic model
Table 9.21	Nicol Creek—1% AEP event estimated peak flow at key locations as applied in the hydraulic model
Table 9.22	Nicol Creek—Existing Case: Overtopping depths of key infrastructure
Table 9.23	Nicol Creek—Proposed bridge location and details
Table 9.24	Nicol Creek—Summary of flood impacts on private land outside the permanent footprint
Table 9.25	Nicol Creek—Extreme events: Depth of water above formation level and over top or rail level
Table 9.26	Nicol Creek—Extreme events: Impacts at flood sensitive receptors
Table 9.27	Nicol Creek—1% AEP event with Representative Concentration Pathways 8.5 conditions: Structure performance
Table 9.28	Nicol Creek—1% AEP event: Culvert blockage assessment
Table 10.1	Bringalily Creek—Summary of URBS model design event inputs
Table 10.2	Rainfall depth (mm) used for the Bringalily Creek catchment centroid
Table 10.3	Bringalily Creek—Areal reduction factor coefficients from ARR 2019 Data Hub
Table 10.4	Bringalily Creek – Adopted temporal patterns
Table 10.5	Bringalily Creek—Adopted pre-burst depths
Table 10.6	Bringalily Creek—Adopted design event losses
Table 10.7	Bringalily Creek—Input data for Probable Maximum Precipitation estimation
Table 10.8	Bringalily Creek—Adopted extreme event rainfall depths (mm)
Table 10.9	Estimate of flows at the Bringalily Creek crossing
Table 10.10	Bringalily Creek—hydraulic model summary
Table 10.11	Bringalily Creek catchment —Adopted roughness values
Table 10.12	Bringalily Creek—Existing culverts within the hydraulic model extent
Table 10.13	Bringalily Creek—Existing bridge
Table 10.14	Bringalily Creek—Critical flood durations and temporal patterns
Table 10.15	Bringalily Creek—1% AEP event peak flow at key locations as applied in the hydraulic model
Table 10.16	Bringalily Creek—Existing Case: Overtopping depths of key infrastructure
Table 10.17	Bringalily Creek—Proposed bridge locations and details
Table 10.18	Bringalily Creek—Summary of flood impacts on State-controlled roads
Table 10.19	Bringalily Creek—Summary of flood impacts on private land outside the permanent footprint
Table 10.20	Bringalily Creek—Extreme events: Estimated depth of water above formation level and over top of rail level
Table 10.21	Bringalily Creek—Extreme events: Estimated impacts at flood sensitive receptors
Table 10.22	Bringalily Creek—1% AEP event Representative Concentration Pathways 8.5 conditions: Impacts at flood sensitive receptors

Table 10.23	Bringalily Creek—Summary of blockage impacts at flood sensitive receptors
Table 11.1	Native Dog Creek—Summary of URBS model design event inputs
Table 11.2	Rainfall depth (mm) used for the Native Dog Creek catchment centroid
Table 11.3	Native Dog Creek—Areal reduction factor coefficients from ARR 2019 Data Hub
Table 11.4	Native Dog Creek—Adopted temporal patterns
Table 11.5	Native Dog Creek—Adopted pre-burst depths
Table 11.6	Native Dog Creek – Adopted design event losses
Table 11.7	Native Dog Creek—Input data for Probable Maximum Precipitation estimation
Table 11.8	Native Dog Creek—Adopted extreme event rainfall depths (mm)
Table 11.9	Native Dog Creek catchment —Model inputs for the Regional Flood Frequency Estimation
Table 11.10	Estimate of flows at the Native Dog Creek crossing
Table 11.11	Native Dog Creek— Hydraulic model summary
Table 11.12	Native Dog Creek catchment —Adopted roughness values
Table 11.13	Native Dog Creek—Existing bridge
Table 11.14	Native Dog Creek—Critical durations and temporal patterns
Table 11.15	Native Dog Creek—1% AEP event peak flow at key locations as applied in the hydraulic model
Table 11.16	Native Dog Creek—Existing Case: Estimated overtopping depths of key infrastructure
Table 11.17	Native Dog Creek—Proposed bridge location and details
Table 11.18	Native Dog Creek—Extreme events: Estimated depth of water above formation and top of rail levels
Table 11.19	Native Dog Creek—1% AEP event with Representative Concentration Pathways 8.5 conditions: Structure performance
Table 12.1	Cattle Creek—Summary of URBS model design event inputs
Table 12.2	Rainfall depth (mm) used for the Cattle Creek catchment centroid
Table 12.3	Cattle Creek—Areal reduction factor coefficients from ARR 2019 Data Hub
Table 12.4	Cattle Creek—Adopted temporal patterns
Table 12.5	Cattle Creek—Adopted pre-burst depths
Table 12.6	Cattle Creek—Adopted design event losses
Table 12.7	Cattle Creek—Input data for Probable Maximum Precipitation estimation
Table 12.8	Cattle Creek—Adopted extreme event rainfall depths (mm)
Table 12.9	Cattle Creek catchment—Model inputs for the Regional Flood Frequency Estimation
Table 12.10	Estimate of flows at the Cattle Creek crossing
Table 12.11	Cattle Creek— Hydraulic model summary
Table 12.12	Cattle Creek catchment—Adopted roughness values
Table 12.13	Cattle Creek—Existing bridge
Table 12.14	Cattle Creek—Critical flood durations and temporal patterns
Table 12.15	Cattle Creek—1% AEP event peak flow at key locations as applied in the hydraulic model
Table 12.16	Cattle Creek—Existing Case: Estimated overtopping depths of key infrastructure
Table 12.17	Cattle Creek—Proposed bridge locations and details
Table 12.18	Cattle Creek—Summary of impacts on local public roads
Table 12.19	Cattle Creek—Extreme events: Estimated depth of water above formation and top of rail levels
Table 12.20	Cattle Creek—1% AEP event with Representative Concentration Pathways 8.5 conditions: Structure performance
Table 12.21	Cattle Creek—1% AEP event: Culvert blockage assessment
Table 13.1	Pariagara Creek—Summary of URBS model inputs
Table 13.2	Pariagara Creek sub-catchment data
Table 13.3	Pariagara Creek—Summary of URBS model design event inputs
Table 13.4	Rainfall depth (mm) used for the Pariagara Creek catchment centroid
Table 13.5	Pariagara Creek—Areal reduction factor coefficients from ARR 2019 Data Hub
Table 13.6	Pariagara Creek—Adopted temporal patterns
Table 13.7	Pariagara Creek—Adopted median pre-burst depths

Table 13.8	Pariagara Creek—Input data for Probable Maximum Precipitation estimation
Table 13.9	Pariagara Creek—Adopted extreme event rainfall depths (mm)
Table 13.10	Estimate of flows at the Pariagara Creek catchment outlet using QRT
Table 13.11	Pariagara Creek catchment—Model inputs for the Regional Flood Frequency Estimation
Table 13.12	Pariagara Creek catchment —Regional Flood Frequency Estimation results obtained from ARR 2019 website
Table 13.13	Estimate of flows at the Pariagara Creek crossing
Table 13.14	Pariagara Creek—Hydraulic model summary
Table 13.15	Pariagara Creek catchment—Adopted roughness values
Table 13.16	Pariagara Creek—Critical flood durations and temporal patterns
Table 13.17	Pariagara Creek—1% AEP event peak flow at key locations as applied in the hydraulic model
Table 13.18	Reassessment of peak flow and critical duration at Ch 66.23 km
Table 13.19	Pariagara Creek—Existing Case: Estimated overtopping depths of key infrastructure
Table 13.20	Pariagara Creek—Proposed bridge location and details
Table 13.21	Pariagara Creek—Summary of flood impacts on State-controlled roads
Table 13.22	Pariagara Creek—Summary of impacts on local public roads
Table 13.23	Pariagara Creek—Summary of flood impacts on private land outside of the permanent footprint
Table 13.24	Pariagara Creek—Extreme events: Depth of water above formation and top of rail levels
Table 13.25	Pariagara Creek—Summary of extreme event impacts at flood sensitive receptors
Table 13.26	Pariagara Creek—1% AEP Event Climate Change Assessment
Table 13.27	Pariagara Creek—1% AEP Event: Culvert blockage assessment
Table 14.1	Macintyre Brook catchment—Summary of key stream gauges used in the model calibration
Table 14.2	Macintyre Brook catchment—Number of rainfall gauges used in the model calibration
Table 14.3	Macintyre Brook catchment—Rainfall losses applied for December 2021 validation event
Table 14.4	Macintyre Brook—hydraulic model setup overview
Table 14.5	Macintyre Brook—Existing structure details
Table 14.6	Macintyre Brook—Manning's n values
Table 14.7	Macintyre Brook catchment—Historical floods gauge comparison—Updated modelling
Table 14.8	Macintyre Brook—1976 Surveyed flood level comparison
Table 14.9	Macintyre Brook—Critical durations for varying AEP events
Table 14.10	Macintyre Brook—1% AEP event peak flow at key locations as applied in the hydraulic model
Table 14.11	Macintyre Brook—Existing Case: Estimated overtopping depths of key infrastructure
Table 14.12	Macintyre Brook—Proposed rail bridges and detail
Table 14.13	Macintyre Brook—flood sensitive receptors impacted by afflux
Table 14.14	Macintyre Brook—flood sensitive receptors impacted by an increase in hazard
Table 14.15	Macintyre Brook—Summary of impacts on State-controlled roads
Table 14.16	Macintyre Brook—Summary of impacts on local public roads
Table 14.17	Macintyre Brook—Summary of impacts on existing rail infrastructure
Table 14.18	Macintyre Brook: Inglewood to Yelarbon—Summary of flood impacts on private land outside the permanent footprint
Table 14.19	Macintyre Brook: Extreme events—Estimated depth of water above formation level and/or over top of rail level
Table 14.20	Macintyre Brook: Extreme events—Impacts to flood sensitive receptors
Table 14.21	Macintyre Brook: 1% AEP event Representative Concentration Pathway 8.5 conditions—Impacts at flood sensitive receptors
Table 14.22	Macintyre Brook: 1% AEP event—Blockage assessment
Table 15.1	Macintyre Brook at Bybera Road—Summary of URBS model inputs

Table 15.2	Macintyre Brook at Bybera Road—Summary of URBS model design event inputs
Table 15.3	Rainfall depth (mm) used for the Macintyre Brook at Bybera Road catchment centroid
Table 15.4	Macintyre Brook at Bybera Road—Areal reduction factor coefficients from ARR 2019 Data Hub
Table 15.5	Macintyre Brook at Bybera Road—Adopted temporal patterns
Table 15.6	Macintyre Brook at Bybera Road—Adopted pre-burst depths
Table 15.7	Macintyre Brook at Bybera Road—Input data for Probable Maximum Precipitation estimation
Table 15.8	Macintyre Brook at Bybera Road—Adopted extreme event rainfall depths (mm)
Table 15.9	Estimate of flows at the outlet of Macintyre Brook at Bybera Road model
Table 15.10	Macintyre Brook at Bybera Road—Hydraulic model summary
Table 15.11	Adopted roughness values for Bringalily Creek catchment
Table 15.12	Macintyre Brook at Bybera Road—Critical durations for vary AEP events
Table 15.13	Macintyre Brook at Bybera Road—1% AEP event peak flow at key locations as applied in the hydraulic model
Table 15.14	Macintyre Brook at Bybera Road: Existing Case—Overtopping depths of key infrastructure
Table 15.15	Macintyre Brook at Bybera Road—Proposed bridge location and details
Table 15.16	Macintyre Brook at Bybera Road—Summary of impacts on local public roads
Table 15.17	Macintyre Brook at Bybera Road—Summary of flood impacts on private land outside the permanent footprint
Table 15.18	Macintyre Brook at Bybera Road: Extreme events—depth of water above rail formation and top of rail levels
Table 15.19	Macintyre Brook at Bybera Road catchment: Extreme events—Impacts at flood sensitive receptors
Table 15.20	Macintyre Brook at Bybera Road – 1% AEP event with Representative Concentration Pathways 8.5 conditions – structure performance
Table 16.1	Macintyre Brook at Cremascos Road—Summary of URBS model inputs
Table 16.2	Macintyre Brook at Cremascos Road—Summary of URBS model design event inputs
Table 16.3	Rainfall depth (mm) used for the Cremascos catchment centroid
Table 16.4	Macintyre Brook at Cremascos Road—Areal reduction factor coefficients from ARR 2019 Data Hub
Table 16.5	Macintyre Brook at Cremascos Road—Adopted temporal patterns
Table 16.6	Macintyre Brook at Cremascos Road—Adopted pre-burst depths
Table 16.7	Macintyre Brook at Cremascos Road—Input data for Probable Maximum Precipitation estimation
Table 16.8	Macintyre Brook at Cremascos Road—Adopted extreme event rainfall depths (mm)
Table 16.9	Estimate of flows at the outlet of Macintyre Brook at Cremascos Road model
Table 16.10	Macintyre Brook at Cremascos Road— Hydraulic model summary
Table 16.11	Macintyre Brook at Cremascos Road catchment—Adopted roughness values
Table 16.12	Macintyre Brook at Cremascos Road—Peak flow at key locations as applied in the hydraulic model
Table 16.13	Macintyre Brook at Cremascos Road—1% AEP event peak flow at key locations as applied in the hydraulic model
Table 16.14	Macintyre Brook at Cremascos Road: Existing Case—Overtopping depths of key infrastructure
Table 16.15	Macintyre Brook at Cremascos Road—Proposed bridge location and details
Table 16.16	Macintyre Brook at Cremascos Road—Summary of impacts on local public roads
Table 16.17	Macintyre Brook at Cremascos Road—Summary of flood impacts on private land outside the permanent footprint
Table 16.18	Macintyre Brook at Cremascos Road: Extreme events—Estimated depth of water above formation and top of rail levels
Table 16.19	Macintyre Brook at Cremascos Road catchment: Extreme events—Impacts at flood sensitive receptors

Table 16.20	Macintyre Brook at Cremascos Road: 1% AEP event with Representative Concentration Pathways 8.5 conditions—Structure performance
Table 17.1	Macintyre River—Data summary
Table 17.2	Macintyre River—Summary of key stream gauges used in the model calibration
Table 17.3	Macintyre River catchment—Number of rainfall gauges used in the model calibration
Table 17.4	Macintyre River—Hydrologic model calibration parameters
Table 17.5	Macintyre River—Summary of URBS model design event inputs
Table 17.6	Macintyre River—Areal reduction factor coefficients from ARR 2019 Data Hub
Table 17.7	Macintyre River—ARR regionalised and adopted at-site flood frequency analysis reconciled losses
Table 17.8	Macintyre River—Reservoir storage details
Table 17.9	Macintyre River—ARR Data Hub baseflow values
Table 17.10	Macintyre River—Flood frequency analysis gauge data summary
Table 17.11	Reconciled and current catchment design flows upstream of Boggabilla
Table 17.12	Macintyre River—TUFLOW hydraulic model setup overview
Table 17.13	Macintyre River—Adopted roughness values
Table 17.14	Macintyre River—1976 event flood marker comparison
Table 17.15	Macintyre River—Summary of calibration against 1976 recorded flood markers
Table 17.16	Macintyre River—1996 flood marker comparison
Table 17.17	Macintyre River—Summary of calibration against 1996 recorded flood markers
Table 17.18	Macintyre River—2011 flood marker comparison
Table 17.19	Macintyre River—Summary of calibration against 2011 recorded flood markers
Table 17.20	Comparison of results at the Boggabilla stream gauge
Table 17.21	Comparison of results at the Goondiwindi stream gauge
Table 17.22	Macintyre River—1% AEP event peak flow at key locations as applied in the hydraulic model
Table 17.23	Macintyre River— Tributary characteristics
Table 17.24	Macintyre River: Existing Case—Estimated overtopping depths of key infrastructure
Table 17.25	Macintyre River—Proposed bridge locations and details
Table 17.26	Macintyre River—Flood Impact Objective exceedances
Table 17.27	Macintyre River—Summary of flood impacts on private land outside the permanent footprint
Table 17.28	Peak flow variation assessment along Section 1
Table 17.29	Macintyre River: Extreme events—Estimated depth of water above formation level and/or over top of rail level
Table 17.30	Macintyre River —Summary of extreme event impacts at flood sensitive receptors
Table 17.31	Macintyre River: 1% AEP event—Climate change assessment
Table 17.32	Macintyre River—Summary of 0% and 50% blockage impacts at flood sensitive receptors
Table 18.1	Local drainage hydraulic model summary
Table 18.2	Fish passage requirements for culvert crossings
Table 18.3	Local catchment drainage—Proposed bridge locations and details
Table 18.4	Local catchment drainage—Estimated impacts to peak water levels at flood sensitive receptors
Table 18.5	Local catchments—Summary of impacts on State-controlled roads
Table 18.6	Local catchments—Summary of impacts on local public roads
Table 18.7	Local catchments—Summary of impacts on existing rail infrastructure
Table 18.8	Local catchments—Summary of flood impacts on private land outside the permanent footprint
Table 18.9	Local catchments—Impact on farm dams
Table 19.1	Millmerran and Pampas emergency evacuation route impact assessment
Table 19.2	Brookstead emergency evacuation route impact assessment
Table 19.3	Southbrook emergency evacuation route impact assessment
Table 19.4	Wellcamp emergency evacuation route impact assessment
Table 19.5	Gowrie Mountain emergency evacuation route impact assessment

Table 19.6	Goondiwindi and Yelarbon emergency evacuation route impact assessment
Table 19.7	Inglewood emergency evacuation route impact assessment
Table 20.1	Adopted 1% AEP IFD depths (mm)
Table 20.2	Adopted 1% AEP design losses and comparison with adjacent Project catchment losses
Table 20.3	Manning's 'n' values
Table 20.4	Adopted 1% AEP IFD depths
Table 20.5	Adopted Manning's 'n' values
Table 20.6	Laydown areas flooding risk
Table 21.1	Geotechnical material parameters
Table 21.2	Geotechnical material parameters—Seepage
Table 21.3	Applicable scenarios
Table 21.4	Flood sections adopted for geotechnical impact analysis
Table 21.5	Geotechnical model
Table 21.6	Stability analysis results for the embankment at Ch 66.3 km (applicable scenario 2b)
Table 21.7	Stability analysis results for the embankment at Ch 147.1 km (applicable scenario 1a)
Table 21.8	Stability analysis results for the embankment at Ch 190.0 km (applicable scenario 2b)
Table 21.9	1,000 mm thick of rock armour required in the 1% AEP event
Table 22.1	Hierarchy of measures to reduce flood impacts
Table 22.2	Mitigation modelling—Site selection
Table 22.3	Site 1 structure details—C62.94 and C63.15
Table 22.4	Site 1 pre- and post-mitigation—Afflux
Table 22.5	Site 1 pre- and post-mitigation—Change in time of inundation
Table 22.6	Site 1 pre- and post-mitigation—Change in velocity
Table 22.7	Site 1 pre- and post-mitigation—Change in hazard
Table 22.8	Site 2 structure details—C77.20, C77.47 and C77.77
Table 22.9	Site 2 pre- and post-mitigation—Afflux
Table 22.10	Site 2 pre- and post-mitigation—Change in time of inundation
Table 22.11	Site 2 pre- and post-mitigation—Change in velocity
Table 22.12	Site 2 pre- and post-mitigation—Change in hazard
Table 22.13	Structure details—C84.38 and C85.35
Table 22.14	Site 3 pre- and post-mitigation—Afflux
Table 22.15	Site 3 pre- and post-mitigation—Change in time of inundation
Table 22.16	Site 3 pre- and post-mitigation—Change in velocity
Table 22.17	Site 3 pre- and post-mitigation—Change in hazard
Table 22.18	Structure details—C119.75, C119.88, C120.08, C120.25 and C120.31
Table 22.19	Site 4 pre- and post-mitigation—Change in time of inundation
Table 22.20	Site 4 pre- and post-mitigation—Change in velocity
Table 22.21	Site 4 pre- and post-mitigation—Change in hazard
Table 24.1	Project hydraulic design criteria outcomes
Table 24.2	Flood Impact Objectives and outcomes

Abbreviations

The following acronyms are used within this document:

Term or acronym	Description
1D	One-dimensional
2D	Two-dimensional
AEP	Annual Exceedance Probability
AHD	Australian Height Datum
AIDR	Australian Institute for Disaster Resilience
ALS	Airborne Laser Scanning
ARF	Areal reduction factor
ARI	Average Recurrence Interval
ARR 1987	<i>Australian Rainfall and Runoff: A Guide to Flood Estimation</i> (Institution of Engineers, 1987)
ARR 2019	<i>Australian Rainfall and Runoff: A Guide to Flood Estimation</i> (Ball et al., 2019)
ARTC	Australian Rail Track Corporation
B2G	Inland Rail Border to Gowrie Project
BoM	Bureau of Meteorology
BRVFMP	<i>Floodplain Management Plan for the Borders River Valley Floodplain</i>
CC	Climate Change
CEMP	Construction Environmental Management Plan
Ch	chainage
CL	Continuing Loss
D/S	Downstream
DEA	Design Event Approach
DEM	Digital Elevation Model
DoR	Department of Resources (Qld)
DPE	NSW Department of Planning and Environment
DPIE	Department of Planning, Industry and Environment (NSW)
DRDMW	Department of Regional Development, Manufacturing and Water (Qld)
DTMR	Department of Transport and Main Roads (Qld)
EAF	Elevation adjustment factor
EIS	Environmental Impact Statement
EMP	Environmental Management Plan
ESCP	Erosion and Sediment Control Plan
ETV	Erosive Threshold Velocity
EY	Exceedance per Year
FFA	flood frequency analysis
FFJV	Future Freight Joint Venture
FIO	Flood Impact Objective
FRA	Flood Risk Assessment
FSR	flood sensitive receptor
GEV	Generalised Extreme Variable

Term or acronym	Description
GIMS	Geospatial Information Management System
GIS	Geographic Information System
GL	gigalitres
GRC	Goondiwindi Regional Council
GSMD	Generalised Short Duration Method
IFD	Intensity-Frequency-Duration
IL	Initial Loss
ILSAX	Time Area Hydrology model with Horton Losses
km	kilometres
km ²	square kilometres
LDMG	Local Disaster Management Group
LFC	Layered flow contraction
lfcs	Layered flow constriction
LGA	Local Government Authority
LiDAR	Light Detection and Ranging
LPIII	Log-Pearson Type III
m	metres
m ³ /s	metres cubed per second
MAF	Moisture Adjustment Factor
mAHD	metres with respect to Australian Height Datum
MBIR	Melbourne to Brisbane Inland Rail
MDC	Material Distribution Centre
ML	megalitres
ML/d	megalitres per day
mm	millimetres
mm/hr	millimetres per hour
NS2B	Inland Rail North Star to Border Project
PAR	Population at risk
PERN	Catchment roughness
PIR	Preferred Infrastructure Report
PMF	Probable Maximum Flood
PMP	Probable Maximum Precipitation
QGIS	Quantum Geographic Information System
Qld	Queensland
QR	Queensland Rail
QRT	Quantile Regression Technique
QT	Quadtree
RAFTS	Runoff-routing modelling
RCBC	Reinforced Concrete Box Culvert
RCP	Reinforced Concrete Pipe
RFFE	Regional Flood Frequency Estimation

Term or acronym	Description
RL	Reduced level
RoG	Rain on Grid
RORB	Runoff and streamflow routing program
SA	Source-Area
SGS	sub-grid sampling
SRTM	Shuttle Radar Topographic Mission
The Project	Inland Rail Border to Gowrie Project
TNSTC	Toowoomba North-South Transport Corridor
TP	Temporal pattern
TRC	Toowoomba Regional Council
TUFLOW	Hydraulic and hydrodynamic modelling software suite
TW	Tailwater
U/S	Upstream
URBS	Unified River Basin Simulator (continuous/event rainfall runoff routing model)
USQ	University of Southern Queensland
VBF	Volume below full
WRM	WRM Water and Environment
WWTP	Wastewater Treatment Plant
XP-RAFTS	Runoff-runoff routing software package

Executive summary

The Inland Rail New South Wales(NSW)/Queensland (QLD) Border to Gowrie (B2G) Project (the 'Project') provides a connection between the northern end of the North Star to Border project and the western end of the Gowrie to Helidon project. The Project is proposed to cross several major rivers, creeks and streams through Goondiwindi Local Government Area (LGA) and Toowoomba LGA.

There are several major waterways within the Study Area, with the key waterways being Gowrie Creek, Condamine River, Macintyre Brook and Macintyre River. Other significant creek crossings include Pariagara Creek, Cattle Creek, Native Dog Creek, Bringalily Creek, Nicol Creek, Back Creek and Westbrook Creek.

The purpose of this assessment is to better understand and quantify the existing flooding characteristics of the floodplains that the Project crosses, and to assess and mitigate potential impacts associated with the construction and operation of the Project on the existing flooding regime of each waterway. The key objective of the report is to provide information on the data investigation, hydrologic and hydraulic calibration, design event modelling, and provide comment on the performance on the Project's revised reference design.

Available background information, including existing hydrologic and hydraulic models, survey, streamflow data, available calibration information and anecdotal flood data, was collected and reviewed. This data was sourced from a wide range of stakeholders and then used to develop calibrated hydrologic and hydraulic models for each waterway. These models were calibrated against multiple historical events and validated through stakeholder and community feedback.

Details regarding each model, including its development, calibration and spatial extent, are provided in the individual catchment sections within this report. Hydrologic modelling for the Project was undertaken using the methodology consistent with *Australian Rainfall and Runoff: A Guide to Flood Estimation* (Ball et al., 2019) (ARR 2019). To support the flood impact assessment, two cases were modelled for each floodplain: An Existing Case representing current floodplain conditions (or 'baseline' conditions), and a Developed Case representing floodplain conditions that include the Project. The Developed Case models were run for the same range of design events as those of the Existing Case, with results compared to determine impacts on peak water levels, flows, flood flow distribution, velocities, hazard, and duration of inundation on each floodplain and, in particular, upon identified flood sensitive receptors (FSRs) and property.

The refinement of the Project reference design was guided using hydraulic design criteria and Flood Impact Objectives (FIOs) for the detailed design, and commissioning and operations stages, which were developed for the Project based on industry practice, in consultation with the International Independent Panel of Flood Experts. Detailed hydrologic and hydraulic modelling was undertaken to meet the hydraulic design criteria and FIOs, with a series of iterations undertaken to incorporate design refinement, and stakeholder and community feedback.

Existing Case hydrologic and hydraulic modelling was used to determine the existing flood risk associated with the proposed non-resident workforce accommodation sites, Turallin facility and Whetstone Material Distribution Centre.

The hydrologic and flooding assessment undertaken has demonstrated that the Project is predicted to result in impacts on the existing flooding regime that generally comply with the FIOs and that the Project's design meets the hydraulic design criteria. In those instances where FIOs are exceeded, this report outlines the proposed mitigation measures to be explored further in detailed design, in consultation with stakeholders and landowners.

A comprehensive consultation exercise was undertaken to provide the community with detailed information and certainty around the flood modelling and the Project revised reference design.

The consultation with stakeholders, including landowners, was undertaken at key stages, including validation of the performance of the modelling in replicating experienced historical flood events, and presentation of the Project's revised reference design outcomes, and impacts on properties and infrastructure. In future stages, ARTC will continue to work with:

- Landowners concerned with hydrology and flooding throughout the detailed design, pre-construction, early works and construction; and commissioning and operations stages of the Project
- Directly impacted landowners affected by the Project throughout the detailed design, pre-construction, early works and construction; and commissioning and operations stages of the Project
- Local councils and State Government departments throughout the detailed design, pre-construction, early works and construction; and commissioning and operations stages of the Project.

Flood maps were prepared to visualise and communicate anticipated flood impacts for the range of modelled flood events. The flood maps are presented in Volume 2 of this report (Appendix T2: Hydrology and Flooding Technical Report—Figures) as well as being available via an [Online Digital Portal](https://anz.planengage.com/inlandrail-border-to-gowrie)¹.

¹ <https://anz.planengage.com/inlandrail-border-to-gowrie>

1 Introduction

1.1 Inland Rail Program

Inland Rail is a once-in-a-generation program connecting regional Australia to domestic and international markets, transforming the way we move freight around the country. It will complete the 'spine' of the national freight network between Melbourne and Brisbane via regional Victoria, New South Wales (NSW) and Queensland. This new 1,700 kilometres (km) line is the largest freight rail infrastructure project in Australia.

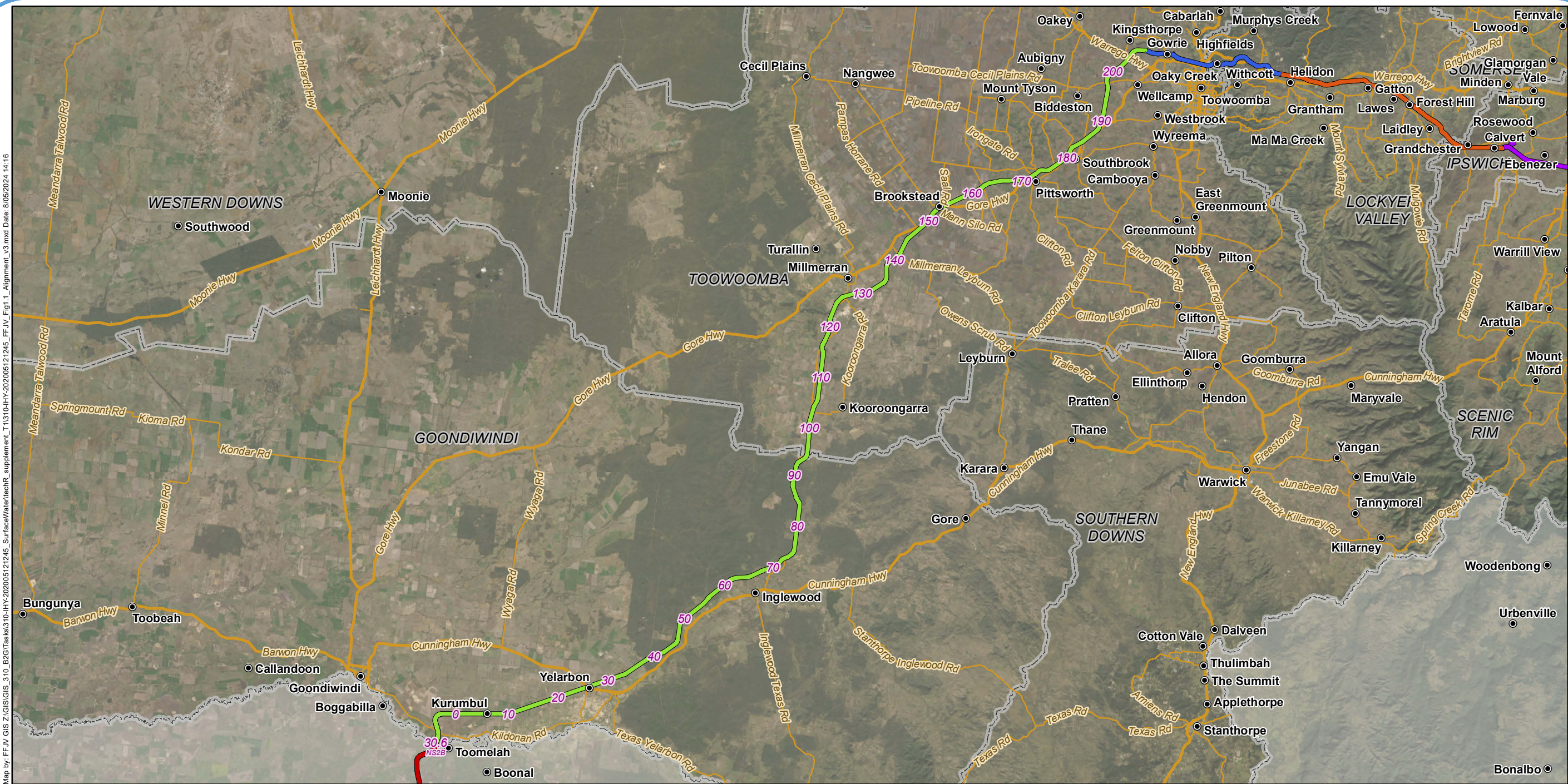
1.2 Border to Gowrie alignment

The Inland Rail NSW/Queensland (QLD) Border to Gowrie (B2G) Project (the Project) provides a connection between the northern end of the North Star to Border (NS2B) project and the western end of the Gowrie to Helidon project. The Project is proposed to cross several major rivers, creeks and streams.

The Project alignment traverses across Goondiwindi Local Government Area (LGA) and Toowoomba LGA. The Project alignment is shown in Figure 1.1.

Key features of the Project include:

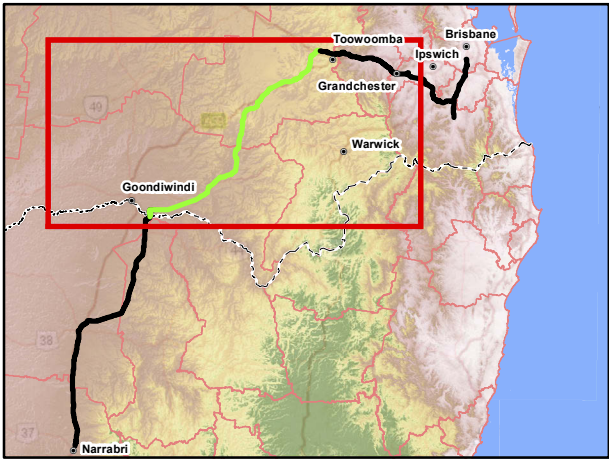
- 217.48 km of new single-track railway comprising of approximately 7 km of standard-gauge rail and 210.48 km of dual-gauge railway (trains travelling in both directions share the same track)
- Bridges to accommodate topographical variation, crossings of waterways and other infrastructure
- Reinforced concrete pipe culverts (RCP) and reinforced concrete box culverts (RCBC)
- Rail crossings, including level crossings, grade separations/rail or road overbridges, occupational/private crossings and fauna crossing structures.



Map by: FF JV GIS Z:\GIS\GIS_310_B203\Task\310-HY-202005121245_SurfaceWater\techR_supplement_T1310-HY-202005121245_FF_JV_Fig1.1_Alignment_v3.mxd Date: 8/05/2024 14:16

Legend

- 5 Chainage (km)
- Localities
- North Star to NSW/QLD border alignment
- Border to Gowrie alignment
- Gowrie to Helidon alignment
- Helidon to Calvert alignment
- Calvert to Kagaru alignment
- Major roads
- Minor roads
- NSW Qld border
- Local Government Areas



A3 scale: 1:800,000
0 6 12 18 24 30 km



Future Freight
Integrating Community, Environment and Engineering

Issue date: 08/05/2024 Version: 3
Coordinate System: GDA 1994 MGA Zone 56

Border to Gowrie
Figure 1.1:
Project rail alignment

1.3 Objectives of this report

The investigation detailed in this report was undertaken to firstly identify high-risk watercourse crossings or floodplain locations that may be impacted by the Project. Secondly a detailed quantitative assessment was undertaken to better understand and quantify the existing flooding characteristics of each of the high-risk waterways in the vicinity of the Project, and to assess and mitigate any potential impacts associated with the Project on the existing flooding regime of each waterway.

The key purpose of this report is to provide details of investigation undertaken, including data collection and review; development and calibration of hydrology and hydraulic models; design event modelling; impact assessment of the Project; development of mitigation measures and to assess proposed Project performance. Consultation with stakeholders and the community was progressively undertaken with feedback used to inform the development and calibration of the models, and to refine the Project's revised reference design.

Key objectives of the hydrology and flooding investigation were to:

- Consult with local authorities regarding existing flood studies relevant to the Project's revised reference design and consider these previous flood studies in the design
- Consult with stakeholders and government agencies to obtain flood data to assist in model development and calibration
- Undertake detailed hydrologic and hydraulic modelling for each major catchment to establish the Base Case (or Existing Case) flood conditions for the range of floods up to 1 per cent (%) Annual Exceedance Probability (AEP) as well as the 1 in 2,000 and 1 in 10,000 AEP and the Probable Maximum Flood (PMF) events
- Determine existing flood conditions, including flood levels, flows, velocities, hazard and time of inundation
- Analyse the Project's road and rail designs, drainage infrastructure and associated infrastructure works
- Assess the impacts of the Project's revised reference design on neighbouring properties, infrastructure and the surrounding environment
- Identify and assess potential mitigation measures. The requirement for mitigation was based on the magnitude of impacts and how this aligned with the Flood Impact Objectives (FIO).

This report has been prepared to support the revised draft Environmental Impact Statement (EIS), specifically EIS Chapter 14: Flooding and Geomorphology. In doing so, this report addresses Sections 11.64 to 11.69 of the Terms of Reference for the Draft EIS: Inland Rail—Border to Gowrie Project (November 2018) and additional information requirements as issued by the office of the Coordinator General, January 2022. A compliance check of this report against each of the relevant components of the terms of reference is presented in EIS Appendix A2: Terms of Reference Cross-reference Table.

1.4 International Independent Panel of Flood Experts

In June 2020 the Australian and the Queensland governments established an independent International Panel of Experts for Flood Studies ('the Expert Flood Panel') to provide advice to the Australian and the Queensland governments on the flood modelling and designs developed by Australian Rail Track Corporation (ARTC) for Inland Rail in Queensland, and to provide assurance to the public that the flood models and revised reference design developed by ARTC meet national guidelines and industry best practice.

As an advisory body to government, the Panel was independent of ARTC in respect of the development, public consultation, and approvals for the Inland Rail EIS process.

The Expert Flood Panel undertook a comprehensive review of all of the hydrologic and hydraulic modelling, supporting technical information, and reporting prepared by ARTC and its consultants. The Expert Flood Panel prepared draft reports for each of the key catchments crossed by the Project, being Macintyre River, Macintyre Brook, Cremascos Road, Bybera Road, Pariagara Creek, Cattle Creek, Native Dog Creek, Bringalily Creek, Nicol Creek, Back Creek, Condamine River, Westbrook, Dry Creek and Gowrie Creek. In each report the Expert Flood Panel documented a series of comments, and rated their risk level, in an Issues Management Register (Appendix A). ARTC, and its consultants, worked closely with the Expert Flood Panel to provide additional technical information and/or undertake sensitivity testing to address the concerns of the Expert Flood Panel. The Expert Flood Panel provided their Final Report to the Queensland and Australian governments on 6 September 2022, and it was released publicly on 7 October 2022. The Final Report can be viewed on the Department of Transport and Main Roads (DTMR) website (tmr.qld.gov.au).

The Final Report presented the outcomes of the Panel's review of the flood models and reference designs developed by ARTC for each of the Queensland projects, including the Border to Gowrie Project. It brings together and addresses the findings of the Panel's initial draft reports for each Project section. The Panel recognised the substantial work undertaken by ARTC to identify existing flooding characteristics and to assess and mitigate potential impacts associated with Inland Rail.

The Expert Flood Panel found ARTC's modelling framework to be in accordance with both national guidelines and current industry best practice. Significant infrastructure projects, like Inland Rail, are accomplished through an iterative process, and the Panel acknowledged that it is normal practice for iterative improvements and changes to occur through the various Project design and approval stages. As such, the Panel identified a number of issues in the Issues Management Register where ARTC has agreed to provide additional information in the respective revised draft EIS or address the issue during the subsequent design stages of the Project.

In respect to the Panel's Terms of Reference, the Panel found the models developed and used by ARTC are 'fit for purpose', meaning suitable for an EIS and, subsequently, detailed design once the recommended actions have been adopted by ARTC.

The Queensland and Australian governments accepted the Expert Flood Panel's final report findings and recommendations. ARTC and the DTMR have agreed a process to ensure each of the recommendations is implemented, including appointment of Flood Experts (former Expert Flood Panel members) to review and certify when each recommendation and Issues Management Register item has been satisfied, in accordance with the timeframes specified in the Final Report for each Queensland project.

The development of FIOs for the Project (as detailed in Section 4.2) was undertaken in consultation with the Expert Flood Panel. The FIOs are quantifiable flooding metrics aimed at identifying locations where the Project has the potential to cause a change in baseline flood characteristics that may result in adverse impacts.

A detailed summary of the Project-related issues and ARTC's responses are presented in the Issues Management Register in the final Expert Flood Panel Report (Appendix B). The Issues Management Register has been replicated in Appendix A with the location where each comment has been addressed and response documented in the revised draft EIS identified.

1.5 Key Project impact assessment terminology

The key Project assessment terminology used within this document is defined below:

- **The Project** refers to the Inland Rail—Border to Gowrie project, the subject of this revised draft EIS. The Project is a 217.48 km section of new dedicated single-track, open-access freight railway between the NSW/QLD Border and Gowrie, Queensland. The Project is described in detail in EIS Chapter 5: Project Description.

- The **Project Footprint** is the area required to accommodate all permanent and temporary components of the Project, being:
 - **Permanent Footprint:** the area required to accommodate permanent infrastructure associated with the Project, including rail, road and other miscellaneous infrastructure. Rail infrastructure includes rail tracks, crossing loops, turnouts, earthworks, bridges, drainage, level crossings, grade separations, rail maintenance access roads, signalling and fencing. Road-related works resulting from the Project encompass new and upgraded roads, realignments and diversions, intersection improvements and closures.
 - **Temporary Footprint:** the area required to accommodate construction activities and facilities of a temporary nature and duration to support the Project. The Temporary Footprint is generally wider than the Permanent Footprint to allow for the construction of Project elements, including fencing; drainage features, including erosion and sediment control measures; temporary stockpiling of soil and cleared vegetation; and to allow for necessary construction access and turnaround provisions.
 - **Temporary Project facilities** include laydowns, site office areas, non-resident workforce accommodation, a material distribution centre (MDC), concrete batch plants and borrow pits.
- The term **Project Alignment** refers to the position given to the centreline of the Project railway track on the ground.
- The Project's **reference design** was developed for the **draft EIS**. Since the draft EIS, the Project's reference design has been revised in response to engagement with key stakeholders, assessment of field-verified survey data and review of design optimisation opportunities. The revised draft EIS has assessed and is seeking approval for the **revised reference design**. The revised reference design is described in EIS Chapter 5: Project Description and EIS Appendix B1: Design Drawings of the revised draft EIS and, with respect to flooding infrastructure, the Cross Drainage Register is documented in Appendix B of this report. The Project's revised reference designs will continue to be further refined post-EIS to become the final design for the Project, during the **detailed design** stage. During the detailed design stage, the general principle will be that design changes either improve the safety or performance of the Project, improve environmental outcomes, or incorporate value engineering opportunities. ARTC and its appointed contractor will be responsible for integrating any EIS conditions into the detailed design.
- The **study area** covers the catchment areas that drain towards the Project footprint. These catchments have been modelled as part of the hydrology and flooding assessment to estimate flood flows, and studied to define the regional geomorphic landscapes and their distribution along the Project footprint.
- The **impact assessment area** refers to the area covered by the regional and local catchment hydraulic models as shown in Figure 3.2 and Figure 3.3.
- Pre-construction, early works and construction works in the context of this chapter refer to:
 - Site preparation for construction
 - Establishment of access roads/tracks
 - Vegetation clearing and other ground-disturbance activities that will be required to comply with relevant legislative requirements, approval conditions, guidelines and plans
 - Additional surveys and geotechnical investigations to inform construction works
 - Relocation or protection of Queensland Rail (QR) assets confirmed through the detailed design stage
 - Utility/service interfaces confirmed through the detailed design stage
 - Modification of biosecurity fencing
 - Installation of boundary fencing
 - Establishment of site offices/facilities and initial laydown areas including Whetstone MDC

- Establishment of non-resident workforce accommodation
- Site set out and pegging within the Project footprint
- Establishment of laydown areas and compounds, including vehicle inspection/workshops, washdown facilities and temporary fencing
- Clearing—using dozers, chainsaws, excavators, trucks and similar equipment
- Establishment of erosion and sediment controls as per approved Erosion and Sediment Control Plan
- Rail corridor works, including track-works turnouts and crossing loops
- Road and road–rail interface works
- Road realignments, grade separations and upgrades works
- Rail maintenance access roads
- Bridge construction
- Fencing
- Fauna habitat connectivity measures (fencing, crossing structures in accordance with detailed design specifications and Appendix N: Draft Fauna Management Plan)
- Signalling and communications
- Ballast—supply, delivery and installation
- Concrete sleepers—supply, delivery and installation
- Utilities and services to support/service the Project
- Bulk earthworks—major cut-to-fill operations include the winning of suitable construction material from sections of cut along the Project alignment or from borrow pits external to the site
- Permanent and temporary drainage controls, including culverts and longitudinal drainage
- Clean-up, landscaping, site restoration and rehabilitation, and any other activities necessary to complete such works
- Demobilisation of construction sites and activities.

For the purposes of the impact assessment, mitigations and risk assessment, the construction stage is assumed to include early works and pre-construction activities.

■ **Commissioning and operations** in the context of this chapter refers to:

- Testing and commissioning (checking) of the rail line and communication/signalling systems
- Bridge and culvert inspections and maintenance
- Sleeper replacement
- Rail replacement, welding and grinding
- Ballast replacement and cleaning
- Track tamping and reconditioning
- Vegetation management
- Signalling systems, and equipment testing and maintenance
- Transportation of equipment, materials and workforce
- Other asset management in accordance with ARTC technical maintenance.

For the purposes of the impact assessment, mitigations and risk assessment, the operations phase is assumed to include commissioning.

- **Regional flooding** happens when widespread, prolonged rain falls over a large river or large creek catchment. As the river/creek channel reaches capacity, excess water flows over its banks causing flooding. Regional flooding downstream can occur hours or days after the rainfall has finished. In a regional system, river or large creek flooding may coincide or exacerbate flooding in adjacent small creeks and tributaries due to backwater effects.
- **Local catchment flooding** can include small creek flooding, overland flow flooding or a combination of both. Small creek flooding happens when intense rain falls over a minor small creek catchment; this is often referred to as 'flash-flooding' due to the rapid catchment response time. Run-off from houses and streets also contributes to creek flooding. The combination of heavy rainfall, run-off and the existing water in the creek causes creek levels to rise. Overland flow is run-off that travels over the land during heavy rainfall events. Overland flow can be unpredictable because it is affected by localised rainfall and urban features such as stormwater pipes, roads, fences, walls and other structures. The actual depth and impact of overland flow varies depending on local conditions but it generally occurs quickly.
- The **Existing Case** is the baseline scenario (i.e. existing environment/before the Project is built).
- The **Developed Case** is the post-development scenario (i.e. once the Project has been built) and is also considered as the initial case within the terms of the impact assessment and risk assessment documented in the revised draft EIS. The Developed Case design (also known as the reference design) is described in EIS Chapter 5: Project Description and EIS Appendix B1: Design Drawings of the revised draft EIS and, with respect to flooding infrastructure, the Cross Drainage Register documented in Appendix B of this report.
- **Flood sensitive receptors** (FSRs) were identified from aerial and satellite imagery and ground-truthed, where possible, during site visits. Flood sensitive receptors include dwellings, sheds, commercial properties/lots (such as petrol stations and shops), silos, hospitals, roads, rail lines, airports, etc. A total of 1,329 FSRs were identified within the Impact Assessment Area and assessed as part of the impact assessment.
- **Waterways** are defined under the *Fisheries Act 1994* (Qld) and include a river, creek, stream, watercourse or inlet of the sea. The definition includes freshwater and tidal waters, both permanent and ephemeral waterways, and includes drainage features. It also includes channels along which fish are expected to move if they connect isolated water bodies to defined waterways during times of flow; however, it does not include isolated waterbodies where no connectivity is available.
- The term '**watercourse**' is defined under the *Water Act 2000* (Qld) as a river, creek, or other stream, which includes a stream in the form of an anabranch or a tributary, where water flows either permanently or intermittently, regardless of flow frequency; however, a watercourse does not include any section of a feature that has a tidal influence or is upstream or downstream from a defined limit between potential estuarine and fresh water; therefore, by definition, all watercourses are waterways but not all waterways are watercourses.
- A **River Basin** is a portion of land that is drained by a river and its tributaries.
- The hydrologic and flooding investigation has adopted the latest approach to design flood terminology, as detailed in *Australian Rainfall and Runoff: A Guide to Flood Estimation* (Ball et al., 2019) (ARR 2019). Accordingly, design events are quoted in terms of **AEP**, with the adopted terminology for the simulated design events shown in bold in Table 1.1.

Table 1.1 Flood event nomenclature (taken from ARR 2019 Book 1)

Exceedances per year (EY)	AEP (%)	AEP (1 in x years)	ARI
0.22	20.00	5	4.48
0.20	18.13	5.52	5.00
0.11	10.00	10	9.49
0.05	5.00	20	20
0.02	2.00	50	50
0.01	1.00	100	100
0.005	0.50	200	200
0.002	0.20	500	500
0.0005	0.05	2,000	2,000
0.0001	0.01	10,000	10,000
-	-	PMF²	

Table notes:

- 1 As an example, in general terms, a 1% AEP event means that there is a 1% chance of a flood event of that magnitude occurring in any given year
- 2 PMF - The term 'probable maximum flood' refers to the largest flood that could conceivably be expected to occur at a particular location, usually estimated from probable maximum precipitation

1.6 Overlap between flooding and geomorphology

The hydrology and flooding and geomorphological assessments undertaken as part of the revised draft EIS both overlap and have distinct differences.

In broad terms:

- The geomorphological assessment provides an overview of long-term trends occurring within waterways both naturally and in response to imposed change (e.g. catchment changes, floodplains works including, but not limited to, the Project).
- The hydrologic/hydraulic assessment describes in very high levels of detail the hydraulic conditions (flow, speed, depth, etc.) over the length of the alignment for specific flood events and enables both structure design and flood mitigation works to be undertaken.

While the hydraulic models are fundamental design and assessment tools, they have no ability to model changes to the physical processes of waterways (erosion/deposition) in response to construction activities during the events modelled. Similarly, they are not used to assess potential changes associated with periods of low flow.

To assess the potential physical change in waterways as a result of background processes and works, it is necessary to undertake both geomorphological assessments and hydrologic/hydraulic assessments.

More specifically, the geomorphology assessment is focused on fluvial geomorphic landforms, specifically assessment of floodplains, channels, and waterways/watercourses.

Due to the overlap between the two disciplines, a combined Flood and Geomorphic Impacts Mitigation Framework has been established and detailed in Section 22.

The detailed step-by-step methodologies followed for the hydrology and flooding and geomorphology impact assessments are discussed in Section 2 and EIS Appendix H: Geomorphology Report, respectively.

2 Assessment methodology

The hydrology and flooding impact assessment involved differing methodologies for the pre-construction, early works and construction works stage, and the commissioning and operations stage. These are detailed below.

2.1 Pre-construction, early works and construction works stage

Pre-construction, early works and construction works are all temporary in nature and will be in various forms as the Project progresses to being fully constructed. As such, flood impacts will be highly influenced by any planned construction staging, planned embankment raises, and cuts and temporary drainage works. At this stage of planning for the Project, detailed construction phasing is not available and will be developed as contractors are onboarded to the Project. As such, the following methodology has been employed for the assessment of impacts from temporary Project facilities within the temporary footprint:

- Assessment of flood risk (via a Flood Risk Assessment (FRA)) utilising Existing Case results including Whetstone MDC, Turallin facility, and Yelarbon and Inglewood non-resident workforce accommodation sites.
- A screening of temporary Project facilities, including concrete batching plants, fuel depots, laydown, stockpile areas etc., against the 1% AEP flood risk mapping was carried out to determine the works and activities that are likely to occur within areas at risk of flooding.

Construction works within the temporary disturbance footprint will be guided by FRA as part of the Outline Environmental Management Plan (EMP) and subsequent Construction Environmental Management Plan (CEMP). The FRA approach is discussed in Section 22 and will be documented via a Flood Risk Management Sub-Plan. Erosion and sediment control will be developed as a component of the CEMP and endorsed by the Environmental Monitor prior to construction works commencing. The CEMP and its relevant Sub-plans including erosion and sediment control plans will be prepared in accordance with statutory and Project requirements to minimise the risk of adverse flood impacts through mitigation determined via an FRA, where required.

2.2 Commissioning and operations stage

The hydrology and flooding impact assessment involved the following activities:

- Collation and review of available background information, including existing hydrologic and hydraulic models, survey, rainfall and streamflow data, calibration information and anecdotal flood related data. This review established which datasets were suitable to use for the Project's revised reference design.
- Determination of critical flooding mechanisms for waterways and drainage paths in vicinity of the Project's temporary and permanent footprint (i.e. regional flooding versus local catchment flooding).
- Determination of high-risk watercourses and overland flow paths, that the Project crosses, qualitatively considering the:
 - Catchment size, resulting flood flows and velocities
 - Land use in the vicinity of the Project
 - Extent and depth of flood inundation
 - Duration of flood events and catchment response time
 - Proximity to and nature of FSRs (e.g. houses, sheds, roads etc).

- Development of tailored hydrologic and hydraulic models for key waterways and overland flow paths. Individual modelling approaches, including justification for the selection of each approach are outlined in the individual modelling sections within this report.
- Validation of the hydrologic and hydraulic models against recorded data for historical flood events, including more recent 2021 events
- Community and stakeholder engagement to validate model performance and gain acceptance of modelling and calibration outcomes. Anecdotal flood event information, such as flood photography, recorded flood markers and personal observations from landowners, were sourced to validate the calibration of the hydrologic and hydraulic models.
- Update of hydrologic models to include ARR 2019 design events. ARR 2019, being the current version of this guideline during the formative stages of Inland Rail and this Project, was adopted as a guiding document for flooding aspects of this assessment to ensure consistency in assessment across the Inland Rail Program.
- Simulation of ARR 2019 design events for the Existing Case and comparison to previous studies to confirm drainage paths, waterways, and associated floodplain areas, and establish the existing flood regime in the vicinity of the Project
- Inclusion of the revised reference design (Developed Case) in the hydraulic models and simulation of ARR 2019 design events including the 20%, 10%, 5%, 2%, 1% AEP events, extreme events including the 1 in 2,000 and 1 in 10,000 AEP events and the PMF
- Tropical cyclone-induced rainfall events are captured in the ARR 2019 Data Hub, and historic rainfall and stream-gauge data used in the hydrologic assessments
- Assessment of impacts of the Project using the suite of design flood events, including consideration of change in flood levels, flow distributions, velocities, hazard and inundation periods
- Determination of initial mitigation measures to manage potential impacts, including refinement of location and dimensions of drainage structures under the Project. Iterations were undertaken in the hydraulic models to achieve a design that addresses the FIOs
- Sensitivity analysis on the Project's revised reference design for factors including climate change and blockage risk.
 - A blockage factor of 25 per cent was applied to all proposed culverts in the revised reference design based on guidelines set out in ARR 2019. The adopted blockage factor for the proposed bridges was between 5 per cent and 10 per cent based on the waterway area blockage due to bridge piers. The sensitivity of the flood impact assessment was tested using two additional blockage scenarios, namely 0 per cent and 50 per cent.
 - The potential impacts of climate change on areas at risk of flooding were assessed for the 1% AEP design event to determine the sensitivity of the Project to the potential long-term changes in climate. The assessment was undertaken in accordance with ARR 2019.

The hydrology and hydraulic impact assessment provided key inputs to the Project's revised reference design where the Project is located within the modelled flood extents. Key dependencies for the Project's revised reference design include:

- Modelling of the Existing Case 1% AEP event to ascertain existing conditions and inform the flood immunity for the Project's alignment, and to size drainage structures
- Modelling of 1 in 2,000 AEP event to provide inputs for bridge design and wider resilience assessment
- Modelling of rare flood events (1 in 10,000 AEP and PMF events) to assist in consideration of overtopping and embankment failure risks
- Modelling of smaller, more frequent events, to ensure the design performance is assessed and refined against these more regular events
- Modelling the full range of flood events to quantify potential impacts and inform mitigation measures

- Input to drainage design including scour protection design—water levels, flows and velocities from this assessment were used to inform the design of scour protection
- Input to structure selection and design for culverts and bridges.

3 Existing environment

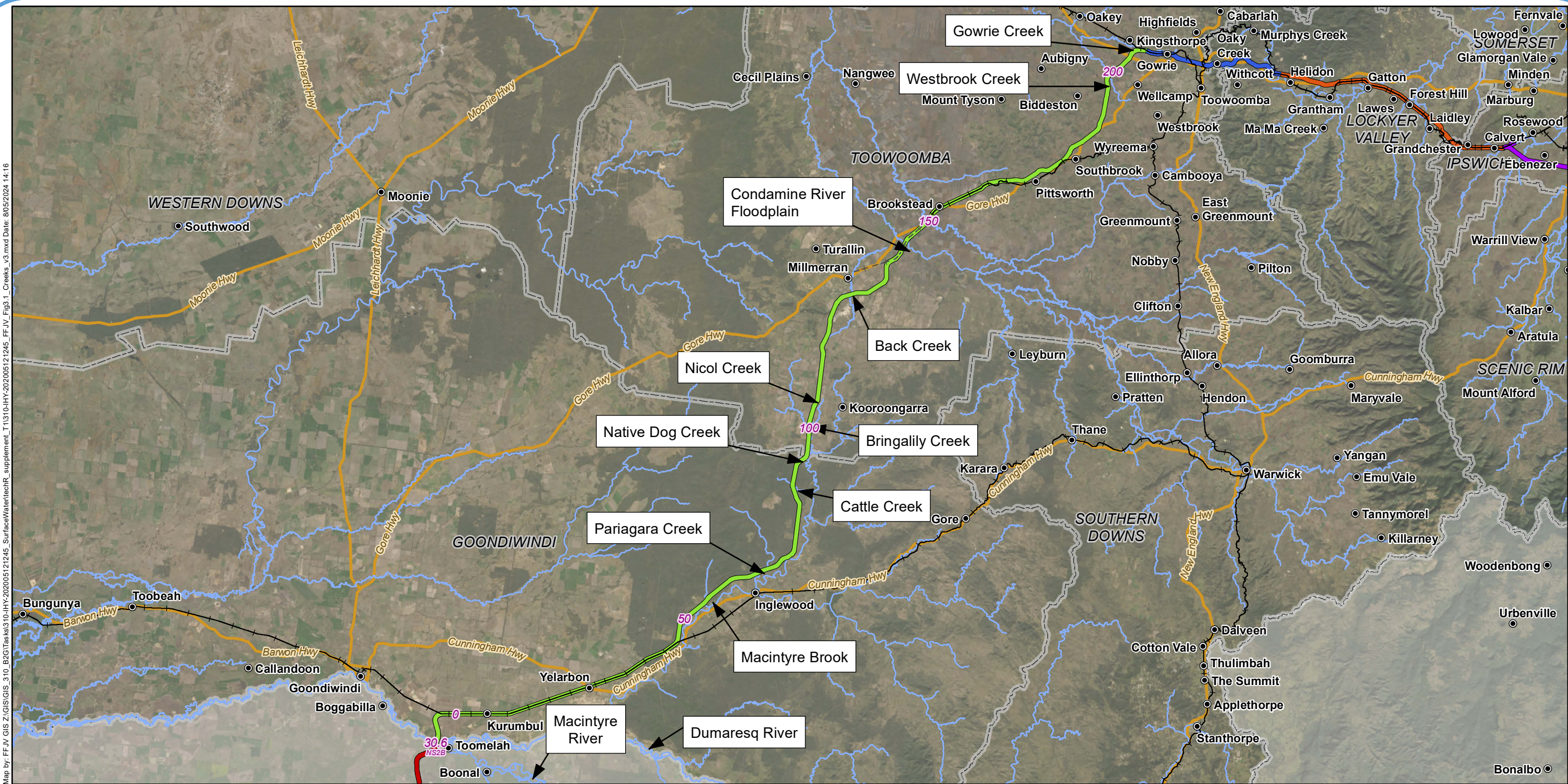
3.1 Waterways

There are several major waterways within the study area, as shown in Figure 3.1, with the key waterways being the Macintyre River, Macintyre Brook, the Condamine River and Gowrie Creek. Other major creek crossings include Pariagara Creek, Cattle Creek, Native Dog Creek, Bringalily Creek, Nicol Creek, Back Creek and Westbrook Creek.

The Border Rivers and Darling Downs floodplains have experienced many floods in recent years, including the 1956, 1976, 2011 and, more recently, the 2021 flood event. The floodplains are generally used for farming practices, and many landowners are reliant on characteristics of flooding across the floodplain for collection and storage of water for irrigation. The Condamine River floodplain, between Millmerran and Brookstead in particular, houses a large number of significant water storages (ring tanks).

Waterways are defined under the *Fisheries Act 1994* (Qld) and include a river, creek, stream, watercourse or inlet of the sea. The definition includes freshwater and tidal waters, both permanent and ephemeral waterways, and includes drainage features. It also includes channels along which fish are expected to move if they connect isolated water bodies to defined waterways during times of flow; however, it does not include isolated waterbodies where no connectivity is available.

The term 'watercourse' is defined under the *Water Act 2000* (Qld) as a river, creek, or other stream, which includes a stream in the form of an anabranch or a tributary where water flows either permanently or intermittently, regardless of flow frequency; however, a watercourse does not include any section of a feature that has a tidal influence or is upstream or downstream from a defined limit between potential estuarine and fresh water; therefore, by definition, all watercourses are waterways but not all waterways are watercourses.



3.2 Floodplain infrastructure

The floodplains located within the study area include several key existing infrastructure² assets near the Project alignment that could potentially influence local flooding behaviour, including:

- Gowrie Creek floodplain:
 - QR West Moreton Line
 - Warrego Highway
 - Kingsthorpe-Haden Road
 - Draper Road
 - Leeson's Road
 - Gowrie Junction Road.
- Westbrook and Dry Creeks floodplain:
 - Toowoomba Wellcamp Airport
 - Toowoomba-Cecil Plains Road
 - Brimblecombe Road.
- Condamine River floodplain:
 - Gore Highway
 - Town of Pampas
 - QR—Wyreema to Millmerran Line
 - Pampas-Horrane Road
 - Millmerran-Leyburn Road
 - Doug Hall Poultry at Yandilla
 - Several stream gauges including Pampas (Department of Regional Development, Manufacturing and Water (DRDMW)), Yarralong Weir (Sunwater), Centenary Bridge (Bureau of Meteorology (BoM)), etc.
- Back Creek floodplain:
 - Commodore Mine
 - Millmerran Power Station
 - Millmerran-Inglewood Road
 - Kooroongarra Road.
- Macintyre Brook floodplain:
 - Cunningham Highway
 - Town of Inglewood
 - Inglewood-Texas Road
 - Texas-Yelarbon Road
 - Desert Creek Road
 - Bybera Road

² This is not an exhaustive list of local roads. All local roads have been assessed as part of this revised draft EIS.

- Cremascos Road
 - Town of Yelarbon
 - Yelarbon-Keetah Road
 - Yelarbon flood levee
 - Kildonan Road.
- Levees, dams, ring tanks and pump houses from farming practices.

3.3 Catchment classification

The Project alignment crosses several existing flowpaths in the different catchment areas that contribute flows to the cross-drainage structures. To determine the appropriate hydrologic methods for the local drainage design, the existing catchments were categorised based on the contributing catchment areas. Table 3.1 presents the drainage catchment classification criteria and number of catchments relating to each classification.

Table 3.1 Drainage catchment classification

Catchment size	Drainage catchment classification	Number of catchments traversed by the Project
Less than or equal to 10 km ²	Minor	165
Greater than 10 km ² and less than or equal to 100 km ²	Moderate	5
Greater than 100 km ²	Major	8

Table note:

km² = square kilometres

Major and moderate floodplains are discussed in Sections 5 to 17, while minor catchments are covered in Section 18.

3.3.1 Major catchments

Major catchments (greater than 100 km²) that are traversed by the Project alignment include Gowrie Creek, the Westbrook/Dry Creeks system, the Condamine River, Back Creek, Bringalily Creek, Pariagara Creek, Macintyre Brook and the Macintyre River.

3.3.2 Moderate catchments

Nicol Creek is one of three moderate waterways that the Project alignment traverses before flowing into Canning Creek. The other waterways are Native Dog Creek and Cattle Creek. Due to their proximity to one another, all four waterways were modelled in a unified Canning Creek model. Two other moderate waterways that are traversed by the Project alignment include tributaries of the Macintyre Brook at Bybera Road and Cremascos Road.

3.3.3 Minor catchments

The local drainage catchments (minor catchments) for the Project were delineated in a 12D model using the following data sets:

- ARTC supplied 1 m Light Detection and Ranging (LiDAR) from 2015
- Queensland Government Shuttle Radar Topographic Mission (SRTM) 25 m hydrologic-enforced shuttle radar
- The Project's earthworks revised reference design.

The cross-drainage structures for the minor catchments are identified as 'Local' in Appendix B. Details of the hydrologic and hydraulic assessments for these local cross-drainage structures are provided in Section 18.

3.4 Catchment classification and model summary

3.4.1 Regional catchments (major and moderate catchments)

The hydrologic and hydraulic models that were developed for major and moderate floodplains (or regional catchments) that are traversed by the Project are summarised in Table 3.2. The hydraulic model locations and extents are shown in Figure 3.2.

Details regarding each model, including its development, calibration and spatial extent, are provided in the individual catchment sections within this report.

Table 3.2 Project hydrologic and hydraulic model summary—Regional catchments

Waterway	Catchment classification	Hydrologic modelling software	Hydraulic modelling software	Report section
Gowrie Creek	Major	RAFTS	TUFLOW	Section 5
Westbrook and Dry Creeks	Major	RAFTS	TUFLOW	Section 6
Condamine River	Major	URBS	TUFLOW	Section 7
Back Creek	Major	URBS	TUFLOW	Section 8
Nicol Creek	Moderate	URBS	TUFLOW	Section 9
Bringalily Creek	Major	URBS	TUFLOW	Section 10
Native Dog Creek	Moderate	URBS	TUFLOW	Section 11
Cattle Creek	Moderate	URBS	TUFLOW	Section 12
Pariagara Creek	Major	URBS	TUFLOW	Section 13
Macintyre Brook Yelarbon to Inglewood	Major	URBS	TUFLOW	Section 14
Macintyre Brook at Bybera Road	Moderate	URBS	TUFLOW	Section 15
Macintyre Brook at Cremascos Road	Moderate	URBS	TUFLOW	Section 16
Macintyre River	Major	URBS	TUFLOW	Section 17

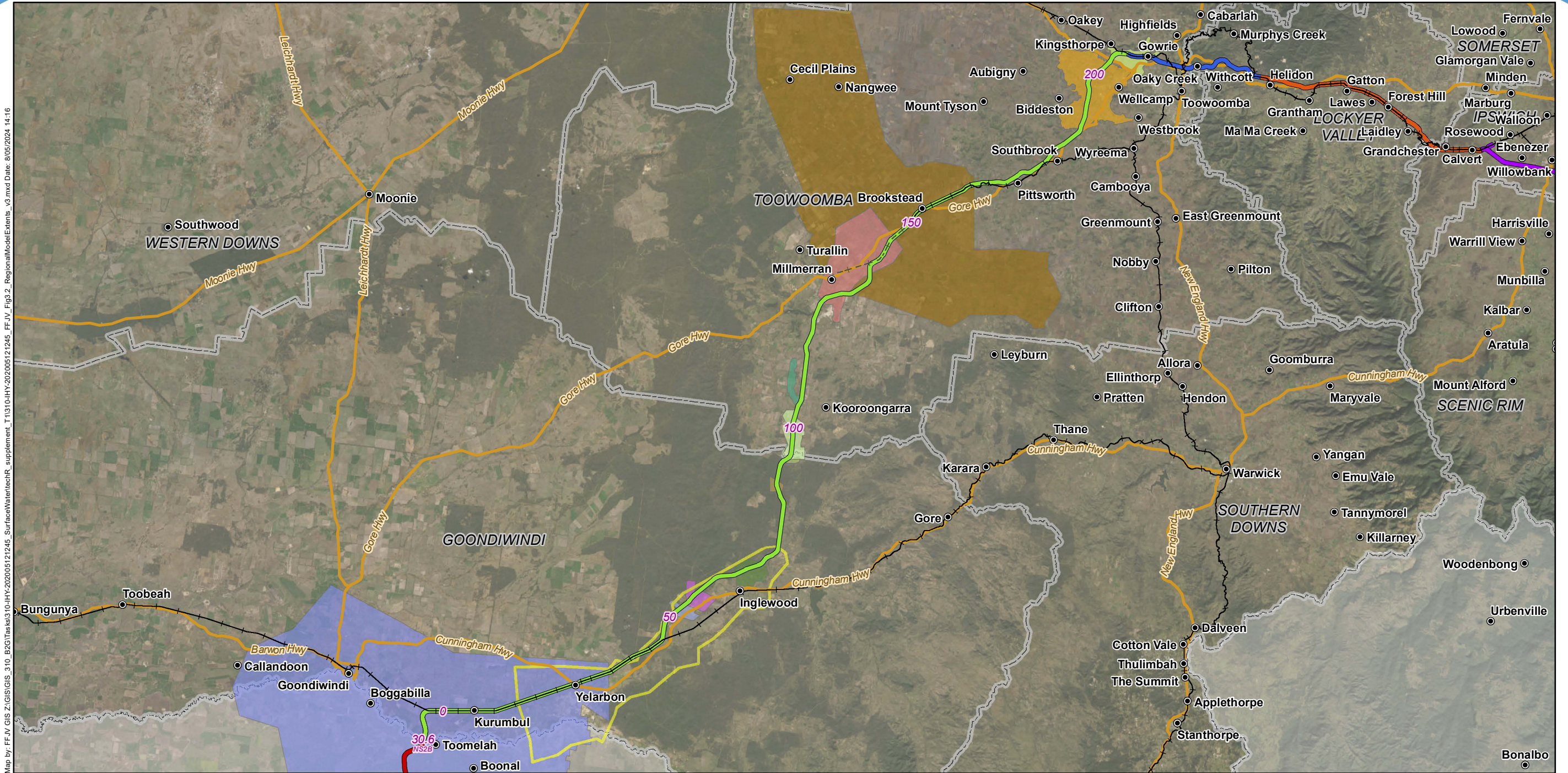
Table notes:

RAFTS: runoff-routing modelling

URBS= Unified River Basin Simulator (continuous/event rainfall runoff routing model)

TUFLOW: Hydraulic and hydrodynamic modelling software suite

Map by: FF JV GIS Z:\GIS\GIS_310_B2G3Tas ks310-HY-202005121245_SurfaceWater\techR_supplement_T11310-HY-202005121245_FF JV_Fig3.2_RegionalHydraulicModelExtents_v3.mxd Date: 8/05/2024 14:16



Legend

- 5 Chainage (km)
- Localities
- North Star to NSW/QLD border alignment
- Border to Gowrie alignment
- Gowrie to Helidon alignment
- Helidon to Calvert alignment
- Calvert to Kagaru alignment
- Existing rail (operational)
- Existing rail (non-operational)

Regional Hydraulic Model Extents

Back Creek	Native Dog Creek
Bringalily Creek	Nicol Creek
Bybera Road	Pariagara Creek
Cattle Creek	Westbrook Creek
Condamine River	Border Rivers - Macintyre River
Cremascos Road	Macintyre Brook
Gowrie Creek	



A3 scale: 1:800,000
0 6 12 18 24 30 km



Future Freight
Integrating Community, Environment and Engineering

Issue date: 08/05/2024 Version: 3
Coordinate System: GDA 1994 MGA Zone 56

Border to Gowrie
Figure 3.2:
Regional Hydraulic Model Extents

3.4.2 Local catchments (minor catchments)

The hydrologic and hydraulic models that were developed for minor (or local catchments) that are traversed by the Project footprint are summarised in Table 3.3. The hydraulic model locations and extents are shown in Figure 3.3.

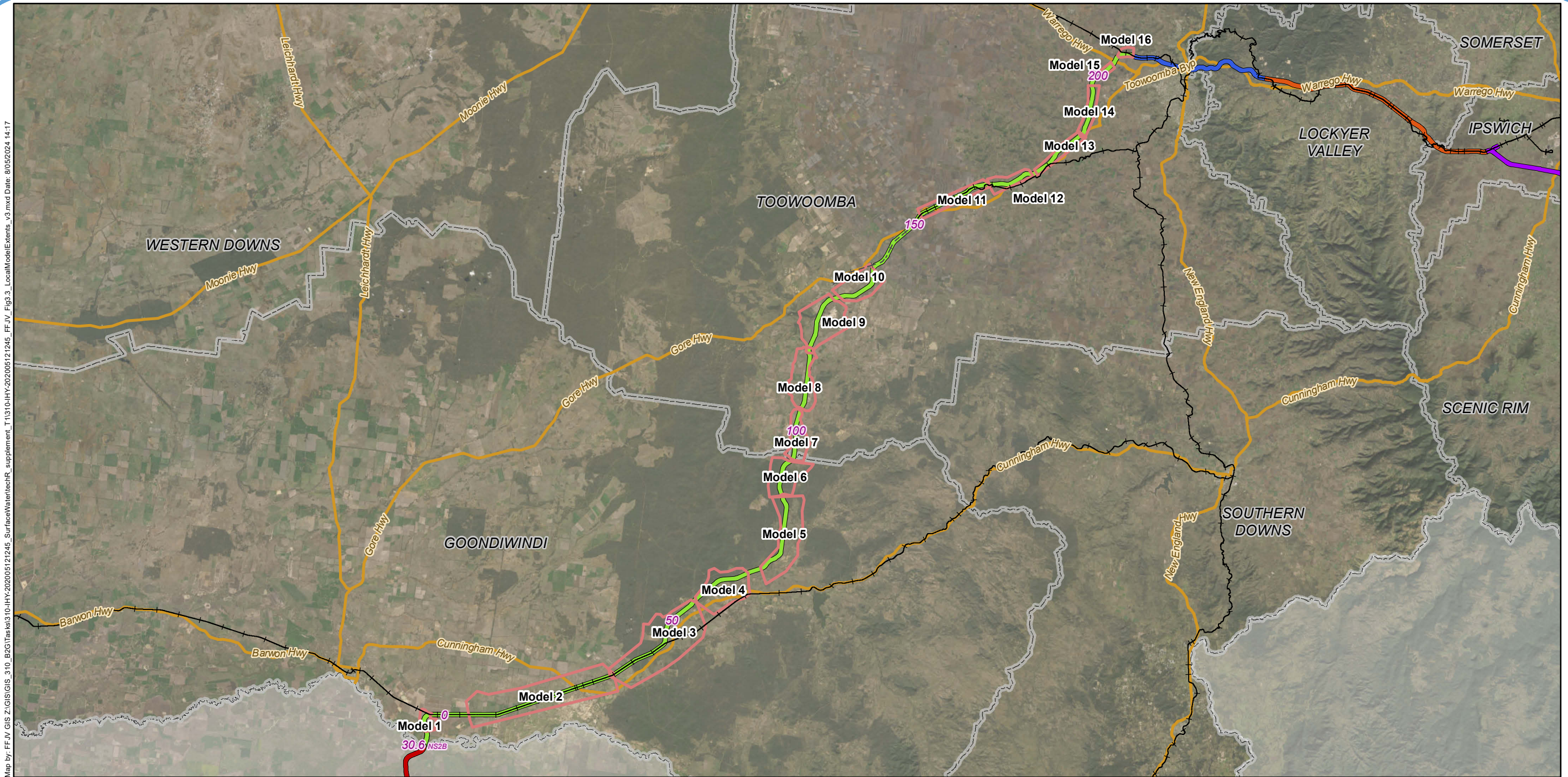
Further detail on the development of local catchment models can be found in Section 18.

Table 3.3 Project hydrologic and hydraulic model summary—Local catchments

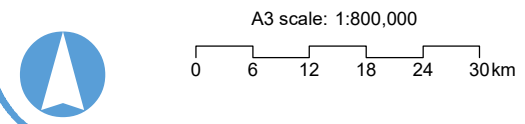
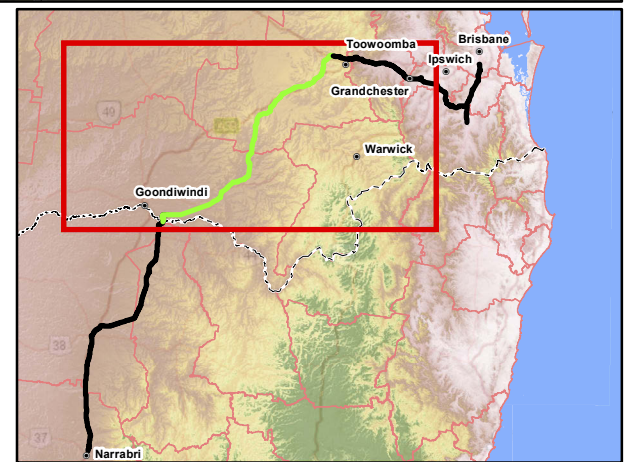
Model ID	Catchment classification ¹	Hydrologic modelling software	Hydraulic modelling software	Report section
1	Minor	12D Drainage Network Editor	TUFLOW	Section 18
2	Minor	12D Drainage Network Editor	TUFLOW	Section 18
3	Minor	12D Drainage Network Editor	TUFLOW	Section 18
4	Minor	12D Drainage Network Editor	TUFLOW	Section 18
5	Minor	12D Drainage Network Editor	TUFLOW	Section 18
6	Minor	12D Drainage Network Editor	TUFLOW	Section 18
7	Minor	12D Drainage Network Editor	TUFLOW	Section 18
8	Minor	12D Drainage Network Editor	TUFLOW	Section 18
9	Minor	12D Drainage Network Editor	TUFLOW	Section 18
10	Minor	12D Drainage Network Editor	TUFLOW	Section 18
11	Minor	12D Drainage Network Editor	TUFLOW	Section 18
12	Minor	12D Drainage Network Editor	TUFLOW	Section 18
13	Minor	12D Drainage Network Editor	TUFLOW	Section 18
14	Minor	12D Drainage Network Editor	TUFLOW	Section 18
15	Minor	12D Drainage Network Editor	TUFLOW	Section 18
16	Minor	12D Drainage Network Editor	TUFLOW	Section 18

Table note:

- 1 Within each Local catchment hydraulic model, multiple catchments exist and have been grouped together for the purposes of modelling.



- Legend**
- 5 Chainage (km)
 - North Star to NSW/QLD border alignment
 - Border to Gowrie alignment
 - Gowrie to Helidon alignment
 - Helidon to Calvert alignment
 - Calvert to Kagaru alignment
 - Existing rail (operational)
 - Existing rail (non-operational)
 - Major roads
 - NSW Qld border
 - Local Model Extents
 - Local Government Areas



Border to Gowrie
Figure 3.3:
Local Drainage Hydraulic Model Extents

4 Design requirements, standards and guidelines

4.1 Hydraulic design criteria

Table 4.1 outlines the hydraulic design criteria that have guided the Project's revised reference design. Detailed hydrologic and hydraulic modelling was undertaken to meet these design criteria with a series of iterations undertaken to incorporate design refinement. The resulting design outcomes relative to these design criteria are detailed in the impact assessment section of each catchment section.

Table 4.1 Project hydraulic design criteria

Performance criteria	Requirement
Flood immunity	Rail line—1% AEP flood immunity to formation level.
Hydraulic analysis and design	Hydrologic and hydraulic analysis and design to be undertaken based on ARR 2019 and State/local government guidelines. ARR 2019 interim climate change guidelines are to be applied with an increase in rainfall intensity to be considered. No sea-level change consideration required due to location outside tidal zone. ARR 2019 blockage assessment guidelines are to be applied.
Longitudinal drainage design	The following criteria has been adopted for the design of the longitudinal drainage and the diversion drains: ■ Embankment drain—1% AEP below formation level. Drain section shall convey a minimum 2% AEP. ■ Cut drain—1% AEP ■ Catch drain and catch bank—1% AEP ■ Batter drain—1% AEP ■ Batter chute—1% AEP ■ Diversion drain—1% AEP ■ Freeboard—300 millimetres (mm) for diversion, cut and embankment drains and zero for others.
Scour protection of structures	All bridges and culverts should be designed to reduce the risk of scour with events up to 1% AEP event considered. Mitigation to be achieved through providing appropriate scour protection or energy dissipation, or by changing the drainage structure design.
Structural design	1 in 2,000 AEP event to be modelled for bridge design purposes.
Extreme events	Damage resulting from overtopping to be minimised.
Flood flow distribution	Locate structures to ensure efficient conveyance and spread of floodwaters.
Sensitivity testing	Consider climate change and blockage in accordance with ARR 2019. Understand risks posed and Project design sensitivity to climate change and blockage of structures.

4.2 Flood Impact Objectives

The nominated FIOs, as detailed in Table 4.2, were developed in consultation with the Expert Flood Panel to provide guidance as to the point at which a more detailed consideration of impacts is required when they are exceeded. The Project will target achieving the FIOs for events up to and including the 1% AEP (without climate change) event for land, FSRs, and/or infrastructure that is potentially impacted by the Project. These FIOs apply to the detailed design, and commissioning and operations stages of the Project only and do not apply to the pre-construction, early works and construction works stage. Pre-construction, early works and construction works will be guided by a FRA as part of the Outline EMP and subsequent CEMP. The FRA approach is discussed in Section 20.6 and will be documented via a Flood Risk Management Plan. ARTC's detailed approach to managing FIO exceedances is detailed in Section 22.4.

Where it is not practicable or feasible to achieve the FIOs at FSRs and/or the nominated land uses, acceptable impacts and/or appropriate mitigation measures will be determined on a case-by-case basis, including through consultation with stakeholders and landowners.

It is important to note that all model results need to be interpreted with a level of engineering judgment, and this is particularly relevant to the application of the FIOs. It is common for models to show localised impacts that are not realistic and that are referred to as model artefacts or model 'noise'. This type of impact should be disregarded and can include isolated impacts far removed from the Project corridor or marginal exceedances around the flood fringe.

The FIO assessment outcomes are detailed in the Impact Assessment section of each catchment section of this report.

Table 4.2 Flood Impact Objectives

Parameter	Objectives					
Change in peak water levels (afflux) ¹	Existing habitable ² and/or commercial and industrial buildings/premises (e.g. dwellings, schools, hospitals, shops) and sensitive infrastructure ³	Yards or gardens of residential or commercial/ industrial properties/lots (excluding habitable ² dwellings/ buildings)	Existing non-habitable structures (e.g. agricultural sheds, pump-houses)	Existing local roads currently in use Existing rail lines	State-controlled roads	Agricultural and grazing land/forest areas and other non-agricultural land
	≤ 10 mm ⁴	≤ 50 mm ⁵	≤ 100 mm ^{4,6}	10 to 100 mm ⁷	10 to 20 mm ⁷	≤ 200 ⁸ mm with localised ⁹ areas up to 400 mm
Changes in peak water levels are to be assessed against the FIOs. Changes in peak water levels can have varying impacts on different infrastructure/land. Flood Impact Objectives were developed to consider the FSRs in the vicinity of the Project. It should be noted that in many locations the presence of existing buildings or infrastructure limits the change in peak water levels. For peak water levels assessed at any structure, the change in peak water level is measured relative to the existing floor level.						
Change in duration of inundation ^{1,4}	Identify changes to time of inundation by determining time of submergence in Existing and Developed¹⁰ Cases. Assess impacts against the following objectives for habitable floors²: - Where existing flood inundation is less than 1 hour—up to 1 hour duration of inundation - Where existing flood inundation of 1 hour or more occurs—up to a 5% increase in duration of inundation. For impacted roads/rail, the duration of inundation can increase by up to 10%, subject to the determination of the time of submergence and consideration of impacts on accessibility/egress during flood events, in consultation with the relevant authority. Assess impacts against the following objectives for all other land uses: - Where existing flood inundation is less than 1 hour—up to 1 hour duration of inundation - Where existing flood inundation of 1 hour or more occurs—up to a 10% increase in duration of inundation. The duration performance targets do not apply to newly flooded land where compliant with afflux criteria (see also General Notes in this table).					
Flood flow distribution ¹	Aim to minimise changes in natural flow patterns and minimise changes to flood-flow distribution across floodplain areas. This includes the objective of maintaining drainage paths that are conveying runoff from adjoining terrain, minor watercourses, and gullies, to avoid ponding of water and/or excessive duration of inundation. Identify any changes and justify acceptability of changes by assessing the risk, with a focus on land use and FSRs. The identification of changes to flow distribution is to include the consideration of a range of floods, from small (frequent) events (for conditions throughout the event) to large and extreme (infrequent) events. This exercise will be undertaken to identify (and, where deemed necessary, mitigate) any increased risk to FSRs associated with flow distribution changes.					

Parameter	Objectives
Velocities ¹	Maintain existing velocities where practical or minimise increases in velocities. Identify changes to velocities and impacts on external properties. The Erosive Threshold Velocity (ETV) for natural ground surfaces should be established from a site-specific assessment by a suitably qualified specialist, and in consideration of engineering guidelines. For sealed surfaces, this same approach could be applied, or through reference to suitable guidelines/specifications. Assess against the following performance objectives: - Sealed surfaces (or surfaces otherwise protected against erosion): - For existing velocities equal to or greater than 1 metre per second (m/s) (or the defined ETV), the increase in velocity is limited to 20% - For existing velocities (or velocities associated with new flowpaths) less than 1 m/s, the maximum design velocity is 1.2 m/s (or the defined ETV). - Natural ground surfaces, including watercourses, agricultural land, unimproved grazing land and other unsealed or unprotected areas: - For existing velocities equal to or greater than 0.5 m/s (or the defined ETV), the increase in velocity is limited to 10% (or within an acceptable range as determined by geomorphological assessment) - For existing velocities (or velocities associated with new flowpaths) less than 0.5 m/s (or the defined ETV), the maximum design velocity is 0.5 m/s (or the defined ETV).
Hazard (velocity x depth product) ¹	Assess against the following objectives: Roads/rail, urban and commercial areas, dwellings: - A 10% increase in velocity x depth product. Other land: - A 20% increase in velocity x depth product. (In both cases where the velocity x depth product is below 0.15 square metres per second (m²/s) in the Developed⁶ Case, no percentage change performance targets apply)
Extreme event risk management	Consider risks posed to neighbouring properties and emergency access/egress for events larger than the 1% AEP event to minimise unexpected or unacceptable impacts. At sites with existing sensitive infrastructure³, uses involving vulnerable people and/or any critical road network that was designed to be immune to flooding in any extreme event, the objective for increase in peak water level under the 0.2% AEP (1 in 500) is 10 mm¹¹. At existing habitable² dwellings and/or commercial and industrial buildings/premises under the 0.05% (1 in 2,000) AEP event, a maximum increase in peak water level of 250 mm applies.
Sensitivity testing	Consider risks posed by climate change and blockage in accordance with ARR 2019. Undertake assessment of impacts associated with Project alignment for both scenarios.
General notes:	- Justification must be provided in the selection of a governing FIO where two or more competing FIO may apply—in the absence of such justification the most limiting of the applicable FIOs shall apply. - Erosive Threshold Velocity means the velocity at which water movement has the potential to create scour or erosion. For natural surfaces, the erosive threshold of the soil/ground cover/vegetative conditions shall be established from an assessment by a suitably qualified specialist. In the absence of such assessment being undertaken an ETV of 0.5 m/s shall be assumed. - The effects of any increased lateral spread of floodwaters (i.e. associated with permissible afflux) beyond 1 hectare (ha) or 5% of the affected lot area should be considered on merit, taking into account affected receptors and land-usage (e.g. flood depth, flood duration, etc.). - The term 'Road' relates to the operational road surface. - Within the rail corridor erosion protection measures will be installed (as required). This is with a view to managing the risk of scour propagating beyond the rail corridor boundary. - Consultation shall be undertaken using the full suite of flood impact information (including afflux, velocity, duration and hazard). In locations where the afflux or velocity FIOs are exceeded, the change in flood hazard shall be communicated to the landowner with respect to the combined flood hazard classifications, as defined in ARR 2019 and the velocity x depth product.

Table notes:

- These FIOs apply for events up to and including the 1% AEP event and relate to land outside the rail corridor (as well as roads and level crossings that lie within/traverse the rail corridor). Where transport corridors are shared or lie immediately adjacent to the Inland Rail corridor, relevant FIOs apply as outlined in Table 1 (i.e. for existing rail, State-controlled roads, local roads, etc.).
- Habitable floors include all lawfully occupied dwellings regardless of land-use zoning.

- 3 Sensitive infrastructure means infrastructure that is an essential service required to operate during emergency events, including water treatment facilities, telecommunications substations and electrical substations.
- 4 Data permitting, and based on a review of calibration outcomes, where negative calibration bias is identified, a suitable tolerance (commensurate with the bias) will be added to the design flood levels to assess afflux and duration acceptability against the nominated FIO threshold.
- 5 This FIO reduces to ≤ 20 mm for developed lots that are less than 1,000 square metres (m^2) in area.
- 6 The current usage of non-habitable structures will be reviewed as design progresses and additional detail becomes available (e.g. animal shelter versus horticultural packing facility). This could influence the application of one of the other FIOs, which may be more suited to the specific usage of the structure.
- 7 Consultation will be undertaken with the relevant asset owner based on the application of the appropriate FIOs. This consultation will present all changes (>10 mm) across the road or rail network to identify specific roads or sections of rail that require local and/or route-specific considerations. In flood sensitive locations this may entail a lower afflux threshold being adopted.
- 8 Where the Project alignment crosses the Condamine River and Lockyer Creek floodplains, this objective reduces to 100 mm (with localised⁹ areas up to 200 mm), in light of the combination of floodplain sensitivity, inundation duration, and land use (intensive horticulture).
- 9 Localised is defined as the lesser of <1.0 ha or 5% of an individual lot.
- 10 Developed scenario implies fully constructed/operational rail line and associated works.
- 11 The 0.2% AEP (1 in 500) event is only required to be assessed where adherence to the FIO cannot be inferred from the 1% AEP and 1 in 2,000 AEP event results.

4.3 Relevant standards and guidelines

The design standards applicable for the hydrologic and hydraulic analysis are listed below:

- *AS7637:2014: Hydrology and Hydraulics* (Rail Industry Safety and Standards Board, 2014)
- *Australian Rainfall and Runoff: A Guide to Flood Estimation*, (Ball, J., Babister, M., Nathan, R., Weeks, W., Weinmann, E., Retallick, M., Testoni, I., (Editors), 2019).
- *Bridge Scour Manual* (DTMR, 2019)
- *Evaluating Scour at Bridges*(United States Department of Transport – Federal Highway Administration(US Department of Transport), 2012
- *Floodplain management for erosion control and high productivity on the Darling Downs* (Department of Primary Industries, 1993)
- *Guide to Road Design Part 5: Drainage – General and Hydrology Considerations* (Austroads, 2021)
- *Handbook 7 – Managing the Floodplain: A Guide to Best Practice in Flood Risk Management in Australia* (Australian Institute for Disaster Resilience (AIDR), 2017)
- *Hydraulic Design of Energy Dissipaters for Culverts and Channels* (US Department of Transport; 2006)
- *Road Planning and Design Manual –Volume 3, Part 5: Drainage and Hydrology Considerations* (DTMR, 2021)
- *Soil conservation guidelines for Queensland* (Department of Science, Information Technology and Innovation, 2015)
- *State Planning Policy – state interest guidance material – Natural hazards, risks and resilience – Flood.* (Department of Infrastructure, Local Government and Planning, 2017).

5 Gowrie Creek

The Project alignment does not cross Gowrie Creek but, instead, is within the study area via bordering the 1% AEP floodplain extents, before connecting into the Gowrie to Helidon section of Inland Rail at Draper Road. Under the 1% AEP event, around the Leeson's Road/Draper Road junction, the Existing Case peak depth of water is approximately 4 to 5 m in the Gowrie Creek channel, and up to around 1 to 2 m on the floodplain at the Project alignment. The floodplain inundation under the 1% AEP event varies between 200 m and 500 m wide. Several small, un-named flow paths draining the area to the north of the Warrego Highway cross the Project alignment.

The location of the Project alignment in relation to Gowrie Creek is shown in Figure A1-A in Volume 2 – Appendix A.

5.1 Data collection and review—Gowrie Creek

Available background information, including existing hydrologic and hydraulic models, survey, streamflow data, available calibration information and anecdotal flood data, has been gathered. Data was sourced from a wide range of stakeholders including:

- Toowoomba Regional Council (TRC)—existing flood studies and stream gauging data
- BoM—rainfall and stream gauging data
- DRDMW—stream gauging data
- QR—existing infrastructure details
- DTMR—existing infrastructure details.

5.1.1 Previous studies

A number of previous hydrology and hydraulic studies were sourced as part of this assessment. A review of each study was undertaken to determine suitability for use on the Project, as documented in the following sections.

5.1.1.1 Technical Report on the Oakey Flood of 10-11 January 2011, BMT WBM, 2011

This study provided information on flooding in relation to the Oakey flood event of 10 to 11 January 2011.

5.1.1.2 Gowrie Creek Flood Risk and Management Study Volume 1, TRC, 2013a

This model covered the upper reaches of the Gowrie Creek catchment within Toowoomba city, including East Creek and West Creek. The model was calibrated for the 2010 and 2011 flood events. The design flood hydrology was based on *Australian Rainfall and Runoff: A Guide to Flood Estimation* (Institution of Engineers, 1987) (ARR 1987); therefore, design flood estimates were not consistent with ARR 2019.

5.1.1.3 Gowrie Creek Flood Risk and Management Peer Review, TRC, 2013b

This study provided a technical review of flood modelling work undertaken as part of the TRC (2013a) study by a Peer Review Panel.

5.1.1.4 Oakey Flood Study Final Report, TRC 2014

A hydrologic and hydraulic model was developed in RAFTS and MIKE FLOOD for the Oakey catchment as part of this study. Gowrie Creek was modelled as one of the Oakey Creek tributaries; however, the area of interest for the Project was not included. The model was calibrated to the 10 January 2011 flood event using historical observed flood marks and flows from Cranley and Oakey stream gauges.

5.1.1.5 Work Package 4, Historical study for Kingsthorpe and Gowrie Junction Final Report, DHI/WRM Water and Environment (WRM) 2014a

This study focused on a small reach of Gowrie Creek between Kingsthorpe and Gowrie Junction and only considered the flood behaviour of the January 2011 flood event. The study involved the development of a coupled one-dimensional (1D)/two-dimensional (2D) MIKE FLOOD hydraulic model.

5.1.1.6 Work Package 8, 2D Flood study for Cotswold Hills (Gowrie Creek Catchment) Final Report, DHI/WRM 2014b

This study focused on small tributaries of Gowrie Creek in the vicinity of Cotswold Hills Township and did not explicitly cover flooding in Gowrie Creek itself. The study involved the development of a hydrologic model (RAFTS) and a coupled 1D/2D MIKE FLOOD hydraulic model. The design flood hydrology was based on ARR (1987); therefore, design flood estimates were not consistent with ARR 2019.

5.1.1.7 Toowoomba North-South Transport Corridor

Hydrologic and hydraulic models for Gowrie Creek were developed as part of studies by DTMR for the Toowoomba North-South Transport Corridor (TNSTC). The hydrologic model has been calibrated to the 2010, 2011 and 2013 events, and design events were modelled in accordance with ARR 2019. The hydraulic model included the 70 per cent reference design for the Project, the InterlinkSQ/Charlton North Urban Stormwater Management Plan channel and the second range crossing. The hydrologic model has been reviewed, modified, and adopted for the revised draft EIS as recommended by the Expert Flood Panel.

5.1.2 Survey

Topographical and survey datasets used within the TNSTC hydraulic model was adopted for this assessment. The hydraulic model was primarily established using 2015 Toowoomba-Lockyer Valley Project LiDAR (one m grid resolution). Secondary datasets were also added including an earlier revision of the Inland Rail reference design, Toowoomba Second Range Crossing as-built survey, proposed Charlton North Urban Stormwater Management Plan's drainage channels and the proposed InterlinkSQ development. Representation of the proposed InterlinkSQ development is shown in Figure 5.1.

The TNSTC study's representation of the revised reference design alignment was updated with Project Digital Elevation Model (DEM) for the revised draft EIS assessment.

The survey data sources, and DEM developed for Gowrie Creek are shown in Figure A1-B in Volume 2 – Appendix A.

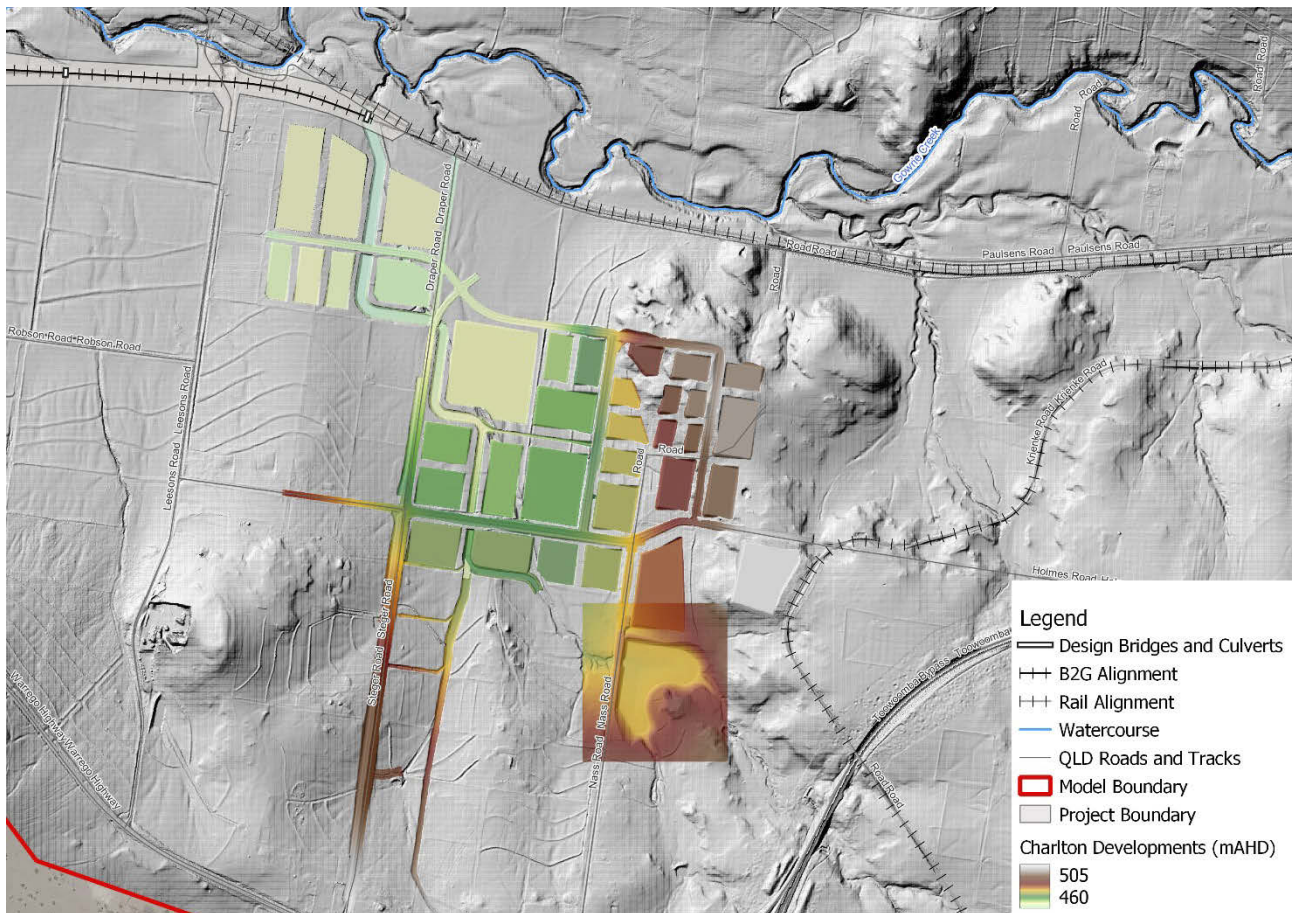


Figure 5.1 InterlinkSQ development terrain representation

5.1.3 Aerial imagery

Aerial imagery captured in 2015 was provided by ARTC, and was used to identify and confirm topographic and vegetative characteristics of the Gowrie Creek study area. Additional imagery outside the study area was sourced from Quantum Geographic Information System (QGIS) imagery in an open-source format.

5.1.4 Existing drainage structure data

Structure data contained within the previous hydraulic model was used in this assessment. Two culverts at Stankes Road and Burkes Road that could influence local flows were identified from the aerial imagery. The details of the culverts at these locations were assumed and used in the hydraulic model. Refer to Section 5.3.5 for additional details on hydraulic structures.

Additional existing rail culvert structure information was sought from QR for inclusion into the Gowrie Creek model.

5.1.5 Stream gauge data

There are two active stream gauges within the Gowrie Creek catchment study area that are operated by the DRDMW. Both gauges were adopted for model calibration and validation of design flows. Key details of each gauge are summarised in Table 5.1. Figure 5.2 shows the locations of each gauge within the Gowrie Creek catchment.

Table 5.1 Stream gauges within the Gowrie Creek catchment

Station number	Station name	DoR catchment area (km ²)	Owner	Latitude (South (°S))	Longitude (East (°E))	Start	End	Project purpose
422332B	Gowrie Creek at Oakey (Open)	142	DRDMW	-27.47	151.74	30/06/1992	Open	Calibration of hydrological model using flow data
422332A	Oakey (Closed)	142	DRDMW	-27.47	151.74	29/11/1967	30/01/1981	Validation of hydrologic models using flow data
422326A	Gowrie Creek at Cranley (Open)	47	DRDMW	-27.51	151.94	1/10/1969	-	Calibration of hydrological model using flow data

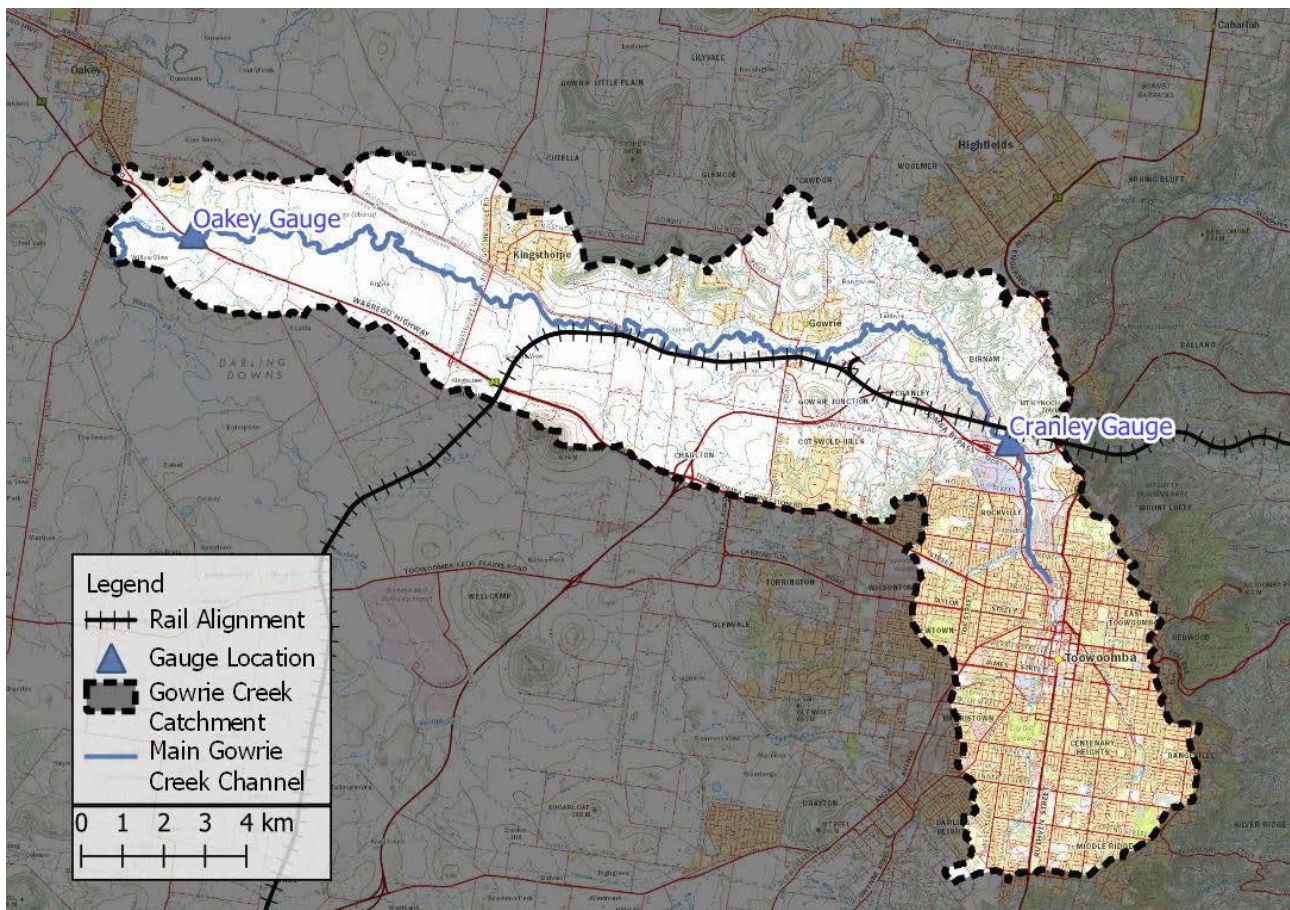


Figure 5.2 Stream gauge locations: Cranley and Oakey

5.1.6 Rainfall data

Table 5.2 outlines the key details of the 11 pluviography stations located within 30 km of the Gowrie Creek catchment (central location). Pluviograph data was used to determine the temporal and spatial rainfall distribution for each calibration event. Available daily rainfall gauges were used to supplement the sub-daily data and provide additional resolution in rainfall depths across the catchment.

Table 5.2 Summary of rainfall gauging stations used for calibration

ID	Station name	Station number	Easting	Northing	Owner
1	Alderley WO8 Rain Gauge REL (aka Alderley Street West Creek)	001	151.9458	-27.5849	TRC
2	Eastern Valley Rain Gauge REL	010	151.9705	-27.58	TRC
3	Gabbinbar Rain Gauge REL	011	151.9557	-27.6116	TRC
4	Gowrie Creek at Oakey	4223328 (541058)	151.74	-27.47	DoR
5	Middle Ridge Rain Gauge REL	016	151.9555	-27.5949	TRC
6	Picnic Tank Rain Gauge REL	018	151.9875	-27.5792	TRC
8	SPS42 Rain Gauge REL (Prince Henry Drive)	025	151.991	-27.556	TRC
9	Toowoomba Airport	41529	151.9134	-27.5425	BoM
10	Toowoomba Alert	540162	151.955	-27.5103	BoM
11	University of Southern Queensland (USQ)	Unknown	151.9332	-27.6034	USQ

5.1.7 Anecdotal and observed flood data

Flood markers represent evidence-based recordings of flood levels typically created by debris marks observed on the ground, buildings, fences and poles. The flood markers were used for hydraulic model validation in this assessment. The accuracy and reliability of debris mark data is inferior to streamflow gauge records (± 200 to 300 mm for flood markers as opposed to ± 100 mm for gauges). Observed flood markers were surveyed after the January 2011 event by TRC. A total of seven markers were available within the extent of the hydraulic model as shown in Figure A2-A in Volume 2 – Appendix A.

5.1.8 Site inspection

A site inspection was undertaken during February 2018. During the site inspection, all major waterway crossings were visited and inspected with photographs taken and details recorded of the crossing, existing drainage structures and surrounding catchment. An assessment of the relative roughness and blockage potential was undertaken during the site inspection.

5.2 Development of hydrologic model—Gowrie Creek

5.2.1 Model setup

As recommended by the Expert Flood Panel, the TNSTC rainfall-runoff routing software package (XP-RAFTS) hydrologic model was selected to inform the revised reference design, after a thorough review by Future Freight Joint Venture (FFJV). The focus of the hydrology analysis was on the tributaries draining towards Gowrie Creek that intersect with the Project alignment, as the alignment is primarily located outside the Gowrie Creek floodplain.

The review by FFJV aimed to confirm the suitability of the TNSTC hydrologic model for use in the current assessment. The results of the review are presented in the subsequent Sections 5.2.1.1 to 5.2.1.5.

5.2.1.1 Sub-catchments

Figure A1-C in Volume 2 – Appendix A shows the 285 sub-catchments that were delineated in the TNSTC hydrologic model. Where the Project alignment crossed sub-catchments, the local sub-catchment inflows were generally split along the alignment and proportioned to the sub catchment area in the hydraulic model. This method is described further in Section 5.3.6.

5.2.1.2 Fraction impervious

The catchment roughness (PERN) and percentage impervious values from the TNSTC hydrologic model were checked against aerial photography and appear to be reasonable. The Gowrie Creek catchment land use delineation is shown in Figure A1-E in Volume 2 – Appendix A.

5.2.1.3 Routing parameters

The TNSTC hydrologic model used lag-links to route peak flows through the catchment. Lag links record the travel time for the peak flow to travel the length of reach, usually the downstream catchment.

During the calibration exercise, lag-links were iterated several times (along with other parameters) until a reasonable fit and time between modelled and historical hydrographs was achieved.

5.2.1.1 Storage coefficient

No storage coefficient (Bx) was applied in the TNSTC hydrologic model. The default value of one was adopted.

5.2.1.2 Design rainfall data

A total of 11 Intensity-Frequency-Duration (IFDs) were extracted from the BoM to capture the spatial variation of rainfall across the Gowrie Creek catchment. Table 5.3 shows the data at four representative locations to demonstrate the variation in 1% AEP rainfall intensity across different parts of catchment for various storm durations.

As the Project is primarily influenced by local catchment draining towards Gowrie Creek, IFD Area 3 and IFD Area 4 are particularly relevant to this investigation.

Table 5.3 Comparison of IFD rainfall intensity for 1% AEP in Gowrie catchment

Duration (hours)	1% AEP rainfall intensity (mm/hr)			
	IFD Area 1 (Perth St – Upper Gowrie Creek)	IFD Area 2 (Cranley Gauge)	IFD Area 3 (B2G)	IFD Area 4 (Model boundary – Henningsen Road)
1	81.2	79.5	78.5	78.3
2	49.2	48.1	47.3	47.1
3	36.0	35.2	34.5	34.4
6	20.9	20.4	19.9	19.8
12	12.4	12.1	11.7	11.6
24	7.8	7.51	7.14	6.95

Table note:

mm/hr = millimetres per hour

5.2.1.3 Extreme event rainfall

Probable Maximum Precipitation (PMP) depths for durations up to 6 hours (for use in modelling the PMF event) were obtained using the methodology presented in *The Estimation of Probable Maximum Precipitation in Australia: Generalised Short-Duration Method* (BoM, 2003). Once the PMP was estimated, rainfall for events between 1 in 2,000 AEP and the PMF was then interpolated. For the 1 in 10,000 AEP event, areal reduction factors (ARF) were applied based on interpolation methodology presented in ARR 2019 Section 3.5 in Book 8, and the rainfall depths were estimated accordingly. The parameters adopted for PMP rainfall estimation are presented in Table 5.4.

Table 5.4 Input data for Probable Maximum Precipitation estimation

Factor	Value
Elevation adjustment factor (EAF)	1
Moisture adjustment factor (MAF)	0.82
Portion of rough catchment	100%
Gowrie Creek location and Catchment Area (km ²) ¹	(24.923824°S, 152.261843°E) and 5 km ²

Table note:

¹ Representative catchment area based on a typical catchment reporting to Project design cross-drainage infrastructure

5.2.1.4 Areal reduction factors

The typical catchments of interest for the Project within the Gowrie catchment ranged from less than 1 km² to approximately 10 km². Given the range and relatively small size of the catchment areas, it was considered practical to apply an ARF of one, instead of applying a minor reduction for each location of interest. This approach is considered conservative, as the ARF of one has also been applied to the coincident regional inflow from Gowrie Creek.

For storm durations between 45 minutes and 6 hours, the ARF would range between 78.3 per cent and 94.3 per cent if only regional flooding influence (i.e. Gowrie Creek inflow) was considered.

5.2.1.5 Design rainfall losses

Design rainfall losses were applied using a standard Initial Loss/Continuing Loss model. Initial losses occur at the start of a rainfall event and are analogous to an initial wetting of the catchment that must occur before the runoff can form, such as interception and the filling of puddles and minor other depression storages within the catchment. Continuing losses represent an ongoing loss of water from the catchment, such as infiltration.

For events up to the 1 in 2,000 AEP, the TNSTC hydrologic model setup was adopted where losses were as follows:

- Pervious Area Initial Storm Loss = 40 mm
- Pervious Area Continuing Loss = 4.5 mm/hr
- Impervious Area Initial Storm Loss = 28 mm
- Impervious Area Continuing Loss = 3.15 mm/hr

These losses were verified against the revised flood frequency analysis (FFA), as described in Section 5.2.3, and deemed suitable for this assessment.

For extreme events (1 in 10,000 AEP and PMP), a conservative mm initial loss and 1 mm/hr continuing loss were adopted.

5.2.2 Model calibration

The adopted TNSTC hydrologic model was calibrated to a number of historical storms. Daily and pluviography data provided an estimate of rainfall variation across the model catchment for each calibration event. Pluviography data was used to capture the temporality of each calibration event. The sub-daily patterns were applied to each model sub-catchment based on its proximity to the gauges.

The rainfall depths and temporal patterns (TP) were applied to the hydrologic model to produce estimates of flood hydrographs at the stream gauges identified for calibration. XP-RAFTS model parameters, PERN, impervious percentage, and lag times remained constant for the calibration events (Section 5.2.1). Initial and continuing losses were adjusted until a good fit was achieved at each gauge.

There are two stream gauges located within the Gowrie Creek catchment suitable for model calibration. The stream gauges are located within the Gowrie Creek catchment and include the Oakey Gauge (422332B) and the Cranley Gauge (422326A), as shown in Figure 5.2.

The historical flood events of December 2010, January 2011, January 2013, and February 2013 were selected as the primary events for model calibration. The January 2011 event was the largest and resulted in significant flood damages within the Toowoomba Region. Two additional historical events, January 2013, and February 2013 were chosen for model validation. These events had longer durations but were significantly smaller than the 2011 event.

A summary of the rainfall data used for model calibration is provided in Table 5.5, noting that:

- For the December 2010 event, more pluviography gauges were used compared to the other events as it was the only event for which TRC gauge data was obtained
- The Oakey TM gauge (422332B) was not available during the February 2013 event and was not able to be utilised.

Calibration hydrographs at each gauge and event are shown in Figure 5.3 to Figure 5.10.

A tabulation of losses used to produce the calibration hydrographs is provided in Table 5.6.

A summary of the overall calibration performance is provided in Table 5.7. A review of the historic streamflow records used for the calibration and their implications on the design is provided in Section 5.2.3.

Table 5.5 Rainfall gauges used for model calibration

Rain gauge	December 2010	January 2011	January 2013	February 2013
Alderley W08 REL (001)	X			
Eastern Valley REL (010)	X			
Gabbinbar REL (011)	X			
Oakey TM (422332B)	X	X	X	
Middle Ridge REL (016)	X			
Picnic Tank REL (018)	X			
Prescott & Goggs REL (008)	X			
SPS42 REL (052)	X			
Toowoomba Airport (041529)	X	X	X	X
Toowoomba Alert (540162)	X	X	X	X
USQ	X	X	X	X

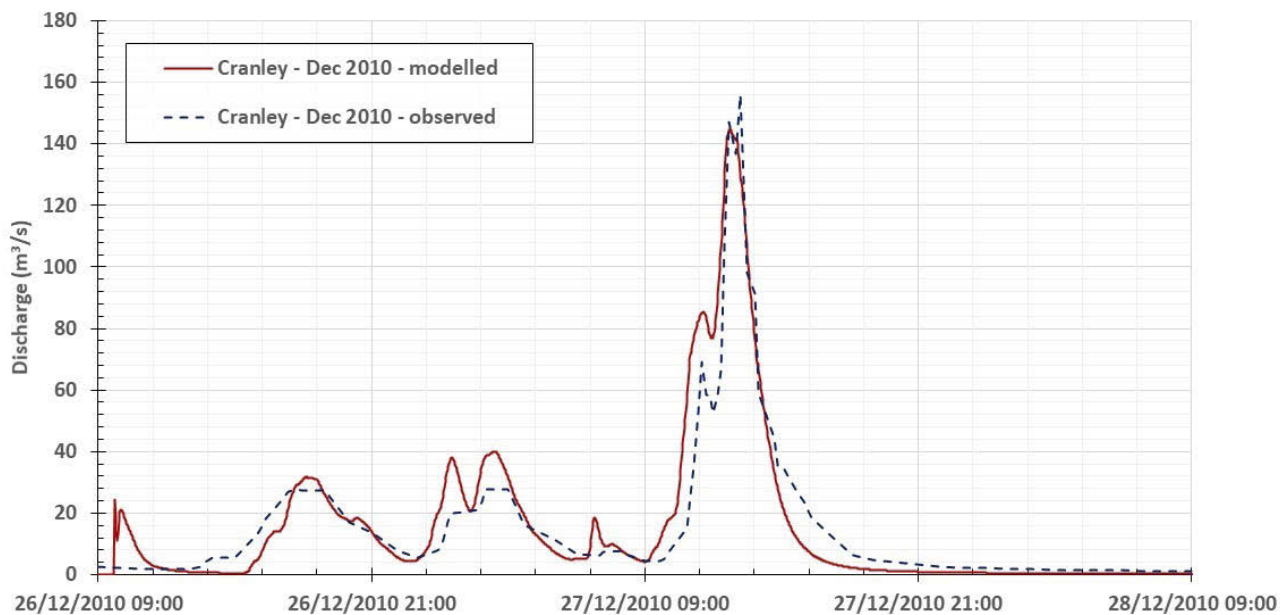


Figure 5.3 Comparison of XP-RAFTS and gauge results at Gowrie Creek at Cranley (422326A)—December 2010 event

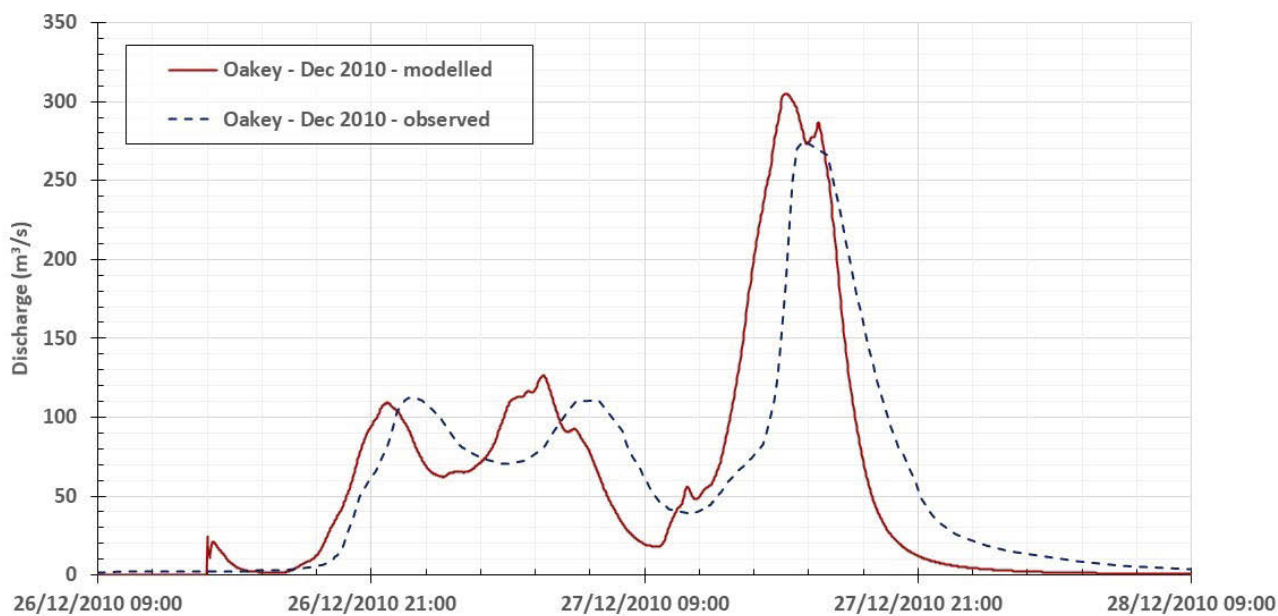


Figure 5.4 Comparison of XP-RAFTS and gauge results at Gowrie Creek at Oakey (422332B)—December 2010 event

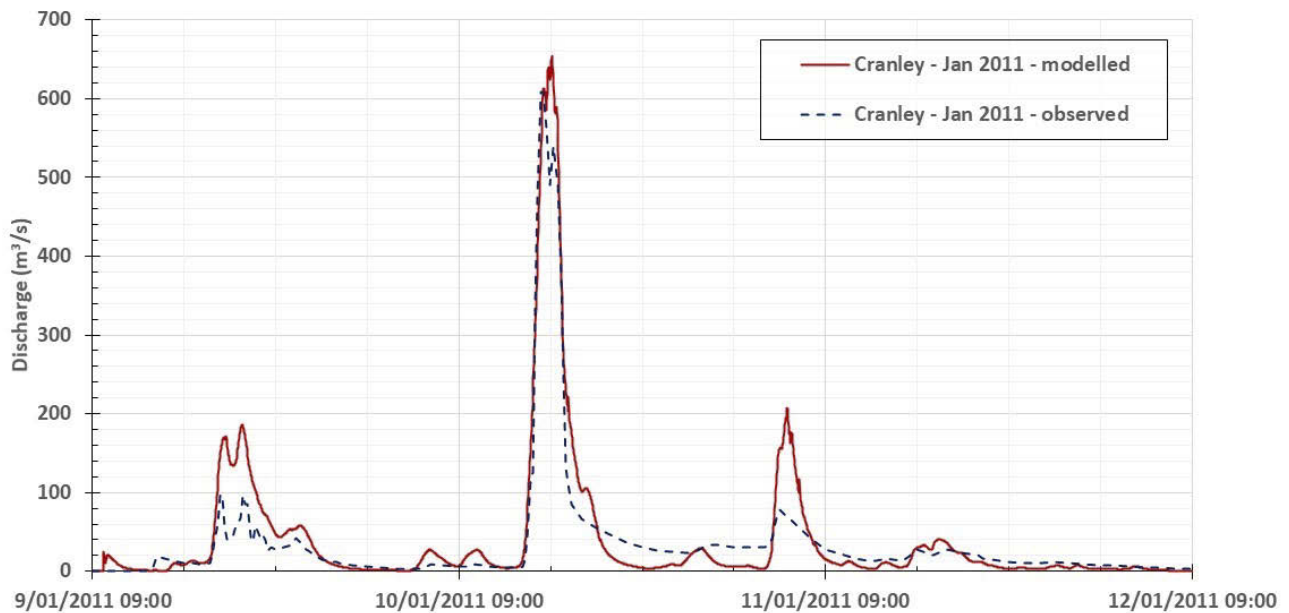


Figure 5.5 Comparison of XP-RAFTS and gauge results at Gowrie Creek at Cranley (422326A)—January 2011 event

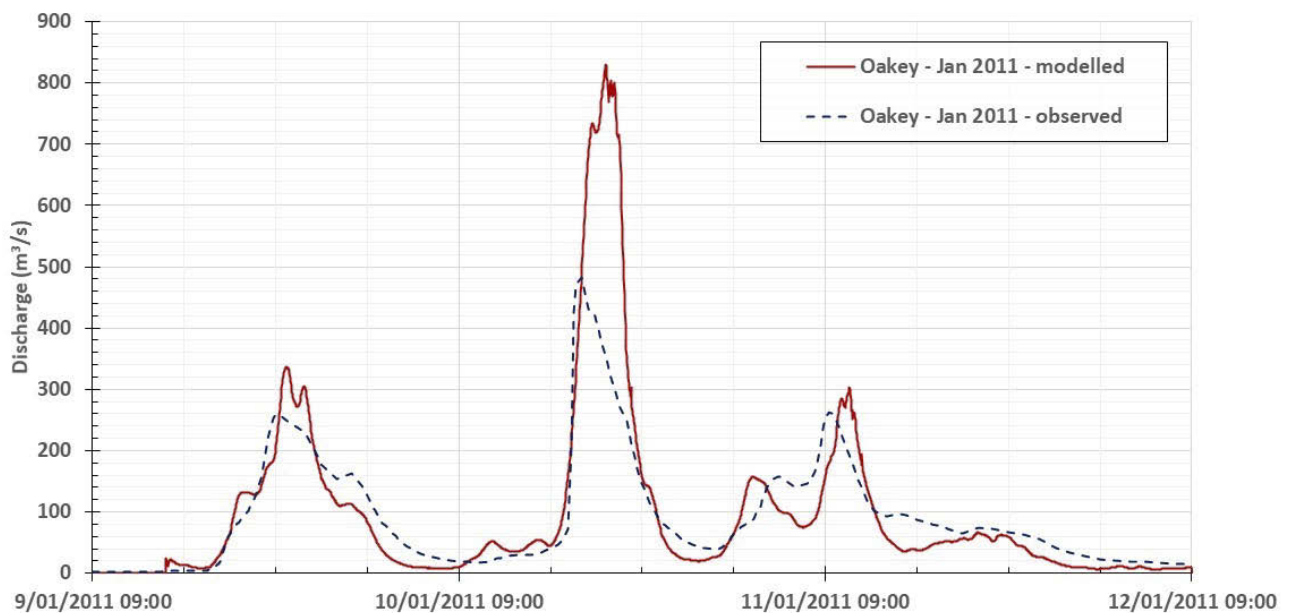


Figure 5.6 Comparison of XP-RAFTS and gauge results at Gowrie Creek at Oakey (422332B)—January 2011 event

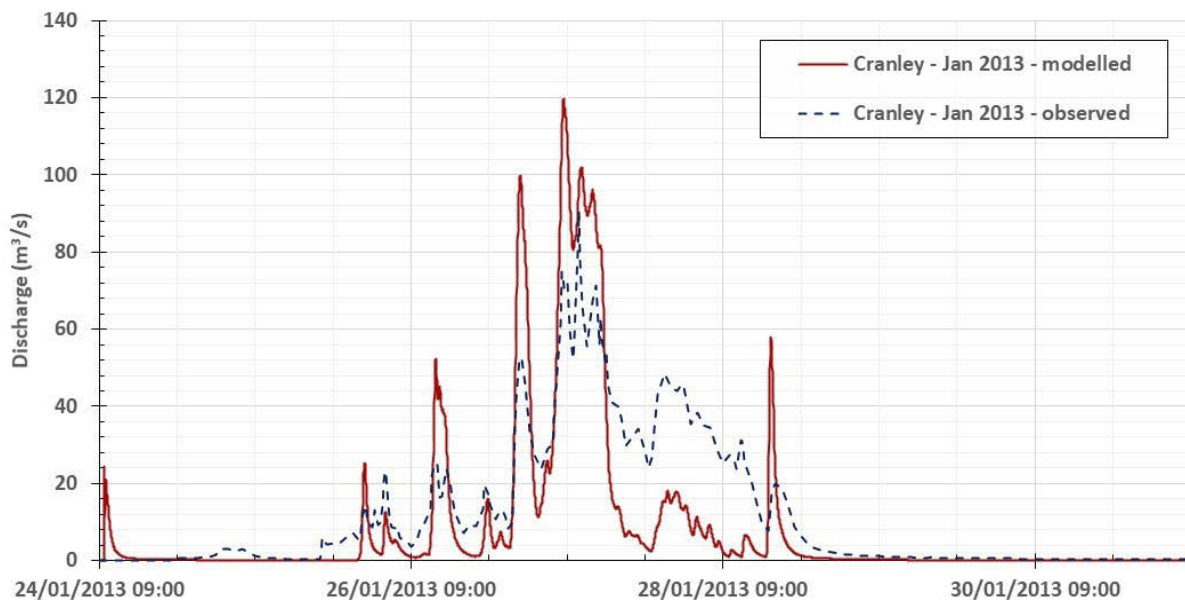


Figure 5.7 Comparison of XP-RAFTS and gauge results at Gowrie Creek at Cranley (422326A)—January 2013 event

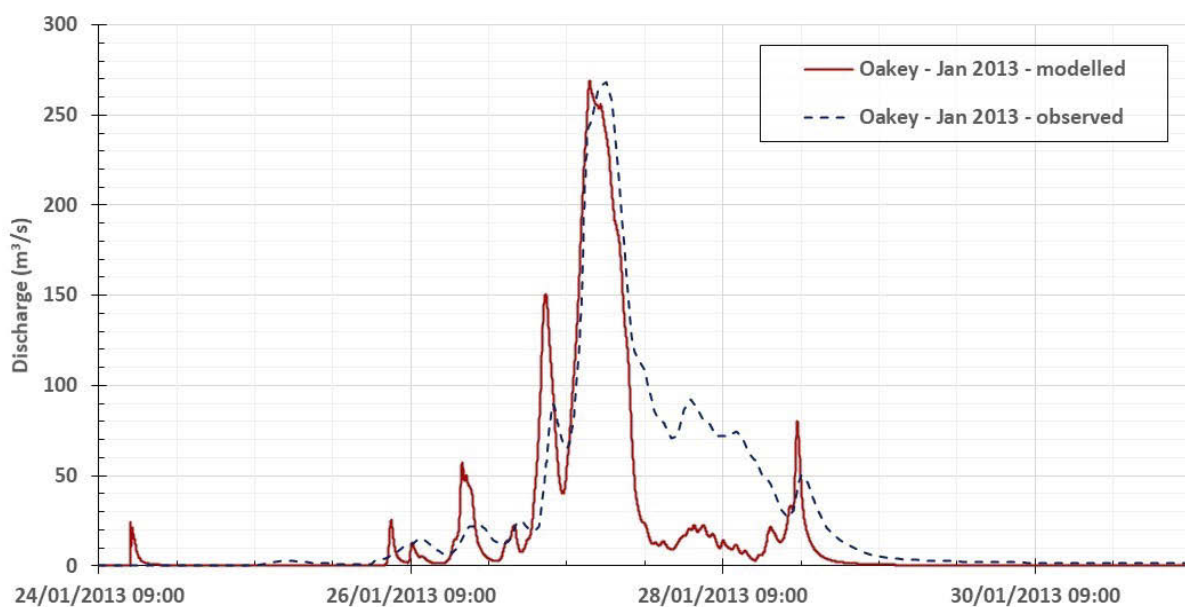


Figure 5.8 Comparison of XP-RAFTS and gauge results at the Gowrie Creek at Oakey (422332B)—January 2013 event

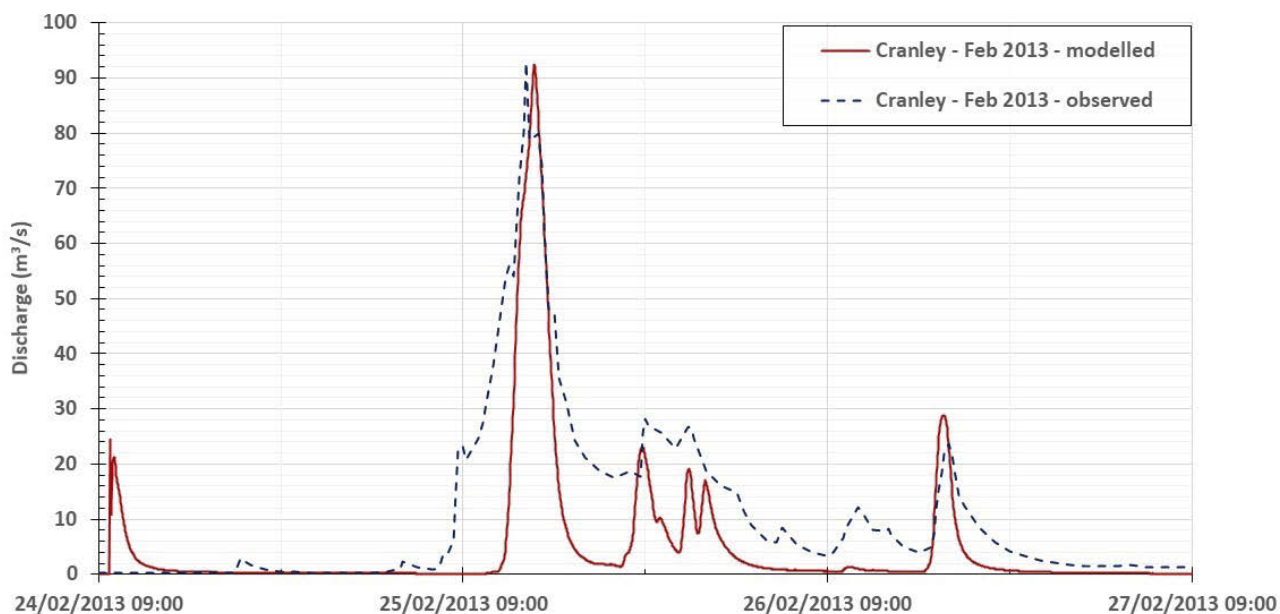


Figure 5.9 Comparison of XP-RAFTS and gauge results at Gowrie Creek at Cranley (422326A)—February 2013 event

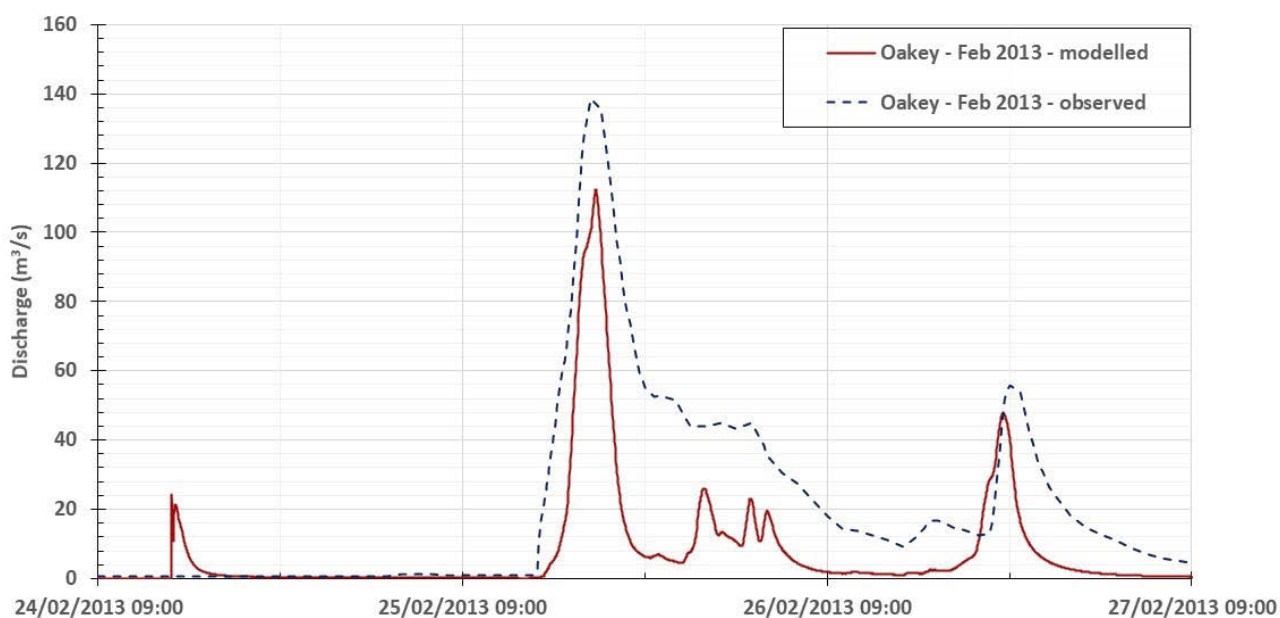


Figure 5.10 Comparison of XP-RAFTS and gauge results at the Gowrie Creek at Oakey (422332B)—February 2013 event

Table 5.6 Summary of calibrated losses

Loss type	December 2010	January 2011	January 2013	February 2013
Pervious IL (mm)	21.4	17	71	71
Pervious CL (mm/hr)	1.43	1.45	6.4	6.4
Impervious IL (mm)	15	12	50	50
Impervious CL (mm/hr)	1	1	4.5	4.5

Table notes:

CL = continuing loss

IL = initial loss

Table 5.7 Summary of Toowoomba North-South Transport Corridor hydrologic model calibration

Event	Approximate AEP ¹	Gauge	Modelled peak flow (m ³ /s)	Observed peak flow (m ³ /s)	Modelled hydrograph volume (GL)	Observed hydrograph volume (GL)	Difference in peak flow (%)	Difference in peak volume (%)
December 2010	9%	Cranley	145	156	2.7	2.6	-7%	4%
		Oakey	305	275	9.6	10.0	11%	-4%
January 2011	1%	Cranley	654	608	10.3	9.0	8%	14%
		Oakey	831	482 ²	25.0	23.8	72%	5%
January 2013	26%	Cranley	120	91	5.3	7.2	32%	-26%
		Oakey	269	268	11.3	17.3	0%	-35%
February 2013	26%	Cranley	92	93	1.3	2.8	-1%	-54%
		Oakey	112	139	2.1	5.4	-19%	-61%

Table notes:

m³/s = metres cubed per second

GL= Gigalitres

1 Based on observed peak flow at Cranley

2 Suspected gauge malfunction during peak

5.2.2.1 Discussion of calibration results

The results of the TNSTC hydrologic model calibrations are generally positive. The significant December 2010 (estimated at around a 9% AEP event) and January 2011 (estimated at around a 1% AEP event) events were well represented in terms of the peak flow, timing to peak and runoff volume. A discrepancy in the recorded peak flow during the January 2011 event was observed, likely due to a malfunction of the Oakey gauge during the event peak. Excluding the suspected faulty peak, the observed and modelled hydrographs for the January 2011 event were found to be well aligned.

The model also captured the peak times and burst shapes for the lesser January 2013 and February 2013 events (both estimated at around a 26% AEP), although the match of peak flow and flood volume at both gauges were not as strong as for the December 2010 and January 2011 events. This may be due to multiple small bursts with long dry periods in between the peaks, which are sensitive to continuing loss and TP selection. The discrepancy may also be improved by refining the channel storage definition in the model.

The TNSTC hydrologic model is considered to be suitably conservative for design flow simulations in intermediate to rare events (i.e. 10% AEP and above). As the Project alignment is generally outside the Gowrie Creek floodplain, the focal catchments (i.e. headwater tributaries draining towards Gowrie Creek and intercepted by the alignment) are likely insensitive to the calibration uncertainties in frequent events (i.e. 20% AEP and below), especially if an ARF of unity was to be adopted for design event simulation. The model calibration will be reviewed and refined during the detailed design stage of the Project.

The calibration accuracy is further influenced by the rating curve quality at both gauge locations, which is discussed further in Section 5.2.3.

5.2.3 Review of stream gauges used for model calibration

The TNSTC hydrologic model was calibrated to two major stream gauges operated by the DRDMW. Table 5.8 summarises the key gauge details, with their locations shown in Figure 5.2.

Table 5.8 Stream gauge details

Gauge No.	Name	Catchment area (km ²)	Period of record	Maximum recorded discharge (m ³ /s)	Maximum recorded height (m)	Maximum gauged discharge (m ³ /s)	Maximum gauged height (m)
422326A	Gowrie Creek at Cranley	47	1/10/1969 – Present	608.3	4.600	10.252	1.328
422323A	Gowrie Creek at Oakey	142	1/10/1967 – 30/01/1981	166.9	4.600	17.880	1.920
422332B	Gowrie Creek at Oakey	142	30/06/1992 – Present	482.1	4.894	122.299	3.500

The results of the hydrologic model calibration to the historical events (Section 5.2.2) indicates general confidence in adopting the catchment routing setup and the routing parameters for design event modelling; however, it is thought that the treatment of the flood of record (i.e. January 2011 event) may impact design-flow estimates at higher AEPs due its significantly higher magnitude compared to the other calibrated events at the gauge. Additionally, the assignment of an AEP to the January 2011 event is further sensitive to the adopted length of record at both gauges, particularly at Oakey (28 years used by TNSTC). There is also uncertainty in the recorded peak discharges during the January 2011 event, which exceeded the maximum gauged height by approximately 3.3 m at Cranley and by 1.4 m at Oakey. These factors should be considered when interpreting the results of the TNSTC model calibration.

To improve the confidence in adopting the TNSTC model's hydrologic estimates for rare events (i.e. 1% AEP and above), FFA was performed using streamflow records at both gauges until June 2021. The streamflow record at Oakey was expanded to 43 years by incorporating additional record from the closed gauge (422332A), which increased the record length beyond the 28 years used in the TNSTC model.

A review of the rating curves at both gauges was also conducted to identify any potential uncertainties in the FFA results for high-flow events, due to the generally low accuracy of the gauge measurements at both gauges. The methodology and outcomes of the FFA re-analysis at each gauge are documented in the following subsections for further review.

5.2.3.1 Cranley stream gauge

The Cranley gauge is located at a small weir approximately 10 km upstream of the Project alignment, as shown in Figure 5.2. The catchment area to the gauge is approximately 47 km² and consists of a highly urbanised catchment (Toowoomba). It is noted that there have been significant land-use changes during the gauge record, including increased urbanisation, construction of detention basins and major road crossings (e.g. Toowoomba Second Range Crossing).

The control section at the Cranley gauge is shown in Figure 5.11. The low flow is controlled by a small V-notch up to approximately 1.2 m, at a flow of 7 m³/s. The Gowrie Creek channel at Cranley has a capacity of approximately 1,000 m³/s, and the 2011 flood of record (4.6 m) was entirely constrained between the high banks of the channel. Due to steepness of the channel (approximately 10 per cent), the ratings at Cranley would be primarily influenced by bed-roughness assumptions, and marginally sensitive to hydrograph shape and channel storage effects.

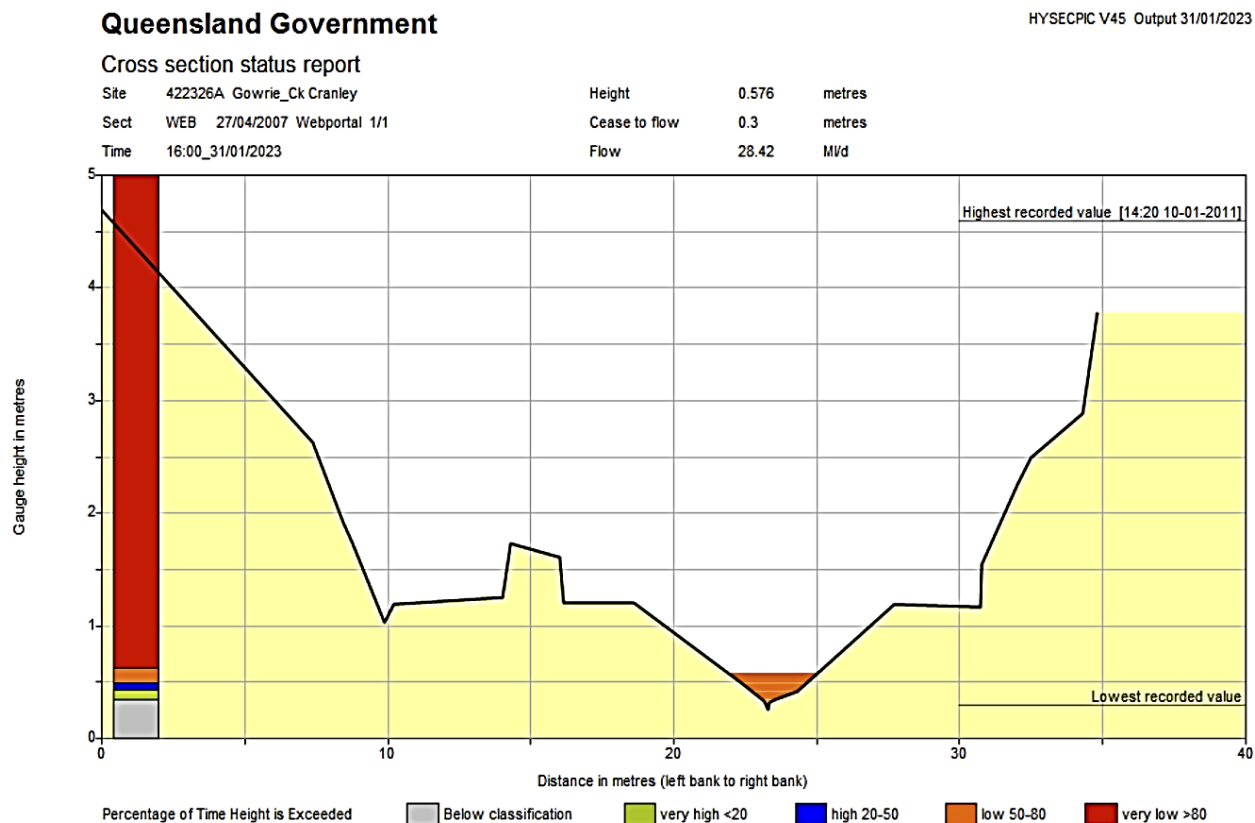


Figure 5.11 Cranley gauge cross-section (source: DRDMW) and reliable range based on visual inspection (blue)

The rating curve at Cranley was reassessed using a fine-scale (5 m) TUFLOW hydraulic model developed from the 1 m Toowoomba 2015 LiDAR dataset. Figure 5.12 plots the DRDMW rating alongside the TUFLOW rating curves generated using one low ($n = 0.03$) and one high ($n = 0.045$) channel roughness based on the commonly applied roughness range (Chow, 1959). The comparison indicates strongly consistent rating characteristics irrespective of the rating method used, with any difference between the ratings explained by small changes in the assumed channel roughness. The DRDMW rating is therefore considered reliable and appropriate for use in the FFA.

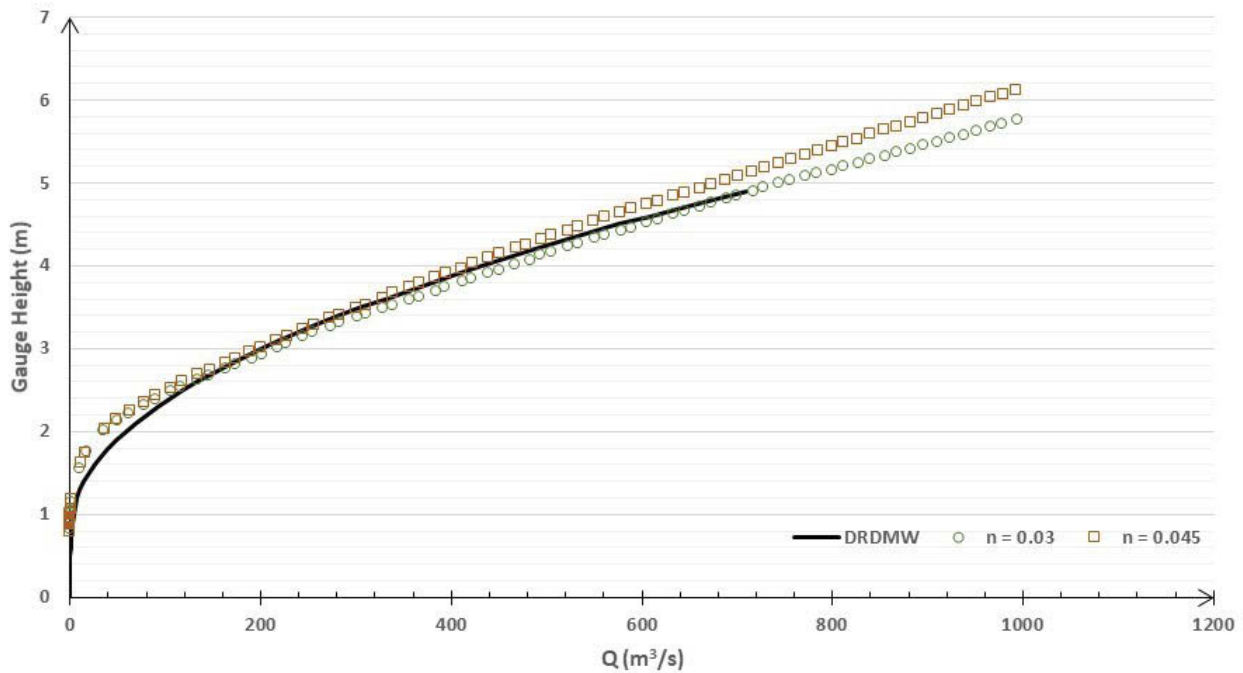


Figure 5.12 Cranley—Comparison between DRDMW and TUFLOW rating curves

The annual maxima series for the period of record based on the DRDMW rating curve is shown in Figure 5.13. The recorded peak flow for the January 2011 event (608 m³/s) is considered accurate based on the rating curve review, refer to the above sections.

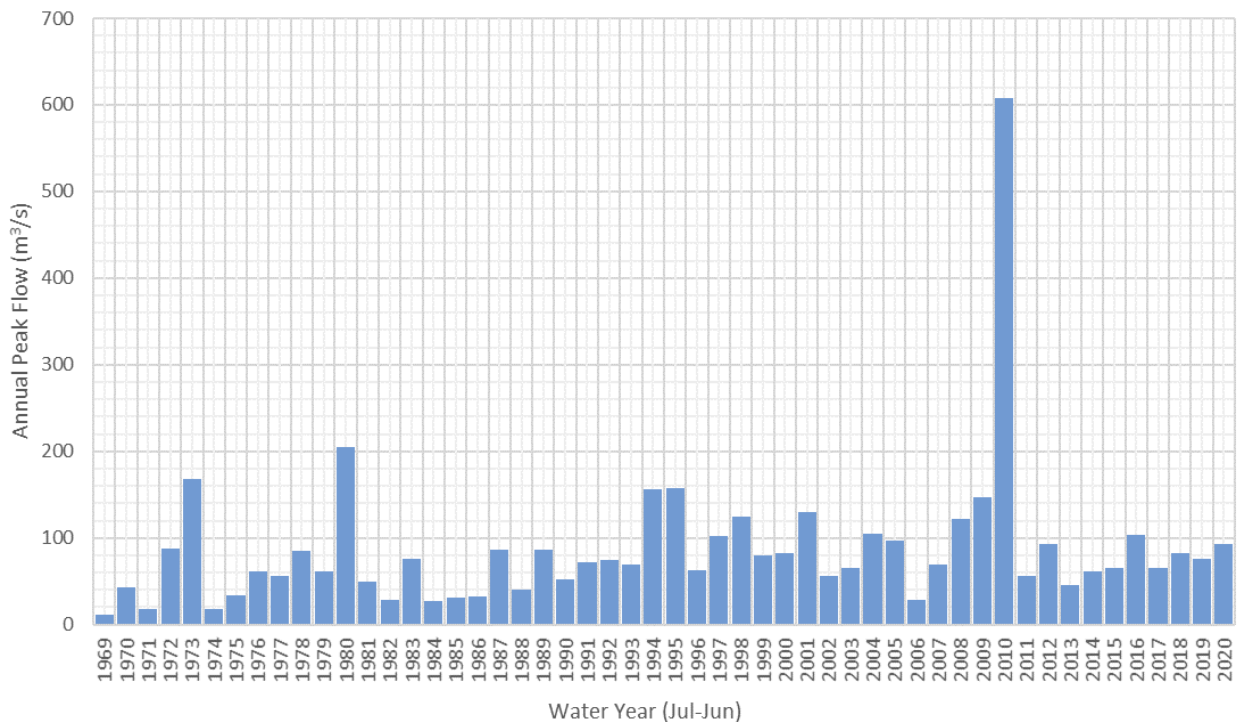


Figure 5.13 Cranley—Adopted annual maxima series

As discussed above, a revised FFA was completed to improve the confidence in adopting the TNSTC model's hydrologic estimates for rare events (i.e. 1% AEP and above), using streamflow records at the Cranley gauge until June 2021. The results of the revised FFA are shown in Figure 5.14 and Table 5.9. The annual peak streamflow data was best-fitted by a Log-Pearson Type III (LPIII) distribution, and no low-flow outliers were found using the multiple Grubbs-Beck test. A good alignment between the revised FFAs and the adopted design event flows was achieved; however, it is still noted that there were several uncertainties in the FFA as follows:

- The January 2011 event is above the confidence limits of the FFA at approximately 1% AEP, which also implies a positive skew despite a negative skew being more typical of catchments west of the dividing range. A more rigorous analysis including more years of data could show that the 2011 event was a high outlier and was in fact a rarer event than the 1% AEP.
- There have been changes to urbanisation and the construction of several detention basins throughout the gauge record.

The revised FFA confirms that the adopted design flow estimates are suitably conservative for rail design and FIA purposes. As the Cranley gauge is associated with a heavily incised upstream reach, the reconciliation between the current design flows and the revised FFA could also be sensitive to the channel storage representation in the hydrologic model. Options to improve channel storage characterisation to refine the current design flow estimates will be further reviewed at the detailed design stage of the Project.

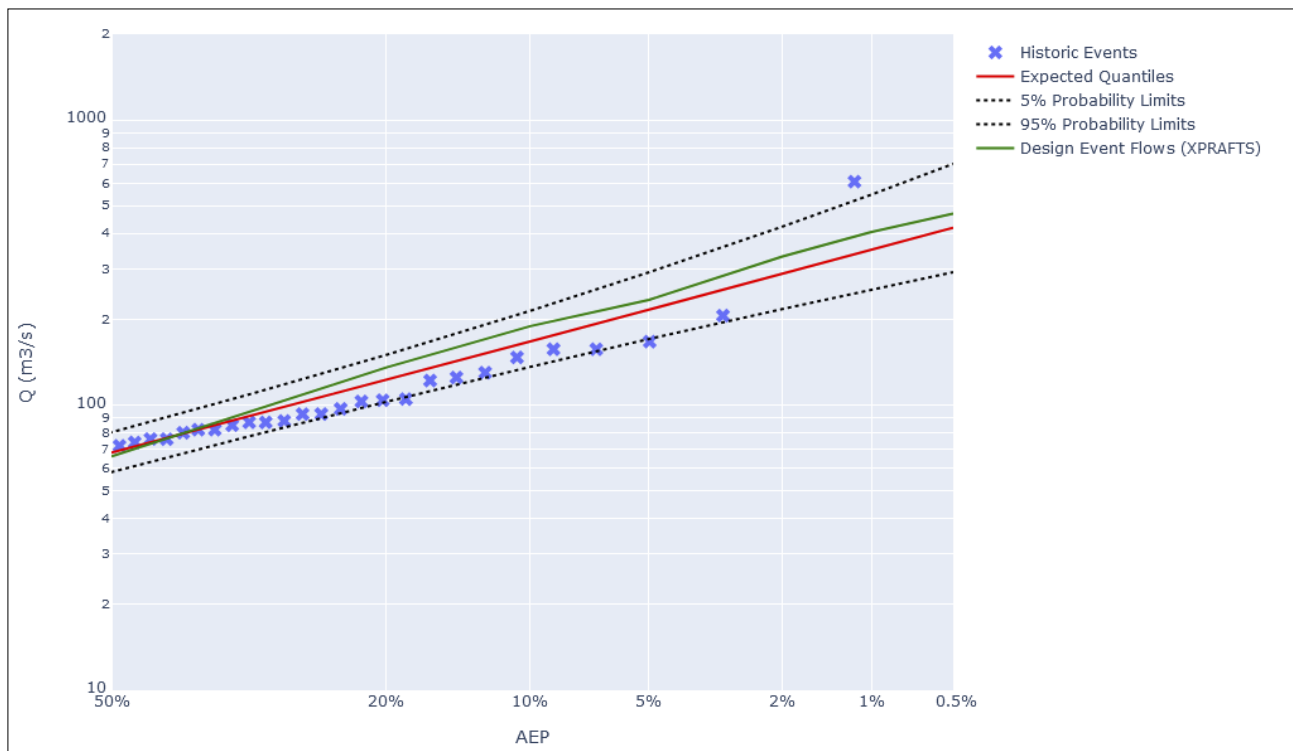


Figure 5.14 Revised flood frequency analysis at Cranley gauge based on LP3 model

Table 5.9 Revised flood frequency analysis at Cranley gauge based on LP3 model

AEP	Expected probability quantile (m³/s)	5% probability limit (m³/s)	95% probability limit (m³/s)	Design event flows (m³/s)
50%	68	58	80	69
20%	123	103	150	139
10%	167	136	214	193
5%	216	171	292	257
2%	289	217	423	356
1%	351	255	548	431

5.2.3.2 Oakey gauge

The Oakey gauge is located at a small concrete causeway immediately upstream of the Warrego Highway, approximately 10 km downstream of the Project alignment, and approximately 25 km downstream of the Cranley gauge, as shown above in Figure 5.2. The catchment area to the gauge is approximately 142 km². The gauge operated from 1967 to 1980, and then was closed until 1992, since which it has been operating until present.

The control section at the gauge is shown in Figure 5.15. In-bank flows at the control section are regulated by an approximately 3 m deep rectangular channel, having an approximate capacity of 350 m³/s at the top of the channel (around 4.5 m high). Floodplain inundation could occur before bankfull height (4.5 m) is reached at the control section due to breakouts from the sinuous Gowrie Creek upstream channel, which has a capacity of approximately 80 m³/s.

The DRDMW ratings at Oakey are therefore considered less reliable above 80 m³/s due to the effects of bypassing floodplain flow and presence of the Warrego Highway.

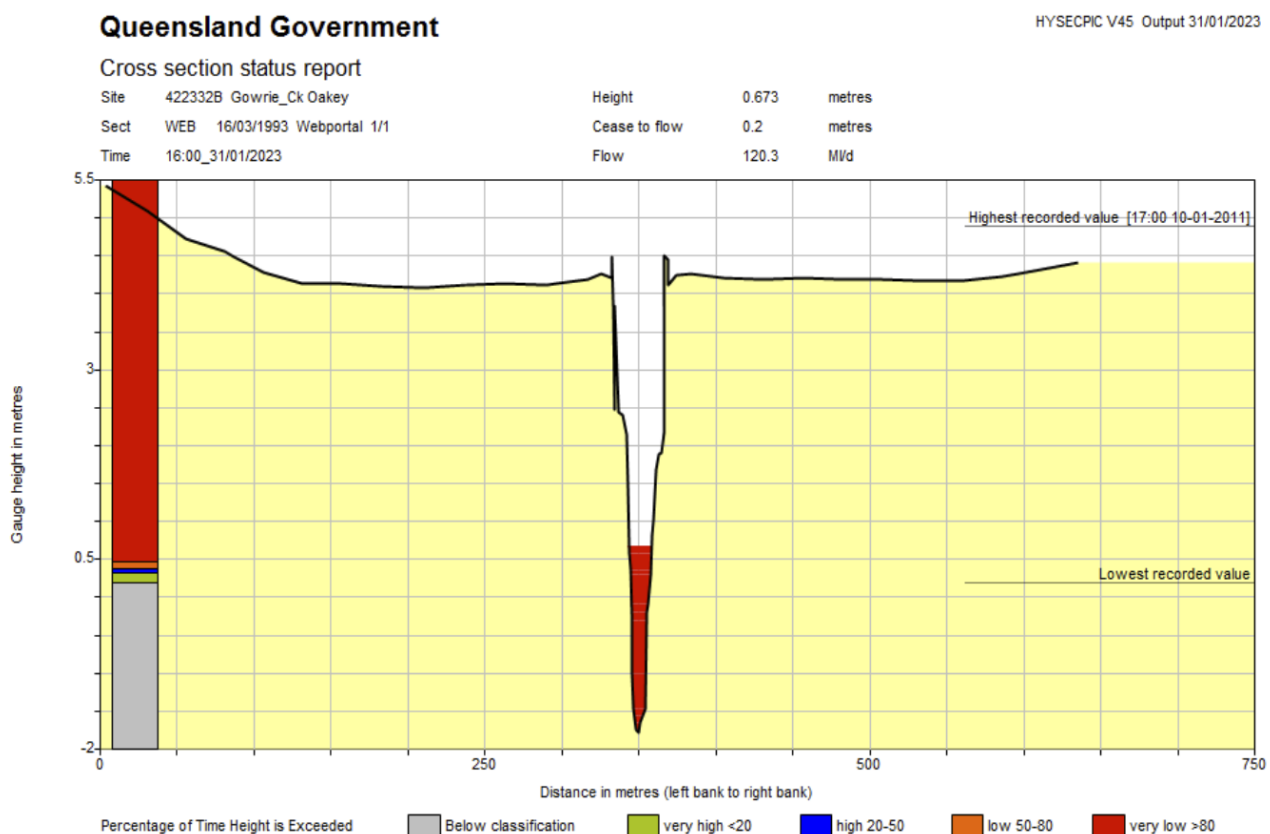


Figure 5.15 Oakey gauge cross-section

Source: DRDMW

The rating curve at Oakey was reassessed using a fine-scale (5 m) TUFLOW hydraulic model developed from the 1 m Toowoomba 2015 LiDAR dataset. A comparison between the DRDMW and revised rating curve is shown in Figure 5.16. The DRDMW rating is shown to be accurate for discharges below 80 m³/s (approximately 3.2 m high), and overestimates flows above 80 m³/s as the effects of Warrego Highway and floodplain breakouts were unaccounted for. Adopting the DRDMW ratings in the FFA would therefore result in more conservative flow estimates, which are considered suitable for feasibility rail design and flood impact assessment purposes.

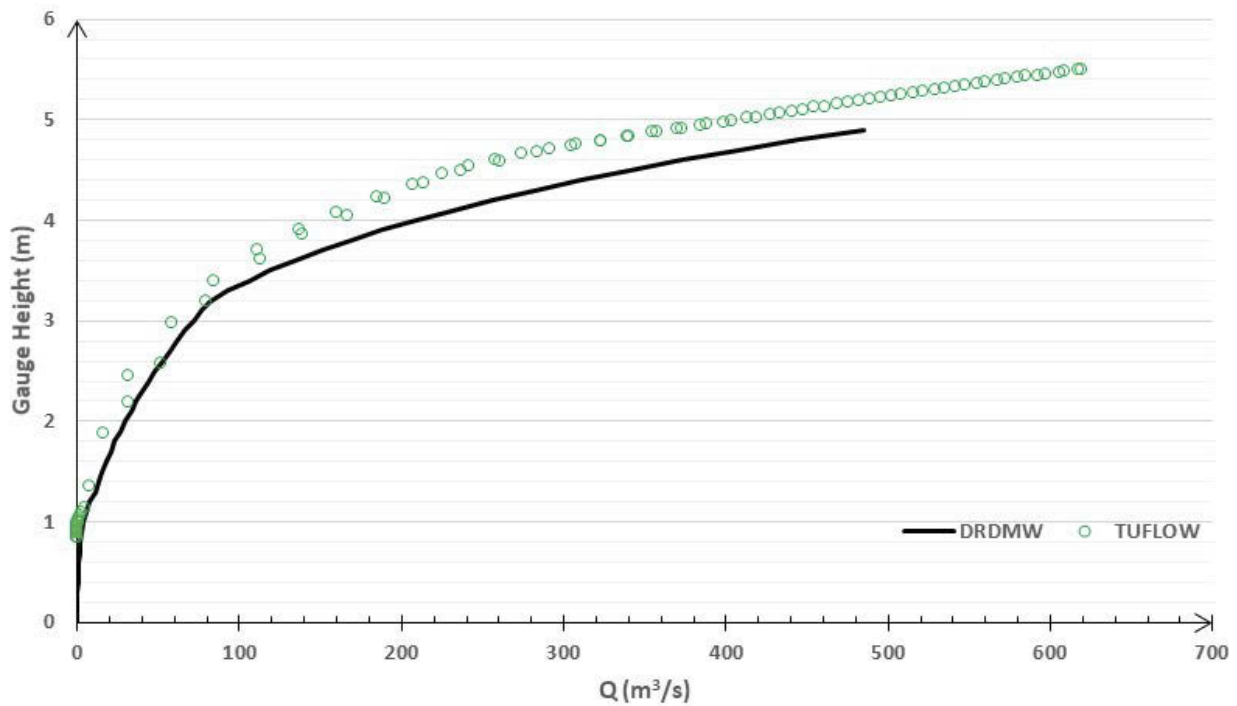


Figure 5.16 Oakey—Comparison between DRDMW and TUFLOW rating curves

The annual maxima series for the period of record based on the DRDMW rating curve is shown in Figure 5.17. The recorded peak flow for the January 2011 event (482 m³/s) has been replaced with the modelled event peak (831 m³/s) due to suspected gauge malfunction during the peak.

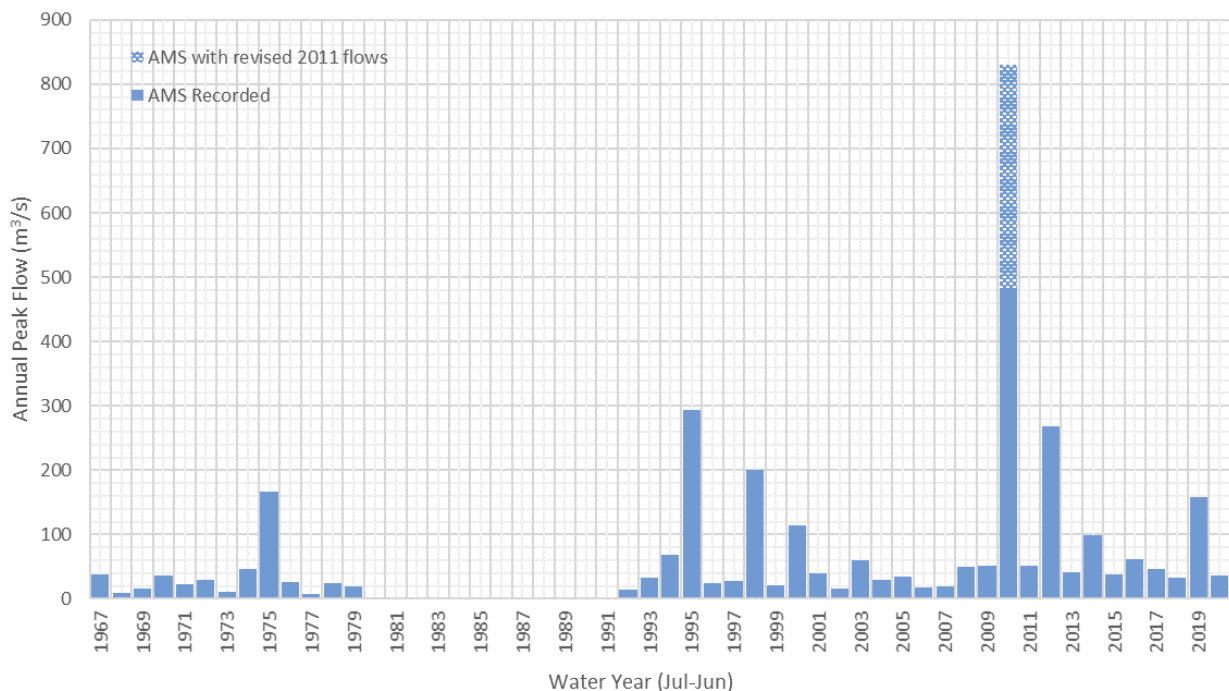


Figure 5.17 Oakey—Adopted annual maxima series

The results of the FFA are shown in Figure 5.18 and Table 5.10. The annual peak streamflow data was best-fitted by a LPIII distribution, and no low-flow outliers were found using the multiple Grubbs-Beck test.

There was a good match between the 1% AEP design event flow and the updated FFA. The 2% and 5% AEP design event flow were within the confidence limits of the updated FFA; however, the smaller events (10% AEP and more frequent) were above the confidence limits of the updated FFA. This was the same outcome as for the TNSTC models and the draft EIS models.

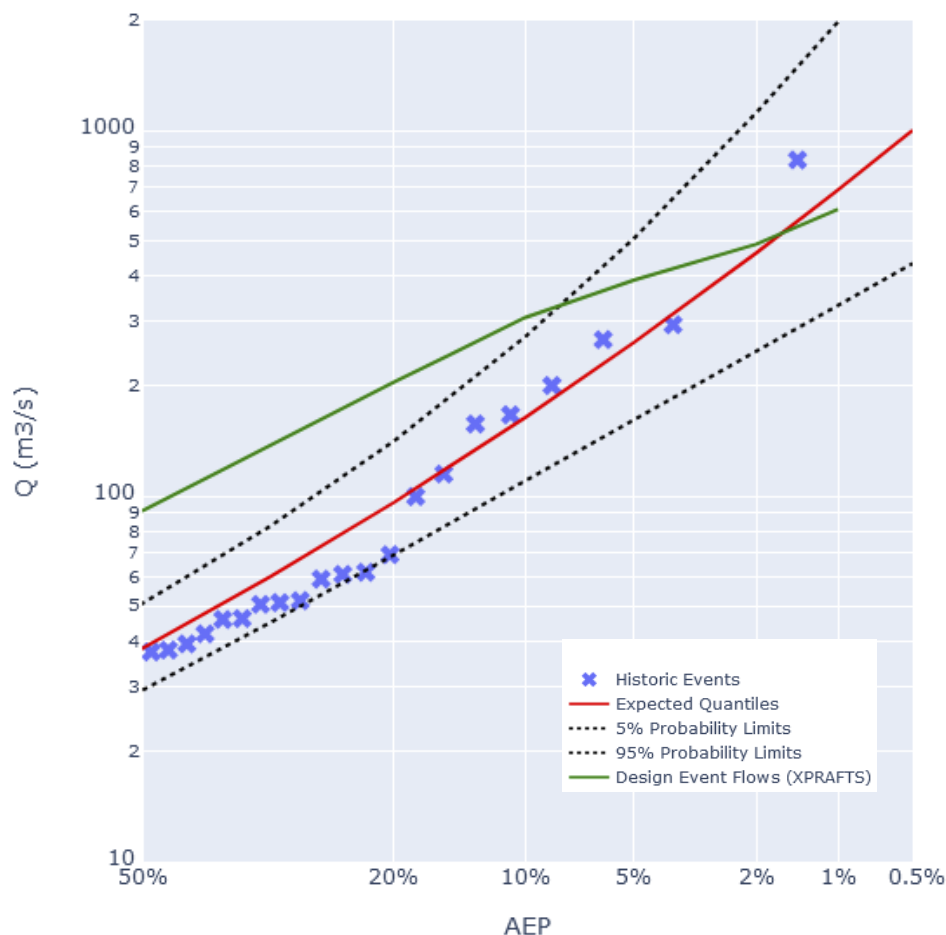


Figure 5.18 Revised flood frequency analysis at Oakey gauge based on LP3 model

Table 5.10 Flood frequency analysis results for Oakey gauge

AEP	Expected probability quantile (m³/s)	5% probability limit (m³/s)	95% probability limit (m³/s)	Design event flows (m³/s)
50%	37	29	48	91
20%	91	66	132	205
10%	157	105	261	308
5%	256	156	498	390
2%	462	245	1,132	487
1%	703	336	2,046	609

The revised FFA further indicates lower peak flows at Oakey than at Cranley, despite the catchment to the former (142 km²) being triple the size of that to the latter (47 km²) and both being located on the same watercourse, at only 11 km apart. Figure 5.19 plots the correlation between the annual peak streamflow data at both gauges, revealing that their peak flow relationship is sensitive to whether out-of-bank flow occurs at Oakey (i.e. lower than 80 m³/s).

For peak flows larger than 100 m³/s at Oakey, the peak flow ratio of Oakey to Cranley is on average of 2:1. It is therefore thought that the design flow estimates at Oakey could be sensitive to potential under-representation of channel storage in the hydrologic model, resulting in more conservative (i.e. higher) design flow estimates at Oakey than the FFA, under the 10% AEP and more frequent events. Options to improve channel storage characterisation to refine the current design flow estimates will be further reviewed at the detailed design stage of the Project.

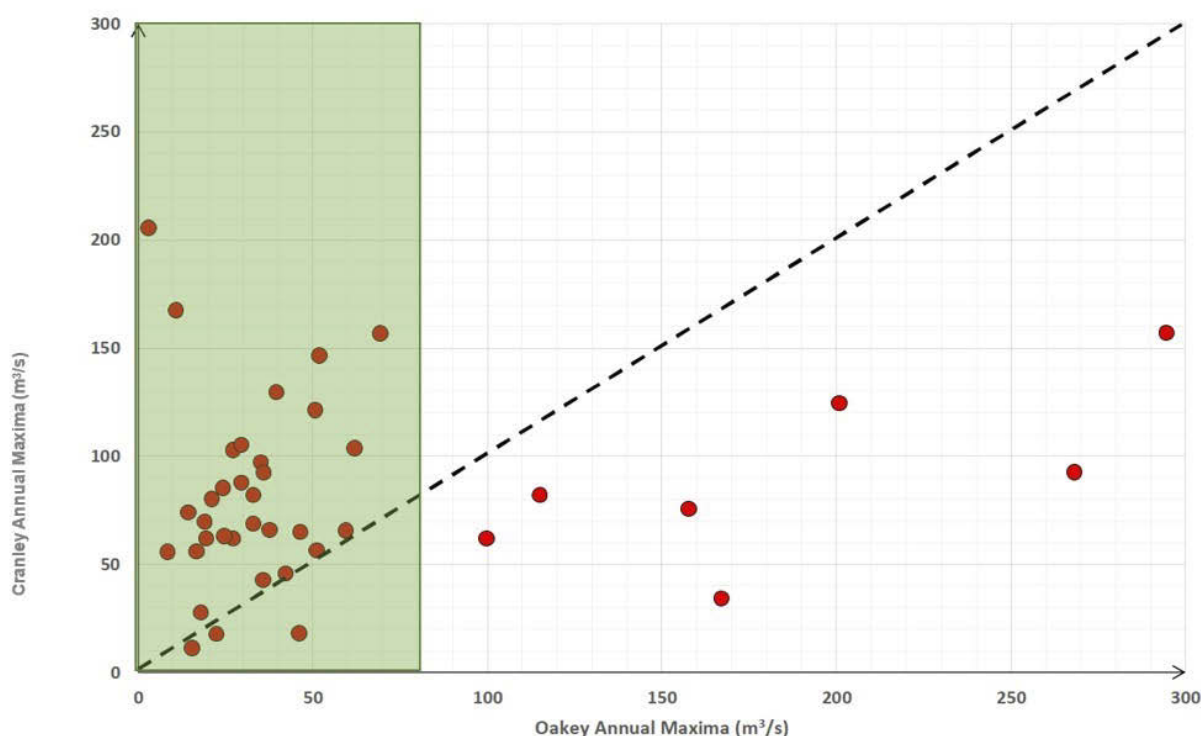


Figure 5.19 Correlation of annual peak streamflow data between Cranley and Oakey—Black dashed line: 1:1 relationship; Green shaded area: in-channel events at Oakey

5.2.4 Design event parameters

Hydrologic information to assist estimation of design event flows is summarised in Table 5.11.

Table 5.11 Gowrie Creek—Summary of URBS model design event inputs

Input parameter	Remarks
Design rainfall	A total of 11 IFDs were extracted from the BoM to capture the spatial variation of rainfall across the Gowrie Creek catchment.
Extreme event rainfall	Probable Maximum Precipitation depths for durations up to 6 hours (for use in modelling the PMF event) were obtained using the method presented in the Bulletin 53 (BoM, 2003). The rainfall depths for the 1 in 10,000 AEP event were estimated using the interpolation method presented in ARR 2019 Book 8 Section 3.5.
Losses	The TNSTC model losses were adopted for events up to and including the 1 in 2,000 AEP, reproduced as follows: ■ Pervious area—IL: 40 mm, CL: 4.5 mm/hr ■ Impervious area – IL:28 mm, CL: 3.15 mm/s. For the 1 in 10,000 AEP and PMP events, 0 mm initial loss and 1 mm/hr continuing loss were adopted.
Areal reduction factor	As the local catchments draining to the Project alignment have typical areas between 1 to 10 km ² , an ARF of unity was applied. The approach is considered conservative as a unity ARF was also applied to the Gowrie Creek inflow.
Ensemble TPs	As the local catchments draining to the Project alignment have typical areas between 1 to 10 km ² , the standard point rainfall patterns from ARR 2019 were applied.
Preburst depths	Median preburst depths were downloaded from the ARR Data Hub for each sub-catchment. Preburst depths vary by design storm event AEP and duration. Preburst depths were applied to the model by reducing the initial losses for each storm event.

5.2.5 Design hydrograph selection

Combinations of eight storm durations (between 1 hour and 12 hours) and 10 TPs were assessed in the calibrated TNSTC hydrologic model to determine the critical storm duration and TP combinations for use in the hydraulic design and impact assessment.

Temporal patterns for the 20% AEP to the 1 in 2,000 AEP events were adopted based on the 10-burst ensemble approach presented in ARR 2019. The TPs and pre-burst data was directly downloaded from the ARR 2019 Data Hub at the centroid of the Gowrie Creek catchment. Temporal patterns for the 1 in 10,000 AEP and PMF design events for durations up to 6 hours were obtained from the *Estimation of Probable Maximum Precipitation Australia: Generalised Short Duration Method* (BoM, 2003).

A single TP for each storm duration was selected for hydraulic simulations. The critical TP was selected as that which produced a peak flow closest to the mean with a bias of +2 (i.e. two above the TP closest to the mean) at the Cranley gauge on Gowrie Creek, located approximately 200 m downstream of Warrego Highway and approximately 6 km north of Toowoomba Central Business District. As the Project alignment also intersects a number of small catchments that could govern local flooding behaviours, the selection of TPs will be further reviewed and refined in detailed design. Table 5.12 provides selected TPs for respective AEPs and durations.

Table 5.12 Gowrie Creek—Summary of selected design event hydrographs

AEP	Duration (mins)	Temporal pattern
20%	60	Pattern 2
	90	Pattern 6
	120	Pattern 6
	180	Pattern 4
	270	Pattern 10
	360	Pattern 3
	540	Pattern 2
	720	Pattern 9
10%	60	Pattern 7
	90	Pattern 7
	120	Pattern 1
	180	Pattern 6
	270	Pattern 6
	360	Pattern 5
	540	Pattern 4
	720	Pattern 7
5%	60	Pattern 7
	90	Pattern 7
	120	Pattern 1
	180	Pattern 8
	270	Pattern 6
	360	Pattern 9
	540	Pattern 4
	720	Pattern 7

AEP	Duration (mins)	Temporal pattern
1%	60	Pattern 7
	90	Pattern 8
	120	Pattern 8
	180	Pattern 6
	270	Pattern 5
	360	Pattern 2
	540	Pattern 3
	720	Pattern 3
1 in 2,000	60	Pattern 2
	90	Pattern 2
	120	Pattern 8
	180	Pattern 6
	270	Pattern 1
	360	Pattern 9
	540	Pattern 6
	720	-

5.2.5.1 Verification of design flows

The design flow for the 1% AEP and extreme events were estimated using the calibrated hydrologic model. Comparison was undertaken between peak flows modelled by the hydrologic model, the FFA results and previous studies to ensure the adopted loss values were within accepted ranges.

Table 5.13 compares the 1% AEP peak flow modelled by the hydrologic model with FFA results at the Oakey and Cranley gauges.

Table 5.13 Cranley and Oakey—Comparison between peak flows from flood frequency analysis with hydrologic model flows

Stream gauge	FFA flow for 1% AEP (m ³ /s)	Hydrologic model 1% AEP flow (m ³ /s)	Difference %
Oakey	610 ¹ to 703 ²	609 ³	-0% to -13%
Cranley	350	431 ³	+23%

Table notes:

- 1 LP3 probability model – using original data
- 2 LP3 probability model – using revised data
- 3 Median value of all 10 TPs (bursts) for the critical duration

A comparison between peak flows modelled in this current investigation and the TRC (2014) study at two locations, Cotswold Hill East catchment and Cotswold West Catchment is shown in Table 5.14.

The discrepancies in results between the two studies may be because of the following factors:

- The previous study used *Australian Rainfall and Runoff: A Guide to Flood Estimation* (Institution of Engineers, 1987) (ARR 1987), while this assessment uses ARR 2019 data and methodologies
- The critical duration adopted in the previous study was 60 minutes while in this assessment (revised draft EIS) it is 90 minutes
- Assumed velocities for channels were higher in the previous model in comparison to this assessment

- The previous study adopted different losses. For the 1% AEP, 10 mm to 15 mm initial losses were applied to the Cotswold Hill model. Varying continuing losses were also applied based on the percentage imperviousness of each sub-catchment:
 - For a closer comparison between the Cotswold Hill Model and the revised draft EIS model, losses were adjusted in the revised EIS to 15 mm initial loss and 2.5 mm/hr continuing loss. The results of this analysis have been presented in Table 5.14.
- The previous model focused on the Cotswold catchment and was not calibrated to any stream gauges, while this assessment focuses on the Gowrie Creek catchment and was calibrated against two historical events.

Table 5.14 Comparison of modelled 1% AEP peak flow in the hydrologic model with TRC (2014) study

Cotswold Hill East		Cotswold Hill West	
TRC (2014b) (m ³ /s)	B2G hydrologic model (m ³ /s)	TRC (2014b) (m ³ /s)	B2G hydrologic model (m ³ /s)
141	76.9 (108 ¹)	28.3	20.9 (31 ¹)

Table note:

1 Revised draft EIS loss conditions altered to 15 mm initial loss, and 2.5 mm/hr continuing loss.

5.3 Development of hydraulic model—Gowrie Creek

A 2D modelling approach was adopted to simulate the flood regime around the proposed rail crossing at Gowrie Creek. The platform used for hydraulic modelling was TUFLOW HPC software package (version 2020-10-AD) with Sub-grid sampling (SGS) and Quadtree (QT) approach. The processes and assumptions adopted throughout the development of the hydraulic model are described in the following sections.

5.3.1 Model setup

The TNSTC hydraulic model was reviewed, modified, and then adopted for the Gowrie Creek assessment. The hydraulic model included the Project's revised design; which were based on the previous revision of the design, therefore required updates. Two model scenarios (Existing Case and Design Case) were created based on the TNSTC model. The modifications were as follows:

- For the Existing Case, the Project rail embankment, culverts, and associated infrastructure were removed to reflect current conditions
- For the Design Case, the Project rail embankment, culverts, and associated infrastructure were updated to correspond with the revised reference design, as presented in the revised draft EIS.

The setup of the TUFLOW model is summarised in Table 5.15.

The hydraulic model setup, including extent and adopted land use, is presented in Figure A1-D and Figure A1-E in Volume 2 – Appendix A. The 2D model topography was modified to adequately represent the drainage flowpaths and the existing road/rail crowns.

Table 5.15 Gowrie Creek—Hydraulic model summary

Parameter	Information
Completion date	August 2022
AEPs assessed	20%, 10%, 5%, 2%, 1%, 1 in 2,000, 1 in 10,000 AEP and PMF
Hydraulic model build	TUFLOW HPC GPU – version 2020-10-AD-iSP-w64
Model extent	Refer to Figure A1-D in Volume 2 – Appendix A
Grid size	4 m
SGS distance	1 m
QT grid size	2 m, 2 nd Order

Parameter	Information
DEM (year flown)	TRC (2015)
Roughness	Spatially varying roughness values compliant with industry norms.
Eddy viscosity	Wu (default)
Model calibration	N/A
Downstream (D/S) model boundary	Height-discharge (HQ) boundary with normal slope approximated based upon topography dataset
Hydraulic model timestep	Adaptive timestep
Modelled scenarios	Existing Case, Developed Case
Sensitivity analysis	Climate change and Blockage (0% and 50% Blockage on culverts)

5.3.2 Model topography and grid size

The topography within the hydraulic model is primarily based on a 1 m DEM data collected during 2015, which was supplied by TRC. A 4 m x 4 m grid size was selected to allow the features of the channels and floodplain to be represented with sufficient accuracy, while maintaining efficient model run times. The 4 m x 4 m grid size is considered adequate to represent floodplain topography and is consistent with the approach typically adopted in similar flood studies. The 2D model topography was modified adjacent to the culverts so that the existing drainage flows, existing road/rail crowns and levee banks were represented accurately in the model.

Additional details of the base terrain models and included survey is documented in Section 5.1.2.

5.3.3 Sub-grid sampling and Quadtree mesh refinement

The Gowrie Creek hydraulic model was established using TUFLOW's 2020-10-AD model version, including SGS and QT mesh add-on modules. The below points provide a brief description of each module, along with the key parameters adopted for this assessment.

- Traditional grid-mesh based 2D hydraulic models sample the underlying DEM by taking a single elevation at the cell centres and cell faces. Sub-grid sampling allows for increased resolution by sampling a greater number of points of the underlying DEM to establish a water surface elevation versus volume relationship for each grid, along with a water surface elevation versus width relationship (wetted perimeter) at cell sides. The adoption of SGS allows increased representation of storage and conveyance between cells. A SGS distance of 1 m was adopted for this assessment.
- Quadtree mesh is constructed by subdividing the base mesh-grid size into four smaller square cells. Cells can be further sub-divided into four, and so on, allowing for additional resolution in location of interest. The base hydraulic model grid cell size (5 m x 5 m) was revised to include additional spatial resolution of approximately 2.5 m, to better represent model velocity at the entry and exit of culverts. Quadtree meshes were applied in the model covering minimum 25 m upstream and 100 m downstream of the design culvert locations, and culverts within the permanent footprint.

5.3.4 Roughness

The hydraulic roughness is reflective of the nature of development and ground cover that exist within the hydraulic model extent. The distribution of roughness categories adopted for this assessment was based on the TNSTC hydraulic model. Specific roughness values applied to the hydraulic model are detailed in Table 5.16. Figure A1-E in Volume 2 – Appendix A shows the spatial breakdown of land use in the 2D model domain.

Table 5.16 Gowrie Creek—Manning's n values

Land use	Manning's n
Roads	0.015
Concrete channel	0.020
Grassed waterways	0.030
Maintained grass	0.030
Floodplain/cropland/grazing areas	0.050
Low density vegetation	0.070
Vegetated waterways	0.090
High-density vegetation/forest	0.120
Low-density residential parcel size <1 ha	0.170
Industrial/Commercial	0.260
Urban—Parcel size < 1,000 m ²	0.300
Buildings	0.500

5.3.5 Hydraulic structures

This study made use of the structure geometry data from the earlier hydraulic models. Dimensions were derived from aerial photography in cases where cross-drainage construction data was not provided. All culverts were modelled using standard inlet and outlet losses, as shown in Table 5.17.

Table 5.17 Gowrie Creek—Adopted culvert hydraulic losses

Structure ID	RCP	RCBC
Height contraction coefficient	n/a	0.6
Width contraction coefficient	1.0	0.9
Entry loss	0.5	0.5
Exit loss	1.0	1.0

Details of existing culverts and bridges identified within the hydraulic model domain are summarised in Table 5.18 and Table 5.19.

Table 5.18 Gowrie Creek—Identified existing structures within the hydraulic model extent

Structure ID	Structure type	Upstream invert (mAHD)	Downstream invert (mAHD)	Width (m)	Height (m)	Number of cells/barrels
GOW30_E001	RCP	465.2	465.0	1.65	-	1
GOW30_E002	RCBC	467.3	467.2	2.8	3	3
GOW30_E003	RCBC	481.3	481.1	1.8	1.8	6
GOW30_E009	RCP	469.0	466.7	1.2	-	1
GOW30_E010	RCP	458.6	458.5	0.97	-	6
GOW30_E017	RCBC	459.1	458.7	2.1	1.8	10
GOW30_E018	RCBC	462.1	461.9	2.1	1.5	11
GOW30_E019	RCP	469.1	468.5	1.8	-	5
GOW30_E020	RCBC	478.3	478.0	2.1	2.1	2
GOW30_E021	RCBC	486.1	485.9	2.4	0.6	2
GOW30_E022	RCBC	491.8	491.3	2.4	0.6	2

Structure ID	Structure type	Upstream invert (mAHD)	Downstream invert (mAHD)	Width (m)	Height (m)	Number of cells/barrels
GOW30_E023	RCBC	481.6	481.4	1.2	0.6	3
GOW30_E024	RCBC	478.2	478.1	2.4	0.6	1
GOW30_E025	RCBC	475.0	474.6	1.2	0.3	5
GOW30_E027	RCBC	472.5	472.2	3	2.1	2
GOW30_E028	RCBC	477.5	477.0	1.2	0.6	2
GOW30_E030	RCP	512.3	511.2	1.5	-	1
GOW30_E031	RCP	510.7	509.4	1.5	-	1
GOW30_E032	RCP	511.0	509.7	2.1	-	4
GOW30_E033	RCP	512.1	511.3	2.1	-	4
GOW30_E034	RCP	516.4	516.0	1.8	-	2
GOW30_E035	RCP	515.6	515.1	1.8	-	2
GOW30_E036	RCP	518.8	518.7	0.6	-	2
GOW30_E037	RCP	518.4	518.0	0.6	-	1
GOW30_E038	RCP	515.1	514.5	0.75	-	1
GOW30_E039	RCP	504.7	504.4	2.4	-	4
GOW30_E040	RCP	504.9	504.5	2.4	-	5
GOW30_E041	RCP	507.0	506.8	0.9	-	1
GOW30_E042	RCP	507.6	507.3	0.9	-	1
GOW30_E043	RCP	501.7	500.7	1.2	-	3
GOW30_E044	RCP	499.6	499.1	0.75	-	3
GOW30_E045	RCP	496.4	495.8	1.95	-	3
GOW30_E046	RCP	506.8	505.6	1.5	-	2
GOW30_E047	RCP	505.4	504.8	1.5	-	2
GOW30_E048	RCP	510.7	510.3	0.75	-	2
GOW30_E049	RCP	521.5	521.3	0.375	-	2
GOW30_E050	RCP	513.3	512.9	0.6	-	1
GOW30_E051	RCP	513.2	512.8	0.6	-	1
GOW30_E052	RCBC	519.1	518.8	1.2	0.3	1
GOW30_E053	RCP	513.2	512.8	0.6	-	2
GOW30_E055	RCP	472.8	472.5	0.6	-	1
GOW11_E006	RCBC	487.2	486.8	1.2	0.32	4
GOW11_E007	RCP	502.2	501.4	0.6	-	2
GOW11_E008	RCBC	501.9	501.4	1.2	0.9	4
GOW11_E014	RCBC	447.9	447.4	2.1	1.12	1
GOW11_E015	RCBC	448.3	448.2	2.1	1.2	2
GOW11_E016	RCP	481.5	481.3	0.6	-	1
GOW11_E017	RCBC	491.6	491.5	0.45	0.45	1
GOW11_E018	RCP	497.1	496.7	0.45	-	3
GOW11_E020	RCBC	497.7	497.4	0.45	0.45	1
GOW11_E023	RCBC	483.9	483.6	1.2	0.6	1
GOW11_E024	RCBC	484.1	484.0	1.8	0.6	1

Structure ID	Structure type	Upstream invert (mAHD)	Downstream invert (mAHD)	Width (m)	Height (m)	Number of cells/barrels
GOW11_E026	RCP	509.7	509.6	0.375	-	1
GOW11_E027	RCP	504.8	504.5	0.375	-	1
GOW11_E028	RCBC	504.8	504.6	1.5	1.5	1
GOW11_E029	RCP	532.9	532.6	0.9	-	1
GOW11_E030	RCP	528.7	528.2	0.6	-	1
GOW11_E031	RCBC	505.0	504.9	0.9	0.6	2
GOW11_E032	RCP	503.3	503.1	0.6	-	1
GOW11_E033	RCBC	518.6	518.2	1.2	0.45	1
GOW11_E034	RCP	498.0	497.7	0.45	-	3
GOW11_E035	RCBC	485.3	485.2	1.5	0.6	1
GOW11_E036	RCBC	495.3	495.2	1.2	0.45	1
GOW11_E040	RCP	522.0	521.5	0.6	-	3
GOW11_E059	RCP	503.8	502.5	0.45	-	2
GOW11_E060	RCP	505.2	504.5	0.75	-	3
GOW11_E061	RCP	504.5	503.9	0.75	-	3
GOW11_E062	RCP	505.0	504.6	1.05	-	2
GOW11_E063	RCP	504.4	502.7	1.05	-	2
GOW11_E064	RCP	504.9	504.5	0.9	-	2
GOW11_E065	RCP	504.3	502.6	0.9	-	2
GOW11_E066	RCP	506.8	506.7	0.675	-	2
GOW11_E067	RCP	506.6	504.6	0.675	-	2
GOW11_E068	RCBC	508.5	508.1	1.2	0.45	1
GOW11_E069	RCBC	508.0	507.8	1.2	0.45	1
GOW11_E070	RCP	507.8	505.0	1.2	-	1
GOW11_E071	RCP	508.3	505.6	1.2	-	1
GOW11_E072	RCP	513.5	512.2	0.6	-	2
GOW11_E073	RCP	509.7	502.6	0.75	-	1
GOW11_E074	RCP	506.8	505.6	0.6	-	2
GOW11_E075	RCP	500.0	498.2	0.6	-	2
GOW11_E076	RCBC	495.2	493.9	1.8	0.6	3
GOW11_E077	RCBC	492.4	488.8	1.2	0.9	3
GOW11_E078	RCBC	499.3	499.3	1.2	0.6	1
GOW11_E079	RCBC	480.3	480.1	0.6	0.6	2
GOW11_E080	RCBC	477.2	476.9	1.2	0.6	2
GOW11_E081	RCBC	472.6	471.8	1.2	1.2	4
GOW11_E082	RCBC	471.8	470.8	1.2	1.2	4
GOW11_E083	RCBC	473.4	473.3	1.2	0.6	4
GOW11_E084	RCBC	449.2	448.7	0.9	0.3	1
GOW11_E085	RCBC	449.2	448.9	0.9	0.3	1
GOW11_E086	RCBC	449.0	448.6	0.9	0.3	1

Structure ID	Structure type	Upstream invert (mAHD)	Downstream invert (mAHD)	Width (m)	Height (m)	Number of cells/barrels
GOW11_E087	RCBC	448.9	448.5	0.9	0.3	1
GOW11_E088	RCBC	448.7	448.3	0.6	0.3	1
GOW11_E089	RCBC	448.5	448.1	0.6	0.3	1
GOW11_E090	RCBC	448.4	448.1	0.9	0.3	1
GOW11_E091	RCBC	448.3	448.1	0.6	0.3	1
GOW11_E092	RCBC	446.4	446.1	1.2	0.9	3
GOW11_E093	RCBC	447.0	446.6	0.9	0.3	1
GOW11_E094	RCP	495.3	495.3	0.6	-	2
GOW11_E095	RCBC	481.4	481.4	1.2	0.6	23
GOW11_E096	RCP	481.0	480.8	0.6	-	1
GOW11_E097	RCP	477.9	477.5	0.6	-	1
GOW11_E098	RCP	480.5	480.3	0.6	-	1
GOW11_E099	RCP	489.4	488.5	0.6	-	2
GOW11_E100	RCP	490.8	490.4	0.6	-	1
GOW11_E101	RCP	492.3	491.9	0.6	-	1
GOW11_E102	RCP	474.0	473.7	0.6	-	1
GOW11_E109	RCP	509.6	509.6	0.45	-	1
GOW11_E110	RCP	482.9	482.0	0.45	-	1
GOW11_E111	RCP	482.0	481.7	0.45	-	1
GOW11_E112	RCP	486.3	483.6	0.375	-	1
GOW11_E113	RCP	486.5	485.0	0.375	-	1
GOW11_E114	RCP	491.0	489.1	0.45	-	1
GOW11_E115	RCP	478.8	477.4	0.6	-	1
GOW11_E117	RCP	483.5	483.3	0.6	-	1
GOW11_E119	RCBC	447.9	447.4	2.1	1.12	1
GOW11_E120	RCBC	447.9	447.4	2.1	1.12	1
GOW11_E122	RCP	482.9	482.8	0.9	-	1
GOW11_E123	RCP	483.0	482.7	1.2	-	1
GOW11_E124	RCP	482.3	482.1	0.9	-	1
GOW11_E125	RCP	482.3	482.0	0.9	-	1
GOW11_E126	RCP	482.0	481.6	0.6	-	1
GOW11_E127	RCBC	447.0	445.6	0.9	0.3	1
GOW11_E009	RCBC	500.5	500.4	1.2	0.9	1
GOW11_E013	RCBC	454.9	454.8	3	2.1	5

Table note:

mAHD = metres above Australian Height Datum

Table 5.19 Gowrie Creek—Identified existing bridges within the hydraulic model extent

Bridge design ID	Bridge length (m)	Obvert level (mAHD)	Deck depth (m)
KIN-GOJ-33	22	492.60	0.60
KIN-GOJ-31-old Homebush	38	478.50	0.70
KIN_GOJ_20	64	454.25	1.75
KIN_GOJ_1	43	435.40	1.10

5.3.6 Boundary conditions

The Gowrie Creek hydrologic model outputs were applied as inflows into the hydraulic model. Total inflows from catchments upstream of the hydraulic model extent were applied at the upstream model boundary and local inflows from areas within the hydraulic model was applied throughout the model. Internal inflow boundaries were applied as Source-Area (SA) polygons, with the flow applied to the lowest point of each SA polygon.

In the hydraulic model, consistent with the TNSTC hydraulic model setup, the local sub-catchment inflows were generally split along the Project alignment in proportion to the sub-catchment area, as shown in Figure 5.20. There were some sub-catchments where this was not the case, for example, with sub-catchments GOW_243 and GOW_77. In these locations, where inflows were not split along the Project alignment, the inflows were placed upstream of the alignment. This is considered slightly conservative and appropriate. A normal depth boundary condition was applied at the downstream boundary.

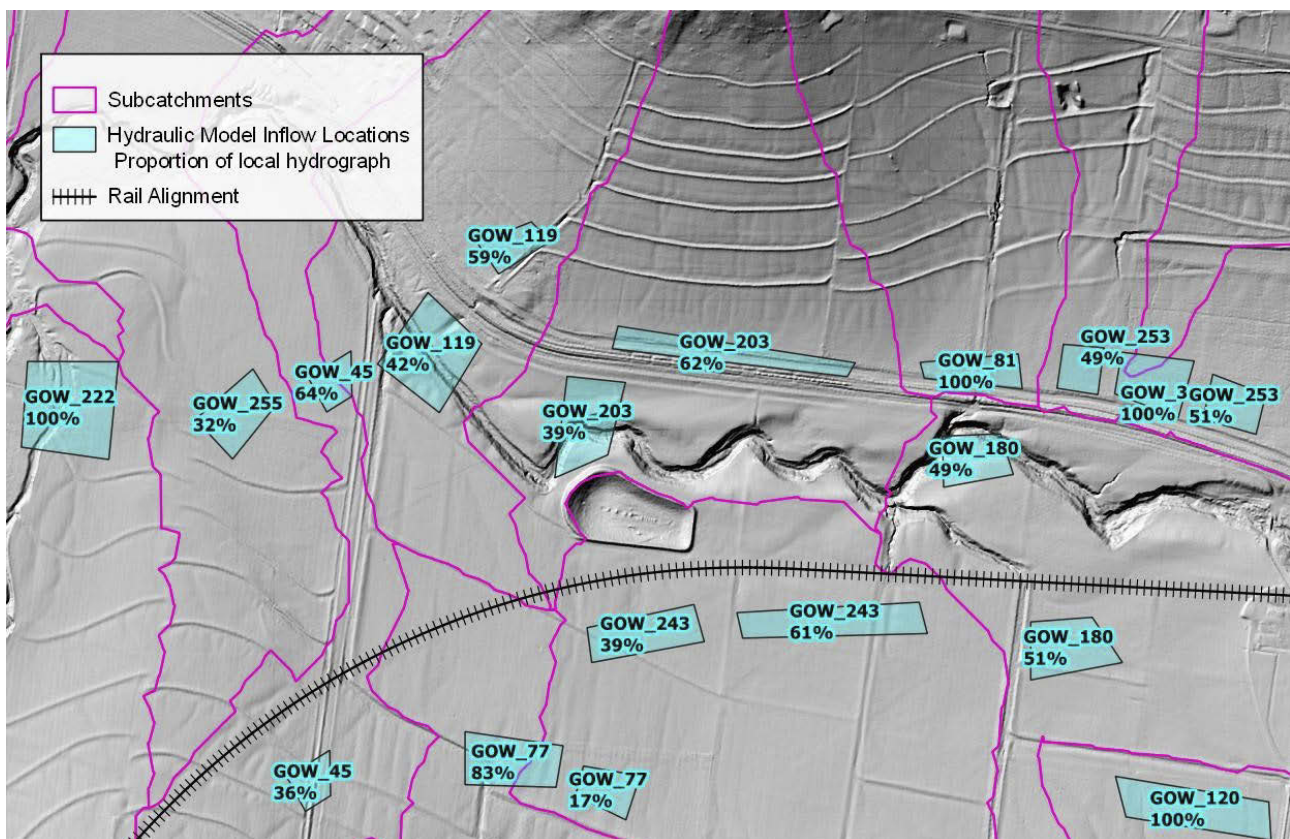


Figure 5.20 Gowrie Creek—Example of local sub-catchment inflow proportions

5.3.7 Critical duration

Two hydraulic models were developed, representing the current state of development (i.e. Existing Case) and a scenario where the Project's revised reference design is implemented, including proposed drainage structures and associated infrastructure works (i.e. Developed Case). All design event flows were modelled (Table 5.12) in the hydraulic model for both cases for the suite of AEP events (20%, 10%, 5%, 2%, 1%, 1 in 2,000, 1 in 10,000 and PMF).

A critical duration assessment was undertaken to determine which storm duration and temporal produced peak water levels across the hydraulic model and, more specifically, the Gowrie Creek study area. The peak water level and hydraulically equivalent critical duration was established by combining the maximum water level raster grids for all durations modelled for separate AEP events.

Figure 5.21 and Figure 5.22 outline an example max statistics critical duration assessment for the 5% AEP and 1% AEP events. The raster grids demonstrate which storms produce the peak water level for flood-affected cells within the hydraulic model.

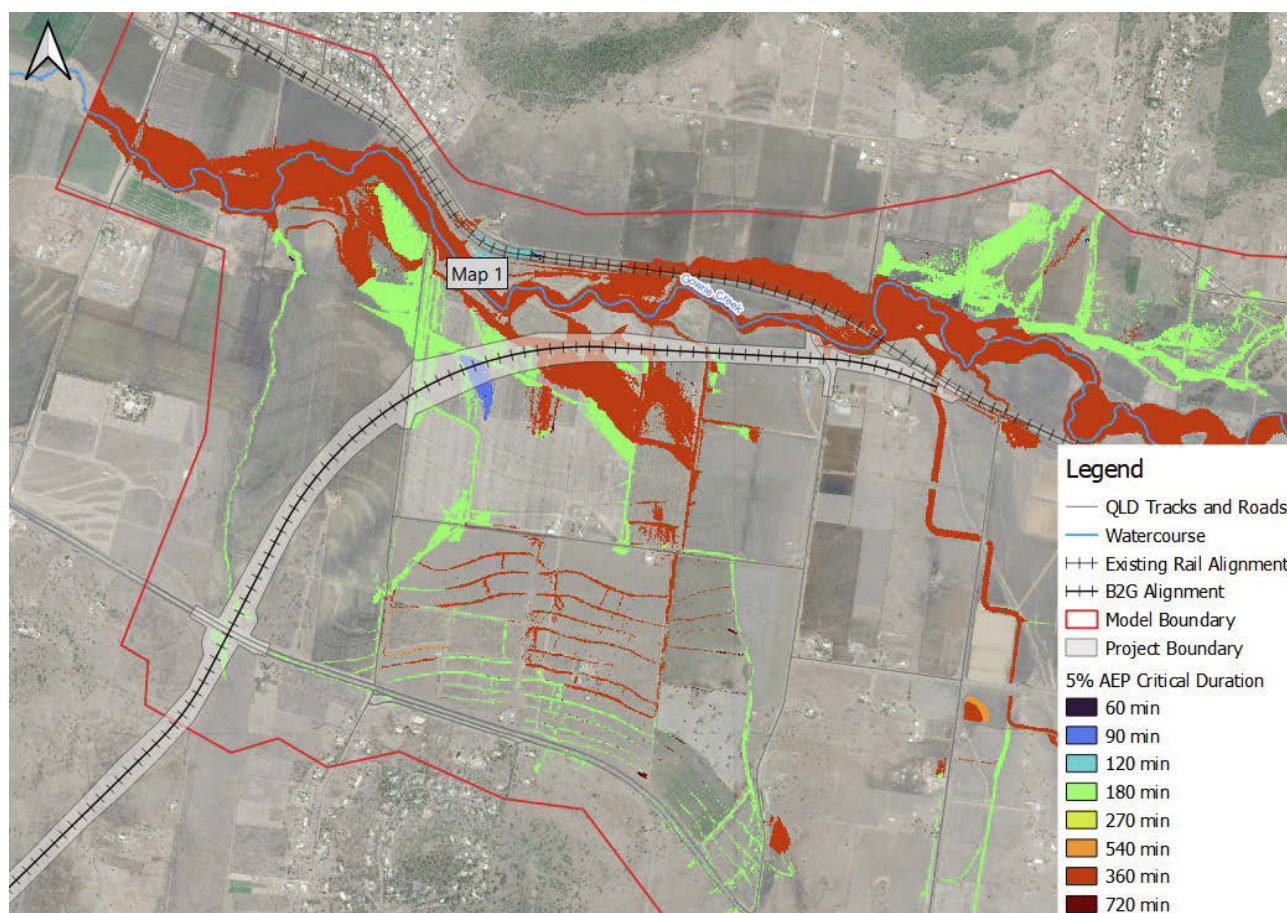


Figure 5.21 Gowrie Creek—5% AEP critical duration assessment—Existing Case

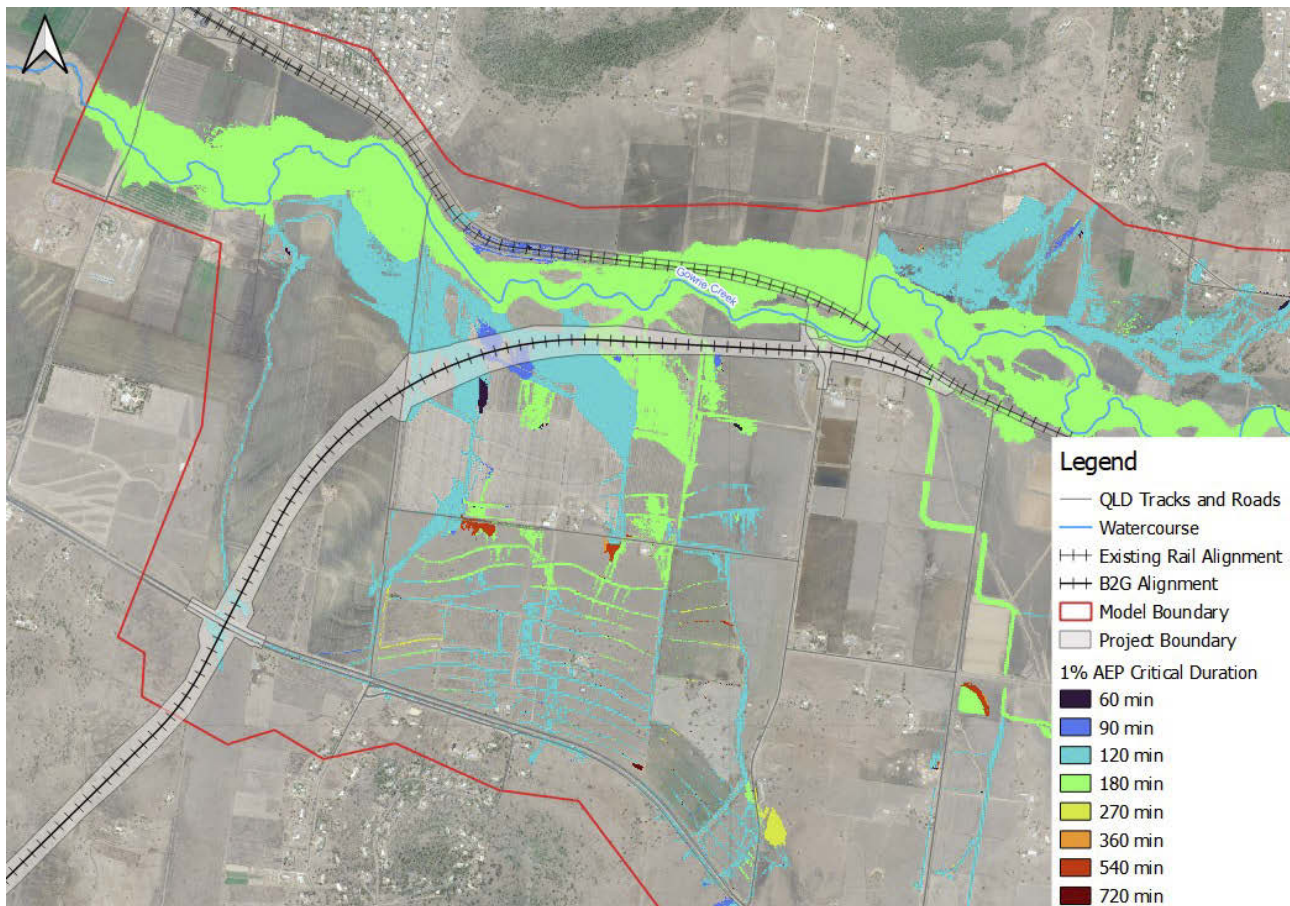


Figure 5.22 Gowrie Creek—1% AEP critical duration assessment—Existing Case

5.3.8 Validation against observed flood markers

Observed flood markers were surveyed after the January 2011 event by TRC (2014). A total of seven flood markers were available within the extent of the hydraulic model, as shown in Figure A2-A in Volume 2 – Appendix A.

The 2011 observed levels typically consisted of debris marks observed on the ground, buildings, fences and poles. The flood markers were used for hydraulic model validation in this assessment. The accuracy and reliability of debris mark data is considered to be inferior to streamflow gauge records (± 200 to 300 mm for flood markers as opposed to ± 100 mm for gauges).

To validate the developed hydraulic model, the hydraulic model was run for the January 2011 flood event using the simulated flows from the hydrologic model. Table 5.20 provides a comparison of the model flood levels and observed levels for debris marks.

These results suggest a reasonable match between the simulated and observed flood debris mark levels. Five out of six flood mark and modelled level comparisons are within the 300 mm range of accuracy. Only one modelled flood level is 370 mm higher and is considered slightly conservative. The hydraulic model does not consistently under or over-estimate the flood levels.

For reference, a flood inundation extent map for the January 2011 flood event is presented in Figure A2-A in Volume 2 – Appendix A.

Table 5.20 Gowrie Creek—Comparison between observed and model flood levels for January 2011 event

Location ID	Observed flood level at flood marks (mAHD)	Simulated water level in hydraulic model (mAHD) ¹	Difference (m)
3	480.18	480.54	0.37
4	482.61	482.82	0.21
5	482.80	Outside of flood extent ²	
6	492.91	492.78	-0.13
7	492.53	492.67	0.15
10	450.17	450.30	0.13
11	455.88	455.69	-0.19

Table notes:

- 1 Values extracted from the nearest flood affected cell
 2 Flood marker outside of the modelled flood extent

5.4 Existing Case modelling results—Gowrie Creek

5.4.1 Existing Case flood maps

Existing conditions flood maps are presented in Volume 2 – Appendix A and via an [Online Digital Portal](#). Appendix A contains the Existing Case 20% and 1% AEP flood maps, while the [Online Digital Portal](#) contains the remainder of the modelled events. The location of each map set is summarised below:

- 20% AEP peak levels: Figure A3-A
- 1% AEP peak levels: Figure A3-B
- 20% AEP peak velocity: Figure A3-C
- 1% AEP peak velocity: Figure A3-D
- 20% AEP peak hazard: Figure A3-E
- 1% AEP peak hazard: Figure A3-F
- 20% AEP peak time of inundation: Figure A3-G
- 1% AEP peak time of inundation: Figure A3-H
- 20% AEP, 10% AEP, 5% AEP, 2% AEP, 1% AEP, 1 in 2,000 AEP, 1 in 10,000 AEP and PMF Existing Case results: [Online Digital Portal](#)
- For higher resolution mapping refer to the [Online Digital Portal](#).

5.4.2 Existing Case flood inundation extent and flood levels

The predicted peak depth of water in the Gowrie Creek channel around the Leeson's Road/Draper Road intersection under the 1% AEP flood event is approximately 4 m to 5 m, and on the floodplain at the Project alignment, approximately 1 m to 2 m. The floodplain inundation caused by the 1% AEP event is from 200 m to 500 m wide.

Figure A3-B in Volume 2 – Appendix A shows the 1% AEP indicative flood extent and peak water levels within the Gowrie Creek floodplain for the Existing Case.

5.4.3 Flood immunity of existing infrastructure

Table 5.21 summarises the expected overtopping depths for important infrastructure (roads and rail) near the Project alignment under various design scenarios. The QR rail line and major roadways around the Project alignment have been assessed against a variety of design events. The existing QR rail formation is being overtopped in many regions by the 1% AEP event.

Table 5.21 Gowrie Creek—Existing Case: Estimated overtopping depths of key infrastructure

Infrastructure	Location	Maximum overtopping depth (m)							
		PMF	1 in 10,000 AEP	1 in 2,000 AEP	1% AEP	2% AEP	5% AEP	10% AEP	20% AEP
Chamberlain Road	At the end of road near Gowrie Creek	4.54	2.49	1.73	0.78	0.44	0.41	0.38	0.35
Kingsthorpe-Tilgonda Road	Near Gowrie Creek (about 650 m west of Lessons Road)	5.66	3.52	2.63	1.56	1.40	0.99	0.66	0.35
Leeson Road	Gowrie Creek Crossing	6.20	3.49	2.80	2.03	1.89	1.64	1.34	0.97
Existing QR Rail Line	About 950 m west of Gowrie Creek/ QR Rail Line crossing	2.47	0.71	0.51	0.39	0.30	0.05	Dry	Dry
Warrego Highway	Approximately 1,100 m east of Kingsthorpe Haden Road Intersection	0.33	0.21	0.19	0.11	0.07	0.04	0.01	Dry

5.4.4 Existing Case velocities

Within Gowrie Creek, north of the Project alignment, the Existing Case 1% AEP peak velocities vary from 3 m/s to 5 m/s. Over the floodplain, velocities that intersect the Project alignment range from 0.5 m/s to 1.5 m/s.

Figure A3-D in Volume 2 – Appendix A shows the 1% AEP peak flood velocities within the Gowrie Creek floodplains for the Existing Case.

5.4.5 Existing Case flood hazard

Within the Gowrie Creek main channel, there are extremely high flood hazard values (velocity-depth product) that exceed 20 m²/s during the 1% AEP event. This result is indicative of the deep, well-incised channel, that experiences high velocities.

Lower hazard zones are noted along the Project permanent footprint, with the velocity-depth product ranging from 0.1 m²/s to 1 m²/s, and 1.4 m²/s at the proposed Charlton drain crossing.

Figure A3-F in Volume 2 – Appendix A shows the 1% AEP peak flood hazard within the Gowrie Creek floodplain for the Existing Case.

5.4.6 Existing Case time of inundation

For the Existing Case 1% AEP event the model demonstrates:

- Gowrie Creek shows a peak time of inundation of approximately 11 to 14 hours
- The floodplains and breakout locations intersecting the Project alignment are typically inundated for approximately 4 to 12 hours.

Figure A3-H in Volume 2 – Appendix A shows the 1% AEP peak flood duration of inundation within the Gowrie Creek floodplains for the Existing Case.

5.5 Developed Case modelling results—Gowrie Creek

5.5.1 Drainage structures

The hydraulic design of the major drainage structures was undertaken using the TUFLOW model (1D and 2D approach).

On the Gowrie Creek floodplain, the Project includes the following floodplain (or regional structures):

- Nine RCP locations (a total of 51 cells)
- One RCBC location (a total of 16 cells)
- Two rail-over-road bridges that also convey flows within minor drainage lines.

Local drainage structures are presented in Appendix B. These structures were sized through the local drainage design and, depending on their interaction with the Gowrie Creek floodplain, were incorporated in the hydraulic model.

Design bridge details are summarised in Table 5.22, and details of the proposed local and regional culvert structures are summarised in the Cross-drainage Register located in Appendix B. The location of proposed cross-drainage features are presented in Figure A1-F in Volume 2 – Appendix A.

Table 5.22 Gowrie Creek—Proposed rail bridge locations and details

Chainage (km)	Structure ID	Approximate span (m)	Deck width (m)	Soffit level (mAHD)	Deck superstructure type	Deck depth (mm)
204.34	310-BR34	132	3.97	481.45	Type D2	2,500
205.83	310-BR35	299	3.97	463.43	Type D1	2,500

A blockage factor of 25 per cent was applied to all proposed culverts based on guidelines set out in ARR 2019. The adopted blockage factor for the proposed bridges was between 5 per cent and 10 per cent based on the waterway area blockage due to bridge piers.

5.5.2 Hydraulic design criteria outcomes

The hydraulic model was run for the Developed Case for the range of events (20%, 10%, 5%, 2%, 1% AEP events and 1 in 2,000 AEP, 1 in 10,000 AEP and PMF events) with the outcomes presented in the following sections.

5.5.2.1 Rail immunity and structures results

The results of flood modelling indicate that a 1% AEP event flood immunity to the proposed rail formation is achieved for the Project alignment across the Gowrie Creek floodplain. There is over 0.7 m freeboard to the rail formation in a 1% AEP event, noting that the formation levels were generally dictated by other engineering criteria and constraints.

Appendix B summarises the peak 1% AEP water levels on the upstream side of the Project alignment.

5.5.3 Flood Impact Objectives outcomes—Gowrie Creek

The impact of the Developed Case was assessed through inclusion of the Project's revised reference design and comparison of model results against the Existing Case results.

Changes to flooding behaviour that were assessed are reported in the following sections, including potential changes in peak water levels, time of inundation, velocity, hazard, and peak flows.

- Changes in 20% AEP and 1% AEP peak water levels are presented in Figures A5-A and A5-B in Volume 2 – Appendix A
- Changes in 20% AEP and 1% AEP velocities are presented in Figures A5-C and A5-D in Volume 2 – Appendix A
- Changes in 20% AEP and 1% AEP hazard are presented in Figures A5-E and A5-F in Volume 2 – Appendix A
- Changes in 20% AEP and 1% AEP time of inundation are presented in Figures A5-G and A5-H in Volume 2 – Appendix A
- Changes in 20% AEP and 1% AEP velocities and percentage change in velocities with cut-offs applied are presented in Figures A6-A to A6-F in Volume 2 – Appendix A
- Percentage changes in 20% AEP and 1% AEP in hazard are presented in Figures A7-A and A7-B in Volume 2 – Appendix A
- Percentage changes in 20% AEP and 1% AEP in time of inundation and with cut-offs applied are presented in Figures A8-A to A8-D in Volume 2 – Appendix A
- Refer to the [Online Digital Portal](#) for other modelled AEP results and higher resolution mapping.

One-on-one consultation with landowners who are expected to experience changes in flooding behaviour on their property was conducted by ARTC, supported by FFJV. Refer to EIS Chapter 14: Flooding and Geomorphology and EIS Appendix E: Consultation Report for further details.

The Project design outcomes relative to the FIOs (Table 4.2) are presented in the following sections.

Flood sensitive receptors referred to in this section are shown in Figures P1 (1 to 59) in Volume 2 – Appendix P.

5.5.3.1 Flood impacts on flood sensitive receptors

Based on the available aerial imagery, no buildings or critical infrastructure are located within the area impacted by afflux and change in flood hazard in the Gowrie Creek floodplain, for all modelled events up to the 1 in 2,000 AEP event.

5.5.3.2 Flood impacts on State-controlled roads

Table 5.23 presents a list of State-controlled road sections within the Gowrie Creek hydraulic model extent that have potential FIO exceedances due to the Project. The impacted road sections are shown in Figure 5.23.

A summary of key impacts, exceedance justification and mitigation measures are presented below, with further detail provided in Table 5.23:

- Three segments of the Warrego Highway, from Nass Road to Chamberlin Road (approximately 700 m), were identified as having potential exceedances.
- Potential afflux exceedances were identified along the eastbound and westbound carriageway of the Warrego Highway heading to and from Toowoomba. Existing flood conditions demonstrate the eastbound lane (road section ID #113) has less than a 20% AEP flood immunity, while the westbound lane (ID #121) does not show overtopping until the 1% AEP. The maximum change in water level occurs during the 1% AEP event, and is approximately 80 mm. The maximum impact is limited to an approximate 30 m chainage along the road.

- Potential velocity and hazard exceedances were also flagged for the eastbound lane of the Warrego Highway. For the 1% AEP, velocity and hazard increased by approximately 49 per cent (+0.79 m/s) and 24 per cent (+0.11 m²/s), respectively. The length of road for each FIO exceedance was limited to 6 m and 2 m.
- The estimated flood impacts are likely due to the changed topography, and loss of storage and conveyance caused by the proposed Project road upgrades. Additional model resolution, survey, and representation of the design road and culvert arrangement will take place during detailed design, to resolve the impacts.

Due to the variability in immunity (and impacts) between the east- and west-bound lanes, a review of the modelling files and aerial imagery indicates that the eastbound lane was constructed in 2017, with LiDAR utilised in the TUFLOW model dated 2015. The discrepancy in data timelines has led to the eastbound lane not being captured within the current flood modelling; hence, demonstrating a lower immunity and greater impacts. It is likely the immunity of the east-bound lane is higher than that modelled and potentially has similar immunity to that of the west-bound lane. This will be further revised in the detailed design stage of the Project.

It should be noted, any impacts associated with the Gowrie to Helidon Project have not been reported as part of this revised draft EIS and are covered in the Gowrie to Helidon EIS.

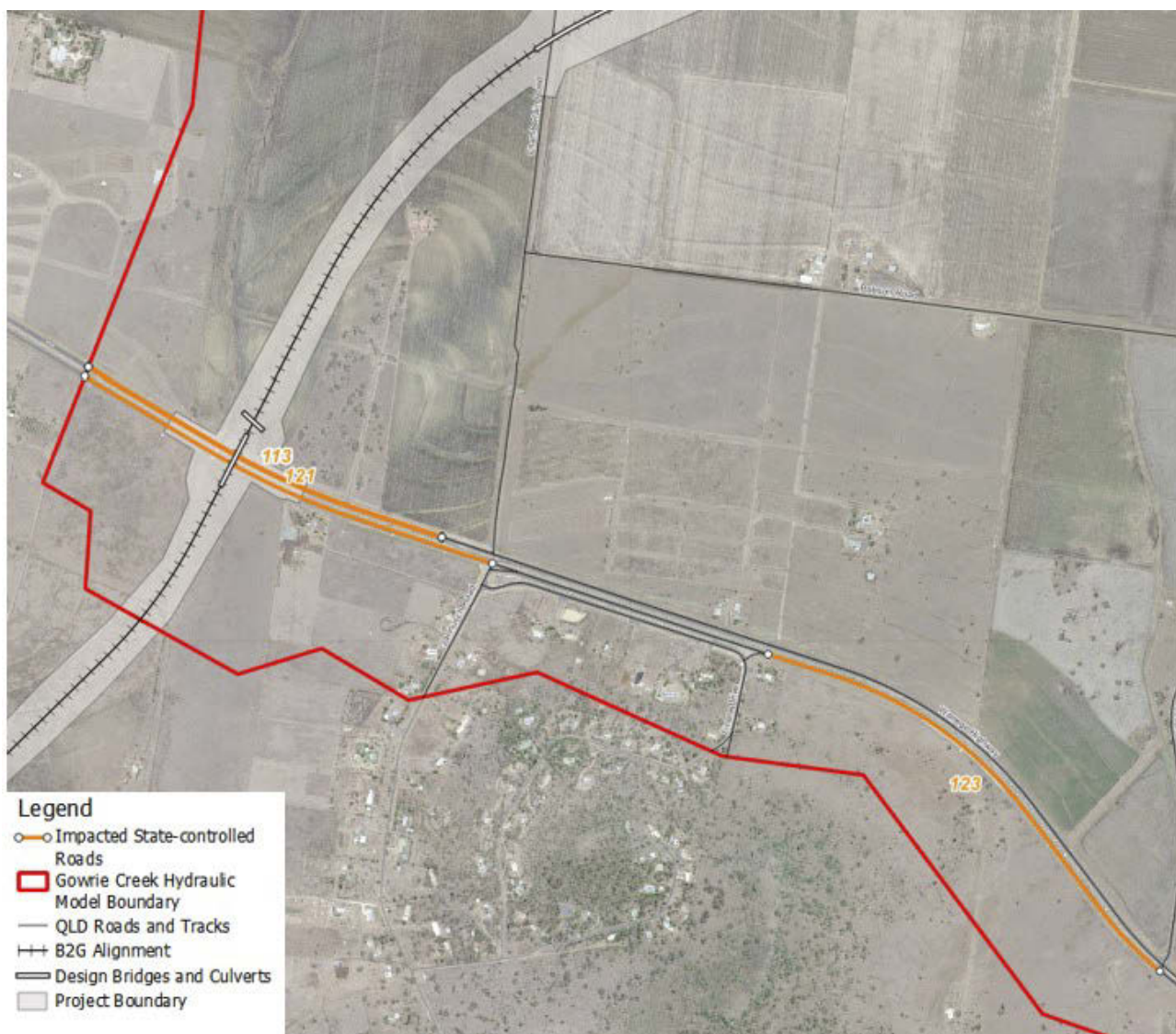


Figure 5.23 Gowrie Creek—Potentially impacted State-controlled roads

5.5.3.3 Flood impacts on local public roads

Table 5.24 presents a list of local public road sections within the Gowrie Creek hydraulic model extent that are expected to experience potential FIO exceedances due to the Project. The impacted road sections are shown in Figure 5.24.

A summary of key impacts, exceedance justification and mitigation measures are presented below, with further detail provided in Table 5.24:

- Afflux, velocity, and hazard FIO exceedances occur along two sections of Draper Road (road section IDs #27 and #124). Both locations are within the immediate vicinity of the permanent footprint; as such, it's likely the exceedances were caused by the redirection of flood water, reduced conveyance and storage on the floodplain caused by the Project rail embankment, and increased storage within the constructed drainage channel from the Charlton North Interlink SQ site between the proposed rail embankment and QR Rail line. Maximum afflux and velocity at these locations ranges between 106 mm to 784 mm, with a flood inundation increase of 5 hours to 9 hours. Potential hazard increases range from 158 per cent (0.18 m²/s to 0.48 m²/s) to 1,367 per cent (0.03 m²/s to 0.49 m²/s). Extremely high values for flood duration and hazard increases are noted when expressed as a percentage; however, these values represent moderate increases from a very small number (i.e. 0.18 m²/s).
- Afflux, velocity and hazard FIO exceedances may occur along Leeson's Road (ID #223) between Draper Road and Kingsthorpe Tilgona Road. Leeson's Road crosses Gowrie Creek, approximately 20 to 50 m downstream of the Project alignment and is highly sensitive to changes in flow conditions. The maximum afflux at this location ranges from 77 mm to 111 mm for the modelled events.
- Afflux, velocity and hazard FIO exceedances occur along Kingsthorpe Haden Road (ID #10). Similar to Leeson's Road, Kingsthorpe Haden Road crosses Gowrie Creek and therefore is sensitive to change in flow conditions. The maximum change in peak water level and velocity results are 55 mm and 4.2 m/s respectively. It is noted the increase in velocity occurs over newly flood-affected cell and demonstrates a flood extent increase at this location. The velocity recorded in the Developed Case is consistent with adjacent velocities in the Existing Case at this location.
- Scattered impacts that just exceed the FIO targets were identified for other local roads, in particular for Kingsthorpe Tilgona Road (IDs #63 and #64) and Robson Road (ID #21). These impacts occur close to the limit of model resolution (i.e. <10 m of pavement) and are typically sensitive to the coarseness of topographic data representation in the hydraulic model, and as such they are generally classified as low-risk technical exceedances. Ongoing model strengthening will help verify such scattered impacts and improve model confidence in hydraulically sensitive areas.
- Flood Impact Objective exceedances are likely due to the changed topography and loss of storage and conveyance caused by the Project (i.e. Draper Road). Additional model resolution, survey, and representation of the design road and culvert arrangement will occur during detailed design to resolve the impacts.
- Scattered and very minor exceedances (often showing the minimum exceedance length of 2 to 10 m) will be amended during detailed design through ongoing model strengthening and revisions of the hydraulic model.

It should be noted, any impacts associated with the Gowrie to Helidon Project have not been reported as part of this revised draft EIS and are covered in the Gowrie to Helidon EIS.

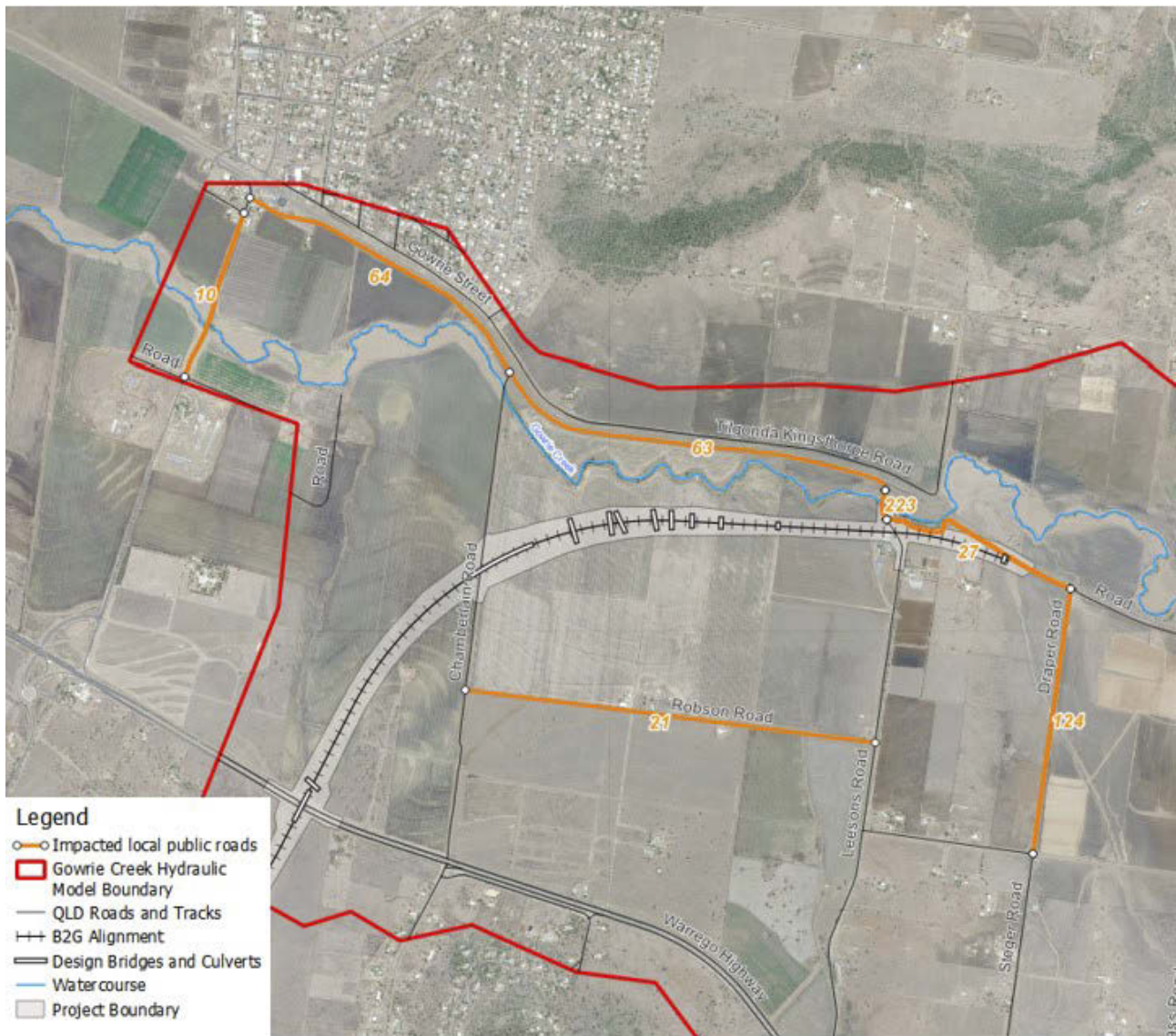


Figure 5.24 Gowrie Creek—Potentially impacted local public roads

Table 5.24 Gowrie Creek– Summary of flood impacts on local public roads

Road Object ID	Road Segment	Road Name	Surface Type	Flood Impact Objective Target					AEP	Maximum FIO Exceedance					Length (m)				Exceedance Justification	Proposed Mitigation Measures ²
				Change in Peak Water Level (mm)	Change in Time of Inundation (%)	Change in Velocity (%)	Change in Velocity (m/s)	Change in Hazard (%)		Change in Peak Water Level (mm)	Change in Time of Inundation (%) ¹	Change in Velocity (%) ¹	Change in Velocity (m/s)	Change in Hazard (%) ¹	Afflux	Time on Inundation	Velocity	Hazard		
27	16015103990000106107	Draper Road	Unsealed	10	10	10	0.5	10	20%	483	135	42	1.03	159	154.00	4.00	56.00	16.00	The exceedances recorded along Draper Road are generally due to the proposed rail embankment impeding the existing overland flow regime. Exceedances occur in two distinct locations, the first being within the constructed drainage channel from the Charlton North Interlink SQ site between the proposed rail embankment and QH Rail line. The likely cause of this exceedance experienced at this location are due to increased storage being available between the two culvert banks in the design case compared to the existing. This is due to modelling nuisances within the design case model which result in increases in water level (afflux and time of inundation). A review of the proposed culvert crossing under the Project embankment will be undertaken in the detailed design stage to better replicate the existing case and reduce exceedances on Draper Road at this location. A review of the existing topography in this area suggests the Interlink SQ project has constructed a 2m deep drain with approximately 1:2H batter slope. This would suggest the trafficability of Draper Road is compromised due to the drain construction. Further investigation and consultation regarding the impact to Draper Road at this location will be undertaken in the Detailed Design stage to determine whether Draper Road will be operational post Inland Rail construction. The second location where exceedances occur are to the south east where Draper Road runs directly adjacent the southern face of the Goerle to Heldon embankment. At this location the embankment prevents runoff towards Goerle Creek resulting in upstream ponding and conveyance of flow along the toe of the embankment towards the next cross drainage structure. At this location the Goerle to Heldon embankment directly crosses Draper Road preventing access towards the west. Further investigation and consultation regarding the impact to Draper Road at this location will be undertaken in the Detailed Design stage to determine whether Draper Road will be operational post Inland Rail construction. The exceedances remain on this road due to the current level of design performed, to a reference design level, and full optimisation of the cross drainage not being completed. Further iteration of cross drainage needs to be undertaken at the detailed design stage, to optimise culvert placement and diversion of flow through the structures. It is anticipated that a reduction in FIO exceedance may occur through mitigation, although some areas of exceedance may remain due to the presence of the project.	To mitigate identified exceedances, a review and further optimisation of the cross drainage at Draper Road will be undertaken in the detailed design phase to better mimic the existing case flooding regime. In addition, ongoing consultation will be undertaken with Interlink SQ and appropriate road manager to identify the operational capacity of Draper Road with respect to the constructed stormwater drainage and the presence of the Project.
									10%	784	53	45	1.19	169	162.00	2.00	82.00	16.00		
									5%	378	5071	159	1.41	1323	168.00	12.00	118.00	50.00		
									2%	455	1458	163	1.52	1343	176.00	18.00	146.00	54.00		
									1%	446	2380	144	1.52	1398	186.00	24.00	160.00	58.00		
									0.05%	-	-	-	-	-	-	-	-	-		
124	16015103990000106108	Draper Road	Sealed	10	10	20	1	10	20%	126	-	-	-	-	4.00	-	-	-	Rail design constraints alter the existing flow regime. Maturity of current rail embankment design is limited by available information (e.g. detailed survey)	Further road and/or rail design optimization investigations are required during detailed design.
									10%	119	-	-	-	-	8.00	-	-	-		
									5%	141	-	-	-	-	8.00	-	-	-		
									2%	154	-	-	-	-	8.00	-	-	-		
									1%	169	-	-	-	-	18.00	-	-	-		
									0.05%	-	-	-	-	-	-	-	-	-		
223	16015103990000202343	Leensons Road	Sealed	10	10	20	1	10	20%	111	-	-	-	-	32.00	-	-	-	Rail design constraints alter the existing flow regime. Maturity of current road design is limited by available information (e.g. detailed survey)	Further road and/or rail design optimization investigations are required during detailed design.
									10%	97	19	-	-	24.00	2.00	-	-	-		
									5%	82	19	20	-	20.00	-	2.00	6.00	-		
									2%	91	15	-	12	38.00	4.00	-	8.00	-		
									1%	77	62	-	59	24.00	12.00	-	14.00	-		
									0.05%	-	-	-	-	-	-	-	-	-		
21	16015103990000317788	Robson Road	Unsealed	10	10	10	0.5	10	20%	-	-	-	-	-	-	-	-	-	Rail design constraints alter the existing flow regime.	Ongoing model strengthening
									10%	-	-	-	-	-	-	-	-	-		
									5%	15	-	-	-	-	2.00	-	-	-		
									2%	-	-	-	-	-	-	-	-	-		
									1%	-	-	-	-	-	-	-	-	-		
									0.05%	-	-	-	-	-	-	-	-	-		
64	16015103990000181160	Kingsathorpe Tilgonda Road	Unsealed	10	10	10	0.5	10	20%	-	-	-	-	-	-	-	-	-	Rail design constraints alter the existing flow regime.	Ongoing model strengthening
									10%	-	-	-	-	-	-	-	-	-		
									5%	18	-	-	-	-	2.00	-	-	-		
									2%	-	-	-	-	-	-	-	-	-		
									1%	-	-	-	0.01	-	-	-	2.00	-		
									0.05%	-	-	-	-	-	-	-	-	-		
10	16015103990000181143	Kingsathorpe Haden Road	Sealed	10	10	20	1	10	20%	-	-	-	-	-	-	-	-	-	Rail design constraints alter the existing flow regime. The Flood Impact Objective exceedance has been identified due to a change in flood extent and a cell which was dry in the existing case and now flood-affected in the developed case.	Ongoing model strengthening
									10%	-	-	-	-	-	-	-	-	-		
									5%	-	-	-	-	-	-	-	-	-		
									2%	23	57	-	-	12	10.00	30.00	-	6.00		
									1%	55	49	21	4.26	28	40.00	48.00	4.00	34.00		
									0.05%	-	-	-	-	-	-	-	-	-		
63	16015103990000181159	Kingsathorpe Tilgonda Road	Unsealed	10	10	10	0.5	10	20%	-	-	-	-	-	-	-	-	-	Rail design constraints alter the existing flow regime.	Ongoing model strengthening
									10%	-	-	-	-	-	-	-	-	-		
									5%	-	-	-	-	-	-	-	-	-		
									2%	-	13	-	-	-	2.00	-	-	-		
									1%	-	-	-	-	-	-	-	-	-		
									0.05%	-	-	-	-	-	-	-	-	-		

[illegible]

Table Notes

1. Time of Submergence has been measured across the full length of the road segment and does not correspond to the exact location of exceedance within that segment.
2. Change in water level (ΔH) is calculated based on the difference in water level difference between the existing and design case resting, and does not consider the underlying topography. Depth values presented are based on the respective underlying topographies and in some cases do not align with afflux calculations. This can be attributed to interpolation calculations of results performed within the hydraulic model.
3. Percentage increases should be read in conjunction with existing and design case results. The FfOs require that change be presented in percentage change for time of inundation, velocity and hazard and can sometimes produce large increases when existing case results are significantly small. Some locations were identified to experience significant percentage increases however the "absolute" difference between Existing and Design case results is small.
4. The Flood Impact Objective exceedance has been identified due to a change in flood extent and a cell which was dry in the existing case and now flood-affected in the designed case.
5. The above table should be read in conjunction with change maps provided as part of Volume II and the Digital Platform.
6. The symbols "-" denotes that no change above Flood Impact Objectives has been determined and therefore no result has been presented.
7. Changes recorded above may only occur during a small fraction of model simulation time.
8. Three decimal places do not infer absolute accuracy in modelling results, but it is necessary to identify small changes.
9. Design optimisations that could be implemented (but not limited to) include adjustment of culvert spacing, realignment of culverts to existing footpaths when more detailed survey is available, modifying the grade of culverts, road height and alignment adjustments, removal of bridge abutments from waterway extents etc.

5.5.3.4 Flood impacts on existing rail infrastructure

Table 5.25 presents a list of rail infrastructure within the Gowrie Creek hydraulic model extent that have potential FIO exceedances due to the Project. The impacted rail sections are shown in Figure 5.25.

The Project constraints cause flow modifications along the proposed embankment, resulting in general FIO exceedances to the existing QR Rosewood to Miles line (route ID #881). Each of these exceedances are at the limit of model resolution (i.e. <10 m of pavement) and are typically sensitive to the coarseness of topographic data representation in the hydraulic model; as such, they are generally classified as low-risk technical exceedances. Ongoing model strengthening will help verify such scattered impacts and improve model confidence in hydraulically sensitive areas.

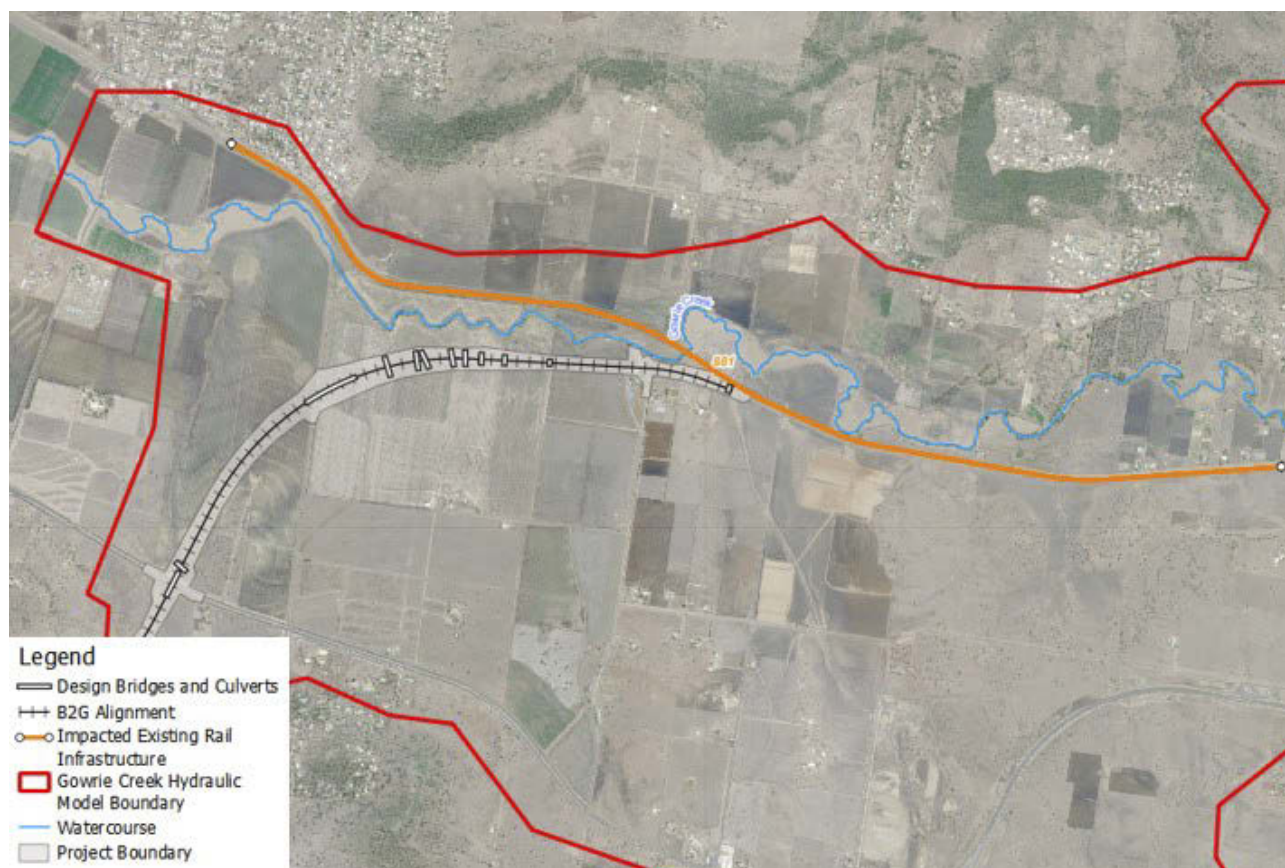


Figure 5.25 Gowrie Creek—Potentially impacted existing rail infrastructure

It should be noted, any impacts associated with the Gowrie to Helidon Project have not been reported as part of this revised draft EIS and are covered in the Gowrie to Helidon EIS.

Table 5.25 Gowrie Creek– Summary of impacts on existing rail infrastructure

Route Name	Section Name	Status	Flood Impact Objective Target					AEP	Maximum FIO Exceedance					Length (m)				Exceedance Justification	Proposed Mitigation Measures ⁹
			Change in Peak Water Level (mm)	Change in Time of Inundation (%)	Change in Velocity (%)	Change in Velocity (m/s)	Change in Hazard (%)		Change in Peak Water Level (mm)	Change in Time of Inundation (%) ²	Change in Velocity (%) ¹	Change in Velocity (m/s)	Change in Hazard (%) ³	Afflux	Time on Inundation	Velocity	Hazard		
881	Rosewood to Miles	Operational	10	10	10	0.5	10	20%	148	10	-	-	-	4.0	2.0	-	-	- Maturity of current cross-drainage design is limited by available information (e.g. detailed survey) - Rail design constraints alter the existing flow regime. - The Flood Impact Objective exceedance has been identified due to a change in flood extent and a cell which was dry in the existing case and now flood-affected in the developed case.	- Ongoing model strengthening - Further cross-drainage optimization investigations are required during detailed design.
								10%	181	14	-	-	-	4.0	2.0	-	-		
								5%	-	-	-	-	-	-	-	-	-		
								2%	-	30	-	-	-	-	8.0	-	-		
								1%	-	30	-	-	-	-	6.0	-	-		
								0.05%	-	-	-	-	-	-	-	-	-		

Table Notes

1. Time of Submergence has been measured across the full length of the road segment and does not correspond to the exact location of exceedance within that segment
2. Change in water level (afflux) is calculated based on the maximum water surface level difference between the existing and design case results, and does not consider the underlying topography. Depth values presented are based on the respective underlying topographies and in some cases do not align with afflux calculations. This can be attributed to interpolation calculations of results performed within the hydraulic model.
3. Percentage increases should be read in conjunction with existing and design case results. The FIOs require that change be presented in percentage change for time of inundation, velocity and hazard and can sometimes produce large increases when existing case results are significantly smal. Some locations were identified to experience significant percentage increases however the "absolute" difference between Existing and Design results are relatively
4. The Flood Impact Objective exceedance has been identified due to a change in flood extent and a cell which was dry in the existing case and now flood-affected in the developed case.
5. The above table should be read in conjunction with change maps provided as part of Volume II and the Digital Platform.
6. The symbol "-" denotes that no change above Flood Impact Objectives has been determined and therefore no result has been presented.
7. Changes recorded above may only occur during a small fraction of model simulation time.
8. Three decimal places do not infer absolute accuracy in modelling results, but it is necessary to identify small changes.
9. Design optimisations that could be implemented (but not limited to) include adjustment of culvert spacing, realignment of culverts to existing flowpaths when more detailed survey is available, modifying the grade of culverts, road height and alignment adjustments, removal of bridge abutments from waterway extents etc

Depth (m) ²		Time of Inundation in hours (related to % exceedance)		Time of Submergence (hours) ¹		Velocity in m/s (related to % exceedance)		Velocity in m/s (related to actual change exceedance)		Hazard in m ³ /s (related to % exceedance)	
Existing Case	Design Case	Existing	Design	Existing	Design	Existing	Design	Existing	Design	Existing	Design
-	0.148	1.98	2.18	13.01	13.01	-	-	-	-	-	-
-	0.181	2.59	2.95	13.01	13.01	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
-	-	7.35	9.58	13.57	13.57	-	-	-	-	-	-
-	-	3.54	4.62	13.02	13.02	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-

5.5.3.5 Flood impacts on private land outside the permanent footprint

Table 5.26 presents a list of private properties outside of the permanent footprint with potential FIO exceedances.

A summary of key impacts, exceedance justification and mitigation measures are provided in the below points, with further detail provided in Table 5.26:

- A total of eight private lots were identified as having a potential impact that exceed the Project FIO targets. The property types vary from 'Vacant Rural Land', 'Cattle Breeding and Fattening', 'Small Crops and Fodder—Irrigation', 'Grains' and 'Dwelling Large—House Site'.
- Potential time of inundation exceedance were identified for six of the properties. Visual inspection of the exceedances show increases are mostly isolated to regions directly upstream and downstream of the Project permanent footprint. Exceedances can be attributed to ponding that is created by alignment crossing a natural waterway, and the redirection and concentration of flow through culvert structures. The maximum increase in time of flood inundation is approximately 10.5 hours on Lot Plan 33SP294200.
- Potential afflux exceedances were identified for three properties. Afflux is notable at inlets and outlets of cross drainage structures. The maximum increase in afflux for the 1% AEP is approximately 0.65 m, located on Lot Plan 33SP294200. It should be noted that most large increases are localised and don't translate to uniform flood level increases across the lot. Ongoing refinement of the cross-drainage design will take place during detailed design to improve potential afflux issues.
- A total of six properties were identified as having velocity exceedances. Impacted properties are located at the outlets of cross-drainage structures and along the banks of Gowrie Creek. The maximum change in velocity is approximately 1.78 m/s on Lot Plan 29SP294200. Similarly to afflux considerations, review of the cross-drainage infrastructure (bridges and culverts) will be undertaken during detailed design to further reduce (or mitigate) velocity impacts.
- Increases in flood hazard are noted for six properties. Exceedances can be attributed to increased flow velocities and flood depths the inlets and outlets of bridges and culverts. The maximum increase for flood hazard is approximately 0.16 m²/s on Lot Plan 44AG109.
- In addition to the mitigation options mentioned in the above points, ongoing stakeholder consultation with impact landowners will also be undertaken during detailed design.

It should be noted, any impacts associated with the Gowrie to Helidon Project alignment have not been reported as part of this revised draft EIS and is to be covered in the Gowrie to Helidon EIS.

Table 5.26 Gowrie Creek- Summary of impacts on private land outside the permanent footprint

[illegible]

Table 1. Floodplain impacts and therefore model results vary from cell to cell. The “Area” is therefore defined as the maximum area over which change above the Flood Impact Objective occurs and does not relate directly to the maximum FIO exceedance recorded.

1. Floodplains are dynamic in nature and therefore model results vary from cell to cell. The “Area” is therefore defined as the maximum area over which change above the Flood Impact Objective occurs and does not relate directly to the maximum FIO exceedance recorded.
2. Change in water level (afflux) is calculated based on the maximum water surface level difference between the existing and design case results, and does not consider the underlying topography. Depth values presented are based on the respective underlying topographies and in some cases do not align with afflux calculations.
3. Percentage increases should be read in conjunction with existing and design case results. The FIOs require that change be presented in percentage change for time of inundation, velocity and hazard and can sometimes produce large increases when existing case results are significantly small. Some locations were identified as Flood Impact in Flood Impact Assessment but were not identified as such in the existing case and no cell which was dry in the existing case and now flood-affected in the development case.
4. The above table should be read in conjunction with change maps provided as part of Volume I and the Digital Platform.
5. The symbol “+/-” denotes that no change above Flood Impact Objectives has been determined and therefore no result has been presented.
6. Three recorded above may occur during a small fraction of model simulation time.
7. Three decimal places do not infer absolute accuracy in modelling results, but it is necessary to identify small changes.
8. Flood impact objectives are assessed based on landuse applied to the entire lot and exceedances identified do not necessarily indicate an impact to a dwelling.
9. Details have been optimised that could not be implemented (but not limited to) include adjustment of culvert spigots, realignment of culverts to existing floodplains when more detailed survey is available, modifying the grade of culverts, road height and alignment adjustments, removal of bridge abutments from waterway extents etc.
10. Landuse data, dated 2018, has been used for this assessment. Due to the changing nature of this dataset, this will be updated to the latest available information in Detailed Design.

5.5.3.6 Flow distribution

A key landowner concern is the potential change in flow distribution. To determine the change in flow distribution maximum velocity-depth modelling results were used to produce change in unit flow maps between Existing and Developed Case conditions. Unit flow is a measure of discharge that is commonly used for assessing peak flow along cross-sections, or to identify the efficient flow corridors within a floodplain.

The presented flow distribution maps were filtered with a velocity-depth product cut-off of 0.15 m²/s. The cut-off is in line with FIO requirements and was used to assist in identifying main channels (high flow zones) from shallow lower flood zones (i.e. outer extents of the flood surface).

The following points summarise the potential change in flow distribution for the Gowrie Creek catchment and should be considered in conjunction with flow distribution figures supplied in Figure Q1-1 to Figure Q1-8 in Volume 2 – Appendix Q.

- Regionally, the proposed Project produces small to moderate changes (+/-10 per cent) within the main channel of Gowrie Creek for all events assessed up to the PMF. Larger changes (positive and negative) are noted along the floodplain edges. The changes are mostly likely due to small increases and decreases in the flood extent.
- Locally, the flow distribution is governed by the relative flood interactions with various hydraulic controls, including the rail embankment, cross-drainage, road levels, and floodplain storage. Local flow redistributions primarily occur along the rail corridor, with a list of key sensitivities summarised below:
 - Flow increases occur upstream and downstream of culverts between Chamberlain Road and Leeson Road for all events. This behaviour is likely caused by the constriction and channelisation of flow through the culverts along this reach. This is particularly evident for all minor to moderate events up to the 1% AEP event, where the Project alignment does not overtop.
- Due to the high immunity of the Chamberlain Road Bridge and approach (>PMF), large changes in flow distribution are noted as the floodplain is restricted to culvert and bridge openings.

Ongoing mitigation measures to reduce changes in flow distribution are likely to occur during future design stages of the Project, and may include further refinement of the bridge abutments and additional survey where necessary.

5.5.4 Extreme events

5.5.4.1 Extreme event immunity

The risk of overtopping of the Project alignment was assessed for a range of extreme events (1 in 2,000 AEP, 1 in 10,000 AEP and PMF) with Table 5.27 presenting the depth of water above the formation level and over the top of rail at each structure. It is noted that the function of the floodplain culverts is to balance flood levels on the upstream and downstream sides of the Project alignment. As such, overtopping of the Project alignment is not predicted to result in significant excessive flows or velocities as would occur in a dam embankment overtopping scenario. Further analysis of the overtopping risk to FSRs can be found in Section 21.

Table 5.27 Gowrie Creek—Extreme events: Estimated depth of water above formation level and/or over top of rail level

Chainage (km)	Structure ID	Depth of water above formation level (m)			Depth of water above top of rail (m)		
		1 in 2,000 AEP	1 in 10,000 AEP	PMF	1 in 2,000 AEP	1 in 10,000 AEP	PMF
204.30	310-BR34	-	-	-	-	-	-
204.45	C204.45	-	-	-	-	-	-
205.80	310-BR35	-	-	-	-	-	-
206.17	C206.17	-	-	-	-	-	-
206.36	C206.36	-	-	-	-	-	-
206.42	C206.42	-	-	-	-	-	-
206.57	C206.57	-	-	-	-	-	-
206.65	C206.65	-	-	-	-	-	-
206.75	C206.75	-	-	-	-	-	-
206.88	C206.88	-	-	-	-	-	-
207.15	C207.15	-	0.01	2.28	-	-	1.58
208.21	C208.21	-	-	0.83	-	-	0.13

5.5.4.2 Impacts during extreme events

Flood immunity of FSRs is discussed in Section 5.5.3.1, and maps demonstrating the impacts during extreme events are shown on the [Online Digital Portal](#).

In accordance with the FIOs, FSRs are considered impacted by extreme flood events if the change in water level is greater than 250 mm in the 1 in 2,000 AEP event, subject to the receptor classification (i.e. at existing habitable dwellings and/or commercial and industrial buildings/premises). In addition, a stricter afflux objective of 10 mm in the 1 in 500 AEP event applies to sites with existing sensitive infrastructure, uses involving vulnerable people and/or any critical road network that was designed to be immune to flooding in any extreme event.

Review of the 1 in 2,000 AEP event results for the Gowrie Creek floodplain against the FIOs show that flood level changes at all FSRs were limited to 250 mm, and no identified existing sensitive infrastructure or uses involving vulnerable people are expected to experience a more than 10 mm increase in flood level. A separate assessment of the 1 in 500 AEP event is not required as they are conservatively represented by the 1 in 2,000 AEP event results. The flood impacts in extreme events due to the Project are therefore compliant against the stipulated FIO limits for extreme events.

Table 5.28 details extreme event results, although it should be noted that FIO targets do not apply to the 1 in 10,000 AEP and PMF events.

Table 5.28 Gowrie Creek—Summary of extreme event impacts at flood sensitive receptors

FSR ID	1 in 2,000 AEP event		1 in 10,000 AEP event		PMF event	
	Change in peak water level (mm)	Existing case flood depth (m)	Change in peak water level (mm)	Existing case flood depth (m)	Change in peak water level (mm)	Existing case flood depth (m)
GOW_ID_2	-	-	+39	0.14	+77	0.75
GOW_ID_4	-	-	-	-	+264	0.74
GOW_ID_5	-	-	-	-	+240	0.15
GOW_ID_6	-	-	-	-	+96	0.21
GOW_ID_7	-	-	-	-	+51	0.07

5.5.5 Sensitivity analysis—Gowrie Creek

The sensitivity of the model to various parameters was assessed using the following two scenarios:

- An increase in rainfall intensity (i.e. to reflect climate change scenario)
- An increase and decrease in blockage of culverts from 25 to 50 per cent and 0 per cent.

5.5.5.1 Climate change

The potential impacts of climate change in the Gowrie Creek floodplain were assessed for the 1% AEP design event to determine the sensitivity of the Project to the potential long-term changes in climate. The assessment was undertaken in accordance with ARR 2019.

The Representative Concentration Pathways 8.5 (2090 horizon) climate change scenario has been adopted for the Project with an associated increase in rainfall intensity of 21 per cent across the catchment area. The increase in rainfall intensity was applied to both the Existing Case and Developed Case scenarios.

Climate change results in increased peak water levels of up to 0.1 m in the vicinity of the Project under the 1% AEP event. The Project is expected to retain a 1% AEP flood immunity, with approximately 0.6 m of freeboard to formation level under the climate change scenario.

The climate change sensitivity result indicate there are no predicted changes in peak water levels at FSRs for the 1% AEP climate change event. Peak water levels for the climate change scenario are presented in Figure A9-C in Volume 2 – Appendix B.

5.5.5.2 Blockage

Blockage was assessed in accordance with ARR 2019. The blockage assessment was undertaken and resulted in a blockage factor of 25 per cent adopted for culverts. A minimum culvert size of 900 mm diameter was adopted to reduce potential for blockage and maintenance. A significant community concern is the potential impacts to flood conditions should the proposed culverts become blocked with debris. The primary concern is the blockage of culverts, which is likely to drive flood levels higher, particularly upstream of the culverts, and divert more flow through residences, across access roads and other infrastructure. A sensitivity analysis was undertaken with 0 per cent and 50 per cent blockage on culverts.

Notably, there are no predicted impacts (in accordance with the FIO objectives) due to the blockage assessment.

During detailed design the blockage factors will be reviewed in line with the final design and local catchment conditions. This may result in a varied and/or lower blockage factors being applied along the Project alignment. Should impacts be determined due to a change in blockage factor, additional mitigation measures will be considered in detailed design.

Maps demonstrating the effects of blockage are shown in Figures A9-A (0 per cent) and A9-B (50 per cent) in Volume 2 – Appendix A.

5.6 Additional modelling items to be considered in detailed design

The following points summarise additional items to be included in the flood models for the detailed design stage of the Project as per agreement with the Expert Flood Panel:

- A review of the hydraulic and hydrologic models shows there is a misalignment in the placement of inflow hydrographs (GOW 19, 220, 24, 26, and GOW 91,184, 146). The placement of the inflow hydrographs (2d_sa) polygons shortens the required hydraulic routing length and consequently increases the conservatism of design flows (reduction in floodplain storage). As the design is not compromised, and flood results are considered conservative, a sensitivity test will be conducted during the detailed design stage, and if required the placement of inflow hydrographs will be updated.

- Following the completion of the FFA at Oakey and Cranley gauges, an additional two years of recorded data was made available. The FFA for both gauges will be updated during the detailed design stage to account for the additional information, including the February 2022 flood event. Necessary updates to the hydrologic models, including validation analysis, will also be included during detailed design.
- The TNSTC XP-RAFTS model was set up using a standard approach of combined floodplain and channel routing (i.e. hydrograph lagging). While the design flow estimates were considered suitability conservative for this assessment, the methodology does not fully capture potential channel storage within Gowrie Creek. During the detailed design stage, channel storage will be investigated and, if necessary, the hydrologic model will be updated to include the Muskingum Routing methodology, or a similar approach.
- The focal point for determining the critical TP (per associated duration and AEP) is the Cranley Gauge. The hydrologic model will be reviewed with a specific focus on confirming the critical durations, and TPs based on a representative catchment, or at design-specific locations for the Project.
- It is noted that some existing and proposed developments within the Gowrie Creek floodplain were not fully captured during the revised draft EIS investigations. Existing development included survey of the railway, Toowoomba Second Range Crossing, and the Nass Road Detention Basin. Future developments that may be constructed before the Project include the InterlinkSQ and Charlton North Urban Stormwater Management Plan. During detailed design, the models will be updated to include relevant developments.
- Undertake additional model strengthening by improving the representation of cross-drainage infrastructure, third-party infrastructure, noise barriers, survey data, and bridge structures within the hydraulic model. Specifically, the modelling of bridges will be included as layered flow contraction (LFC) shape files within the hydraulic model.
- Gowrie Junction Road, Old Homebush Road, and Gowrie Creek Rail bridges were modelled using LFC shape polygons. Losses for Gowrie Creek bridges are considered to be conservative and will be updated during detailed design. It should be noted, that due to bridge configuration, form loss values did not have an impact on flood levels up to the 1% AEP, as the peak water levels were below the soffit level.
- Where FIO exceedances have been identified, additional analysis of cross-drainage infrastructure and other mitigation options will be investigated to reduce impacts. Land and stakeholder consultation will also form part of the mitigation process.
- Review of the modelling files and aerial imagery showed the eastbound lane of the Warrego Highway was constructed in 2017, with LiDAR utilised in the TUFLOW model dated 2015. The discrepancy in data timelines has led to the eastbound lane not being captured within the current flood modelling; hence, demonstrating a lower immunity and greater impacts. It is likely the immunity of the east-bound lane is higher than that modelled and potentially has a similar immunity to that of the west-bound lane. This will be further revised during detailed design.
- A review of the latest TUFLOW version will be undertaken at the commencement of detailed design to determine whether updates of the TUFLOW version from 2020-10-AD-iSP-w64 are beneficial to modelling.

Further details of mitigation measures can be found in Section 22. A review of proposed mitigation measures will be completed during the detailed design stage of the Project after hydraulic models have been updated to capture the above changes.

5.7 Planned and future roads

Consultation with road managers in and around Gowrie Creek was undertaken to determine if any planned or future roads need to be considered within the Project impact assessment. A response was received from DTMR identifying upgrades to the Warrego Highway as part of a future-proofing project. The inclusion of this project has not been considered in modelling completed as part of this revised draft EIS, due to the unknown timing of the Project. Ongoing engagement with DTMR will occur throughout the detailed design stage, to capture any future designs associated with this upgrade, as well as any future projects that maybe completed prior to construction of the Project. These will be included within flood modelling to be completed as part of detailed design, as required.

TRC identified rehabilitation works being undertaken on Kingsthorpe Silverleigh Road. A review of this location has identified that any works associated with this road are outside the hydraulic model domain of Gowrie Creek and therefore no impact on the hydraulic modelling is anticipated for this road rehabilitation work. ARTC will continue to consult and engage with TRC to determine any future or planned road projects that may need to be considered as part of the detailed design hydraulic modelling.

6 Westbrook and Dry creeks

The Project footprint and alignment crosses Westbrook Creek and Dry Creek to the west of Toowoomba Wellcamp Airport. The western section of the Toowoomba Wellcamp Airport is protected by a flood levee. The proposed Inland Rail crossing location is approximately 800 m upstream of the confluence of Westbrook Creek and Dry Creek. The combined Existing Case 1% AEP event inundated floodplain width at the proposed crossing point is approximately 1.7 km. At the Brimblecombe Road crossing of Dry Creek the inundated floodplain width is approximately 500 m. At the Toowoomba-Cecil Plains Road crossing of Westbrook Creek the inundated floodplain width is approximately 800 m.

The Westbrook Creek and Dry Creek floodplains study area is well defined with a few minor localised breakouts and small tributary drainage lines. Under the 1% AEP event, around the Project alignment crossing of Westbrook Creek, the Existing Case peak depth of water is approximately 4.5 m in the channel and up to 3 m on the floodplain. At Dry Creek the 1% AEP event water depth in the channel is around 3.5 m deep and up to one m on the floodplain.

The location of the Project alignment in relation to Westbrook Creek and Dry Creek is shown in Figure B1-A in Volume 2 – Appendix B.

6.1 Data collection and review—Westbrook and Dry creeks

Available background information including existing hydrologic and hydraulic models, survey, streamflow data, available calibration information and anecdotal flood data has been gathered. Data was sourced from a wide range of stakeholders including:

- TRC—existing flood studies and stream gauging data
- BoM—rainfall and stream gauging data
- DRDMW—stream gauging data
- QR—existing infrastructure details
- DTMR—existing infrastructure details.

6.1.1 Previous studies

Several previous hydrology and hydraulic studies were sourced as part of this assessment. A review of each study was undertaken to determine suitability for use on the Project as documented in the following sections.

6.1.1.1 Work Package 8, 2D Flood Study for Westbrook Final Report, DHI/TRC, 2014

This study focused on Westbrook town and included the development of a 2D rain-on-grid MIKE21 hydraulic model. A hydrologic model was not developed for this study.

6.1.1.2 Spring Creek Flood Study—Rev 2, TRC, 2017

This study focused on the upstream catchment area of Spring Creek and reported on estimated flow at a location 10 km upstream of the Inland Rail study corridor. The study focuses on a small, localised area with catchment characteristics (e.g. urbanisation and catchment slope) different to the overall Westbrook Creek catchment.

6.1.1.3 2D Flood Study for Dry Creek, TRC, 2015a

This study focused on the upstream catchment area of Dry Creek and reported on estimated flow at a location 7 km upstream of the Inland Rail study corridor. The study focuses on a small, localised area with catchment characteristics (e.g. urbanisation and catchment slope) different to the overall Westbrook Creek catchment.

6.1.1.4 Engineering Report Hydraulic Assessment—Westbrook Creek Wellcamp Project No 10525, RMA Engineers, 2015

A hydraulic assessment was undertaken in support of the Wellcamp Airport development at 1511 Toowoomba-Cecil Plains Road. The study explored the impacts of Westbrook Creek on the Wellcamp Airport site. To support the study a XP-STORM model was developed for Westbrook Creek and calibrated against the ARR Regional Flood Frequency Estimation (RFFE) model. A 2D hydraulic model was developed in TUFLOW covering an area from upstream of the Westbrook and Dry Creek confluence to upstream of the Wellcamp Airport site.

6.1.1.5 Gowrie Creek Flood Risk and Management Study Volume 1, TRC, 2013a

This model covered the upper reaches of the Gowrie Creek catchment within Toowoomba city including East Creek and West Creek. The model was calibrated for the 2010 and 2011 flood events. The design flood hydrology was based upon ARR (1987); therefore, design flood estimates were not consistent with ARR 2019.

6.1.1.6 Gowrie Creek Flood Risk and Management Peer Review, TRC, 2013b

This study provided a technical review of flood modelling work undertaken as part of the TRC (2013a) study by a Peer Review Panel.

6.1.2 Survey

ARTC provided LiDAR data from 2015 in the form of 1 m grid DEM tiles. The 1 m tiles were combined using Geographic Information System (GIS) software for use in the Project. The DEM was used for modelling within the permanent footprint, and where relevant, up to the full extent of the dataset.

A 12D model of planned works within the Toowoomba Wellcamp Airport site was provided by the Wagner Corporation. The topographical model was used to reflect infilling, and earthworks along Westbrook Creek. The model containing surveyed points and design tins was exported to a 1 m DEM and was used in the hydraulic model.

In areas that were not covered by the supplied LiDAR and Wellcamp datasets, additional DEMs were sourced from Geoscience Australia. The downloaded datasets included survey flow between 2009 and 2015. Where the datasets overlapped, preference was given to the 2015 dataset.

SRTM data was used for catchment delineation where no LiDAR data could be sourced. The primary use of the SRTM data was to inform the hydrologic modelling.

The survey data sources, and DEM developed for Gowrie Creek is shown in Figure B1-B in Volume 2 – Appendix B.

6.1.3 Aerial imagery

Aerial imagery of the Westbrook and Dry creeks study area was provided by ARTC and was used to identify and confirm topographic and vegetative characteristics of the study area. Aerial imagery captured in 2015 was made available. Additional imagery outside the study area was sourced from QGIS imagery in an open-source format.

6.1.4 Existing drainage structure data

At the time of documenting this report, there was no information available on existing drainage structures within the Westbrook Creek and Dry Creek catchments. Critical locations were identified based on existing flow paths and aerial imagery. The DEM was modified at several locations to represent natural drains at road or embankment crossing locations where the existing cross-drainage structure data was not available (Section 6.3.4).

6.1.5 Streamflow gauges

There are no streamflow gauges located in the Westbrook Creek or Dry Creek catchments study area.

6.1.6 Rainfall data

A number of daily and sub-daily rainfall stations are located in and around the Westbrook Creek and Dry Creek catchments; however, no historical rainfall data was sought for calibration purposes because other required data for calibration was not available.

6.1.7 Anecdotal and observed flood data

No anecdotal or observed flood data was available for this area of Westbrook and Dry creeks.

6.1.8 Site inspection

A site inspection was undertaken during February 2018. During the site inspection, all major waterway crossings were visited and inspected with photographs taken and details recorded of the crossing, existing drainage structures and surrounding catchment. An assessment of the relative roughness and blockage potential was undertaken during the site inspection.

6.2 Hydrologic model development—Westbrook and Dry creeks

6.2.1 Model setup

A hydrologic model for the combined Westbrook Creek and Dry Creek catchments was developed using RAFTS software Version 2018.1.2. The RAFTS software is a rainfall runoff routing model that estimates catchment flows generated by rainfall depths applied to a network of parameterised sub-catchments and waterways.

As part of the Project hydrology and hydraulic assessment, a RAFTS hydrologic model was also developed and calibrated for the Gowrie Creek catchment located to the north of the Westbrook and Dry creeks catchment (Section 5.2.2). For the development of the Westbrook and Dry creeks RAFTS model, the calibrated parameters of the Gowrie Creek hydrologic model were used as guidance along with the parameters presented in the previous studies and relevant data downloaded from the ARR 2019 Data Hub. Other adopted parameters are discussed in the following sections.

The Westbrook and Dry creeks RAFTS hydrologic model was validated against the RFFE method flow predictions. The estimated flows were also compared with reported flows in the previous studies, where the information was available.

6.2.2 Sub-catchments

Catchment-based information used in the hydrologic model depends on the geographic, physical and topographic characteristics. The catchment data includes catchment area, catchment slope, fraction impervious, catchment roughness and routing parameters.

The delineation of sub-catchments for the Westbrook and Dry creeks catchment was based on topographic data (SRTM data) using CatchmentSIM 3.5 software.

The hydrologic model extent and sub-catchment map is presented in Figure B1-C in Volume 2 – Appendix B.

6.2.3 Fraction impervious and roughness

The catchment roughness (PERN) and percentage impervious were based on the land-use layer from the Queensland Globe database and aerial imagery. Fraction imperviousness of all sub-catchments was initially estimated based on land-use data downloaded from Queensland Globe in April 2018. Then, for urbanised sub-catchments, the impervious area was estimated manually using aerial imagery. Accordingly, the impervious area was calculated as 80 per cent of the estimated urbanised area considering the area comprises low-medium density residential, roads, district and local centres (based on fraction impervious values presented in TRC, 2017 study).

6.2.4 Routing parameters

Routing between sub-catchments was modelled using the channel routing link approach (K and X parameters). The parameter K is a storage constant expressing the ratio between storage and flow, and is usually expressed in hours. It may also be viewed as the lag or travel time through the reach. The dimensionless parameter X is indicative of the relative importance of inflow and outflow to storage.

Initial estimates of K parameters were based on approximate flow distance between sub-catchment nodes and average flow velocities based on catchment slope. The adopted value for X was 0.25, which is a typical value for natural streams. The K values were lag time in hours for each routing link based on assumed flow velocity. The routing parameter K was slightly adjusted to account for large flow for a 1% AEP event. The TUFLOW hydraulic model (Section 6.3) was used to adjust the routing parameter K further to match the peak time in the hydrologic model with the modelled peak time in the hydraulic model (Section 6.3).

6.2.5 Loss model

In this assessment, an initial and continuing loss model was applied, where the lost rainfall depth does not contribute to the catchment runoff. In the loss model, it was assumed that there is an initial loss of rainfall depth followed by a constant continuing loss (mm/hr) throughout the rainfall event.

During the development of the draft EIS, a hydrologic model was developed for the Gowrie Creek catchment, located in the north of the Westbrook and Dry Creeks catchment. This model was utilised as part of a joint calibration exercise with the Gowrie Creek hydraulic model. The development of this hydrological model was informed by previous Gowrie Creek studies with Rainfall Loss Parameters reviewed. In the TRC (2013a) study, impervious areas were divided into 'old urban' and 'new urban' with different loss values. In the TRC (2013b) study, 40 per cent of the urban area (low-density urban area) is considered as impervious.

The Gowrie catchment was divided into two areas where Area 2 (downstream of the Gowrie catchment). Imperviousness for these two areas was defined as follow:

- Area 1:
 - Old urban impervious: Refers to areas of the catchment that were developed for over 20 years and generally have no defined or formalised overland flow path. Initial and continuous rainfall losses for this area type were defined based on TRC (2013a) study.

- New urban impervious: Refers to all impervious areas except for the old urban impervious. Initial and continuous rainfall losses for this area type were defined based on TRC (2013a) study.
- Pervious U/S: Refers to all pervious areas. Initial and continuous rainfall losses for this area type were defined based on TRC (2013a) study.

■ Area 2:

- New urban impervious: Refers to all impervious area types within Area 2. Initial and continuous rainfall losses for this area type were defined based on the TRC (2013b) study.
- Pervious D/S: Refers to all pervious area types within the Area 2. Initial and continuous rainfall losses for this area type were defined based on TRC (2013b) and ARR 2019 Data Hub, where applicable.

A range of applicable losses, following the review of the previous studies, were identified and used as specified in Table 6.1.

Table 6.1 Westbrook and Dry creeks catchment—Range of rainfall losses used in previous studies for 5% AEP to 1% AEP design storms

Location	Area type	Initial loss range (mm)	Continuing loss range (mm/hr)
Area 1	Old urban impervious	8.0	4.0
	New urban impervious	1.5	0
	Pervious U/S	37.0	2.5 to 6.0
Area 2	New urban impervious	0 to 1.5	0
	Pervious U/S	15.0 to 40.0	1.0 to 2.5

Gowrie Creek design rainfall losses were selected as a result of an improved correlation between estimated flow from FFA results and the RAFTS model for 1% AEP event at the two streamflow gauging stations, Oakey and Cranley. For the 1 in 10,000 AEP and PMF events, zero initial and 1.6 mm/hr continuous rainfall loss rates were assumed for all areas. Comparison between FFA and the hydrologic model results was not undertaken for the 1 in 10,000 AEP event as the number of recorded peak flows at Oakey and Cranley gauges were not sufficient to estimate a rare event peak flow.

During development of the Westbrook and Dry Creek hydrological model, it was identified that catchment characteristics were similar to that of Area 2 of the Gowrie Creek catchment. Therefore, the adopted design rainfall losses for the Gowrie Creek catchment (Area 2) along with data downloaded from the ARR 2019 Data Hub were used as loss estimates for the Westbrook and Dry creeks hydrologic model and are summarised in Table 6.2.

Table 6.2 Westbrook and Dry creeks catchment—Range of rainfall losses applicable

Source	Impervious area		Pervious area	
	Initial loss (mm)	Continuous loss (mm/hr)	Initial loss (mm)	Continuous loss (mm/hr)
Adopted value for 1% AEP design storm for Gowrie Creek – Area 2	1.0	0.0	15.0	2.5
Data downloaded from ARR 2019 Data Hub	N/A	N/A	40.0	1.0

6.2.6 Hydrologic modelling—Westbrook and Dry creeks

The hydrologic model was used to estimate the design hydrographs for the 20%, 10%, 5%, 2%, 1%, 1 in 2,000, 1 in 10,000 AEP and PMF events. A climate change scenario was also considered for the 1% AEP event based on Representative Concentration Pathways 8.5 scenario for 2090 (Section 6.5.5.1).

Details of the adopted hydrologic model parameters are presented in the following sections.

6.2.6.1 Design rainfall for up to 1 in 2,000 AEP event

Spatial distribution

The ARR 2019 design event methodology was adopted for this assessment. ARR 2019 recommends adopting the spatially varying method for catchments larger than 20 km². The Westbrook and Dry creeks catchment is approximately 317 km²; therefore, the catchment was divided into four IFD areas as shown in Figure B1-C in Volume 2 – Appendix B. Table 6.3 presents latitude and longitude of the centroid of IFD areas.

Table 6.3 Westbrook and Dry creeks catchment—IFD areas—total area longitude and latitude of the centroid of each area

IFD area number	Area (km ²)	Longitude (°E)	Latitude (°S)
1	42.9	151.841673	-27.534918
2	81.7	151.844230	-27.572356
3	100.1	151.828434	-27.617558
4	92.7	151.737083	-27.535577

Rainfall depth estimation

For 20% AEP to 1% AEP events, the design rainfalls were directly downloaded by the RAFTS software from the ARR 2019 Data Hub at the centroid of each IFD area and assumed to be uniform across each IFD area.

At the time this model was developed, the RAFTS software could not download design rainfalls for rare events with short durations (e.g. the 1 in 2,000 AEP event). Therefore, for the 1 in 2,000 AEP event, the design rainfalls directly downloaded from ARR 2019 Data Hub at the centroid of each IFD area and manually added to the RAFTS hydrologic model. Table 6.4 to Table 6.7 show the adopted rainfall intensities for each IFD area.

Table 6.4 Westbrook and Dry creeks catchment—Rainfall intensity (mm/hr) used for the IFD1 area

Duration (hr)	Annual Exceedance Probability (% AEP)						
	50	20	10	5	2	1	0.05
0.25	71.6	98.0	116.4	134.0	157.6	175.6	272.4
0.50	49.0	67.6	80.4	93.0	109.6	122.6	190.0
1.00	31.0	42.8	51.0	59.2	70.2	78.9	122.0
2.00	18.7	25.6	30.4	35.4	42.1	47.5	73.5
3.00	13.7	18.6	22.1	25.7	30.6	34.7	53.7
4.50	10.0	13.6	16.1	18.6	22.2	25.1	38.9
6.00	8.1	10.8	12.8	14.8	17.7	19.8	31.0
12.00	4.8	6.4	7.6	8.8	10.3	11.7	18.1
24.00	2.9	3.9	4.6	5.3	6.3	7.1	11.0
48.00	1.8	2.4	2.9	3.4	4.0	4.5	6.9
72.00	1.3	1.8	2.2	2.6	3.1	3.5	5.4

Table 6.5 Westbrook and Dry creeks catchment—Rainfall intensity (mm/hr) used for the IFD2 area

Duration (hr)	Annual Exceedance Probability (% AEP)						
	50	20	10	5	2	1	0.05
0.25	71.6	98.0	116.4	134.0	157.6	175.6	272.4
0.50	49.0	67.6	80.4	93.0	109.6	122.6	190.0
1.00	31.0	42.8	51.0	59.2	70.2	78.9	122.0
2.00	18.7	25.6	30.4	35.4	42.1	47.5	73.5
3.00	13.7	18.6	22.1	25.7	30.6	34.7	53.7
4.50	10.0	13.6	16.1	18.6	22.2	25.1	38.9
6.00	8.1	10.8	12.8	14.8	17.7	19.8	31.0
12.00	4.8	6.4	7.6	8.8	10.3	11.7	18.1
24.00	2.9	3.9	4.6	5.3	6.3	7.1	11.0
48.00	1.8	2.4	2.9	3.4	4.0	4.5	6.9
72.00	1.3	1.8	2.2	2.6	3.1	3.5	5.4

Table 6.6 Westbrook and Dry creeks catchment—Rainfall intensity (mm/hr) used for the IFD3 area

Duration (hr)	Annual Exceedance Probability (% AEP)						
	50	20	10	5	2	1	0.05
0.25	71.2	98.0	116.4	134.0	158.0	176.0	274.0
0.50	49.0	67.6	80.4	93.0	110.0	123.0	190.8
1.00	30.9	42.7	50.9	59.2	70.3	79.1	122.0
2.00	18.6	25.5	30.4	35.3	42.1	47.5	73.5
3.00	13.7	18.6	22.1	25.6	30.6	34.7	53.7
4.50	10.0	13.5	16.0	18.6	22.1	24.9	38.9
6.00	8.1	10.8	12.8	14.8	17.5	19.8	30.8
12.00	4.8	6.4	7.5	8.7	10.3	11.6	18.0
24.00	2.9	3.9	4.6	5.3	6.3	7.0	11.0
48.00	1.8	2.4	2.9	3.4	4.0	4.5	6.9
72.00	1.3	1.8	2.2	2.6	3.1	3.4	5.3

Table 6.7 Westbrook and Dry creeks catchment—Rainfall intensity (mm/hr) used for the IFD4 area

Duration (hr)	Annual Exceedance Probability (% AEP)						
	50	20	10	5	2	1	0.05
0.25	71.6	98.4	116.4	134.0	157.6	175.2	268.4
0.50	49.2	67.6	80.2	92.8	109.2	122.0	186.6
1.00	31.0	42.6	50.7	58.8	69.7	78.3	120.0
2.00	18.6	25.4	30.2	35.1	41.8	47.1	72.0
3.00	13.6	18.5	21.9	25.5	30.4	34.3	52.7
4.50	9.9	13.4	15.9	18.4	22.0	24.9	38.0
6.00	8.0	10.7	12.7	14.7	17.5	19.8	30.3
12.00	4.7	6.3	7.4	8.6	10.3	11.5	17.7
24.00	2.8	3.8	4.5	5.2	6.2	6.9	10.6
48.00	1.7	2.3	2.8	3.2	3.8	4.3	6.5
72.00	1.2	1.7	2.0	2.4	2.8	3.2	4.9

Areal reduction factors

The catchment area of Westbrook and Dry creeks upstream of the Project alignment is 216 km². This area was used to estimate the ARF.

The ARF was estimated using the coefficients presented in Table 6.8 and was applied to all IFD areas. The RAFTS software directly downloaded these coefficients from ARR 2019 Data Hub to estimate the ARF using Equation 2.4.1 of ARR 2019 Book 2.

Table 6.8 Westbrook and Dry creeks catchment—Aerial reduction factor coefficients from ARR 2019 Data Hub

a	b	c	d	e	f	g	h	i
0.159	0.283	0.25	0.308	7.3e-07	1	0.039	0	0

Temporal patterns

Temporal patterns for the 20% AEP to 1 in 2,000 AEP events were adopted based on the 10-burst ensemble approach presented in the ARR 2019. The TPs and pre-burst rainfall data were directly downloaded using the RAFTS software from the ARR 2019 Data Hub at the centroid of the Westbrook and Dry creeks catchment.

Median pre-burst depths for durations from one hour to four hours for a 1% AEP design event are between 2 mm and 8.4 mm (ARR 2019 Data Hub). Due to the small depth of pre-burst in this area, the impact of pre-burst rainfall is considered to be minimal in this catchment; therefore, the pre-burst rainfall was spread over three time steps at the start of the model simulation.

6.2.6.2 Design rainfall for 1 in 10,000 AEP and PMF events

Rainfall depth estimation

Probable Maximum Precipitation (PMP) depths for durations up to 6 hours (for use in modelling the PMF event) were calculated using the methodology presented in *The Estimation of Probable Maximum Precipitation in Australia: Generalised Short-Duration Method* (BoM, 2003). Once the PMP was estimated, rainfall for events between 1 in 2,000 AEP and the PMF was then interpolated. The parameters adopted for PMP rainfall estimation are presented in Table 6.9.

Table 6.9 Westbrook and Dry creeks catchment—Location data for Probable Maximum Precipitation estimation

Factor	Value
Topographic adjustment factor	1.02
Decay amplitude factor	0.94
Extreme precipitable water	84.49
Moisture adjustment factor	0.81

For the 1 in 10,000 AEP event, ARF were applied based on the interpolation methodology presented in ARR 2019 (Section 3.5, Book 8) and the rainfall depths were estimated accordingly as shown in Table 6.10.

Table 6.10 Westbrook and Dry creeks catchment—Rainfall intensity for the 1 in 10,000 AEP and Probable Maximum Precipitation events

Duration (hrs)	Areal reduction factor	1 in 10,000 AEP rainfall intensity (mm/hr)	PMP rainfall intensity (mm/hr)
0.25	0.56	156.9	440.0
0.50	0.63	123.6	320.0
0.75	0.70	113.9	280.0

Duration (hrs)	Areal reduction factor	1 in 10,000 AEP rainfall intensity (mm/hr)	PMP rainfall intensity (mm/hr)
1.00	0.77	102.9	250.0
1.50	0.81	88.0	206.7
2.00	0.84	75.8	180.0
2.50	0.86	67.7	160.0
3.00	0.88	61.8	146.7
4.00	0.89	51.4	122.5
5.00	0.91	45.3	106.0
6.00	0.91	40.2	95.0

Table note:

No ARF was applied to the PMF rainfall.

Temporal patterns

Temporal patterns for the 1 in 10,000 AEP and PMF design events for up to 6 hours duration were obtained from *The Estimation of Probable Maximum Precipitation in Australia: Generalised Short-Duration Method* (BoM, 2003).

6.2.6.3 Design rainfall losses

A range of design rainfall losses and routing parameters were trialled for the design events in accordance with the ranges detailed in Section 6.2.5. Loss rates shown in Table 6.11 were selected as a result of:

- Improved correlation between the estimated flow in the hydrologic model and the RFFE results
- Comparison between the hydrologic and hydraulic modelled hydrographs, and peak flows at a few critical locations along the Project alignment.

For impervious areas, the same loss parameters were applied to the entire catchment. There is variability in characteristics of the pervious area with the upstream area of the Westbrook and Dry Creeks catchment being steeper and more urbanised than the rest of the catchment. Two different loss models were applied in the upstream catchment and the rest of catchment as follow:

- Pervious upstream (U/S): applied to the upstream catchment where the loss values are smaller
- Pervious: applied to the rest of catchment where loss values are higher.

Table 6.11 Westbrook and Dry creeks catchment—Adopted rainfall losses

Design event	Area type	Initial loss (mm)	Continuous loss (mm/hr)
20% AEP to 5% AEP	Impervious	1.0	0.0
	Pervious U/S	20.0	1.0
	Pervious	35.0	1.5
1% AEP and 2% AEP	Impervious	1.0	0.0
	Pervious U/S	15.0	1.0
	Pervious	25.0	1.5
1 in 2,000 AEP	Impervious	0.0	0.0
	Pervious U/S	10.0	1.0
	Pervious	15.0	1.0
1 in 10,000 AEP and PMF	Impervious	0.0	0.0
	Pervious U/S	0.0	1.0
	Pervious	0.0	1.0

6.2.6.4 Hydrologic model validation

The hydrologic model was not calibrated due to unavailability of observed streamflow and anecdotal flood level data in the catchment; however, the loss and routing parameters were adjusted to match the RFFE results and through comparison of the modelled hydrographs in the hydrologic and hydraulic models. The estimated flows were also compared with reported flows in the previous studies where the information was available.

Regional Flood Frequency Estimation

Since there is no stream gauge data available within the Westbrook and Dry Creeks catchment, it was not possible to undertake a FFA; however, the RFFE method from ARR 2019 was used to validate the hydrologic model.

The online RFFE tool was used to obtain an estimation of peak flows at the outlet of three sub-catchments including S1.09 and S11.05 where the Project alignment crosses Westbrook Creek and Dry Creek, respectively, and at the outlet of sub-catchment S1.11 where the two creeks merge. The RFFE inputs and outputs are summarised in Table 6.12 and Table 6.13, respectively.

Table 6.12 Westbrook and Dry creeks sub-catchments—Model inputs for the Regional Flood Frequency Estimation

Model parameter	Input value
Sub-catchment S1.11	
Region name	East Coast
Region code	1
Latitude at S1.11 (degree)	-27.53
Longitude at S1.11 (degree)	151.77
Latitude at catchment centroid (degree)	-27.57
Longitude at catchment centroid (degree)	151.83
Distance of the nearest gauged catchment in the database (km)	16.8
Catchment area (km ²)	247
Design rainfall intensity, 50% AEP and 6 hr duration (mm/hr)	8.07
Design rainfall intensity, 2% AEP and 6 hr duration (mm/hr)	17.61
Sub-catchment S1.09	
Region name	East Coast
Region code	1
Latitude at S1.11 (degree)	-27.539
Longitude at S1.11 (degree)	151.77
Latitude at catchment centroid (degree)	-27.594
Longitude at catchment centroid (degree)	151.824
Distance of the nearest gauged catchment in the database (km)	16.9
Catchment area (km ²)	205
Design rainfall intensity, 50% AEP and 6 hr duration (mm/hr)	7.99
Design rainfall intensity, 2% AEP and 6 hr duration (mm/hr)	17.46
Sub-catchment S11.05	
Region name	East Coast
Region code	1
Latitude at S1.11 (degree)	-27.53

Model parameter	Input value
Longitude at S1.11 (degree)	151.78
Latitude at catchment centroid (degree)	-27.53
Longitude at catchment centroid (degree)	151.85
Distance of the nearest gauged catchment in the database (km)	15.8
Catchment area (km ²)	41
Design rainfall intensity, 50% AEP and 6 hr duration (mm/hr)	8.06
Design rainfall intensity, 2% AEP and 6 hr duration (mm/hr)	17.62

Table 6.13 Westbrook and Dry creeks sub-catchments—Regional Flood Frequency Estimation results obtained from ARR 2016 website

AEP (%)	Flow (m³/s)	95% probability limit	
Sub-catchment S1.11			
50	82.0	31.5	213
20	189.0	76.2	470
10	296.0	113.0	781
5	431.0	151.0	1,210
2	665.0	206.0	2,140
1	891.0	250.0	3,150
Sub-catchment S1.09			
50	68.2	26.2	177
20	157.0	63.4	392
10	246.0	94.2	652
5	359.0	126.0	1,010
2	554.0	172.0	1,790
1	743.0	208.0	2,630
Sub-catchment S11.05			
50	19.9	7.6	51.8
20	45.4	18.3	114.0
10	70.8	27.0	188.0
5	103.0	35.9	290.0
2	158.0	48.8	509.0
1	211.0	59.2	744.0

Hydrologic model validation against hydraulic model

The hydrologic and hydraulic model hydrographs were compared at critical locations at the Project alignment to adjust rainfall losses and routing parameters accordingly. The final values were adopted as a result of improved correlation between the hydrologic and hydraulic modelling results.

Figure 6.1 to Figure 6.4 illustrate the comparison of modelled hydrographs and peak flow estimates from the hydrologic and hydraulic models. The graphs demonstrate that the models correlate well.

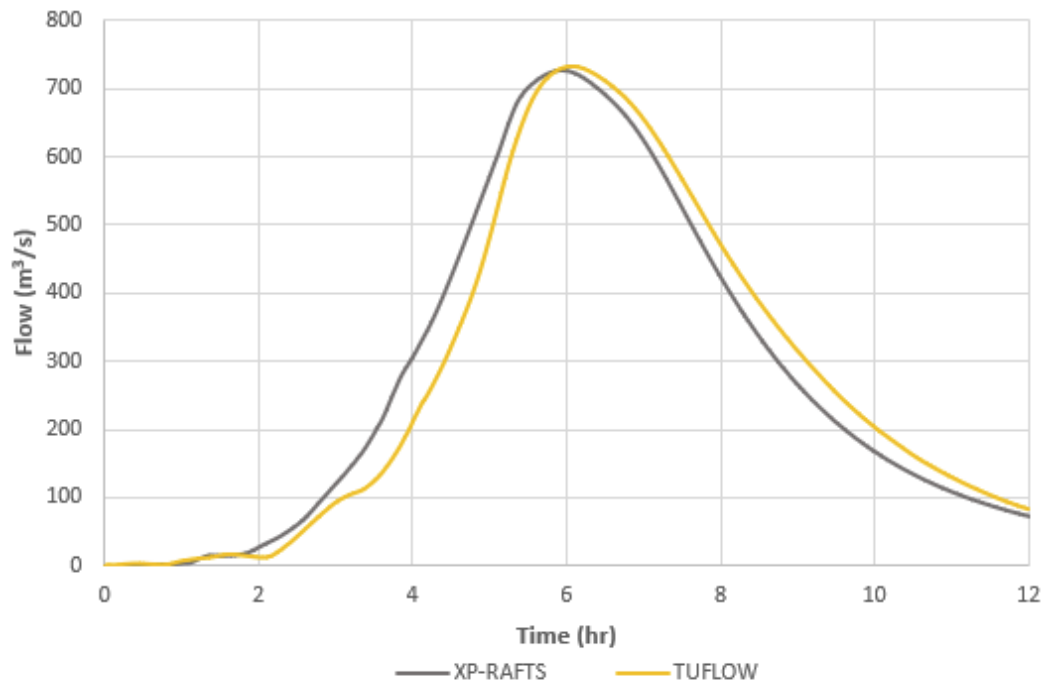


Figure 6.1 Westbrook and Dry creeks catchment—Comparison of sub-catchment S1.08 modelled hydrographs in TUFLOW and RAFTS models—storm 4.5 hrs burst 9

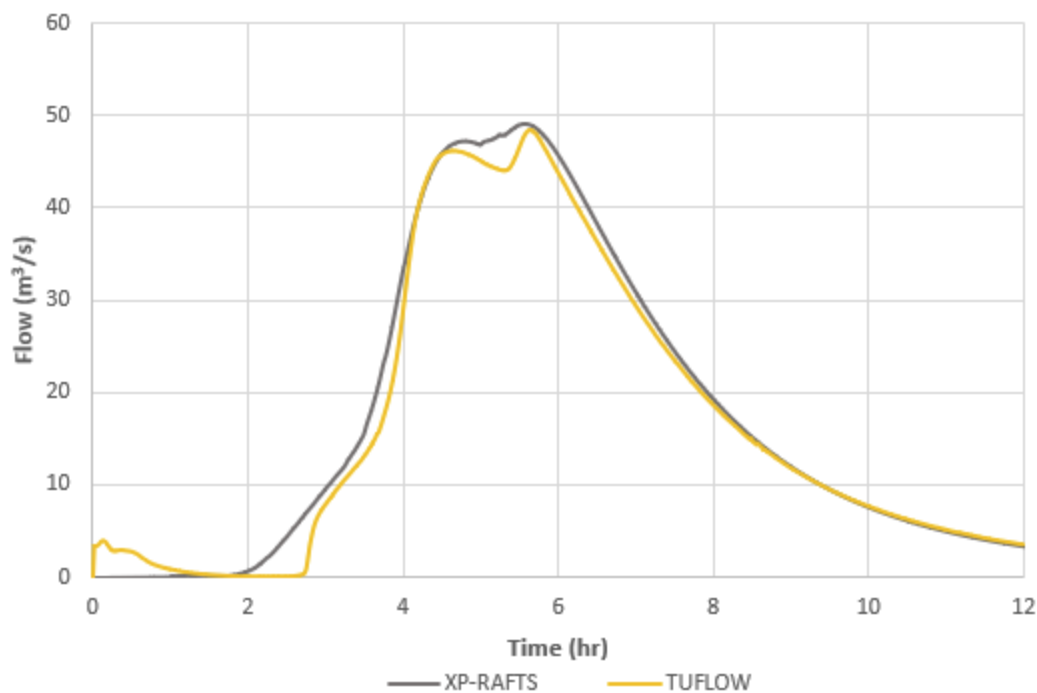


Figure 6.2 Westbrook and Dry creeks catchment—Comparison of sub-catchment S1.10 modelled hydrographs in TUFLOW and RAFTS models— storm 4.5 hrs burst 9

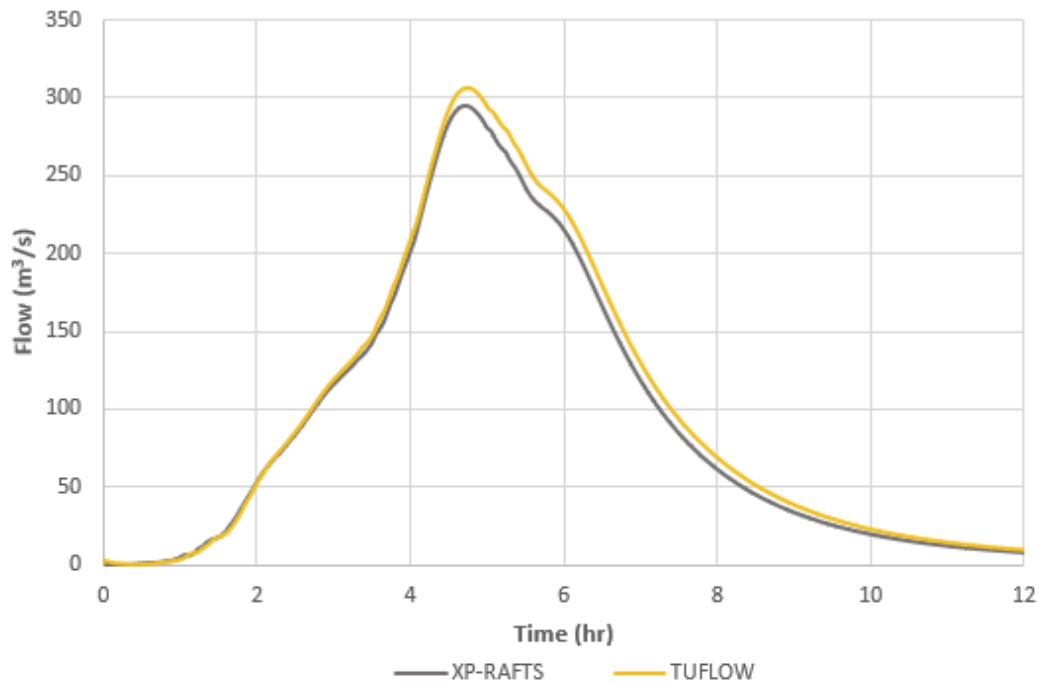


Figure 6.3 Westbrook and Dry creeks catchment—Comparison of sub-catchment S3.04 modelled hydrographs in TUFLOW and RAFTS models— storm 4.5 hrs burst 9

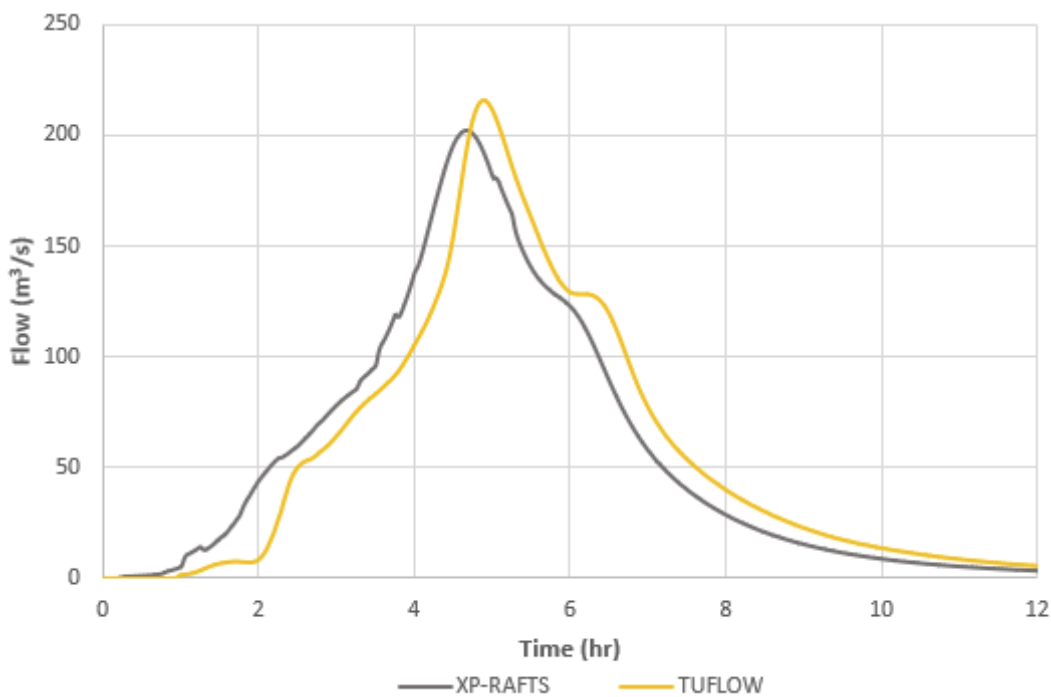


Figure 6.4 Westbrook and Dry creeks catchment—Comparison of sub-catchment S1.05 modelled hydrographs in TUFLOW and RAFTS models—storm 4.5 hrs burst 9

6.2.6.5 Estimated design flows

The design flow for the 1% AEP event was estimated using the hydrologic model. The modelled peak flow estimates were compared against the RFFE estimates and the results from the previous studies to ensure the estimated flows are within the accepted ranges.

Table 6.14 provides comparison of the peak flow modelled by hydrologic model and RFFE estimates at three sub-catchments. The results show that the difference between the RFFE and hydrologic model estimates at the area of interest is less than 6 per cent for the 1% AEP event and less than 15 per cent for the 2% AEP event. The difference between the RFFE and hydrologic model estimates for the 10% AEP event is up to 45 per cent.

Table 6.14 Westbrook and Dry creeks sub-catchments—Comparison between estimated peak flows by Regional Flood Frequency Estimation with the RAFTS model

Sub-catchment	Peak flow by RFFE (m ³ /s)			Peak flow by RAFTS model (m ³ /s) ¹			Difference (%)		
	1% AEP	2% AEP	10% AEP	1% AEP	2% AEP	10% AEP	1% AEP	2% AEP	10% AEP
S1.11	891	665	296	921	756	429	+ 3.4	+13.7	+44.9
S1.09	743	554	246	770	633	356	+ 3.6	+14.3	+44.7
S11.05	211	158	71	200	167	85	- 5.2	+5.7	+19.7

Table note:

¹ Median value of all 10 TPs (bursts) for the critical duration

Table 6.15 shows a comparison between the reported 1% AEP peak flows in this assessment with estimates from the previous studies. The differences between the previous study and the current study results can be related to the following factors:

- The RMA Engineers (2015) study used ARR 1987 methodology while the current study used ARR 2019 methodology. The rainfall depths used in RMA Engineers (2015) are smaller than the ones used in this current study, and the ensemble storm approach was not used in the ARR 1987 methodology.
- The location of reported peak flows in the previous studies is not the same as those used in this current investigation; therefore, the total catchment areas for the reported locations are slightly different from the area used in this assessment.

Table 6.15 Westbrook and Dry creeks catchment—Comparison between flows for 1% AEP event in previous studies with this study's hydrologic model

B2G hydrologic model sub-catchment	1% AEP peak flow (m ³ /s) ¹	Previous investigation and the associated sub-catchment in that study	B2G hydrologic model (m ³ /s)	Difference (%)
S1.09	770	RMA Engineers (2015) at catchment outlet	691	- 10%
S3.02	211	TRC (2017) at R3A7 sub-catchment	213	+ 0.1%

Table note:

¹ Median value of all 10 TPs (bursts) for the critical duration

The estimated peak flows in this assessment at the Project alignment have a reasonable correlation with the RFFE model results and results from TRC (2017) and RMA Engineers (2015) studies. This comparison provides confidence in the current 1% AEP event flow estimates.

6.3 Hydraulic model development—Westbrook and Dry Creeks

A 2D modelling approach was adopted to simulate the flood regime around the Project alignment crossing at Westbrook Creek and Dry Creek. The platform used for hydraulic modelling was TUFLOW HPC software package (version 2020-10-AD) with SGS and QT approach. The processes and assumptions adopted throughout the development of the hydraulic model are described in the following sections.

6.3.1 Model setup

The setup of the hydraulic model is summarised in Table 6.16.

The model extent and the spatial distribution of land use in the 2D model domain is presented in Figure B1-D and Figure B1-E in Volume 2 – Appendix B, respectively.

Table 6.16 Westbrook and Dry creeks—Hydraulic model setup overview

Parameter	Information
Completion date	November 2022
AEPs assessed	20% AEP, 10%, 5%, 2%, 1%, 1 in 2,000 and 1 in 10,000 AEP and PMF
Hydraulic model build	TUFLOW software version Build 2020-10-AD-iSP-w64
Model extent	Refer to Figure B1-D in Volume 2 – Appendix E
Grid Size	5 m
SGS distance	1 m
QT grid size	2.5 m, 2 nd Order
DEM (year flown)	1 m DEM – 2015
Roughness	Spatially varying roughness values compliant with industry norms. Refer to Table 6.17.
Eddy viscosity	Wu (default)
Model Calibration	N/A
U/S model boundary	Flow/Time boundary obtained from RAFTS model
D/S model boundary	Height/Flow boundary based on the Westbrook Creek stream slope D/S of the model
Internal boundaries	SA polygons and internal Flow/Time boundary lines based on the RAFTS model
Hydraulic model timestep	Adaptive timestep
Modelled scenarios	Existing Case, Developed Case
Sensitivity analysis	Climate change and Blockage (0% and 50% Blockage on culverts)

6.3.2 Model topography and grid size

The topography within the hydraulic model was primarily based on a 1 m DEM data collected during 2015, which was supplied by ARTC. A 5 m x 5 m grid size was selected to allow the features of the channels and floodplain to be represented with sufficient accuracy, while maintaining efficient model run times. The 5 m x 5 m grid size is considered adequate to represent floodplain topography and is consistent with the approach typically adopted in similar flood studies.

The 2D model topography was modified adjacent to the culverts so that the existing drainage flows, existing road/rail crowns and levee banks were represented accurately in the model.

For the Developed Case, the topography was modified to include the Project rail embankment, drainage structures and associated infrastructure (Section 6.5.1).

6.3.2.1 Sub-grid sampling and Quadtree mesh refinement

The hydraulic model was established using TUFLOW's 2020-10-AD model version, including SGS and QT add-on modules. The below points provide a brief description of each module, along with the key parameters adopted for this assessment.

- Traditional grid-mesh based 2D hydraulic models sample the underlying DEM by taking a single elevation at the cell centres and cell faces. Sub-grid sampling allows for increased resolution by sampling a greater number of points of the underlying DEM to establish a water surface elevation versus volume relationship for each grid, along with a water surface elevation versus width relationship (wetted perimeter) at cell sides. The adoption of SGS allows increased representation of storage and conveyance between cells. A SGS distance of 1 m was adopted for this assessment.
- The QT mesh is constructed by subdividing the base mesh-grid size into four smaller square cells. Cells can be further sub-divided into four, and so on, allowing for additional resolution in location of interest. The base hydraulic model grid cell size (5 m x 5 m) was revised to include additional spatial resolution of approximately 2.5 m, to better represent model velocity at the entry and exit of culverts. Quadtree meshes were applied in the model covering minimum 25 m upstream and 100 m downstream of the design culvert locations, and culverts within the permanent footprint.

6.3.3 Roughness

The hydraulic roughness generally reflects the types of development and ground cover that exists within the hydraulic model extent. The distribution of roughness categories adopted for this assessment was based on the information supplied in the land-use layer from Queensland Globe database, aerial imagery and confirmed during the site inspection. Specific roughness values applied to the model are detailed in Table 6.17.

Table 6.17 Westbrook and Dry creeks—Adopted roughness values

Category	Manning's 'n' value
Floodplain	0.050
Roads	0.025
Developed area	0.083
Vegetated waterways	0.050
Waterways	0.033
Dense vegetation	0.100
Rail embankment	0.040

Figure B1-E in Volume 2 – Appendix B shows the spatial breakdown of land use within the hydraulic model.

6.3.4 Hydraulic structures

There was no information available about the existing drainage structures within the Westbrook Creek and Dry Creek catchments. Initial 2D modelling results show ponding at road and embankment crossings and in some other areas. These locations were further inspected using aerial imagery and reviewing the topographic representation of open channels and hydraulic structures. This clearly showed that the model topography required adjustment to provide a more realistic reorientation of flow paths; therefore, the DEM was lowered at five locations to create the identified flow paths.

6.3.5 Boundary conditions

6.3.5.1 External and internal boundary conditions

The inflow boundary and internal source boundaries were based on results of the hydrologic modelling. A normal depth boundary was applied for the downstream outflow model boundary. It was confirmed that it was a sufficient distance away from the area of interest so as not to impact on the model results.

Dry Creek is a tributary of Westbrook Creek running east to west and joining Westbrook Creek from the north-east on the downstream side of the Project alignment. Several Westbrook Creek and Dry Creek sub-catchments, including S1.04, S11.02, S3.04, S12.02, S9.01, S10.01 and S14.01, are located upstream of the hydraulic model domain. Accordingly, the upstream boundary was applied as a flow/time boundary with total flow of these sub-catchments obtained from the hydrologic model.

Delineated sub-catchments in the hydrologic model were used as internal inflow boundaries within the hydraulic model extent. The modelled flows in the hydrologic model were applied to the lowest point of each SA polygon. Since the lowest point of the SA polygons for sub-catchments S2.02, S6.02, S7.01, S8.01 and S1.11 lie downstream of the Project alignment, extra internal inflow boundaries were added as a flow/time boundary and the total flow was divided accordingly based on catchment area upstream and downstream of the Project alignment.

The same boundary conditions were used for both the Existing Case and the Developed Case.

Figure B1-D in Volume 2 – Appendix B shows the hydraulic model boundaries.

6.4 Existing Case modelling results—Westbrook and Dry Creeks

6.4.1 Hydrologic model peak flow assessment

A critical duration assessment was undertaken to determine which storm duration(s) produced peak flood flows where the major waterways are intersected by the Project alignment. To assess the critical storm duration, the following methodology was adopted:

- The models were simulated for a range of AEP events: 20% AEP, 10%, 5%, 2%, 1%, 1 in 2,000, 1 in 10,000 AEP and PMF:
 - Each AEP was modelled for a range of durations
 - Each duration was modelled for each of the 10 associated TPs
- A critical duration assessment was undertaken at the locations mentioned above to determine which duration produced the highest median flow of the 10 TPs for each event.

Table 6.18 outlines the critical duration and TPs for each AEP event. Table 6.19 presents a sample of estimated peak flows applied to the hydraulic model at outlets of several sub-catchments for the 1% AEP event.

Table 6.18 Westbrook and Dry creeks—Modelled critical duration and temporal patterns

AEP (%)	Critical storm duration and temporal pattern ¹
20	9 hour – Pattern 3
	12 hour – Pattern 5
10	9 hour – Pattern 1
	9 hour – Pattern 3

AEP (%)	Critical storm duration and temporal pattern ¹
5	6 hour – Pattern 7
	9 hour – Pattern 1
	9 hour – Pattern 8
2	3 hour – Pattern 6
	3 hour – Pattern 7
	12 hour – Pattern 9
1	3 hour – Pattern 6
	4.5 hour – Pattern 1
	4.5 hour – Pattern 8
	4.5 hour – Pattern 9
1 in 2,000	1.5 hour – Pattern 9
	3 hour – Pattern 4
	3 hour – Pattern 5
1 in 10,000	3 hour – PMP Pattern
	4.5 hour – PMP Pattern
PMF	2 hour – PMP Pattern
	3 hour – PMP Pattern

Table note:

1 Multiple durations are reported per event due to the selection of various focal points along the alignment

Table 6.19 Westbrook and Dry creeks—1% AEP event peak flow at key locations as applied in the hydraulic model

Sub-catchment name	1% AEP event peak flow (m ³ /s)	Critical storm (hrs) duration and temporal pattern
S1.04_TOT	232.8	4.5 hrs T8
S1.05_LOC	36.8	4.5 hrs T1
S1.06_LOC	15.1	4.5 hrs T1
S1.07_LOC	44.0	4.5 hrs T9
S1.08_LOC	11.2	4.5 hrs T1
S1.09_LOC	14.0	4.5 hrs T9
S1.10_LOC	13.9	4.5 hrs T1
S1.11_LOC	22.3	4.5 hrs T1
S1.12_LOC	18.1	4.5 hrs T1
S1.13_LOC	23.6	4.5 hrs T1
S1.14_LOC	14.9	4.5 hrs T1
S1.15_LOC	14.6	4.5 hrs T1
S10.01_LOC	14.8	4.5 hrs T1
S11.02_TOT	142.4	3 hrs T6
S11.03_LOC	35.6	4.5 hrs T9
S11.04_LOC	35.3	4.5 hrs T1
S11.05_LOC	33.9	4.5 hrs T1
S12.02_TOT	63.7	4.5 hrs T1
S13.01_LOC	16.2	4.5 hrs T1
S14.01_LOC	14.6	4.5 hrs T1

Sub-catchment name	1% AEP event peak flow (m ³ /s)	Critical storm (hrs) duration and temporal pattern
S15.01_LOC	13.1	4.5 hrs T1
S2.01_LOC	26.8	3 hrs T6
S2.02_LOC	29.4	4.5 hrs T1
S3.04_TOT	301.6	3 hrs T6
S3.05_LOC	45.7	4.5 hrs T1
S6.01_LOC	37.2	4.5 hrs T1
S6.02_LOC	21.9	4.5 hrs T1
S7.01_LOC	17.5	4.5 hrs T1
S8.01_LOC	16.8	4.5 hrs T1
S9.01_LOC	26.0	4.5 hrs T1

6.4.2 Existing Case flood maps

Existing conditions flood maps are presented in Volume 2 – Appendix B and via an [Online Digital Portal](#). Appendix B contains the Existing Case 20% and 1% AEP flood maps, while the [Online Digital Portal](#) contains the remainder of the modelled events. The location of each map set is summarised below:

- 20% AEP peak levels: Figure B2-A
- 1% AEP peak levels: Figure B2-B
- 20% AEP peak velocity: Figure B2-C
- 1% AEP peak velocity: Figure B2-D
- 20% AEP peak hazard: Figure B2-E
- 1% AEP peak hazard: Figure B2-F
- 20% AEP peak time of inundation: Figure B2-G
- 1% AEP peak time of inundation: Figure B2-H
- 20% AEP, 10% AEP, 5% AEP, 2% AEP, 1% AEP, 1 in 2,000 AEP, 1 in 10,000 AEP and PMF Existing Case results: [Online Digital Portal](#)
- For higher resolution mapping refer to the [Online Digital Portal](#).

6.4.3 Existing Case flood inundation extent and flood levels

For the Existing Case 1% AEP event:

- The Westbrook Creek and Dry Creek floodplains are well defined with a few minor localised breakouts and small tributary drainage lines
- Under the 1% AEP event, the Project crosses Westbrook Creek, where the peak depth of water is approximately 4.5 m in the channel and up to 3 m on the floodplain
- At Dry Creek the 1% AEP event water depth in the channel is around 3.5 m deep and up to 1 m on the floodplain. The peak water levels at the proposed Westbrook and Dry creeks are 426.5 mAHD and 425.3 mAHD, respectively.

Figure B2-B in Volume 2 – Appendix B shows the indicative peak flood extent and water levels within the Westbrook and Dry floodplain for the Existing Case.

6.4.4 Flood immunity of existing infrastructure

Toowoomba-Cecil Plains Road is located upstream of the Project alignment. Table 6.20 presents a summary of overtopping depths for key roads near the Project alignment under a range of design events. Modelling results indicate that Toowoomba Wellcamp Airport is not flooded under a 1% AEP event.

Table 6.20 Westbrook and Dry creeks—Existing Case: Estimated overtopping depths of key infrastructure

Infrastructure	Location	Maximum estimated overtopping depth (m)							
		PMF	1 in 10,000 AEP	1 in 2,000 AEP	1% AEP	2% AEP	5% AEP	10% AEP	20% AEP
Anderson Road	D/S Westbrook Bridge	7.03	5.44	4.63	3.90	3.62	3.21	2.79	2.30
Brimblecombe Road	Dry Creek Crossing	1.74	1.18	0.98	0.79	0.73	0.63	0.52	0.43
Toowoomba-Cecil Plains Road	~300 m west of Westbrook Creek Crossing	2.44	1.65	1.1	0.75	0.67	0.60	0.45	0.22

6.4.5 Existing Case velocities

Peak Existing Case velocities for the 1% AEP event in the channels of Westbrook and Dry Creeks at the Project alignment are in the order of 2 to 3 m/s, and on the floodplain, velocities are generally in the order of 1 to 1.5 m/s.

Figure B2-D in Volume 2 – Appendix B shows the 1% AEP peak flood velocities within the Westbrook and Dry floodplain for the Existing Case.

6.4.6 Existing Case flood hazard

For the 1% AEP event, high hazard zones within the catchment occur within the main channels of Westbrook and Dry Creeks. Estimated peak velocity-depth product values of 10 m²/s and 7.5 m²/s are noted within the respective waterways at the proposed Project crossings.

Lower hazard zones are located on the floodplains with velocity-depth product values ranging between 0.1 m²/s to 0.6 m²/s.

Figure B2-F in Volume 2 – Appendix B shows the 1% AEP peak flood hazard within the Westbrook and Dry floodplains for the Existing Case.

6.4.7 Existing Case flood time of inundation

The flood duration of inundation is influenced by the critical duration selection, which has been assessed as between 90 min (1.5 hours) and 720 min (12 hours) for this study. Flood duration results are conservative at locations subject to standing water, such as along the main channel bed or at ground depressions, as ground infiltration and evaporation are not represented in the flood model.

For the Existing Case 1% AEP event the model demonstrates:

- Westbrook and Dry Creeks are inundated for approximately 8 to 12 hours
- The floodplains and breakout locations are typically inundated for approximately 2 to 4 hours.

Figure B2-H in Volume 2 – Appendix B shows the 1% AEP peak flood duration of inundation within the Westbrook and Dry floodplains for the Existing Case.

6.5 Developed Case modelling results—Westbrook and Dry creeks

6.5.1 Drainage structures

The hydraulic design of the major drainage structures was undertaken using the TUFLOW hydraulic model (1D and 2D approach).

On the Westbrook and Dry Creek floodplain, the Project includes the following floodplain (or regional structures):

- Two waterway bridges
- 10 RCP locations (a total of 94 cells)
- Two RCBC locations (a total of 13 cells)
- Two rail-over-road bridges that also convey flood flows in large events.

Local drainage structures presented in Appendix B were sized through the local drainage design and depending on their interaction with the Westbrook and Dry Creek floodplain were incorporated in the hydraulic model. For more details on the local culverts refer to Section 18.

Design bridge details are summarised in Table 6.21, and details of the proposed local and regional culvert structures are summarised in the Cross-Drainage Register located in Appendix B. The location of proposed cross-drainage features are presented in Figure B1-F in Volume 2 – Appendix B.

Table 6.21 Westbrook and Dry creeks—Proposed bridge locations and details

Chainage (km)	Structure ID	Approximate span (m)	Deck width (m)	Soffit level (mAHD)	Deck superstructure type	Deck depth (mm)
197.40	310-BR20	92	3.97	439.40	Type D1	2,000
198.50	310-BR31	230	3.97	427.70 ¹	Type D1	2,000
199.20	310-BR32	184	3.97	430.10 ¹	Type D1	2,000
200.00	310-BR33	75	3.97	437.95 ¹	Type D1	2,000+2,450

Table note:

- 1 Average bridge soffit level reported due to changing soffit level of bridges due to rail grade. The reported values may differ from that specified within the hydraulic model.

Bridges were represented within the TUFLOW model through use of LFC. Each bridge within the model has had a flow constriction coefficient applied to represent obstruction of waterway area due to the piers.

A blockage factor of 25 per cent was applied to all proposed culverts based on guidelines set out in ARR 2019. The adopted blockage factor for the proposed bridges was between 5 per cent and 10 per cent based on the waterway area blockage due to bridge piers.

6.5.2 Hydraulic design criteria outcomes

The hydraulic model was run for the Developed Case for the range of events (20%, 10%, 5%, 2%, 1% AEP events and 1 in 2,000 AEP, 1 in 10,000 AEP and PMF events) with the outcomes presented in the following sections.

6.5.2.1 Rail immunity and structures results

Appendix B summarises the peak 1% AEP event water levels on the upstream side of the Project alignment.

The results of flood modelling indicate that a 1% AEP event flood immunity to the Project formation level is achieved at all cross-drainage location within the Westbrook and Dry floodplains. There is over 0.9 m freeboard above the culvert obvert levels to the rail formation level in a 1% AEP event, noting that the formation levels were generally dictated by other engineering criteria and constraints. At the bridge locations, the 1% AEP peak water levels are below the bridge soffit levels.

6.5.3 Flood Impact Objectives outcomes—Westbrook and Dry Creeks

The impact of the Developed Case was assessed through inclusion of the proposed Project rail design and comparison of model results against the Existing Case results.

Changes to flooding behaviour that were assessed, and are reported in the following sections, include potential changes in peak water levels, time of inundation, velocity, hazard, and peak flows:

- Changes in 20% AEP and 1% AEP peak water levels are presented in Figures B4-A and B4-B in Volume 2 – Appendix B
- Changes in 20% AEP and 1% AEP velocities are presented in Figures B4-C and B4-D in Volume 2 – Appendix B
- Changes in 20% AEP and 1% AEP hazard are presented in Figures B4-E and B4-F in Volume 2 – Appendix B
- Changes in 20% AEP and 1% AEP time of inundation are presented in Figures B4-G and B4-H in Volume 2 – Appendix B
- Changes in 20% AEP and 1% AEP velocities and percentage change in velocities with cut-offs applied are presented in Figures B5-A to B5-F in Volume 2 – Appendix B
- Percentage changes in 20% AEP and 1% AEP in hazard are presented in Figures B6-A and B6-B in Volume 2 – Appendix B
- Percentage changes in 20% AEP and 1% AEP in time of inundation and with cut-offs applied are presented in Figures B7-A to B7-D in Volume 2 – Appendix B
- Refer to the [Online Digital Portal](#) for other modelled AEP results and higher resolution mapping.

One-on-one consultation with landowners who are expected to experience changes in flooding behaviour on their property was conducted by ARTC supported by FFJV. Refer to EIS Chapter 14: Flooding and Geomorphology and EIS Appendix E: Consultation Report for further details.

The Project design outcomes relative to the FIOs (Table 4.2) are presented in the following sections.

Flood sensitive receptors referred to in this section are shown in Figures P1 (1 to 59) in Volume 2 – Appendix P.

6.5.3.1 Flood impacts on flood sensitive receptors

Based on the available aerial imagery, no buildings or critical infrastructure are located within the area impacted by afflux and change in flood hazard in the Westbrook and Dry Creeks floodplain for all modelled events up to the 1 in 2,000 AEP event.

6.5.3.2 Flood impacts on State-controlled roads

Within the extent of the hydraulic model, the only State-controlled road that is influenced by flooding and the Project alignment is Toowoomba-Cecil Plains Road. The extent of the hydraulic model developed for Westbrook and Dry creeks and impacted State-controlled roads is shown in Figure 6.5. The following points provide a summary of key roads:

- The Existing Case model results indicate that Toowoomba-Cecil Plains Road has a very low existing flood immunity, with sections overtopping with small flood depths in the 20% AEP event. The impact of the Project has minimal impact on the overall flood immunity. The maximum afflux experienced on Toowoomba-Cecil Plains Road is 102 mm across a length of 27.5 m (road section ID #134), with a maximum change in time of inundation of 364 per cent, from 0.21 hours in the Existing Case to 1.01 hours in the Developed Case. It is noted that in the 1% AEP event, the Project does result in other small road segments being overtopped; however, it should be noted that the road is already submerged in other parts of the link with the time of inundation changing from 11.65 hours in the Existing Case to 11.67 hours in the Developed Case.

Table 6.22 details the State-controlled roads assessment relative to Project FIOs including justification and mitigation of exceedances.

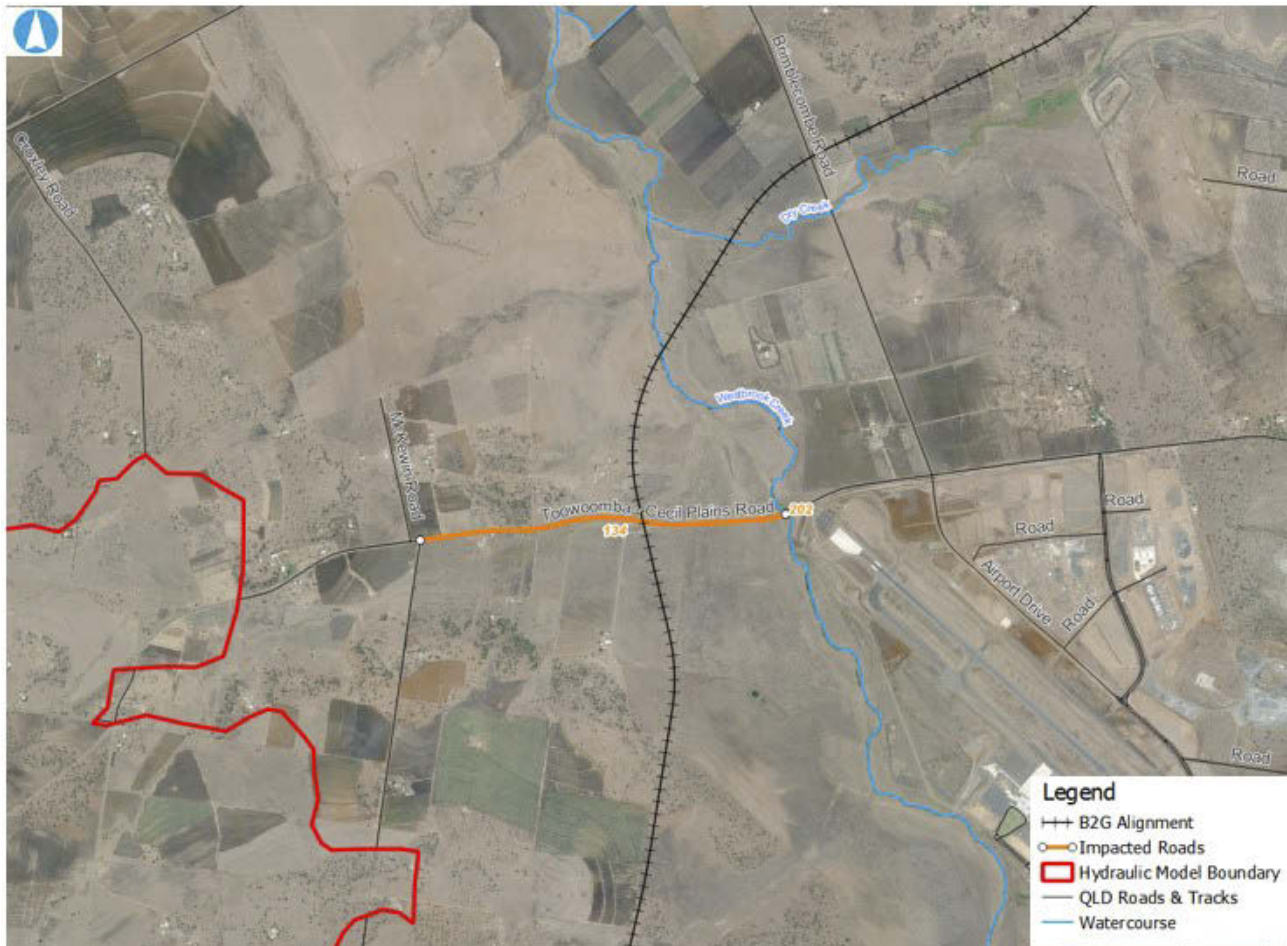


Figure 6.5 Westbrook and Dry creeks—Impacted State-controlled roads

Table 6.22 Westbrook and Dry Creek - Summary of flood impacts on State-Controlled roads

Road Object ID	Road Segment	Road Name	Surface Type	Flood Impact Objective Target					AEP	Maximum FIO Exceedance					Length (m)				Exceedance Justification	Proposed Mitigation Measures ²
				Change in Peak Water Level (mm)	Change in Time of Inundation (%)	Change in Velocity (%)	Change in Velocity (m/s)	Change in Hazard (%)		Change in Peak Water Level (mm)	Change in Time of Inundation (%) ¹	Change in Velocity (%) ¹	Change in Velocity (m/s)	Change in Hazard (%) ¹	Afflux	Time on Inundation	Velocity	Hazard		
134	16015103990000366753	Toowoomba - Cecil Plains Road	Sealed	10	10	20	1	10	20%	16	51	-	0.32	13	57.50	10.00	2.50	10.00	Exceedances experienced on this road segment are due to the representation of existing and design culvert structures and their interface with road upgrades within the hydraulic model. The main area of exceedance along this road segment occur directly under Bridge 310-BR20 where an upgrade to Toowoomba-Cecil Plains Road is proposed. In the existing case a culvert is present under the road draining north towards Westbrook Creek. This flowpath is impeded due to the presence of the proposed road upgrade which is causing fringe impacts on Toowoomba Cecil Plains Road. The Existing Case model results indicate that Toowoomba-Cecil Plains Road has a very low existing flood immunity, with sections overtopping with small flood depths in the 20% AEP event. The impact of the Project alignment has minimal impact on the overall flood immunity. It is noted that in the 1% AEP event, the Project alignment does result in other small road segments being overtopped; however, it should be noted that the road is already submerged in other parts of the link with the time of inundation changing from 11.65 hours in the Existing Case to 11.67 hours in the Developed Case. Exceedances remain on this road segment due to the maturity of the hydraulic model and the representation of existing and proposed road cross drainage infrastructure within the model.	In detailed design, model improvements are to be undertaken to better represent existing culvert structures and their interface with proposed road upgrades. Detailed survey will be obtained to better represent existing structures under Toowoomba-Cecil Plains Road. A review of the bridge abutment location adjacent the flowpath will also be undertaken to mitigate these fringe impacts on the road.
									10%	65	25	45	-	14	5.00	17.50	15.00	2.50		
									5%	-	-	32	-	-	-	-	15.00	-		
									2%	102	-	30	-	-	27.50	-	17.50	-		
									1%	83	373	22	-	-	52.50	27.50	7.50	-		
									0.05%	-	-	-	-	-	-	-	-	-		
202	16015103990000366754	Toowoomba - Cecil Plains Road	Sealed	10	10	20	1	10	20%	38	-	-	-	-	15.00	-	-	-	- Maturity of current cross-drainage design is limited by available information (e.g. detailed survey). - Maturity of current road design is limited by available information (e.g. detailed survey).	Further cross-drainage optimization investigations are required during detailed design. Further road and/or rail design optimization investigations are required during detailed design.
									10%	-	-	-	-	-	-	-	-	-		
									5%	-	-	-	-	-	-	-	-	-		
									2%	-	-	-	-	-	-	-	-	-		
									1%	-	-	-	-	-	-	-	-	-		
									0.05%	-	-	-	-	-	-	-	-	-		

Table Notes

1. Time of Submergence has been measured across the full length of the road segment and does not correspond to the exact location of exceedance within that segment.

2. Change in water level (afflux) is calculated based on the maximum water surface level difference between the existing and design case results, and does not consider the underlying topography. Depth values presented are based on the respective underlying topographies and in some cases do not align with afflux calculations. This can be attributed to interpolation calculations of results performed within the hydraulic model.

3. Percentage increases should be read in conjunction with existing and design case results. The FIOs require that change be presented in percentage change for time of inundation, velocity and hazard and can sometimes produce large increases when existing case results are significantly small. Some locations were identified to experience significant percentage increases however the "absolute" difference between Existing and Design results are relatively small.

4. The above table should be read in conjunction with change maps provided as part of Volume II and the Digital Platform.

5. The symbol "-" denotes that no change above Flood Impact Objectives has been determined and therefore no result has been presented.

6. Changes recorded above may only occur during a small fraction of model simulation time.

7. Three decimal places do not infer absolute accuracy in modelling results, but it is necessary to identify small changes.

8. Design optimisations that could be implemented (but not limited to) include adjustment of culvert spacing, realignment of culverts to existing flowpaths when more detailed survey is available, modifying the grade of culverts, road height and alignment adjustments, removal of bridge abutments from waterway extents etc

Depth (m) ³		Time of Inundation in hours (related to % exceedance)		Time of Submergence (hours) ²		Velocity in m/s (related to % exceedance)		Velocity in m/s (related to actual change exceedance)		Hazard in m/s (related to % exceedance)	
Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case
0.782	0.801	0.39	0.59	16.92	16.97	-	-	0.887	1.205	0.46	0.52
-	0.065	0.55	0.69	16.90	16.93	1.044	1.512	-	-	0.66	0.74
-	-	-	-	-	-	1.944	2.575	-	-	-	-
0.008	0.023	-	-	-	-	2.148	2.789	-	-	-	-
0.018	0.031	0.21	1.01	11.65	11.67	2.309	2.821	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
0.421	0.424	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-

6.5.3.3 Flood impacts on local public roads

Within the extent of the hydraulic model, two local public roads are influenced by flooding (Figure 6.6). Table 6.23 identifies the location of potential FIO exceedances and the details for each exceedance, including afflux, velocity, time of inundation and hazard. Justification and proposed mitigation measures are also detailed.

The following provides a summary of the key FIO exceedances:

- The maximum water level increase over the local roads is estimated at less than 40 mm for all modelled events up to the 1% AEP Event. This occurs at Brimblecombe Road (road section ID #46) during the 1% AEP flood event. In addition to afflux exceedances, Brimblecombe Road also exceeds flood hazard FIOs with an increase of 364 per cent from 0.05 m²/s in the Existing Case to 0.22 m²/s in the Developed Case.
- Anderson Road (ID #1) is subject to afflux of up to 22 mm during the 20% AEP event. The road is also subject to increases in the duration of inundation, registering an increase of up to 45 per cent for the same flood AEP event, from 0.74 hours in the Existing Case to 1.07 hours in the Developed Case.
- Exceedances on local roads is due to the Project's design constraints altering the existing flow regime (i.e. the corridor crossing Westbrook and Dry Creeks), concentration of runoff through culverts and bridge openings and the current maturity of cross-drainage design.
- During detailed design, proposed mitigation measures including site-specific treatments, and additional cross-drainage optimisation investigations are likely to be undertaken. Stakeholder consultation is also likely to inform potential mitigation options.

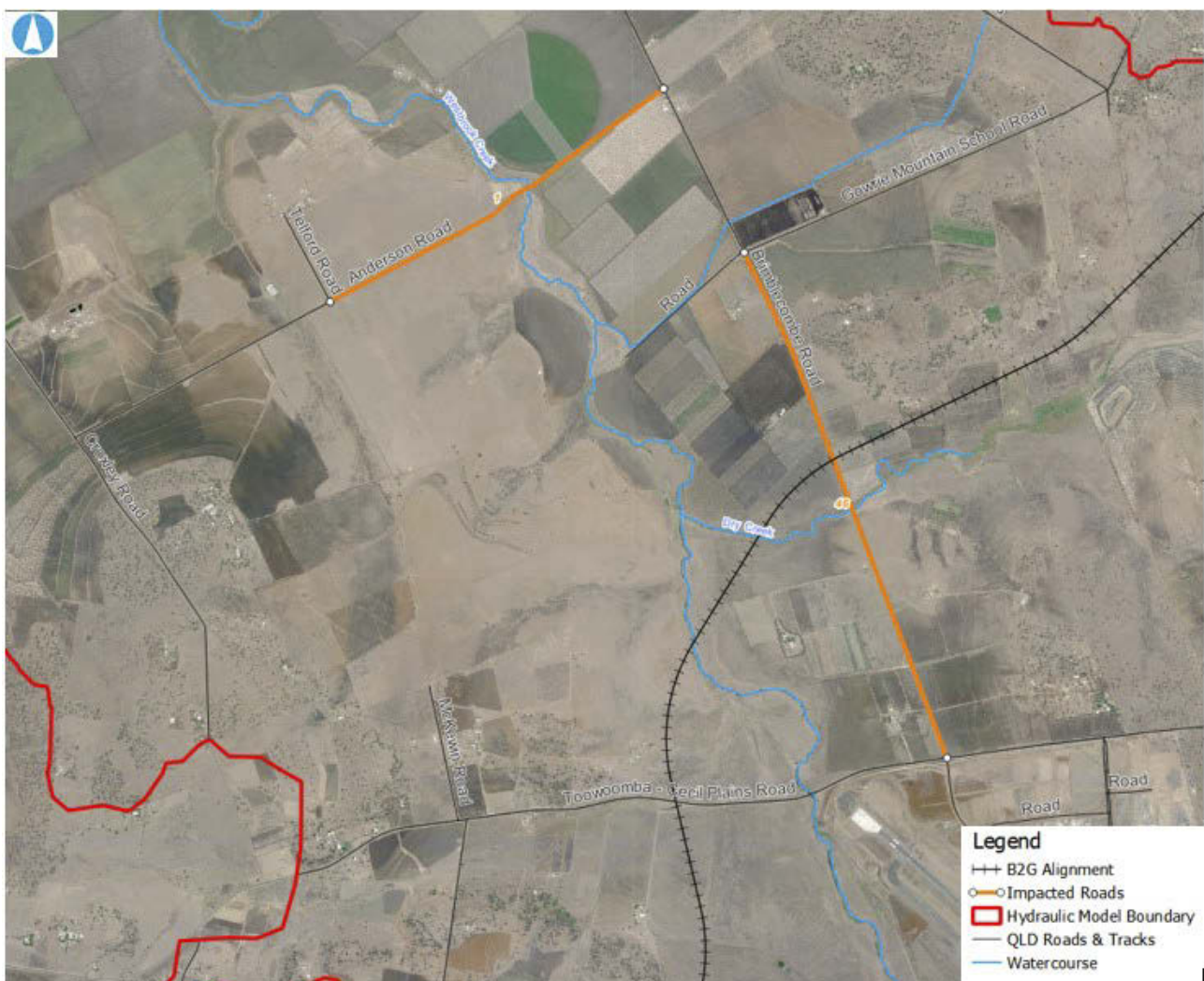


Figure 6.6 Westbrook and Dry creeks—Impacted local public roads

6.5.3.4 Flood impacts on existing rail infrastructure

There is no third-party rail infrastructure located within the Westbrook and Dry creeks hydraulic model boundary.

6.5.3.5 Flood impacts on private land outside the permanent footprint

Table 6.24 presents a list of private properties located outside the permanent footprint with potential FIO exceedances due to the potential rail impact. A summary of key impacts, exceedance justification and mitigation measures is provided in the below points, with further detail provided in Table 6.24:

- A total of 35 private lots were flagged as having a potential impact that exceed the Project FIO criteria. The majority of recorded exceedances were identified on land-use types 'Cattle Breeding and Fattening' and 'Grains'.
- Change in time of inundation is the primary exceedance category for the Westbrook and Dry catchments. A total of 33 properties have been identified. Review of the exceedances show increases are mostly isolated to regions directly upstream and downstream of the permanent footprint. Exceedances can be attributed to ponding that is created by the alignment crossing a natural waterway, and the localised redirection of flow through culvert structures creating a concentrated flow path downstream. The maximum increase in time of flood inundation is approximately 13.38 hours on Lot Plan 2AG3200, with the total impacted area being 0.17 ha (less than 1 per cent of total lot area). It should be noted that the above change is the peak only and does not necessarily occur across the whole impacted area.
- Potential afflux exceedances were identified for six properties. Afflux has been identified at cross-drainage inlets, and approximately 1.2 km upstream of the permanent footprint along Westbrook Creek. The maximum increase in afflux is approximately 0.72 m on Lot Plan 2RP149961, with the total impacted area being 0.71 ha (less than 1 per cent of total lot area). This area of afflux is due to a change in inundation extent and impacts a newly flood-affected cell. It should be noted that most large increases are localised and don't translate to uniform flood level increases across the lot.
- A total of 21 properties were identified as having potential velocity exceedances. Impacted properties are located at the outlets of cross-drainage structures, and within Westbrook Creek. The maximum velocity increase is approximately 2.6 m/s on Lot Plan 1RP155674.
- Increases in flood hazard are noted for nine properties that are predominantly located in the immediate vicinity of the permanent footprint. Exceedances can be attributed to increased flow velocities and flood depths for the inlets and outlets of bridges and culverts. The maximum increase for flood hazard is approximately 0.34 m²/s on Lot Plan 10RP47857.

Potential mitigation measures include further cross-drainage optimisation investigations and ongoing stakeholder communication.

Table 6.24 Westbrook and Dry Creek - Summary of flood impacts on private land outside the permanent footprint

ID	ID (Lotplan ID)*FSR ID/ Culvert ID)	Sensitive Receptor Type	Catchment Type	Landuse ¹	Lot Area (m2)	Flood Impact Objective Target						AEP	Maximum FIO Exceedance						Area (ha) ²				Exceedance Justification	Proposed Mitigation Measures ¹⁰	
						Change in Peak Water Level (mm)	Change in Time of Inundation (%)	Change in Time of Inundation (Hours)	Change in Velocity (%)	Change in Velocity (m/s)	Change in Hazard (%) ³		Change in Peak Water Level (mm)	Change in Time of Inundation (%) ³	Change in Time of Inundation (Hours)	Change in Velocity (%) ³	Change in Velocity (m/s)	Change in Hazard (%) ³	Afflux	Time on Inundation	Velocity	Hazard			
1	2AG3200	Lot Type Parcel	Regional	GRANIS	841,252	200	10	1	10	0.5	20	20%	79	15	12.93	11	1.34	-	0.02	0.04	0.151	-	The Flood Impact Objective exceedance has been identified due to a change in flood extent and a cell which was dry in the existing case and now flood-affected in the developed case. The FIOs require that change be presented in percentage change for time of inundation, velocity and hazard and can sometimes produce large increases when existing case results are significantly small. Some locations were identified to experience significant percentage increases however the "absolute" difference between Existing and Design results are relatively small.	The property parcel has been acquired by ARTC.	
												10%	278	412	11.30	15	1.22	38440	0.07	0.17	0.08	0.04			
												5%	375	259	13.38	96	1.34	185401	0.28	0.17	0.34	0.27			
												2%	369	440	9.93	138	1.39	130597	0.36	0.36	0.47	0.44			
												1%	345	450	10.62	161	1.56	46735	0.41	0.59	0.59	0.55			
												-	-	-	-	-	-	-	-	-	-	-			
												-	-	-	-	-	-	-	-	-	-	-			
2	10RP47857	Lot Type Parcel	Regional	CATTLE BREEDING & FATTENING	509,248	200	10	1	10	0.5	20	20%	454	363	7.69	108	1.06	461	0.10	0.24	0.16	0.16	The exceedances experienced on this Lot are due to the concentration of runoff through the proposed culvert structures. In the existing case, water ponds in a dam upstream of the alignment and is allowed to overtop into a downstream floodpath in all modelled events from the 20% AEP to the 1% AEP events. In the design case, the proposed rail embankment is to be constructed over the existing dam highway with 2 cross drainage structures proposed under the embankment to provide a floodpath and protect the immunity of the alignment. The introduction of these cross drainage structures results in increased conveyance downstream from the existing case and subsequently afflux, time of inundation, velocity and hazard increases. A comparison between the existing and design case floodpaths shows that there is some increase in flood extent although this is generally confined to an existing channel across this Lot.	In detailed design, a review of the interaction between the rail embankment, existing dam and proposed drainage structures will be undertaken to mitigate impacts. Ongoing consultation with the upstream landowner (with the existing farm dam on their land) will also be undertaken to understand the long term need associated with the dam. The review will require a balance to be obtained between the proposed design, yield from the existing farm dam, rail line immunity and impacts on this Lot. Due to this interface, it is anticipated some impact on the Lot will remain post optimisation of the design and landowner consultation will occur to agree any site specific mitigation measures.	
												10%	407	387	6.74	112	1.11	639	0.10	0.27	0.17	0.16			
												5%	444	355	6.01	119	1.72	1019	0.17	0.29	0.19	0.18			
												2%	510	398	5.13	105	1.86	1119	0.18	0.31	0.20	0.19			
												1%	554	421	5.58	131	2.01	995	0.19	0.31	0.21	0.20			
												-	-	-	-	-	-	-	-	-	-	-			
												-	-	-	-	-	-	-	-	-	-	-			
3	14RP24627	Lot Type Parcel	Regional	DWELLING - LARGE HOUSESITE	181,981	10	5	1	10	0.5	10	20%	78	-	-	-	-	0.01	-	-	-	-	Maturity of current road design is limited by available information (e.g. detailed survey)	Further road and or rail design optimization investigations are required during detailed design.	
												10%	-	-	-	-	-	-	-	-	-	-			-
												5%	-	-	-	-	-	-	-	-	-	-			-
												2%	-	-	-	-	-	-	-	-	-	-			-
												1%	-	-	-	-	-	-	-	-	-	-			-
												-	-	-	-	-	-	-	-	-	-	-			-
												-	-	-	-	-	-	-	-	-	-	-			-
4	2RP419961	Lot Type Parcel	Regional	CATTLE FATTENING	536,299	200	10	1	10	0.5	20	20%	437	989	12.20	-	0.51	-	0.19	0.95	0.03	-	This Lot is split by the proposed alignment. In the existing case, flood waters sheet flow across the land north east towards Westbrook Creek. The proposed rail embankment intercepts this flow creating ponding on the upstream side and directing flow along the toe of the embankment to proposed cross drainage structures C196.92, C197.18 and C197.31 and bridge (300-BR20). The current placement and grouping of the proposed culverts leads to a concentration of flow resulting in downstream velocity, hazard and time of inundation increases and upstream afflux impacts due to ponding and change in flood extent. This is particularly prevalent around culvert C196.92 where 14 cells of 1.25m RCPs are proposed.	It is envisaged a revision of the proposed cross drainage in detailed design at this location, in particular C196.92, to spread the culverts cells across the embankment will significantly reduce the impact on this Lot. Distributing the flow to replicate the existing flow regime will minimise a change in flood extent as well as concentration of flow through only a few locations.	
												10%	478	882	10.65	-	0.53	126	0.25	1.01	0.03	0.00			
												5%	587	823	12.62	34	0.70	1494	0.43	1.39	0.09	0.04			
												2%	663	675	9.31	34	0.72	953	0.58	1.59	0.10	0.05			
												1%	723	687	10.01	29	0.76	758	0.71	1.84	0.13	0.08			
												-	-	-	-	-	-	-	-	-	-	-			
												-	-	-	-	-	-	-	-	-	-	-			
5	2RP82906	Lot Type Parcel	Regional	VACANT - LARGE HOUSESITE	183,999	10	5	1	0	0.0	0	20%	16	-	-	-	-	0.003	-	-	-	-	Maturity of current bridge design is limited by available information (e.g. detailed survey)	Further cross-drainage optimization investigations are required during detailed design.	
												10%	-	54	-	-	-	-	-	0.003	-	-			
												5%	-	-	-	-	-	-	-	-	-	-			
												2%	-	-	-	-	-	-	-	-	-	-			
												1%	-	-	-	-	-	-	-	-	-	-			
												-	-	-	-	-	-	-	-	-	-	-			
												-	-	-	-	-	-	-	-	-	-	-			
6	2RP94706	Lot Type Parcel	Regional	GRANIS	899,056	200	10	1	10	0.5	20	20%	-	474	0.23	-	0.06	-	0.08	0.06	-	-	Maturity of current road design is limited by available information (e.g. detailed survey)	Further road and or rail design optimization investigations are required during detailed design.	
												10%	-	76	4.14	-	1.41	-	0.03	0.04	-	-			
												5%	-	-	0.05	-	-	-	0.003	-	-				
												2%	-	-	-	-	-	-	-	-	-				
												1%	-	13	-	-	-	-	0.003	-	-				
												-	-	-	-	-	-	-	-	-	-				
												-	-	-	-	-	-	-	-	-	-				
7	31RP24805	Lot Type Parcel	Regional	GRANIS	750,356	200	10	1	10	0.5	20	20%	-	17	0.20	-	0.02	-	0.02	0.03	-	-	Maturity of current road design is limited by available information (e.g. detailed survey)	Further road and or rail design optimization investigations are required during detailed design.	
												10%	-	16	-	-	0.01	-	0.004	0.01	-	-			
												5%	-	-	-	-	-	-	-	-	-	-			
												2%	-	-	-	-	-	-	-	-	-	-			
												1%	-	-	-	-	-	-	-	-	-	-			
												-	-	-	-	-	-	-	-	-	-	-			
												-	-	-	-	-	-	-	-	-	-	-			
8	2RP49965	Lot Type Parcel	Regional	CATTLE FATTENING	755,012	200	10	1	10	0.5	20	20%	-	586	6.57	-	0.53	-	8.17	0.32	-	-	Maturity of current bridge design is limited by available information (e.g. detailed survey)	Further cross-drainage optimization investigations are required during detailed design.	
												10%	-	469	6.10	-	0.57	-	5.70	0.10	-	-			
												5%	-	409	5.42	32	0.30	-	4.71	0.12	-	-			
												2%	-	426	3.11	34	0.74	25	3.84	0.26	0.01	-			
												1%	-	316	-	28	0.80	-	3.40	0.32	-	-			
												-	-	-	-	-	-	-	-	-	-	-			
												-	-	-	-	-	-	-	-	-	-	-			
9	2RP48192	Lot Type Parcel	Regional	GRANIS	260,731	200	10	1	10	0.5	20	20%	-	88	0.36	-	0.04	37	-	0.28	0.04	0.07	Maturity of current bridge design is limited by available information (e.g. detailed survey)	Further cross-drainage optimization investigations are required during detailed design.	
												10%	-	177	2.66	14	0.02	28	0.42	0.02	0.04	0.04			
												5%	-	168	2.44	17	0.66	41	-	0.58	0.14	0.05			
												2%	-	149	1.82	23	0.02	185	-	1.12	0.34	0.27			
												1%	-	124	1.52	27	0.03	158	-	1.50	0.69	0.54			
												-	-	-	-	-	-	-	-	-	-	-			
												-	-	-	-	-	-	-	-	-	-	-			
10	14RP24607	Lot Type Parcel	Regional	GRANIS	713,224	200	10	1	10	0.5	20	20%	-	23	0.49	-	-	-	0.02	-	-	-	Maturity of current bridge design is limited by available information (e.g. detailed survey)	Further cross-drainage optimization investigations are required during detailed design.	
												10%	-	131	1.36	-	-	-	-	0.04	-	-			
												5%	-	143	1.51	-	0.06	-	-	0.05	0.06	-			
												2%	-	96	1.40	17	0.18	95	-	0.22	0.21	0.36			
												1%	-	86	1.15	24	0.21	98	-	0.30	0.28	0.48			
												-	-	-	-	-	-	-	-	-	-	-			
												-	-	-	-	-	-	-	-	-	-	-			
11	2RP24615	Lot Type Parcel	Regional	GRANIS	88,168	200	10	1	10	0.5	20	20%	-	15	0.15	-	0.01	-	0.01	0.00	-	-	Maturity of current bridge design is limited by available information (e.g. detailed survey)	Further cross-drainage optimization investigations are required during detailed design.	
												10%	-	19	-	-	-	-	-	0.01	-	-			
												5%	-	-	-	-	-	-	-	-	-	-			
												2%	-	-	-	-	-	-	-	-	-	-			
												1%	-	-	-	-	-	-	-	-	-	-			
												-	-	-	-	-	-	-	-	-	-	-			
												-	-	-	-	-	-	-	-	-	-	-			
12	7RP36554	Lot Type Parcel	Regional	GRANIS	757,373	200	10	1	10	0.5	20	20%	-	16	0.74	-	0.01	-	0.01	0.02	-	-	Maturity of current bridge design is limited by available information (e.g. detailed survey)	Further cross-drainage optimization investigations are required during detailed design.	
												10%	-	-	-	-	-	-	-	-	-	-			
												5%	-	-	-	-	-	-	-	-	-	-			
												2%	-	-	-	-	-	-	-	-	-	-			
												1%	-	-	-	-	-	-	-	-	-	-			
												-	-	-	-	-	-	-	-	-	-	-			
												-	-	-	-	-	-	-	-	-	-	-			
13	8RP36554	Lot Type Parcel	Regional	GRANIS	557,011	200	10	1	10	0.5	20	20%	-	15	0.20	-	0.05	-	0.03	0.06	-	-	Maturity of current bridge design is limited by available information (e.g. detailed survey)	Further cross-drainage optimization investigations are required during detailed design.	
												10%	-	16	-	-	0.01	-	-	0.01	0.00	-			-
												5%	-	-	-	-	-	-	-	-	-	-			
												2%	-	-	-	-	-	-	-	-	-	-			
												1%	-	-	-	-	-	-	-	-	-	-			
												-	-	-	-	-	-	-	-	-	-	-			
												-	-	-	-	-	-	-	-	-	-	-			
14	42RP24623	Lot Type Parcel	Regional	MLK-NO QUOTA	264,479	200	10	1	10	0.5	20	20%	-	47	-	-	-	-	0.003	-	-	-	Maturity of current road design is limited by available information (e.g. detailed survey)	Further road and or rail design optimization investigations are required during detailed design.	
												10%	-	25	-	-	-	-	-	-	-	-			
												5%	-	-	-	-	-	-	-	-	-	-			
												2%	-	-	-	-	-	-	-	-	-	-			
												1%	-	-	-	-	-	-	-	-	-	-			
												-	-	-	-	-	-	-	-	-	-	-			
												-	-	-	-	-	-	-	-	-	-	-			
15	42RP24623	Lot Type Parcel	Regional	GRANIS	264,479	200	10	1	10	0.5	20	20%	-	47	-	-	-	-	0.003	-	-	-	Maturity of current road design is limited by available information (e.g. detailed survey)	Further road and or rail design optimization investigations are required during detailed design.	
												10%	-	25	-	-	-	-	-	-	-	-			
												5%	-	-	-	-	-	-	-	-	-	-			
												2%	-	-	-	-	-	-	-	-	-	-			
												1%	-	-	-	-	-	-	-	-	-	-			
												-	-	-	-	-	-	-	-	-	-	-			
												-	-	-	-	-	-	-	-	-	-	-			
16	45RP24605	Lot Type Parcel	Regional	VACANT RURAL LAND (EXCL 01 & 04)	20,236	200	10	1	10	0.5	20	20%	-	16	0.19	-	-	-	0.01	-	-	-	Maturity of current bridge design is limited by available information (e.g. detailed survey)	Further cross-drainage optimization investigations are required during detailed design.	
												10%	-	-	-	-	-	-	-	-	-	-			
												5%	-	-	-	-	-	-	-	-	-	-			
												2%	-	-	-	-	-	-	-	-	-	-			
												1%	-	-	-	-	-	-	-	-	-	-			
												-	-	-	-	-	-	-	-	-	-	-			
												-	-	-	-	-	-	-	-	-	-	-			
17	2RP61982	Lot Type Parcel	Regional	CATTLE FATTENING	4,275,799	200	10	1	10	0.5	20	20%	-	16	0.22	-	0.02	-	0.04	0.11	-	-	Maturity of current bridge design is limited by available information (e.g. detailed survey)	Further cross-drainage optimization investigations are required during detailed design.	
												10%	-	18	0.17	-	0.81	-	-	0.02	0.004	-			-
												5%	-	-	0.06	-	-	-	-	0.003	-	-			
												2%	-	-	-	-	-	-	-	-	-	-			
												1%	-	-	-	-	-	-	-	-	-	-			
												-	-	-	-	-	-	-	-	-	-	-			
												-	-	-	-	-	-	-	-	-	-	-			
18	1RP155674																								

Table Notes:

- Floodplains are dynamic in nature and therefore model results vary from cell to cell. The "Area" is therefore defined as the maximum area over which change above the Flood Impact Objective occurs and does not relate directly to the maximum FIO exceedance recorded.
- The water level (ft/lm) is calculated based on the maximum water surface level difference between existing topography and design case results, and does not consider the underlying topography. Depth values presented are based on the resulting topographies and in some cases do not align with afflux calculations. This can be attributed to interpolation calculations of results performed within the hydraulic model.
- Percentage increases should be read in conjunction with existing and design case results. The FIOs require that change be presented in percentage change for time of inundation, velocity and hazard and can sometimes produce large increases when existing case results are significantly small. Some locations were identified to experience significant percentage increases however the "absolute" difference between Existing and Design results are relatively small.
- The Flood Impact Objective exceedance has been identified due to a change in flood extent and a cell which was dry in the existing case and now flood-affected in the developed case.
- The above table should be read in conjunction with change maps provided as part of Volume II and the Digital Platform.
- The symbol "--" denotes that no change above Flood Impact Objectives has been determined and therefore no result has been presented.
- Changes recorded above may only occur during a small fraction of model simulation time.
- Three decimal places do not infer absolute accuracy in modelling results, but it is necessary to identify small changes.
- Flood impact objectives are assessed based on landuse applied to the entire lot and exceedances identified do not necessarily indicate an impact to a dwelling.
- Design optimisations that could be implemented (but not limited to) include adjustment of culvert spacing, to existing flow depths, or to existing road cross-sections. More detailed studies is available, modifying the grade of culverts, road height and alignment adjustments, removal of bridge abutments from waterway extents etc
- Landuse data, dated 2018, has been used for this assessment. Due to the changing nature of this dataset, this will be updated to the latest available information in Detailed Design.

[illegible]

6.5.3.6 Flow distribution

A key landowner concern is the potential change in flow distribution. To determine the change in flow distribution, maximum velocity-depth modelling results were used to produce change in unit flow maps between Existing and Developed Case conditions. Unit flow is a measure of discharge that is commonly used for assessing peak flow along cross-sections, or to identify the efficient flow corridors within a floodplain.

The presented flow distribution maps were filtered with a velocity-depth product cut-off of 0.15 m²/s. The cut-off is in line with FIO requirements and was used to identify main channels (high-flow zones), and shallow lower flood zones (i.e. outer extents of the flood surface).

The following points summarise the change in flow distribution for Westbrook and Dry creeks and should be considered in conjunction with flow distribution figures supplied in Figure Q2-1 to Figure Q2-8 of Volume 2 – Appendix Q.

- Minor flow decreases (upstream) and increases (downstream) are localised to cross-drainage infrastructure. This behaviour is expected due to the constriction of the floodway through the culvert and bridge openings. This is particularly evident for all flood events up to the 1% AEP event, where the Project alignment is not overtopped.
- While the Project does impact flow conditions immediately upstream and downstream of the alignment, regional conditions remain relatively unchanged up to the 1% AEP event.
- During the rarer events (1 in 2,000 AEP, 1 in 10,000 AEP and PMF events), there is generally a positive increase in flow distribution immediately adjacent to the Project alignment, with a negative change throughout the majority of the remainder of the catchment due to the presence of the alignment.

Ongoing mitigation measures to reduce changes in flow distribution are likely to occur during future design stages of the Project, and may include further refinement of the bridge abutments and additional survey where necessary.

6.5.4 Extreme events

6.5.4.1 Flood immunity for extreme events

The risk of overtopping of the Project alignment was assessed for a range of extreme events (1 in 2,000 AEP, 1 in 10,000 AEP and PMF events), with Table 6.25 presenting the depth of water above the rail formation level and over the top of rail at each structure location. It is noted that the function of the floodplain culverts is to balance flood levels on the upstream and downstream sides of the Project alignment. As such, overtopping of the rail is not predicted to result in significant excessive flows or velocities as would occur in a dam embankment overtopping scenario. Further analysis of the overtopping risk to FSRs can be found in Section 21.

Table 6.25 Westbrook and Dry creeks—Extreme events: Estimated depth of water above formation and top of rail levels

Chainage (km)	Structure ID	Depth of water above formation level (m)			Depth of water over top of rail (m)		
		1 in 2,000 AEP	1 in 10,000 AEP	PMF	1 in 2,000 AEP	1 in 10,000 AEP	PMF
189.99	C189.99	-	-	-	-	-	-
193.10	C193.10	-	-	-	-	-	-
194.66	C194.66	0.27	0.60	1.04	-	-	0.34
194.69	C194.69	0.46	0.86	1.30	-	0.16	0.60
196.92	C196.92	-	-	-	-	-	-
197.18	C197.18	-	-	-	-	-	-

Chainage (km)	Structure ID	Depth of water above formation level (m)			Depth of water over top of rail (m)		
		1 in 2,000 AEP	1 in 10,000 AEP	PMF	1 in 2,000 AEP	1 in 10,000 AEP	PMF
197.31	C197.31	-	-	-	-	-	-
197.4	310-BR20	-	-	-	-	-	-
198.5	310-BR31	0.77	1.80	4.01	0.07	1.10	3.31
198.68	C198.68	-	0.47	2.21	-	-	1.51
198.77	C198.77	-	0.74	2.40	-	0.04	1.70
198.80	C198.80	-	0.35	1.76	-	-	1.06
198.98	C198.98	-	-	0.76	-	-	0.06
199.20	310-BR32	-	-	0.42	-	-	-
199.53	C199.53	-	-	-	-	-	-
200.00	310-BR33	-	-	-	-	-	-

6.5.4.2 Impacts during extreme events

Flood immunity of FSRs is discussed in Section 6.5.3.1, and maps demonstrating the impacts during extreme events are shown on the [Online Digital Portal](#).

In accordance with the FIOs, FSRs are considered impacted by extreme flood events if the change in water level is greater than 250 mm in the 1 in 2,000 AEP event, subject to the receptor classification (i.e. at existing habitable dwellings and/or commercial and industrial buildings/premises). In addition, a stricter afflux objective of 10 mm in the 1 in 500 AEP event applies to sites with existing sensitive infrastructure, uses involving vulnerable people and/or any critical road network that was designed to be immune to flooding in any extreme event.

Review of the 1 in 2,000 AEP event results for the Westbrook and Dry creeks floodplain against the FIOs show that flood level changes at all FSRs were limited to 250 mm, and no identified existing sensitive infrastructure or uses involving vulnerable people are expected to experience a more than 10 mm increase in flood level. A separate assessment of the 1 in 500 AEP event is not required as they are conservatively represented by the 1 in 2,000 AEP event results. The flood impacts in extreme events due to the Project are therefore compliant against the stipulated FIO limits for extreme events.

Table 6.26 details extreme event results, although it should be noted that FIO targets do not apply to the 1 in 10,000 AEP and PMF events.

Table 6.26 Westbrook and Dry creeks catchment—Extreme events: Impacts at flood sensitive receptors

FSR ID	Receptor type	1 in 2,000 AEP event ¹		1 in 10,000 AEP event		PMF event	
		Change in peak water level (mm)	Existing Case flood depth (m)	Change in peak water level (mm)	Existing Case flood depth (m)	Change in peak water level (mm)	Existing Case flood depth (m)
WES_ID_1	Shed	-	0.50	+98	1.16	+373	3.00
WES_ID_2	House	-	0.26	+184	0.95	+445	2.87
WES_ID_3	Shed	-	0.30	+166	0.97	+431	2.88

Table note:

¹ Note only those FSRs impacted are reported

6.5.5 Sensitivity analysis—Westbrook and Dry creeks

The sensitivity of the model to various parameters was assessed using the following two scenarios:

- An increase in rainfall intensity (i.e. to reflect climate change scenario)
- An increase and decrease in blockage of culverts from 25 per cent to 50 per cent and 0 per cent.

6.5.5.1 Climate change

The potential impacts on FSR locations were assessed for the 1% AEP climate change event to determine the Project's long-term hydraulic performance and sensitivity to changes in climate. The assessment was undertaken in accordance with ARR 2019.

The Representative Concentration Pathways 8.5 (2090 horizon) climate change scenario was adopted for the Project with an associated increase in rainfall intensity of 21 per cent across the catchment area.

The Project is expected to retain a 1% AEP flood immunity, with approximately 0.9 m of freeboard to formation level under the climate change scenario.

The results of the analysis show there are no changes to peak water levels at FSRs for the 1% AEP climate change event, as shown in Table 6.27. In addition to Table 6.27, peak water levels for the climate change scenario are presented in Figure B8-C in Volume 2 – Appendix B.

Table 6.27 Westbrook and Dry creeks—1% AEP event with Representative Concentration Pathways 8.5 conditions Climate Change assessment: flood sensitive receptor impacts

FSR ID ¹	FSR type	1% AEP climate change event	
		Change in peak water level (mm)	Existing Case flood depth (m)
WES_ID_1	Shed	-	0.12
WES_ID_4	Shed	-	0.02
WES_ID_17	House	-	0.16
WES_ID_18	Shed	-	0.16
WES_ID_19	Shed	-	0.09
WES_ID_21	House	-	0.28
WES_ID_22	Shed	-	0.19
WES_ID_23	Shed	-	0.14
WES_ID_24	Shed	-	0.12
WES_ID_26	Shed	-	0.13
WES_ID_27	Shed	-	0.17
WES_ID_31	Shed	-	0.02
WES_ID_42	Private Access Road	-	0.21
WES_ID_61	Shed	-	0.15

Table note:

1 Only FSR locations that recorded a flood depth during pre-Project climate change conditions, and post climate change conditions have been recorded in the above table.

6.5.5.2 Blockage

Blockage was assessed in accordance with ARR 2019. The blockage assessment resulted in a blockage factor of 25 per cent being adopted for culverts, along with a minimum culvert size of 900 mm diameter to reduce potential for blockage and maintenance.

A significant community concern is the potential impacts to flood conditions should the proposed culverts become blocked with debris. The primary concern is the blockage of culverts, which is likely to drive flood levels higher, particularly upstream of the culverts, and divert more flow through residences, across access roads and other infrastructure. A sensitivity analysis was undertaken with 0 per cent and 50 per cent blockage on culverts.

In addition, the culvert blockage sensitivity assessment results indicated that FSRs have not been impacted by blockage, as detailed in Table 6.28.

During detailed design the blockage factors will be reviewed in line with the final design and local catchment conditions. This may result in a varied and/or lower blockage factors being applied along the Project alignment. Should impacts be determined due to a change in blockage factor, additional mitigation measures will be considered in detailed design.

Table 6.28 Westbrook and Dry creeks—1% AEP event culvert blockage assessment

FSR ID	FSR type	Existing Case flood depth (m)	1% AEP Afflux (mm)		
			0% blockage	Developed Case (25% blockage)	50% blockage
WES_ID_1	Shed	0.12	-	-	-
WES_ID_4	Shed	0.03	-	-	-
WES_ID_17	House	0.16	-	-	-0.01
WES_ID_18	Shed	0.16	-	-	-
WES_ID_19	Shed	0.09	-	-	-
WES_ID_21	House	0.28	-	-	-0.01
WES_ID_22	Shed	0.19	-	-	-
WES_ID_23	Shed	0.14	-	-	-
WES_ID_24	Shed	0.12	-	-	-
WES_ID_26	Shed	0.13	-	-	-
WES_ID_27	Shed	0.17	-	-	-
WES_ID_31	Shed	0.02	-	-	-
WES_ID_42	Private Access Road	0.22	-	-	-
WES_ID_61	Shed	0.15	-	-	-

6.6 Additional modelling items to be considered in detailed design

Detailed design provides the opportunity to incorporate further detail in the modelling and using the improved detail to assess the elements of the design further.

Items to be incorporated into the detailed design for this section of the Project are:

- Blockage factors will be reviewed in line with the final design and local catchment conditions
- Undertake additional model strengthening by improving the representation of cross-drainage infrastructure, third-party infrastructure, noise barriers, survey data, and bridge structures within the hydraulic model
- The Westbrook and Dry Creek catchments have a number of new developments, with development approval, that are likely to affect local hydrology and floodplain behaviour before the Project is constructed. Developments for inclusion in the updated flood models will be discussed and agreed with the applicable authority.

Further details of mitigation measures can be found in Section 22. A review of proposed mitigation measures will be completed during the detailed design stage of the Project after hydraulic models have been updated to capture the above changes.

6.7 Planned and future roads

Consultation with DTMR has not identified any planned or future road upgrades within the Westbrook and Dry creeks catchment. ARTC will continue to consult and engage with DTMR to determine any future or planned road projects that may need to be considered as part of the detailed design hydraulic modelling.

TRC identified rehabilitation works being undertaken on Louttit Court at Gowrie Mountain. A review of this location has identified that these works are upstream of the Project alignment and outside the 1% AEP extents. No impact on the hydraulic modelling is anticipated for this road rehabilitation work and therefore it has not been included within hydraulic modelling. ARTC will continue to consult and engage with TRC to determine any future or planned road projects that may need to be considered as part of the detailed design hydraulic modelling.

7 Condamine River

The Condamine River drains the northern portion of the Darling Downs and forms part of the Balonne River catchment that, in turn, drains into the Murray-Darling River basin. The river rises on the western slopes of the Great Dividing Range and drains a total catchment area of approximately 14,000 km².

The upper Condamine catchments rise in elevated country around Killarney, with elevations up to 1,400 m above sea level, but about two-thirds of the catchment is flat floodplain country where elevations are around 100 to 200 m above sea level. The lower part of the catchment consists of a complex system of rivers and creeks. The eastern part of the catchment has an annual average rainfall of 600 to 800 mm and the floodplains of the south-west have an annual average rainfall of 300 to 500 mm.

Upstream of the Project alignment, between Millmerran and Pittsworth where the proposed rail crosses the Condamine River floodplain, the river breaks out around the area of Tummaville into a braided and intricate system covering an area around 13 km wide. The floodplain is formed by three main river branches: the Condamine River North Branch, main Condamine River and a southern branch known as Grasstree Creek.

On the Condamine River floodplain, the terrain is flat and the sinuous creek channels begin to break their banks in a 50% AEP event, and then flow between branches in 20% AEP and larger events. Due to the minimal slope throughout the majority of the floodplain, flooding in this area is typically characterised by slow-moving flood waters. The Existing Case 1% AEP event inundated width along the Project alignment between Grasstree Creek and Brookstead is approximately 12.5 km. Along the Gore Highway, the 1% AEP floodplain length can exceed 20 km when Back Creek flooding is combined with Condamine River flooding.

Under existing conditions, there are multiple pieces of infrastructure impacted by flooding, including the existing QR Rail Line, multiple State-controlled roads, and various structures, including houses and sheds. The existing State-controlled roads within the floodplain, which includes the Gore Highway, Millmerran Leyburn Road and Pampas-Horrane Road, all have low flood immunity.

Under the 1% AEP event, the Existing Case peak depth of water is approximately 8 m in the Condamine River channel, 2 m in the Condamine River North Branch, and approximately 5 m deep in the Grasstree Creek channel.

The location of the Project alignment in relation to the Condamine River is shown in Figure C-1A in Volume 2 – Appendix C.

7.1 Data collection and review—Condamine River

Available background information, including existing hydrologic and hydraulic models, survey, streamflow data, available calibration information and anecdotal flood data was gathered. Data was sourced from a wide range of stakeholders including:

- TRC—existing flood studies
- BoM—rainfall data
- DTMR—existing infrastructure details
- DRDMW/Sunwater—streamflow and gauge details.

7.1.1 Previous studies

A number of previous hydrology and hydraulic studies were sourced as part of this assessment. A review of each study was undertaken to determine suitability for use on the Project, as documented in the following sections.

7.1.1.1 Upper Condamine River Flood Study, SKM, 2013a

In 2013, SKM undertook a 2D flood study for the Upper Condamine River catchment on behalf of TRC. This flood study included historical and design event modelling and was based on an URBS hydrologic model. The hydraulic model utilised a 60 m grid due to lack of LiDAR information at the time. The BoM developed a number of URBS models for use in its flood forecasting and flood warning system in 2003. The model was calibrated to the 1976 flood event. SKM undertook a review and revision of the BoM URBS model and developed a new Upper Condamine URBS model in 2012. This model was calibrated to a number of flood events, including the 2010/11 event.

In 2013 an URBS model for TRC was developed by extending the 2012 Upper Condamine URBS model to Cecil Plains using catchment data from the BoM 2003 URBS model. The extended URBS model was validated against the 1976 and 2010/11 flood events. In 2013 SKM used the extended URBS model to derive the design flows for a number of design events (10%, 5%, 1% and 0.2% AEP) for input into the hydraulic model. The critical duration of the design events was identified as 72 hours. Information regarding observed flood extents during the December 2010/January 2011 flood event was collected around the township of Ellangowan, located approximately 5 km upstream of the SKM model extent, as part of a community consultation process.

7.1.1.2 SP051 Flood Studies— – Work Package 7 — 2D Flood Study for areas within the Upper Condamine River Floodplain Broad-Scale Hydraulic Model study area including Cecil Plains, SKM, 2014

The Upper Condamine River Flood Study (2013) was updated and published as the TRC SP051 Flood Studies—Work Package 7—2D Flood Study for areas within the Upper Condamine River Floodplain in June 2014.

7.1.1.3 Historical Study for Brookstead, WRM Water & Environment, 2014

This study focused on the hydraulic analysis of flooding in the town of Brookstead. The hydraulic modelling was undertaken using a coupled MIKE FLOOD 1D/2D hydrodynamic model and utilised a steady-state peak flow of 700 m³/s with no hydrologic modelling as basis. As part of the study, a sensitivity analysis was carried out which determined that peak water levels are not very sensitive to changes in flow, roughness or blockage of hydraulic structures.

This study recommended that calibration for at least two historical flood events should be performed to improve the model accuracy. The model developed for this study was validated by adjusting steady-state flows to achieve a match with historic flood marks within ± 0.5 m. A total of eight flood marks for the December 2010 flood event were used for validation, and the modelled peak water levels achieved a match for seven out of eight flood mark heights within the targeted tolerance of ± 0.5 m.

This study also provides a good explanation with regards to the uncertainty associated with observed flood marks, and states: 'Available recorded historical flood information was supplied by TRC. The flood information was collected following the January 2011 event. However, it should be noted that significant flooding occurred in the study area in December 2010 as well as January 2011. It is therefore assumed that the historical flood data collected following the January 2011 event could have originated from either of the two events. Please note that TRC has collected flood data for this study from a variety of sources including debris marks, flood marks visible and accessible at the time of survey after the January 2011 flood, eyewitness accounts, community consultation, etc. It is possible that some the flood data available to TRC may not be accurate or complete. Information used is the best information available at this time for the purposes of this study. Marks observed and other anecdotal information obtained after flood events have been obtained from a range of sources and have varying degrees of certainty'.

7.1.1.4 Addendum 1 Updated 2D Hydraulic Modelling to include LiDAR and Flexible Mesh GPU Approach, Water Modelling Solutions, 2015

Water Modelling Solutions built a new hydraulic model of the Upper Condamine River catchment using the MIKE FLOOD FlexiMesh software on behalf of TRC in 2015. This model used LiDAR data and extended the SKM model domain to include the township of Ellangowan. The model domain extends from approximately 10 km south of Ellangowan to approximately 14 km north of Cecil Plains and includes the Condamine River study area for the Project's hydrology and flooding assessment.

The model includes cross-drainage structures under the Gore Highway downstream of the Project alignment. Dimensional data for these structures were used in the development of the TUFLOW model for the Project's hydrology and flooding assessment.

The MIKE FLOOD model was validated against observed records for the Upper Condamine River catchment by WMS. The model results had reasonable (-0.25 m) fits against observed records at the Department of Natural Resources and Mines gauge (422347—North Condamine River at Pampas) within the study area.

7.1.1.5 Inland Rail, Border to Gowrie Phase 1 Report, AECOM, 2017

During 2017, AECOM was commissioned by ARTC to undertake a hydraulic assessment of the Condamine River and floodplain at its intersection with the proposed Inland Rail corridor at the time. The hydraulic assessment was undertaken to establish existing flood conditions, determine potential flood impacts, and inform the design of cross-drainage infrastructure to establish comparative cost estimates and enable a Multi-Criteria Analysis of the four corridor options under consideration at the time.

The hydrology inflows adopted for this study for the Condamine River and its tributaries were based on flow hydrographs extracted from this WMS MIKE FLOOD model. No additional hydrologic assessment was undertaken. For the hydraulic assessment AECOM developed a hydraulic model using the TUFLOW CPU software package. The model was used to provide an estimate of existing flood levels, extents and velocities; inform the design of the cross-drainage solutions for the Inland Rail Phase 1 concept; and estimate any potential impacts on flooding as a result of the Project. Tributaries of the Condamine River, such as Rocky Creek, Back Creek, Punches Creek and Hermitage Creek, were not modelled as part of this work package.

7.1.2 Survey

ARTC provided LiDAR data from 2015 as 1 m grid DEM tiles. Using GIS software, a DEM was generated with a 1 m grid resolution for use in the Project based on the 2015 dataset. This was used for modelling within the permanent footprint and up to the full extent of the 2015 LiDAR, where relevant.

The DEM datasets used for modelling were based on surveys flown between 2009 and 2015. SRTM data was used for catchment delineation where no LiDAR data could be sourced, to inform the hydrologic modelling.

The LiDAR data that was used for the Condamine River model development is summarised in Table 7.1.

Table 7.1 Condamine River—LiDAR datasets

LiDAR project	LiDAR location	Date flown	Vertical accuracy	Output grid resolution
Melbourne to Brisbane Inland Rail LiDAR	Millmerran to Toowoomba	26/3/2015	0.15 m	1 m
Inland_Towns_Stage_6_2014	CondamineRiv_Twmba_2014_Prj	30/8/2014	0.15 m @ 68% CL	1 m
Inland_Towns_Stage_4_2012	Brookstead_2012_Twn	25/7/2012	0.15 m @ 68% CL	1 m
Toowoomba_2010	Millmerran_2010_Twn	16/7/2010	0.15 m @ 68% CL	1 m

LiDAR project	LiDAR location	Date flown	Vertical accuracy	Output grid resolution
Inland_Towns_Stage_2_2011	Tummalville_2011_Loc	5/6/2011	0.13 m @ 67% CL	1 m
Inland_Towns_Stage_2_2011	Leyburn_2011_Twn	5/6/2011	0.13 m @ 67% CL	1 m
Southern_Downs_2010	Southern_Downs_2010_Rgn	23/10/2010	0.15 m @ 68% CL	1 m
Toowoomba_2010	Tummalville_2010_Loc	16/7/2010	0.15 m @ 68% CL	1 m
Inland_Towns_Stage_6_2014	Clifton_2014_Twn	30/8/2014	0.15 m @ 68% CL	1 m
Toowoomba_2010	Clifton_2010_Twn	16/7/2010	0.15 m @ 68% CL	1 m
Inland_Towns_Stage_4_2012	Maryvale_to_Goomburra_2012_Rgn	31/7/2012	0.15 m @ 68% CL	1 m

Detailed ground and structure survey was incorporated at the Gore Highway crossing at the main Condamine River branch.

The survey data sources and DEM developed for the Condamine River are shown in Figure C1 – B in Volume 2 – Appendix C.

Discrepancies between differently dated LiDAR datasets were observed, which may be attributed to land use practises and seasonal vegetation cover. Updated LiDAR data for the whole model domain will be acquired to facilities model updates during detailed design.

7.1.3 Aerial imagery

Aerial imagery of the Condamine River study area was provided by ARTC and used to identify and confirm topographic and vegetative characteristics of the study area. Aerial imagery captured in 2015 was made available. Additional imagery outside the study area was sourced from QGIS imagery in an open-source format.

7.1.4 Existing drainage structure data

The DTMR Geospatial Information Management System (GIMS) database contained structure information for the following bridges:

- Back Creek Bridge on the Gore Highway—Bridge ID 24899. Dwg. 114676
- Condamine River Bridge on the Gore Highway—Bridge ID 361. Dwg. 149608B
- Bridges on Millmerran-Leyburn Road—Bridge ID 243. Dwg. 166904, Bridge ID 244. Dwg. 42359, Bridge ID 245. Dwg. 42359
- Bridge on Toowoomba-Karara Road—Bridge ID 582. Dwg. 276940, Bridge ID 581. Dwg. 150712, Bridge ID 580. Dwg. 51744.

This structure data was built into the hydraulic model.

7.1.5 Stream gauge data

There are 21 major stream gauges within the Condamine River study area, with 16 operated by the Department of Resources (DoR), four by Sunwater, and one by TRC. Although a large number of stream gauges exist within the study catchment area, most are of limited value to the development of the flood model. Many of the stream gauges either have a short or incomplete data records or are positioned on a small tributary of the Condamine River. The gauges used for model calibration were selected by their data quality and quantity, proximity to the Project alignment and the proportion of catchment area that drains to them. Key gauges used, and their purpose, are summarised in Table 7.2. Data at these gauges was sourced from DoR Water Monitoring Information Portal and requested from Sunwater directly.

Due to floodwaters in larger events breaking out and recombining within the Condamine floodplain, particularly downstream of Tummaville, flood flow estimates at some gauges are reliable only for small events. Despite this, the water levels recorded at the gauges are reliable and are useful for hydraulic model calibration. Gauges at Tummaville, Yarramalong and Pampas were used for hydraulic model development. Flood height data from the Centenary Bridge BoM logbook was used for the 1991 event.

The accuracy of the stream gauges varies considerably in the Upper Condamine catchment. The stream gauge at Warwick is rated as reliable up to a depth of 10.61 m, which corresponds to a flood flow of 3,160 m³/s. The highest recorded gauge height was 3.79 m on 23 December 1975.

Moving downstream to Talgai Tailwater (TW), the gauge is situated upstream of a stream confluence, as shown in Figure 7.1. Talgai TW is rated as reliable up to a depth of 4.77 m, which corresponds to a flood flow of 910 m³/s. Above this height, flows are approximated using log-log extrapolation up to 3,000 m³/s at 13.8 m deep. The highest record gauge height was 6.58 m on 6 May 1996.

Adding uncertainty to the reliability of data at Talgai TW gauge is the Dalrymple Creek catchment to the east (approximately 500 km² in size). Due to upstream breakouts, flood flows from Dalrymple Creek may get recorded at the Talgai TW gauge during larger events.



Figure 7.1 Talgai TW gauge location

Table 7.2 Condamine River—Stream gauges

Gauge	Name	Catchment area (km ²)	Owner	Latitude (°S)	Longitude (°E)	Start	End	Purpose
422310C	Condamine River at Warwick	1,360	DRDMW	28.214	152.049	1/10/1960	Not closed	Calibration of hydrological model using flow data
422355A	Condamine River at Talgai TW	3,105	DRDMW	27.990	151.758	26/10/1989	Not closed	Calibration of hydrological model using flow data
422323A	Condamine River at Tummaville	6,475	DRDMW	27.870	151.511	29/08/1961	Not closed	Calibration of hydrological using flow data and hydraulic model using height data
422347B	North Condamine River at Pampas	378 ¹	DRDMW	27.783	151.424	25/03/1988	Not closed	Calibration of hydraulic model using height data
422353A	Yarramalong Weir TW	-	Sunwater	27.836	151.449	28/10/1989	Not closed	Calibration of hydraulic model using height data
422936	Centenary Bridge on Gore Highway	-	TRC/ Lyndon Pfeffer	27.809	151.363	1/2/1982	-	Validation of hydraulic model using height data. Note: Automatic data logging begun December 2017. Prior events from February 1982 were recorded by logbook.
422316A	Condamine River at Cecil Weir	7,795 ¹	DRDMW	27.534	151.203	24/10/1947	Not closed	Calibration of hydrological model using flow data and validation of hydraulic model using height data.
422345	North Condamine River at Lone Pine	710 ¹	DRDMW	27.669	151.347	13/10/1978	Not closed	Calibration of hydrological model using flow data

Table note:

- 1 Catchment areas stated in the table above are approximate. In larger events, floodwaters from the Condamine River breakout near Tummaville and flow into the North Branch where the Pampas and Lone Pine gauges are located

The Tummaville gauge is rated as reliable up to a depth of 4.23 m, which corresponds to a flood flow of 160 m³/s. The highest recorded gauge height was 8.75 m on 30 January 2013. A flood flow of 700 m³/s at 8.93 m deep is rated a fair estimate but around this level flood waters have already broken out significantly across the floodplain to the north and all estimates of flood flows are unreliable.

The Tummaville gauge was re-rated several times during its record, which has resulted in considerable variation in flow estimates, particularly at large depths. The most recent rating curve was adopted some time in 2008, and subsequent flood events have smaller recorded flows for a given depth than equivalent floods during the 1970s, 1980s and 1990s. For example, the depths recorded at Tummaville during the 1976 and 2010 events were very similar—11.19 m and 11.14 m, respectively—yet the estimated flows were 1,788.4 m³/s and 957.9 m³/s, respectively.

Adding further uncertainty to the flows estimated at the Tummaville gauge, there are substantial breakouts upstream of the Tummaville gauge that allow floodwaters to completely bypass the gauge (Figure 7.2) and not be recorded; therefore, using this gauge for hydrologic model development was only considered for small events and with caution.

Downstream of the Project alignment are the Cecil Weir and Lone Pine gauges. Cecil Weir is rated as reliable up to a depth of 5.55 m, which corresponds to a flood flow of 2,025 m³/s. The highest recorded gauge height was 4.84 m on 6 April 1988.

Lone Pine is a very small gauge used for monitoring low flows and is rated as accurate up to a depth of 1.5 m with a flow of 1.0 m³/s. Above 1.5 m, the flood flows are estimated to be 1,000 m³/s at 3.7 m deep. The highest recorded gauge height was 1.66 m on 16 February 1988.

Flows at these gauges were combined to represent all floodwaters exiting the Condamine River Study area. These combined flows were used to aid in the calibration of the hydrologic model. The limited accuracy of the Lone Pine gauge was outweighed significantly by the higher accuracy of the Cecil Weir gauge.

7.1.6 Historical rainfall data

Historical rainfall information used for model development came in two forms: gauging station records and gridded daily rainfall data.

There are over 200 daily rainfall gauging stations in and around the Upper Condamine catchment. Data at these gauges was sourced from the BoM website. Sub-daily data (pluviography data) is available at one- or 6-minute intervals and this resolution facilitates a better understanding of the TPs of storms for calibration.

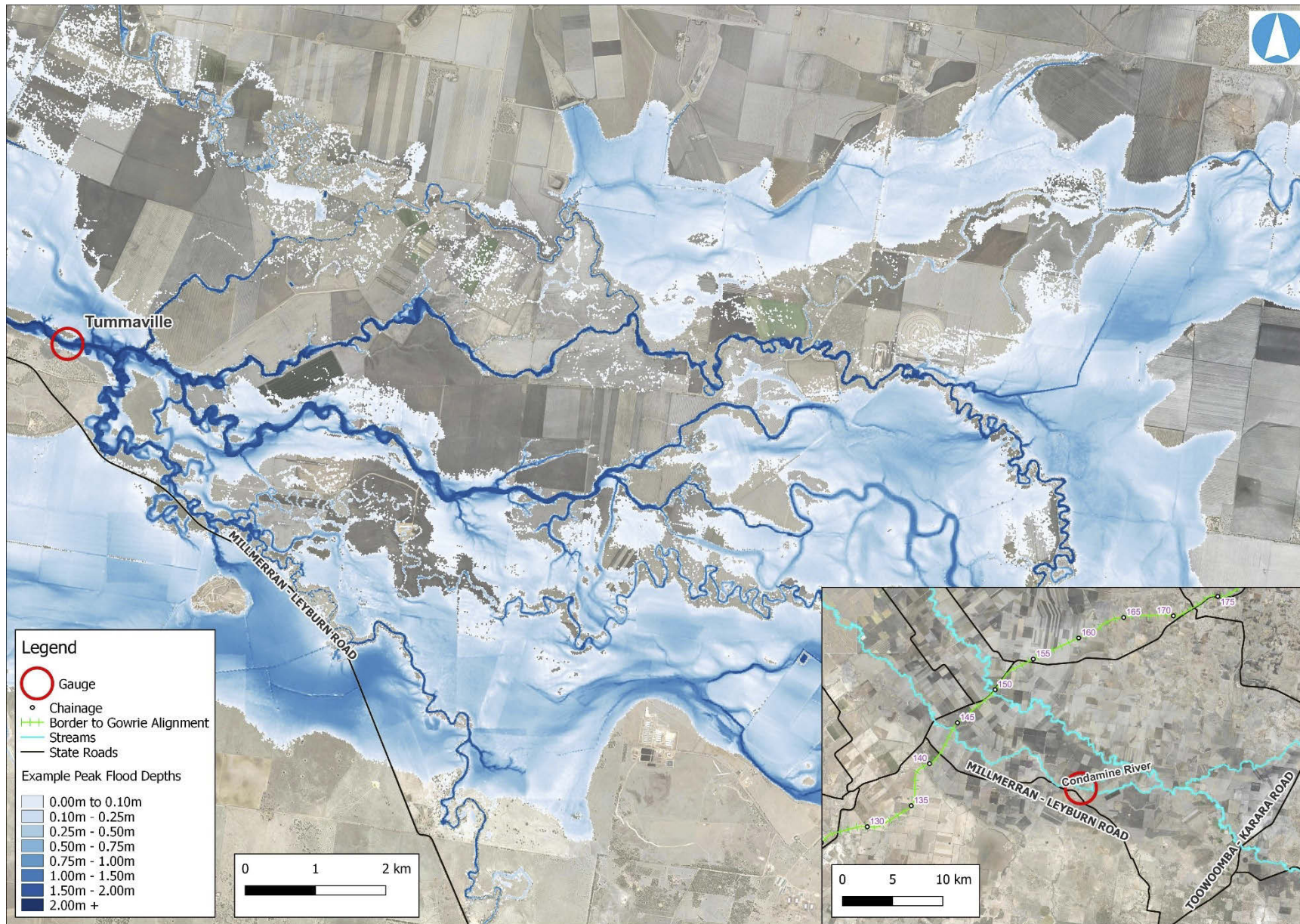


Figure 7.2 Tummaville gauge location

Pluviograph data for the gauges as listed in Table 7.3 was sourced from the BoM.

Table 7.3 **Condamine River gauges—Pluviograph data**

Gauge No.	Name	Longitude	Latitude	Record start	Record end ¹
041018	Clifton Post Office	-27.932	151.906	19/09/1972	31/07/2010
041044	Hermitage	-28.206	152.100	18/02/1952	28/02/2001
041056	Killarney Post Office	-28.334	152.295	24/09/1972	31/01/2010
041445	Leslie Dam	-28.214	151.919	30/04/1986	30/06/2009
041060	Leyburn	-28.009	151.586	1/01/1959	21/11/1996
041063	Leyburn Post Office	-28.011	151.586	1/03/1997	31/05/2010
041361	Pittsworth DPI	-27.717	151.629	9/09/1970	14/12/1984
041082	Pittsworth	-27.716	151.633	25/03/1959	24/02/1996
041525	Warwick	-28.206	152.100	1/04/1999	31/01/2022
041467	Toowoomba City Council	-27.567	151.885	1/01/1957	1/01/1984
040082	University of Queensland Gatton	-27.544	152.338	1/01/2021	31/01/2022
041529	Toowoomba Airport	-27.543	151.913	1/01/2021	31/01/2022
041175	Applethorpe	-28.622	151.953	1/01/2021	31/01/2022
041359	Oakey Aero	-27.403	151.741	1/01/2021	31/01/2022
040004	Amberley	-27.630	152.711	1/01/2021	31/01/2022
041522	Dalby Airport	-27.161	151.263	1/01/2021	31/01/2022

Table note:

- 1 Record end refers to the end date of the period of data used for this assessment and does not indicate the operational status of the gauge

The Queensland Government also provides gridded climatic data through the Scientific Information for Land Owners online portal. The data uses advanced splining or kriging techniques that spatially interpolate between data points—in this case, rainfall gauge information, in order to provide an estimate of daily rainfall depths across Australia.

7.1.7 Anecdotal and observed flood data

Local landowners provided the following anecdotal flood information, which was invaluable for flood model calibration and validation:

- Flood markers on local landowners' properties. With landowner permission, some of these flood markers were surveyed and their heights assisted in validating the hydraulic model. Validation of hydraulic modelling results against flood marker data is discussed further in Section 7.3.7.1.
- Timestamped photographs of historical floods:
 - Some photographs were taken from aircraft, which provided information on broader flood extents
 - Some photographs show flood heights at buildings and fences
 - The results of the hydraulic model were compared against the information shown in these photos.
- Anecdotes of flood extents and behaviour. For instance, landowners have provided insight into:
 - Where and how floods breakout from the main Condamine flow paths
 - The velocity of flows, which tend to vary considerably across the floodplain
 - How flood direction can be significantly affected by accumulated flood debris and crop types
 - The impact that the Gore Highway upgrades have had on flood behaviour
 - Their concern about scour at culverts and other locations of concentrated flow.

7.1.8 Site inspection

A site inspection was undertaken in February 2018. During the site inspection, all major waterway crossings were visited and inspected with photographs taken and details recorded of the crossing, existing drainage structures and surrounding catchment. An assessment of the relative roughness and blockage potential was undertaken during the site inspection.

Flood markers were surveyed on several occasions. During these surveys, additional inspections were undertaken of locations previously affected by flooding, including interviews with landowners to inform model validation.

An additional site visit to the Lone Pine Gauge was undertaken in May 2021 to gain insights from landowners on flooding experienced in the vicinity of the gauge during the 2010 event. Visits were conducted on two separate farms adjacent to the gauge with an objective to utilise on-ground observations (including additional flood markers surveyed during the visit) during the flood event to validate hydrologic and hydraulic modelling. A thorough assessment of the gauge and its implications on the hydrology model development are discussed in Section 7.2.7.

7.2 Hydrologic model development—Condamine River

7.2.1 Model setup

It is understood that TRC, Sunwater and BoM possess calibrated hydrologic models of the Upper Condamine catchment; however, as none of these models were made available for use on this assessment, a new, calibrated model was built to establish historical event flows and design flood event flows for input into the hydraulic model.

A runoff routing network model (hydrologic model) was established using the latest procedures detailed in ARR 2019 and using the latest rainfall data from BoM. The URBS software package was chosen for estimating flood flows in the Condamine River catchment. URBS is a sophisticated rainfall runoff program that was widely used in the hydrology industry for over 25 years.

The hydrologic model covers approximately 8,542 km² of the Condamine River catchment, spanning the upper reaches at Killarney and extending 146 km downriver to Cecil Plains. The catchment was delineated into 147 sub-catchments to capture the variability of hydrologic parameters such as losses and rainfall, and to better represent the network of creeks and streams within the catchment.

These sub-catchments were grouped into four subzones according to the stream gauges (Table 7.4) being used for calibration (Section 7.2.3). The hydrologic model setup, including extent and sub-catchment map, is presented in Figure C1-C in Volume 2 – Appendix C.

Table 7.4 Condamine River—Stream gauges adopted for calibration

URBS sub-model	Stream gauges used for calibration
A	422310C—Condamine River at Warwick
B	422355A—Condamine River at Talgai TW
C	422323A—Condamine River at Tummaville
D	422345A—North Condamine River at Lone Pine, 422316A – Condamine River at Cecil Weir

URBS can accommodate up to seven routing variables that have varying degrees of influence on the size and shape of flood hydrographs produced by the model. This model uses only two routing variables (catchment areas and reach lengths) to estimate flood flows. During the model calibration process, two additional routing variables were tested (channel slope and catchment slope) with negligible improvement to model outputs. Therefore, these additional parameters were omitted from the model.

It must be noted that Leslie and Connolly Dams are situated in the upper reaches of the Condamine River catchment. Given their catchment sizes compared to the size of the overall catchment are small and their storage volume is also small, any contributing attenuation from these dams would be negligible. Therefore, both dams have not been included in the hydrologic model.

A summary of the hydrologic model inputs is provided in Table 7.5.

Table 7.5 Condamine River—Summary of URBS model calibration inputs

Input parameter	Remarks
URBS model type	Split
Routing variables	Catchment area, stream lengths
Rainfall data used	Daily data at over 200 rainfall gauges downloaded from the BoM's database.
Pluviograph data used	BoM pluviographs – 041018, 041044, 041056, 041445, 041060, 041063, 041361, 041082, 041525, 041467, 041457, 040677, 041359, 041352, 041536, 422310C, 422313B, 422355A, 422394A, 541041, 541062, 041175, 041522, 041529, 0422319, 422323, 4223001
Gridded rainfall data used	Daily rainfall data was sourced from Scientific Information for Land Owners online portal
Stream gauge data used	DRDMW's gauges at 422310C, 422355A, 422323A, 422347B, 422316A

7.2.2 Event selection for calibration

Stream gauges within the catchment were prioritised by quality of gauge record and their position within the catchment. Five gauges were identified as high priority for model calibration (Table 7.4). Flood frequency analysis using Log Pearson Type III to fit the data were conducted at each of these gauges to determine key flood events that would be suitable for model calibration purposes. The aim was to select one large and one small flood event for calibration to provide confidence that the final, calibrated model can replicate events of these magnitudes and be used to interpolate/extrapolate floods of other magnitudes.

The two largest events on record appear to be the 1956 and 1976 events, of which several community members recall their magnitudes and impacts; however, available quality data for these events is scarce and the floodplain has changed significantly in the last 30 to 50 years. Therefore, calibrating to these events would not produce a reliable model. Instead, the focus shifted to the December 2010 and January 2011 events. Both events were large and there exists a large amount of stream gauge and rainfall information for these events.

Furthermore, local landowners were able to provide valuable anecdotal information in the form of photographs and flood markers on their properties. The December 2010 event was finally chosen since the January 2011 event began before the 2010 event had fully finished.

Selecting a smaller event was somewhat more difficult than choosing a large event. Farmers are legally permitted to pump floodwater from the Condamine River and its various branches into dams (or ring tanks) for irrigation. Given the scale of agriculture in the Condamine floodplain, the amount of water harvested can be significant compared to the size of the flood. For instance, in the 2004 event, flood volume recorded at the Cecil Weir gauge was approximately half of what was recorded at the Talgai TW gauge 75 km upstream. Calibrating a hydrologic model to events like 2004 is risky and may produce an unreliable model.

Using historical aerial photography, it can be seen that the number of ring tanks in the area grew substantially in the mid-to-late 1990s, which correlates to larger volumes of floodwater being pumped from the floodplain during flood events.

Within the extent of the regional TUFLOW model, as described in Section 7.3, a total of 63 significant ring tanks were identified. The construction dates were estimated using historical aerial photographs from Google Earth®. The number of ring tanks built since 1984, as well as the cumulative increase, is shown in Figure 7.3.

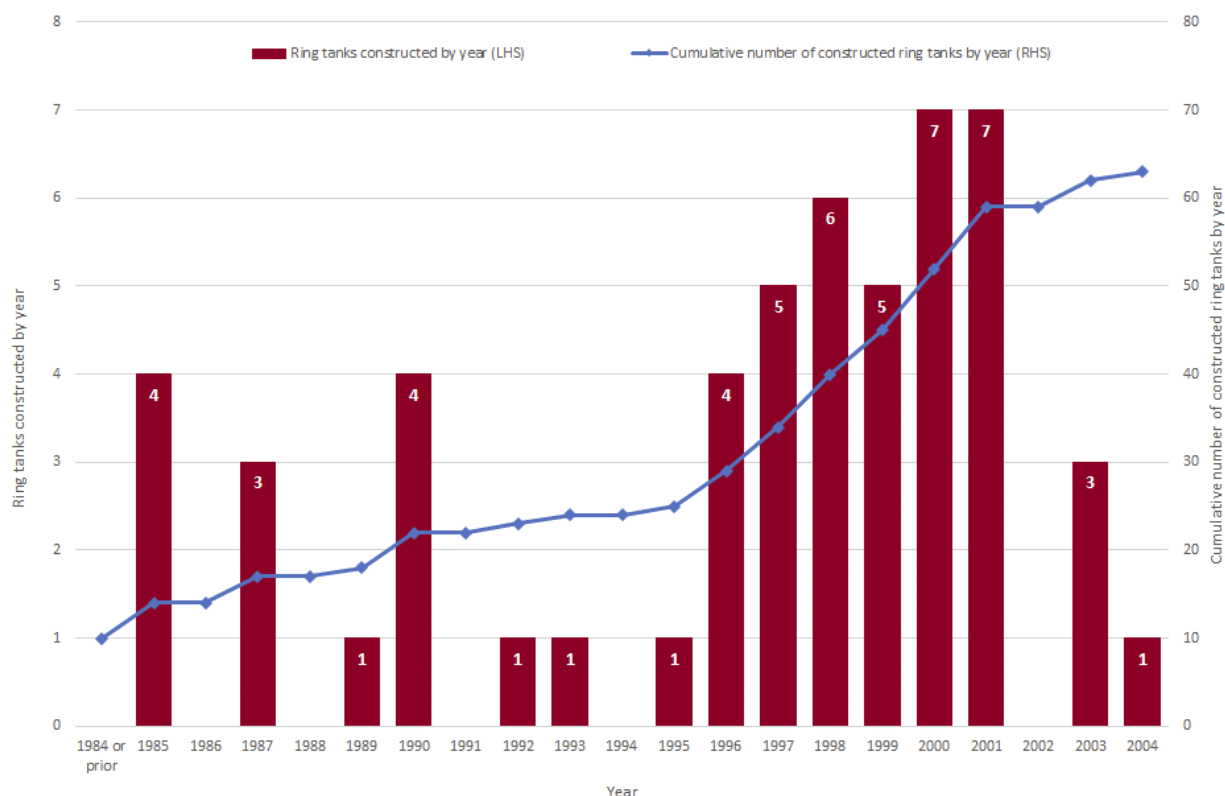


Figure 7.3 Condamine River—Estimated proliferation of ring tanks within the regional flood model extent

The February 1991 event and December 2010 event were selected for calibration as they represented a relatively small flood event and a relatively large flood event, respectively. The January 2013 and recent December 2021 events were selected to validate the hydrologic model performance under intermediate weather conditions. A summary of peak depths and flows recorded at gauges for each flood event, and their estimated AEP, is summarised in Table 7.6. For comparison purposes, the 1956 and 1976 events were included.

Table 7.6 Condamine River—Historic gauged results

Event	Warwick (422310C)	Talgai TW (422355A)	Tummalville (422323A)	Cecil Weir (422316A)
1956	No data	No data	No data	8.8 m – 1,717.8 m ³ /s 3.7% AEP
1976	8.3 m – 1,519.4 m ³ /s 0.6% AEP	No data	11.2 m – 1,788.4 m ³ /s 1.0% AEP	9.2 m – 2,023.9 m ³ /s 2.2% AEP
1991	3.9 m – 236.4 m ³ /s 19.6% AEP	14.2 m – 290.6 m ³ /s 26.0% AEP	8.3 m – 602.2 m ³ /s 25.1% AEP	7.5 m – 872.4 m ³ /s 23.3% AEP
2010	7.1 m – 1,029.9 m ³ /s 2.6% AEP	15.7 m – 2,264.2 m ³ /s 8.9% AEP	11.1 m – 957.9 m ³ /s 13.0% AEP	9.2 m – 2,046.8 m ³ /s 0.8% AEP
2013	6.9 m – 956.1 m ³ /s 3.7% AEP	15.7 m – 2,375 m ³ /s 2.1% AEP	10.8 m – 866.5 m ³ /s 19.9% AEP	8.3 m – 1,342.9 m ³ /s 12.1% AEP
2021	4.9 m – 354.6 m ³ /s 10.4% AEP	15.0 m – 725.1 m ³ /s 13.5% AEP	10.1 m – 715.3 m ³ /s 15.4% AEP	8.1 m – 1,009 m ³ /s 15.2% AEP

7.2.2.1 February 1991 event

The 1991 event appeared to be of suitably small magnitude, early enough to have negligible pumping intervention, and recent enough to have good rainfall and stream-gauge data coverage.

Historical aerial photography indicates that very few private storages had been constructed prior to the 1991 event. It is estimated that as few as three had been constructed along the Condamine River between Tummaville and the Gore Highway; one on Grasstree Creek; and one on the North Branch near Pampas.

The potential influence of irrigation pumping would be minimal on this event. Flood frequency analysis at the Warwick and Cecil Weir gauges estimate the February 1991 event was a 50% to 20% AEP event. The spatial distribution of rainfall during the 1991 event is shown in Figure 7.4.

7.2.2.2 December 2010 event

In the months leading up to the 2010 flood event there were a series of large convective storms over South East Queensland. It is highly likely the catchment was saturated, and all private storages were at full capacity, despite the large number of private storages constructed by that time.

Additionally, the rainfall event that contributed to the flood lasted at least three weeks, which suggests that if any storages were not already full, any pumping to top them up would have ceased prior to the flood peak. The potential influence of irrigation pumping would be minimal on this event. Flood frequency analysis at the Warwick and Cecil Weir gauges estimate the December 2010 was approximately a 5% to 1% AEP event. The spatial distribution of rainfall during the 2010 event is shown in Figure 7.5.

7.2.2.3 January 2013 event

The 2013 flood event in the Upper Condamine catchment was one of the largest on record. The event reached a peak depth of 10.76 m at Cecil Weir at approximately 1.00 pm on 29 January 2013. The rainfall burst that contributed to the peak flood event lasted approximately 6 days.

On average, 160 mm fell across the entire Upper Condamine catchment during this time. Some areas upstream of Warwick received rainfall in excess of 540 mm during this period. The event is estimated to be a 16% AEP flood event at Cecil Weir. The spatial distribution of rainfall during the 2013 event is shown in Figure 7.6.

7.2.2.4 December 2021 event

The 2021 flood event in the Upper Condamine catchment was the most recent event considered for model validation. The event reached a peak depth of 8.0 m at Cecil Weir at approximately 8.00 am on 3 December 2021. This prolonged event extended for 61 days with approximately 80 per cent of rainfall occurring during the first 25 days.

On average, 350 mm fell across the entire Upper Condamine catchment during this time. Some areas upstream of Warwick received rainfall in excess of 610 mm during this period. The event is estimated to be a 15.2% AEP flood event at Cecil Weir. The spatial distribution of rainfall during the 2021 event is shown in Figure 7.7.

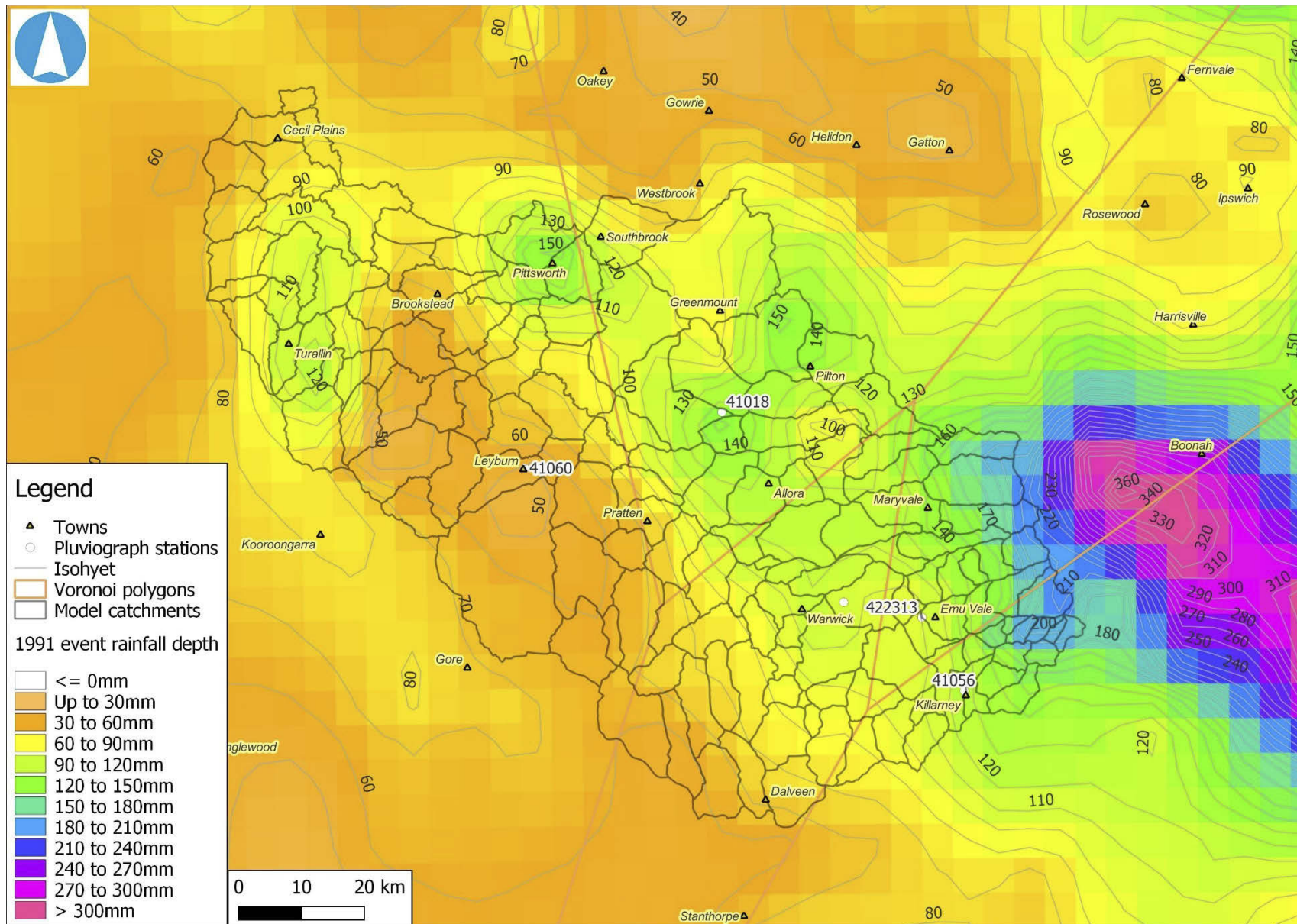


Figure 7.4 Condamine River catchment—Gridded rainfall depths during 1991 event

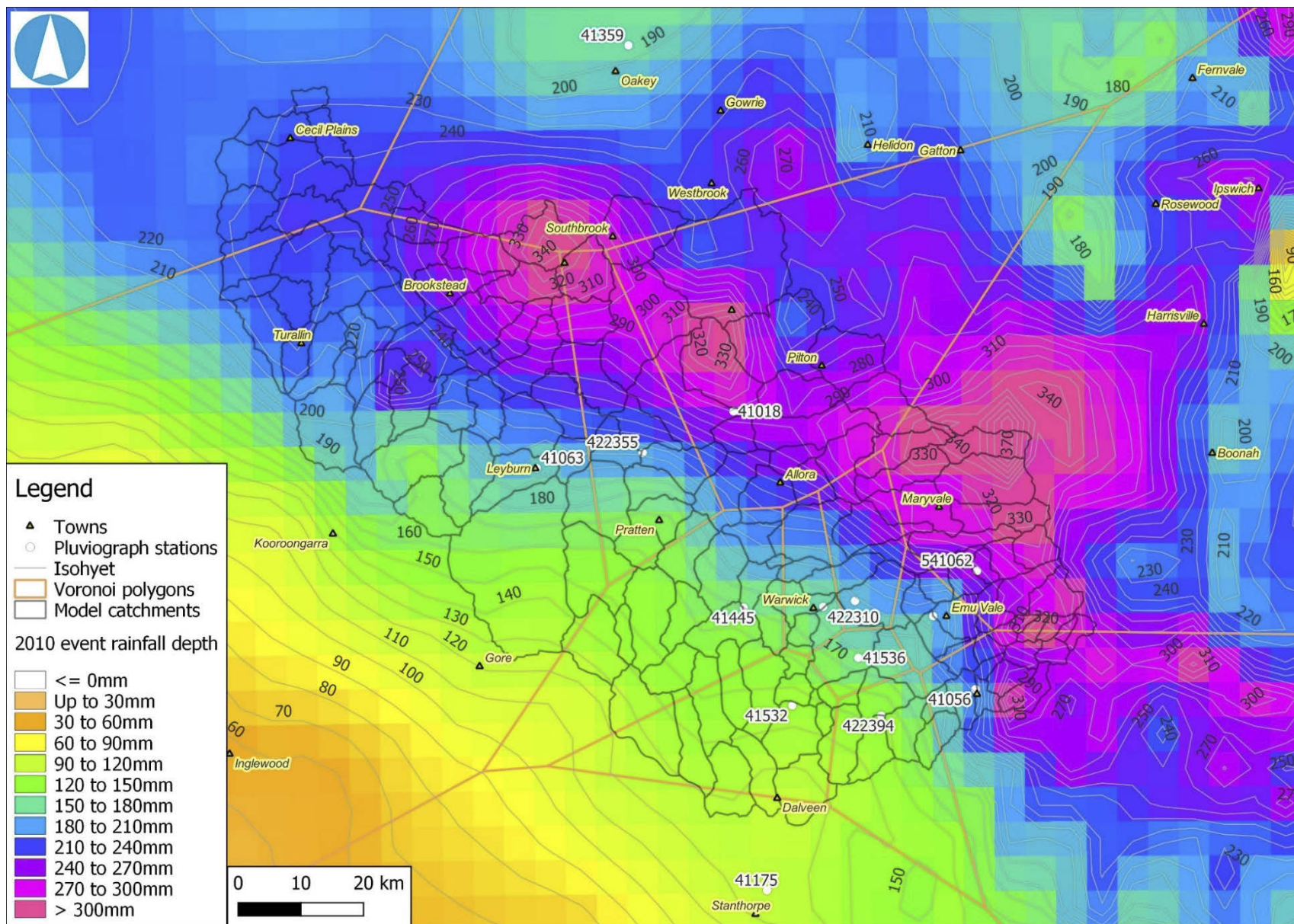


Figure 7.5 Condamine River catchment—Gridded rainfall depths during 2010 event

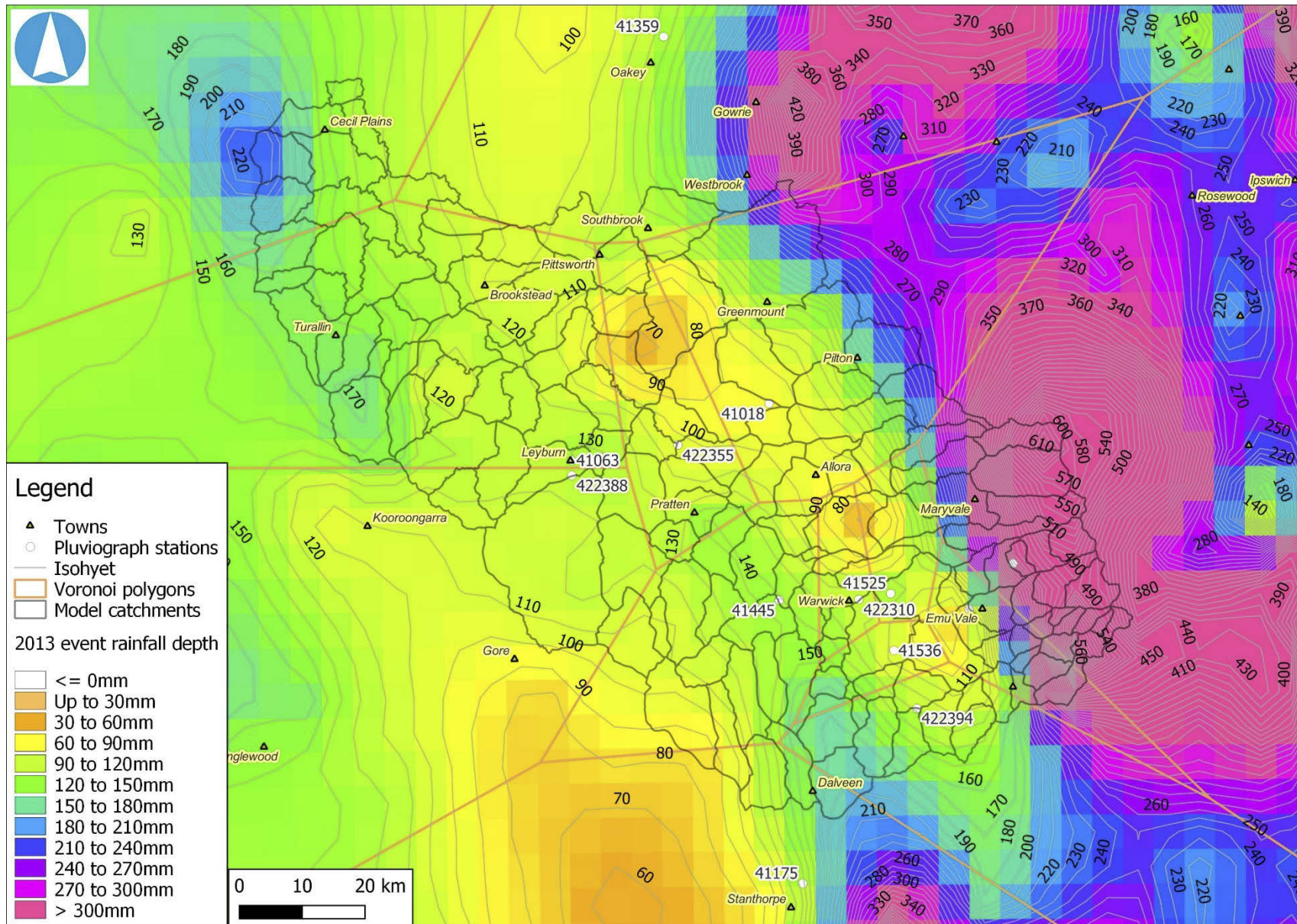


Figure 7.6 Condamine River catchment—Gridded rainfall depths during 2013 event

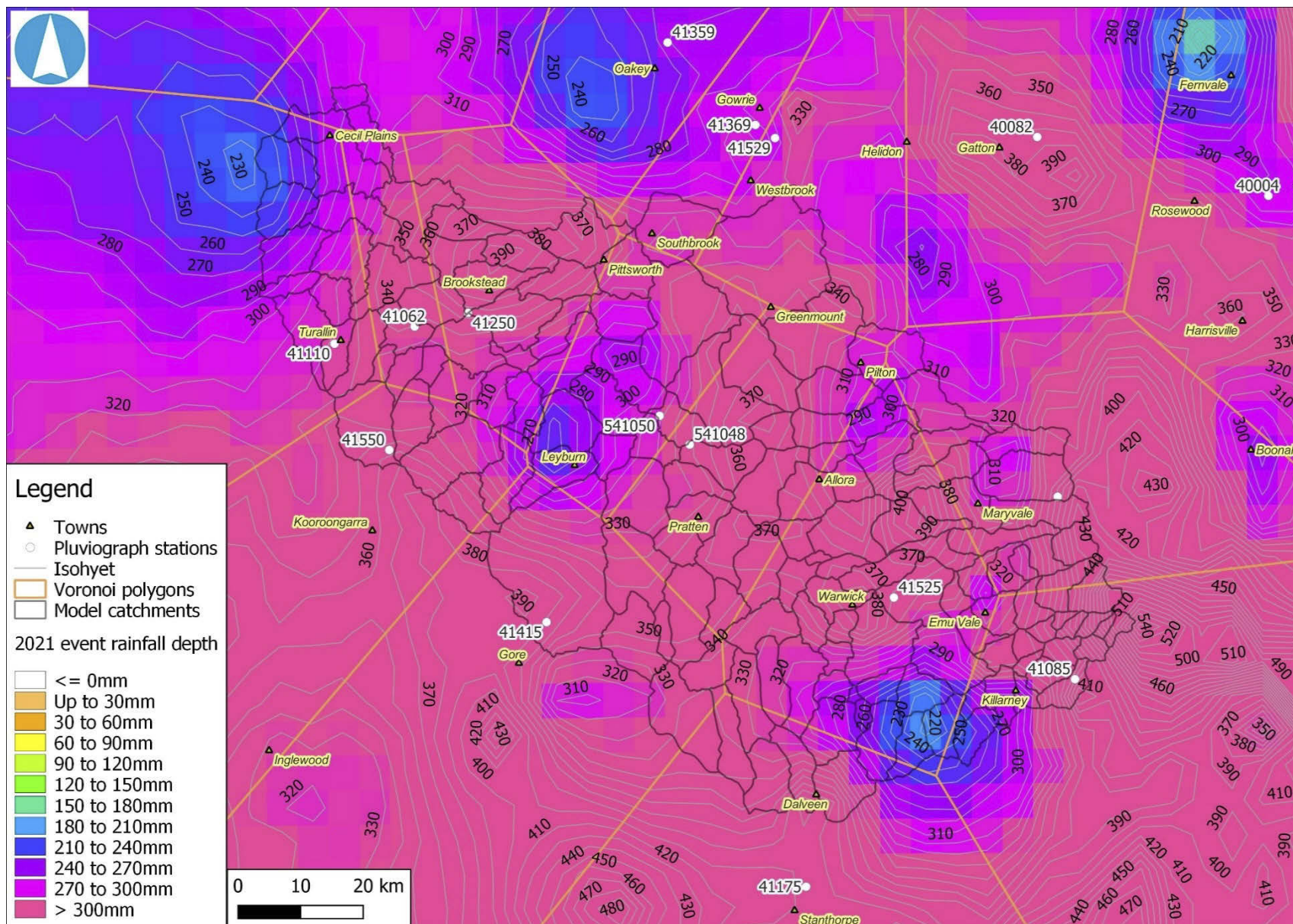


Figure 7.7 Condamine River catchment—Gridded rainfall depths during 2021 event

7.2.3 Model calibration

Gridded rainfall data provided an estimate of rainfall variation across the entire model catchment for each calibration event (Figure 7.4, Figure 7.5, Figure 7.6 and Figure 7.7). Area-weighted averages of gridded rainfall depths were applied to each model sub-catchment. Pluviograph data was used to capture the temporality of each calibration event. The sub-daily rainfall patterns from the pluviographs were applied to each model sub-catchment based on their proximity to the gauges using Voronoi analysis.

The rainfall depths and TPs were applied to the hydrologic model to produce estimates of flood hydrographs at the stream gauges identified for calibration. URBS model routing parameters alpha, beta, m, rf, and the IL and CL were adjusted until good fits to historical stream gauge hydrographs was obtained.

Measures of good fit to historical data were:

- Peak flow
- Hydrograph volume
- Time to peak
- Matching shape of flood event.

As an overall check of flood flow estimation within the model, the URBS hydrographs at Cecil Weir and Lone Pine were combined and compared against a combined stream gauge hydrograph. This combination is referred to as the model 'Outlet'. Comparing the model outputs to separate gauge observations is not considered a better estimate of the flow distribution between the North Branch and Condamine River as the rating curve at Lone Pine was found to be of poor quality. The rating quality at Lone Pine and its implications on the flow distribution between Condamine River and North Branch are discussed in detail in Section 7.2.7.

Calibration hydrographs at each gauge and calibration event are displayed in Figure 7.8 to Figure 7.11 (1991 event) and Figure 7.12 to Figure 7.15 (2010 event).

A summary of the parameters used to achieve the calibration hydrographs is contained in Table 7.7 (1991 event) and Table 7.9 (2010 event).

The performance metrics of the calibration are summarised in Table 7.8 (1991 event) and Table 7.10 (2010 event). A discussion on results follows the figures and tables.

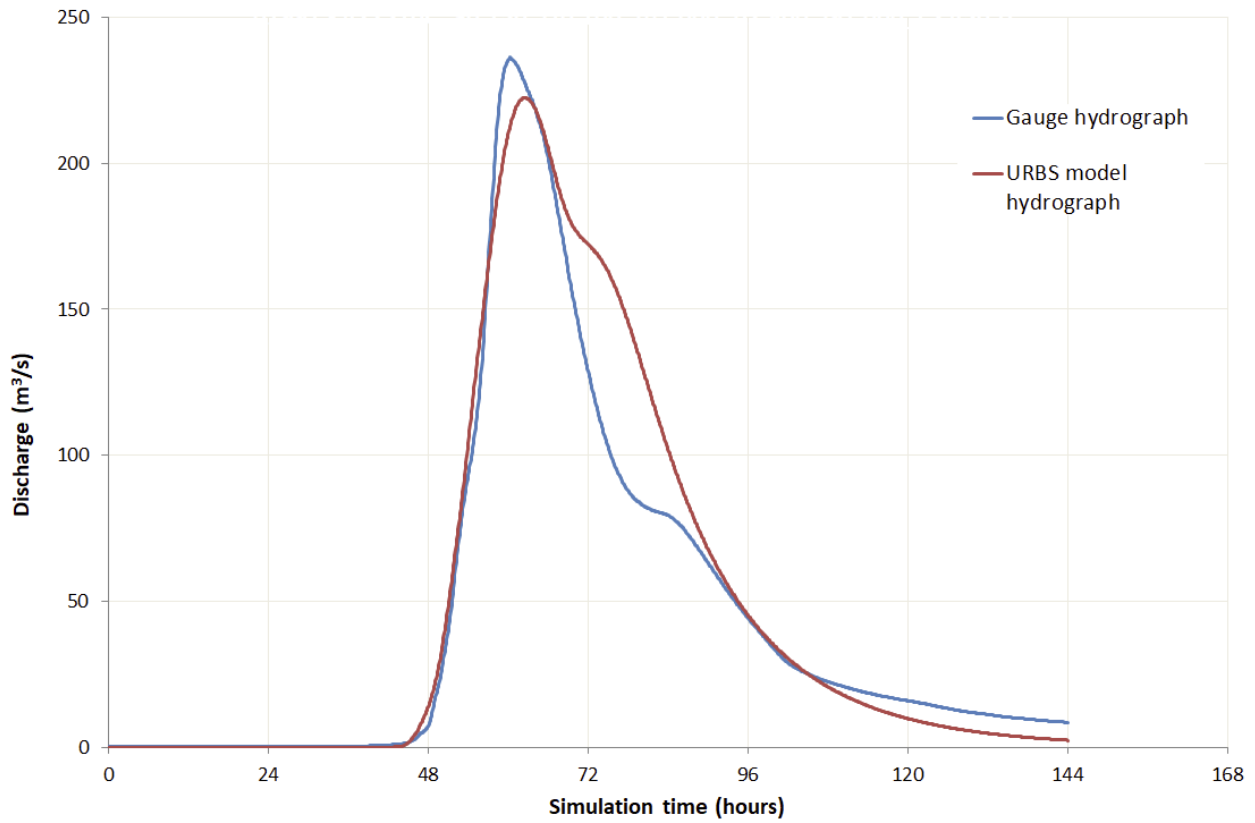


Figure 7.8 Calibration hydrographs at Warwick 422310C—1991 event

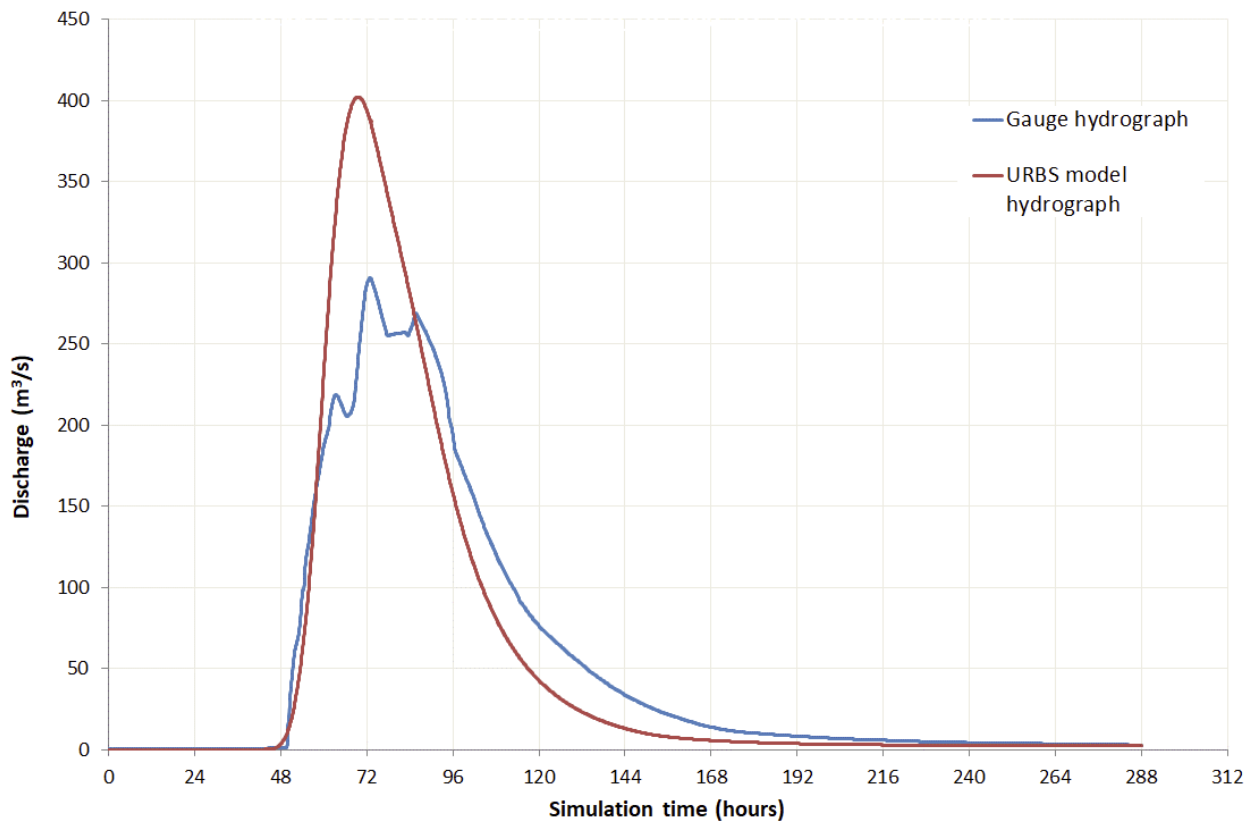


Figure 7.9 Calibration hydrographs at Talgai TW 422355A—1991 event

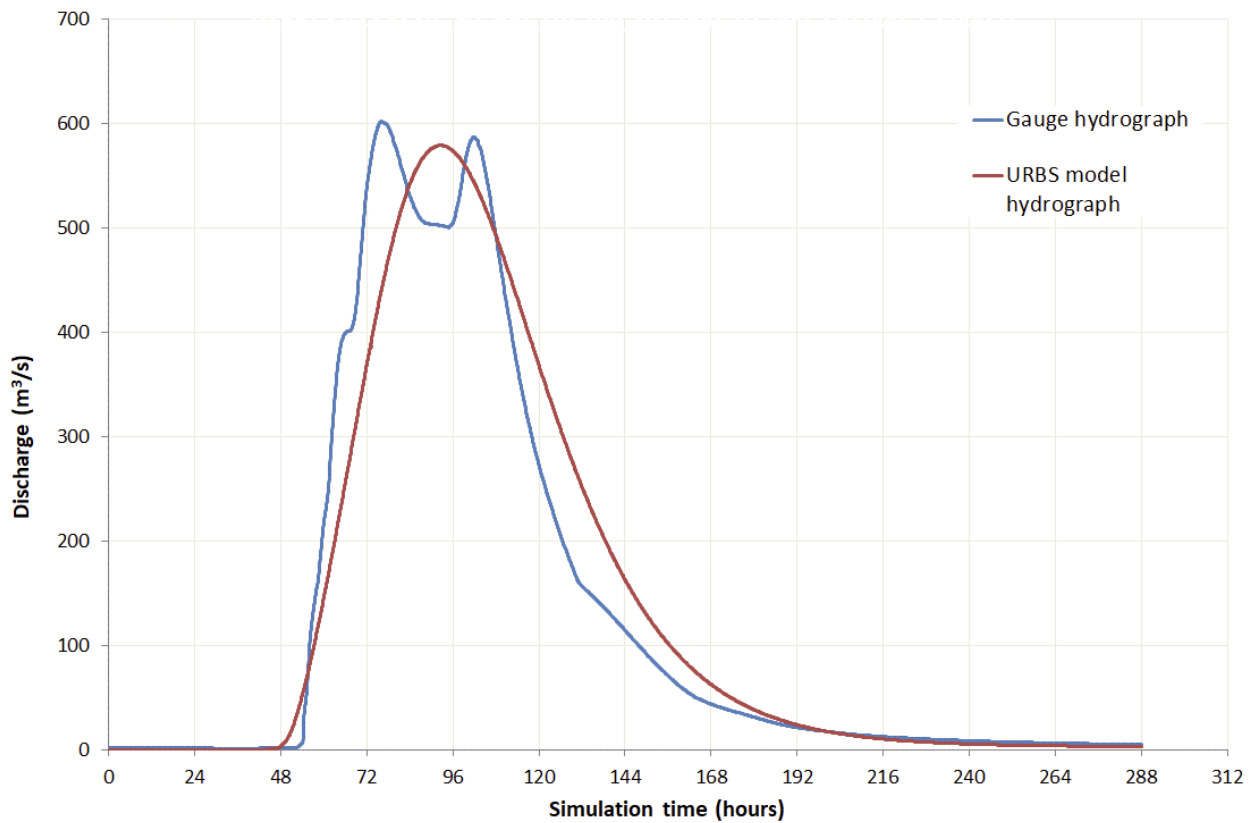


Figure 7.10 Calibration hydrographs at Condamine River at Tummalville 422323A—1991 event

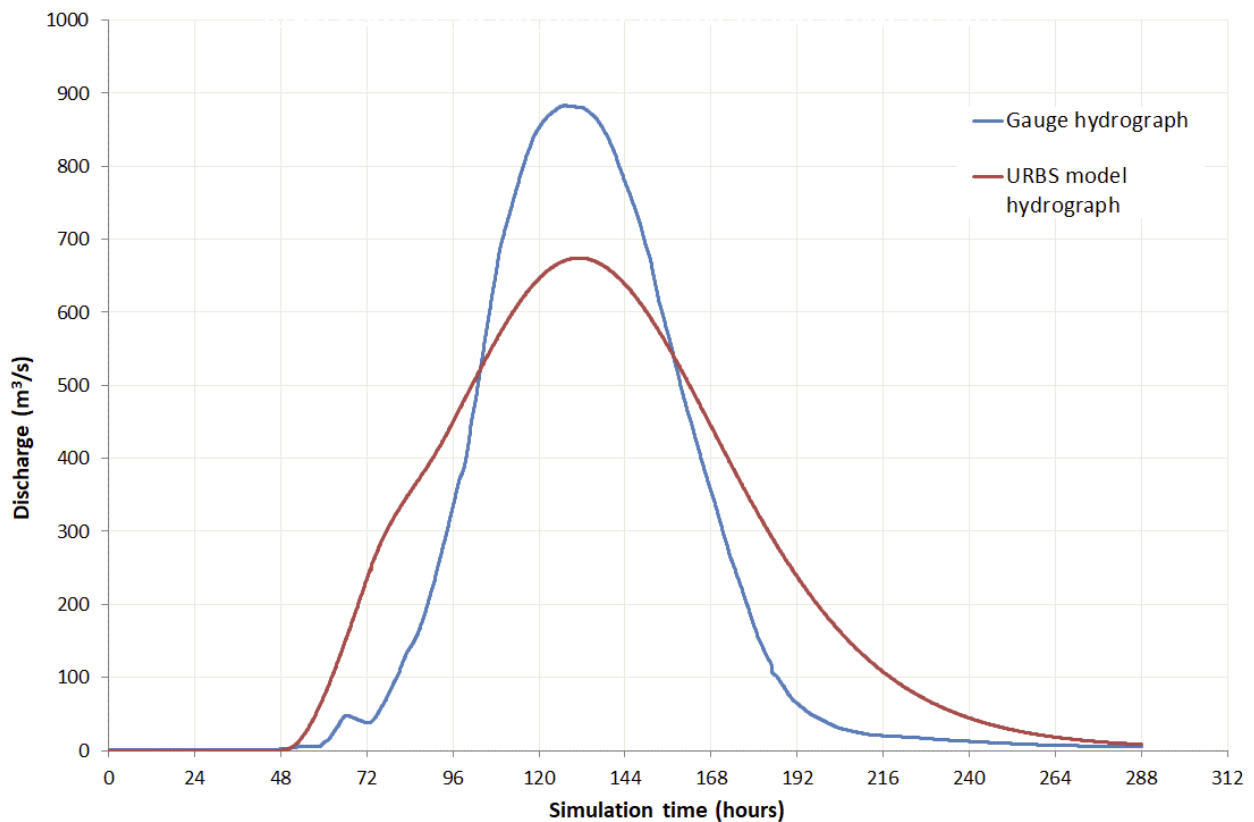


Figure 7.11 Calibration hydrographs at combined 'outlet'—1991 event

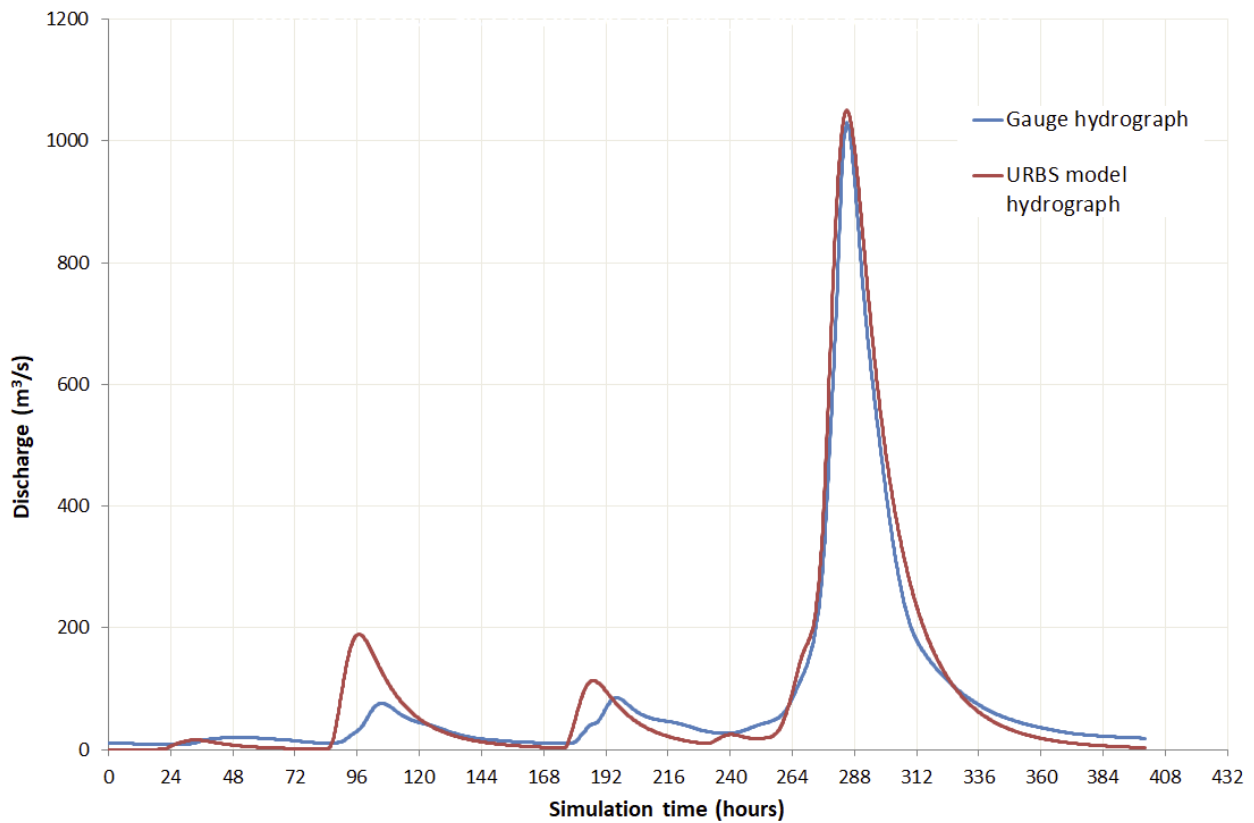


Figure 7.12 Calibration hydrographs at Warwick 422310C—2010 event

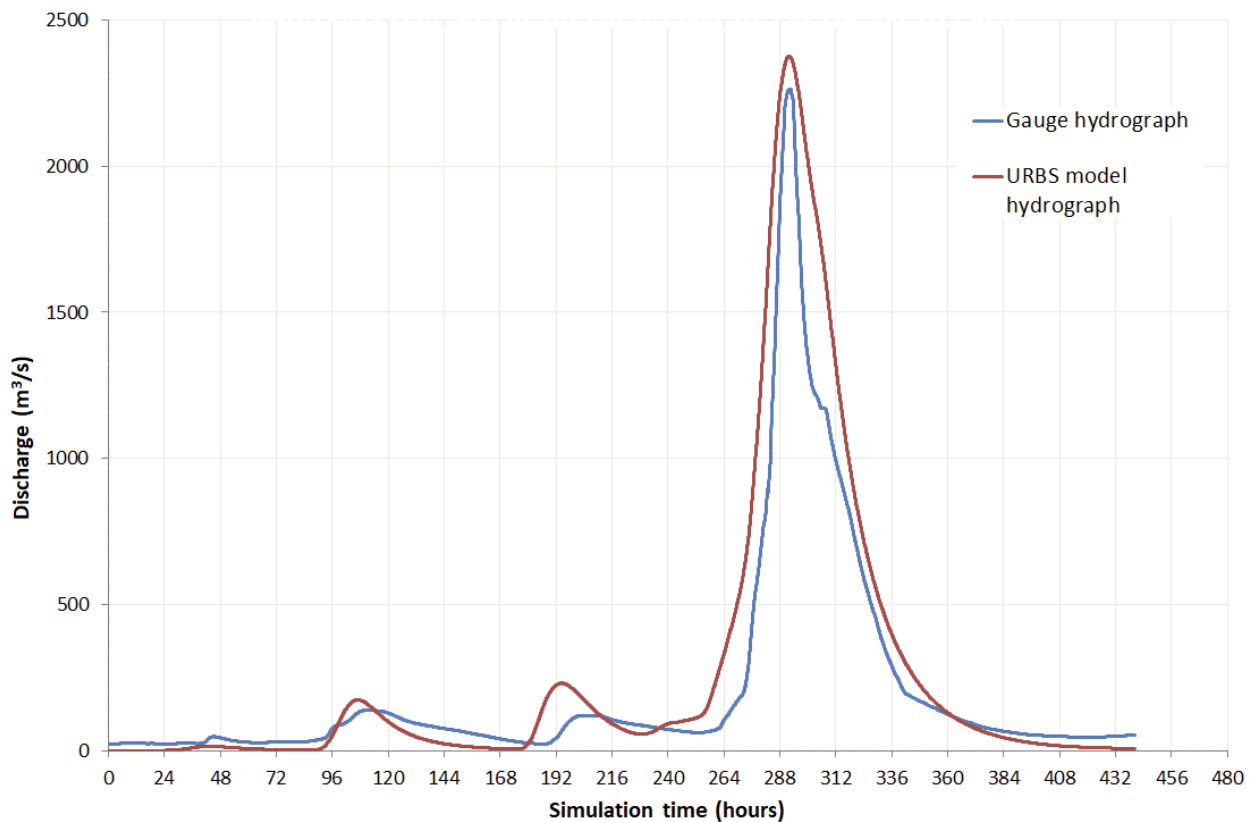


Figure 7.13 Calibration hydrographs at Talgai TW 422355A—2010 event

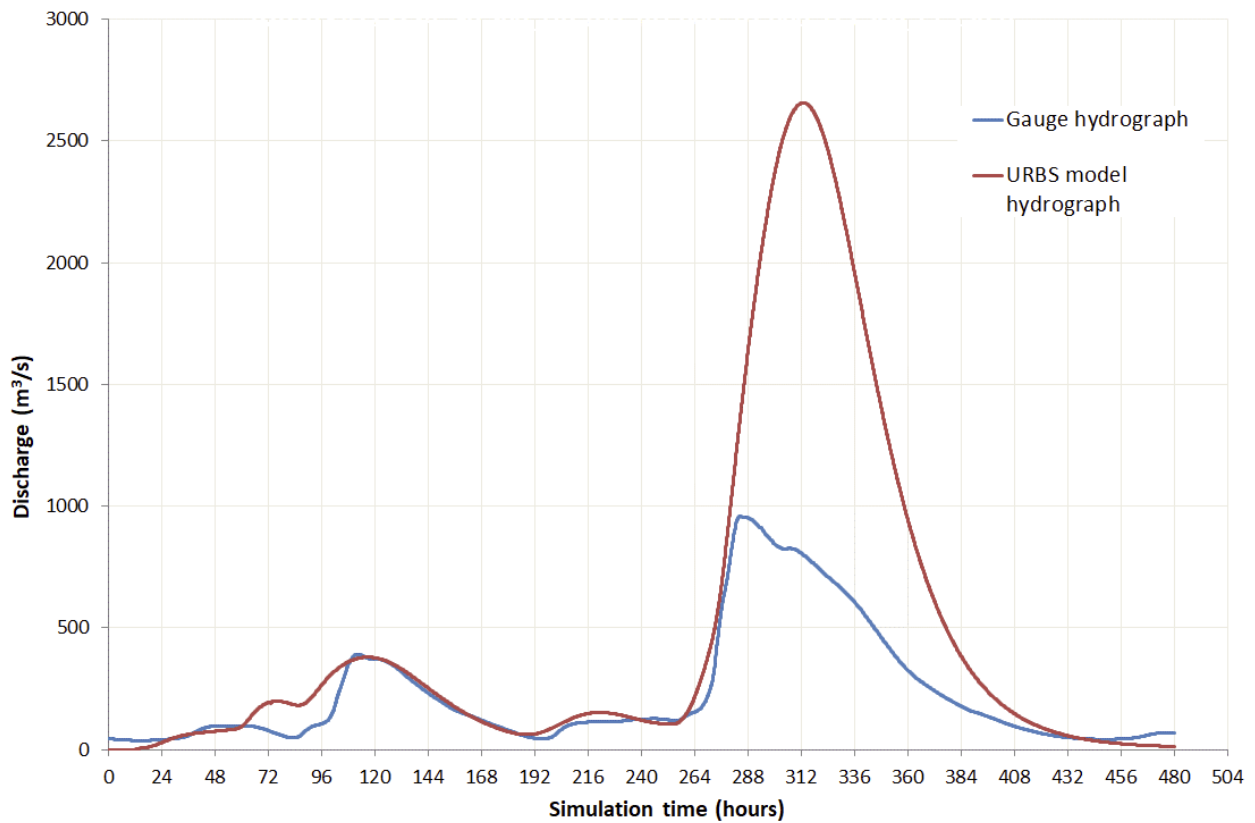


Figure 7.14 Calibration hydrographs at Condamine River at Tummalville 423323A—2010 event

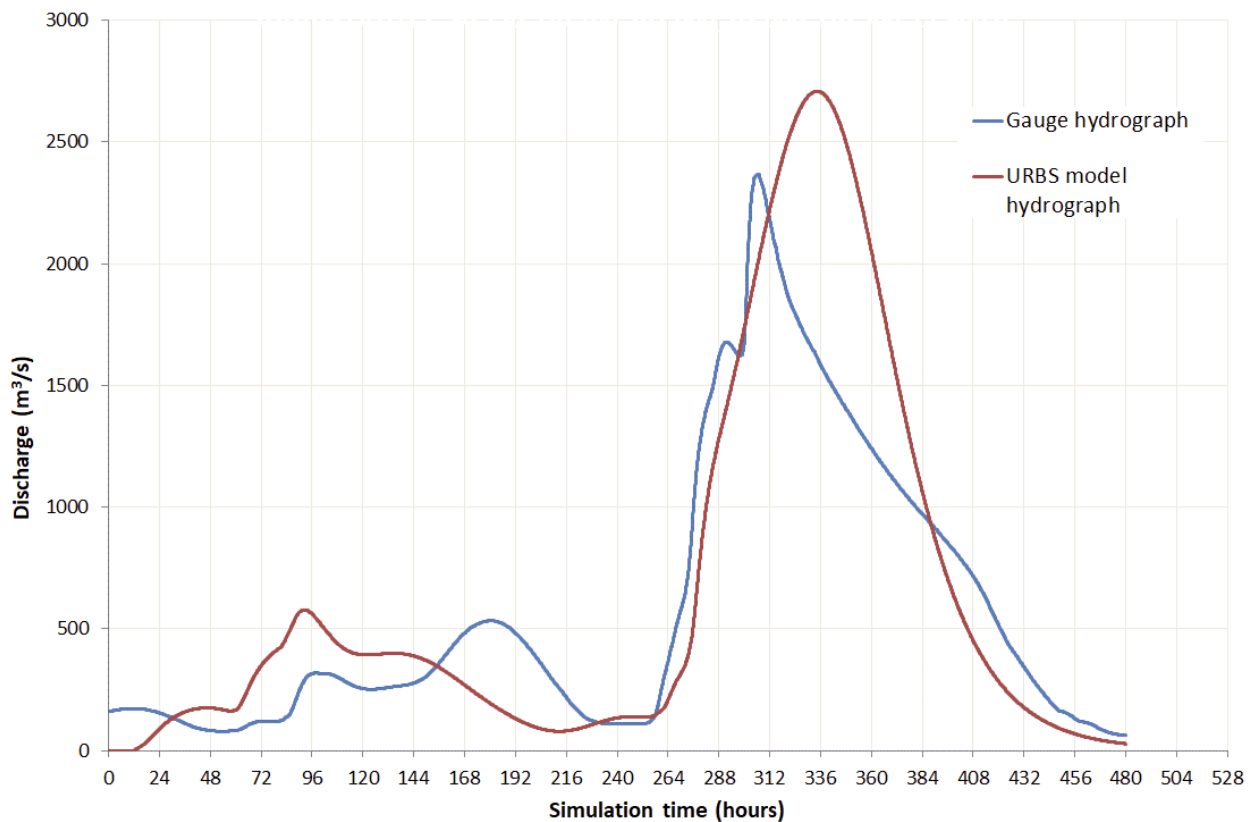


Figure 7.15 Calibration hydrographs at combined 'outlet'—2010 event

Table 7.7 Condamine River catchment—Summary of URBS model calibration parameters—1991 event

Parameter description		Sub-model A	Sub-model B	Sub-model C	Sub-model D
Channel lag parameter	α	0.15	0.12	0.30	0.26
Catchment lag parameter	β	2.80	2.75	2.00	2.00
Initial loss recovery factor	rf	0.10	0.11	0.10	0.10
Catchment non-linearity parameter	m	1.00	1.00	1.00	1.00
Initial loss	IL	91.0	70.0	50	4
Continuing loss	CL	3.45	4.00	3.0	0.5

Table 7.8 Condamine River catchment—URBS model calibration performance—1991 event

Gauge	Difference in peak flow	Difference in hydrograph volume	Difference in time to peak (hours)
Warwick 422310C	-5.8%	9.0%	2.20
Talgai TW 422355A	38.4%	-1.4%	-3.40
Tummaville 422323A	-3.8%	3.2%	16.50
OUTLET	-23.6%	14.0%	3.70

Table 7.9 Condamine River catchment—Summary of URBS model calibration parameters—2010 event

Parameter description		Sub-model A	Sub-model B	Sub-model C	Sub-model D
Channel lag parameter	α	0.15	0.12	0.30	0.13
Catchment lag parameter	β	2.80	2.75	2.00	2.00
Initial loss recovery factor	rf	0.10	0.11	0.1	0.10
Catchment non-linearity parameter	m	1.00	1.00	1.00	1.00
Initial loss	IL	18.0	78.5	25.5	20.5
Continuing loss	CL	2.00	0.23	1.15	1.80

Table 7.10 Condamine River catchment—URBS model calibration performance—2010 event

Gauge	Difference in peak flow	Difference in hydrograph volume	Difference in time to peak (hours)
Warwick 422310C	2.0%	9.3%	0.00
Talgai TW 422355A	5.0%	26.9%	-0.50
Tummaville 422323A	177.3%	109.8%	28.90
OUTLET	14.4%	9.8%	28.30

7.2.3.1 Discussion of calibration results

The primary aim of calibration in the upper catchment was to match the hydrologic model peak flows, hydrograph volumes and times to peak to those recorded at the gauge. A reasonably good calibration was achieved at Warwick and Talgai TW in the upper catchment, as shown in Figure 7.8, Figure 7.9, Figure 7.12 and Figure 7.13.

Aside from the inherent approximation of physical processes in any hydrologic model, the level of accuracy a calibration can achieve is largely governed by the quality and coverage of data available. As stated in Section 7.1.5 (stream gauges), the reliability of the gauge at Talgai TW is limited by its location upstream of the confluence with Dalrymple Creek and the potential for breakouts in that area. During both calibration events there is uncertainty surrounding how much flow Dalrymple Creek is contributing to the flood event.

The accuracy of results declines as the model moves past the Tummalville gauge into the Condamine floodplain and down to Cecil Weir and Lone Pine gauges. Due to the complex braided nature of the main Condamine floodplain, calibration of the hydrology model in this area tends to be unproductive. URBS, nor any hydrologic modelling package, cannot sufficiently capture flows breaking out and recombining during different flood events.

As an example of the complex flood behaviour around Tummalville, Figure 7.2, illustrates how a substantial proportion of any significant flood event can break out and completely bypass the Tummalville gauge in multiple locations. During the 2010 event, the peak flow at the Talgai TW gauge is higher than that recorded at Tummalville, which strongly suggests that either water is bypassing the gauge; flows derived at Tummalville are inaccurate; or some combination of both. Therefore, the validity in adopting flow data from the Tummalville gauge (especially in large events like 2010) is very low.

The complex flood behaviour continues for approximately 60 km until the Condamine River and the North Branch recombine 5 km downstream of Cecil Weir. Calibrating peak flow and timing of peak at Tummalville, Cecil Weir and Lone Pine was not pursued, and instead the focus shifted to the calibration of flood volumes, particularly at Cecil Weir and Lone Pine.

Calibrating flood volumes was feasible, and it was important to achieve a match in flood volumes since the flood water levels in the broad, flat floodplain are driven predominantly by volume. The URBS model calibrated well to flood volumes at the 'Outlet'. A flood hydrograph volume match within 10 per cent at the 'Outlet' was considered satisfactory.

The URBS model produces estimates of local runoff in the Condamine floodplain but all complex routing is determined by the hydraulic model; therefore, calibration to gauges in the floodplain was made using gauge heights in the 2D hydraulic model (Section 7.2.7).

Since there was negligible variability in routing parameters across calibration events for each sub-model, it was simple to adopt a single set of URBS parameters for the validation and design event hydrology. Only the channel lag parameter in Sub-model D differed between 1991 and 2010, but a test performed in the TUFLOW hydraulic model showed that the model results are not sensitive to that parameter in that part of the model. Table 7.11 summarises the URBS routing parameters that were adopted for the validation and design events.

Table 7.11 Condamine River—URBS routing parameters adopted for validation and design events

Parameter description		Sub-model A	Sub-model B	Sub-model C	Sub-model D
Channel lag parameter	α	0.15	0.12	0.30	0.13
Catchment lag parameter	β	2.80	2.75	2.00	2.00
Initial loss recovery factor	rf	0.10	0.11	0.10	0.10
Catchment non-linearity parameter	m	1.00	1.00	1.00	1.00

7.2.4 Model validation

The results of the 2013 event and 2021 event validations are shown in Figure 7.16 to Figure 7.23 (hydrographs) and Table 7.12 to Table 7.13 (model validation performance metrics). The validation results indicate that the hydrological model outputs using parameters adopted in Table 7.11 achieve a good match with the historical event.

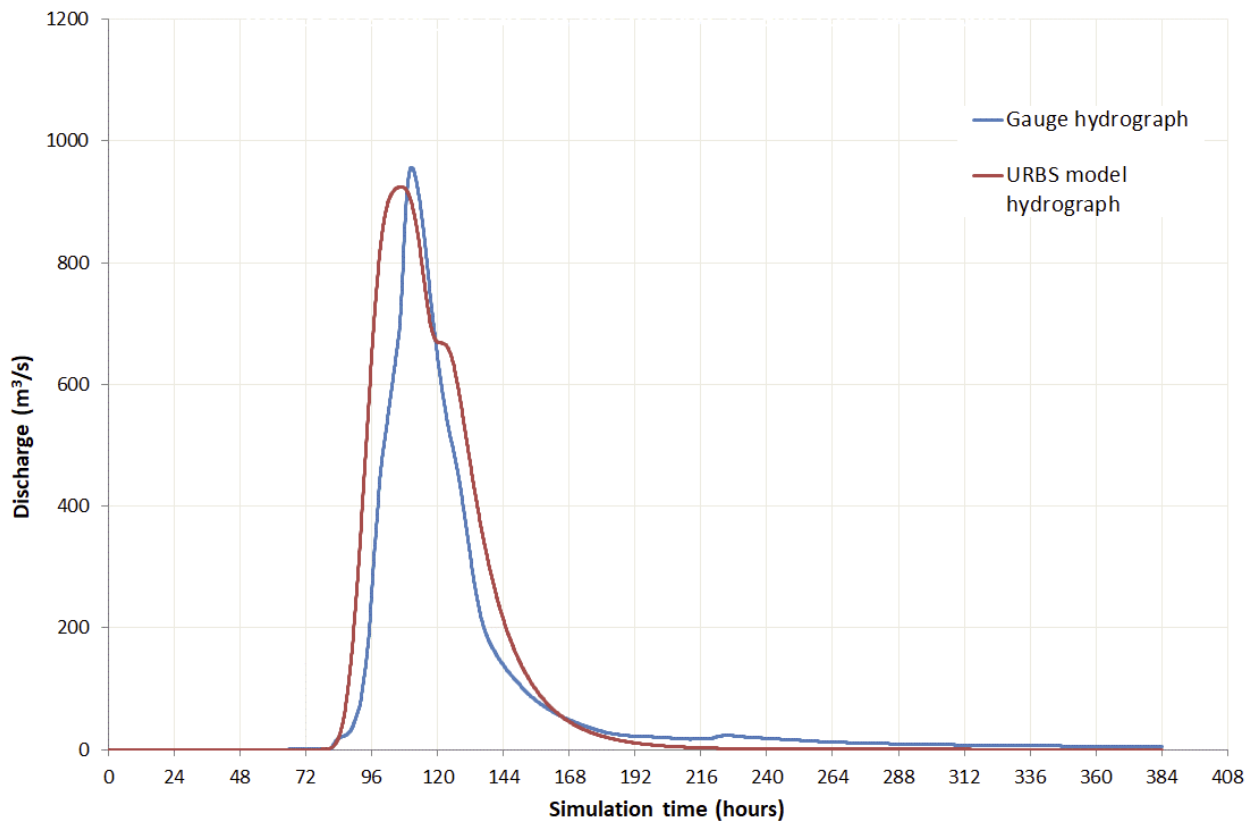


Figure 7.16 Validation hydrographs at Warwick 422310C—2013 event

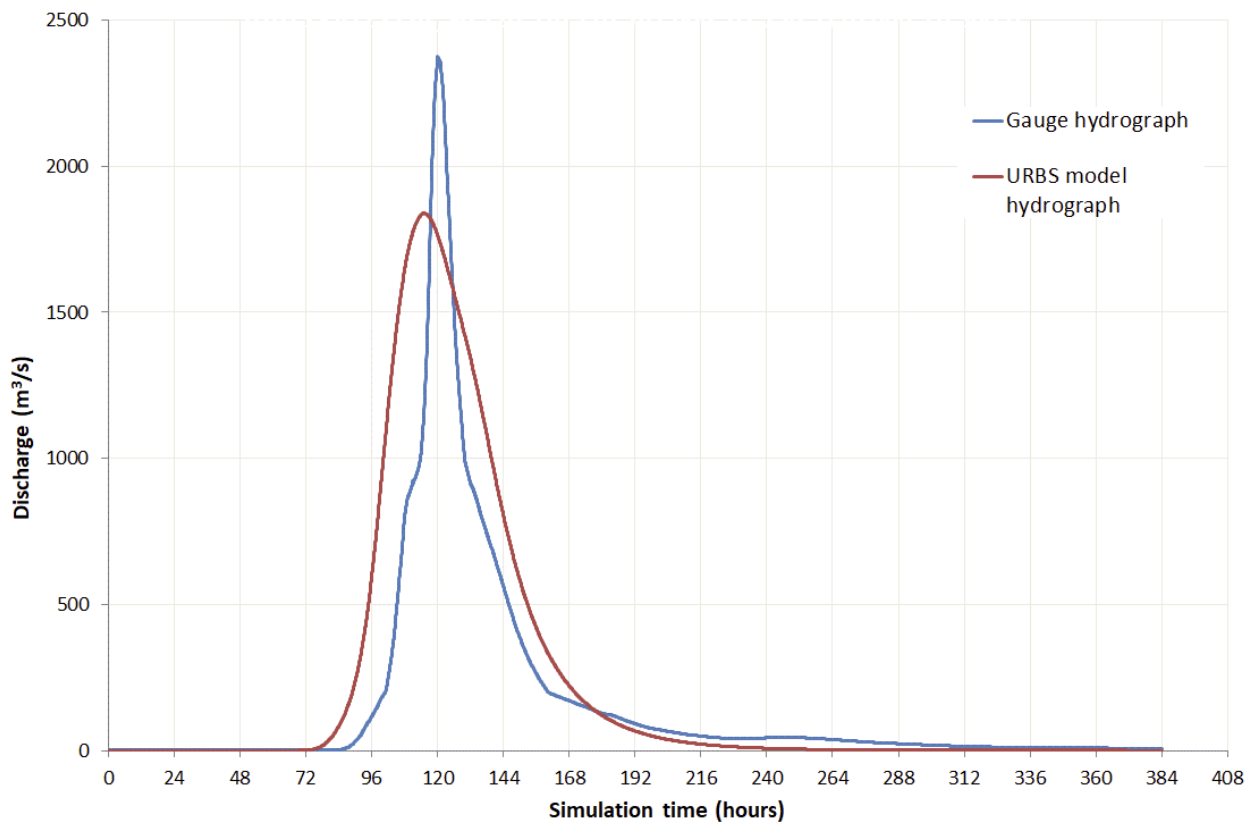


Figure 7.17 Validation hydrographs at Talgai TW 422355A—2013 event

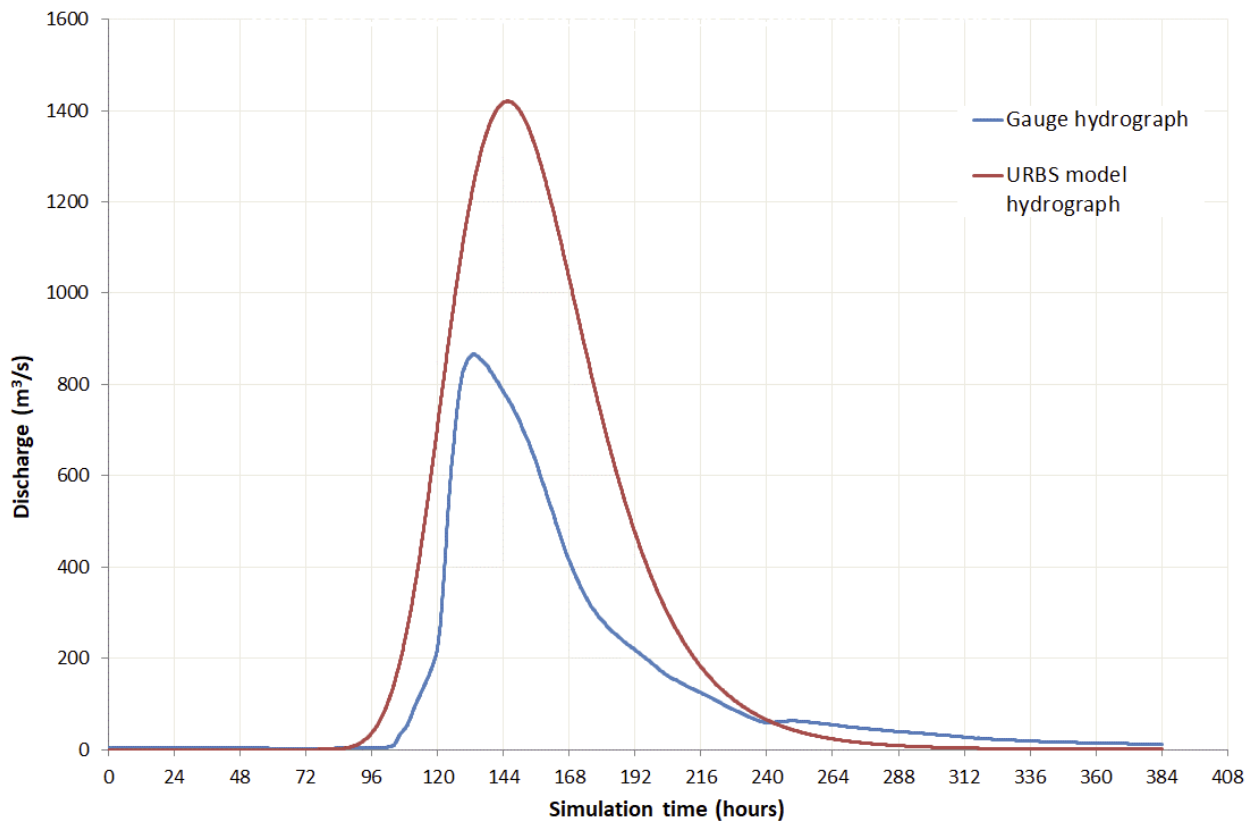


Figure 7.18 Validation hydrographs at Tumnaville 423323A—2013 event

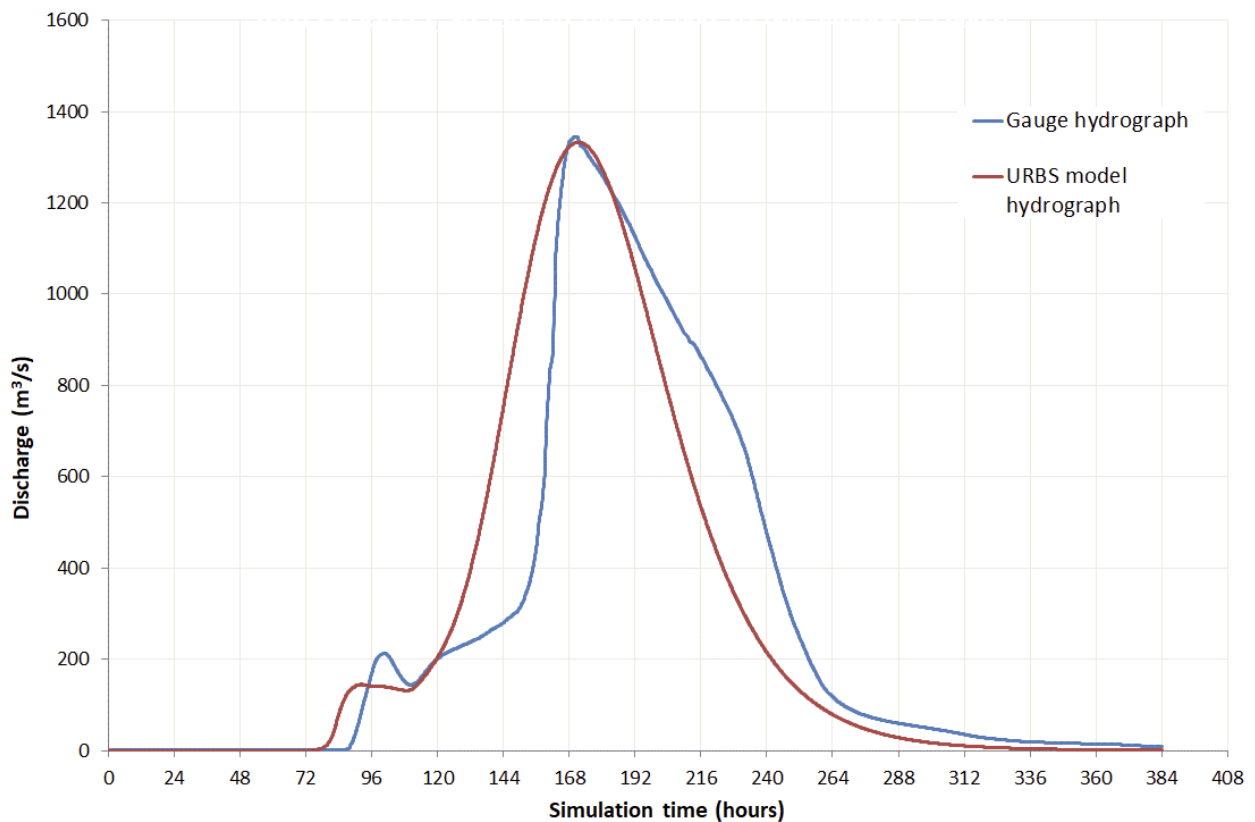


Figure 7.19 Validation hydrographs at combined 'outlet'—2013 event

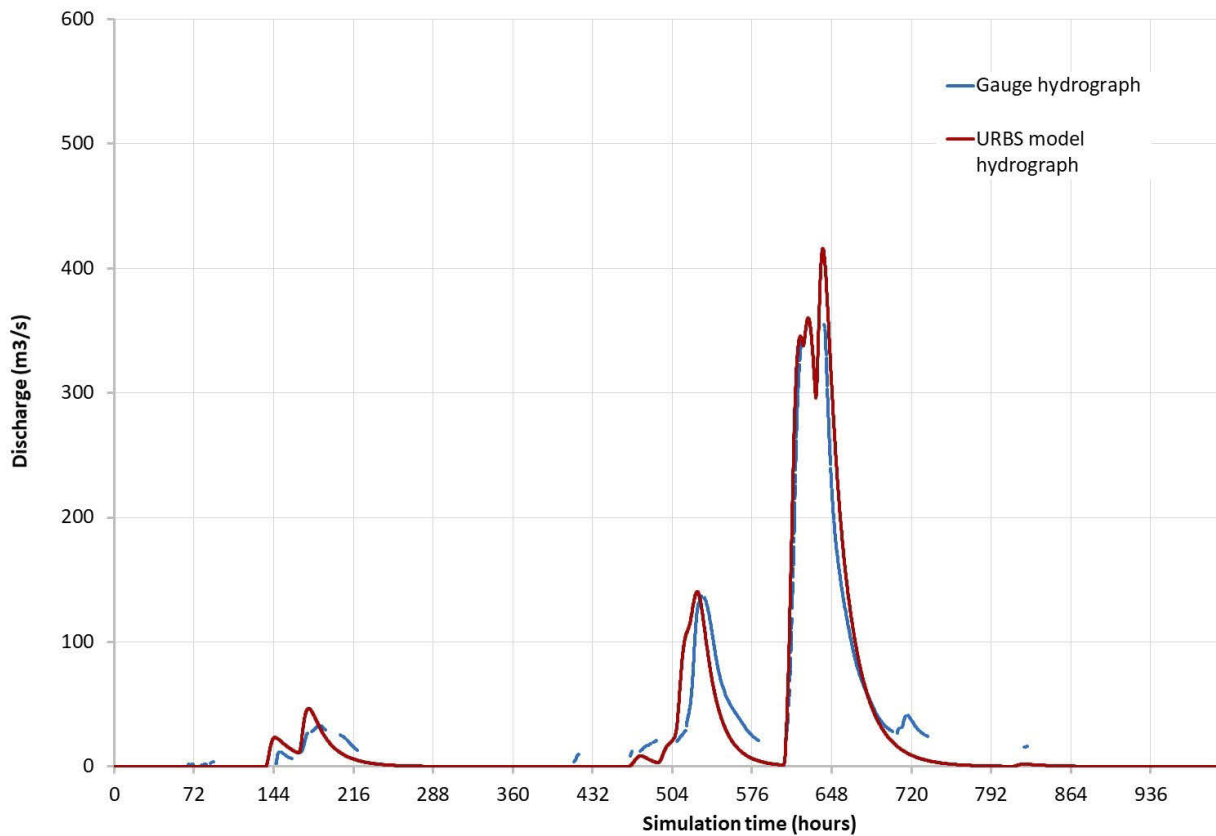


Figure 7.20 Validation hydrographs at Warwick 422310C—2021 event

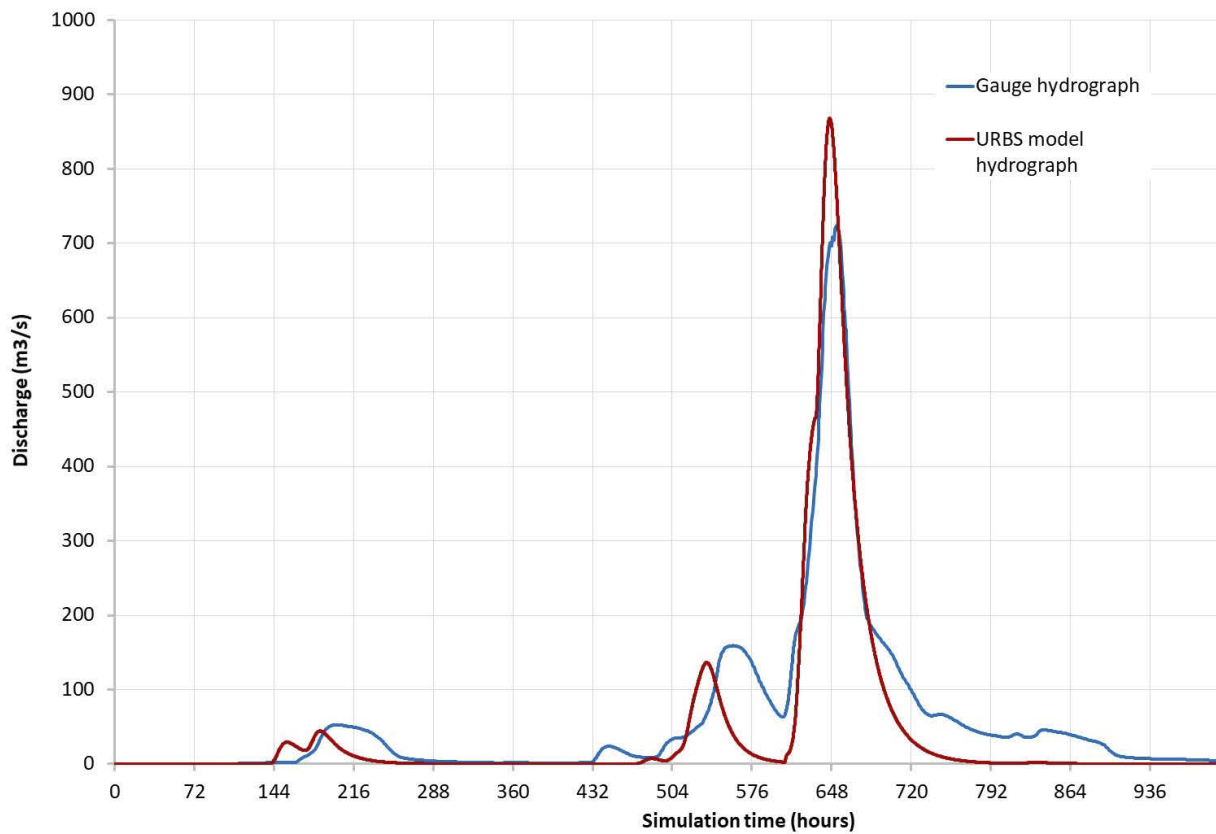


Figure 7.21 Validation hydrographs at Talgai TW 422355A—2021 event

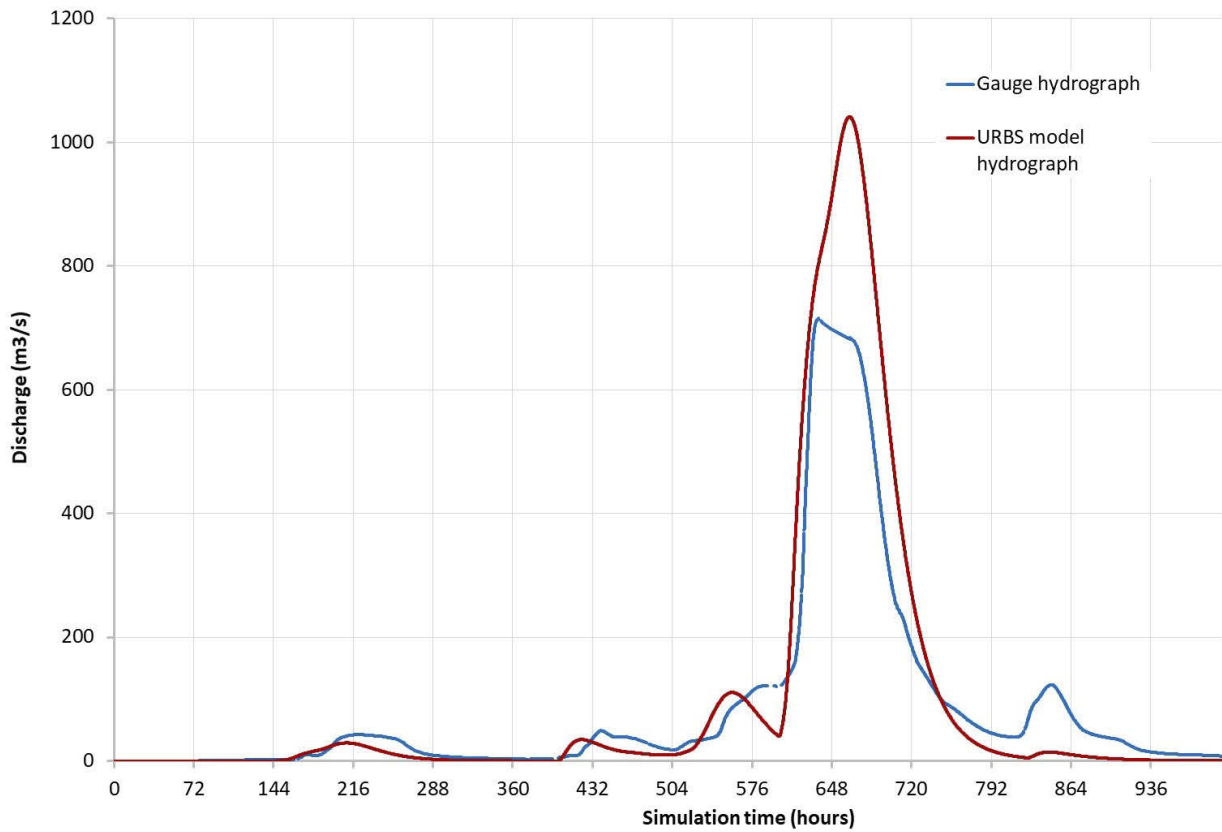


Figure 7.22 Validation hydrographs at Tumnaville 423323A—2021 event

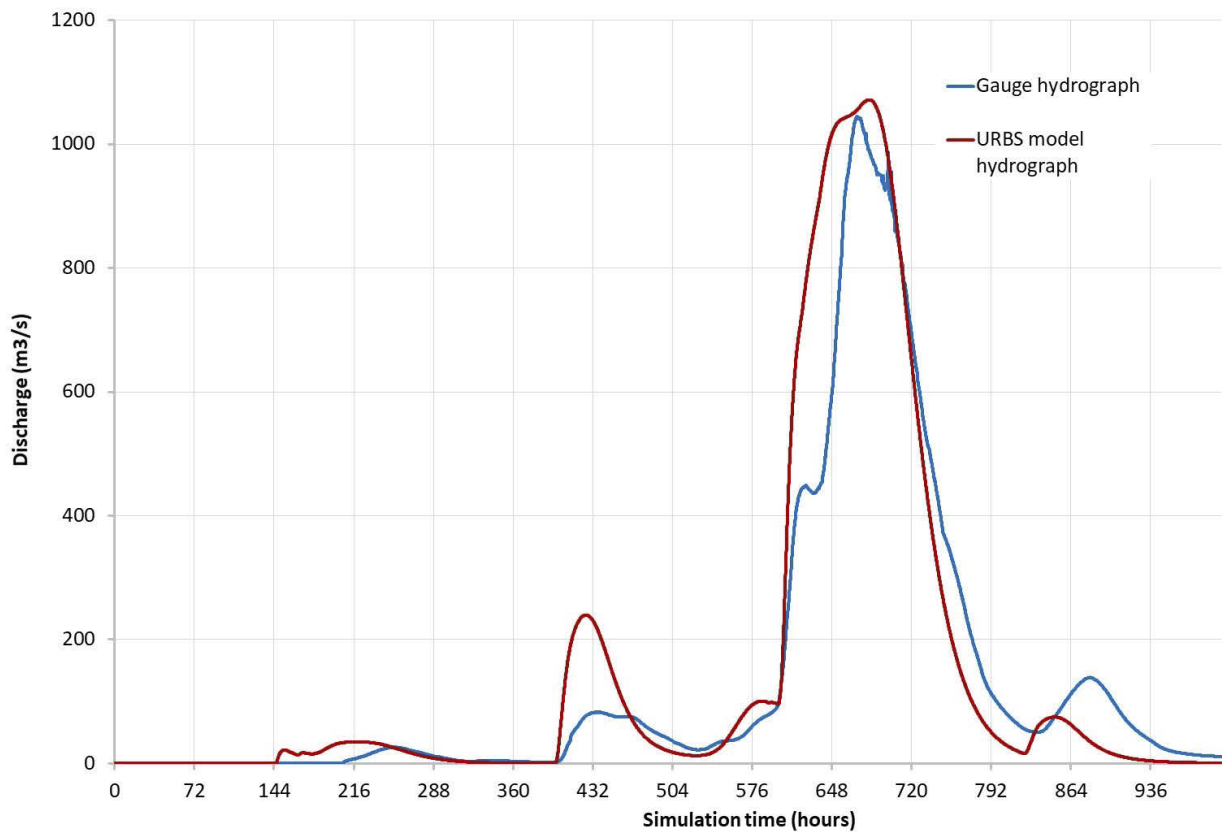


Figure 7.23 Validation hydrographs at combined 'outlet'—2021 event

Table 7.12 Condamine River—URBS model validation performance metrics—2013 event

Gauge	Difference in peak flow	Difference in hydrograph volume	Difference in time to peak (hours)
Warwick 422310C	-3.3%	22.4%	-4.40
Talgai TW 422355A	-22.6%	34.5%	-4.90
Tummaville 422323A	63.9%	82.7%	12.50
OUTLET	-0.9%	-2.4%	0.30

Table 7.13 Condamine River—URBS model validation performance metrics—2021 event

Gauge	Difference in peak flow	Difference in hydrograph volume	Difference in time to peak (hours)
Warwick 422310C	-17%	17%	11
Talgai TW 422355A	-20%	29%	6
Tummaville 422323A	-46%	-12%	14
OUTLET	-3%	-8%	12

7.2.5 Design event parameters

Hydrologic information to assist estimation of design event flows was sourced from the ARR Data Hub as summarised in Table 7.14.

Table 7.14 Condamine River—Summary of URBS model design event inputs

Input parameter	Remarks
Design rainfall	Intensity-Frequency-Durations for each sub-catchment were downloaded from the BoM's website to simulate the relatively high variation in rainfall across the catchment.
Extreme event rainfall	Probable Maximum Precipitation depths for durations up to 6 hours (for use in modelling the PMF event) were obtained using the method presented in the Bulletin 53 (BoM, 2003). The rainfall depths for the 1 in 10,000 AEP event were estimated using the interpolation method presented in ARR 2019 Book 8 Section 3.5.
Losses	Rainfall loss parameters were downloaded from the ARR Data Hub for each sub-model. Sub-model A – IL: 31 mm, CL: 3.0 mm/hr Sub-model B – IL: 25 mm, CL: 1.7 mm/hr Sub-model C – IL: 37 mm, CL: 1.0 mm/hr Sub-model D – IL: 46 mm, CL: 0.4 mm/hr
Areal reduction factor	Parameters were adopted for the semi-arid inland Queensland region. The catchment area U/S of the proposed rail crossing on the Condamine River is approximately 7,064 km ² , which yields an ARF between 78.5% and 86.3% depending on design storm event AEP and duration.
Ensemble TPs	As the study catchment area exceeds 75 km ² , the standard ensemble rainfall patterns from ARR 2019 do not apply to this catchment. These were replaced with the areal TPs for the Central Slopes region.
Pre-burst depths	Median pre-burst depths were downloaded from the ARR Data Hub for each sub-catchment. Pre-burst depths vary by design storm event AEP and duration. Pre-burst depths were applied to the model by reducing the initial losses for each storm event.

The calibrated URBS model was set up to run the 10 ensemble TPs for the 9 AEPs (50%, 20%, 10%, 5%, 2%, 1%, 1 in 2,000, 1 in 10,000, PMF) and 7 storm durations (720, 1440, 2880, 4320, 5760, 7200, 8640 minutes), which amounted to 560 individual design storms per sub-model area. Each storm was passed from one sub-model to the next in the series.

Initially, the URBS model was run with the loss values obtained from the ARR Data Hub. The losses derived during calibration were not adopted since it was difficult to confidently judge antecedent conditions and the true magnitude of the losses in each calibration event.

The results of the 560 design storms were statistically analysed to determine which TP generated the fifth-highest (Rank-5) flow for each duration and AEP. Then the adopted design flow for a given AEP was the maximum value across the corresponding Rank-5 flows. The results informed how much these losses needed to be scaled (up or down) to reconcile the differences between FFAs and the flows estimated by the URBS model at the gauges.

The design event initial and continuing losses were selected based on the ARR Datahub (to account for regional conditions) and verified against the at-site FFA (to account for local conditions). Adjustments to rainfall losses were made only in Sub-model A to reconcile modelled flows to the FFA at Warwick which had the highest reliability. Several iterations were necessary to reconcile differences to less than 5 per cent for events smaller than the 1% AEP, and to less than 10 per cent for events larger than the 1% AEP. The broader tolerance for the rarer, larger events reflects the larger uncertainty in the FFAs at those AEPs. Although it is expected that initial and continuing losses for calibration events are of similar magnitude to design event losses, the calibration losses were not adopted for design events as calibration losses do not account for probabilities of antecedent conditions given a particular AEP.

Figure 7.24 and Table 7.15 summarise the required adjustments to the loss values in order to achieve a good match to the FFAs.

Figure 7.25 and Figure 7.26 display how the design event flows plot against each FFA and Table 7.16 and Table 7.17 summarise the differences. Note that due to the much higher reliability of the FFAs at Warwick and Cecil Weir, compared to the FFAs at Talgai TW and Tummaville, reconciliation was performed at an upstream pivot (Warwick) and a downstream pivot (URBS model outlet) only. At the URBS model outlet, the combined FFA from Cecil Weir and Lone Pine was used because of the low FFA reliability at Lone Pine, which is discussed separately in Section 7.2.7.

Floodfrequency analysis was not undertaken at Lemon Tree Weir, Centenary Bridge, Talgai and Yarramalong Weir due to the short periods of record. Floodfrequency analysis was not undertaken at Tummaville and Pampas as the rating curves were unreliable and influenced by cross-catchment interaction.

As an additional check, DTMR's Quantile Regression Technique (QRT) was used. Although QRT is applicable only to catchments up 1,000 km², it serves as a reasonable, arbitrary point of comparison.

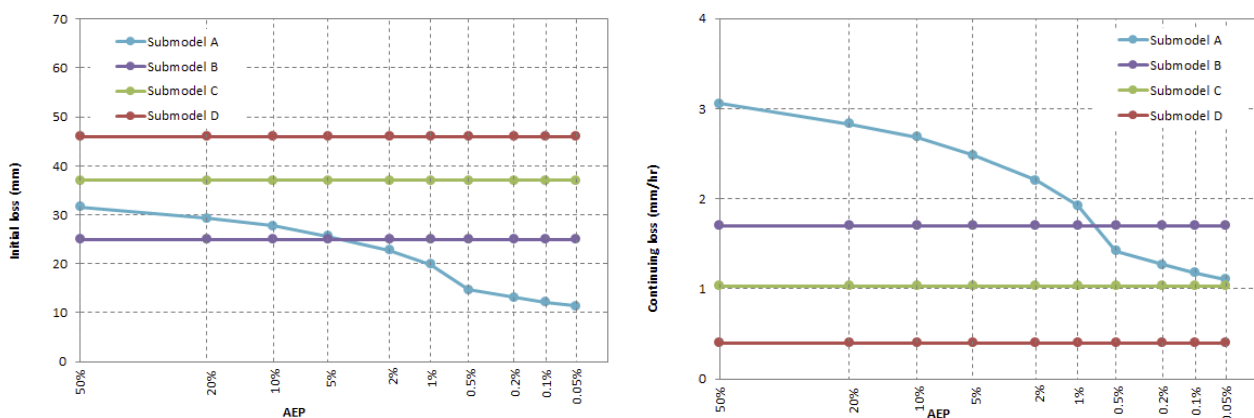


Figure 7.24 Condamine River—Adjusted design event losses

Table 7.15 Condamine River—Adjusted design event losses

AEP		ARR Data Hub value	50%	20%	10%	5%	2%	1%	1 in 2,000
Sub-model A	IL	31.0	31.57	29.28	27.74	25.68	22.77	19.88	11.40
	CL	3.0	3.05	2.83	2.68	2.49	2.20	1.92	1.10
Sub-model B	IL	25.0	25.00	25.00	25.00	25.00	25.00	25.00	25.00
	CL	1.7	1.70	1.70	1.70	1.70	1.70	1.70	1.70

AEP		ARR Data Hub value	50%	20%	10%	5%	2%	1%	1 in 2,000
Sub-model C	IL	37.0	37.00	37.00	37.00	37.00	37.00	37.00	37.00
	CL	1.0	1.03	1.03	1.03	1.03	1.03	1.03	1.03
Sub-model D	IL	46.0	46.00	46.00	46.00	46.00	46.00	46.00	46.00
	CL	0.4	0.40	0.40	0.40	0.40	0.40	0.40	0.40

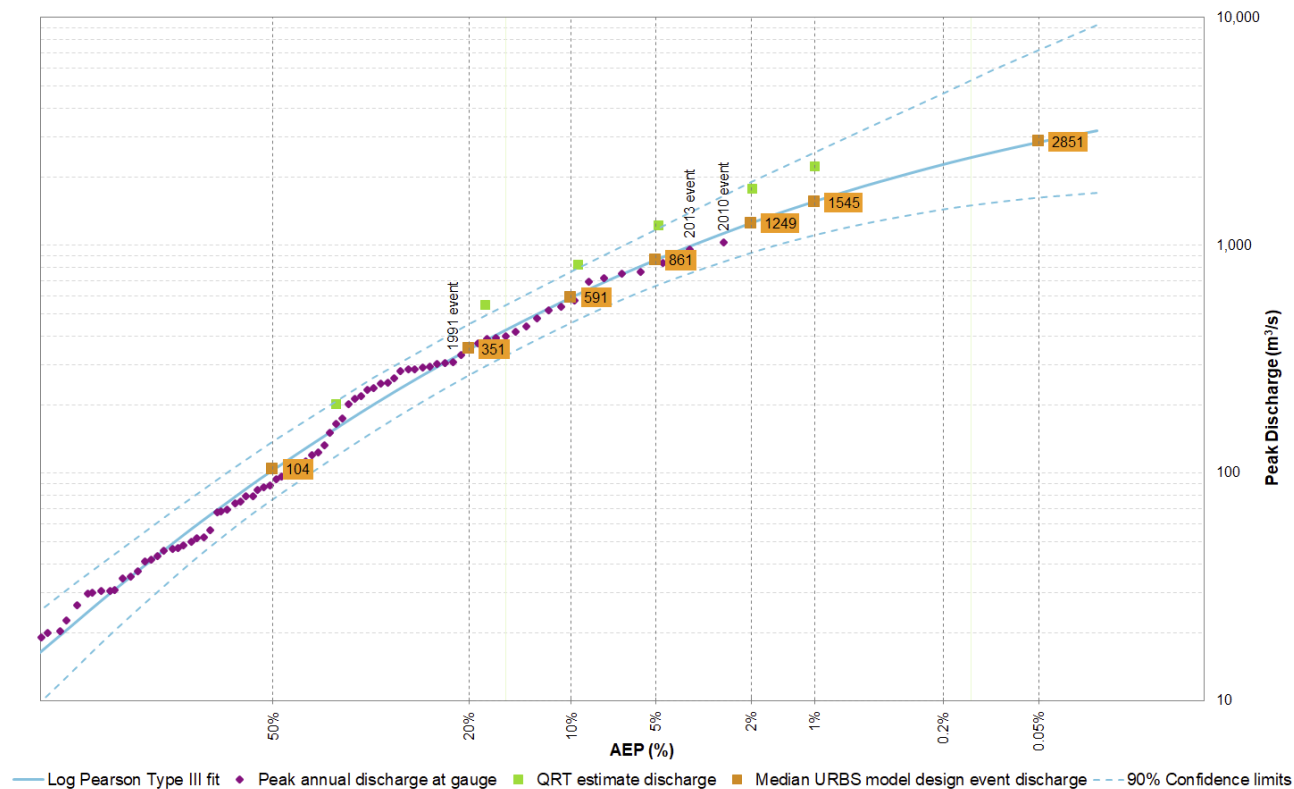


Figure 7.25 Flood frequency analysis at Warwick (422310C)

Table 7.16 Data output from flood frequency analysis at Warwick (422310C)

AEP	FFA estimate – lower bound 90% confidence interval (m³/s)	FFA – estimate of flow (m³/s)	FFA estimate – upper bound 90% confidence interval (m³/s)	DTMR quantile regression technique (m³/s)	URBS model flows (m³/s)	Difference between FFA estimate and URBS model
50%	77	103	138	-	104	0.5%
20%	269	349	452	-	351	0.7%
10%	457	589	767	-	591	0.3%
5%	664	861	1,176	1,224	861	0.0%
2%	926	1,250	1,892	1,767	1,249	-0.1%
1%	1,106	1,554	2,559	2,213	1,545	-0.6%
1 in 2,000	1,616	2,826	7,213	-	2,851	-1.2%

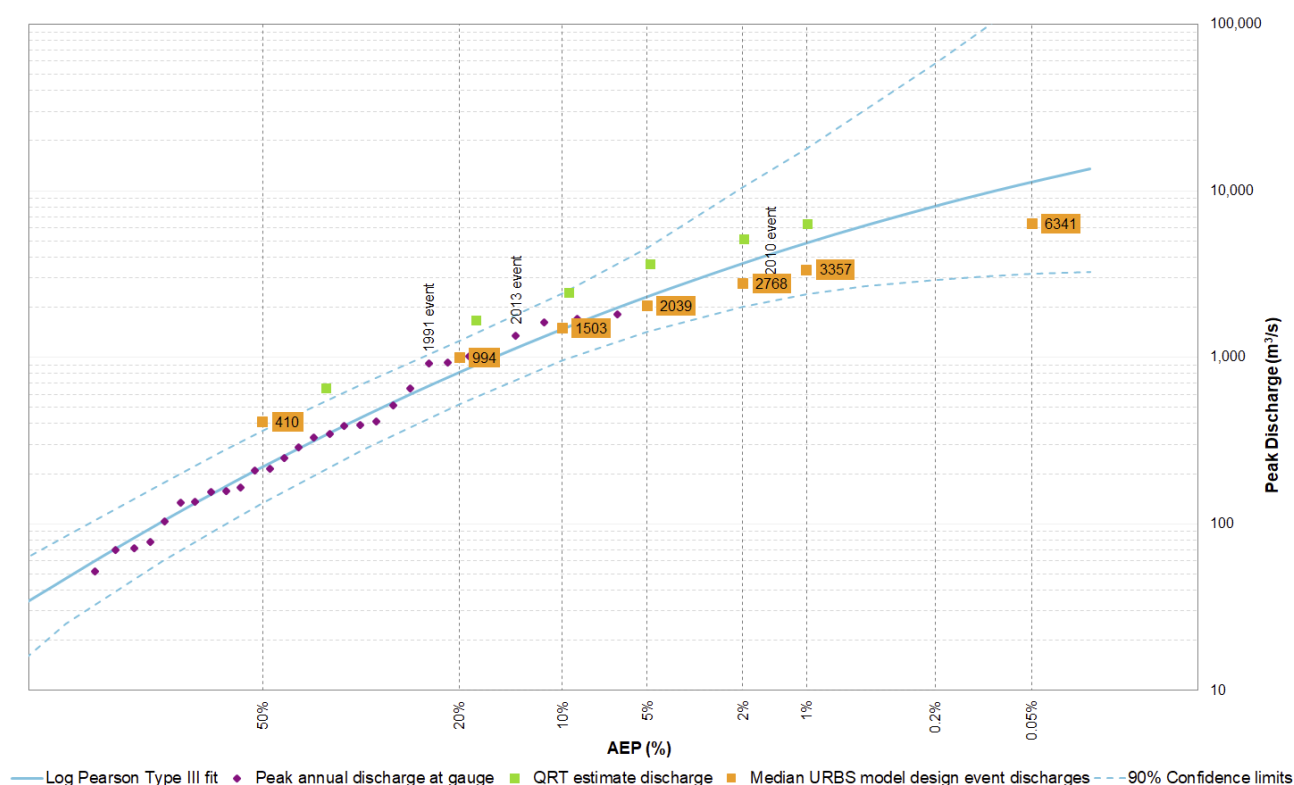


Figure 7.26 Condamine River—Combined flood frequency analysis at URBS model outlet (422316A and 422345A)

Table 7.17 Condamine River—Data output from combined flood frequency analysis at URBS model outlet

AEP	FFA estimate – lower bound 90% confidence interval (m³/s)	FFA – estimate of flow (m³/s)	FFA estimate – upper bound 90% confidence interval (m³/s)	DTMR quantile regression technique (m³/s)	URBS model flows (m³/s)	Difference between FFA estimate and URBS model
50%	133	221	362	-	410	+85.7%
20%	524	817	1,256	-	994	+21.7%
10%	959	1,483	2,437	-	1,503	+1.4%
5%	1,432	2,323	4,565	3,624	2,039	-12.2%
2%	2,020	3,681	10,472	5,139	2,768	-24.8%
1%	2,380	4,878	17,968	6,354	3,357	-31.2%
1 in 2,000	3,173	11,348	152,272	-	6,341	-44.1%

7.2.6 Design hydrograph selection

Three storm durations were selected for each AEP for testing in the TUFLOW model to account for potential subtle differences between design event hydrographs causing localised differences to the flood hydraulics.

The critical durations that produced the highest peak flow tended to be between 2,880 and 7,200 minutes, while the critical durations that produced the highest flood volume tended to be between 2,880 and 8,640 minutes. Storm event durations of 2,880, 4,320 and 5,760 minutes were chosen as their flood peaks and volumes were consistently among the highest for all AEP at locations within the hydraulic model domain.

A single TP for each critical duration was selected for hydraulic simulation. The critical TP was chosen, which produced the rank 6 (one above the median) peak flow at Tumnaville in URBS.

The URBS results used for design event selection were taken at the outlet of Sub-model C (Tummaville) as this is the last point of reference upstream of the Project alignment and upstream of the braided floodplain. The peaks and the TP chosen for design event hydrographs are summarised in Table 7.18.

As recommended by the Expert Flood Panel (Appendix A), sensitivity tests were performed using the draft EIS hydraulic model to verify that the hydrologically selected storm duration and TPs are also representative for hydraulic design and impact assessment. Verification of the selected storm durations is discussed in Section 7.2.6.1. Verification of the selected single TP is discussed in Section 7.2.6.2.

Table 7.18 **Condamine River—Summary of selected design event hydrographs**

AEP	Peak flow (m³/s)			Ensemble pattern (0 to 9)		
	Storm duration (minutes)					
	2,880	4,320	5,760	2,880	4,320	5,760
50%	284	284	296	6	6	3
20%	761	752	744	0	6	3
10%	1,147	1,111	1,152	0	4	9
5%	1,553	1,530	1,590	0	0	9
2%	2,223	2,148	2,182	4	0	9
1%	2,765	2,655	2,707	4	0	7
1 in 2,000	4,927	5,186	5,421	5	7	7

7.2.6.1 Verification of critical duration selection

Three storm durations (2,880, 43,20 and 5,760 minutes) were selected for simulation in the TUFLOW hydraulic model. These storm durations were selected based on the durations in the hydrologic model that produced the highest peak flows and volumes. As the hydrologically modelled catchment area to the Project alignment (~7,000 km²) is an order of magnitude larger than the hydraulically routed catchment area to the Project alignment (~500 km²), the hydrologic model is expected to provide reasonable and robust critical duration estimates for the hydraulic model.

To verify that the hydrologically selected critical durations are also hydraulically representative, five storm durations between 1,440 minute and 7,200 minute storm durations for the 1% AEP event were simulated in the hydraulic model. Figure 7.27 presents the test results, which confirm that a single duration (i.e. 2,880 minutes) within the selected range of storms was indeed critical along the Project alignment and on most of the adjacent floodplain. A longer duration in the selected range (5,760 min) was critical along the slower-moving North Branch.

The above assessment indicates that the adopted range of storm durations are hydraulically representative, and suitable for hydraulic design and impact assessment. As part of the ongoing model strengthening, an additional long storm duration (i.e. 7,200 minutes) will be considered in the hydraulic design and assessment of flood impact in the detailed design stage for the 2% AEP and more frequent events that may experience slightly longer critical durations.

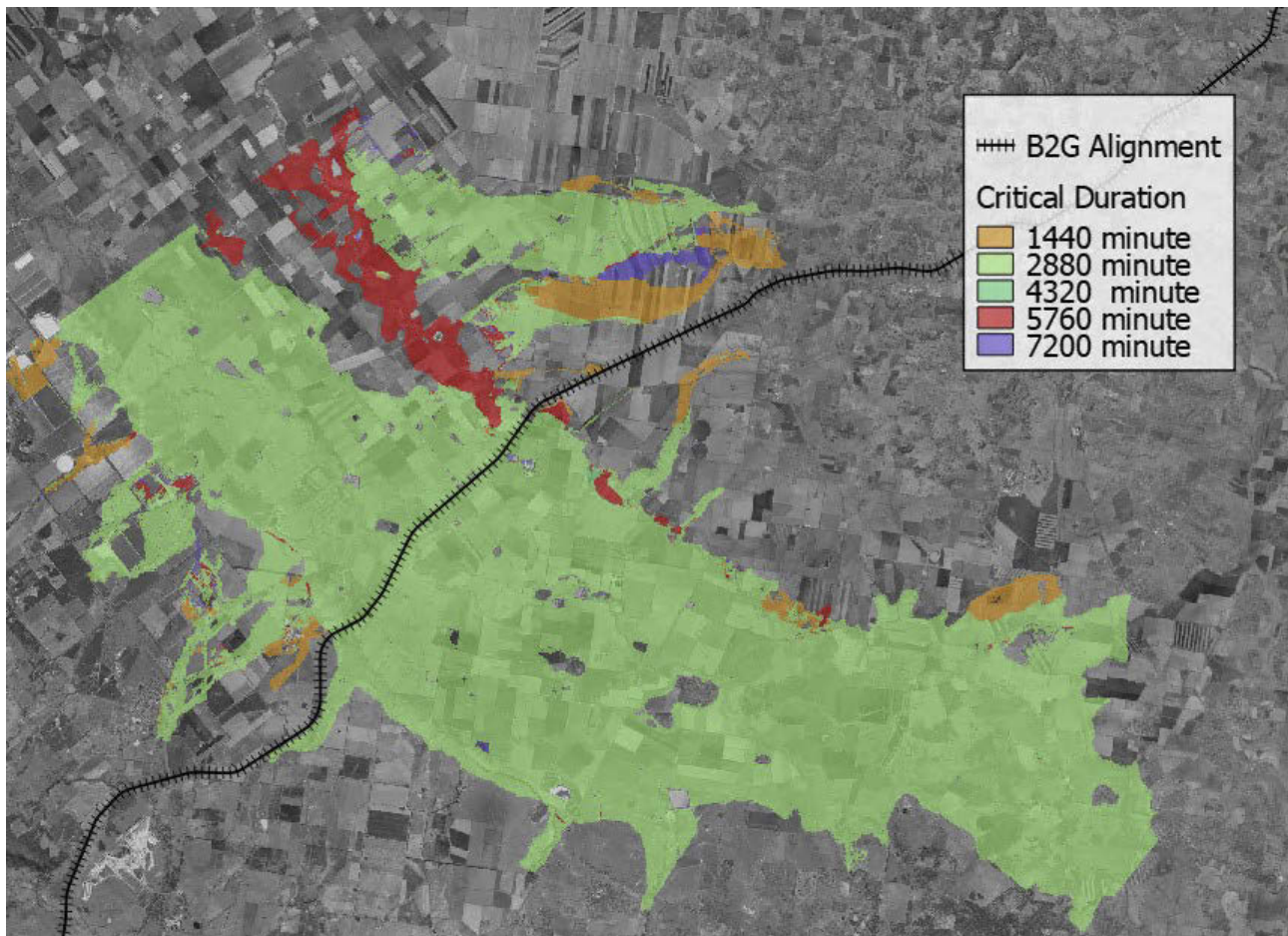


Figure 7.27 Condamine River—Critical durations for the 1% AEP event

7.2.6.2 Verification of critical temporal pattern selection

Simulating all 10 design TPs for each storm duration in the hydraulic model while maintaining a good resolution for impact assessment was not considered a practical approach due to the significant catchment scale and relative insensitivity of volume-driven flooding behaviours to TPs. The selection of a single TP per critical duration for the purposes of improving simulation efficiency is described in Book 2, Chapter 5.9.2 of ARR 2019.

To assess the sensitivity of flood levels to TPs, all 10 design TPs were simulated in the hydraulic model for the 1% AEP 2880-minute event, which dominates the Condamine River floodplain. The results are shown in Figure 7.28 and Figure 7.29. For most of the floodplain and within the vicinity of the Project alignment, TP one was selected as the critical temporal pattern. This is not consistent with the TP selected in URBS (TP four); however, the URBS TP was generally higher by 0 to 50 mm as shown in Figure 7.29. These differences are minor, and the adopted approach for the design modelling is considered to be slightly conservative.

These results suggest that the critical TP is likely to be consistent across the area of interest and the selection of a single TP in URBS is a reasonable approximation.

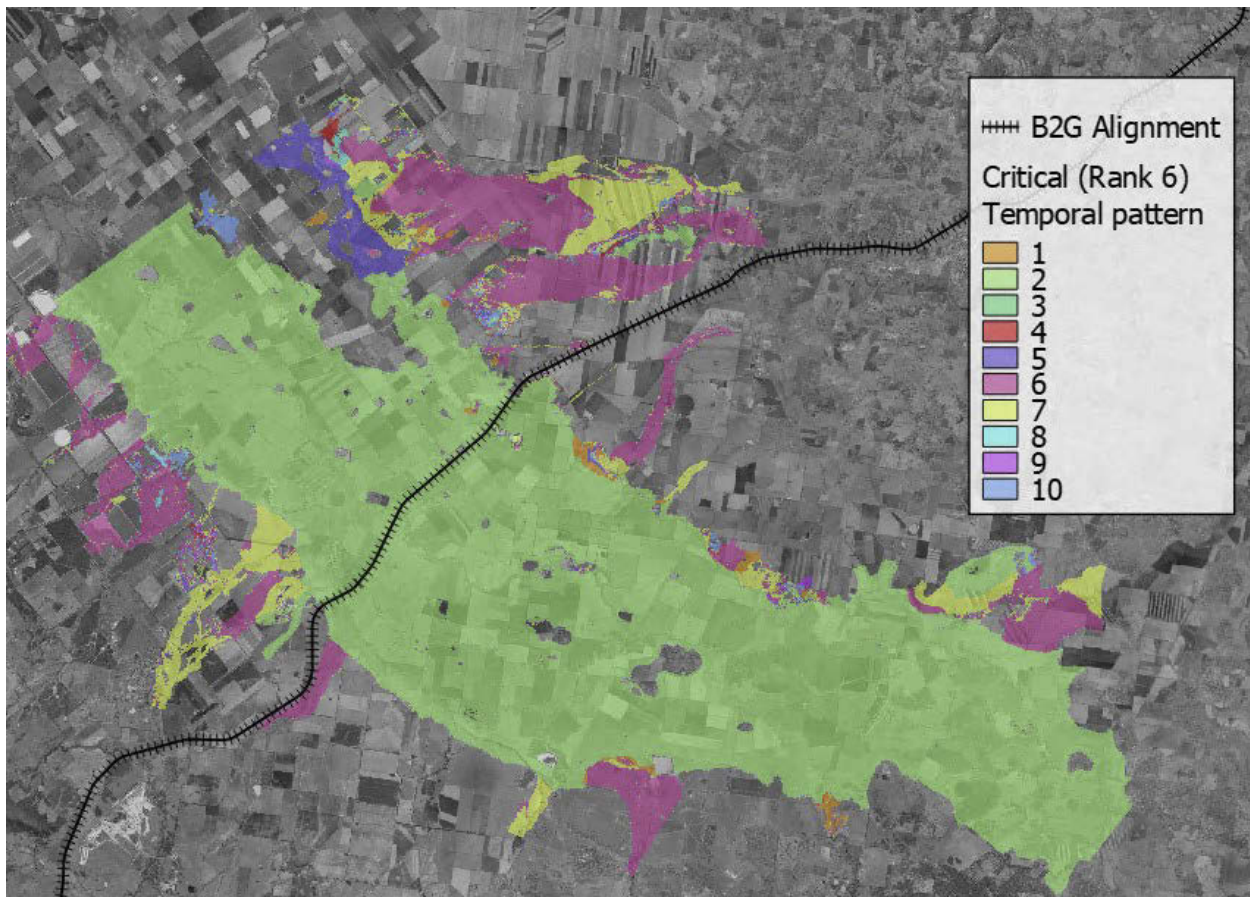


Figure 7.28 Condamine River—Selected temporal patterns for the 1% AEP 2880-minute event (TUFLOW)

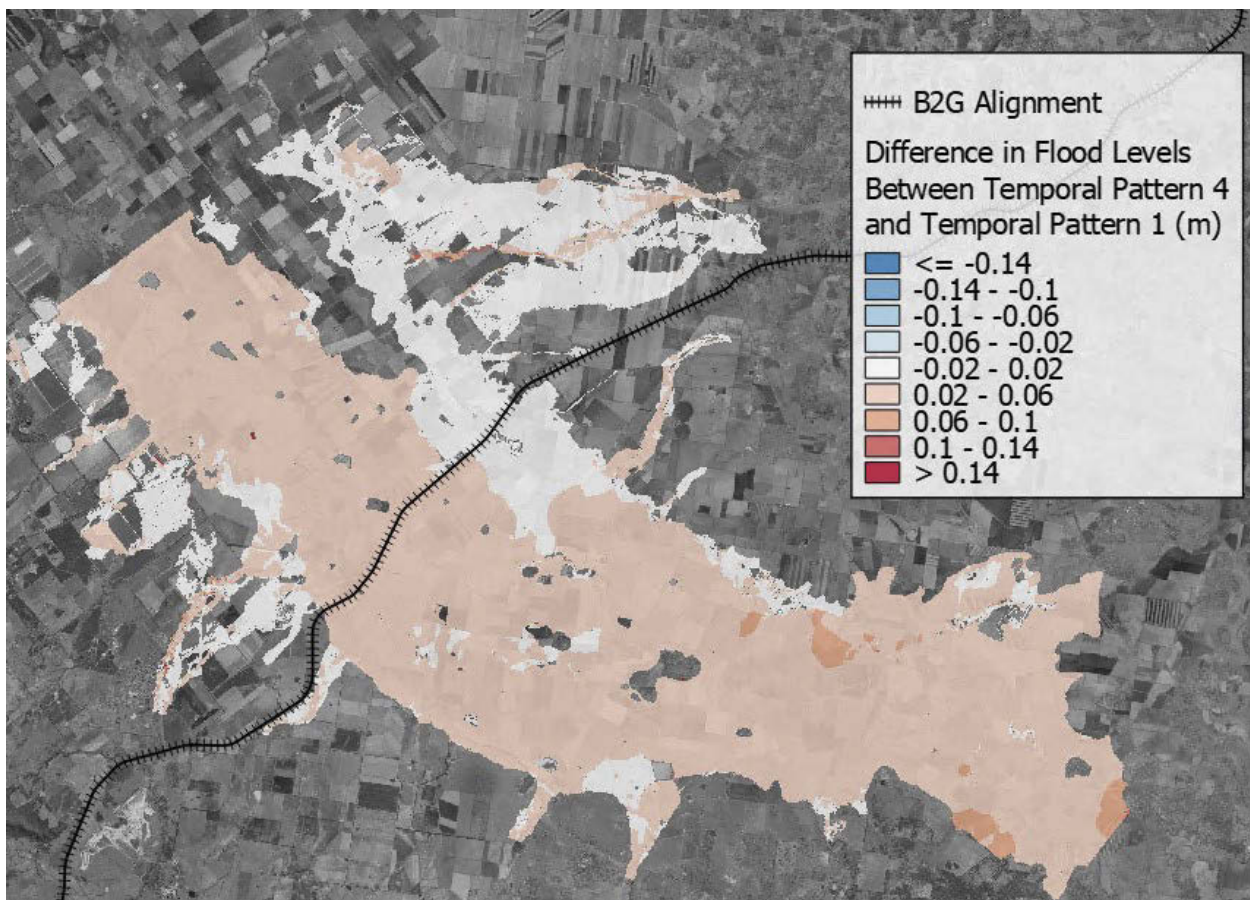


Figure 7.29 Condamine River—Flood level differences for the 1% AEP 2880-minute event: Temporal pattern 4 (selected in URBS); Temporal pattern 1 (selected in TUFLOW)

7.2.7 Flow distribution between Condamine River and North Branch

The hydrologic model was calibrated to a combined outlet due to the significantly better quality gauging at Cecil Weir. A comparison between hydrologic model output and gauging at Lone Pine is considered useful for reviewing whether calibrating to the respective gauges (i.e. Cecil Weir and Lone Pine) would improve the estimate of the flow distribution between Condamine River and North Branch.

Lone Pine is a very small gauge used for monitoring low flows, which has been gauged up to 3.47 m at 21 m³/s and has a comparatively short record length (only 43 years compared to 74 years at Cecil Weir and 98 years at Warwick). The flood level records at the gauge are considered suitable for model output comparison as no instrument malfunction or significant changes to the channel geometry was identified from a desktop review.

On 28 December 2010, the gauge recorded the highest stage at 6.37 m, which was significantly above the second highest (1996 at 5.69 m) and the third highest (1988 at 5.43 m) floods on record, and therefore would have a significant effect on the estimation of flood frequency. The 2010 flood was also significantly higher than the low channel level (at approximately 3.7 m) and the highest surveyed overbank level (at approximately 5.2 m); therefore, a hydraulic assessment is required to improve the rating-curve confidence at Lone Pine.

The rating curve at Lone Pine was reassessed using a fine-scale (5 m) TUFLOW model for the extent shown in Figure 7.30, which includes both flow through and that which bypasses the gauge. Sensitivity tests found that the hydraulic model setup, including orientation and resolution, did not affect the adopted rating, and only the rising limb was sensitive to the hydrograph shape. The revised rating curve is plotted alongside the official Water Monitoring Information Portal rating in Figure 7.31. Water Monitoring Information Portal is a portal that provides latest and historical river height and stream data, supplied by the DRDMW.

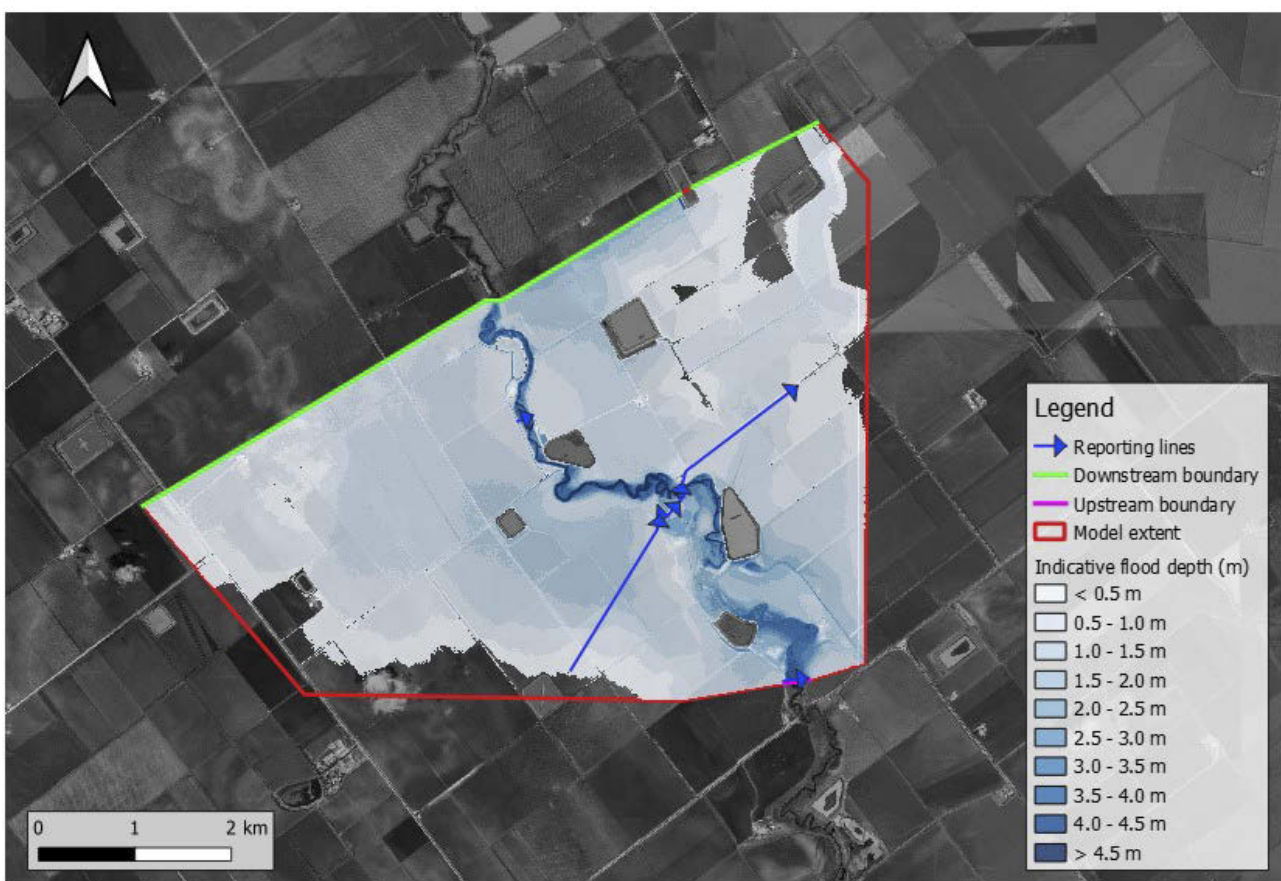


Figure 7.30 Lone Pine TUFLOW model configuration

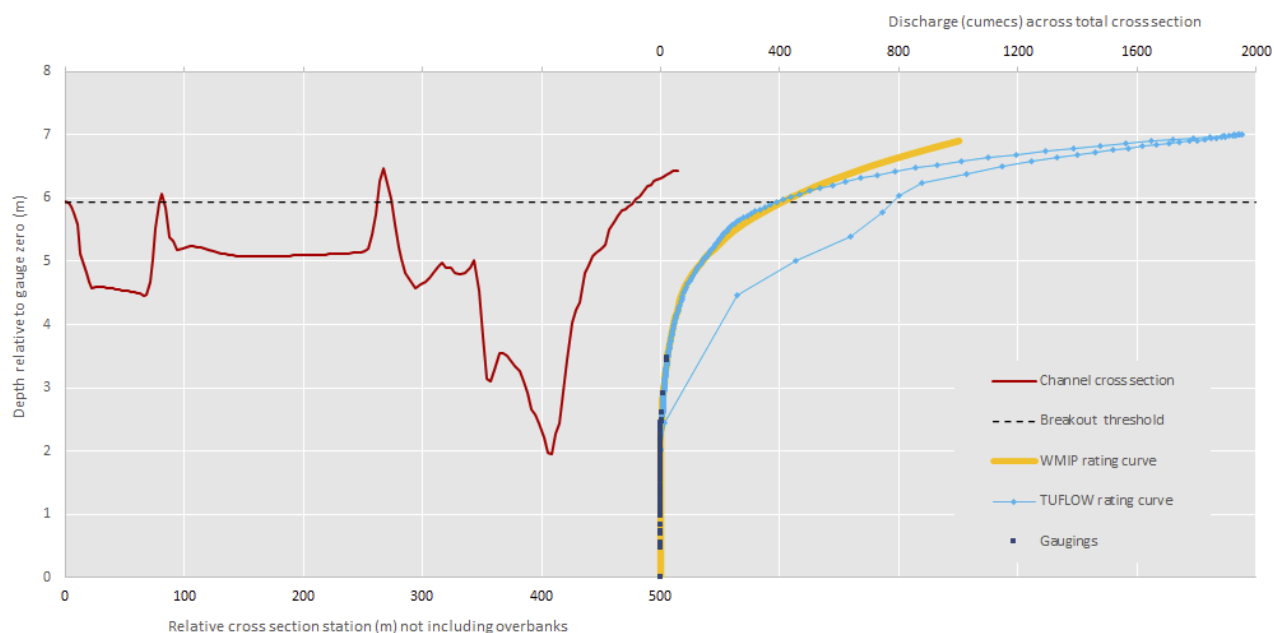


Figure 7.31 Adopted revised rating curve at Lone Pine

Depending on whether the rising or the falling limb was selected, the 2010 event peak flow would range from 700 to 1,010 m³/s, which is 17 per cent to 67 per cent higher than the Water Monitoring Information Portal gauging. The 2010 event peak flow was also significantly larger than the second largest event (1996, ranging from 250 m³/s to 655 m³/s), suggesting that it may be a high outlier.

To account for these uncertainties, FFAs were performed using both the rising (i.e. high flow) and falling limbs (i.e. low flow) of the rating curve, treating the 2010 event as a potential outlier. Table 7.19 summarises the 1% AEP expected peak flow quantiles based on FFAs at Lone Pine, highlighting the significant uncertainty underlying the gauge records that requires explanation before they can be used for model output comparison.

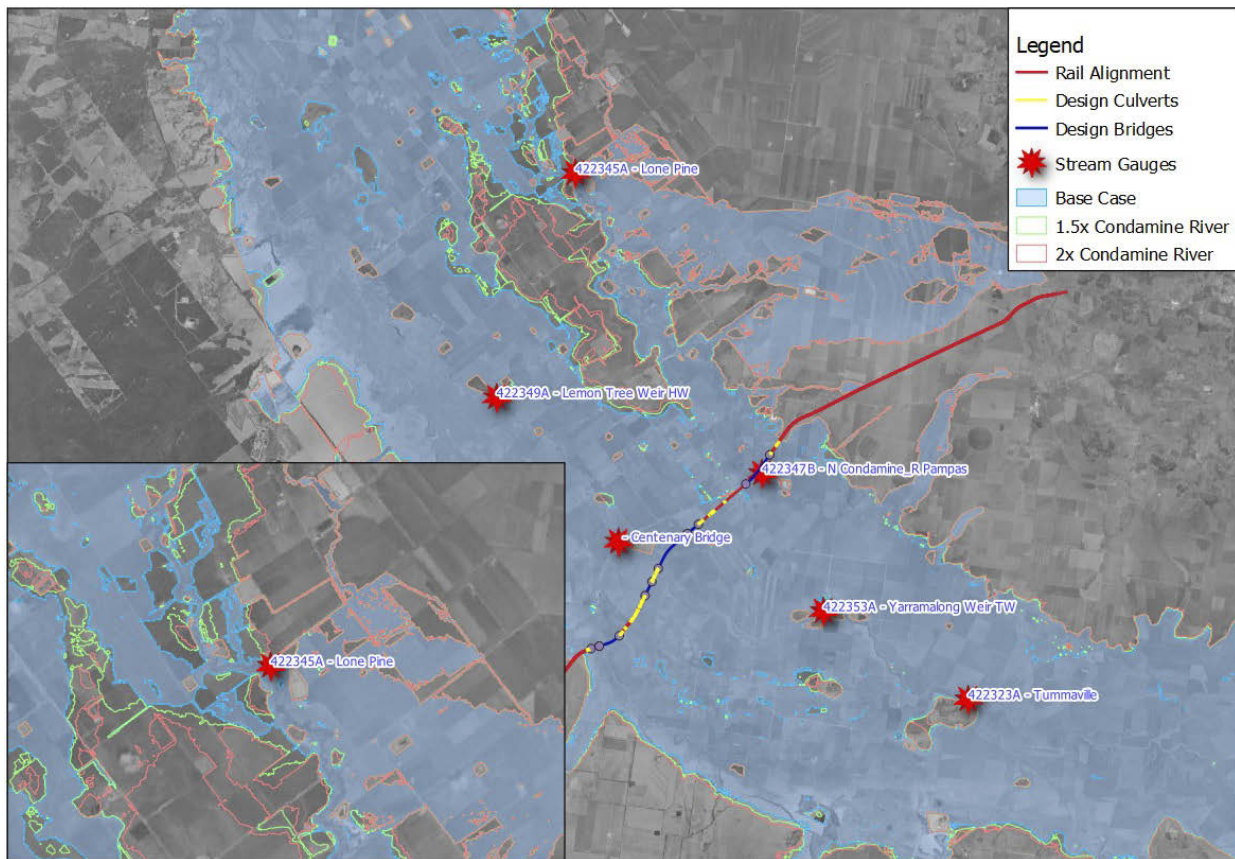
Table 7.19 1% AEP expected peak flow quantiles at Lone Pine

Rating curve	2010 event assessed as outlier	FFA expected quantile (m ³ /s)
Rising limb (high flow)	No	2,700
	Yes	1,500
Falling limb (low flow)	No	450
	Yes	389

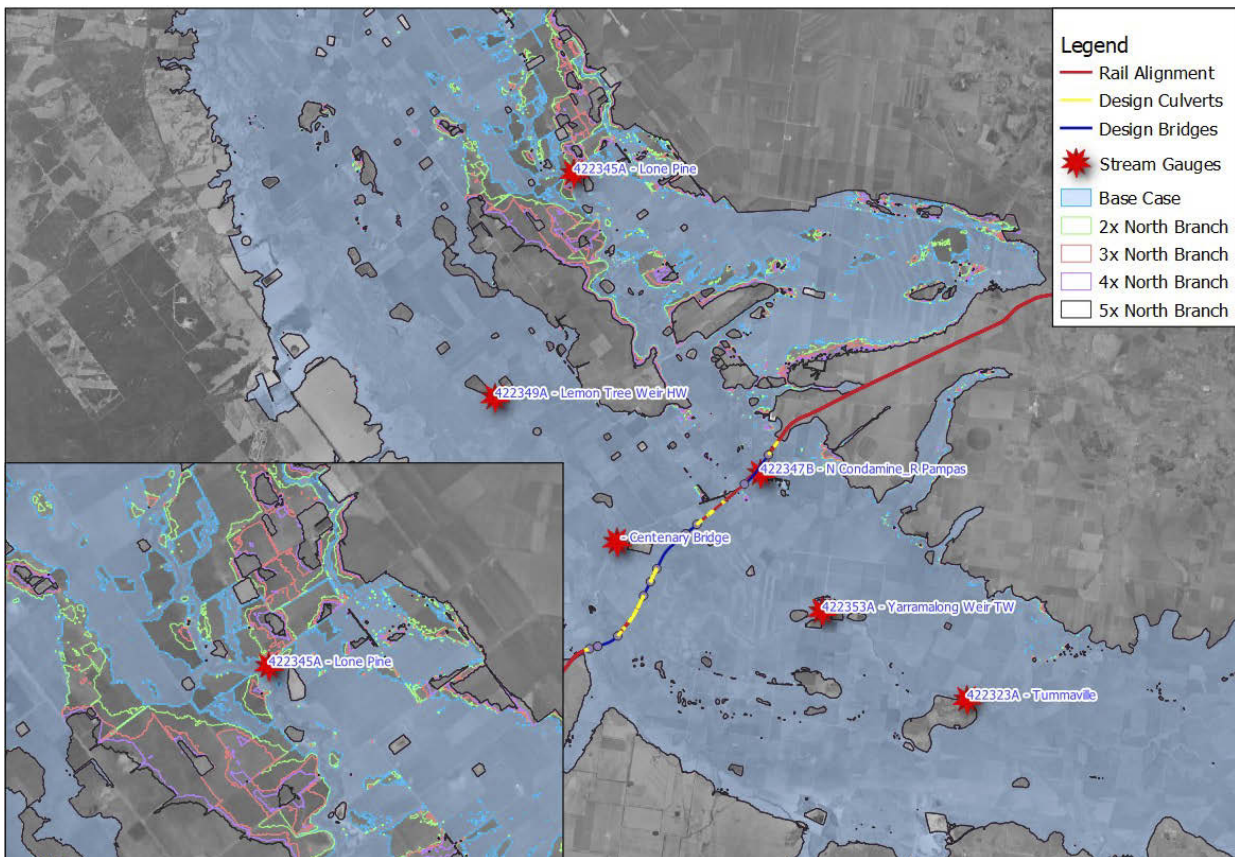
On 21 May 2021, FFJV and ARTC visited the Lone Pine gauge in order to gain a better understanding of the flooding that occurred in the area during the 2010 event, and to validate hydraulic modelling. The main purpose of the visit was to clarify the significant uncertainty surrounding the gauge records at Lone Pine. Interviews with adjacent landowners were also conducted. Observations made during the site visit suggested that the gauge levels recorded at Lone Pine were accurate with regard to the extent of the flood as experienced by local landowners. It was also noted that significant overland flow occurred on the afternoon of the 26 December 2010 (a day prior to the regional flood peak on the 27 December 2010) from the Yarranlea direction (south of the Project alignment).

To determine the cause of uncertainty in the 2010 event at Lone Pine, and assess the need for adjustments to the current design parameters in the hydrologic model, the draft EIS hydraulic model was refined and used to test flow increases along both the Condamine River and the North Branch in an effort to replicate the onsite observations from the 2010 event. The refined draft EIS model configuration is identical to the revised hydraulic model (Section 7.3), except that QT was not used within the draft EIS model. Specifically, the Condamine River design event inflows generated using the hydrologic model yielded increases of between 50 per cent and 100 per cent (Figure 7.32 (a)), and the North Branch inflows were increased by between 100 per cent and 400 per cent, (Figure 7.32 (b)). The flood extents produced by the inflow sensitivity tests are shown in Figure 7.32 (b), with a detailed tabulation of simulated and gauged flood levels provided in Table 7.20.

The sensitivity tests indicate that the flood extent around Lone Pine is primarily sensitive to the North Branch inflows, with a +400 per cent increase yielding the closest match (+0.062 m). However, applying this adjustment would negatively impact the calibration at gauges both upstream and downstream of Lone Pine, particularly at Pampas (+0.066 m) and Cecil Weir (+0.256 m) where the current design parameters already produce very good matches to the gauges (-0.018 m at Pampas, +0.086 m at Cecil Weir). In comparison, increasing the Condamine River inflows has relatively little effect at Lone Pine (-0.541 m at +100 per cent inflow), but significant changes elsewhere (e.g. +0.202 m at Pampas, +1.129 m at Cecil Weir), resulting in a significantly reduced quality in regional calibration.



a) Increase in Condamine River URBS inflows



b) Increase in North Branch URBS inflows

Figure 7.32 Condamine River and North Branch—2010 event flood extent sensitivity to increases in hydraulic model inflows

Table 7.20 **Condamine River—Comparison between simulated and recorded peak flood levels in 2010 event¹**

Inflow adjustment	Tummalville (high quality)		Yarramalong Weir (medium quality)		Pampas (medium quality)		Lone Pine (medium quality)		Cecil Weir (high quality)	
	Model level ² (mAHD)	Difference to gauge level (m)	Model level ² (mAHD)	Difference to gauge level (m)	Model level ² (mAHD)	Difference to gauge level (m)	Model level ² (mAHD)	Difference to gauge level (m)	Model level ² (mAHD)	Difference to gauge level (m)
No change (i.e. design inflows)	392.082 ¹	0.021	385.999	-0.193	382.576	-0.018	370.289	-1.173	356.821	0.086
North Branch URBS +100%	392.083	0.022	386.004	-0.188	382.597	0.003	370.749	-0.713	356.877	0.142
North Branch URBS +200%	392.083	0.022	386.008	-0.184	382.615	0.021	371.114	-0.348	356.921	0.186
North Branch URBS +300%	392.083	0.022	386.012	-0.180	382.633	0.039	371.359	-0.103	356.958	0.223
North Branch URBS +400%	392.084	0.023	386.017	-0.175	382.660	0.066	371.524	0.062	356.991	0.256
Condamine River URBS + 50%	392.513	0.452	386.280	0.088	382.687	0.093	370.700	-0.762	357.436	0.701
Condamine River URBS +100%	392.837	0.776	386.519	0.327	382.796	0.202	370.921	-0.541	357.864	1.129
<i>Recorded</i>	<i>392.061</i>	-	<i>386.192</i>	-	<i>382.594</i>	-	<i>371.462</i>	-	<i>356.735</i>	-

Table notes:

- 1 Values in the tables above are shown to three decimal places for comparative purposes only, and not to imply absolute accuracy of results.
- 2 Modelled water levels may differ slightly from those reported for calibration (Section 7.3.6) as QT (Section 7.3.2.1) was not applied during sensitivity testing.

To assess the feasibility of achieving a +400 per cent increase in North Branch inflows, the hydrologic model design event parameters were adjusted for two extreme scenarios:

- No initial and continuing losses
- No initial and continuing losses, and reduction of routing parameters to the lowest end of the typical range of values for unrealistically steep catchments ($\alpha = 0.10$, $\beta = 1.00$) in the URBS model.

The sensitivity test results are summarised in Table 7.21. The results show that even with unrealistically steep routing parameters and no design event losses only, an approximate 100 per cent increase in flow can be achieved.

Table 7.21 URBS sensitivity scenarios—Peak Flows at Lone Pine (m³/s)¹

AEP	Current design parameters: $\alpha = 0.13$ $\beta = 2.00$ IL= 46 mm – median preburst CL= 0.4 mm/hr	Scenario 1 parameters: $\alpha = 0.13$ $\beta = 2.00$ IL= 0 mm CL= 0.0 mm/hr	Scenario 2 parameters: $\alpha = 0.10$ $\beta = 1.00$ IL= 0 mm CL= 0.0 mm/hr	Required flows to match Lone Pine (approximately 400% increase) (m ³ /s)
1%	373	473	621	1,865
2%	313	450	635	1,565
5%	241	380	537	1,205
10%	193	295	368	965

Table note:

1 URBS peak flows at Lone Pine do not result in the same TUFLOW peak flows at Lone Pine. The reported flows are for URBS vs URBS comparisons only.

The sensitivity tests indicate that the flood levels around Lone Pine are primarily influenced by local rainfall-runoff behaviours within the 300 km² of local catchment area east of the gauge, rather than regional flooding that drives the flood levels surrounding the Project alignment and on the Condamine River floodplain. Since the gauge is located more 15 km downstream of the Project alignment, and flood levels upstream of the Project alignment are primarily driven by inflows from the Condamine River (which is expected given the order-of-magnitude difference in catchment sizes), it is considered reasonable to assume that the distribution of flows between the Condamine River and North Branch is accurately represented in a regional flood event. Therefore, the current validation of the hydrologic model to the combined outlet is considered appropriate.

7.3 Hydraulic model development—Condamine River

Due to the expansive floodplain and braided flow paths associated with the lower reaches of the Condamine River catchment, particularly in the agricultural land adjacent to Millmerran and Brookstead, a 2D modelling approach was adopted to appropriately simulate flood mechanisms within the catchment. The platform that was utilised for hydraulic modelling for the Condamine system is the TUFLOW HPC software (version 2020-10-AD) with SGS and QT approach. The processes and assumptions adopted throughout the development of the hydraulic model are described in the following sections.

7.3.1 Model setup

Integrated 1D/2D numerical TUFLOW HPC hydraulic models were developed to simulate flood behaviour within the Condamine River study area. Two hydraulic models were developed, one being a regional model and the second being the local model, which covered a smaller area within the extents of the regional model to enable faster design optioneering.

The local model was used to test a large number of design configurations; however, following the selection of the preferred reference design, the selected option was run using the full regional model.

The regional model was developed to simulate flows throughout the broader Condamine floodplain, extending over 100 km along the Condamine River main channel from just downstream of the Talgai Weir to approximately 16 km downstream of Cecil Weir stream gauge, and spans over 34 km at its widest point. This model was used for calibration, and to understand flood depths and velocities during design events in existing and design conditions. The regional model was developed from the original draft EIS hydraulic model by incorporating community feedback and Expert Flood Panel recommendations, with key improvements including:

- Solver update to 2020-10-AD-iSP-w64
- Significant extension of downstream boundary past Cecil Weir to enable hydraulic validation at an additional high-quality gauge stream gauge (i.e. Cecil Weir) and eliminate potential TW uncertainties at Lone Pine in the earlier model
- Refinement to conveyance modelling on flow paths narrower than the base cell size (20 m) by adopting SGS at a 1 m resolution, which enabled a full 2D representation for all watercourses on the floodplain
- Application of QT refinement at a very fine resolution (2.5 m) around culvert inlets and outlets to improve linking of 1D culverts to the 2D floodplain, which enabled a higher resolution impact assessment in key areas with a high risk of geomorphological impact.

The hydraulic model extent set up is shown in Figure C1-D in Volume 2 – Appendix C.

An overview of the model setup and key parameters for the regional model is provided in Table 7.22.

Table 7.22 Regional Condamine River hydraulic model summary

Parameter	Information
Completion date	April 2022
AEPs assessed	20%, 10%, 5%, 2%, 1%, 1 in 2,000, 1 in 10,000 AEP and PMF
Hydrologic modelling approach	Inflows determined through aforementioned URBS model. Inflows applied to model as SA polygons within 2d domain utilising the SA ALL command
IFD input parameters	Hydrologic approach based upon ARR 2019
Hydraulic modelling approach	TUFLOW HPC GPU – version 2020-10-AD-iSP-w64
Model extent	Figure C1-D in Volume 2 – Appendix C
Grid size	20 m
QT size	2.5 m, 2 nd Order
SGS size	1 m
DEM (year flown)	Multiple Topographic datasets, varying from 2010 to 2015.
Roughness	Spatially varying roughness values compliant with industry norms.
Eddy viscosity	Wu (default)
Model calibration	1991 Event, 2010 Event, 2013 Event, 2021 Event.
D/S model boundary	Height-Discharge (HQ) Boundary with normal slope approximated based upon topography dataset.
Hydraulic model timestep	Adaptive Timestep
Hydraulic model wetting and drying depths	Cell centre set at 0.0002 m Cell side set at 0.0001 m
Mapping cut-off depth	50 mm post processed cut-off depth applied to flood duration/not applied otherwise.
Modelled scenarios	Existing and design conditions
Sensitivity analysis	Inflow application approach/cropping roughness/climate change/culvert blockage/Back Creek interactions

7.3.2 Model topography and grid size

The topography within the hydraulic model was based on 1 m DEM data collected during 2015, which was supplied by ARTC. A 20 m x 20 m base grid was adopted to define the general floodplain topography, which is consistent with the typical range of model resolutions (i.e. 10 to 40 m) adopted in similar flood studies. Conveyance and storage effects less than the base cell size were captured using SGS at a 1 m resolution, which further improves the model accuracy and helps maintain reasonable model run times. Local refinements were applied to improve the connectivity between small drainage channels and definition of road/rail crown or levee banks as required. For the Developed Case, the topography was modified to include the Project rail embankment, drainage structures and associated infrastructure.

7.3.2.1 Sub-grid sampling and Quadtree mesh refinement

The hydraulic model was established using TUFLOW's 2020-10-AD model version, including SGS and QT add-on modules. The below points provide a brief description of each module, along with the key parameters adopted for this assessment.

- Traditional grid-mesh based 2D hydraulic models sample the underlying DEM by taking a single elevation at the cell centres and cell faces. Sub-grid sampling allows for increased resolution by sampling a greater number of points of the underlying DEM to establish a water surface elevation versus volume relationship for each grid, along with a water surface elevation versus width relationship (wetted perimeter) at cell sides. The adoption of SGS allows increased representation of storage and conveyance between cells. A SGS distance of 1 m was adopted for this assessment.
- Quadtree mesh is constructed by subdividing the base mesh-grid size into four smaller square cells. Cells can be further sub-divided into four, and so on, allowing for additional resolution in location of interest. The base hydraulic model grid cell size (20 m x 20 m) was revised to include additional spatial resolution of approximately 2.5 m, to better represent model velocity at the entry and exit of culverts. Quadtree meshes were applied in the model covering minimum 25 m upstream and 100 m downstream of the design culvert locations, and culverts within the permanent footprint.

7.3.3 Roughness

The roughness values adopted in both the regional and local model were spatially varied based on both aerial imagery and land-use classifications sourced from the Qspatial and published by the Department of State Development, Infrastructure, Local Government and Planning. Information gained from landowners during the site inspection, such as typical crop types and sowing/harvesting timeframes, was also used to inform the material roughness values. The adopted roughness values are listed in Table 7.23.

Table 7.23 Condamine River hydraulic model—Manning's n values

Material	Manning's n value
River	0.035
Riparian vegetation	0.04
Sealed road corridor	0.015
Unsealed road corridor	0.025
Rural residential	0.090
Urban residential	0.100
Commercial	0.060
Open space—Minimum vegetation	0.045
Open space—Moderate vegetation	0.055
Open space—Heavy vegetation	0.065
Open water	0.025

Material	Manning's n value
Rail	0.025
Crops—Uniform ¹	0.1085

Table note:

- 1 A uniform value was adopted for the crop areas, due to the seasonal and spatial variations in vegetation distribution and the uncertainty this brings to the design event approach.

The values above were tested through the calibration and validation runs, particularly in regard to the crop roughness value, and the values adopted were observed to result in relatively good fits for timings of flow peaks and floodplain attenuation.

In Table 7.23, the uniform Manning's n value of 0.1085 for crop was chosen as the best representation of the expected average conditions due to multiple sensitivity tests that took into account the impact of spatial and seasonal variations in crop distribution on peak flood levels. The sensitivity of the results to spatial variations in roughness parameters is discussed in Section 7.5.5.4.

The hydraulic model extent and the spatial distribution of land use in the 2D model domain is presented in Figure C1-E in Volume 2 – Appendix C.

7.3.4 Hydraulic structures

Numerous data sources were used in defining existing structures within the hydraulic model. Survey information was utilised where possible along with DTMR GIMS data. In addition to this information, records from site visits undertaken in February 2018 (Section 7.1.8) and structure geometry information contained within the previous hydraulic studies (Section 7.1.1) were used. Where no data was available and culverts were visible within topography and aerial imagery, assumptions of structure sizes were made based on measured structure widths and topography levels. In total, 15 existing culverts and eight existing bridges were identified within the hydraulic model domain as summarised in Table 7.24 and Table 7.25.

Table 7.24 Condamine River—Identified existing structures within the hydraulic model extent

Structure modelling ID	Type	U/S invert (mAHD)	D/S invert (mAHD)	Diameter/width (m)	Height (m)	Number of cells
G02	RCBC	379.67	379.65	2.4	1.2	8
G03	RCBC	376.20	376.20	2.4	1.2	8
G04	RCBC	376.29	376.29	2.4	1.2	2
G05	RCBC	376.39	376.39	2.4	1.2	8
G06	RCBC	376.80	376.80	2.4	1.2	3
P01	RCBC	381.75	381.72	1.2	0.3	6
P02	RCBC	381.73	381.67	1.2	0.3	6
P03	RCBC	381.74	381.67	1.2	0.3	6
P04	RCBC	381.67	381.66	1.2	0.3	6
P05	RCBC	381.71	381.68	1.2	0.3	6
P06	RCBC	381.72	381.68	1.2	0.3	6
P07	RCBC	381.72	381.69	1.2	0.3	6
P08	RCBC	381.69	381.65	1.2	0.3	6
P09	RCBC	381.69	381.69	1.2	0.3	6
P10	RCBC	381.76	381.64	1.2	0.3	6

Table 7.25 **Condamine River—Identified existing bridges within the hydraulic model extent**

Bridge name	Obvert level (mAHD)	Deck depth (m)
Grass Tree Creek (Millmerran-Leyburn Road)—BIS ID 243	377.80	1.00
Dogtrap Creek (Millmerran-Leyburn Road)—BIS ID 244	392.20	0.61
Canal Creek (Millmerran-Leyburn Road)—BIS ID 245	394.59	0.61
Back Creek (Gore Highway)—BIS ID 24899	396.10	1.00
Condamine River (Gore Highway)—BIS ID 361	376.91	1.00
Condamine River (Toowoomba-Karara Road)—BIS ID 580	406.09	0.55
Middle Creek (Toowoomba-Karara Road)—BIS ID 581	406.10	1.00
Thanes Creek (Toowoomba-Karara Road)—BIS ID 582	405.51	1.00

7.3.5 Boundary conditions

The hydrologic model outputs were applied as inflows into the hydraulic model. Total inflows from catchments upstream of the hydraulic model extent were applied at the upstream model boundary and local inflows from areas within the hydraulic model were applied throughout the model.

Internal inflow boundaries were applied as SA polygons along the streams and rivers within the hydraulic model extent. The 'ALL' flag was applied to the internal SA boundaries to enable an even distribution of inflows along physical streamlines. This approach was utilised to enable a physical representation of conveyance along known flow paths, particularly where they cross the Project alignment. Although the 'SA ALL' approach may negate some of the routing calculation within the hydrologic model, this occurs at a local subarea scale (about 50 km² on average) that is less than 1 per cent of the total upstream catchment area (about 7,000 km²). Therefore, the floodplain conditions within the hydraulic model are dominated by the upstream inflows, and the hydraulic re-routing in each subarea would have only a marginal impact on the hydraulic results.

As demonstrated in Section 7.5.5.3, this approach tends to produce slightly more conservative flood level estimates than the TUFLOW default approach, which concentrates inflows at the bottom of each subarea.

Downstream boundaries were modelled under normal depth conditions based on topographically derived channel slopes. Downstream boundary lines were separated to represent the independent flow paths that would each incur their own normal depth condition. Backwater effects are not anticipated at the Cecil Weir stream gauge as the model has been extended 16 km downstream of Cecil Plains.

Boundary locations are shown in Figure C1-D in Volume 2 – Appendix C.

7.3.6 Calibration

Model calibration was undertaken for the February 1991, December 2010, January 2013 and December 2021 events. This process involved adjusting the model parameters, primarily roughness, to obtain the best average match to the recorded hydrographs at key gauge locations. The calibrated model was then validated and verified before it can be used confidently to describe flooding behaviour on the regional Condamine River floodplain and support impact assessment efforts. Validation and verification of the calibrated hydraulic model to surveyed flood markers, gauge records and historical photographs are presented in Sections 7.3.7 and 7.3.8, respectively.

The floodplain topography has changed considerably over the past 50+ years and is expected to continue to change, which limits the ability of the model to recreate past events and forecast future events. Large sections of the Gore Highway were damaged in the 2010/2011 flood events, which were resurfaced between October 2012 and July 2013. To accurately represent the pre-2014 floodplain conditions, the June 2012 LiDAR was adopted for all calibration events besides the December 2021 event. For the January 2021 event, the Base Case topography adopted for the Existing Case and Developed Case model scenarios was used. This topography is a composite of the latest LiDAR tiles available, which provides the most robust representation of the present-day floodplain topography, including the existing QR railway line embankment.

The comparison of modelled and gauged hydrographs for the 1991, 2010, 2013 and 2021 events are shown in Figure 7.33 to Figure 7.36 (1991 event), Figure 7.37 to Figure 7.40 (2010 event), Figure 7.41 to Figure 7.45 (2013 event) and Figure 7.46 to Figure 7.48 (2021 event).

A summary of modelled and observed flood heights are shown in Table 7.26, Table 7.27, Table 7.28 and Table 7.29.

Inundation extent maps for the calibration results are shown in Figure C2-A to C2-D in Volume 2 – Appendix C.

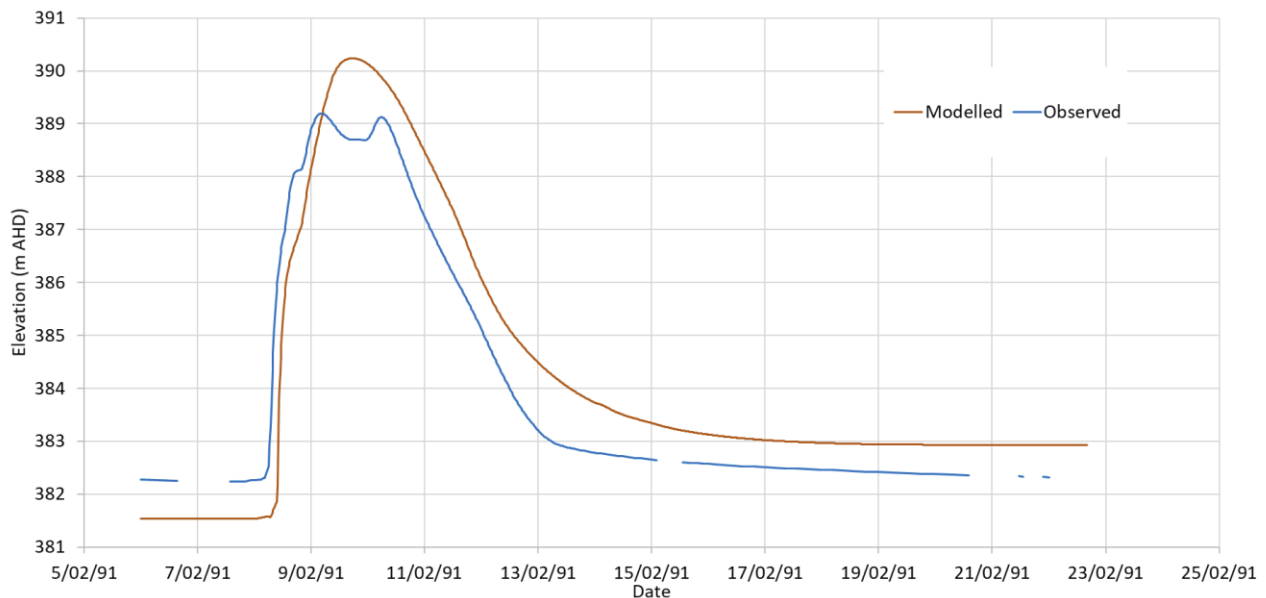


Figure 7.33 Hydraulic model calibration hydrographs at Tumnaville 422323A—1991 event

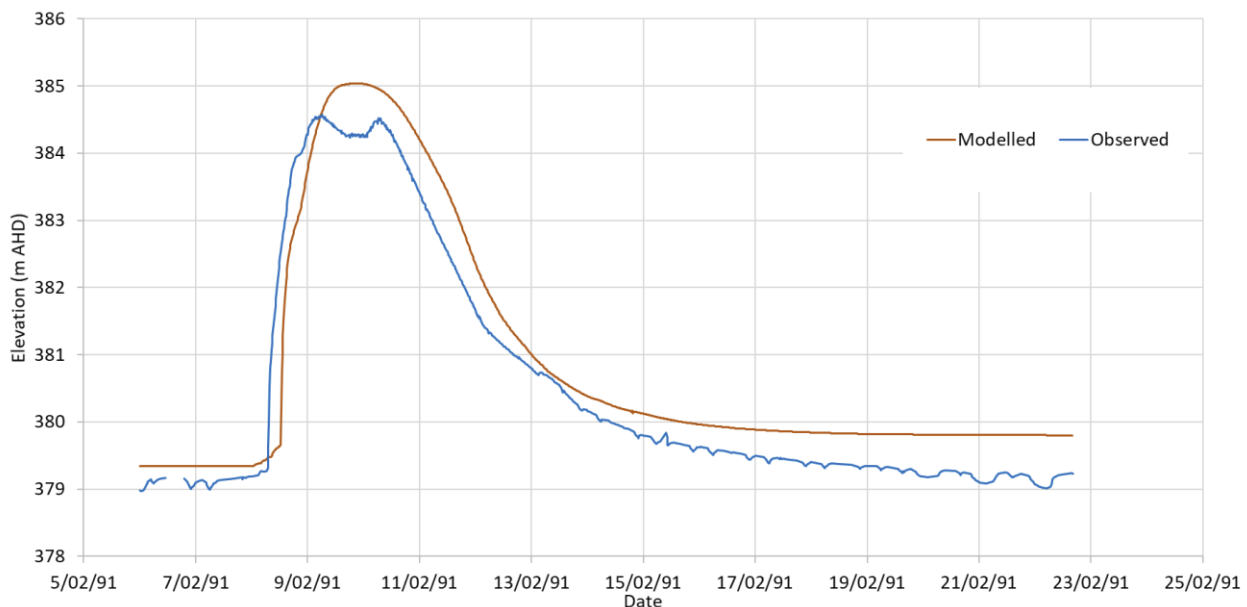


Figure 7.34 Hydraulic model calibration hydrographs at Yarramalong Weir 422353A—1991 event

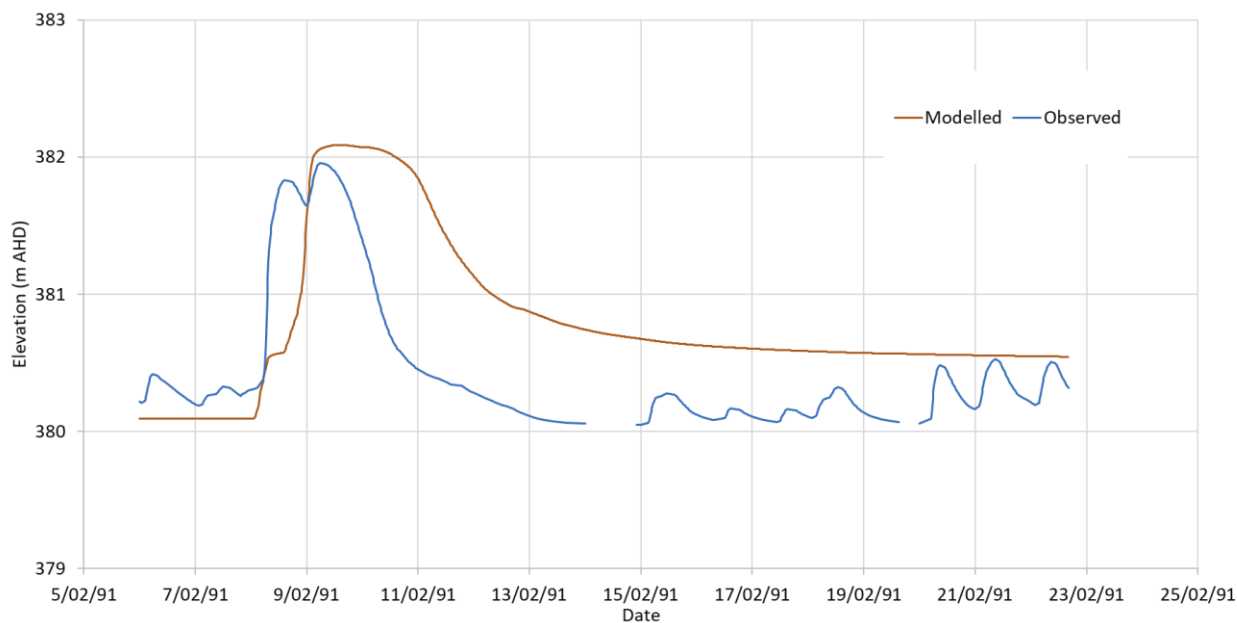


Figure 7.35 Hydraulic model calibration hydrographs at Pampas 422347B—1991 event

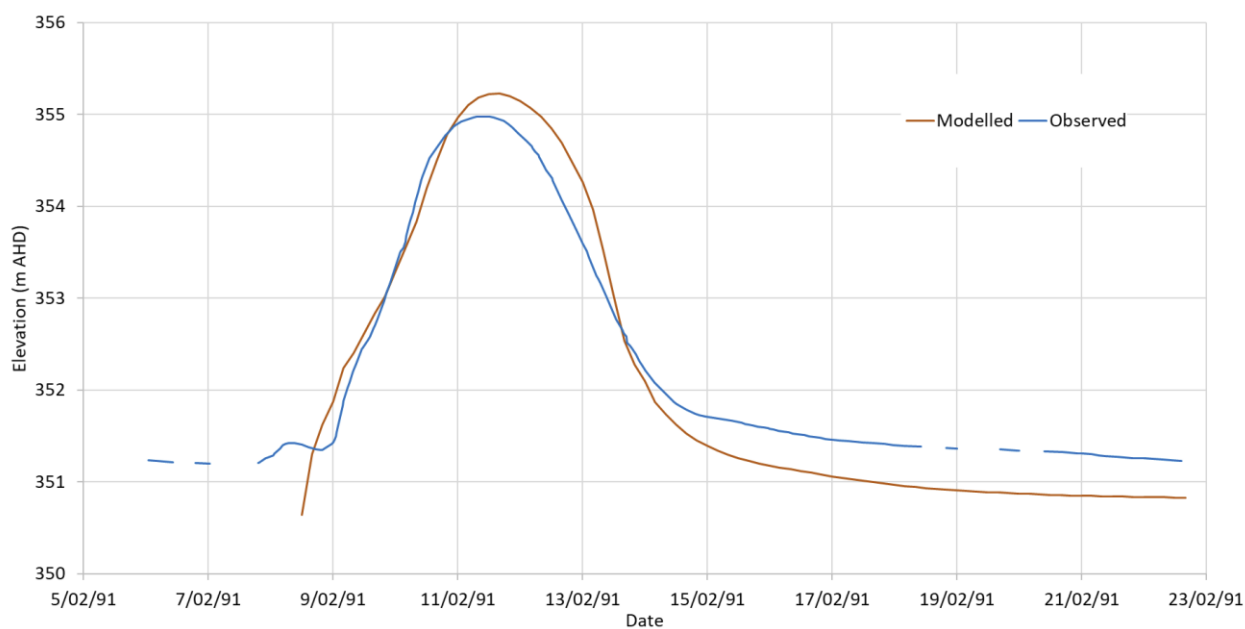


Figure 7.36 Hydraulic model calibration hydrographs at Cecil Weir 422316A—1991 event

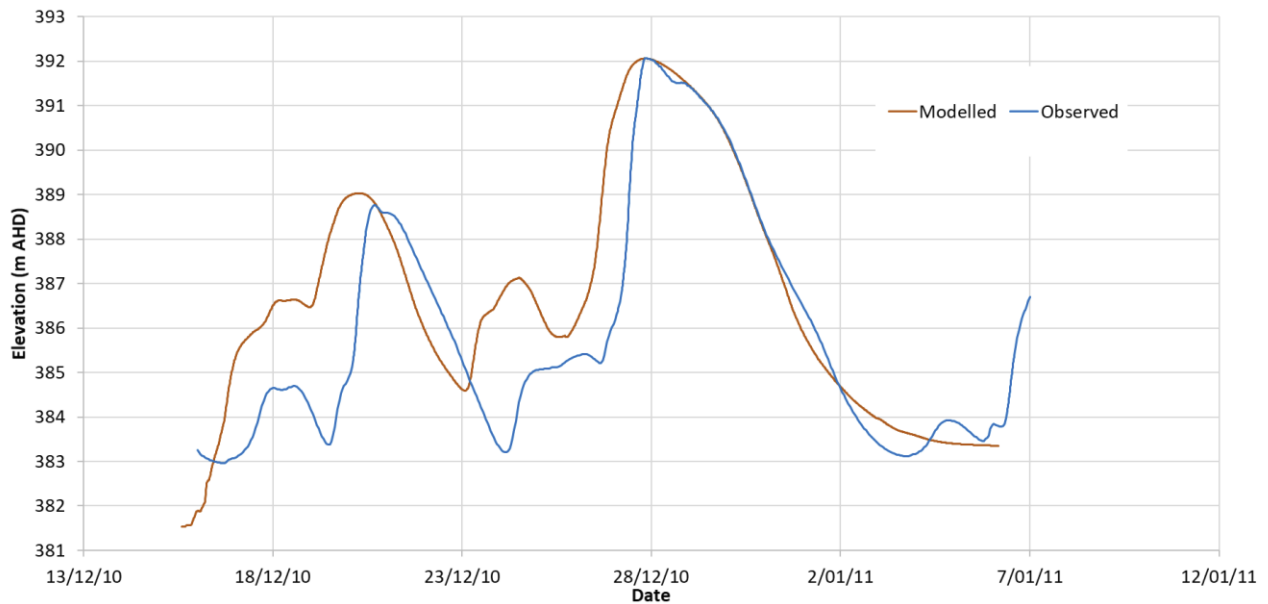


Figure 7.37 Hydraulic model calibration hydrographs at Tumnaville 422323A—2010 event

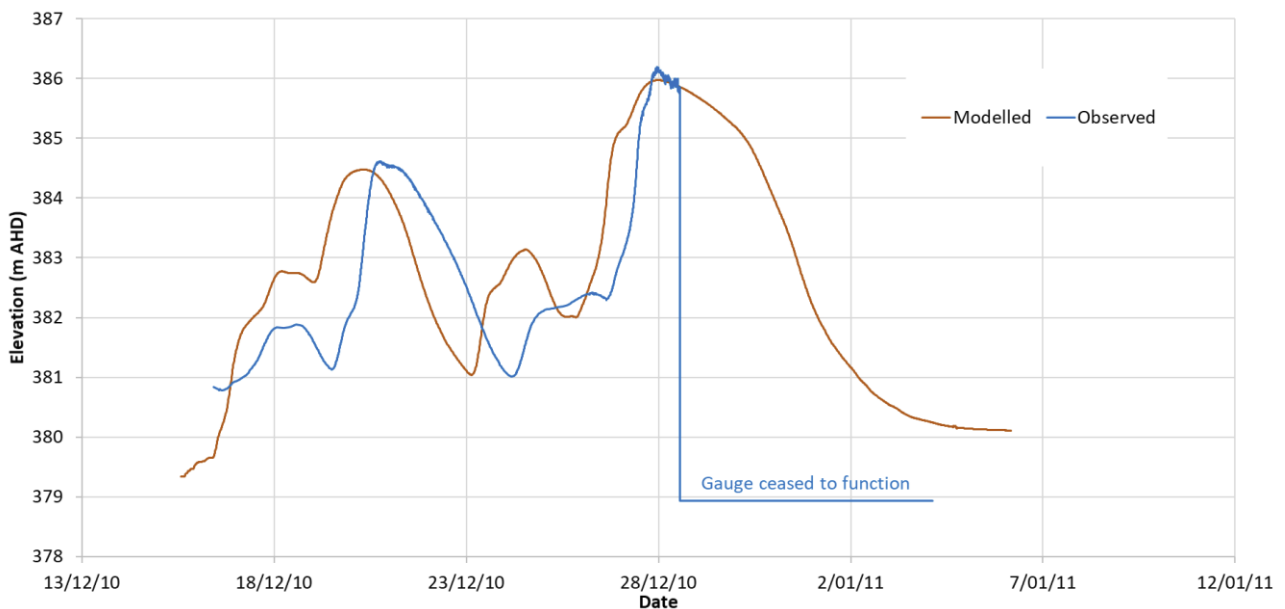


Figure 7.38 Hydraulic model calibration hydrographs at Yarramalong Weir 422353A—2010 event

Figure note:

Yarramalong gauge ceased to operate during the event as can be seen on the 'observed hydrograph' plot

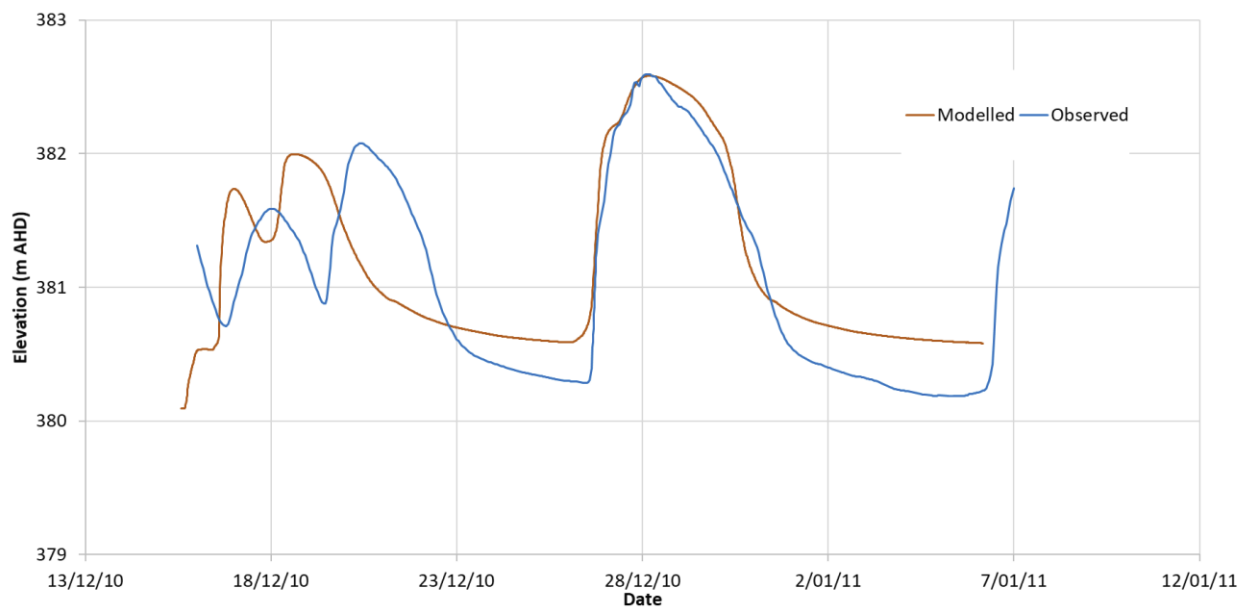


Figure 7.39 Hydraulic model calibration hydrographs at Pampas 422347B—2010 event

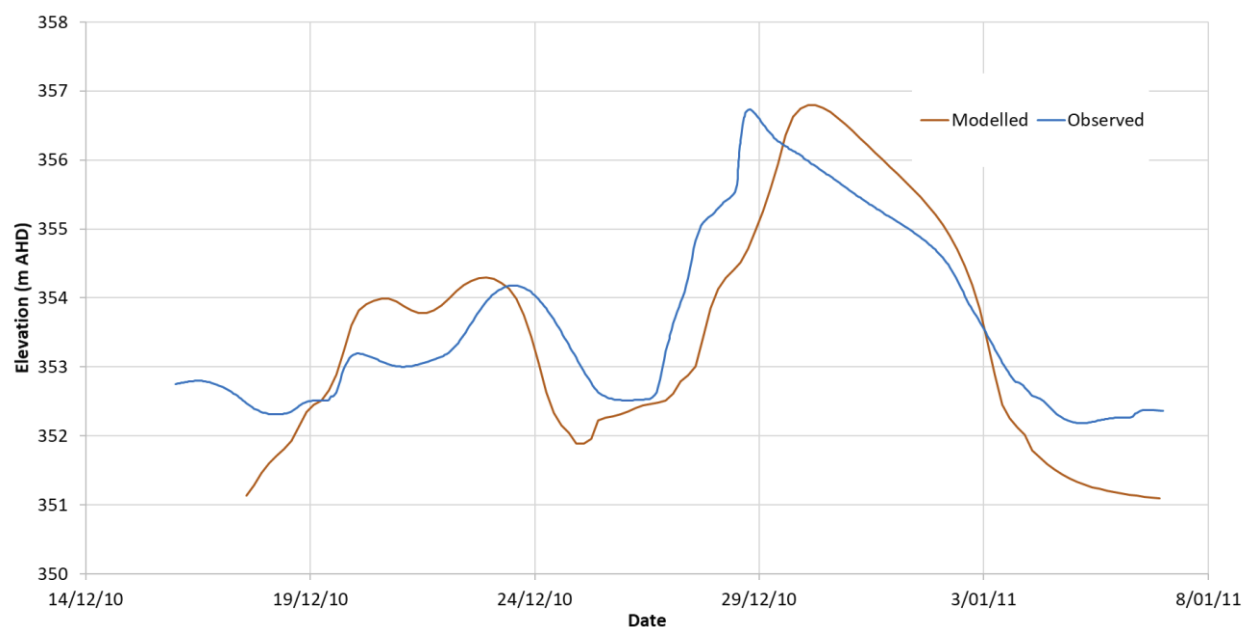


Figure 7.40 Hydraulic model calibration hydrographs at Cecil Weir 422316A—2010 event

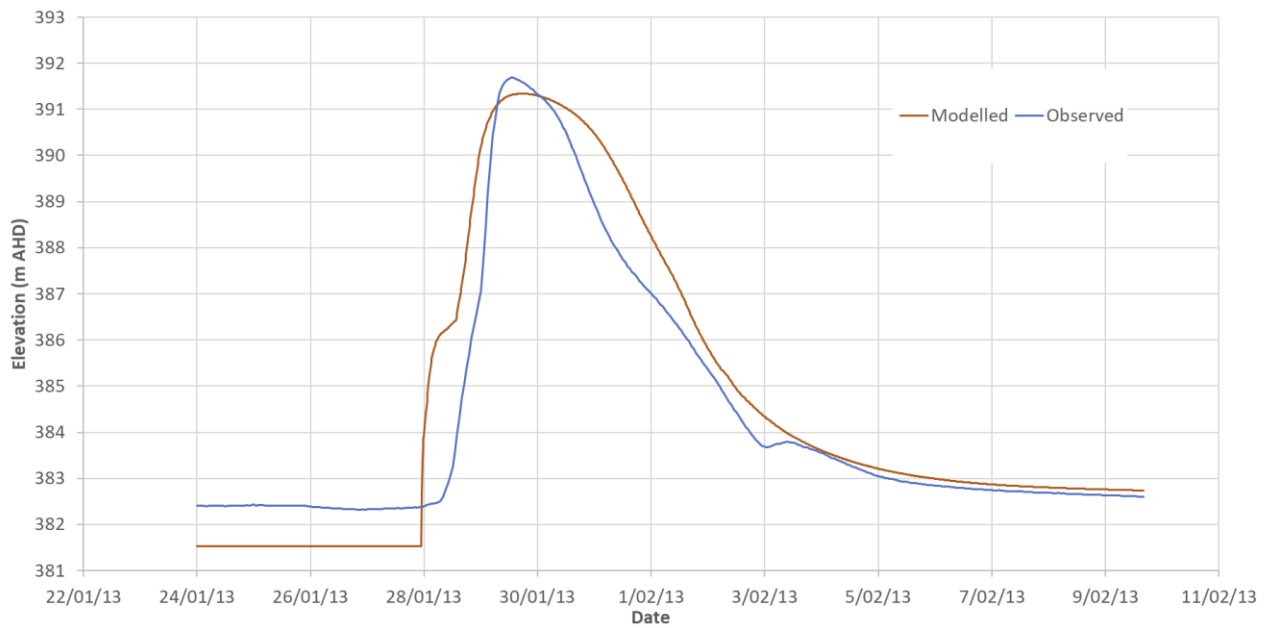


Figure 7.41 Hydraulic model calibration hydrographs at Tummalville 422323A—2013 event

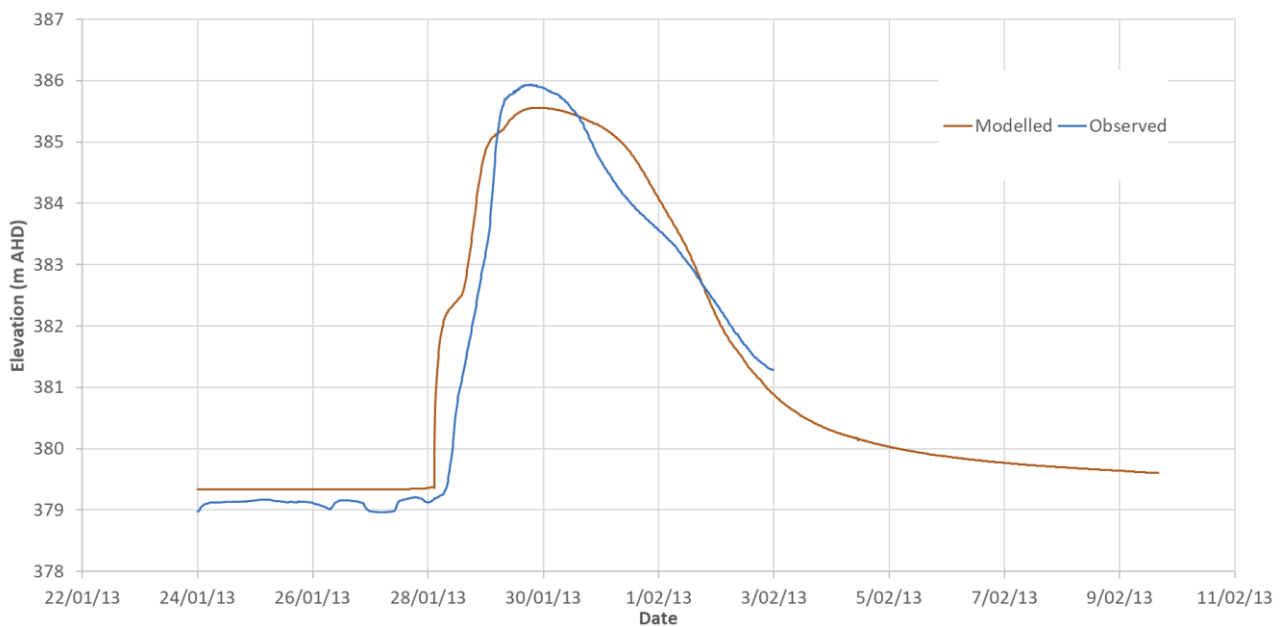


Figure 7.42 Hydraulic model calibration hydrographs at Yarramalong Weir 422353A—2013 event

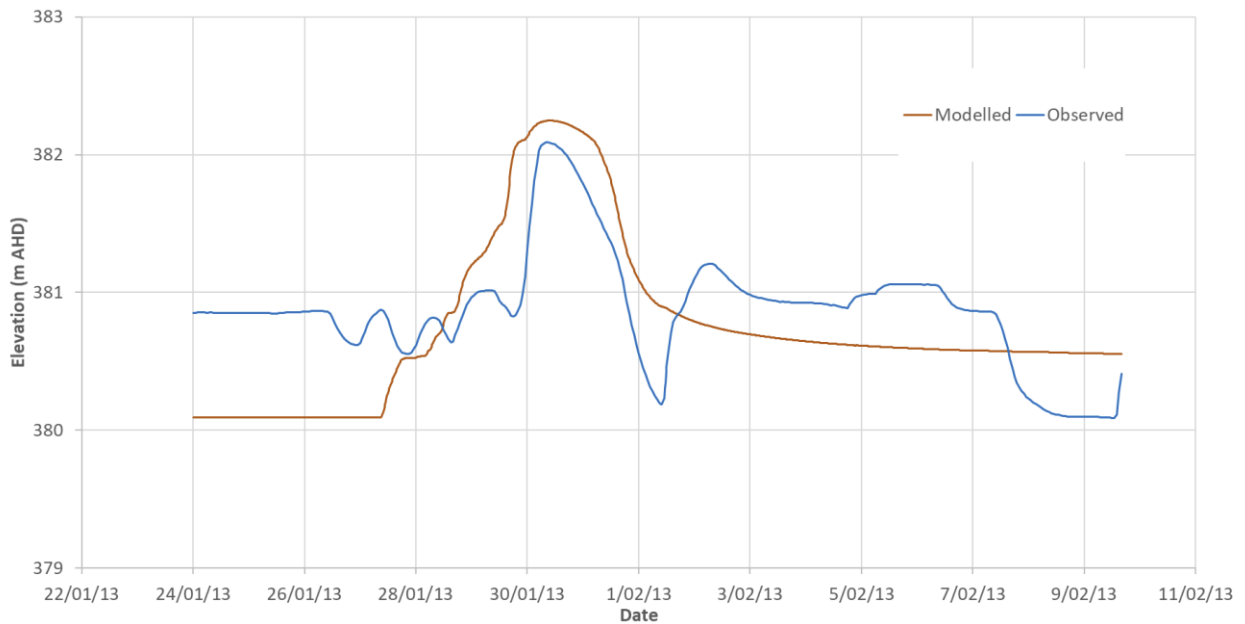


Figure 7.43 Hydraulic model calibration hydrographs at Pampas 422347B—2013 event

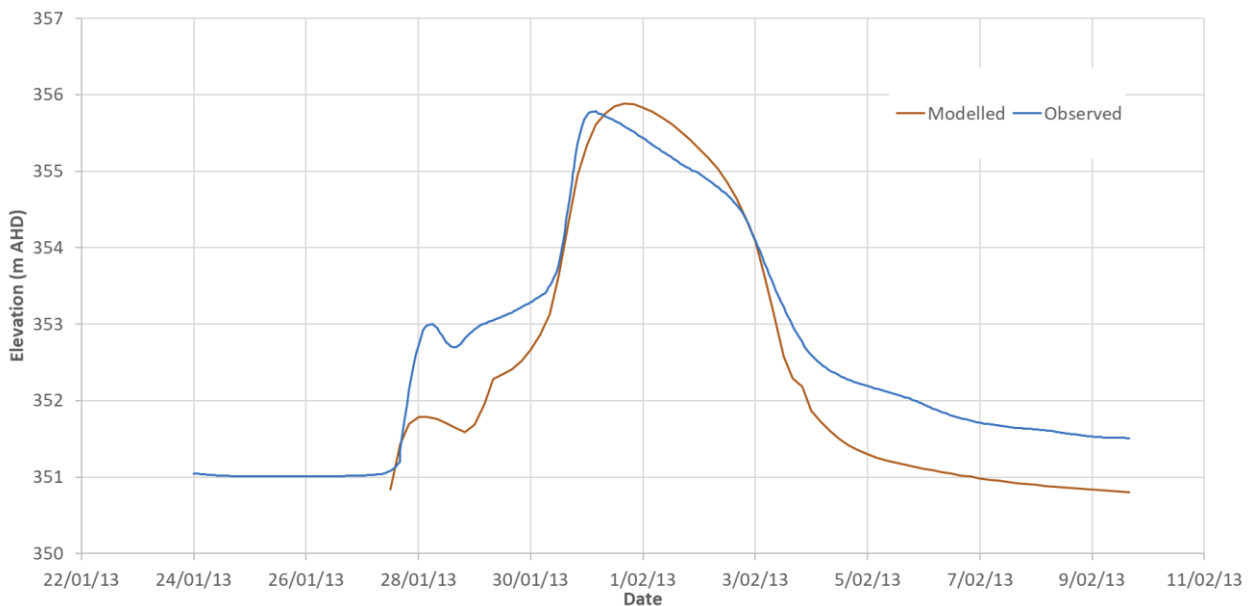


Figure 7.44 Hydraulic model calibration hydrographs at Cecil Weir 422316A—2013 event

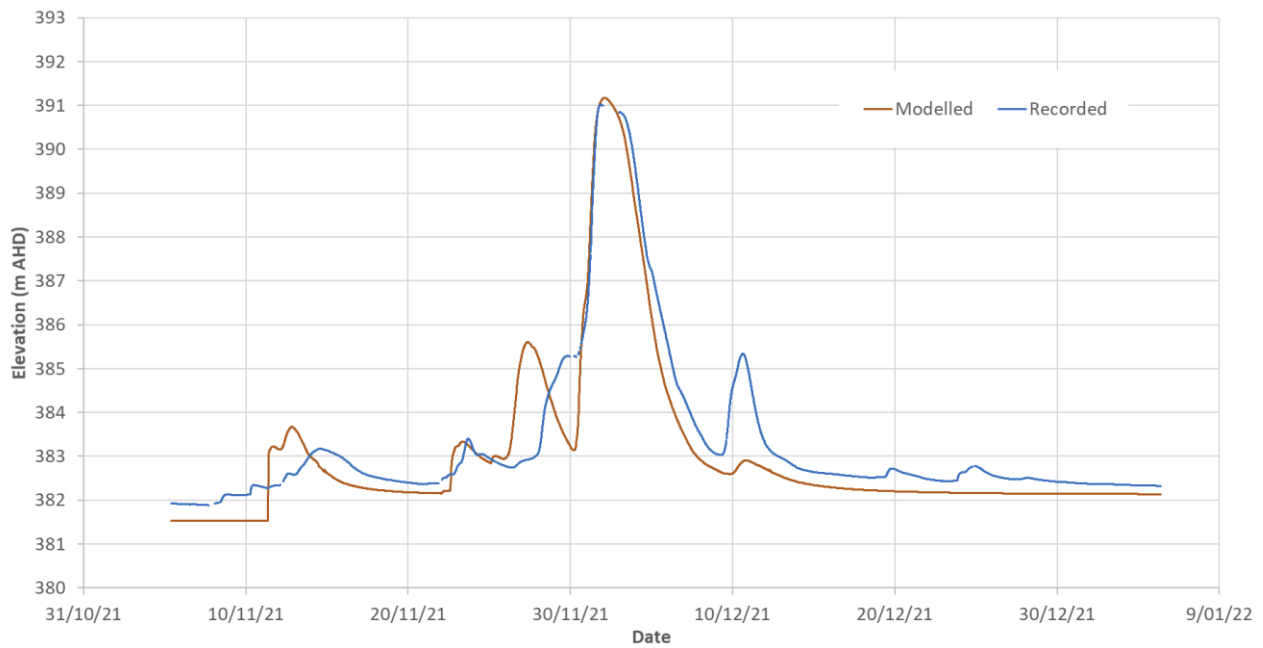


Figure 7.45 Hydraulic model calibration hydrographs at Tumnaville 422323A—2021 event

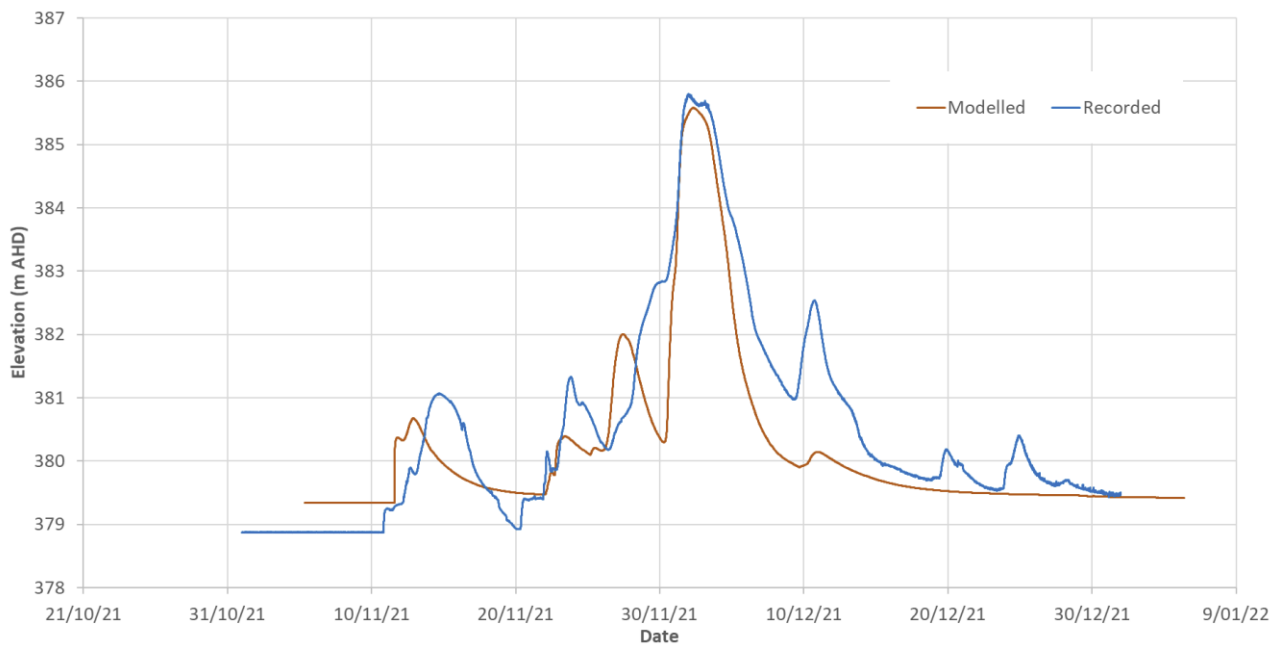


Figure 7.46 Hydraulic model calibration hydrographs at Yarramalong Weir 422353A—2021 event

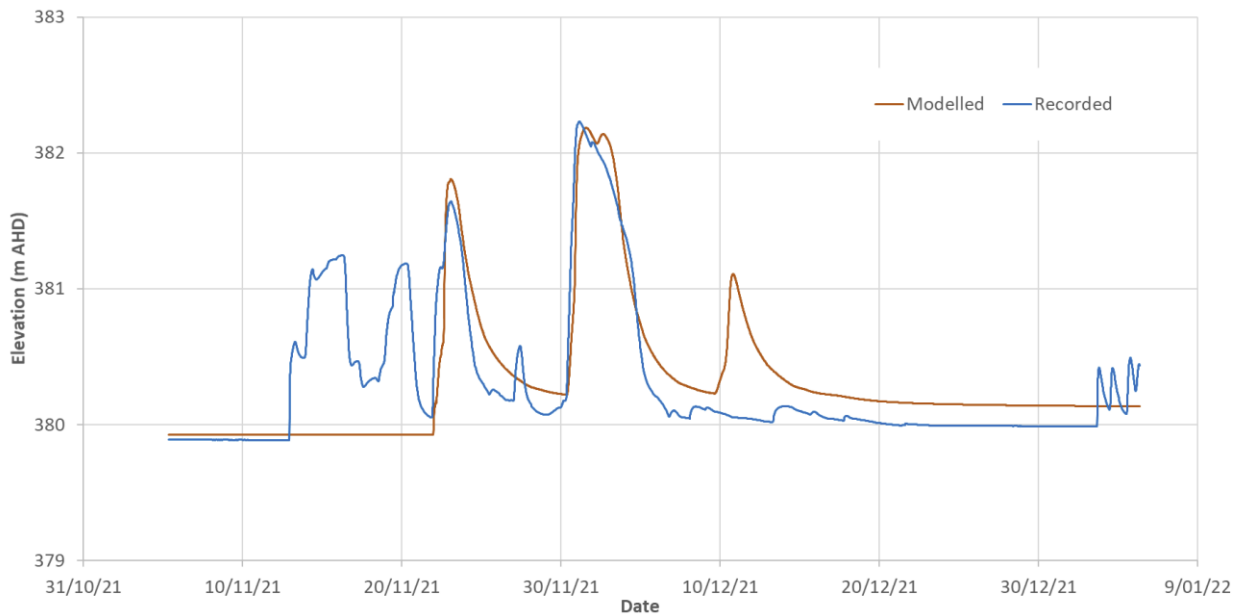


Figure 7.47 Hydraulic model calibration hydrographs at Pampas 422347B—2021 event

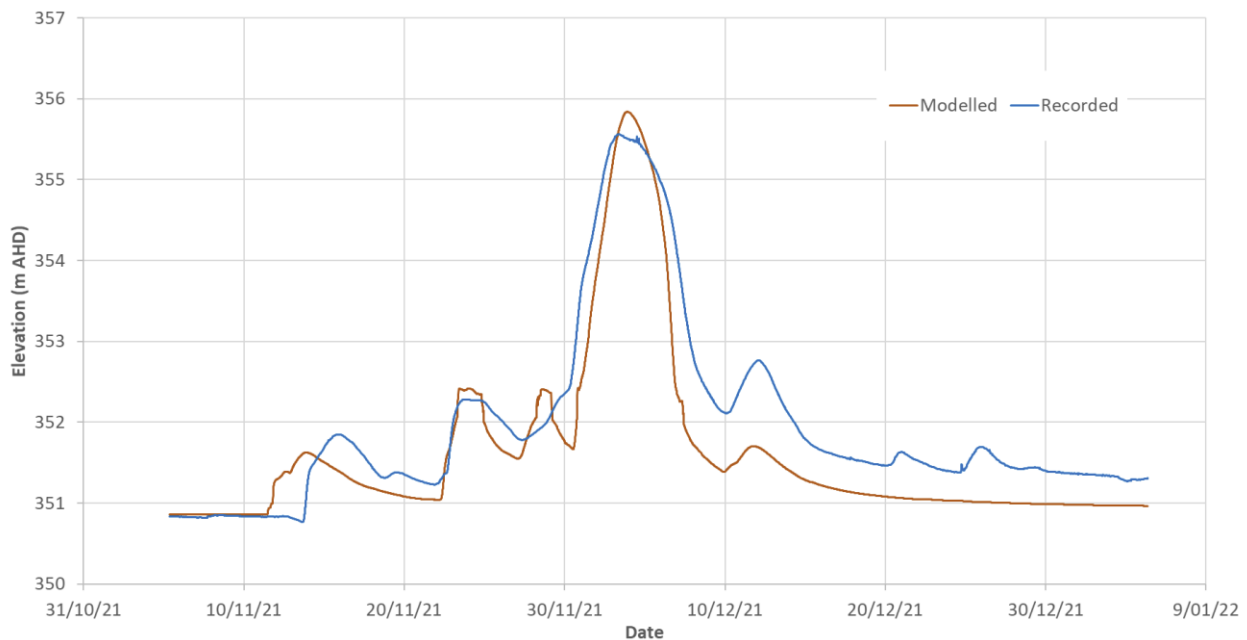


Figure 7.48 Hydraulic model calibration hydrographs at Cecil Weir 422316A—2021 event

Table 7.26 Condamine River—Summary of hydraulic model calibration—1991 event¹

Gauge	Ground surface level (mAHD)	Observed water level (mAHD)	Modelled water level (mAHD)	Observed water depth (m)	Modelled water depth (m)	Difference in water level (m)	Difference in water depth (%)
Tummalville	380.926	389.195	390.233	8.27	9.31	+1.038	+12.6%
Yarramalong	378.930	384.577	385.038	5.65	6.11	+0.461	+8.2%
Pampas	379.000	381.956	382.087	2.96	3.09	+0.131	+4.4%
Cecil Weir	347.507	354.975	355.226	7.47	7.72	+0.251	+3.4%

Table 7.27 Condamine River—Summary of hydraulic model calibration—2010 event¹

Gauge	Ground surface level (mAHD)	Observed water level (mAHD)	Modelled water level (mAHD)	Observed water depth (m)	Modelled water depth (m)	Difference in water level (m)	Difference in water depth (%)
Tummalville	380.926	392.061	392.059	11.14	11.13	-0.002	0.0%
Yarramalong	378.930	386.192	385.972	7.26	7.04	-0.221	-3.0%
Pampas	379.000	382.594	382.582	3.59	3.58	-0.012	-0.3%
Cecil Weir	347.507	356.735	356.794	9.23	9.29	+0.059	+0.6%

Table 7.28 Condamine River—Summary of hydraulic model calibration—2013 event¹

Gauge	Ground surface level (mAHD)	Observed water level (mAHD)	Modelled water level (mAHD)	Observed water depth (m)	Modelled water depth (m)	Difference in water level (m)	Difference in water depth (%)
Tummalville	380.926	391.690	391.339	10.76	10.41	-0.351	-3.3%
Yarramalong	378.930	385.939	385.557	7.01	6.63	-0.382	-5.5%
Pampas	379.000	382.090	382.247	3.09	3.25	+0.157	+5.1%
Cecil Weir	347.507	355.782	355.886	8.27	8.38	+0.104	+1.3%

Table 7.29 Condamine River—Summary of hydraulic model calibration—2021 event¹

Gauge	Ground surface level (mAHD)	Observed water level (mAHD)	Modelled water level (mAHD)	Observed water depth (m)	Modelled water depth (m)	Difference in water level (m)	Difference in water depth (%)
Tummalville	380.926	391.019	391.170	10.09	10.24	+0.151	+1.5%
Yarramalong	378.930	385.798	385.577	6.87	6.65	-0.221	-3.2%
Pampas	379.000	382.232	382.184	3.23	3.18	-0.048	-1.5%
Cecil Weir	347.507	355.560	355.837	8.05	8.33	+0.277	+3.4%

Tables note:

1 Values in the tables above are shown to three decimal places for comparative purposes only, and not to imply absolute accuracy of results

Throughout the development of the model, it was observed that changes made throughout calibration iterations have not resulted in significant changes to corresponding design event runs, because the broad and complex floodplain is not overly sensitive to tested parameters such as inflow application methods or hydraulic roughness parameters (Section 7.5.5).

The results produced by this calibrated hydraulic model are considered an acceptable estimation in determining the impacts of various feasibility design scenarios on existing floodplain conditions during the current stage of the Project. A recalibration may be required if significant topographic changes occur during future design stages of the Project.

7.3.7 Validation

The calibrated hydraulic model has been validated to surveyed flood markers and gauge records during the 2010 flood event, as presented in Sections 7.3.7.1 and 7.3.7.2, respectively. The validation process involved comparing the model output to a separate set of actual flood data points than those used in calibration, to ensure the calibrated model's accuracy and reliability in representing the actual flood conditions that would occur on the Condamine River regional floodplain. This validation is important to ensure that the model can be used with confidence for impact assessment purposes.

7.3.7.1 Flood markers

Through consultation, a total of 47 flood markers of the December 2010 flood event were supplied by landowners on the Condamine River floodplain. These flood markers were surveyed by a professional surveying company. Further confidence in model calibration is gained when validating the anecdotal and surveyed flood markers against modelling results.

Additional validation was performed using 25 independently surveyed flood markers provided by the Expert Flood Panel. A GIS analysis has identified these as either exact or highly probable duplicates of a subset of the FFJV flood markers; as such, they have been considered as a separate dataset.

Figure 7.49 illustrates the geographical location of the flood markers, including the relative quality of each. In addition, data at three streamflow gauges, namely Tummaville, Yarramalong Weir and Pampas, were also available for validation. As previously stated by WRM (2014), in their study of Condamine River flooding at Brookstead, there is uncertainty involved in using historic flood markers for validation and verification (Section 7.1.1).

Table 7.30 summarises the validation grouping of flood markers.

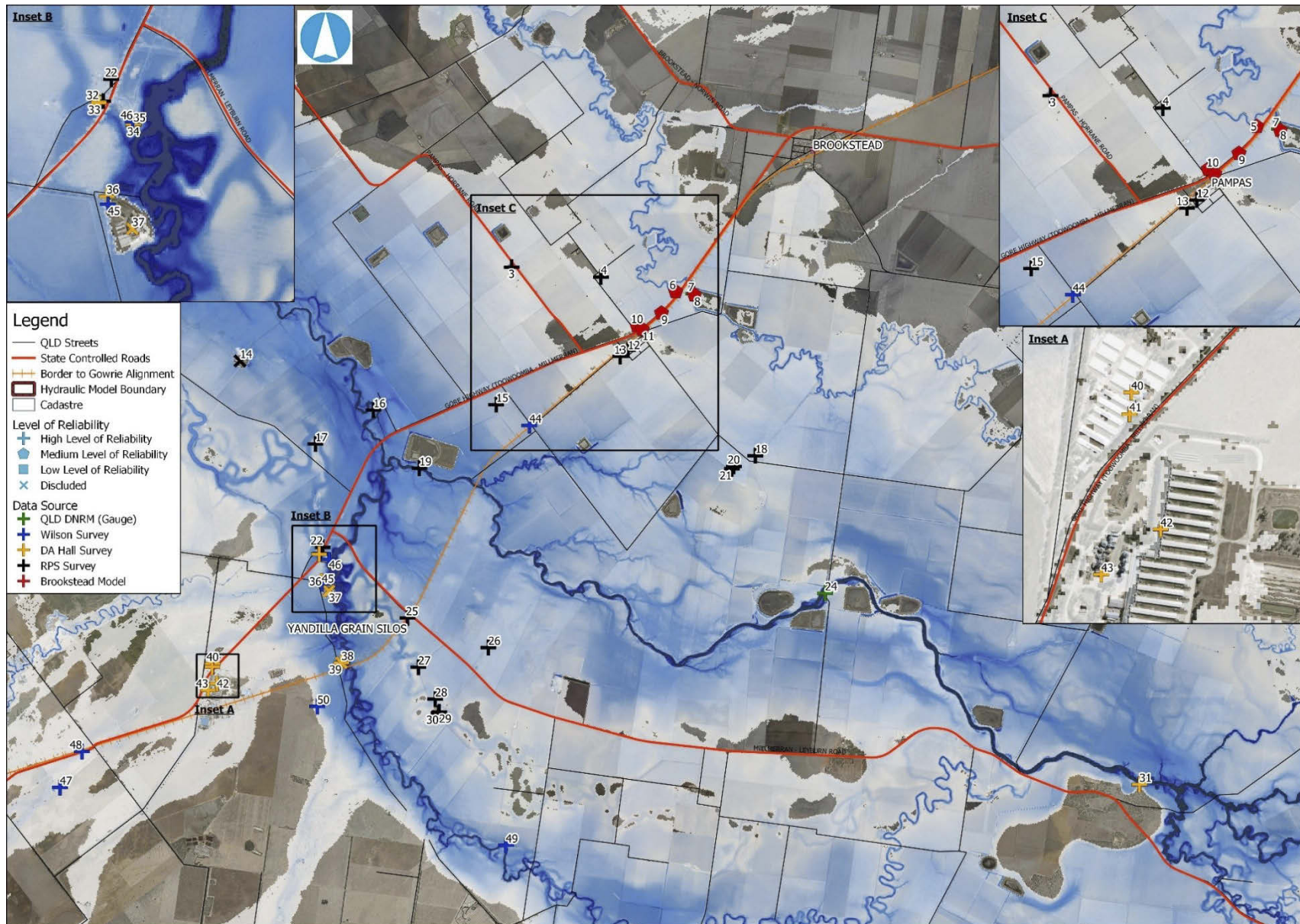


Figure 7.49 Condamine River—Flood marker locations and relative quality

Table 7.30 Flood marker grouping

Description	Grouping
Good match	±200 mm
Acceptable match	±200 – 300 mm
Poor match	±300 – 500 mm

In accordance with industry best practice, flood markers matched to within ± 200 mm in model validation are considered to be a good match, with matches within ± 300 mm considered to be an acceptable match. These targeted tolerances were developed to account for typical topographic and modelling uncertainties and had previously been adopted for other large infrastructure projects, such as the Yeppen floodplain crossing, and industry benchmark flood studies such as the Brisbane River Flood Study.

Figure 7.50 shows the frequency distribution of differences between modelled flood levels and flood marker data, where one dry flood marker (#1) and two (#2) spurious entries (#14, #39) were excluded from the analysis. A tabulation of the validation and commentary on the poor matches and outliers is provide in Table 7.31. The spatial distribution of the validation results is shown Figure 7.52.

Of the 47 flood markers included in the comparison, 78.7 per cent achieved a good match and 87.2 per cent achieved an acceptable or good match. The average difference of -25 mm between modelled and observed flood levels is considered excellent, as it is well within the typical vertical error of airborne topographic surveys (± 100 mm within one standard deviation).

The spatial distribution of the validation results (Figure 7.52) show a good reproduction of flood levels at all main points of interactions between Gore Highway and the regional floodplain, including along Grasstree Creek (cluster around flood marker #22), Condamine River (flood marker #16), and North Branch (cluster around flood marker #5), indicating that the flow balance between major and minor conveyance paths across the regional floodplain has been well reproduced. The spatial results demonstrate that the model has reproduced historic flood levels across the significant Condamine River floodplain with minimal global bias.

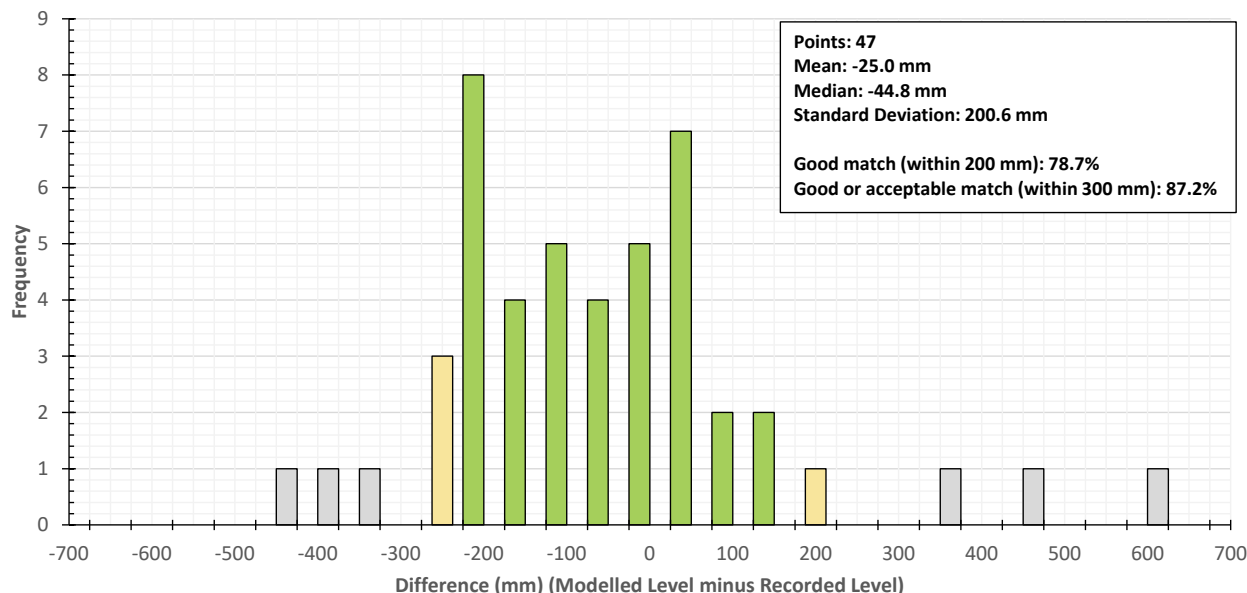


Figure 7.50 Condamine River—2010 event validation to FFJV flood markers

Additional model validation was undertaken against 25 independently surveyed flood markers provided by the Expert Flood Panel. A GIS analysis identified 6 exact duplications and 17 approximate duplications (i.e. located on the same hydraulic model cell) to the flood markers obtained by FFJV. The remaining two flood markers are located in very close proximity to, but further than one hydraulic model cell away from, nearby FFJV flood markers. For these reasons, the additional flood markers have been treated as a separate dataset to avoid introducing systematic survey uncertainties into the validation.

Figure 7.51 shows the frequency distribution of differences between modelled flood levels and the Expert Flood Panel flood marker dataset, where three dry marks have been excluded. Of the 22 flood markers included in the comparison, 86.4 per cent achieved a good or acceptable match. The average difference of -68.5 mm between modelled and observed flood levels is considered a very good match. As this dataset is essentially a re-survey of a subset of the FFJV flood markers, the lower average calibration (-68.5 mm vs -25.0 mm average difference) indicates a systematic error introduced as a result of the independent surveys.

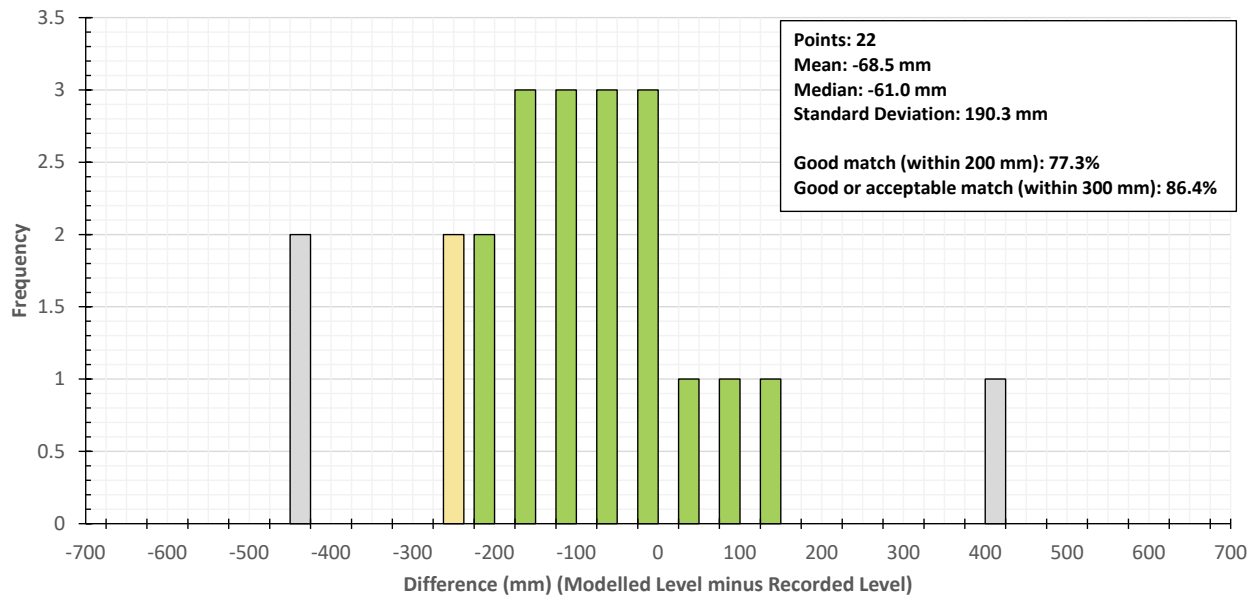


Figure 7.51 Condamine River—2010 event validation to Expert Flood Panel flood markers

Table 7.31 Condamine River—Surveyed flood marker comparison—2010 event

Flood marker ID (FFJV)	Easting	Northing	2010 peak water level (mAHD)	Modelled peak water level (mAHD)	Difference in peak water level (mm)	Classification	Comments
1	347851	6933298	389.61	-	-	Outlier	Sensitive to local runoff assumptions that could not be verified due to a lack of rainfall data.
2	348557	6932762	392.13	391.93	-200	Good	
3	341255	6926305	378.72	378.41	-310	Poor	Possible survey uncertainties along Pampas-Horrane Road. Water level is 300 mm higher on eastern side of Pampas Horrane Road.
4	342977	6926114	382.01	381.84	-165	Good	
5 ¹	344418	6925831	381.78	382.28	498	Poor	Same location as #6. Poor match likely due to survey uncertainty.
6 ¹	344418	6925831	382.44	382.28	-162	Good	Same location as #5
7	344762	6925794	382.594	382.62	25	Good	Pampas Gauge
8	344779	6925771	382.59	382.58	-9	Good	
9	344145	6925418	382.21	382.43	224	Acceptable	
10	343673	6925174	382.3	382.26	-45	Good	
11	343774	6925102	382.63	382.52	-112	Good	
12	343498	6924703	382.33	381.95	-378	Poor	Possible survey uncertainties along Fysh Road. Water level is 400 mm higher on eastern side of Fysh Road.
13	343349	6924581	382.09	381.87	-220	Acceptable	
14 ²	336017	6924495	376.27	-	-	Outlier	Level surveyed within failed levee during 2010 event. Levee failure was not assumed in the flood model.
15	340957	6923652	379.98	379.99	8	Good	
16	338590	6923546	379	379.06	62	Good	
17	337470	6922892	378.42	379.05	633	Poor	Possible local effects due to levee upstream of property.
18	345953	6922666	384.09	384.26	166	Good	
19	339476	6922425	379.43	379.82	390	Poor	Possible local effects due to ring tank embankment.
20	345539	6922459	383.48	383.57	91	Good	
21	345501	6922414	383.54	383.49	-48	Good	

Flood marker ID (FFJV)	Easting	Northing	2010 peak water level (mAHD)	Modelled peak water level (mAHD)	Difference in peak water level (mm)	Classification	Comments
22	337612	6920898	380.01	380.09	78	Good	
23	337567	6920775	380.08	380.13	52	Good	
24	347300	6920011	386.19	385.98	-209	Acceptable	Yarramalong Gauge
25	339247	6919533	380.41	380.52	105	Good	
26	340806	6918963	381.16	381.07	-86	Good	
27	339456	6918583	380.78	380.75	-35	Good	
28	339779	6917966	381.15	380.97	-179	Good	
29	339858	6917719	381.68	381.44	-240	Acceptable	
30	339845	6917698	381.63	381.44	-187	Good	
31	353364	6916318	392.061	392.09	32	Good	Tummaville Gauge
32	337541	6920771	380.092	380.13	42	Good	
33	337543	6920764	380.078	380.14	58	Good	
34	337736	6920641	380.221	380.17	-54	Good	
35	337737	6920644	380.22	380.17	-53	Good	
36	337586	6920239	380.187	380.25	67	Good	
37	337731	6920056	380.496	380.37	-131	Good	
38	337972	6918669	380.684	380.56	-127	Good	
39 ³	337972	6918669	-	380.56	-	Outlier	2013 flood level
40	335486	6918651	384.088	383.92	-169	Good	
41	335481	6918589	384.238	384.04	-198	Good	
42	335569	6918265	384.812	384.63	-179	Good	
43	335402	6918138	385.425	385.42	-1	Good	
44	341597	6923248	380.988	380.57	-420	Poor	Possible local effects at intersection between Gilgai Lane and another access track.
45	337593	6920194	380.308	380.25	-57	Good	
46	337733	6920658	380.213	380.16	-55	Good	

Flood marker ID (FFJV)	Easting	Northing	2010 peak water level (mAHD)	Modelled peak water level (mAHD)	Difference in peak water level (mm)	Classification	Comments
47	332543	6916262	393.927	394.09	160	Good	
48	332968	6916955	392.07	392.19	120	Good	
49	341143	6915145	382.752	382.83	80	Good	
50	337506	6917826	380.989	380.86	-131	Good	

Table notes:

- 1 Identical location with different surveyed flood levels.
- 2 Flood markers surveyed within flood levee which had failed during 2010 flood event. The flood model assumes flood levee to be structurally sound.
- 3 2013 flood level at abandoned hut; therefore, discounted.

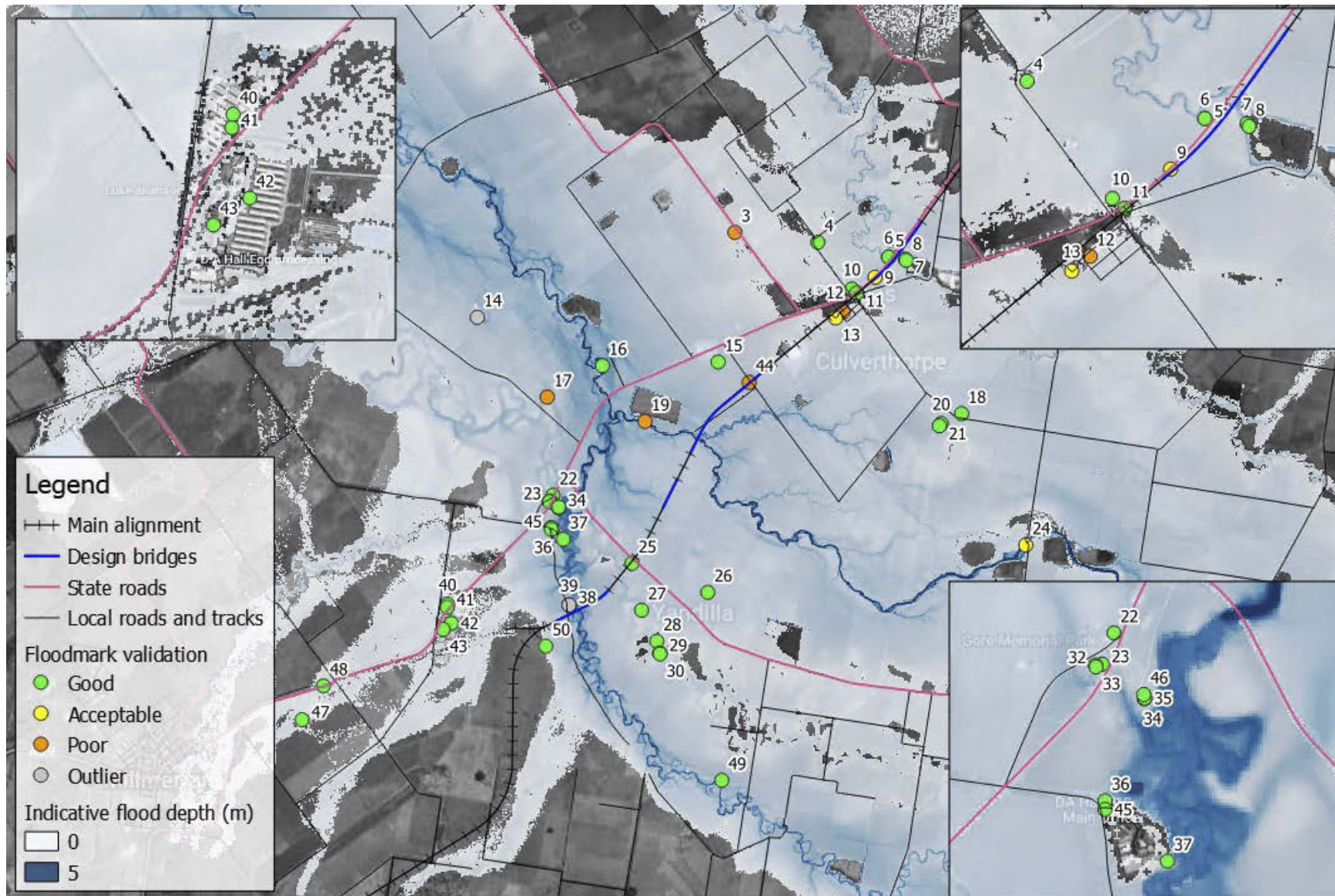


Figure 7.52 Condamine River—Spatial distribution of flood marker validation results

7.3.7.2 Manual gauge at Centenary Bridge

A manual gauge at Centenary Bridge was in operation during the December 2010 flood event. Table 7.32 compares the modelled peak water levels for the December 2010 calibration event with the observed peak water level (as per SKM, 2014) at Centenary Bridge. It also includes a comparison with the modelled peak water levels obtained by SKM in their flood study for TRC in 2014.

SKM (2014) concluded that the lower simulated level at Centenary Bridge is '*likely to be due to the model not implicitly representing the bridge structures and embankments in these locations due to the 60 m grid size*'. The FFJV hydraulic model has adopted a base grid size of 20 m with topographic details captured using SGS at 1 m resolution and therefore achieved a better match with the observed historic flood level.

Table 7.32 Recorded and modelled peak flood levels for December 2010 event at Centenary Bridge (SKM observed water level)

Gauge	Observed water level as per SKM (mAHD)	Flood Study (SKM, 2014)		This Study (FFJV)	
		Modelled water level (mAHD)	Difference in water level (mm)	Modelled water level (mAHD)	Difference in water level (mm)
Centenary Bridge	379.59 ¹	378.85	-740	379.38	-210

Table note:

- 1 The observed water level quoted by SKM in their 2014 report seems to be incorrect. In their report they quote a difference of 440 mm but 379.59 mAHD minus 378.85 mAHD is 740 mm. However, if one adds 440 mm to 378.85 mAHD, one gets 379.29 mAHD.

Given the uncertainty associated with the observed water level quoted by SKM, the formal record for the Centenary Bridge manual gauge was requested from the BoM. Figure 7.53 presents the manual gauged readings during the December 2010 flood event. It is unable to tell whether the flood peak was captured. The last data point before the record peak depth of 8.3 m was 11 hours prior.

Based on the gauge zero of 370.99 mAHD a peak flood depth of 8.3 m yields a peak flood level of 379.29 mAHD. This level corresponds with the 'corrected' SKM level (see table note 1 above), providing further confidence.

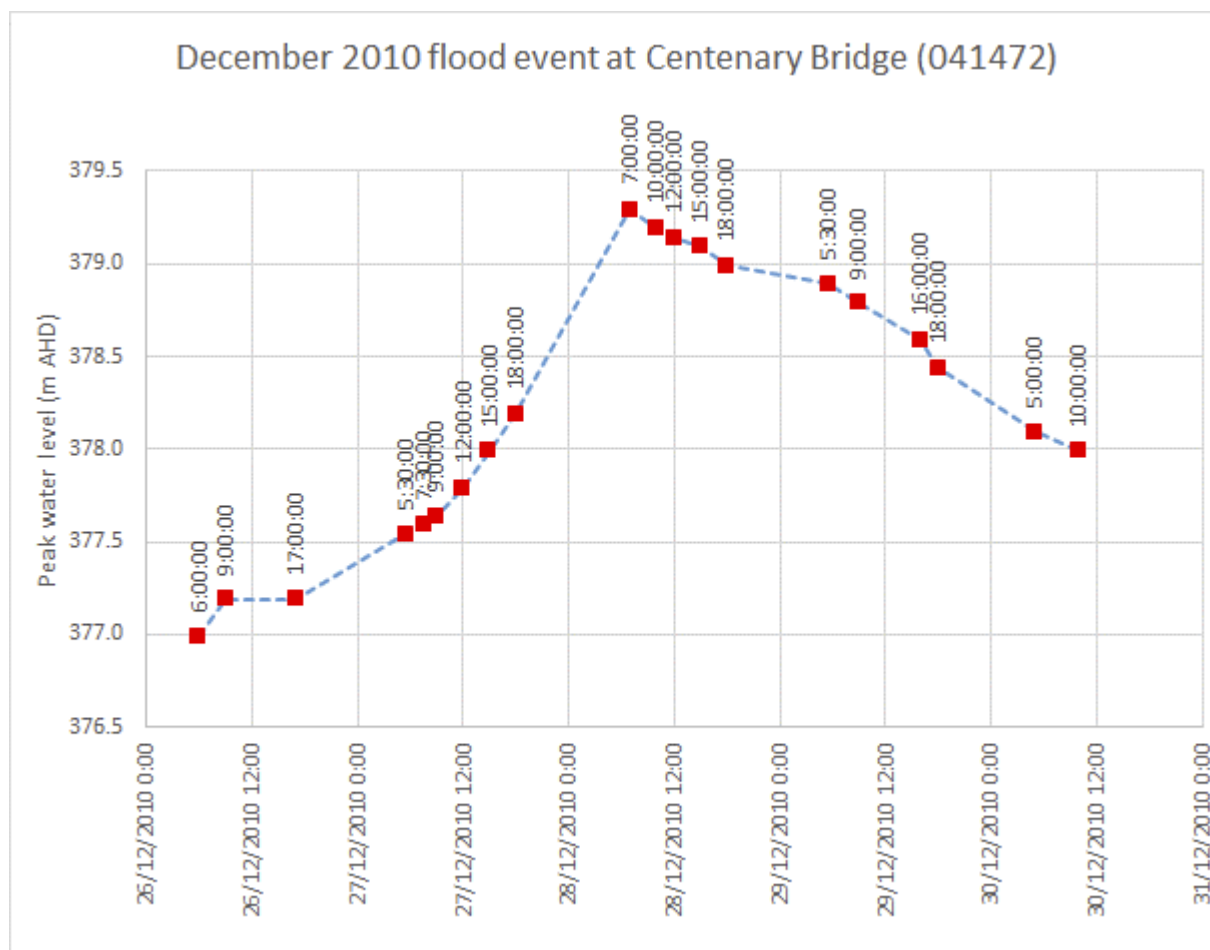


Figure 7.53 Manual gauged readings at Centenary Bridge (041472) during the Dec 2010 flood event

Table 7.33 compares the BoM December 2010 flood peak level with the FFJV modelled water level at Centenary Bridge. The BoM data confirms that the FFJV flood model validates well against observed data at Centenary Bridge, which was the main hydraulic control on Condamine River during the December 2010 event.

The corroboration between this and flood marker #16 downstream of the control (Section 7.3.7.1) confirms that the correct flow proportion along the Condamine River has been represented.

Table 7.33 Recorded and modelled peak flood levels for December 2010 event at Centenary Bridge (BoM observed water level)

Gauge	Observed water level as per BoM (mAHD)	This Study (FFJV)	
		Modelled water level (mAHD)	Difference in water level (mm)
Centenary Bridge	379.29	379.38	+90

7.3.8 Verification

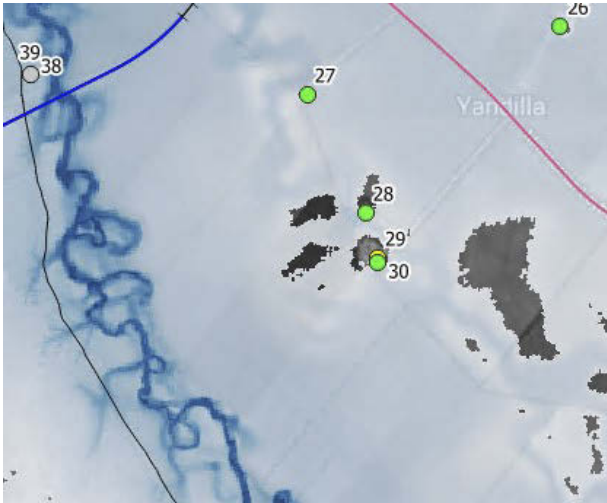
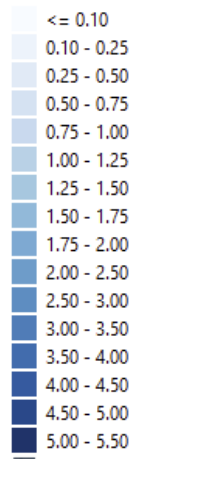
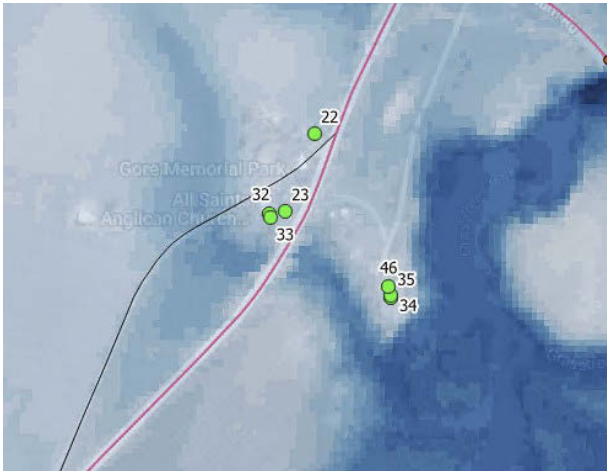
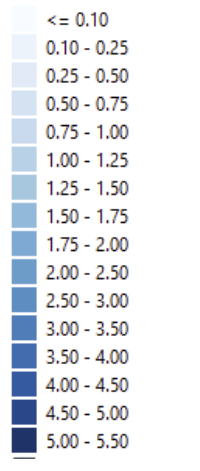
In addition to model validation to flood marker and gauge data recorded during the 2010 event, an assessment was undertaken to verify modelling results using available photographs of the 2010, 2013 and 2021 flood events. While the model validation process confirms the accuracy and reliability of the model in the 2010 event, the verification process involves comparing the modelled flood conditions to a broader range of historical flood events to ensure that the model can also reliably and consistently represent flood conditions across different events.

7.3.8.1 2010 flood event

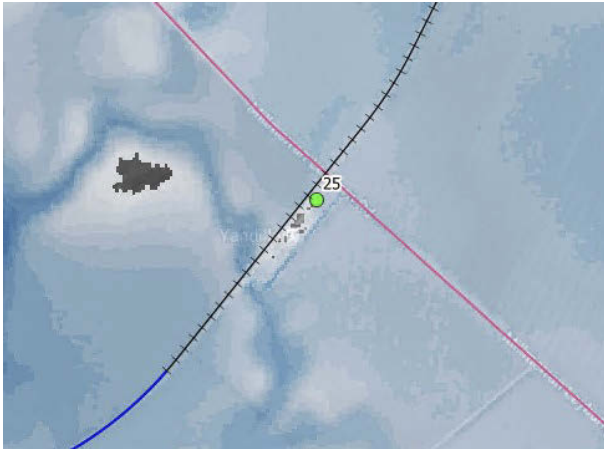
Photographs of the 2010 event were supplied directly by some landowners and acquired by FFJV or from an exhibition in the Millmerran Library (displayed 2017/18). Out of 400+ available photographs, 40+ photographs were obtained that relate directly to available flood markers. Except for the three gauges, all the flood markers are surveyed levels based on information provided by landowners. A degree of uncertainty therefore exists in terms of the accuracy of the flood markers, given the variables involved such as timing of the observation, wave action, localised hydraulic effects, infrastructure failure, etc.

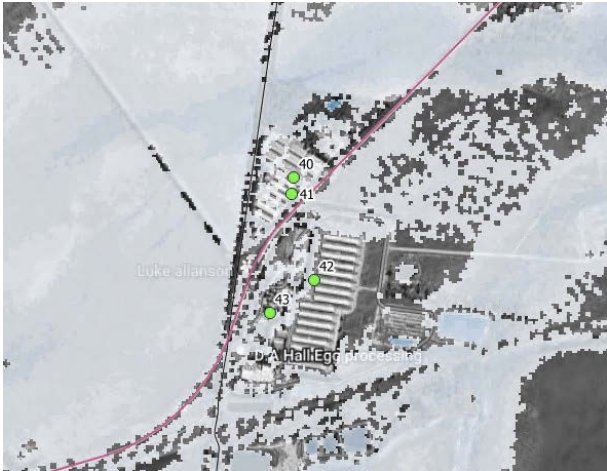
A summary of the flood photo verification is presented in Table 7.34. The small subset of photos was obtained after removal of duplicates or dry areas. Comparisons between the model results and flood photos demonstrate the strong capability of the model to reproduce flood extents and hydraulic behaviours along the Gore Highway between Pampas Road and Brose Lane.

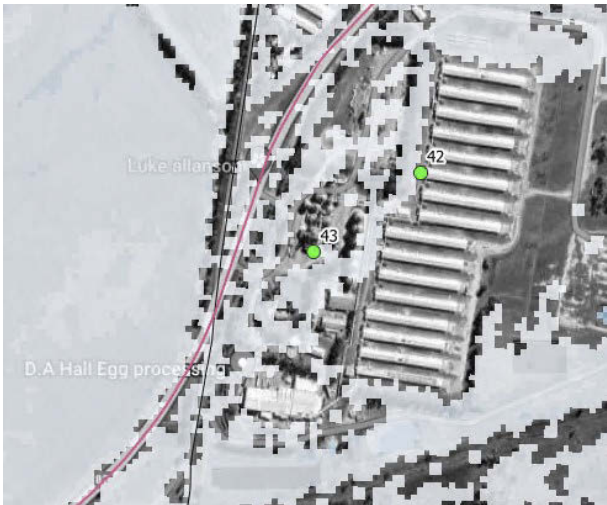
Table 7.34 Condamine River—Model verification against photographs—2010 flood event

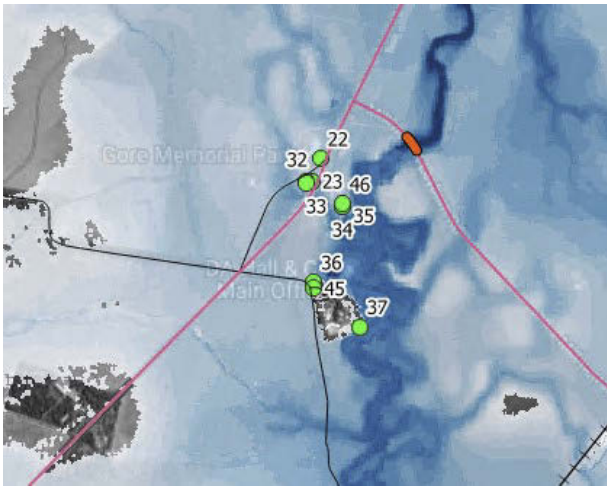
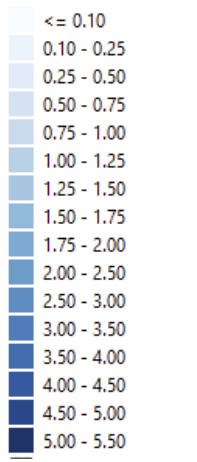
Photograph	Flood model extract (estimated depth, m)	Estimated flood depth (m)	Commentary
 House and sheds on Lot Plan 3RP35147, Millmerran-Leyburn Road			Photographed in 2010 during the flooding event at an unknown time. The flood model generally reproduced the peak flood levels within ± 200 mm, except a slight underestimation at #29 which was susceptible to runoff from within the property.
 House on Lot plan 1DY939, Yandilla			Photo taken after peak. Peak modelled levels are within ± 200 mm of surveyed peak flood levels.

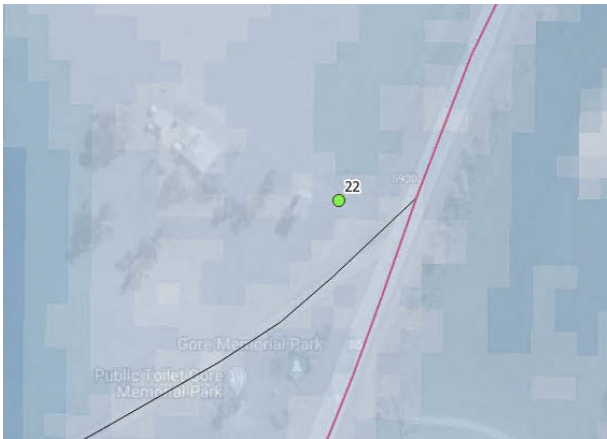
Photograph	Flood model extract (estimated depth, m)	Estimated flood depth (m)	Commentary
 Hall's Poultry offices and sheds surrounded by levee bank. 2010 level came right up to levee crest but did not overtop 		<= 0.10 0.10 - 0.25 0.25 - 0.50 0.50 - 0.75 0.75 - 1.00 1.00 - 1.25 1.25 - 1.50 1.50 - 1.75 1.75 - 2.00 2.00 - 2.50 2.50 - 3.00 3.00 - 3.50 3.50 - 4.00 4.00 - 4.50 4.50 - 5.00 5.00 - 5.50	Peak modelled level is within $\pm$ 200 mm of surveyed flood marker. Note that ID location 37 is a projected level by Hall's to determine where the flood level would have come up to had there been a breach in the levee bank. The area inside the levee bank did not actually flood.

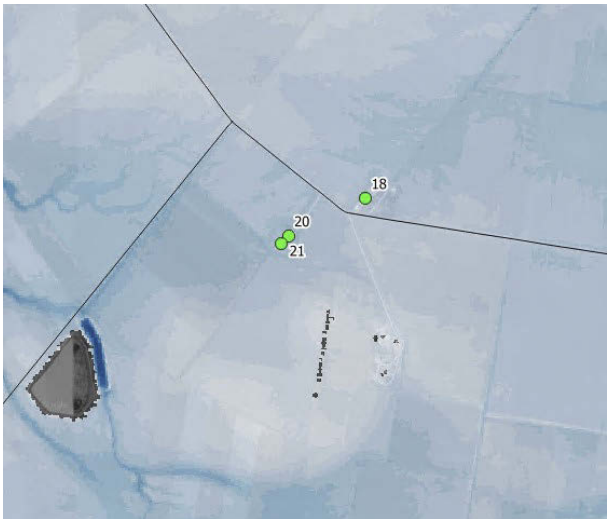
Photograph	Flood model extract (estimated depth, m)	Estimated flood depth (m)	Commentary
 Aerial photograph after peak (time unknown)		<= 0.10 0.10 - 0.25 0.25 - 0.50 0.50 - 0.75 0.75 - 1.00 1.00 - 1.25 1.25 - 1.50 1.50 - 1.75 1.75 - 2.00 2.00 - 2.50 2.50 - 3.00 3.00 - 3.50 3.50 - 4.00 4.00 - 4.50 4.50 - 5.00 5.00 - 5.50	Peak modelled level is within $\pm$ 200 mm of surveyed flood marker. Silos pad is dry but surrounded by flood water.
 Photograph of debris line at Yandilla Grain Silos denoting peak flood level			

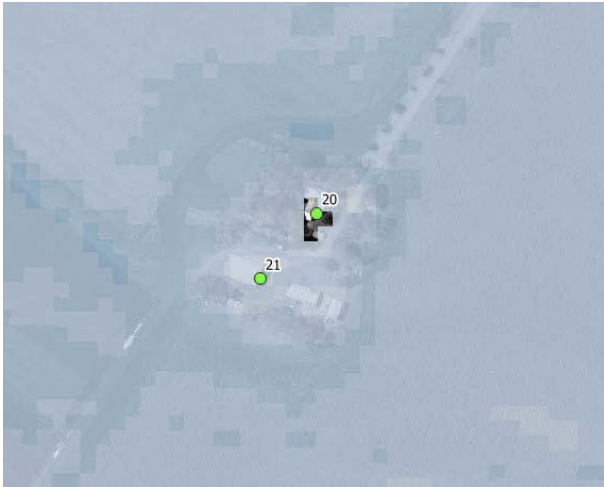
Photograph	Flood model extract (estimated depth, m)	Estimated flood depth (m)	Commentary
 Halls Poultry Sheds—north of Gore Highway		<= 0.10 0.10 - 0.25 0.25 - 0.50 0.50 - 0.75 0.75 - 1.00 1.00 - 1.25 1.25 - 1.50 1.50 - 1.75 1.75 - 2.00 2.00 - 2.50 2.50 - 3.00 3.00 - 3.50 3.50 - 4.00 4.00 - 4.50 4.50 - 5.00 5.00 - 5.50	Peak modelled level is within $\pm$ 200 mm of surveyed flood markers.

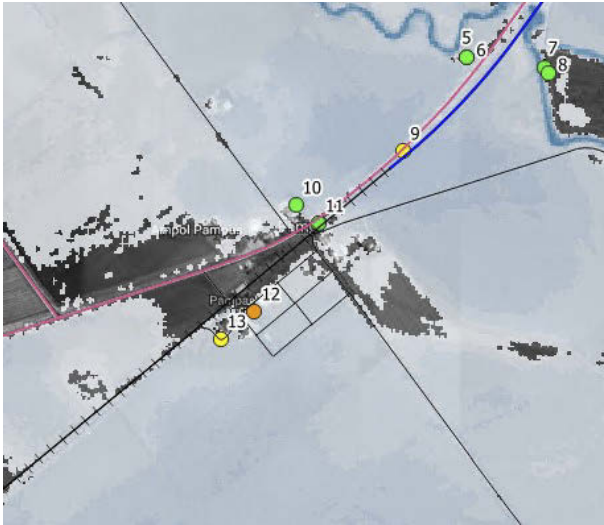
Photograph	Flood model extract (estimated depth, m)	Estimated flood depth (m)	Commentary
 Halls Poultry feedmill—south of Gore Hwy		<= 0.10 0.10 - 0.25 0.25 - 0.50 0.50 - 0.75 0.75 - 1.00 1.00 - 1.25 1.25 - 1.50 1.50 - 1.75 1.75 - 2.00 2.00 - 2.50 2.50 - 3.00 3.00 - 3.50 3.50 - 4.00 4.00 - 4.50 4.50 - 5.00 5.00 - 5.50	Peak modelled level is within $\pm$ 200 mm of surveyed flood marker.

Photograph	Flood model extract (estimated depth, m)	Estimated flood depth (m)	Commentary
 Yandilla Church			Good match $\pm$ 200 mm (one marker in building and two on fence).

Photograph	Flood model extract (estimated depth, m)	Estimated flood depth (m)	Commentary
 Letterbox in driveway of Lot Plan 2RP71762, Gore Hwy		<= 0.10 0.10 - 0.25 0.25 - 0.50 0.50 - 0.75 0.75 - 1.00 1.00 - 1.25 1.25 - 1.50 1.50 - 1.75 1.75 - 2.00 2.00 - 2.50 2.50 - 3.00 3.00 - 3.50 3.50 - 4.00 4.00 - 4.50 4.50 - 5.00 5.00 - 5.50	Peak modelled level is within ± 200 mm of surveyed flood marker.
 316 Fysh Road, Pampas		<= 0.10 0.10 - 0.25 0.25 - 0.50 0.50 - 0.75 0.75 - 1.00 1.00 - 1.25 1.25 - 1.50 1.50 - 1.75 1.75 - 2.00 2.00 - 2.50 2.50 - 3.00 3.00 - 3.50 3.50 - 4.00 4.00 - 4.50 4.50 - 5.00 5.00 - 5.50	Model underestimated peak flood level at house by over -300 mm. Possible survey uncertainties along Fysh Road, which regulates flow received at this location.

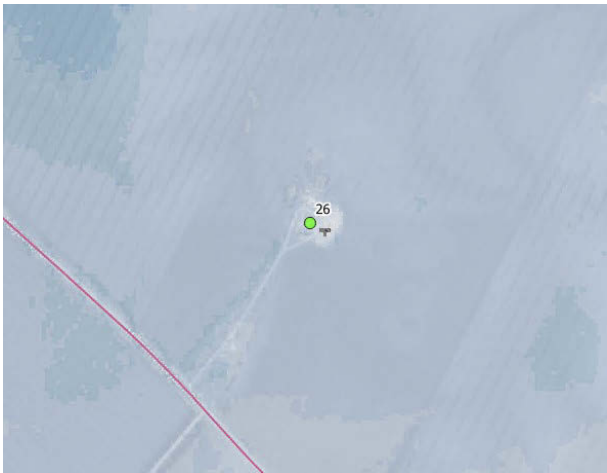
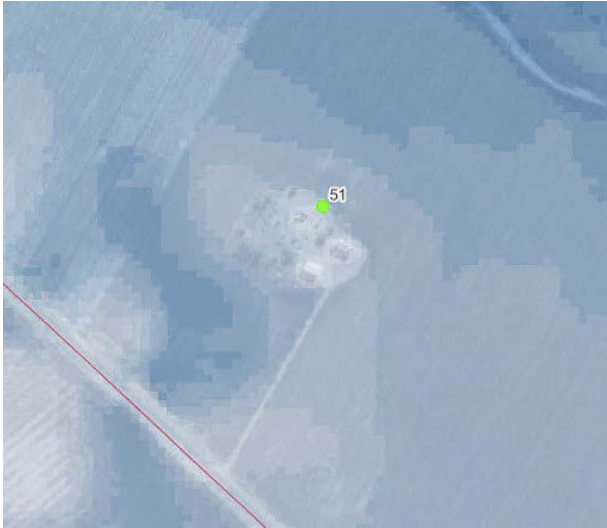
Photograph	Flood model extract (estimated depth, m)	Estimated flood depth (m)	Commentary
 Car travelling on Fysh Road  House on Lot Plan 1RP42427, Fysh Road		<= 0.10 0.10 - 0.25 0.25 - 0.50 0.50 - 0.75 0.75 - 1.00 1.00 - 1.25 1.25 - 1.50 1.50 - 1.75 1.75 - 2.00 2.00 - 2.50 2.50 - 3.00 3.00 - 3.50 3.50 - 4.00 4.00 - 4.50 4.50 - 5.00 5.00 - 5.50	Peak modelled level is within $\pm$ 200 mm of surveyed flood marker.

Photograph	Flood model extract (estimated depth, m)	Estimated flood depth (m)	Commentary
 House on Lot Plan 1RP48984, Fysh Road after peak  Shed on Lot Plan 1RP48984, Fysh Road		<= 0.10 0.10 - 0.25 0.25 - 0.50 0.50 - 0.75 0.75 - 1.00 1.00 - 1.25 1.25 - 1.50 1.50 - 1.75 1.75 - 2.00 2.00 - 2.50 2.50 - 3.00 3.00 - 3.50 3.50 - 4.00 4.00 - 4.50 4.50 - 5.00 5.00 - 5.50	Peak modelled level is within $\pm$ 200 mm of surveyed flood marker.

Photograph	Flood model extract (estimated depth, m)	Estimated flood depth (m)	Commentary
 Pampas		<= 0.10 0.10 - 0.25 0.25 - 0.50 0.50 - 0.75 0.75 - 1.00 1.00 - 1.25 1.25 - 1.50 1.50 - 1.75 1.75 - 2.00 2.00 - 2.50 2.50 - 3.00 3.00 - 3.50 3.50 - 4.00 4.00 - 4.50 4.50 - 5.00 5.00 - 5.50	This image was taken on the 28th December 2010, around 3 hrs after the peak. Modelled peak levels are within ± 200 mm of the surveyed flood levels at ID locations 10 and 11. ID locations 12 and 13 are discussed above under 316 Fysh Road, Pampas.
 House on Lot Plan 4RP55115, Pampas-Horrane Road		<= 0.10 0.10 - 0.25 0.25 - 0.50 0.50 - 0.75 0.75 - 1.00 1.00 - 1.25 1.25 - 1.50 1.50 - 1.75 1.75 - 2.00 2.00 - 2.50 2.50 - 3.00 3.00 - 3.50 3.50 - 4.00 4.00 - 4.50 4.50 - 5.00 5.00 - 5.50	Photo taken at 11am Tuesday 28th December ('few hours after the peak'). Flood extent matches observed flood extent; however, the modelled levels do not match the recorded levels. Possible survey uncertainties along Pampas-Horrane Road, which regulates flow received at this location.

Photograph	Flood model extract (estimated depth, m)	Estimated flood depth (m)	Commentary
 Gore Hwy between Pampas and North Branch		<= 0.10 0.10 - 0.25 0.25 - 0.50 0.50 - 0.75 0.75 - 1.00 1.00 - 1.25 1.25 - 1.50 1.50 - 1.75 1.75 - 2.00 2.00 - 2.50 2.50 - 3.00 3.00 - 3.50 3.50 - 4.00 4.00 - 4.50 4.50 - 5.00 5.00 - 5.50	During peak of flood. Peak modelled level generally within ± 200 mm of surveyed flood marker, except at ID locations 5 and 9 where the model was conservative. ID locations 5 and 6 appear to be the same location which had been surveyed twice, and the model would return a strong match if the average level was assumed at this location.
 Bunded property near Lemontree Road/Bostock Road intersection		<= 0.10 0.10 - 0.25 0.25 - 0.50 0.50 - 0.75 0.75 - 1.00 1.00 - 1.25 1.25 - 1.50 1.50 - 1.75 1.75 - 2.00 2.00 - 2.50 2.50 - 3.00 3.00 - 3.50 3.50 - 4.00 4.00 - 4.50 4.50 - 5.00 5.00 - 5.50	Good match despite flood marker not being available.

Photograph	Flood model extract (estimated depth, m)	Estimated flood depth (m)	Commentary
 Driveway on Lot Plan 2SP119342 north of Gore Hwy, between Pampas and Brookstead		<= 0.10 0.10 - 0.25 0.25 - 0.50 0.50 - 0.75 0.75 - 1.00 1.00 - 1.25 1.25 - 1.50 1.50 - 1.75 1.75 - 2.00 2.00 - 2.50 2.50 - 3.00 3.00 - 3.50 3.50 - 4.00 4.00 - 4.50 4.50 - 5.00 5.00 - 5.50	Timing unknown but debris line on air conditioning unit suggests the photo was taken after the peak. Despite flood markers not being available there is a good match between modelling information and photographs.
 Lot Plan 2SP119342 north of Gore Hwy, between Pampas and Brookstead (water reaching air conditioning unit)			

Photograph	Flood model extract (estimated depth, m)	Estimated flood depth (m)	Commentary
 House on Lot Plan 1RP53346 at Millmerran-Leyburn Road		<= 0.10 0.10 - 0.25 0.25 - 0.50 0.50 - 0.75 0.75 - 1.00 1.00 - 1.25 1.25 - 1.50 1.50 - 1.75 1.75 - 2.00 2.00 - 2.50 2.50 - 3.00 3.00 - 3.50 3.50 - 4.00 4.00 - 4.50 4.50 - 5.00 5.00 - 5.50	Peak modelled level is within $\pm$ 200 mm of surveyed flood marker.
 Lot Plan 3RP53023 located to the north east of Yandilla Silos. Fence post (water reaching just below the top of the fence railing) and bore pump station shed located within proximity to the residence.		<= 0.10 0.10 - 0.25 0.25 - 0.50 0.50 - 0.75 0.75 - 1.00 1.00 - 1.25 1.25 - 1.50 1.50 - 1.75 1.75 - 2.00 2.00 - 2.50 2.50 - 3.00 3.00 - 3.50 3.50 - 4.00 4.00 - 4.50 4.50 - 5.00 5.00 - 5.50	Taken close to the peak of the flood. Despite flood markers not being available there is a good match between modelling information and photographs.

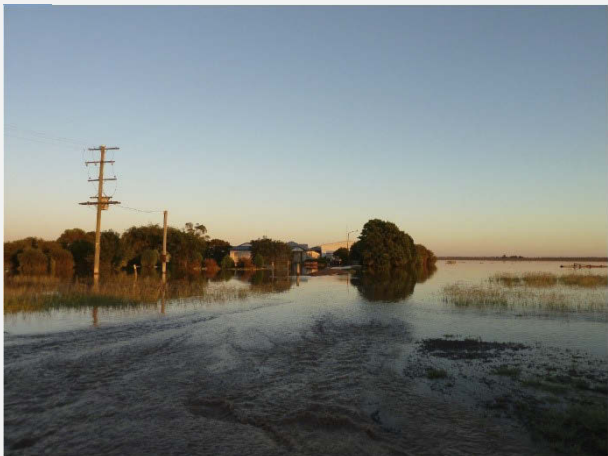
7.3.8.2 2013 flood event

An assessment was undertaken to verify the model results against additional flood photos of the 2013 flood event obtained from landowners. Of the 90 photos obtained, 20 were taken within a 10-hour window around the 2013 event flood peak, during which the peak flood level fell by approximately 0.5 m at the Tummalville gauge. Most of these photos were taken along Gore Highway between Pampas Road and Brose Lane, which remained trafficable by 4WD during the flood event.

A summary of the flood photo verification is presented in Table 7.35, which also indicates the approximate location and bearing of each photo taken. The small subset of photos was obtained after removal of duplicates or dry areas. Comparisons between the model results and flood photos demonstrate the strong capability of the model to reproduce flood extents and hydraulic behaviours along Gore Highway between Pampas Road and Brose Lane.

A direct validation of the 2013 flood event could not be undertaken because of the scarcity and uncertainty associated with flood marker data as most historical evidence of the flood was masked by the high water marks and damage left by the much rarer 2010 flood event.

Table 7.35 Condamine River—Model verification against photographs—2013 flood event

Photograph	Flood model extract (estimated depth, m)	Estimated flood depth (m)	Commentary
 Gore Highway looking west (Date taken: 30/01/2013 5:38 am)		<= 0.10 0.10 - 0.25 0.25 - 0.50 0.50 - 0.75 0.75 - 1.00 1.00 - 1.25 1.25 - 1.50 1.50 - 1.75 1.75 - 2.00 2.00 - 2.50 2.50 - 3.00 3.00 - 3.50 3.50 - 4.00 4.00 - 4.50 4.50 - 5.00 5.00 - 5.50	Photo taken 3.3 hours before flood peak. Both model and photo show shallow flooding above Gore Highway with negligible turbulence. The road remained trafficable on a 4WD with a typical depth around 30 mm.
 Gore Highway looking north (Date taken: 30/01/2013 5:40 am)		<= 0.10 0.10 - 0.25 0.25 - 0.50 0.50 - 0.75 0.75 - 1.00 1.00 - 1.25 1.25 - 1.50 1.50 - 1.75 1.75 - 2.00 2.00 - 2.50 2.50 - 3.00 3.00 - 3.50 3.50 - 4.00 4.00 - 4.50 4.50 - 5.00 5.00 - 5.50	Photo taken 3.3 hours before flood peak. Both model and photo show flooding up to the perimeter of the house. The model predicts shallow inundation over the surrounding paddocks which matches the photo.

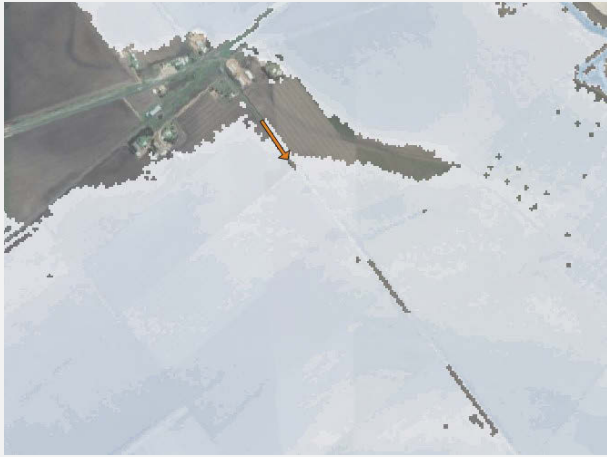
Photograph	Flood model extract (estimated depth, m)	Estimated flood depth (m)	Commentary
 Fysh Road at the first floodway looking southeast (Date taken: 30/01/2013 at 5:45 am)		<= 0.10 0.10 - 0.25 0.25 - 0.50 0.50 - 0.75 0.75 - 1.00 1.00 - 1.25 1.25 - 1.50 1.50 - 1.75 1.75 - 2.00 2.00 - 2.50 2.50 - 3.00 3.00 - 3.50 3.50 - 4.00 4.00 - 4.50 4.50 - 5.00 5.00 - 5.50	Photo taken 3.2 hours before the flood peak. The flow across the road appears slightly turbulent. The model shows a slightly wider overflow extent and generally consistent wet/dry patterns along the line of sight as in the photo.
 Gore Highway looking west (Date taken: 30/01/2013 at 11:26 am)		<= 0.10 0.10 - 0.25 0.25 - 0.50 0.50 - 0.75 0.75 - 1.00 1.00 - 1.25 1.25 - 1.50 1.50 - 1.75 1.75 - 2.00 2.00 - 2.50 2.50 - 3.00 3.00 - 3.50 3.50 - 4.00 4.00 - 4.50 4.50 - 5.00 5.00 - 5.50	Photo taken 2.4 hours after flood peak. Both the model and the photo show the shed remains dry despite the surrounding floodwater up to its perimeter. The model predicts a wider flood extent dominated with differences mostly accounted for by shallow standing water below 100 mm.

Table note:

The orange arrow indicates the directional view of the photograph

7.3.8.3 2021 flood event

Following a period of heavy rainfall, ARTC inspected the extent of inundation by plane on 1 December 2021, approximately 5 hours after the peak flood at Pampas. These photos are thought to be indicative of the actual flood peak during the event except on floodplain fringes, as the Tummalville gauge recorded a slow drawdown of only approximately 0.5 m within 2 days after the peak.

ARTC also visited major flooded areas along Gore Highway 2 weeks after the event on 14 December 2021, during which additional post-flood photographs were acquired. The observation covered Millmerran Leyburn Road (Grasstree Creek) and Gore Highway Bridge (Condamine River), both located within the Condamine River regional model domain.

A summary of the flood photo verification is presented in Table 7.36, which also indicates the approximate location and bearing of each photo taken. Although the 2021 event is much more frequent (15.3% AEP at Cecil Weir) than the 2010 event (0.8% AEP at Cecil Weir) that the flood model has been calibrated to, comparisons between the model results and flood photos show a good match between the observed and modelled flood extent, demonstrating that the model calibration is robust for flood assessment under a broad range of hydrologic conditions.

Table 7.36 Condamine River—Model verification against photographs—2021 flood event

Photograph	Flood model extract (estimated depth, m)	Legend	Commentary
 14 Dec 2021 at 9:13:40 am 22° N Pampas QLD Australia Index number:0 Photo TC_00000		★ Photo Location Indicative Depth (m) ≤ 0.25 0.25 - 0.50 0.50 - 0.75 0.75 - 1.00 1.00 - 1.25 1.25 - 1.50 1.50 - 1.75 1.75 - 2.00 2.00 - 2.25 2.25 - 2.50 2.50 - 2.75 2.75 - 3.00 3.00 - 3.25 3.25 - 3.50 3.50 - 3.75 3.75 - 4.00 4.00 - 4.25 4.25 - 4.50 4.50 - 4.75 > 4.75	Photo was taken 2 weeks after the flood. The model predicts road overtopping by 0.6 m during the event.
 14 Dec 2021 at 9:38:57 am 24° N Gore Highway Yendilla QLD 4352 Australia Index number:39 Photo TC_00039		★ Photo Location Indicative Depth (m) ≤ 0.25 0.25 - 0.50 0.50 - 0.75 0.75 - 1.00 1.00 - 1.25 1.25 - 1.50 1.50 - 1.75 1.75 - 2.00 2.00 - 2.25 2.25 - 2.50 2.50 - 2.75 2.75 - 3.00 3.00 - 3.25 3.25 - 3.50 3.50 - 3.75 3.75 - 4.00 4.00 - 4.25 4.25 - 4.50 4.50 - 4.75 > 4.75	Photo was taken 2 weeks after the flood. The model predicts road overtopping by 0.4 m during the event.

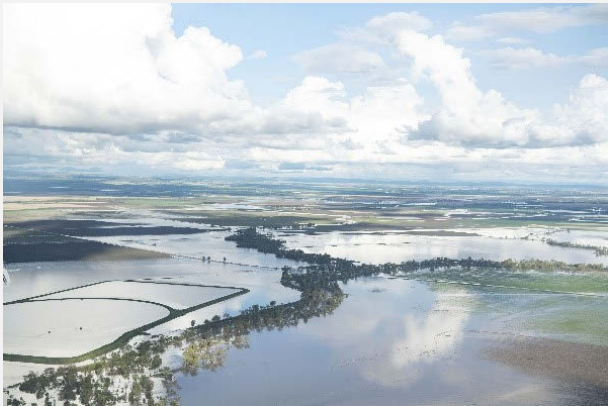
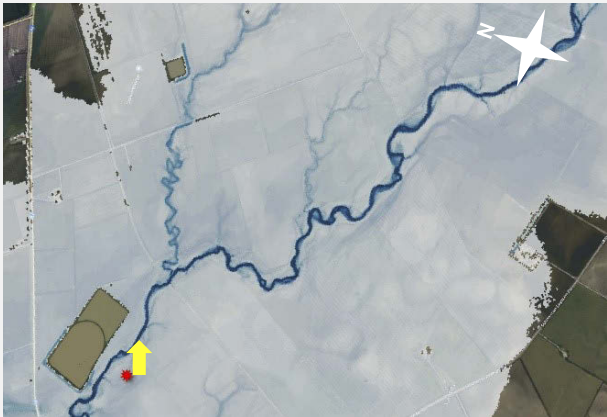
Photograph	Flood model extract (estimated depth, m)	Legend	Commentary
 <i>Aerial photo looking west towards Pampas floodplain</i>		★ Photo Location Indicative Depth (m) □ ≤ 0.25 □ 0.25 - 0.50 □ 0.50 - 0.75 □ 0.75 - 1.00 □ 1.00 - 1.25 □ 1.25 - 1.50 □ 1.50 - 1.75 □ 1.75 - 2.00 □ 2.00 - 2.25 □ 2.25 - 2.50 □ 2.50 - 2.75 □ 2.75 - 3.00 □ 3.00 - 3.25 □ 3.25 - 3.50 □ 3.50 - 3.75 □ 3.75 - 4.00 □ 4.00 - 4.25 □ 4.25 - 4.50 □ 4.50 - 4.75 □ > 4.75	The photo was taken approximately 5 hours after the flood peak. There is a good agreement between modelled and observed flood extents.
 <i>Aerial photo looking west towards Condamine River</i>		★ Photo Location Indicative Depth (m) □ ≤ 0.25 □ 0.25 - 0.50 □ 0.50 - 0.75 □ 0.75 - 1.00 □ 1.00 - 1.25 □ 1.25 - 1.50 □ 1.50 - 1.75 □ 1.75 - 2.00 □ 2.00 - 2.25 □ 2.25 - 2.50 □ 2.50 - 2.75 □ 2.75 - 3.00 □ 3.00 - 3.25 □ 3.25 - 3.50 □ 3.50 - 3.75 □ 3.75 - 4.00 □ 4.00 - 4.25 □ 4.25 - 4.50 □ 4.50 - 4.75 □ > 4.75	The photo was taken approximately 5 hours after the peak flood weeks after the flood. There is a reasonable agreement between modelled and observed flood extent.

Table note:

The yellow arrow indicates the directional view of the photograph

7.4 Existing Case modelling results—Condamine River

7.4.1 Existing Case flood maps

Existing conditions flood maps are presented in Volume 2 – Appendix C and via an [Online Digital Portal](#). Appendix C contains the Existing Case 20% and 1% AEP flood maps, while the [Online Digital Portal](#) contains the remainder of the modelled events. The location of each map set is summarised below:

- 20% AEP peak levels: Figure C3-A
- 1% AEP peak levels: Figure C3-B
- 20% AEP peak velocity: Figure C3-C
- 1% AEP peak velocity: Figure C3-D
- 20% AEP peak hazard: Figure C3-E
- 1% AEP peak hazard: Figure C3-F
- 20% AEP peak time of inundation: Figure C3-G
- 1% AEP peak time of inundation: Figure C3-H
- 20% AEP, 10% AEP, 5% AEP, 2% AEP, 1% AEP, 1 in 2,000 AEP, 1 in 10,000 AEP and PMF Existing Case results: [Online Digital Portal](#)
- For higher resolution mapping refer to the [Online Digital Portal](#).

7.4.2 Existing Case flood inundation extent and flood levels

Figure C3-B in Volume 2 – Appendix C shows the 1% AEP event indicative flood extent and peak water levels within the Condamine River floodplain for the Existing Case.

The Condamine River floodplain is characterised by flat terrain and sinuous channels, which begin to break their banks in a 50% AEP event, and then flow between branches in 20% AEP and larger events. The regional floodplain is formed by three interacting river branches: the Condamine River North Branch, Condamine River and a southern branch known as Grasstree Creek. Grasstree Creek discharges into Condamine River approximately 2.5 km downstream of the alignment at Centenary Bridge on Gore Highway. The confluence between North Branch and Condamine River is located approximately 45 km downstream of the Project alignment, or 6 km downstream of Cecil Plains. The 1% AEP event inundated width along the Project alignment between Grasstree Creek and Brookstead is approximately 12.5 km.

The Condamine River hydraulic model predicts an average 1% AEP event flood depth of around 8 m in the Condamine River main channel.

Grasstree Creek was assessed to have less capacity than Condamine River, with an average 1% AEP event flood depth of around 5 m predicted in the main channel.

The North Branch was assessed to have the smallest capacity, with an average 1% AEP event flood depth of 2 m predicted in the main channel.

The total floodplain conveyance is split around Pampas into a primary floodplain formed by interactions between Condamine River and Grasstree Creek (i.e. Condamine River floodplain), and a secondary floodplain of North Branch itself (i.e. North Branch floodplain). Under the 1% AEP event, the Condamine River floodplain is characterised by deep and slow-moving inundation, with typical flood depths between 1 to 1.5 m; the North Branch floodplain is both shallower and more stagnant, with typical depths under 0.5 m.

In a large event where the Condamine River and North Branch floodplains begin to combine, the model predicts a westerly flow diversion from the North Branch floodplain onto the Condamine River floodplain along Fysh Road (upstream of Gore Highway) and along Pampas-Horrane Road (downstream of Gore Highway). The overflow above Pampas-Horrane Road is minimal compared to the total flow conveyed through North Branch due to shallow overflow depth (around 200 mm) combined with high TW on the receiving floodplain resulting in partial submergence of the road.

7.4.3 Existing Case flood immunity of existing infrastructure

Table 7.37 presents a summary of overtopping depths for the existing QR Rail Line and key roads near the Project alignment under a range of design events. Generally, modelling results demonstrate that most roads and infrastructure in the Condamine River floodplain have a low existing flood immunity.

Table 7.37 Condamine River—Existing Case: Estimated overtopping depths of key infrastructure

Infrastructure	Location	Maximum overtopping depth (m)							
		PMF	1 in 10,000 AEP	1 in 2,000 AEP	1% AEP	2% AEP	5% AEP	10% AEP	20% AEP
Gore Highway	D/S of the proposed main branch bridge	4.26	2.94	2.08	1.37	1.18	0.94	0.70	0.39
Millmerran-Leyburn Road	Immediately U/S of proposed alignment	4.20	2.81	1.86	1.05	0.84	0.67	0.58	0.47
Pampas-Horrane Road	D/S of Project alignment	3.37	2.08	1.03	0.38	0.30	0.25	0.12	-
Existing QR Rail Line	Adjacent to Main Condamine Crossing	4.08	2.80	1.95	1.30	1.14	0.93	0.72	0.40
Existing QR Rail Line	Adjacent to North Branch Condamine Crossing	2.18	1.39	1.06	0.89	0.84	0.69	0.45	0.35
Yandilla Grain Silos	Yandilla, Immediately U/S of Project alignment	3.45	2.06	1.11	0.30	0.10	Dry	Dry	Dry

7.4.4 Existing Case velocities

Figure C3-D in Volume 2 – Appendix C shows the 1% AEP peak flood velocities within the Condamine River floodplain for the Existing Case.

For the 1% AEP:

- The model predicts typical velocities between 1.5 m/s and 2 m/s in the Condamine River channel, around 1 m/s in the Grasstree Creek channel, and less than 1 m/s in the North Branch channel.
- Floodplain velocities generally remain below 0.5 m/s.

7.4.5 Existing Case hazard

Figure C3-F in Volume 2 – Appendix C shows the 1% AEP peak flood hazard within the Condamine River floodplain for the Existing Case.

For the 1% AEP, the model demonstrates that:

- High flood hazard in excess of 1.0 m²/s occurs along all major channels, including Condamine River, Grasstree Creek and North Branch.
- Moderate flood hazard between 0.3 m²/s and 1.0 m²/s occurs over an approximate 5 km width of the Condamine River and Grasstree Creek floodplain.
- Low flood hazard below 0.3 m²/s occurs across approximate 50 per cent of the total regional floodplain width. These occurrences are associated with shallow inundations, typically 300 mm or less, combined with slow moving flows, at typically 1 m/s or less, which are primarily distributed along fringes on the Condamine River floodplain and along North Branch outside the main channel.

7.4.6 Existing Case time of inundation

Figure C3-H in Volume 2 – Appendix C shows the 1% AEP flood duration of inundation within the Condamine River floodplain for the Existing Case.

For the 1% AEP, the model demonstrates that:

- Most of the Condamine River floodplain and the Grasstree Creek floodplain are inundated by around 96–120 hours (4 to 5 days). Floodplain fringes are typically inundated by up to 48 hours (2 days).

Most of the North Branch floodplain is inundated by around 72 to 96 hours (3 to 4 days). Floodplain fringes are typically inundated by up to 48 hours (2 days).

7.5 Developed Case modelling results—Condamine River

7.5.1 Drainage structures

The hydraulic design of the major drainage structures was undertaken using the hydraulic model (1D and 2D approach).

Regionally, the Project provides a total opening width of more than 7,000 m on the Condamine River floodplain where most of the opening width (86 per cent) is delivered by bridges. At the Condamine River floodplain crossing the following structures are included:

- Four waterway bridges (with a combined opening width of >6 km)
- 69 RCP locations (a total of 525 cells with a total opening width of approximately 900 m)
- 14 RCBC locations (a total of 76 cells with a total opening width of approximately 105 m).

Local drainage structures are presented in the cross-drainage register in Appendix B. These structures were sized through the local drainage design and depending on their interaction with the Condamine River floodplain were incorporated in the hydraulic model.

Bridges and viaduct structures are summarised in Table 7.38 and details of the proposed culvert structures are summarised in the Cross-Drainage Register located in Appendix B. The location of cross-drainage features has been presented in Figures C1-F to C1-K in Volume 2 – Appendix C.

Table 7.38 Condamine River—Proposed bridge locations and details

Chainage (km)	Structure ID	Approximate span (m)	Deck width (m)	Soffit level (m)	Deck superstructure type	Deck depth (mm)
139.29	310-BR21/22	1,330	3.97	382.05	Type D5	1,000
142.63	310-BR24	658	3.97	382.05	Type D5	1,000
143.89	310-BR25/26	2,577	3.97	380.60	Type D1	2,450
149.05	310-BR27	1,568	3.97	383.79	Type D5	1,000

Bridges were represented within the hydraulic model through use of LFC. Each bridge/viaduct within the model has had a flow constriction coefficient and percentage blocked value applied to represent obstruction of waterway area due to the piers, which were estimated based on the physical waterway area blockage from ARTC bridge drawings and standard details. No specific allowance for debris blockage at bridges was made, as the likelihood of significant blockage is low, given the nature of the catchments and the predominant vegetation of the waterways. The optimisation of bridge lengths was balanced between minimising the changes to the hydraulic regime, primarily afflux and velocities, and the cost of replacing bridge spans by large earth embankments.

Noise Barriers were incorporated into the hydraulic model through Brookstead and are represented within the TUFLOW model by raising the existing ground levels by 4 m along the length of the barrier, running parallel to the Project alignment and crossing Brookstead Scrubby Road. The barrier is located outside the PMP regional flood extent and does not contribute to an increase in impacts.

Blockage of hydraulic structures was assessed in accordance with ARR 2019 and can vary with respect to event magnitude and the catchment conditions, such as slopes and ambient vegetation. However, as the catchment characteristics were determined to be fairly consistent throughout the catchment, a uniform blockage factor at each structure was expected. After each culvert was independently assessed, a blockage factor of 25 per cent was confirmed as suitably conservative to be adopted across all culverts.

7.5.2 Hydraulic design criteria outcomes

The hydraulic model was run for the Developed Case for the range of events (20%, 10%, 5%, 2%, 1% AEP events and 1 in 2,000 AEP, 1 in 10,000 AEP and PMF events) with the outcomes presented in the following sections.

7.5.2.1 Rail immunity and structures results

Appendix B summarises the peak 1% AEP water levels on the upstream side of the proposed alignment.

Local drainage structures (i.e. those not included in the flood model) and road culverts are not reported. The results of flood modelling indicate that a 1% AEP event flood immunity to the proposed rail formation level is achieved for the Project alignment across the Condamine River floodplain.

For the 1% AEP flood event:

- The freeboard to bridge deck levels ranges from 2.24 m to 2.66 m
- The freeboard to rail formation in embankment areas ranges from 0.74 m to 1.90 m.

It should be noted that the formation levels were generally dictated by other engineering criteria and constraints.

7.5.3 Flood impact objectives outcomes—Condamine River

The impact of the Developed Case was assessed through inclusion of the proposed Project rail design and comparison of model results against the Existing Case results.

Changes to flooding behaviour that were assessed, and are reported in the following sections, include potential changes in peak water levels, duration of inundation, velocities, hazard and peak flows within the floodplain.

- Changes in 20% AEP and 1% AEP peak water levels are presented in Figures C5-A and C5-B in Volume 2 – Appendix C
- Changes in 20% AEP and 1% AEP velocities are presented in Figures C5-C and C5-D in Volume 2 – Appendix C
- Changes in 20% AEP and 1% AEP hazard are presented in Figures C5-E and C5-F in Volume 2 – Appendix C

- Changes in 20% AEP and 1% AEP time of inundation are presented in Figures C5-G and C5-H in Volume 2 – Appendix C
- Changes in 20% AEP and 1% AEP velocities and percentage change in velocities with cut-offs applied are presented in Figures C6-A to C6-F in Volume 2 – Appendix C
- Percentage changes in 20% AEP and 1% AEP in hazard are presented in Figures C7-A and C7-B in Volume 2 – Appendix C
- Percentage changes in 20% AEP and 1% AEP in time of inundation and with cut-offs applied are presented in Figures C8-A to C8-D in Volume 2 – Appendix C
- Refer to the [Online Digital Portal](#) for other modelled AEP results and higher resolution mapping.

One-on-one consultation with landowners who are expected to experience changes in flooding behaviour on their property was conducted by ARTC supported by FFJV. Refer to EIS Chapter 14: Flooding and Geomorphology and EIS Appendix E: Consultation Report for further details.

The Project design outcomes relative to the FIOs (Table 4.2) are presented in the following sections.

Flood-sensitive receptors referred to in this section are shown in Figures P1 (1 to 59) in Volume 2 – Appendix P.

7.5.3.1 Flood impacts on flood sensitive receptors

Flood sensitive receptors were identified from aerial imagery and include buildings and assets such as dwellings, sheds, pump stations and silos.

A tabulation of FSRs which are likely to experience afflux FIO exceedances in events up to and including the 1 in 2,000 AEP event are presented in Table 7.39.

Of a total of 384 FSRs assessed on the Condamine River regional floodplain, potential afflux FIO exceedances are predicted at 12 FSRs, including six houses and six non-habitable structures. These exceedances occur due to flow retardation upstream of Project rail embankment in infrequent to rare flood events (i.e. 2% and 1% AEP events). The FIO afflux limit is not exceeded during extreme events (i.e. 1 in 2,000 AEP event) on the Condamine River floodplain.

Exceedances experienced at CON_ID_275, CON_ID_277 (both houses) and COND_ID_160 occur due to the complex interface between the proposed rail embankment, Fysh Road upgrade and the existing Gore Highway. This area of the Condamine River floodplain acts as a flood storage area during flood events with existing culverts acting as balancing culverts across the Gore Highway rather than flowpaths. The presence of the design in this location reduces available flood storage area resulting in an increase in water level and extent in the design case. Within this area, significant optimisation of the proposed rail and cross-drainage design has been undertaken during the reference design development to minimise impacts on associated land and infrastructure in this area and mimic the existing case flood dynamics. Although this optimisation has occurred in this area, residual impacts on both FSRs detailed above will likely persist through the detailed design stage even with further optimisation of the design. In order to mitigate these exceedances, ongoing consultation with the impacted landowner will be undertaken to identify at property specific mitigation measures to reduce residual impacts.

No potential flood hazard exceedances were identified on the Condamine River floodplain. Flood hazard changes on floodplains are generally induced by flow redistribution around large waterway openings (i.e. bridges). Within the Condamine River floodplain, bridges have been optimised and positioned to minimise flow regime disturbances around habitable and commercial structures.

Mitigation of impacts on FSRs is individual to each landholder and will require ongoing consultation with those impacted to agree on appropriate mitigation specific to each site. ARTC will continue through the next stages of design to engage with impacted landowners to negotiate site specific mitigation measures.

Table 7.39 **Condamine River—Flood Impact Objective exceedances at flood sensitive receptors**

FSR ID	Description	FIO target (up to 1% AEP)	FIO target (1 in 2,000-year AEP)	Afflux (mm)					
				20%	10%	5%	2%	1%	1 in 2,000
CON_ID_78	House	10 mm	250 mm	-	-	-	+18	+25	-
CON_ID_120	House			-	-	-	+20	+29	-
CON_ID_205	House			-	-	-	-	+12	-
CON_ID_275	House			-	-	-	+26	+34	-
CON_ID_277	House			-	-	-	+28	+37	-
CON_ID_148	House			-	-	-	-	+21	-
CON_ID_118	Shed	100 mm	250 mm	-	-	-	-	+126	-
CON_ID_160	Shed			-	-	-	-	+188 ¹	-
CON_ID_5	Silos			-	-	-	-	+146	-
CON_ID_6	Silos							+35 ¹	
CON_ID_7	Silos			-	-	-	-	+144	-
CON_ID_8	Silos					+1 ¹	-	-	-

Table note:

1 Newly wet.

7.5.3.2 Flood impacts on State-controlled roads

Table 7.40 presents a list of State-controlled road sections within the Condamine River hydraulic model extent where potential FIO exceedances due to the Project were identified. The impacted road sections are shown in Figure 7.54. A summary of key impacts, exceedance justification and mitigation measures is presented below with further detail provided in Table 7.40:

- Time of inundation and afflux FIO exceedances occur on Gore Highway between Pampas Road and Elsdon Road (road section ID #331), which is assessed to have a less than 20% AEP flood immunity in the Existing Case. The impacts are related to an increase in ponding at Pampas, which also drives a slightly higher proportion of flow through North Branch onto its northern floodplain. This increase flow is concentrated through proposed culverts and bridge openings resulting in these increases. The FIO exceedances occur on non-critical sections of the road and results in negligible increases (i.e. < 1 per cent) to the time of inundation, which governs the overall accessibility and egress of this section of Gore Highway. Future improvement in cross-drainage and road design maturity around Pampas are expected to mitigate these exceedances.
- Time of inundation and afflux FIO exceedances occur on over 2 km of pavement along Millmerran-Leyburn Road (ID #483), which is assessed to have a less than 20% AEP immunity in the Existing Case. The impacts are primarily driven by increases in flood levels on both sides of the road due to the proposed road upgrade. The FIO exceedances occur on non-critical sections of the road and have negligible impact on the time of inundation of the Millmerran-Leyburn Road section between Reichle Road and Gore Highway. Future improvement in cross-drainage and road design maturity at this location are expected to mitigate these exceedances.
- Time of inundation exceedances occur on isolated sections along Pampas-Horrane Road (ID #350), which is assessed to have a less than 10% AEP flood immunity in the Existing Case. In the Developed Case, Pampas-Horrane Road would experience slightly reduced flood levels to the upstream side associated with small time of inundation reductions along the road. Ongoing model strengthening is expected to verify these exceedances.



Figure 7.54 Condamine River—Impacts to State-controlled roads

Table 7.40 Condamine River - Summary of flood impacts on State-controlled roads

Road Object ID	Road Segment	Road Name	Surface Type	Flood Impact Objective Target					AEP	Maximum FID Exceedance					Length (m)				Exceedance Justification	Proposed Mitigation Measures ¹
				Change in Peak Water Level (mm)	Change in Time of Inundation (%)	Change in Velocity (%)	Change in Velocity (m/s)	Change in Hazard (%)		Change in Peak Water Level (mm)	Change in Time of Inundation (%) ²	Change in Velocity (%) ³	Change in Velocity (m/s)	Change in Hazard (%) ³	Afflux	Time on Inundation	Velocity	Hazard		
483	16015103990000232940	Millmeran - Leyburn Road	Sealed	10	10	20	1	10	20%	151	135	-	-	-	490.00	120.00	-	-	Millmeran-Leyburn Road is subjected to exceedances due to interface between the rail embankment and road upgrades limiting conveyance of water downstream and creating ponding on the upstream side of the alignment. Exceedances experienced are directly south east of the proposed Millmeran-Leyburn Road upgrade along the existing road alignment. A review of the time of submergence results in both the existing and design case for this road segment demonstrates that the localised increase in water level along this road due to the presence of the Project do not detrimentally impact the operational capacity of the road. Exceedances remain on this segment are due to the maturity of the rail/rail interface and proposed cross drainage design only being a reference design level. To date, significant optimisation of the design around this location has been undertaken and therefore it is anticipated that there will be residual exceedances on this Lot after design optimisation in the detailed design phase.	Further road and/or rail design optimisation investigations will be undertaken in detailed design, but it is anticipated residual exceedances will remain. In this case, discussions with the appropriate road authority and relevant stakeholders will be undertaken to discuss the impacts and their overall impact on the operational capacity of the road link.
									10%	61	65	-	-	-	930.00	190.00	-	-		
									5%	82	55	-	-	-	1270.00	160.00	-	-		
									2%	90	35	-	-	-	1850.00	70.00	-	-		
									1%	99	30	-	-	-	2480.00	50.00	-	-		
									0.05%	-	-	-	-	-	-	-	-	-		
331	16015103990000145822	Gore Highway	Sealed	10	10	20	1	10	20%	78	17	-	-	-	20.00	20.00	-	-	Exceedances occur along Gore Hwy due to the proposed rail embankment impeding floodwater and diverting flow to the bridge and culvert structures. The Gore Highway is located immediately downstream and runs parallel to the proposed rail embankment. The exceedances experienced on the highway are concentrated around the outlets of culverts C150.08 to C151.30 and adjacent bridge 310-BR27. Due to the presence of the embankment, flow is redirected to cross drainage structures resulting in more concentrated flowpaths than the existing case. A review of the time of submergence results in both the existing and design case for this road segment demonstrates that the localised increase in water level and time of inundation along this road due to the presence of the Project do not detrimentally impact the operational capacity of the road. The exceedances remain due to the current design being at a reference design stage and culvert placement and spacing being grouped (rather than spaced across the embankment), resulting in concentrated flow. Further iteration of cross drainage and placement needs to be undertaken at detailed design stage. Significant optimisation of the design around this location has been undertaken to date and therefore it is anticipated that there will be some residual exceedance on this road after design optimisation in the detailed design phase.	Further rail design optimisation investigations will be undertaken in detailed design, but it is anticipated residual exceedances will remain. In this case, discussions with the appropriate road authority and relevant stakeholders will be undertaken to discuss the impacts and their overall impact on the operational capacity of the road link.
									10%	177	216	-	-	-	70.00	30.00	-	-		
									5%	148	11	-	-	-	60.00	20.00	-	-		
									2%	148	12	-	-	16	140.00	20.00	-	140.00		
									1%	89	31	-	-	39	260.00	70.00	-	280.00		
									0.05%	-	-	-	-	-	-	-	-	-		
350	16015103990000270389	Pampas - Horrane Road	Sealed	10	10	20	1	10	20%	-	-	-	-	-	-	-	-	-	Lack of available data for model validation and ongoing model strengthening.	Ongoing model strengthening.
									10%	-	-	-	-	-	-	-	-	-		
									5%	-	38	-	-	-	-	60.00	-	-		
									2%	-	16	-	-	-	-	10.00	-	-		
									1%	-	-	-	-	-	-	-	-	-		
									0.05%	-	-	-	-	-	-	-	-	-		

Table Notes

1. Time of Submergence has been measured across the full length of the road segment and does not correspond to the exact location of exceedance within that segment

2. Change in water level (afflux) is calculated based on the maximum water surface level difference between the existing and design case results, and does not consider the underlying topography. Depth values presented are based on the respective underlying topographies and in some cases do not align with afflux calculations. This can be attributed to interpolation calculations of results performed within the hydraulic model.

3. The above table should be read in conjunction with change maps provided as part of Volume II and the Digital Platform.

4. The symbol "-" denotes that no change above Flood Impact Objectives has been determined and therefore no result has been presented.

5. Changes recorded above may only occur during a small fraction of model simulation time.

6. Three decimal places do not infer absolute accuracy in modelling results, but it is necessary to identify small changes.

7. Design optimisations that could be implemented (but not limited to) include adjustment of culvert spacing, realignment of culverts to existing flowpaths when more detailed survey is available, modifying the grade of culverts, road height and alignment adjustments, removal of bridge abutments from waterway extents etc

Depth (m) ²		Time of Inundation in hours (related to % exceedance)		Time of Submergence (hours) ²		Velocity in m/s (related to % exceedance)		Velocity in m/s (related to actual change exceedance)		Hazard in m/s (related to % exceedance)	
Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case
-	0.151	47.08	110.85	121.62	122.95	-	-	-	-	-	-
0.713	0.192	75.53	134.25	132.55	132.55	-	-	-	-	-	-
0.827	0.328	83.28	128.70	140.55	140.63	-	-	-	-	-	-
1.022	0.530	94.75	128.38	146.48	146.44	-	-	-	-	-	-
1.104	0.182	101.04	131.78	150.52	150.74	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
0.001	0.001	32.50	38.03	139.04	139.42	-	-	-	-	-	-
0.013	0.012	2.53	7.99	144.03	144.32	-	-	-	-	-	-
0.021	0.021	22.93	25.44	146.53	147.23	-	-	-	-	-	-
0.002	0.002	24.34	27.31	126.02	103.61	-	-	-	-	0.16	0.19
0.030	0.030	22.82	29.90	131.44	112.14	-	-	-	-	0.13	0.19
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
-	-	64.48	88.86	102.68	102.28	-	-	-	-	-	-
-	-	7.23	8.40	106.78	107.24	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-

7.5.3.3 Flood impacts on local public roads

Table 7.41 presents a list of local public road sections within the Condamine River hydraulic model extent where potential FIO exceedances due to the Project were identified. The impacted road sections are shown in Figure 7.55. A summary of key impacts, exceedance justification and mitigation measures is presented below with further detail provided in Table 7.41:

- Afflux related FIO exceedances occur on two sections of Hall Road outside the left bank of Grasstree Creek (road section IDs #524 and #675). While long pavement sections (up to nearly 3 km) may experience flood level increases in the order of 10 to 20 mm due to the flow constriction at the proposed Grasstree Creek Bridge, the overtopping duration is not impacted as the existing hydrograph shape through Grasstree Creek is maintained.
- Afflux-related FIO exceedances occur on three Fysh Road sections (IDs #715, #716 and #214) and one Elsdon Road section between Fysh Road and Gibbs Road (ID #712). These are related to increased ponding at Pampas and may be mitigated by future development in cross-drainage maturity to improve the flow balance between Pampas and North Branch. Scattered duration impacts also occur on Elsdon Road, which do not impact the access and egress of this road section as the time of inundation on this section is negligibly affected.
- Afflux and time of inundation-related FIO exceedances occur on Gibbs Road (ID #708) and Elsdon Road between Gibbs Road and Gore Highway (ID #713), which are due to increased ponding from local flow redistribution around the right abutment of the proposed North Branch Bridge. The storage area formed between Gore Highway and Elsdon Road further increases the afflux sensitivity. Future development in cross drainage maturity along the northern approach of North Branch bridge are expected to mitigate the impacts in the area.
- Scattered impacts that just exceed the FIO target were identified on Hall Road downstream of the proposed Project embankment (ID #521), on Gilgai Lane (ID #517) and Brose Road (ID #172) along the Condamine River, and on Hanlon Road (ID #171), Bostock Road (ID #547) and Pampas Road (ID #556) along the North Branch. These potential impacts occur close to the limit of model resolution (i.e. 10 to 30 m of pavement) and are typically sensitive to the coarseness of topographic data representation in the hydraulic model, and as such they are generally classified as low risk technical exceedances. Ongoing model strengthening will help verify such scattered impacts and improve model confidence in hydraulically sensitive areas.

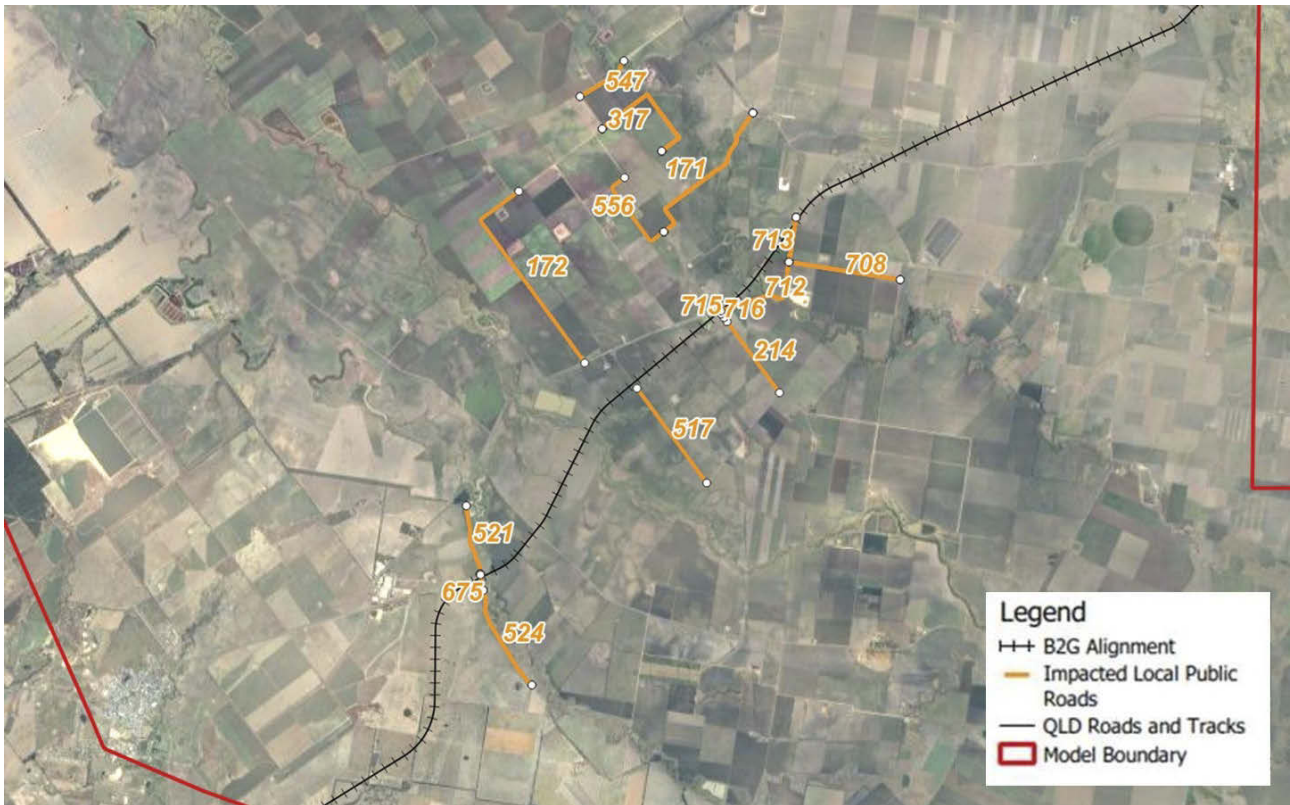


Figure 7.55 Condamine River—Impacts to local public roads

Table 7.41 Condamine River - Summary of impacts on local public roads

Object ID	Road Segment	Road Name	Surface Type	Flood Impact Objective Target					AEP	Maximum FIO Exceedance					Length (m)				Exceedance Justification	Proposed Mitigation Measures ²
				Change in Peak Water Level (mm)	Change in Time of Inundation (%)	Change in Velocity (m/s)	Change in Velocity (m/s)	Change in Hazard (%)		Change in Peak Water Level (mm)	Change in Time of Inundation (%) ³	Change in Velocity (%) ³	Change in Velocity (m/s)	Change in Hazard (%) ³	Afflux	Time on Inundation	Velocity	Hazard		
713	16015103990000114440	Eldston Road	Sealed	10	10	20	1	10	20%	138	233	-	-	-	660.00	60.00	-	-	Eldston Road is immediately upstream of the rail embankment and adjacent culverts C150.68 to C151.30. At this location the rail embankment impedes floodwaters and redirects flow towards the culvert structures. This redirection causes localised upstream ponding (afflux) and increases in time of inundation. In this location the rail embankment impedes the existing overland flow regime diverting flow to the bridge structure. A review of the time of submergence results in both the existing and design case for this road segment demonstrates that the localised increase in water level and time of inundation along this road due to the presence of the Project do not detrimentally impact the operational capacity of the road. The exceedances remain due to the current design being at a reference design stage and culvert placement and spacing being grouped (rather than spaced across the embankment), resulting in concentrated flow. Further iteration of cross drainage and placement needs to be undertaken at detailed design stage. Significant optimisation of the design around this location has been undertaken to date and therefore it is anticipated that there will be some residual exceedance on this road after design optimisation in the detailed design phase.	Further road and/or rail design optimisation investigations will be undertaken in detailed design, but it is anticipated residual exceedances will remain. In this case discussions with the appropriate road authority and relevant stakeholders will be undertaken to discuss the impacts and their overall impact on the operational capacity of the road link.
									10%	118	444	-	-	-	680.00	100.00	-	-		
									5%	23	247	-	-	-	1000.00	150.00	-	-		
									2%	14	123	-	-	-	570.00	240.00	-	-		
									1%	16	63	-	-	230	950.00	200.00	-	80.00		
									0.05%	-	-	-	-	-	-	-	-	-		
171	16015103990000155862	Hanton Road	Unsealed	10	10	10	0.5	10	20%	52	53	-	-	-	20.00	60.00	-	-	Maturity of current bridge design is limited by available information (e.g. detailed survey)	Further cross-drainage optimization investigations are required during detailed design.
									10%	125	20	-	-	-	40.00	30.00	-	-		
									5%	74	-	-	-	-	-	-	-	-		
									2%	-	-	-	-	-	-	-	-	-		
									1%	-	-	-	-	-	-	-	-	-		
									0.05%	-	-	-	-	-	-	-	-	-		
517	16015103990000139007	Gilgal Lane	Unsealed	10	10	10	0.5	10	20%	-	-	-	-	-	-	-	-	-	Lack of available data for model validation and ongoing model strengthening.	Ongoing model strengthening.
									10%	-	-	-	-	-	-	-	-	-		
									5%	-	-	-	-	-	-	-	-	-		
									2%	-	-	-	-	-	-	-	-	-		
									1%	-	-	-	-	-	-	-	-	-		
									0.05%	-	-	-	-	-	-	-	-	-		
708	16015103990000138467	Gibbs Road	Unsealed	10	10	10	0.5	10	20%	13	17	-	-	-	20.00	10.00	-	-	Maturity of current bridge design is limited by available information (e.g. detailed survey)	Further cross-drainage optimization investigations are required during detailed design.
									10%	59	46	-	-	-	400.00	60.00	-	-		
									5%	11	19	-	-	-	400.00	90.00	-	-		
									2%	-	-	-	-	-	-	-	-	-		
									1%	-	-	-	-	-	-	-	-	-		
									0.05%	-	-	-	-	-	-	-	-	-		
547	16015103990000039598	Bostock Road	Unsealed	10	10	10	0.5	10	20%	-	-	-	-	-	-	-	-	-	Lack of available data for model validation and ongoing model strengthening.	Ongoing model strengthening.
									10%	28	-	-	-	-	20.00	-	-	-		
									5%	-	-	-	-	-	-	-	-	-		
									2%	-	-	-	-	-	-	-	-	-		
									1%	-	-	-	-	-	-	-	-	-		
									0.05%	-	-	-	-	0.002	-	-	-	10.00		
556	16015103990000270409	Pampus Road	Unsealed	10	10	10	0.5	10	20%	-	-	-	-	-	-	-	-	-	Lack of available data for model validation and ongoing model strengthening.	Ongoing model strengthening.
									10%	-	-	-	-	-	-	-	-	-		
									5%	-	-	-	-	-	-	-	-	-		
									2%	-	-	-	-	-	-	-	-	-		
									1%	-	-	-	-	-	-	-	-	-		
									0.05%	-	-	-	-	-	-	-	-	-		
712	16015103990000114439	Eldston Road	Sealed	10	10	20	1	10	20%	11	58	-	-	-	570.00	10.00	-	-	Maturity of current bridge design is limited by available information (e.g. detailed survey)	Further cross-drainage optimization investigations are required during detailed design.
									10%	-	-	-	-	-	20.00	-	-	-		
									5%	29	40	-	-	-	1360.00	20.00	-	-		
									2%	-	-	-	-	-	1290.00	20.00	-	-		
									1%	40	39	-	-	-	-	-	-	-		
									0.05%	-	-	-	-	-	-	-	-	-		
675	163837962500000000011	Hall Road	Unsealed	10	10	10	0.5	10	20%	-	-	-	-	-	-	-	-	-	Maturity of current bridge design is limited by available information (e.g. detailed survey)	Further cross-drainage optimization investigations are required during detailed design.
									10%	-	-	-	-	-	-	-	-	-		
									5%	-	-	-	-	-	-	-	-	-		
									2%	-	-	-	-	-	-	-	-	-		
									1%	-	-	-	-	-	-	-	-	-		
									0.05%	-	-	-	-	-	-	-	-	-		
715	16015103990000123521	Fysh Road	Sealed	10	10	10	1	10	20%	-	-	-	-	-	-	-	-	-	Afflux exceedances occur on this road segment due to the presence of the proposed Fysh Road Upgrade. The current road design restricts flow moving north west towards the Gore Highway resulting in upstream afflux on the road segment. It is proposed to undertake a review of the Fysh Road Upgrade cross drainage design in the detailed design phase to reduce the afflux exceedances on this road segment. Exceedances remain on this road due to the maturity of the road and proposed cross drainage design only being at a reference design level with further optimisation to occur in the detailed design phase. It is anticipated that a reduction in afflux and time of inundation on this link could be achieved through a review and optimisation of the road cross drainage design in this location.	Further cross-drainage optimization investigations are required during detailed design.
									10%	-	-	-	-	-	-	-	-	-		
									5%	90	-	-	-	-	40.00	-	-	-		
									2%	132	-	-	-	-	50.00	-	-	-		
									1%	150	-	-	-	-	50.00	-	-	-		
									0.05%	-	-	-	-	-	-	-	-	-		
716	16015103990000133522	Fysh Road	Sealed	10	10	20	1	10	20%	-	-	-	-	-	-	-	-	-	Maturity of current road design is limited by available information (e.g. detailed survey)	Further road and or rail design optimization investigations are required during detailed design.
									10%	-	-	-	-	-	-	-	-	-		
									5%	-	-	-	-	-	-	-	-	-		
									2%	20	-	-	-	-	10.00	-	-	-		
									1%	28	-	-	-	-	10.00	-	-	-		
									0.05%	-	-	-	-	-	-	-	-	-		
172	16015103990000047407	Broze Lane	Sealed	10	10	20	1	10	20%	-	-	-	-	-	-	-	-	-	Lack of available data for model validation and ongoing model strengthening.	Ongoing model strengthening.
									10%	-	-	-	-	-	-	-	-	-		
									5%	-	-	-	-	-	-	-	-	-		
									2%	-	-	-	-	-	-	-	-	-		
									1%	-	-	-	-	-	-	-	-	-		
									0.05%	-	-	-	-	-	-	-	-	-		
521	163837962500000000013	Hall Road	Unsealed	10	10	10	0.5	10	20%	-	-	-	-	-	-	-	-	-	Lack of available data for model validation and ongoing model strengthening.	Ongoing model strengthening.
									10%	-	-	-	-	-	-	-	-	-		
									5%	-	-	-	-	-	-	-	-	-		
									2%	-	-	-	-	-	-	-	-	-		
									1%	-	-	-	-	-	-	-	-	-		
									0.05%	-	-	-	-	-	-	-	-	-		
524	16015103990000154478	Hall Road	Unsealed	10	10	10	0.5	10	20%	-	-	-	-	-	-	-	-	-	Maturity of current bridge design is limited by available information (e.g. detailed survey)	Further cross-drainage optimization investigations are required during detailed design.
									10%	-	-	-	-	-	-	-	-	-		
									5%	-	-	-	-	-	-	-	-	-		
									2%	11	-	-	-	-	420.00	-	-	-		
									1%	35	-	-	-	-	2080.00	-	-	-		
									0.05%	-	-	-	-	-	-	-	-	-		
214	16015103990000113526	Fysh Road	Sealed	10	10	20	1	10	20%	-	-	-	-	-	-	-	-	-	Maturity of current road design is limited by available information (e.g. detailed survey)	Further road and or rail design optimization investigations are required during detailed design.
									10%	-	-	-	-	-	-	-	-	-		
									5%	-	-	-	-	-	-	-	-	-		
									2%	-	-	-	-	-	-	-	-	-		
									1%	-	-	-	-	-	-	-	-	-		
									0.05%	32	-	-	-	-	90.00	-	-	-		

7.5.3.4 Flood impacts on existing rail infrastructure

Table 7.42 presents a summary of existing rail infrastructure within the Condamine River hydraulic model extent where potential FIO exceedances due to the Project were identified. The impacted rail infrastructure sections are shown in Figure 7.56.

The Project constraints cause flow modifications along the proposed embankment, resulting in general FIO exceedances to the existing QR rail line (rail section ID #3384). This section of existing QR rail is currently non-operational. The changes determined are in part due to the proposed design providing a higher standard of flood immunity with a significant increase in opening ratio along the embankment than the existing QR line. Stakeholder consultation will be undertaken to inform the mitigation of the impacted section (#3384) which is located outside the Project corridor.

General FIO exceedances also impact the privately owned Yandilla Siding (#4042, #4043, #4044), which connects into the QR line and is also identified as currently non-operational. The flood impacts in this area are driven by increased ponding at the Millmerran-Leyburn Road intersection, which is proposed to be raised as part of the Project. In the Existing Case, this rail is only partially inundated with flood waters. In the design case, due to the presence of the proposed embankment, flood waters are detained upstream of the alignment resulting in the impact. Significant optimisation of the design around this location has been undertaken to date and therefore it is anticipated that there will be residual exceedances on this rail line after design optimisation in the detailed design stage. Stakeholder consultation will be undertaken to inform the mitigation of this impact.

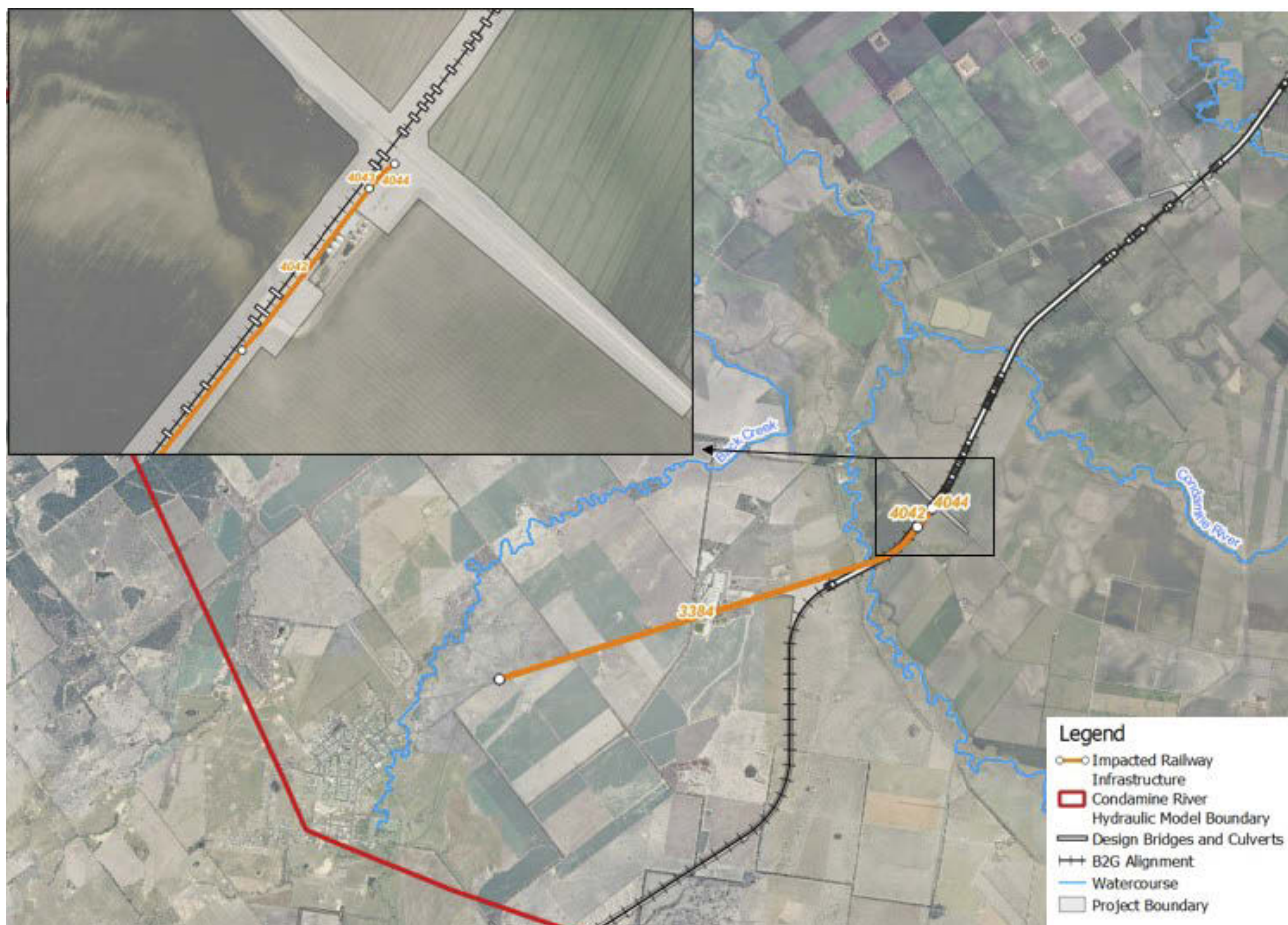


Figure 7.56 Condamine River—Impacted existing rail infrastructure

Table 7.42 Condamine River - Summary of impacts on existing rail infrastructure

ID	Route Name	Section Name	Status	Flood Impact Objective Target				AEP	Maximum FIO Exceedance					Length (m)			Exceedance Justification	Proposed Mitigation Measures ³			
				Change in Peak Water Level (mm)	Change in Time of Inundation (%)	Change in Velocity (%)	Change in Velocity (m/s)		Change in Hazard (%)	Change in Peak Water Level (mm)	Change in Time of Inundation (%) ²	Change in Velocity (m/s)	Change in Velocity (m/s)	Change in Hazard (%) ²	Afflux	Time on Inundation			Hazard		
																			Time on Inundation	Velocity	Hazard
1	3384	Milnerian Branch	Abandoned	10	10	10	0.5	10	20%	-	265	-	-	-	-	-	50.00	-	-	Maturity of current road design is limited by available information (e.g. detailed survey)	Stakeholder Consultation (Landowner/ Government).
									10%	12	72	-	-	-	10.00	30.00	-	-			
									5%	37	70	20	0.11	88	250.00	40.00	20.00	50.00			
									2%	79	47	0.13	120	920.00	40.00	30.00	570.00				
									1%	82	32	0.28	218	1180.00	20.00	70.00	680.00				
									0.05%	-	-	-	-	-	-	-	-				
2	4042	Yandilla Siding	Abandoned	10	10	10	0.5	10	20%	362	-	-	-	-	40.00	-	-	-	Exceedances on this rail segment are generally due to the proximity of the rail directly adjacent to the proposed rail alignment and Milnerian Leyburn Road Level Crossing Upgrade. In the existing case this rail is only partially inundated with flood waters. In the design case, due to the presence of the proposed embankment, flood waters are detained upstream of the alignment resulting in the impact.	Further road and or rail design optimisation investigations will be undertaken in detailed design but it is anticipated residual exceedances will remain. In this case, discussions with the asset owner will be undertaken to identify appropriate site specific mitigation to reduce the impact at this location.	
									10%	501	36	-	-	-	100.00	20.00	-	-			
									5%	663	164	-	-	-	180.00	70.00	-	-			
									2%	143	383	-	0.38	384	460.00	200.00	10.00	50.00			
									1%	212	198	-	0.25	96	560.00	360.00	10.00	50.00			
									0.05%	-	-	-	-	-	-	-	-				
3	4044	Yandilla Siding	Abandoned	10	10	10	0.5	10	20%	-	-	-	-	-	-	-	-	-	Maturity of current road design is limited by available information (e.g. detailed survey)	Further road and or rail design optimisation investigations are required during detailed design / Stakeholder Consultation (Landowner/ Government).	
									10%	-	31	-	0.22	-	-	-	10.00	10.00			-
									5%	82	127	-	-	-	100.00	-	-	-			
									2%	152	127	-	-	-	100.00	20.00	-	-			
									1%	137	44	-	-	-	100.00	20.00	-	-			
									0.05%	-	-	-	-	-	-	-	-				
4	4043	Yandilla Siding	Abandoned	10	10	10	0.5	10	20%	-	-	-	-	-	-	-	-	-	Maturity of current road design is limited by available information (e.g. detailed survey)	Further road and or rail design optimisation investigations are required during detailed design / Stakeholder Consultation (Landowner/ Government).	
									10%	-	-	-	-	-	-	-	-	-			
									5%	82	-	-	-	-	40.00	-	-	-			
									2%	152	127	-	-	-	40.00	20.00	-	-			
									1%	137	44	-	-	-	40.00	20.00	-	-			
									0.05%	-	-	-	-	-	-	-	-	-			

Table Notes

1. Time of Submergence has been measured across the full length of the road segment and does not correspond to the exact location of exceedance within that segment
2. Change in water level based on the maximum water surface case result and does not consider the underlying topography. Depth values presented are based on the respective underlying topographies and in some cases do not align with afflux calculations. This can be attributed to interpolation calculations of results performed within the hydraulic model.
3. Percentage increases should be read in conjunction with existing and design case results. The FfOs require that change be presented in percentage change for time of inundation, velocity and hazard and can sometimes produce large increases when existing case results are significantly small. Some locations were identified to experience significant percentage increases however the 'absolute' difference between Existing and Design results are relatively small
4. **The Flood Impact Objective exceedance has been identified due to a change in flood extent and a cell which was dry in the existing case and now flood-affected in the developed case.**
5. The above table should be read in conjunction with change maps provided as part of Volume II and the Digital Platform.
6. The symbol "<" denotes that no change above Flood Impact Objectives has been determined and therefore no result has been presented.
7. Changes recorded above may only occur during a small fraction of model simulation time.
8. Three decimal places do not infer absolute accuracy in modelling results, but it is necessary to identify small changes.
9. Design optimisations that could be implemented (but not limited to) include adjustment of culvert spacing, realignment of culverts to existing flowpaths when more detailed survey is available, modifying the grade of culverts, road height and alignment adjustments, removal of bridge abutments from waterway extents etc

7.5.3.5 Flood impacts on private land outside the permanent footprint

Table 7.43 presents a summary of private properties located outside the permanent footprint with potential FIO exceedances due to the potential Project rail impacts. A summary of key impacts, exceedance justification and mitigation measures are provided in the below points with further detail provided in Table 7.43:

- Out of a total of 2,556 individual properties assessed on the Condamine River regional floodplain, 103 private lots were flagged as having a potential impact exceeding the Project FIO criteria. The majority of potential exceedances (74) were identified on cropping land (grain and cotton). An additional 26 instances of potential exceedances occurred on residential land.
- Potential flood duration FIO exceedances were identified on 88 lots. These are spread along floodplain fringes and in areas with slow moving floodwater which are generally sensitive to minor disturbances to flood timing or storage modification. Flood duration exceedances may coincide with improvements in flood levels (i.e. reduced afflux) as a result of improved upstream floodwater retention. Future improvement in road and cross-drainage design maturity around Yandilla and west of Pampas is expected to mitigate the flood duration FIO exceedances.
- Potential afflux FIO exceedances were identified on 25 lots. These are concentrated within a narrow corridor upstream of the Project footprint due to topographic changes associated with the proposed alignment which have relatively more impact in rarer events. The afflux impacts are associated with general flood level improvements (i.e. reductions) downstream of the alignment, sometimes resulting from balancing the upstream and downstream impacts between frequent and rare events (e.g. oversizing an opening for a rare event may lead to increased impact in another frequent event). Future improvement in road and cross-drainage design maturity around Yandilla and west of Pampas is expected to mitigate the afflux FIO exceedances.
- Potential velocity FIO exceedances were identified on four lots. These occur in rarer events (i.e. 2% AEP and above) outside main channels or along floodplain fringes in response to small flow redistribution in areas of high topography sensitivity (e.g. where a hydraulic control such as the channel high bank or structure is present). Ongoing model strengthening will help refine the area of velocity impacts and inform the appropriate site-specific flood mitigation as required.
- Potential hazard FIO exceedances were identified on 13 lots. Hazard exceedances are primarily associated with local flow redistributions around structural openings, such as bridge abutments, and storage reconfigurations, such as corners of rail/road intersections. On the Condamine River floodplain, flood hazard exceedances do not generally coincide with afflux or velocity exceedances. Ongoing model strengthening will help refine the area of hazard impacts and inform the appropriate site-specific flood mitigation or awareness management required.

The current assessment indicate that the proposed Project rail design achieves generally compliant FIO outcomes on lands across the expansive Condamine River regional floodplain. Future design optimisation and model strengthening will help mitigate and improve the risk categorisation of the current flood impacts. Relevant stakeholder consultation will be undertaken in support of the development of impact mitigation or management measures as required.

Table 7.43 Condamine River – Summary of flood impacts on private land outside the permanent footprint

ID	ID (Lotplan ID*/FSR ID/ Culvert ID)	Sensitive Receptor Type	Catchment Type	Landuse ¹¹	Lot Area (m2)	Flood Impact Objective Target						AEP	Maximum FIO Exceedance						Area (ha) ¹				Exceedance Justification	Proposed Mitigation Measures ¹⁰	
						Change in Peak Water Level (mm)	Change in Time of Inundation (%)	Change in Time of Inundation (Hours)	Change in Velocity (%)	Change in Velocity (m/s)	Change in Hazard (%)		Change in Peak Water Level (mm)	Change in Time of Inundation (%) ²	Change in Time of Inundation (hours)	Change in Velocity (%)	Change in Velocity (m/s)	Change in Hazard (%)	Afflux	Time on Inundation	Velocity	Hazard			
1	3RP37102	Lot Type Parcel	Regional	GRAINS	1,019,556	100	10	1	10	0.5	20	20%	-	788	-	-	-	-	-	6.62	-	-	Maturity of current road design is limited by available information (e.g. detailed survey)	Further road and or rail design optimization investigations are required during detailed design.	
												10%	-	188	-	-	-	-	-	-	9.64	-			-
												5%	-	56	-	-	-	-	-	-	5.08	-			-
												2%	-	35	-	-	-	-	33	-	1.45	-			0.14
												1%	-	28	-	-	-	61	-	0.94	-	0.98			
												0.05%	-	-	-	-	-	-	-	-	-	-			-
2	2RP61876	Lot Type Parcel	Regional	GRAINS	2,752,451	100	10	1	10	0.5	20	20%	-	205	-	-	-	-	-	2.39	-	-	Maturity of current road design is limited by available information (e.g. detailed survey)	Further road and or rail design optimization investigations are required during detailed design.	
												10%	-	198	-	-	-	-	-	-	3.88	-			-
												5%	-	113	-	-	-	-	-	-	2.55	-			-
												2%	-	107	-	-	-	68	-	2.22	-	0.21			
												1%	-	142	-	-	-	51	-	1.18	-	0.62			
												0.05%	-	-	-	-	-	-	-	-	-	-			-
3	41SP177895	Lot Type Parcel	Regional	GRAINS	3,280,101	100	10	1	10	0.5	20	20%	-	30	-	-	-	-	-	1.35	-	-	Maturity of current bridge design is limited by available information (e.g. detailed survey)	Further cross-drainage optimization investigations are required during detailed design.	
												10%	-	-	-	-	-	-	32	-	-	-			0.04
												5%	-	-	-	-	-	-	24	-	-	-			0.01
												2%	-	-	-	-	-	-	26	-	-	-			0.01
												1%	-	-	-	-	-	40	-	-	-	0.22			
												0.05%	-	-	-	-	-	-	-	-	-	-			-
4	11SP254028	Lot Type Parcel	Regional	GRAINS	2,786,834	100	10	1	10	0.5	20	20%	-	65	-	-	-	-	-	0.36	-	-	Maturity of current bridge design is limited by available information (e.g. detailed survey)	Further cross-drainage optimization investigations are required during detailed design.	
												10%	-	17	-	-	-	-	-	-	0.05	-			-
												5%	-	-	-	-	-	-	-	-	-	-			-
												2%	-	65	-	-	-	-	-	-	0.77	-			-
												1%	-	15	-	-	-	-	-	-	0.07	-			-
												0.05%	-	-	-	-	-	-	-	-	-	-			-
5	4RP53023	Lot Type Parcel	Regional	GRAINS	567,987	100	10	1	10	0.5	20	20%	-	35	-	-	-	-	-	0.41	-	-	Maturity of current road design is limited by available information (e.g. detailed survey)	Further road and or rail design optimization investigations are required during detailed design.	
												10%	-	-	-	-	-	-	-	-	-	-			-
												5%	-	-	-	-	-	-	-	-	-	-			-
												2%	-	-	-	-	-	-	-	-	-	-			-
												1%	-	-	-	-	-	-	-	-	-	-			-
												0.05%	-	-	-	-	-	-	-	-	-	-			-
6	2SP202199	Lot Type Parcel	Regional	GRAINS	942,683	100	10	1	10	0.5	20	20%	-	125	-	-	-	-	-	0.16	-	-	Lack of available data for model validation and ongoing model strengthening.	Ongoing model strengthening.	
												10%	-	75	-	-	-	-	-	-	0.52	-			-
												5%	-	63	-	-	-	-	-	-	0.31	-			-
												2%	-	-	-	-	-	-	-	-	-	-			-
												1%	-	-	-	-	-	-	-	-	-	-			-
												0.05%	-	-	-	-	-	-	-	-	-	-			-
7	3RP35147	Lot Type Parcel	Regional	GRAINS	2,803,306	100	10	1	10	0.5	20	20%	-	34	-	-	-	-	-	0.09	-	-	Maturity of current road design is limited by available information (e.g. detailed survey)	Further road and or rail design optimization investigations are required during detailed design.	
												10%	-	91	-	-	-	-	-	-	1.06	-			-
												5%	-	135	-	-	-	-	-	-	0.50	-			-
												2%	-	140	-	-	-	-	-	-	1.14	-			-
												1%	-	151	-	-	-	-	-	-	2.54	-			-
												0.05%	-	-	-	-	-	-	-	-	-	-			-
8	5RP37102	Lot Type Parcel	Regional	GRAINS	3,203	100	10	1	10	0.5	20	20%	-	25	-	-	-	-	-	0.05	-	-	Maturity of current road design is limited by available information (e.g. detailed survey)	Further road and or rail design optimization investigations are required during detailed design.	
												10%	-	14	-	-	-	-	-	-	0.05	-			-
												5%	-	14	-	-	-	-	-	-	0.05	-			-
												2%	-	12	-	-	-	-	-	-	0.04	-			-
												1%	-	-	-	-	-	-	-	-	-	-			-
												0.05%	-	-	-	-	-	-	-	-	-	-			-
9	3RP53023	Lot Type Parcel	Regional	GRAINS	649,822	100	10	1	10	0.5	20	20%	-	632	-	-	-	-	-	27.98	-	-	Maturity of current road design is limited by available information (e.g. detailed survey)	Further road and or rail design optimization investigations are required during detailed design.	
												10%	-	170	-	-	-	-	34	-	14.73	-			0.42
												5%	-	38	-	-	-	-	56	-	13.41	-			1.00
												2%	-	24	-	-	-	-	34	-	6.48	-			0.66
												1%	-	19	-	-	-	-	31	-	3.92	-			0.24
												0.05%	-	-	-	-	-	-	-	-	-	-			-
10	2RP53023	Lot Type Parcel	Regional	GRAINS	436,576	100	10	1	10	0.5	20	20%	-	65	-	-	-	-	-	6.99	-	-	Maturity of current road design is limited by available information (e.g. detailed survey)	Further road and or rail design optimization investigations are required during detailed design.	
												10%	-	18	-	-	-	-	36	-	0.97	-			0.01
												5%	-	21	-	-	-	-	29	-	1.05	-			0.01
												2%	-	14	-	-	-	-	-	-	0.42	-			-
												1%	-	11	-	-	-	-	-	-	0.12	-			-
												0.05%	-	-	-	-	-	-	-	-	-	-			-
11	1RP83100	Lot Type Parcel	Regional	COTTON	707,156	100	10	1	10	0.5	20	20%	-	88	-	-	-	-	-	0.43	-	-	Maturity of current bridge design is limited by available information (e.g. detailed survey)	Further cross-drainage optimization investigations are required during detailed design.	
												10%	-	-	-	-	-	-	-	-	-	-			-
												5%	-	-	-	-	-	-	-	-	-	-			-
												2%	-	-	-	-	-	-	-	-	-	-			-
												1%	-	-	-	-	-	-	-	-	-	-			-
												0.05%	-	-	-	-	-	-	-	-	-	-			-
12	2AG1803	Lot Type Parcel	Regional	DWELLING - LARGE HOUSESITE	63,172	10	5	1	10	0.5	10	20%	-	40	-	-	-	-	-	0.05	-	-	Maturity of current bridge design is limited by available information (e.g. detailed survey)	Further cross-drainage optimization investigations are required during detailed design.	
												10%	-	-	-	-	-	-	-	-	-	-			-
												5%	-	13	-	-	-	-	-	-	0.05	-			-
												2%	-	-	-	-	-	-	-	-	-	-			-
												1%	-	-	-	-	-	-	26	-	-	-			0.07
												0.05%	-	-	-	-	-	-	-	-	-	-			-
13	1AG1803	Lot Type Parcel	Regional	GRAINS	211,675	100	10	1	10	0.5	20	20%	-	1,113	-	-	-	-	-	0.52	-	-	- Maturity of current bridge design is limited by available information (e.g. detailed survey) - Percentage increases should be read in conjunction with existing and design case results. The FIOs require that change be presented in percentage change for time of inundation, velocity and hazard and can sometimes produce large increases when existing case results are significantly small. Some locations were identified to experience significant percentage increases however the "absolute" difference between Existing and Design results are relatively small.	Further cross-drainage optimization investigations are required during detailed design.	
												10%	-	-	-	-	-	-	-	-	-	-			-
												5%	-	-	-	-	-	-	-	-	-	-			-
												2%	-	-	-	-	-	-	-	-	-	-			-
												1%	-	-	-	-	-	-	-	-	-	-			-
												0.05%	-	-	-	-	-	-	-	-	-	-			-
14	64RP14239	Lot Type Parcel	Regional	GRAINS	414,196	100	10	1	10	0.5	20	20%	-	24	-	-	-	-	-	0.04	-	-	Maturity of current bridge design is limited by available information (e.g. detailed survey)	Further cross-drainage optimization investigations are required during detailed design.	
												10%	-	62	-	-	-	-	-	-	0.15	-			-
												5%	-	-	-	-	-	-	-	-	-	-			-
												2%	-	-	-	-	-	-	-	-	-	-			-
												1%	-	-	-	-	-	-	-	-	-	-			-
												0.05%	-	-	-	-	-	-	-	-	-	-			-

[illegible]

ID	ID (Lotplan ID ⁹ /FSR ID/ Culvert ID)	Sensitive Receptor Type	Catchment Type	Landuse ¹¹	Lot Area (m2)	Flood Impact Objective Target						AEP	Maximum FIO Exceedance						Area (ha) ⁷				Exceedance Justification	Proposed Mitigation Measures ¹⁰			
						Change in Peak Water Level (mm)	Change in Time of Inundation (%)	Change in Time of Inundation (Hours)	Change in Velocity (%)	Change in Velocity (m/s)	Change in Hazard (%)		Change in Peak Water Level (mm)	Change in Time of Inundation (%) ²	Change in Time of Inundation (hours)	Change in Velocity (%) ³	Change in Velocity (m/s)	Change in Hazard (%) ⁴	Afflux	Time on Inundation	Velocity	Hazard					
30	14SP254028	Lot Type Parcel	Regional	GRAINS	1,801,784	100	10	1	10	0.5	20	20%	-	39	-	-	-	-	-	0.31	-	-	Maturity of current bridge design is limited by available information (e.g. detailed survey)	Further cross-drainage optimization investigations are required during detailed design.			
												10%	-	-	-	-	-	-	-	-	-	-			-	-	-
												5%	-	-	-	-	-	-	-	-	-	-			-	-	-
												2%	-	-	-	-	-	-	-	-	-	-			-	-	-
												1%	-	20	-	-	-	-	-	0.06	-	-			-	-	-
												0.05%	-	-	-	-	-	-	-	-	-	-			-	-	-
31	2343A341070	Lot Type Parcel	Regional	GRAINS	326,124	100	10	1	10	0.5	20	20%	-	40	-	-	-	-	-	0.13	-	-	Maturity of current bridge design is limited by available information (e.g. detailed survey)	Further cross-drainage optimization investigations are required during detailed design.			
												10%	-	67	-	-	-	-	-	-	-	-			-	-	-
												5%	-	-	-	-	-	-	-	-	-	-			-	-	-
												2%	-	-	-	-	-	-	-	-	-	-			-	-	-
												1%	-	-	-	-	-	-	-	-	-	-			-	-	-
												0.05%	-	-	-	-	-	-	-	-	-	-			-	-	-
32	2RP37134	Lot Type Parcel	Regional	GRAINS	190,156	100	10	1	10	0.5	20	20%	-	22	-	-	-	-	-	0.13	-	-	Maturity of current bridge design is limited by available information (e.g. detailed survey)	Further cross-drainage optimization investigations are required during detailed design.			
												10%	-	132	-	-	-	-	-	-	-	-			-	-	-
												5%	-	-	-	-	-	-	-	-	-	-			-	-	-
												2%	-	-	-	-	-	-	-	-	-	-			-	-	-
												1%	-	-	-	-	-	-	-	-	-	-			-	-	-
												0.05%	-	-	-	-	-	-	-	-	-	-			-	-	-
33	115A341191	Lot Type Parcel	Regional	COTTON	202,316	100	10	1	10	0.5	20	20%	-	179	-	-	-	-	-	0.24	-	-	Maturity of current bridge design is limited by available information (e.g. detailed survey)	Further cross-drainage optimization investigations are required during detailed design.			
												10%	-	125	4.15	-	-	-	-	-	0.67	-			-	-	-
												5%	-	14	-	-	-	-	-	-	0.05	-			-	-	-
												2%	-	-	-	-	-	-	-	-	-	-			-	-	-
												1%	-	-	-	-	-	-	-	-	-	-			-	-	-
												0.05%	-	-	-	-	-	-	-	-	-	-			-	-	-
34	2342A341070	Lot Type Parcel	Regional	COTTON	335,370	100	10	1	10	0.5	20	20%	-	53	-	-	-	-	-	0.15	-	-	Maturity of current bridge design is limited by available information (e.g. detailed survey)	Further cross-drainage optimization investigations are required during detailed design.			
												10%	-	202	-	-	-	-	-	-	1.19	-			-	-	-
												5%	-	-	-	-	-	-	-	-	-	-			-	-	-
												2%	-	-	-	-	-	-	-	-	-	-			-	-	-
												1%	-	-	-	-	-	-	-	-	-	-			-	-	-
												0.05%	-	-	-	-	-	-	-	-	-	-			-	-	-
35	9RP37106	Lot Type Parcel	Regional	GRAINS	372,665	100	10	1	10	0.5	20	20%	-	51	-	-	-	-	-	1.35	-	-	Maturity of current bridge design is limited by available information (e.g. detailed survey)	Further cross-drainage optimization investigations are required during detailed design.			
												10%	-	-	-	-	-	-	-	-	-	-			-	-	-
												5%	-	-	-	-	-	-	-	-	-	-			-	-	-
												2%	-	-	-	-	-	-	-	-	-	-			-	-	-
												1%	-	-	-	-	-	-	-	-	-	-			-	-	-
												0.05%	-	-	-	-	-	-	-	-	-	-			-	-	-
36	3RP803554	Lot Type Parcel	Regional	GRAINS	368,794	100	10	1	10	0.5	20	20%	-	89	-	-	-	-	-	0.25	-	-	Maturity of current bridge design is limited by available information (e.g. detailed survey)	Further cross-drainage optimization investigations are required during detailed design.			
												10%	-	-	-	-	-	-	-	-	-	-			-	-	-
												5%	-	-	-	-	-	-	-	-	-	-			-	-	-
												2%	-	-	-	-	-	36	-	-	0.01	-			-	-	-
												1%	-	-	16	0.04	44	-	-	0.02	0.03	-			-	-	-
												0.05%	-	-	-	-	-	-	-	-	-	-			-	-	-
37	1AP13223	Lot Type Parcel	Regional	GRAINS	66,323	100	10	1	10	0.5	20	20%	-	57	-	-	-	-	-	0.04	-	-	Maturity of current bridge design is limited by available information (e.g. detailed survey)	Further cross-drainage optimization investigations are required during detailed design.			
												10%	-	145	-	-	-	-	-	-	0.17	-			-	-	-
												5%	-	-	-	-	-	-	-	-	-	-			-	-	-
												2%	-	-	-	-	-	-	-	-	-	-			-	-	-
												1%	-	-	-	-	-	-	-	-	-	-			-	-	-
												0.05%	-	-	-	-	-	-	-	-	-	-			-	-	-
38	3AG1845	Lot Type Parcel	Regional	GRAINS	503,349	100	10	1	10	0.5	20	20%	-	282	-	-	-	-	-	0.34	-	-	Maturity of current bridge design is limited by available information (e.g. detailed survey)	Further cross-drainage optimization investigations are required during detailed design.			
												10%	-	117	-	-	-	-	-	-	1.01	-			-	-	-
												5%	-	-	-	-	-	-	-	-	-	-			-	-	-
												2%	-	21	-	-	-	-	-	-	0.06	-			-	-	-
												1%	-	-	-	-	-	-	-	-	-	-			-	-	-
												0.05%	-	-	-	-	-	-	-	-	-	-			-	-	-
39	1RP14242	Lot Type Parcel	Regional	GRAINS	397,577	100	10	1	10	0.5	20	20%	-	262	-	-	-	-	-	1.73	-	-	Maturity of current bridge design is limited by available information (e.g. detailed survey)	Further cross-drainage optimization investigations are required during detailed design.			
												10%	-	234	-	-	-	-	-	-	4.36	-			-	-	-
												5%	-	59	-	-	-	-	-	-	2.16	-			-	-	-
												2%	-	-	-	-	-	-	-	-	-	-			-	-	-
												1%	-	-	-	-	-	-	-	-	-	-			-	-	-
												0.05%	-	-	-	-	-	-	-	-	-	-			-	-	-
40	2AP13242	Lot Type Parcel	Regional	GRAINS	41,354	100	10	1	10	0.5	20	20%	-	29	-	-	-	-	-	0.04	-	-	Maturity of current bridge design is limited by available information (e.g. detailed survey)	Further cross-drainage optimization investigations are required during detailed design.			
												10%	-	-	-	-	-	-	-	-	-	-			-	-	-
												5%	-	-	-	-	-	-	-	-	-	-			-	-	-
												2%	-	-	-	-	-	-	-	-	-	-			-	-	-
												1%	-	-	-	-	-	-	-	-	-	-			-	-	-
												0.05%	-	-	-	-	-	-	-	-	-	-			-	-	-
41	2353A341070	Lot Type Parcel	Regional	GRAINS	447,750	100	10	1	10	0.5	20	20%	-	75	-	-	-	-	-	0.68	-	-	Maturity of current bridge design is limited by available information (e.g. detailed survey)	Further cross-drainage optimization investigations are required during detailed design.			
												10%	-	151	-	-	-	-	-	-	1.00	-			-	-	-
												5%	-	33	-	-	-	-	-	-	0.09	-			-	-	-
												2%	-	-	-	-	-	-	-	-	-	-			-	-	-
												1%	-	-	-	-	-	-	-	-	-	-			-	-	-
												0.05%	-	-	-	-	-	-	-	-	-	-			-	-	-
42	126A341070	Lot Type Parcel	Regional	GRAINS	207,805	100	10	1	10	0.5	20	20%	-	43	-	-	-	-	-	0.11	-	-	Maturity of current bridge design is limited by available information (e.g. detailed survey)	Further cross-drainage optimization investigations are required during detailed design.			
												10%	-	110	-	-	-	-	-	-	0.85	-			-	-	-
												5%	-	-	-	-	-	-	-	-	0.04	-			-	-	-
												2%	-	-	-	-	-	-	-	-	-	-			-	-	-
												1%	-	-	-	-	-	-	-	-	-	-			-	-	-
												0.05%	-	-	-	-	-	-	-	-	-	-			-	-	-
43	7RP835792	Lot Type Parcel	Regional	COTTON	840,024	100	10	1	10	0.5	20	20%	-	67	-	-	-	-	-	0.13	-	-	Lack of available data for model validation and ongoing model strengthening.	Ongoing model strengthening.			
												10%	-	28	-	-	-	-	-	-	0.21	-			-	-	-
												5%	-	34	-	-	-	-	-	-	0.51	-			-	-	-
												2%	-	-	-	-	-	-	-	-	-	-			-	-	-
												1%	-	-	-	-	-	-	-	-	-	-			-	-	-
												0.05%	-	-	-	-	-	-	-	-	-	-			-	-	-
44	3RP37103	Lot Type Parcel	Regional	GRAINS	1,359,583	100	10	1	10	0.5	20	20%	-	-	-	-	32	-	-	-	0.05	-	Maturity of current bridge design is limited by available information (e.g. detailed survey)	Further cross-drainage optimization investigations are required during detailed design.			
												10%	-	-	-	-	-	-	-	30	-	-			-	0.11	-
												5%	-	-	-	-	-	-	-	28	-	-			-	0.05	-
												2%	-	-	-	-	-	-	-	31	-	-			-	0.12	-
												1%	-	-	-	-	-	-	-	50	-	-			-	0.33	-
												0.05%	-	-	-	-	-	-	-	-	-	-			-	-	-

ID	ID (Lotplan ID ⁹ /FSR ID/ Culvert ID)	Sensitive Receptor Type	Catchment Type	Landuse ¹¹	Lot Area (m2)	Flood Impact Objective Target						AEP	Maximum FIO Exceedance						Area (ha) ⁷				Exceedance Justification	Proposed Mitigation Measures ¹⁰
						Change in Peak Water Level (mm)	Change in Time of Inundation (%)	Change in Time of Inundation (Hours)	Change in Velocity (%)	Change in Velocity (m/s)	Change in Hazard (%)		Change in Peak Water Level (mm)	Change in Time of Inundation (%) ²	Change in Time of Inundation (hours)	Change in Velocity (%) ³	Change in Velocity (m/s)	Change in Hazard (%) ⁴	Afflux	Time on Inundation	Velocity	Hazard		
45	3RP55115	Lot Type Parcel	Regional	GRAINS	2,053,655	100	10	1	10	0.5	20	20%	-	-	-	-	-	-	-	-	-	-	Maturity of current bridge design is limited by available information (e.g. detailed survey)	Further cross-drainage optimization investigations are required during detailed design.
												10%	-	83	1.62	-	-	-	-	0.77	-	-		
												5%	-	-	-	-	-	-	-	-	-	-		
												2%	-	-	-	-	-	-	-	-	-	-		
												1%	-	-	-	-	-	-	-	-	-	-		
												0.05%	-	-	-	-	-	-	-	-	-	-		
46	2RP53442	Lot Type Parcel	Regional	COTTON	2,024,816	100	10	1	10	0.5	20	20%	-	-	-	-	-	-	-	-	-	-	Maturity of current bridge design is limited by available information (e.g. detailed survey)	Further cross-drainage optimization investigations are required during detailed design.
												10%	-	101	1.21	-	-	-	-	3.82	-	-		
												5%	-	-	-	-	-	-	-	-	-	-		
												2%	-	-	-	-	-	-	-	-	-	-		
												1%	-	-	-	-	-	-	-	-	-	-		
												0.05%	-	-	-	-	-	-	-	-	-	-		
47	55RP14239	Lot Type Parcel	Regional	COTTON	875,570	100	10	1	10	0.5	20	20%	-	-	-	-	-	-	-	-	-	-	Maturity of current bridge design is limited by available information (e.g. detailed survey)	Further cross-drainage optimization investigations are required during detailed design.
												10%	-	51	-	-	-	-	-	0.05	-	-		
												5%	-	-	-	-	-	-	-	-	-	-		
												2%	-	-	-	-	-	-	-	-	-	-		
												1%	-	208	-	-	-	-	-	0.25	-	-		
												0.05%	-	-	-	-	-	-	-	-	-	-		
48	87AG895	Lot Type Parcel	Regional	DWELLING - LARGE HOUSESITE	107,141	10	5	1	10	0.5	10	20%	-	-	-	-	-	-	-	-	-	-	Maturity of current bridge design is limited by available information (e.g. detailed survey)	Further cross-drainage optimization investigations are required during detailed design.
												10%	-	21	-	-	-	-	-	0.05	-	-		
												5%	-	-	-	-	-	-	-	-	-	-		
												2%	-	-	-	-	-	-	-	-	-	-		
												1%	-	-	-	-	-	-	-	-	-	-		
												0.05%	-	-	-	-	-	-	-	-	-	-		
49	2612A341191	Lot Type Parcel	Regional	GRAINS	202,060	100	10	1	10	0.5	20	20%	-	-	-	-	-	-	-	-	-	-	Maturity of current bridge design is limited by available information (e.g. detailed survey)	Further cross-drainage optimization investigations are required during detailed design.
												10%	-	76	-	-	-	-	-	0.34	-	-		
												5%	-	12	-	-	-	-	-	0.04	-	-		
												2%	-	-	-	-	-	-	-	-	-	-		
												1%	-	-	-	-	-	-	-	-	-	-		
												0.05%	-	-	-	-	-	-	-	-	-	-		
50	116A341191	Lot Type Parcel	Regional	GRAINS	202,460	100	10	1	10	0.5	20	20%	-	-	-	-	-	-	-	-	-	-	Maturity of current bridge design is limited by available information (e.g. detailed survey)	Further cross-drainage optimization investigations are required during detailed design.
												10%	-	82	-	-	-	-	-	0.24	-	-		
												5%	-	19	-	-	-	-	-	0.04	-	-		
												2%	-	-	-	-	-	-	-	-	-	-		
												1%	-	-	-	-	-	-	-	-	-	-		
												0.05%	-	-	-	-	-	-	-	-	-	-		
51	6RP37103	Lot Type Parcel	Regional	GRAINS	160,581	100	10	1	10	0.5	20	20%	-	-	-	-	-	-	-	-	-	-	Maturity of current road design is limited by available information (e.g. detailed survey)	Further road and or rail design optimization investigations are required during detailed design.
												10%	-	-	-	-	-	37	-	-	0.10	-		
												5%	-	-	-	-	-	32	-	-	0.07	-		
												2%	-	-	-	-	-	34	-	-	0.08	-		
												1%	-	-	-	-	-	55	-	-	0.15	-		
												0.05%	-	-	-	-	-	-	-	-	-	-		
52	202SP124721	Lot Type Parcel	Regional	WAREHOUSE & BULK STORES	7,995	10	10	1	20	0.5	10	20%	-	-	-	-	-	-	-	-	-	-	Exceedances on this Lot are generally due to the proximity of the Lot directly adjacent to the proposed rail alignment and Milmeran Leyburn Road Level Crossing Upgrade. In the existing case this Lot is only partially inundated with flood waters. In the design case, due to the presence of the proposed embankment, flood waters are detained upstream of the alignment resulting in the impact.	Further road and or rail design optimisation investigations will be undertaken in detailed design but it is anticipated residual exceedances will remain. In this case, discussions with the Landowner will be undertaken to identify appropriate site specific mitigation to reduce the impact at this location.
												10%	-	-	-	-	-	-	-	-	-	-		
												5%	49	-	-	-	-	-	0.29	-	-	-		
												2%	110	165	-	-	-	-	0.60	0.28	-	-		
												1%	157	1,282	-	-	-	-	0.82	0.33	-	-		
												0.05%	-	-	-	-	-	-	-	-	-	-		
53	4RP55115	Lot Type Parcel	Regional	GRAINS	2,043,659	100	10	1	10	0.5	20	20%	-	-	-	-	-	-	-	-	-	-	Maturity of current cross-drainage design is limited by available information (e.g. detailed survey)	Further cross-drainage optimization investigations are required during detailed design.
												10%	-	-	-	-	-	-	-	-	-	-		
												5%	-	29	-	-	-	-	-	0.05	-	-		
												2%	-	-	-	33	-	-	-	0.11	-	-		
												1%	-	-	-	59	-	-	-	0.21	-	-		
												0.05%	-	-	-	-	-	-	-	-	-	-		
54	12RP37111	Lot Type Parcel	Regional	GRAINS	1,013	100	10	1	10	0.5	20	20%	-	-	-	-	-	-	-	-	-	-	Maturity of current road design is limited by available information (e.g. detailed survey)	Further road and or rail design optimization investigations are required during detailed design.
												10%	-	-	-	-	-	-	-	-	-	-		
												5%	-	73	-	-	-	-	-	0.05	-	-		
												2%	-	-	-	-	-	-	-	-	-	-		
												1%	-	-	-	-	-	-	-	-	-	-		
												0.05%	-	-	-	-	-	-	-	-	-	-		
55	76RP37111	Lot Type Parcel	Regional	DWELLING - LARGE HOUSESITE	1,013	10	5	1	10	0.5	10	20%	-	-	-	-	-	-	-	-	-	-	Maturity of current road design is limited by available information (e.g. detailed survey)	Further road and or rail design optimization investigations are required during detailed design.
												10%	-	-	-	-	-	-	-	-	-	-		
												5%	-	31	-	-	-	-	-	0.08	-	-		
												2%	12	-	-	-	-	-	0.10	-	-	-		
												1%	27	-	-	-	-	-	0.10	-	-	-		
												0.05%	-	-	-	-	-	-	-	-	-	-		
56	75RP37111	Lot Type Parcel	Regional	DWELLING - LARGE HOUSESITE	1,013	10	5	1	10	0.5	10	20%	-	-	-	-	-	-	-	-	-	-	Maturity of current road design is limited by available information (e.g. detailed survey)	Further road and or rail design optimization investigations are required during detailed design.
												10%	-	-	-	-	-	-	-	-	-	-		
												5%	-	20	-	-	-	-	-	0.07	-	-		
												2%	12	-	-	-	-	-	0.09	-	-	-		
												1%	27	-	-	-	-	-	0.10	-	-	-		
												0.05%	-	-	-	-	-	-	-	-	-	-		
57	102AG2442	Lot Type Parcel	Regional	VACANT - LARGE HOUSESITE	24,178	10	5	1	10	0.5	10	20%	-	-	-	-	-	-	-	-	-	-	Maturity of current bridge design is limited by available information (e.g. detailed survey)	Further cross-drainage optimization investigations are required during detailed design.
												10%	-	-	-	-	-	-	-	-	-	-		
												5%	-	56	-	-	-	-	-	0.04	-	-		
												2%	-	-	-	-	-	-	-	-	-	-		
												1%	-	-	-	-	-	-	-	-	-	-		
												0.05%	-	-	-	-	-	-	-	-	-	-		

7.5.3.6 Flow distribution

A key landowner concern is the potential change in flow distribution. To determine the change in flow distribution maximum velocity-depth modelling results were used to produce change in unit flow maps between Existing and Developed Case conditions. Unit flow is the measure of discharge (velocity-depth product) and is often used for assessing peak flow along a cross-section, or to determine the effective flow corridors within a floodplain.

The presented flow distribution maps were filtered with a velocity-depth product cut-off of 0.15 m²/s. The cut-off is in line with FIO requirements and was used to assist in identifying main channels (high flow zones) from shallow lower flood zones (i.e. outer extents of the flood surface).

The following points summarise the potential change in flow distribution for the Condamine River catchment and should be considered in conjunction with flow distribution figures supplied in Figure Q3-1 to Figure Q3-8 in Volume 2 – Appendix Q.

Regionally, the Project produces generally small to negligible flow changes (i.e. within ±5 per cent) along all key flow paths, including Condamine River, Grasstree Creek, and North Branch, across the Condamine River regional floodplain under all events assessed (20%, 10%, 5%, 2%, 1% AEP events and 1 in 2,000 AEP, 1 in 10,000 AEP and PMF events).

Locally, the flow distribution is governed by the relative flood interactions with various hydraulic controls, including the rail embankment, cross-drainage, road levels, and floodplain storage. Local flow redistributions primarily occur along the Project rail corridor, with a list of key sensitivities summarised below:

- Local flow redistributions were identified around all design bridge abutments under all modelled events. The bridge abutment induces a local flow concentration along the bridge opening edge, which corresponds to a flow reduction along the approach on the same side. Flow redistributions of this type are typically localised around the abutments and have small to negligible impact on the overall channel conveyance.
 - Areas surrounding the Gore Highway—Millmerran-Leyburn Road intersection would experience flow redistribution exceeding 50 per cent in some localised locations. These impacts are related to modifications of floodplain storage by the raised Millmerran-Leyburn Road intersection. It is expected that further improvement in cross-drainage design maturity during detailed design will mitigate the current impact at this location in frequent and infrequent events (e.g. below 1% AEP event). The flow redistribution at this location is only marginally influenced by storage modification in very rare and extreme events (i.e. above 1 in 2,000 AEP event), where the overtopping depth along the raised Millmerran-Leyburn Road would significantly exceed the raise above the existing road. The flow redistribution is then governed by the flow concentration around the Grasstree Creek Bridge and Condamine River South Branch Bridge abutments, essentially becoming independent of the road drainage provision.
- Flow reductions were identified along the left floodplain of North Branch downstream of Pampas, which are in part related to small flow increases under the North Branch bridge. Further model strengthening, including refining the model representation of local roads in this area based on detailed survey, and improvement to drainage maturity, including increasing the opening length between Pampas and North Branch, are expected to mitigate the impact.

In frequent events (e.g. 10% AEP event), moderate flow redistributions (i.e. within ±20 per cent) were notable on fringes along the North Branch downstream of the permanent footprint. These areas are characterised by slow-moving inundation which are sensitive to small changes in hydrograph timing, despite the overall conveyance (i.e. peak flow) being preserved. It is expected that further improvement to the cross-drainage design maturity during detailed design around Pampas will improve the current flow distribution sensitivities along North Branch fringe downstream of the permanent footprint.

Ongoing mitigation measures to reduce changes in flow distribution are likely to occur during future design stages of the Project and may include further refinement of the bridge abutments and additional survey where necessary.

7.5.4 Extreme events

7.5.4.1 Extreme event immunity

The risk of overtopping of the Project alignment was assessed for a range of extreme events (1 in 2,000 AEP, 1 in 10,000 AEP and PMF). Table 7.44 presents the estimated depth of water above the formation level and over the top of rail at each structure. It is noted that the function of the floodplain culverts is to balance flood levels on the upstream and downstream sides of the Project alignment.

As such, overtopping of the Project rail is not predicted to result in significant excessive flows or velocities as would occur in a dam embankment overtopping scenario. Further analysis of the overtopping risk to FSRs can be found in Section 21.

Table 7.44 Condamine River—Extreme events: Estimated depth of water above formation and top of rail levels

Chainage (km)	Structure ID	Depth of water above formation level (m)			Depth of water above top of rail (m)		
		1 in 2,000 AEP	1 in 10,000 AEP	PMF	1 in 2,000 AEP	1 in 10,000 AEP	PMF
139.13	C139.13	-	0.28	1.46	-	-	0.76
139.17	C139.17	-	0.24	1.33	-	-	0.63
139.21	C139.21	-	0.20	1.29	-	-	0.59
139.5	310-BR21/22	-	0.20	1.56	-	-	0.85
140.66	C140.66	-	0.13	1.34	-	-	0.64
140.73	C140.73	-	0.14	1.30	-	-	0.60
140.79	C140.79	-	0.16	1.31	-	-	0.61
140.85	C140.85	-	0.15	1.30	-	-	0.60
141.00	C141.00	-	0.14	1.24	-	-	0.54
141.02	C141.02	-	-	1.23	-	-	0.53
141.07	C141.07	-	0.17	1.26	-	-	0.56
141.38	C141.38	-	0.15	1.23	-	-	0.53
141.40	C141.40	-	0.10	1.19	-	-	0.49
141.46	C141.46	-	0.08	1.14	-	-	0.44
141.49	C141.49	-	0.10	1.18	-	-	0.48
141.52	C141.52	-	-	1.16	-	-	0.46
141.54	C141.54	-	0.10	1.20	-	-	0.50
141.56	C141.56	-	-	1.16	-	-	0.46
141.61	C141.61	-	-	1.17	-	-	0.47
141.67	C141.67	-	0.12	1.20	-	-	0.50
141.69	C141.69	-	0.11	1.19	-	-	0.49
141.72	C141.72	-	0.06	1.14	-	-	0.44
141.75	C141.75	-	-	1.13	-	-	0.43
141.78	C141.78	-	0.11	1.24	-	-	0.54
141.80	C141.80	-	0.08	1.15	-	-	0.45
141.84	C141.84	-	0.10	1.17	-	-	0.47
141.88	C141.88	-	0.11	1.22	-	-	0.52
141.93	C141.93	-	0.10	1.19	-	-	0.49
141.96	C141.96	-	0.10	1.16	-	-	0.46

Chainage (km)	Structure ID	Depth of water above formation level (m)			Depth of water above top of rail (m)		
		1 in 2,000 AEP	1 in 10,000 AEP	PMF	1 in 2,000 AEP	1 in 10,000 AEP	PMF
142.07	C142.07	-	-	1.07	-	-	0.37
142.12	C142.12	-	0.04	1.05	-	-	0.35
142.16	C142.16	-	0.02	1.03	-	-	0.33
142.20	C142.20	-	0.03	1.02	-	-	0.32
142.27	C142.27	-	0.05	1.11	-	-	0.41
142.32	C142.32	-	-	0.96	-	-	0.26
142.36	C142.36	-	-	0.98	-	-	0.28
142.40	C142.40	-	0.01	1.12	-	-	0.42
142.49	C142.49	-	-	1.09	-	-	0.39
142.53	C142.53	-	-	1.09	-	-	0.39
142.58	C142.58	-	-	1.09	-	-	0.39
142.61	C142.61	-	-	0.99	-	-	0.29
142.65	310-BR24	-	-	1.14	-	-	0.44
143.29	C143.29	-	-	0.86	-	-	0.16
143.33	C143.33	-	-	0.95	-	-	0.25
143.37	C143.37	-	-	0.87	-	-	0.17
143.42	C143.42	-	-	0.79	-	-	0.09
143.44	C143.44	-	-	0.90	-	-	0.20
143.48	C143.48	-	-	0.91	-	-	0.21
143.51	C143.51	-	-	0.89	-	-	0.19
143.54	C143.54	-	-	0.92	-	-	0.22
143.57	C143.57	-	-	0.78	-	-	0.08
143.65	C143.65	-	-	0.77	-	-	0.07
143.70	C143.70	-	-	0.87	-	-	0.17
143.73	C143.73	-	-	0.76	-	-	0.06
143.77	C143.77	-	-	0.87	-	-	0.17
143.79	C143.79	-	-	0.88	-	-	0.18
143.83	C143.83	-	-	0.79	-	-	0.09
143.87	C143.87	-	-	0.77	-	-	0.07
145.5	310-BR25/26	-	-	-	-	-	-
146.45	C146.45	-	-	0.83	-	-	0.13
146.50	C146.50	-	-	0.65	-	-	-
146.54	C146.54	-	-	0.87	-	-	0.17
146.61	C146.61	-	0.05	0.80	-	-	0.10
146.69	C146.69	-	-	0.65	-	-	-
147.01	C147.01	-	0.19	0.76	-	-	0.06
147.12	C147.12	-	0.25	0.84	-	-	0.14
147.18	C147.18	-	0.26	0.87	-	-	0.17
147.21	C147.21	-	0.28	0.89	-	-	0.19
147.27	C147.27	-	0.23	0.78	-	-	0.08

Chainage (km)	Structure ID	Depth of water above formation level (m)			Depth of water above top of rail (m)		
		1 in 2,000 AEP	1 in 10,000 AEP	PMF	1 in 2,000 AEP	1 in 10,000 AEP	PMF
147.31	C147.31	-	0.26	0.82	-	-	0.12
147.85	C147.85	-	0.44	1.10	-	-	0.40
147.91	C147.91	-	0.52	1.18	-	-	0.48
148.87	C148.87	-	-	0.44	-	-	-
148.92	C148.92	-	-	0.29	-	-	-
148.96	C148.96	-	-	0.15	-	-	-
149.02	C149.02	-	-	0.12	-	-	-
149.6	310-BR27	-	-	-	-	-	-
150.68	C150.68	-	-	1.89	-	-	1.19
150.71	C150.71	-	-	1.95	-	-	1.25
150.74	C150.74	-	-	1.98	-	-	1.28
151.08	C151.08	-	-	2.06	-	-	1.36
151.12	C151.12	-	-	2.09	-	-	1.39
151.14	C151.14	-	-	2.08	-	-	1.38
151.17	C151.17	0.67	1.25	2.10	-	0.55	1.40
151.20	C151.20	-	-	2.02	-	-	1.32
151.27	C151.27	-	-	2.04	-	-	1.34
151.30	C151.30	-	-	-	-	-	-

Table note:

1 Assuming top of rail is 700 mm above formation level

7.5.4.2 Impacts during extreme events

Flood immunity of FSRs is discussed in Section 7.5.3.1, and maps demonstrating the impacts during extreme events are shown on the [Online Digital Portal](#).

In accordance with the FIOs, FSRs are considered impacted by extreme flood events if the change in water level is greater than 250 mm in the 1 in 2,000 AEP event, subject to the receptor classification (i.e. at existing habitable dwellings and/or commercial and industrial buildings/premises). In addition, a stricter afflux objective of 10 mm in the 1 in 500 AEP event applies to sites with existing sensitive infrastructure, uses involving vulnerable people and/or any critical road network that was designed to be immune to flooding in any extreme event.

Review of the 1 in 2,000 AEP event results for the Condamine River floodplain, against the FIOs show that flood level changes at all FSRs were limited to 250 mm, and no identified existing sensitive infrastructure or uses involving vulnerable people are expected to experience a more than 10 mm increase in flood level. A separate assessment of the 1 in 500 AEP event is not required as they are conservatively represented by the 1 in 2,000 AEP event results on the Condamine River floodplain. The flood impacts in extreme events due to the Project are therefore compliant against the stipulated FIO limits for extreme events.

Table 7.45 details extreme event results, although it should be noted that FIO targets do not apply to the 1 in 10,000 AEP and PMF events.

Table 7.45 **Condamine River—Extreme events: Estimated impacts at flood sensitive receptors**

FSR ID	FSR type	1 in 2,000 AEP event		1 in 10,000 AEP event		PMF event	
		Change in peak water level (mm)	Existing Case flood depth (m)	Change in peak water level (mm)	Existing Case flood depth (m)	Change in peak water level (mm)	Existing Case flood depth (m)
CON_ID_4	Silos	+205	0.97	+301	1.92	+66	3.31
CON_ID_5	Silos	+213	0.90	+304	1.85	+65	3.24
CON_ID_6	Silos	+210	0.79	+302	1.75	+65	3.14
CON_ID_7	Silos	+217	0.93	+307	1.89	+65	3.28
CON_ID_8	Silos	+205	1.20	+302	2.15	+66	3.53
CON_ID_9	Silos	+206	1.06	+302	2.01	+66	3.40
CON_ID_10	Silos	+203	1.04	+300	1.99	+66	3.38
CON_ID_63	Shed	+4	1.33	+6	2.17	+4	3.45
CON_ID_64	Shed	+4	1.53	+6	2.37	+4	3.66
CON_ID_65	Shed	+4	1.38	+6	2.22	+4	3.50
CON_ID_66	House	+4	1.31	+6	2.15	+4	3.43
CON_ID_67	Shed	+4	1.33	+6	2.17	+4	3.45
CON_ID_68	Shed	+81	1.35	+163	2.31	+89	3.71
CON_ID_69	Shed	+2	0.75	+3	1.58	+3	2.86
CON_ID_70	House	+2	0.96	+3	1.78	+3	3.07
CON_ID_71	House	+2	0.83	+3	1.66	+3	2.95
CON_ID_72	Shed	+1	1.24	+2	2.05	+3	3.32
CON_ID_73	Shed	+2	1.06	+3	1.87	+3	3.13
CON_ID_74	Shed	-3	1.99	-7	2.89	-	4.20
CON_ID_75	House	-3	1.92	-7	2.82	+1	4.13
CON_ID_76	Shed	-3	1.88	-7	2.77	+1	4.09
CON_ID_77	Shed	-3	1.97	-7	2.87	+1	4.18
CON_ID_78	House	+98	1.25	+188	2.21	+94	3.61
CON_ID_79	House	+19	1.20	+33	2.04	+17	3.34
CON_ID_80	Shed	+21	1.05	+37	1.89	+18	3.19
CON_ID_81	Shed	+22	0.84	+39	1.66	+18	2.96
CON_ID_82	Shed	+21	1.30	+37	2.13	+18	3.44
CON_ID_83	Shed	+1	0.84	+2	1.65	+3	2.93
CON_ID_84	Shed	+1	0.94	+2	1.73	+3	3.02
CON_ID_85	House	+1	0.72	+2	1.52	+3	2.81
CON_ID_86	Shed	+1	0.93	+2	1.73	+3	3.02
CON_ID_89	Shed	-28	0.32	-17	1.34	+4	2.75
CON_ID_90	House	-29	0.52	-18	1.54	+4	2.94
CON_ID_91	Shed	-30	0.74	-18	1.76	+4	3.17
CON_ID_92	House	-31	0.54	-19	1.55	+4	2.96
CON_ID_93	Shed	-28	0.49	-17	1.52	+4	2.92
CON_ID_94	Shed	-29	0.39	-17	1.41	+4	2.82
CON_ID_95	Shed	-29	0.40	-17	1.42	+4	2.83

FSR ID	FSR type	1 in 2,000 AEP event		1 in 10,000 AEP event		PMF event	
		Change in peak water level (mm)	Existing Case flood depth (m)	Change in peak water level (mm)	Existing Case flood depth (m)	Change in peak water level (mm)	Existing Case flood depth (m)
CON_ID_96	Shed	+49	0.41	+115	1.37	+52	2.80
CON_ID_97	Shed	+48	0.71	+114	1.66	+53	3.10
CON_ID_98	Shed	+48	0.50	+114	1.46	+53	2.89
CON_ID_99	House	+65	0.77	+127	1.76	+55	3.20
CON_ID_100	House	+64	0.61	+126	1.60	+55	3.04
CON_ID_101	Shed	+64	1.10	+126	2.09	+55	3.53
CON_ID_102	Shed	+65	1.19	+127	2.18	+56	3.62
CON_ID_103	Shed	+65	0.88	+127	1.88	+55	3.32
CON_ID_104	Shed	+64	0.81	+127	1.80	+55	3.24
CON_ID_105	Shed	-26	1.23	-42	2.17	+4	3.54
CON_ID_106	House	-22	1.50	-37	2.44	+4	3.81
CON_ID_118	Shed	+218	0.87	+309	1.83	+66	3.22
CON_ID_119	Shed	+88	1.26	+159	2.24	+61	3.66
CON_ID_120	House	+91	1.15	+162	2.13	+61	3.55
CON_ID_122	Shed	-3	1.62	-6	2.51	+1	3.83
CON_ID_123	House	-3	1.61	-6	2.50	+1	3.82
CON_ID_124	Shed	-3	1.69	-6	2.59	+1	3.91
CON_ID_125	Shed	-1	1.81	-2	2.68	+2	3.98
CON_ID_126	House	-	1.87	-1	2.74	+2	4.02
CON_ID_127	Shed	+1	1.77	+1	2.62	+2	3.89
CON_ID_128	Shed	+1	1.91	+1	2.75	+2	4.03
CON_ID_129	House	-	1.77	-	2.61	+2	3.89
CON_ID_130	House	-	1.77	-	2.62	+2	3.89
CON_ID_145	Shed	-3	1.95	-7	2.85	+1	4.16
CON_ID_146	Shed	+103	0.47	+200	1.00	+90	1.83
CON_ID_147	Shed	+103	0.47	+201	0.99	+88	1.80
CON_ID_148	House	+105	0.52	+202	1.04	+91	1.86
CON_ID_149	House	+88	0.26	+193	0.80	+134	1.61
CON_ID_150	House	-15	0.98	-12	1.81	+16	3.11
CON_ID_151	Shed	-17	0.89	-14	1.72	+16	3.02
CON_ID_152	Shed	-17	0.92	-13	1.76	+15	3.06
CON_ID_153	House	+15	1.09	+26	1.97	+13	3.32
CON_ID_154	Shed	-79	0.82	-69	1.66	+14	2.95
CON_ID_155	House	-88	0.72	-74	1.57	+14	2.86
CON_ID_156	Shed	-91	0.54	-77	1.39	+14	2.68
CON_ID_157	Shed	+62	0.05	+202	0.41	+156	1.21
CON_ID_158	Shed	+61	0.05	+194	0.40	+148	1.22
CON_ID_159	Shed	+3	0.51	+25	0.86	+50	1.61
CON_ID_160	Shed	+23	0.45	+70	0.87	+46	1.70

FSR ID	FSR type	1 in 2,000 AEP event		1 in 10,000 AEP event		PMF event	
		Change in peak water level (mm)	Existing Case flood depth (m)	Change in peak water level (mm)	Existing Case flood depth (m)	Change in peak water level (mm)	Existing Case flood depth (m)
CON_ID_161	House	+25	0.63	+72	1.06	+49	1.88
CON_ID_162	Shed	+24	0.63	+72	1.06	+48	1.88
CON_ID_163	Shed	+2	0.03	+6	0.24	-1	0.79
CON_ID_164	Shed	+1	0.05	+4	0.24	-1	0.78
CON_ID_165	House	-	-	+6	0.18	-1	0.76
CON_ID_166	Shed	-	-	+3	0.90	+6	2.07
CON_ID_167	Silos	-	-	+3	0.60	+6	1.76
CON_ID_168	Silos	-	-	+3	0.66	+6	1.83
CON_ID_169	Silos	-	-	+3	0.72	+6	1.90
CON_ID_170	Silos	-	0.05	+3	0.75	+6	1.92
CON_ID_171	House	-	0.21	+3	0.96	+6	2.13
CON_ID_172	Shed	-	0.22	+3	0.97	+6	2.14
CON_ID_173	Shed	-	0.18	+3	0.94	+6	2.12
CON_ID_174	House	-	0.48	+1	0.92	+5	1.75
CON_ID_175	Shed	-	0.39	+1	0.82	+5	1.65
CON_ID_176	Shed	-	0.55	+1	0.97	+6	1.81
CON_ID_177	Shed	-	0.50	+1	0.93	+5	1.76
CON_ID_178	Shed	-	0.69	+1	1.13	+5	1.95
CON_ID_179	House	-	0.38	+1	0.82	+5	1.65
CON_ID_180	Shed	-	0.71	+1	1.17	+4	1.99
CON_ID_181	Shed	-	0.52	+1	0.95	+5	1.77
CON_ID_182	Shed	-	0.50	+1	0.94	+5	1.76
CON_ID_183	Silos	-	0.46	+1	0.89	+6	1.74
CON_ID_184	Shed	-	0.00	+168	0.23	+52	1.03
CON_ID_185	Shed	-	0.11	-73	0.66	+43	1.43
CON_ID_186	Shed	-121	0.66	-93	1.53	+13	2.81
CON_ID_187	House	-132	0.61	-100	1.49	+13	2.78
CON_ID_188	Shed	-130	0.62	-98	1.50	+13	2.78
CON_ID_189	Shed	-129	0.65	-97	1.52	+13	2.81
CON_ID_190	Shed	-129	0.59	-96	1.46	+13	2.74
CON_ID_191	Shed	-125	0.55	-95	1.42	+13	2.71
CON_ID_192	Shed	-120	0.51	-92	1.37	+13	2.66
CON_ID_193	Shed	+1	1.11	+10	1.73	+14	2.75
CON_ID_194	Shed	+1	1.21	+10	1.82	+13	2.83
CON_ID_195	Shed	+1	1.14	+9	1.75	+13	2.76
CON_ID_196	House	+1	1.19	+10	1.79	+14	2.80
CON_ID_197	Shed	+1	1.15	+9	1.75	+13	2.76
CON_ID_198	Shed	-	-	+2	0.10	-2	1.15
CON_ID_199	Shed	-	-	+2	0.30	-3	1.32

FSR ID	FSR type	1 in 2,000 AEP event		1 in 10,000 AEP event		PMF event	
		Change in peak water level (mm)	Existing Case flood depth (m)	Change in peak water level (mm)	Existing Case flood depth (m)	Change in peak water level (mm)	Existing Case flood depth (m)
CON_ID_200	Shed	-	-	+3	0.28	-3	1.26
CON_ID_201	Shed	-	-	+3	0.15	-3	1.10
CON_ID_202	House	-	-	+3	0.09	-4	1.06
CON_ID_203	Shed	+57	1.18	+124	2.15	+48	3.56
CON_ID_204	Shed	+57	1.30	+124	2.26	+48	3.67
CON_ID_205	House	+58	1.52	+125	2.48	+48	3.89
CON_ID_206	Shed	+58	1.26	+126	2.22	+48	3.63
CON_ID_207	Shed	+58	1.59	+126	2.55	+48	3.96
CON_ID_208	Shed	+16	0.43	+66	1.37	+34	2.81
CON_ID_209	Shed	+20	0.44	+68	1.41	+35	2.85
CON_ID_210	Shed	+17	0.39	+66	1.33	+34	2.77
CON_ID_211	Shed	+17	0.21	+79	0.71	+111	1.61
CON_ID_212	House	+14	0.13	+72	0.36	+104	0.94
CON_ID_213	Shed	+15	0.11	+75	0.62	+107	1.51
CON_ID_214	Shed	+15	0.47	+74	0.98	+107	1.88
CON_ID_215	Shed	+16	0.76	+58	1.73	+30	3.14
CON_ID_216	Shed	+16	1.43	+59	2.39	+30	3.80
CON_ID_217	Shed	+16	1.21	+59	2.16	+30	3.57
CON_ID_218	Shed	+15	1.07	+58	2.02	+30	3.43
CON_ID_219	Shed	+15	0.99	+58	1.94	+30	3.34
CON_ID_220	Shed	+14	0.85	+57	1.80	+29	3.20
CON_ID_221	Shed	+15	0.94	+57	1.89	+29	3.29
CON_ID_222	Shed	+14	0.91	+55	1.86	+29	3.26
CON_ID_223	Shed	+15	0.96	+57	1.91	+30	3.32
CON_ID_224	House	+14	0.85	+56	1.81	+29	3.22
CON_ID_225	House	+15	1.02	+57	1.98	+30	3.39
CON_ID_226	Shed	-	0.58	-	1.04	+5	1.76
CON_ID_227	Shed	-	0.64	-	1.11	+6	1.83
CON_ID_231	Shed	-4	0.01	-11	0.15	+12	1.09
CON_ID_232	Shed	-4	0.06	-11	0.30	+10	1.22
CON_ID_233	Shed	-4	0.10	-10	0.37	+8	1.26
CON_ID_234	House	-	-	-11	0.25	+18	1.24
CON_ID_235	Shed	-4	0.09	-11	0.35	+7	1.26
CON_ID_236	Shed	-3	-	-9	0.21	+14	1.10
CON_ID_237	Shed	-	0.58	+4	1.33	+7	2.51
CON_ID_238	Shed	-	0.66	+4	1.41	+7	2.58
CON_ID_239	House	-	0.61	+4	1.36	+6	2.53
CON_ID_240	House	-	0.60	+3	1.35	+6	2.53
CON_ID_241	House	-	0.65	+4	1.40	+6	2.58

FSR ID	FSR type	1 in 2,000 AEP event		1 in 10,000 AEP event		PMF event	
		Change in peak water level (mm)	Existing Case flood depth (m)	Change in peak water level (mm)	Existing Case flood depth (m)	Change in peak water level (mm)	Existing Case flood depth (m)
CON_ID_242	Shed	-	0.86	+4	1.62	+6	2.80
CON_ID_243	Shed	-	0.69	+4	1.44	+6	2.62
CON_ID_244	Shed	+140	0.34	+235	0.86	+83	1.71
CON_ID_245	House	+122	0.24	+220	0.75	+84	1.58
CON_ID_246	Shed	+113	0.35	+208	0.88	+86	1.72
CON_ID_247	Shed	+107	0.43	+203	0.96	+87	1.80
CON_ID_248	Shed	+106	0.38	+203	0.90	+87	1.73
CON_ID_256	Shed	-	0.75	+4	1.27	+10	2.19
CON_ID_257	Shed	-	1.00	+4	1.50	+9	2.40
CON_ID_258	House	-	0.76	+3	1.25	+9	2.13
CON_ID_259	Shed	-	0.85	+3	1.33	+9	2.21
CON_ID_260	Shed	-	0.79	+3	1.27	+9	2.14
CON_ID_261	Shed	-	0.78	+3	1.26	+9	2.12
CON_ID_262	Shed	-	0.01	-	0.38	+1	1.20
CON_ID_263	Shed	-	0.10	-	0.60	+1	1.43
CON_ID_264	House	-	-	-	0.46	+1	1.27
CON_ID_265	Shed	-	0.23	-	0.70	+1	1.53
CON_ID_266	Shed	-	0.03	-	0.45	+1	1.27
CON_ID_267	Shed	-	0.15	-	0.56	+1	1.38
CON_ID_275	House	+68	0.04	+183	0.38	+145	1.22
CON_ID_276	Shed	+64	0.01	+175	0.23	+138	1.07
CON_ID_277	House	+74	0.22	+202	0.54	+165	1.37
CON_ID_278	Shed	+13	0.76	+68	1.07	+69	1.85
CON_ID_279	Shed	+14	0.65	+69	0.95	+78	1.74
CON_ID_280	Shed	-	-	-16	0.21	+8	1.23
CON_ID_285	Shed	-	0.05	-90	0.58	+45	1.33
CON_ID_286	Shed	-	0.13	-99	0.67	+42	1.41
CON_ID_300	House	-	-	-	-	+20	0.59
CON_ID_302	Silos	+1	0.08	+14	0.75	+16	1.99
CON_ID_303	Shed	+1	0.37	+14	1.05	+17	2.30
CON_ID_307	Shed	-	3.38	-	3.96	+1	4.84
CON_ID_308	Shed	-	3.03	-	3.61	+1	4.49
CON_ID_309	Shed	-	1.11	-	1.72	+1	2.62
CON_ID_310	Pump Station	-	1.00	-	1.59	+1	2.48
CON_ID_311	Pump Station	-	4.38	-	4.98	+1	5.88
CON_ID_312	Shed	-	1.26	-	1.85	+1	2.75
CON_ID_313	Shed	-	1.27	-	1.86	+1	2.75
CON_ID_314	Shed	-	2.20	+1	2.78	+3	3.78

FSR ID	FSR type	1 in 2,000 AEP event		1 in 10,000 AEP event		PMF event	
		Change in peak water level (mm)	Existing Case flood depth (m)	Change in peak water level (mm)	Existing Case flood depth (m)	Change in peak water level (mm)	Existing Case flood depth (m)
CON_ID_316	Shed	-	0.98	+4	1.79	+6	3.00
CON_ID_317	Shed	-	0.93	+4	1.73	+6	2.94
CON_ID_318	Shed	-	0.89	+4	1.69	+6	2.90
CON_ID_319	Shed	-	0.90	+4	1.70	+6	2.91
CON_ID_320	Shed	-	0.86	+4	1.67	+6	2.87
CON_ID_321	Shed	-	0.88	+3	1.68	+6	2.89
CON_ID_322	Shed	-	0.38	+1	1.27	+3	2.54
CON_ID_323	Shed	-	-	+1	0.80	+2	2.10
CON_ID_324	Silos	-	0.08	-	0.72	+2	2.00
CON_ID_325	Shed	-	0.30	-	0.93	+2	2.18
CON_ID_326	Shed	-	0.24	+1	0.88	+2	2.16
CON_ID_327	Shed	-	0.17	-	0.80	+2	2.06
CON_ID_328	House	-	0.16	-	0.79	+2	2.06
CON_ID_329	Shed	-	0.47	+1	1.33	+3	2.57
CON_ID_330	Silos	-	0.31	+1	1.16	+3	2.39
CON_ID_331	Shed	-	0.41	+1	1.21	+3	2.44
CON_ID_332	Shed	-	0.31	+1	1.13	+3	2.36
CON_ID_333	Shed	-	0.32	+1	1.14	+3	2.37
CON_ID_334	Shed	-	0.36	+1	1.17	+3	2.40
CON_ID_335	Shed	-	0.67	+2	1.48	+4	2.70
CON_ID_336	Silos	-	1.30	+2	2.04	+4	3.21
CON_ID_337	Shed	-	0.92	-	1.64	+2	2.78
CON_ID_338	Shed	-	0.79	-	1.50	+2	2.64
CON_ID_339	Silos	-	0.83	-	1.55	+2	2.69
CON_ID_340	House	-	0.65	-	1.37	+1	2.51
CON_ID_341	Shed	-	0.93	-	1.66	+1	2.80
CON_ID_342	Shed	-	1.06	-	1.78	+1	2.92
CON_ID_343	Shed	-	0.94	-	1.67	+1	2.80
CON_ID_344	Shed	-	0.83	-	1.55	+1	2.69
CON_ID_345	House	-	0.69	-	1.42	+1	2.56
CON_ID_346	House	-	0.42	-	1.08	+1	2.19
CON_ID_347	Shed	-	0.68	-	1.35	+1	2.47
CON_ID_348	Shed	-	0.67	-	1.34	+1	2.45
CON_ID_349	Shed	-	-	-	-	+1	2.61
CON_ID_350	House	-	-	+1	0.47	+2	1.66
CON_ID_351	Shed	-	0.11	+1	0.90	+2	2.10
CON_ID_352	Pump Station	-	1.33	+1	2.12	+1	3.28
CON_ID_353	Shed	-	0.28	+1	1.08	+2	2.28
CON_ID_354	Shed	-	0.35	+1	1.15	+2	2.35

FSR ID	FSR type	1 in 2,000 AEP event		1 in 10,000 AEP event		PMF event	
		Change in peak water level (mm)	Existing Case flood depth (m)	Change in peak water level (mm)	Existing Case flood depth (m)	Change in peak water level (mm)	Existing Case flood depth (m)
CON_ID_355	Shed	-	0.38	+1	1.18	+2	2.38
CON_ID_361	Substation	-3	0.67	+1	1.63	+2	2.81
CON_ID_368	Railway	+3	0.88	+65	1.22	+33	2.05
CON_ID_371	Shed	+1	1.86	+3	2.81	+4	4.13
CON_ID_372	Electricity Pole	+7	2.97	+11	3.84	+5	5.15
CON_ID_375	Weighbridge	+218	0.70	+311	1.66	+68	3.04
CON_ID_376	Shed	+46	0.49	+113	1.43	+52	2.87
CON_ID_399	Shed	-	0.19	-	0.66	+1	1.48
CON_ID_409	Shed	+15	0.62	+72	1.13	+105	2.03
CON_ID_410	Silos	+28	0.41	+91	0.95	+119	1.86
CON_ID_411	Shed	-	1.01	+3	1.72	+7	2.88
CON_ID_412	Silos	+14	0.90	+58	1.84	+30	3.24
CON_ID_413	Shed	-	0.47	+1	1.36	+3	2.62
CON_ID_414	Silos	-	0.42	+1	1.31	+3	2.58

7.5.5 Sensitivity analysis—Condamine River

The following sensitivity analyses were completed:

- An increase in rainfall intensity (i.e. to reflect climate change scenario)
- A decrease and increase in blockage of culverts from 25 to 0 per cent and 50 per cent
- Flood level sensitivity to inflow distribution method
- Variability of crop distributions by type, maturity and planting orientation
- Regional flood level sensitivity to Back Creek inflow.

7.5.5.1 Climate change

The potential impacts of climate change on FSRs on the Condamine River floodplain were assessed for the 1% AEP design event. The assessment was undertaken in accordance with ARR 2019.

The Representative Concentration Pathways 8.5 (2090 horizon) climate change scenario has been adopted for the Project with an associated increase in rainfall intensity of 22.8 per cent across the catchment area. The Project is expected to retain a 1% AEP flood immunity, with at least 0.6 m of freeboard to formation level under the climate change scenario.

The climate change sensitivity results predict some increases in flood levels at FSRs, as summarised in Table 7.46. Note, only FSRs that experience change are reported on.

For the 1% AEP event, the change in peak water levels for the RCP 8.5 climate change scenario is presented in Figure C9-C in Volume 2 – Appendix C. The flood impacts under the climate change scenario are expected to increase in magnitude but exhibit a consistent pattern to the present day 1% AEP event scenario.

Table 7.46 Condamine River—1% AEP event for 2090 horizon under the RCP 8.5 scenario: flood sensitive receptors impacts

FSR ID	FSR type	1% AEP climate change event	
		Change in peak water level (mm)	Existing case flood depth (m)
CON_ID_4	Silos	+141	0.62
CON_ID_5	Silos	+157	0.53
CON_ID_6	Silos	+152	0.39
CON_ID_7	Silos	+162	0.57
CON_ID_8	Silos	+142	0.82
CON_ID_9	Silos	+142	0.69
CON_ID_10	Silos	+139	0.68
CON_ID_11	Shed	+1	0.12
CON_ID_12	Shed	+1	0.10
CON_ID_13	Shed	+1	0.17
CON_ID_14	Shed	+1	0.21
CON_ID_15	Shed	+1	0.16
CON_ID_16	Shed	+1	0.10
CON_ID_17	House	+1	0.02
CON_ID_19	Shed	+1	0.13
CON_ID_20	Shed	+1	0.19
CON_ID_23	Shed	+1	0.03
CON_ID_24	Shed	+1	0.10
CON_ID_25	Shed	+1	0.02
CON_ID_26	Shed	+1	0.12
CON_ID_64	Shed	+1	1.23
CON_ID_65	Shed	+1	1.07
CON_ID_66	House	+1	1.01
CON_ID_67	Shed	+1	1.03
CON_ID_68	Shed	+32	0.99
CON_ID_69	Shed	-1	0.46
CON_ID_70	House	-1	0.67
CON_ID_71	House	-1	0.54
CON_ID_72	Shed	-1	0.96
CON_ID_73	Shed	-1	0.79
CON_ID_74	Shed	-2	1.66
CON_ID_75	House	-2	1.59
CON_ID_76	Shed	-2	1.54
CON_ID_77	Shed	-2	1.64
CON_ID_78	House	+43	0.89
CON_ID_79	House	+8	0.91
CON_ID_80	Shed	+9	0.77
CON_ID_81	Shed	+9	0.57
CON_ID_82	Shed	+9	1.01

FSR ID	FSR type	1% AEP climate change event	
		Change in peak water level (mm)	Existing case flood depth (m)
CON_ID_83	Shed	-1	0.58
CON_ID_84	Shed	-1	0.68
CON_ID_85	House	-1	0.45
CON_ID_86	Shed	-1	0.66
CON_ID_87	House	-2	1.58
CON_ID_88	House	-2	0.97
CON_ID_90	House	-7	0.19
CON_ID_91	Shed	-8	0.39
CON_ID_92	House	-8	0.22
CON_ID_93	Shed	-8	0.15
CON_ID_94	Shed	-8	0.04
CON_ID_95	Shed	-7	0.05
CON_ID_96	Shed	+13	0.10
CON_ID_97	Shed	+12	0.41
CON_ID_98	Shed	+12	0.20
CON_ID_99	House	+30	0.39
CON_ID_100	House	+29	0.23
CON_ID_101	Shed	+29	0.72
CON_ID_102	Shed	+29	0.81
CON_ID_103	Shed	+30	0.50
CON_ID_104	Shed	+30	0.43
CON_ID_105	Shed	-23	0.87
CON_ID_106	House	-20	1.14
CON_ID_107	House	+2	0.20
CON_ID_108	Shed	+2	0.15
CON_ID_118	Shed	+162	0.50
CON_ID_119	Shed	+45	0.89
CON_ID_120	House	+47	0.78
CON_ID_121	Shed	-2	1.06
CON_ID_122	Shed	-2	1.29
CON_ID_123	House	-2	1.27
CON_ID_124	Shed	-2	1.36
CON_ID_125	Shed	-2	1.49
CON_ID_126	House	-2	1.55
CON_ID_127	Shed	-1	1.46
CON_ID_128	Shed	-1	1.60
CON_ID_129	House	-1	1.45
CON_ID_130	House	-2	1.45
CON_ID_145	Shed	-2	1.62
CON_ID_146	Shed	+30	0.29
CON_ID_147	Shed	+29	0.26

FSR ID	FSR type	1% AEP climate change event	
		Change in peak water level (mm)	Existing case flood depth (m)
CON_ID_148	House	+34	0.34
CON_ID_149	House	+29	0.07
CON_ID_150	House	-6	0.69
CON_ID_151	Shed	-7	0.60
CON_ID_152	Shed	-6	0.63
CON_ID_153	House	+7	0.79
CON_ID_154	Shed	-36	0.46
CON_ID_155	House	-40	0.36
CON_ID_156	Shed	-175 ²	0.17
CON_ID_157	Shed	+44	0.01
CON_ID_158	Shed	+54	0.02
CON_ID_159	Shed	+1	0.40
CON_ID_160	Shed	+13	0.33
CON_ID_161	House	+13	0.51
CON_ID_162	Shed	+12	0.51
CON_ID_186	Shed	-41	0.28
CON_ID_187	House	-45	0.21
CON_ID_188	Shed	-43	0.23
CON_ID_189	Shed	-43	0.26
CON_ID_190	Shed	-42	0.20
CON_ID_191	Shed	-41	0.17
CON_ID_192	Shed	-40	0.13
CON_ID_193	Shed	+1	0.90
CON_ID_194	Shed	+1	1.00
CON_ID_195	Shed	+1	0.93
CON_ID_196	House	+1	0.98
CON_ID_197	Shed	+1	0.94
CON_ID_203	Shed	+27	0.82
CON_ID_204	Shed	+27	0.93
CON_ID_205	House	+28	1.15
CON_ID_206	Shed	+28	0.90
CON_ID_207	Shed	+28	1.23
CON_ID_208	Shed	+1	0.20
CON_ID_209	Shed	+2	0.15
CON_ID_210	Shed	+1	0.14
CON_ID_211	Shed	+6	0.07
CON_ID_212	House	+5	0.07
CON_ID_215	Shed	+4	0.41
CON_ID_216	Shed	+5	1.07
CON_ID_217	Shed	+5	0.86
CON_ID_218	Shed	+4	0.72

FSR ID	FSR type	1% AEP climate change event	
		Change in peak water level (mm)	Existing case flood depth (m)
CON_ID_219	Shed	+4	0.64
CON_ID_220	Shed	+4	0.51
CON_ID_221	Shed	+4	0.59
CON_ID_222	Shed	+4	0.56
CON_ID_223	Shed	+4	0.60
CON_ID_224	House	+4	0.49
CON_ID_225	House	+4	0.66
CON_ID_228	House	-23	0.08
CON_ID_230	Shed	-27	0.14
CON_ID_232	Shed	-11 ²	0.01
CON_ID_233	Shed	-6	0.04
CON_ID_235	Shed	-6	0.04
CON_ID_237	Shed	+1	0.31
CON_ID_238	Shed	+1	0.39
CON_ID_239	House	+1	0.35
CON_ID_240	House	+1	0.34
CON_ID_241	House	+1	0.38
CON_ID_242	Shed	+1	0.59
CON_ID_243	Shed	+1	0.42
CON_ID_244	Shed	+20	0.18
CON_ID_245	House	+11	0.06
CON_ID_246	Shed	+22	0.17
CON_ID_247	Shed	+25	0.14
CON_ID_248	Shed	+24	0.16
CON_ID_249	Shed	-2	0.11
CON_ID_250	Shed	-2	0.13
CON_ID_251	Shed	-3	0.21
CON_ID_252	Shed	-2	0.08
CON_ID_253	House	-2	0.22
CON_ID_254	Shed	-2	0.17
CON_ID_255	Shed	-2	0.23
CON_ID_256	Shed	+1	0.57
CON_ID_257	Shed	+1	0.84
CON_ID_260	Shed	+1	0.64
CON_ID_266	Shed	-1	0.01
CON_ID_269	Shed	-4	0.04
CON_ID_270	Shed	-5	0.30
CON_ID_271	House	-8	0.10
CON_ID_273	House	-8	0.04
CON_ID_274	Shed	-9	0.25
CON_ID_275	House	+51	0.02

FSR ID	FSR type	1% AEP climate change event	
		Change in peak water level (mm)	Existing case flood depth (m)
CON_ID_276	Shed	+3 ¹	-
CON_ID_277	House	+55	0.16
CON_ID_278	Shed	+7	0.69
CON_ID_279	Shed	+8	0.55
CON_ID_283	Shed	-7	0.02
CON_ID_284	House	-7	0.07
CON_ID_303	Shed	+1	0.15
CON_ID_310	Pump Station	+1	0.78
CON_ID_314	Shed	+1	2.00
CON_ID_316	Shed	+1	0.67
CON_ID_317	Shed	+1	0.62
CON_ID_318	Shed	+1	0.58
CON_ID_319	Shed	+1	0.59
CON_ID_320	Shed	+1	0.56
CON_ID_321	Shed	+1	0.58
CON_ID_329	Shed	+1	0.12
CON_ID_331	Shed	+1	0.15
CON_ID_336	Silos	+1	1.01
CON_ID_355	Shed	+1	0.05
CON_ID_361	Substation	-1	0.28
CON_ID_368	Railway	-1	0.68
CON_ID_371	Shed	+1	1.53
CON_ID_372	Electricity Pole	+1	2.66
CON_ID_375	Weighbridge	+155	0.35
CON_ID_376	Shed	+13	0.18
CON_ID_409	Shed	+6	0.48
CON_ID_410	Silos	+15	0.24
CON_ID_412	Silos	+4	0.57

Table notes:

1 Newly wet in Developed Case

2 Newly dry in Developed Case

7.5.5.2 Culvert blockage

Blockage was assessed in accordance with ARR 2019. The blockage assessment undertaken resulted in a blockage factor of 25 per cent being adopted for culverts. A minimum culvert size of 900 mm diameter was adopted to reduce potential for blockage and maintenance. A significant community concern is that the potential impacts on flood conditions should the proposed culverts become blocked with debris. The primary concern is that the blockage of culverts is likely to drive flood levels higher, particularly upstream of the culverts, and divert more flow through residences, across access roads and other infrastructure. A sensitivity analysis was undertaken with 0 per cent and 50 per cent blockage.

Information has been requested from the DTMR and QR with regards to maintenance records, and historical incidences of debris accumulation, culvert blockage or scour at their respective assets, namely the Gore Highway and the QR rail line through the Condamine River floodplain. The photographic evidence provided by QR is shown in Figure 7.57. The DTMR South West Region do not have any records of debris management at the Gore Highway but the department provided information on historical road closures due to flooding, including December 2010, January 2011, January 2013, February 2013, March 2014 and March 2017.



Figure 7.57 Condamine River floodplain—Evidence of historical debris deposition and scour provided by Queensland Rail (actual flood event unknown)

Table 7.47 summarises the blockage sensitivity results for the 1% AEP scenario. Based on these results, the current FSR outcomes are not expected to be sensitive to the blockage assumptions, unless a very conservative blockage assumption (i.e. 50 per cent) was adopted. During detailed design the blockage factors will be reviewed in line with the final design and local catchment conditions. This may result in a varied and/or lower blockage factors being applied along the Project alignment. Should impacts be determined due to a change in blockage factor, additional mitigation measures will be considered in detailed design.

Maps demonstrating the effects of blockage are shown in Figures C9-A (0 per cent) and C9-B (50 per cent) in Volume 2 – Appendix C.

Table 7.47 **Condamine River—1% AEP event blockage sensitivity: flood sensitive receptor impacts**

FSR ID	FSR type	Existing Case flood depth (m)	Change in peak water level (mm)		
			0% blockage	25% blockage	50% blockage
CON_ID_4 ¹	Silos	0.17	+71	+82	+102
CON_ID_5 ¹	Silos	0.03	+135	+146	+168
CON_ID_6	Silos	-	+25 ¹	+36 ¹	+57 ¹
CON_ID_7 ¹	Silos	0.08	+132	+144	+166
CON_ID_8 ¹	Silos	0.37	+75	+86	+107
CON_ID_9 ¹	Silos	0.24	+74	+85	+106
CON_ID_10 ¹	Silos	0.24	+70	+80	+101
CON_ID_11	Shed	0.02	+1	+1	+1
CON_ID_13	Shed	0.09	+1	+1	+1
CON_ID_14	Shed	0.12	+1	+1	+1
CON_ID_15	Shed	0.06	+1	+1	+1
CON_ID_16	Shed	0.02	+1	+1	+1
CON_ID_19	Shed	0.06	+1	+1	+1
CON_ID_20	Shed	0.10	+1	+1	+1
CON_ID_24	Shed	0.01	+1	+1	+1
CON_ID_26	Shed	0.05	+1	+1	+1
CON_ID_63	Shed	0.64	-1	-1	-1
CON_ID_64	Shed	0.83	-1	-1	-1
CON_ID_65	Shed	0.67	-1	-1	-1
CON_ID_66	House	0.62	-1	-1	-1
CON_ID_67	Shed	0.63	-1	-1	-1
CON_ID_68	Shed	0.46	+17	+18	+21
CON_ID_69	Shed	0.11	-1	-1	-1
CON_ID_70	House	0.33	-1	-1	-1
CON_ID_71	House	0.19	-1	-1	-1
CON_ID_72	Shed	0.60	-1	-1	-1
CON_ID_73	Shed	0.43	-1	-1	-1
CON_ID_74	Shed	1.20	-1	-1	-2
CON_ID_75	House	1.13	-1	-1	-2
CON_ID_76	Shed	1.08	-1	-1	-2
CON_ID_77	Shed	1.18	-1	-1	-2
CON_ID_78 ¹	House	0.37	+2	+25	+28
CON_ID_79	House	0.67	+3	+3	+4
CON_ID_80	Shed	0.52	+3	+3	+4
CON_ID_81	Shed	0.32	+3	+3	+4
CON_ID_82	Shed	0.75	+3	+3	+4
CON_ID_83	Shed	0.23	-1	-1	-1
CON_ID_84	Shed	0.34	-1	-1	-1

FSR ID	FSR type	Existing Case flood depth (m)	Change in peak water level (mm)		
			0% blockage	25% blockage	50% blockage
CON_ID_85	House	0.09	-1	-1	-1
CON_ID_86	Shed	0.30	-1	-1	-1
CON_ID_87	House	1.12	-1	-1	-1
CON_ID_88	House	0.58	-1	-1	-1
CON_ID_91	Shed	0.11	-3	-3	-3
CON_ID_97	Shed	0.06	+3	+4	+5
CON_ID_98	Shed	0.00	+3	+4	+5
CON_ID_101	Shed	0.21	+14	+16	+21
CON_ID_102	Shed	0.30	+14	+16	+21
CON_ID_103	Shed	0.06	+14	+16	+20
CON_ID_105	Shed	0.39	-14	-16	-21
CON_ID_106	House	0.66	-12	-14	-18
CON_ID_118 ¹	Shed	0.02	+116	+128	+150
CON_ID_119	Shed	0.42	+23	+26	+34
CON_ID_120 ¹	House	0.30	+25	+29	+37
CON_ID_121	Shed	0.61	-1	-1	-1
CON_ID_122	Shed	0.83	-1	-1	-2
CON_ID_123	House	0.81	-1	-1	-2
CON_ID_124	Shed	0.89	-1	-1	-2
CON_ID_125	Shed	1.04	-1	-2	-2
CON_ID_126	House	1.10	-1	-2	-2
CON_ID_127	Shed	1.04	-1	-1	-2
CON_ID_128	Shed	1.17	-1	-1	-2
CON_ID_129	House	1.02	-1	-1	-2
CON_ID_130	House	1.02	-1	-1	-2
CON_ID_145	Shed	1.16	-1	-1	-1
CON_ID_146	Shed	0.09	+19	+19	+19
CON_ID_147	Shed	0.06	+21	+21	+21
CON_ID_148 ¹	House	0.13	+21	+21	+22
CON_ID_150	House	0.37	+4	+3	+2
CON_ID_151	Shed	0.28	+4	+3	+2
CON_ID_152	Shed	0.29	+4	+3	+2
CON_ID_153	House	0.46	+3	+3	+4
CON_ID_154	Shed	0.09	+3	+2	-1
CON_ID_155	House	0.06	+3	+2	-1
CON_ID_159	Shed	0.26	+0	+0	+1
CON_ID_160 ¹	Shed	-	+189 ¹	+189 ¹	+189 ¹
CON_ID_161	House	0.36	+4	+4	+4
CON_ID_162	Shed	0.36	+3	+3	+3
CON_ID_203	Shed	0.41	+10	+12	+15
CON_ID_204	Shed	0.52	+10	+12	+16

FSR ID	FSR type	Existing Case flood depth (m)	Change in peak water level (mm)		
			0% blockage	25% blockage	50% blockage
CON_ID_205 ¹	House	0.74	+10	+12	+16
CON_ID_206	Shed	0.50	+10	+12	+16
CON_ID_207	Shed	0.83	+10	+12	+16
CON_ID_208	Shed	0.08	+1	+1	+1
CON_ID_215	Shed	0.05	+1	+1	+1
CON_ID_216	Shed	0.72	+1	+1	+1
CON_ID_217	Shed	0.52	+1	+1	+1
CON_ID_218	Shed	0.38	+1	+1	+1
CON_ID_219	Shed	0.32	+1	+1	+1
CON_ID_220	Shed	0.21	+1	+1	+1
CON_ID_221	Shed	0.26	+1	+1	+1
CON_ID_222	Shed	0.26	+1	+1	+1
CON_ID_223	Shed	0.26	+1	+1	+1
CON_ID_224	House	0.14	+1	+1	+1
CON_ID_225	House	0.30	+1	+1	+1
CON_ID_235	Shed	0.01	-6	-6	-6
CON_ID_247	Shed	0.03	+14	+14	+14
CON_ID_248	Shed	0.02	+16	+16	+16
CON_ID_250	Shed	0.03	-1	-1	-1
CON_ID_253	House	0.10	-1	-1	-1
CON_ID_254	Shed	0.05	-1	-1	-1
CON_ID_255	Shed	0.12	-1	-1	-1
CON_ID_270	Shed	0.22	-6	-6	-6
CON_ID_274	Shed	0.16	-8	-8	-7
CON_ID_275 ¹	House	0.02	+34	+34	+34
CON_ID_277 ¹	House	0.10	+36	+36	+37
CON_ID_278	Shed	0.59	+1	+1	+2
CON_ID_279	Shed	0.45	+1	+1	+1
CON_ID_283	Shed	0.01	-7	-7	-7
CON_ID_284	House	0.04	-7	-7	-7
CON_ID_368	Railway	0.60	-2	-2	-2
CON_ID_372	Electricity Pole	2.26	-1	-0	-0
CON_ID_409	Shed	0.26	+1	+1	+1
CON_ID_410	Silos	0.04	+2	+2	+3
CON_ID_412	Silos	0.26	+1	+1	+1

Table notes:

- 1 Newly wet in Developed Case
- 2 Newly dry in Developed Case

7.5.5.3 Local inflow approach

The assessment of cross-drainage response along the Project alignment requires appropriate proportioning of subarea inflows between the upstream and downstream sides of the Project alignment. This is commonly achieved by re-splitting the existing hydrological subarea at the Project alignment so that two inflows, one upstream and another downstream of the alignment, can be generated in the hydrology model and applied in the hydraulic model (i.e. 'default' approach). The process implicitly assumes that the presence of the Project has negligible influence on the natural runoff behaviour of the catchment, except reducing the catchment area. Assumptions must also be made on where these 'split' inflows are best applied in the hydraulic model.

As described in Section 7.2.1, a custom approach was used to simultaneously apply subarea inflows on both sides of the Project alignment without having to re-split the catchment at the alignment, while accounting for the presence of the proposed rail. This is achieved by evenly distributing the hydrologically generated subarea inflows along known streams and rivers within the hydraulic model extent (i.e. 'SA ALL' approach), particularly where they cross the Project alignment. The partial re-routing of the subarea inflows during the hydraulic redistribution is expected to have minimal impact on the regional flooding behaviour given the significantly larger regional catchment size (~7,000 km²) than the local subarea size (~50 km²).

Figure 7.58 shows the effect of applying local inflow using the default approach in terms of differences in peak flood levels for the 1% AEP event. The flood level differences are minor (i.e. within 20 mm) across most of the floodplain, with the distributed inflow approach resulting in slightly higher flood levels along the edges and on the islands of the floodplain. This demonstrates that the floodplain conditions are primarily determined by the upstream catchment inflows and are insensitive to the local inflow approach used.

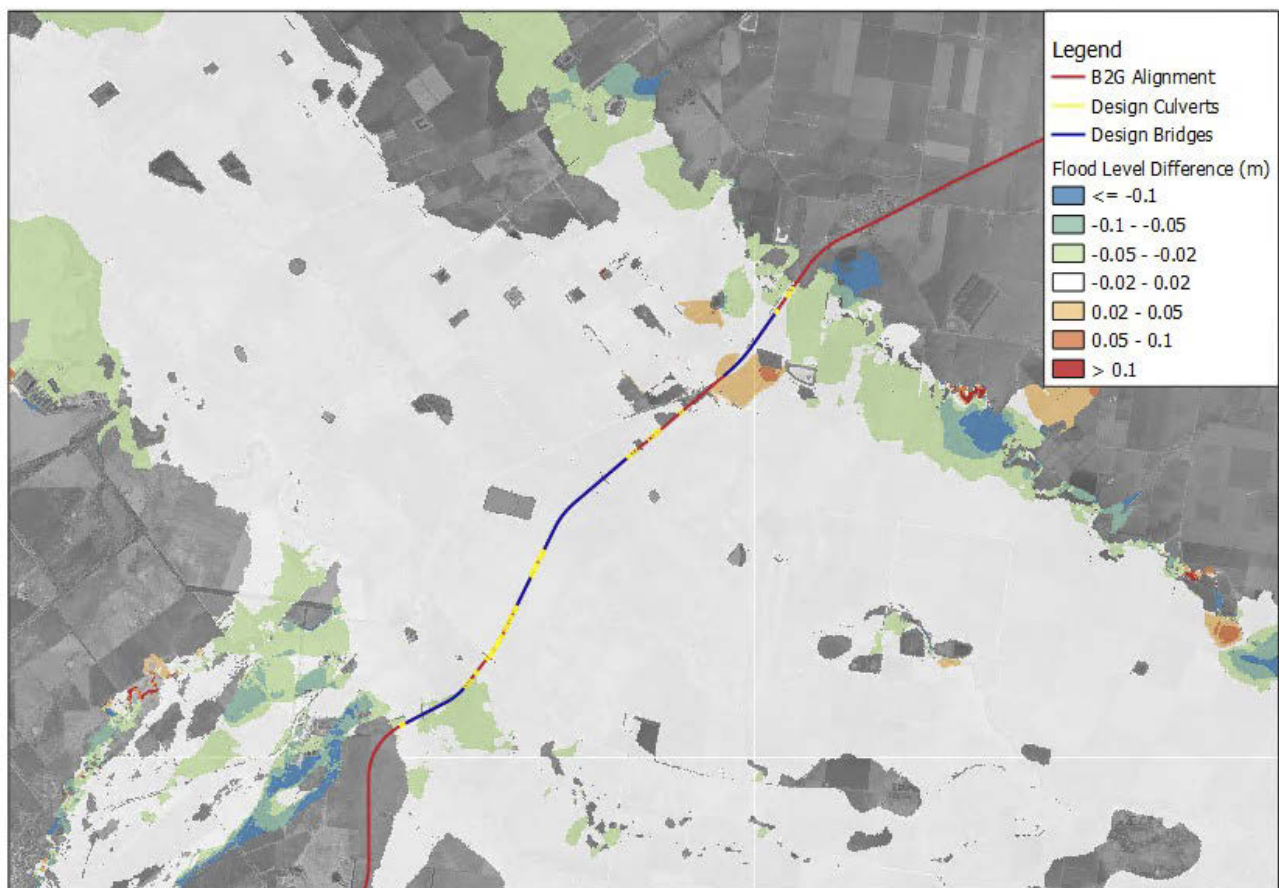


Figure 7.58 Condamine River—Flood level sensitivity to local inflow approach – 1% AEP: default approach minus 'SA ALL' approach

Figure 7.59 shows the effect of applying local inflow using the default approach in terms of differences in differences (i.e. default SA approach afflux – ‘SA ALL’ approach afflux) in the 1% AEP afflux, with the locations of FSRs shown as blue points. The afflux differences at all FSRs and across most of the floodplain are negligible (i.e. within 10 mm). Only one area on the North Branch downstream of the Project alignment exhibits slightly larger afflux differences (between 10 mm and 50 mm), which is likely due to the limitations of the default inflow application approach in applying inflow only at the subarea outlet.

The results of the sensitivity tests show that the floodplain conditions are primarily determined by the upstream catchment inflows and the flooding impacts are insensitive to the local inflow approach; therefore, the adopted ‘SA ALL’ inflow approach is deemed suitable and representative for use.

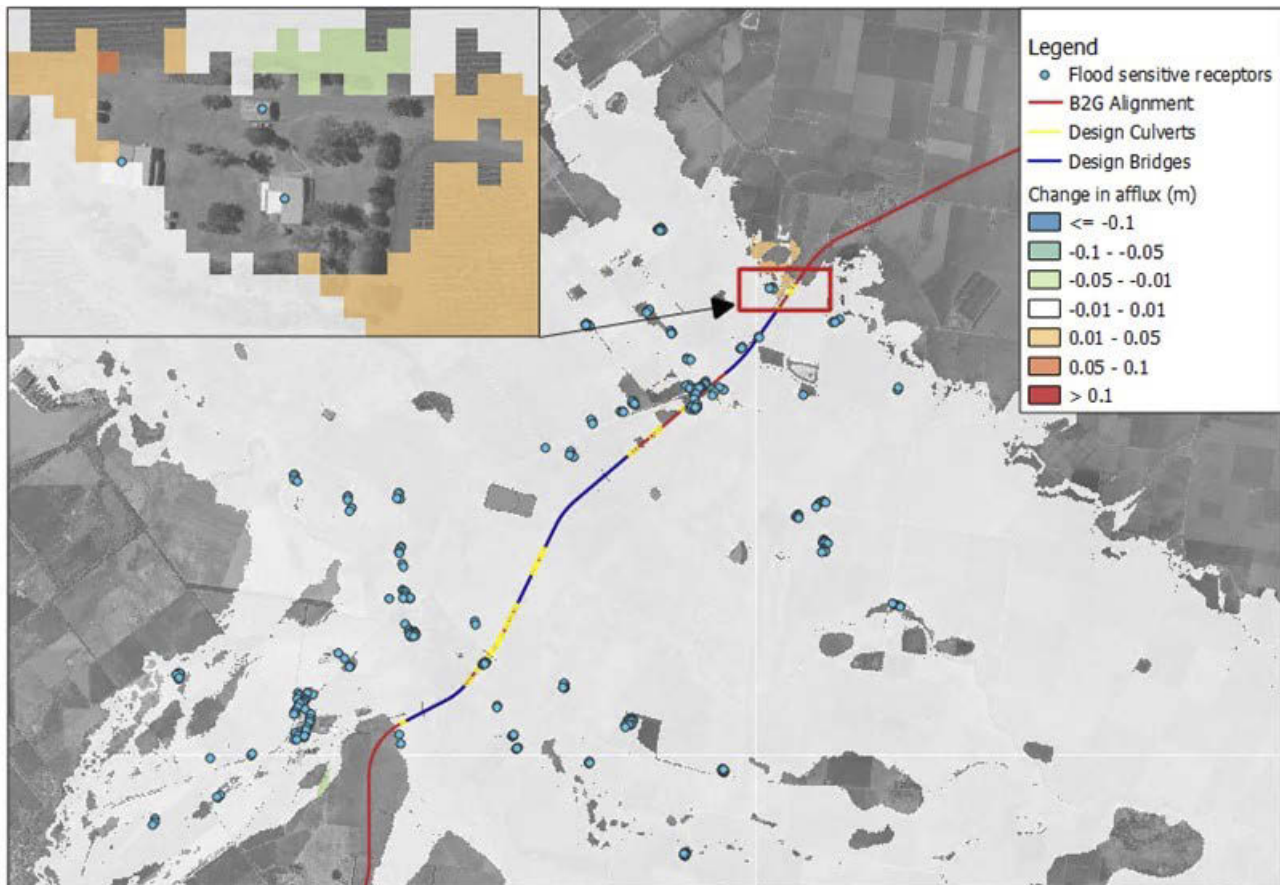


Figure 7.59 Condamine River—Afflux sensitivity to local inflow approach—1% AEP: default SA approach afflux minus ‘SA ALL’ approach afflux

7.5.5.4 Spatial and seasonal variability in crops

The local community has raised concerns that historical floods have differed significantly from one another and that one of the factors that drives this variation is the different types of crops that are being cultivated at the time of flood. Furthermore, several community members have stated that the height of each crop (or its maturity) and its planting orientation has had visibly obvious impacts on flow direction and velocities at different times of a flood.

Research was undertaken into the types of crops grown in the area and their cycles—including times of fallow—to understand what range of ground conditions could be expected during the times of year when floods were likely. The effect of spatial and seasonal variabilities in crop type and maturity was then tested in the model by randomly assigning the roughness values spatially, based on the areal weighting of each crop and its probability of occurrence in each season (wheat, barley, chickpeas in winter, and sorghum, cotton, corn in summer) produced from the crop cycle research.

This process made it possible to stochastically generate crop patterns across the floodplain that were statistical representations of crop patterns likely to occur at the time of a flood. In total, 10 different crop patterns were generated, using the crop types and roughness values summarised in Table 7.48. The detailed roughness values in Table 7.48 were obtained using Cowan's method (Cowan, 1956) by modifying a base Manning's value, obtained following best industry practice from Chow (1959), to account for additional roughness and cross-section characteristics including surface irregularity, size and shape variation, obstructions, vegetation and meandering.

Table 7.48 **Condamine River—Crop Manning's n values used for spatial variability test**

Crop type	Roughness
Barley – Mature – Perpendicular	0.155
Barley – Mature – Parallel	0.113
Barley – Seedling – Perpendicular	0.082
Barley – Seedling – Parallel	0.060
Chickpeas – Mature – Perpendicular	0.110
Chickpeas – Mature – Parallel	0.080
Chickpeas – Seedling – Perpendicular	0.082
Chickpeas – Seedling – Parallel	0.060
Corn – Mature – Perpendicular	0.247
Corn – Mature – Parallel	0.180
Corn – Seedling – Perpendicular	0.082
Corn – Seedling – Parallel	0.060
Cotton – Mature – Perpendicular	0.155
Cotton – Mature – Parallel	0.113
Cotton – Seedling – Perpendicular	0.110
Cotton – Seedling – Parallel	0.080
Fava Beans – Mature – Perpendicular	0.155
Fava Beans – Mature – Parallel	0.113
Fava Beans – Seedling – Perpendicular	0.082
Fava Beans – Seedling – Parallel	0.060
Forage Sorghum – Mature – Perpendicular	0.155
Forage Sorghum – Mature – Parallel	0.113
Forage Sorghum – Seedling – Perpendicular	0.082
Forage Sorghum – Seedling – Parallel	0.060
Millet – Mature – Perpendicular	0.155
Millet – Mature – Parallel	0.113
Millet – Seedling – Perpendicular	0.110
Millet – Seedling – Parallel	0.080
Mung Beans – Mature – Perpendicular	0.110
Mung Beans – Mature – Parallel	0.080
Mung Beans – Seedling – Perpendicular	0.082
Mung Beans – Seedling – Parallel	0.060
Oats – Mature – Perpendicular	0.155
Oats – Mature – Parallel	0.113
Oats – Seedling – Perpendicular	0.082

Crop type	Roughness
Oats – Seedling – Parallel	0.060
Sorghum – Mature – Perpendicular	0.155
Sorghum – Mature – Parallel	0.113
Sorghum – Seedling – Perpendicular	0.082
Sorghum – Seedling – Parallel	0.060
Soyabeans – Mature – Perpendicular	0.110
Soyabeans – Mature – Parallel	0.080
Soyabeans – Seedling – Perpendicular	0.082
Soyabeans – Seedling – Parallel	0.060
Wheat – Mature – Perpendicular	0.247
Wheat – Mature – Parallel	0.180
Wheat – Seedling – Perpendicular	0.082
Wheat – Seedling – Parallel	0.060

Figure 7.60 shows differences between flood levels generated by subtracting the upper median result (i.e. Rank 6) from the adopted uniform Manning's n (0.1085) and generated using all 10 crop patterns, incorporating different seasonal and spatial variations. The flood level changes include newly wet or newly dry areas along the floodplain fringe. Overall, flood levels and flood extent on the Condamine River floodplain are insensitive to crop variability.

Such variability may result in flood level variations of ± 200 mm at a local scale, however. These localised effects are triggered by specific roughness application patterns that are unlikely to occur regularly, and the global sensitivity to crop variability remains neutral. It is noted that a small pocket of increase (>200 mm) occurs adjacent to the alignment, and a cluster of houses and sheds. Upon closer inspection, this increase is localised to farmland and does not impact the associated houses and sheds.

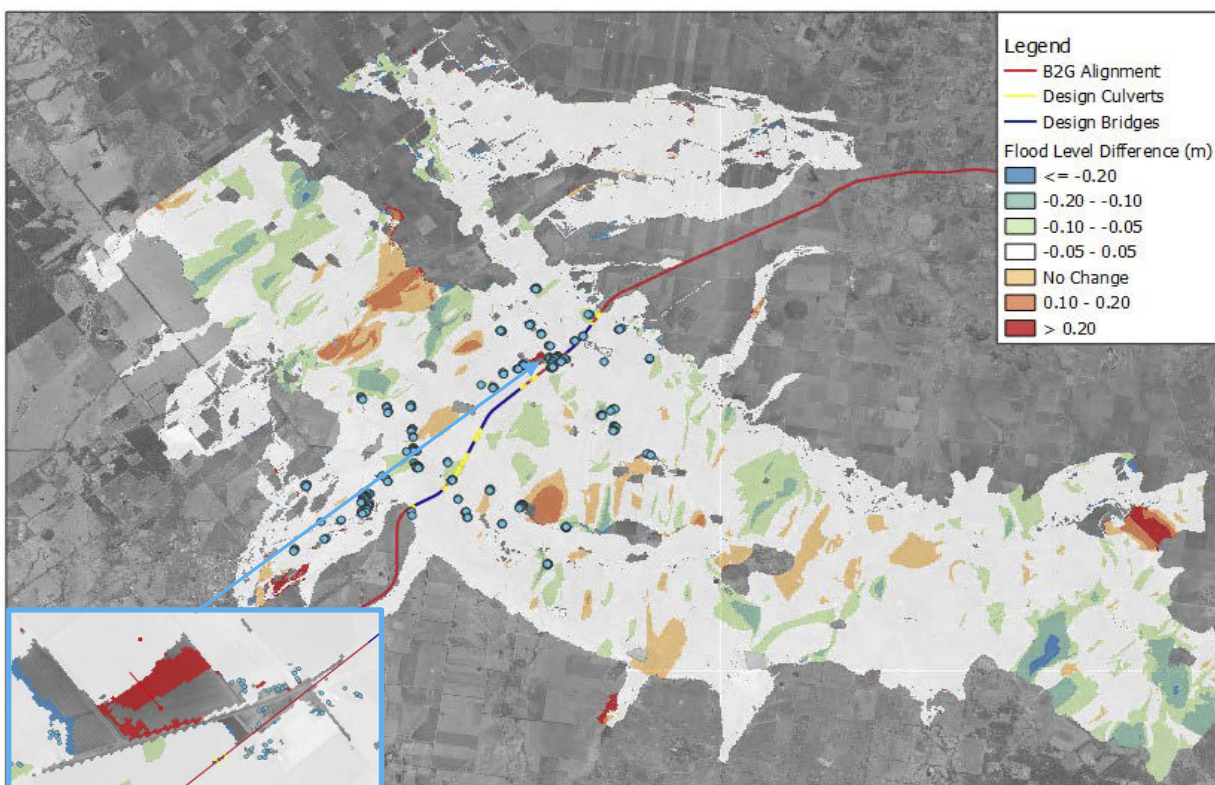


Figure 7.60 Condamine River—Crop variability effects on flood level—1% AEP 2880 min: single uniform roughness: 6th lowest ranked sensitivity test result (i.e. one above median)

Figure 7.61 and Figure 7.62 show the crop variability effects on flood extent and depth, where highly sensitive areas correlate with a low 'likelihood of inundation' (i.e. a 20 per cent likelihood of inundation represents a 1 in 5 chance that the area would remain wet if a different crop distribution is assumed). The majority of the regional floodplain is not sensitive to crop variations (i.e. 100 per cent likelihood of inundation), and higher sensitivities only occur on fringes and islands along the floodplain. Adopting a single uniform roughness is therefore considered a robust assumption for reproducing the floodplain inundation under average conditions.

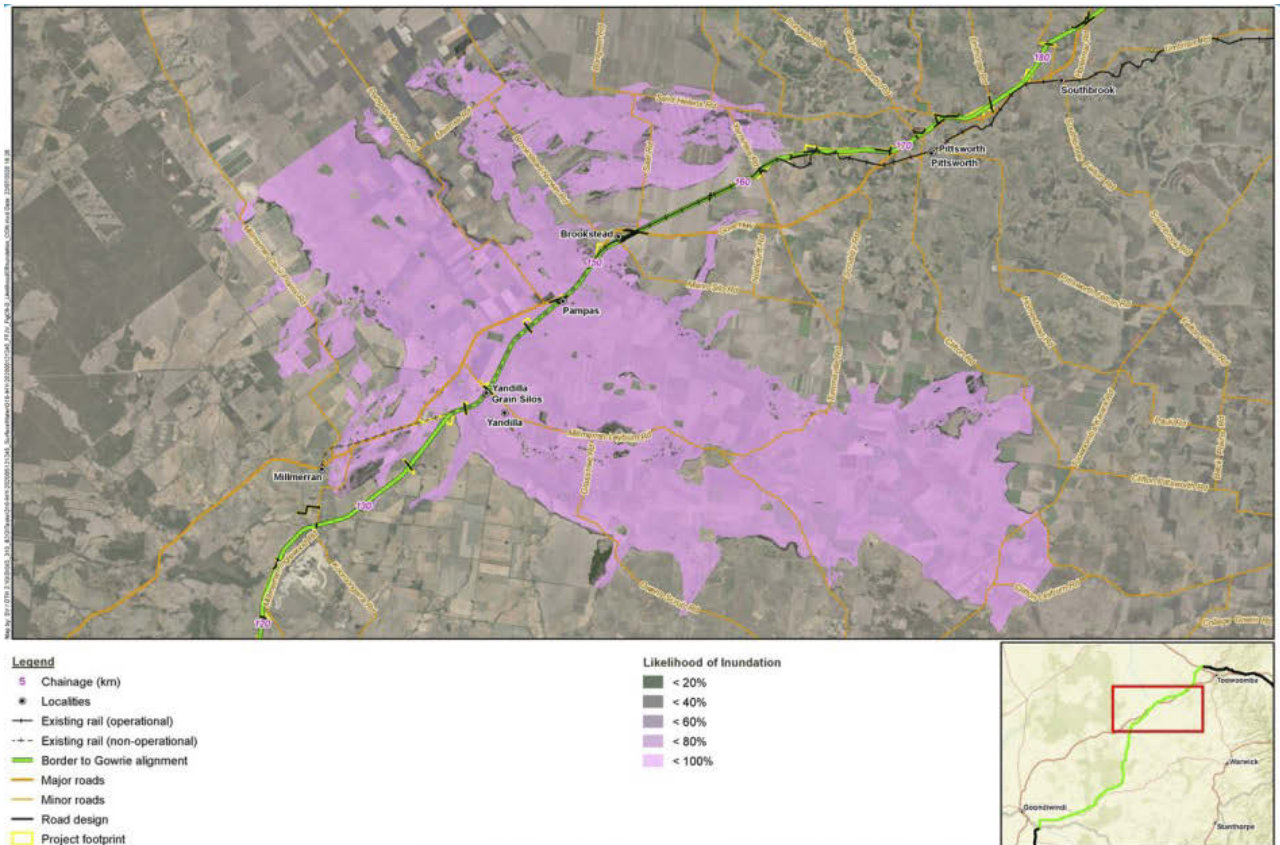


Figure 7.61 Condamine River—Crop variability effects on flood extent—1% AEP 2880 min (reproduced from the draft EIS)

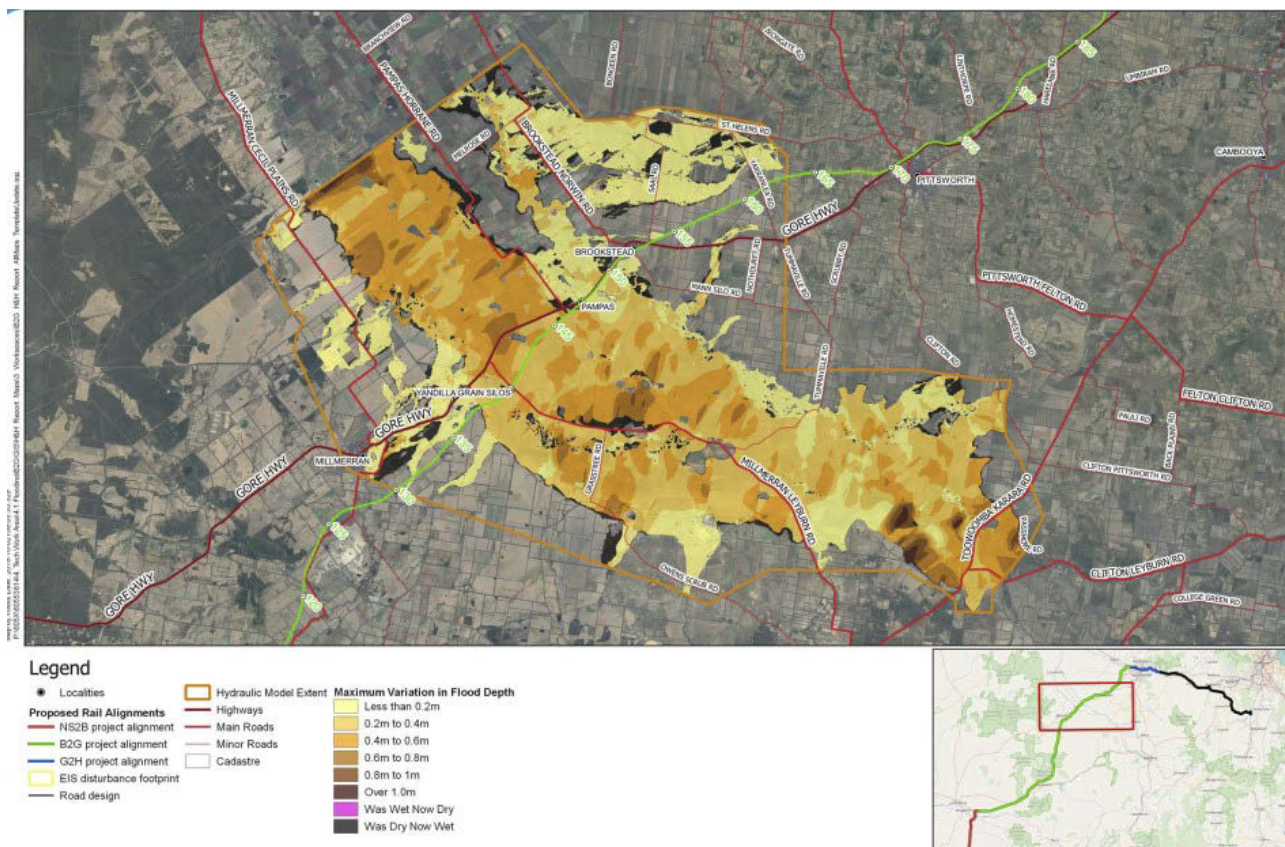


Figure 7.62 Condamine River—Crop variability effects on flood depth—1% AEP event (reproduced from the draft EIS)

Figure 7.63 summarises the impact on flood levels on the identified FSRs due to crop variability. The results demonstrate negligible overall impact on FSRs, with average and median difference both below 10 mm.

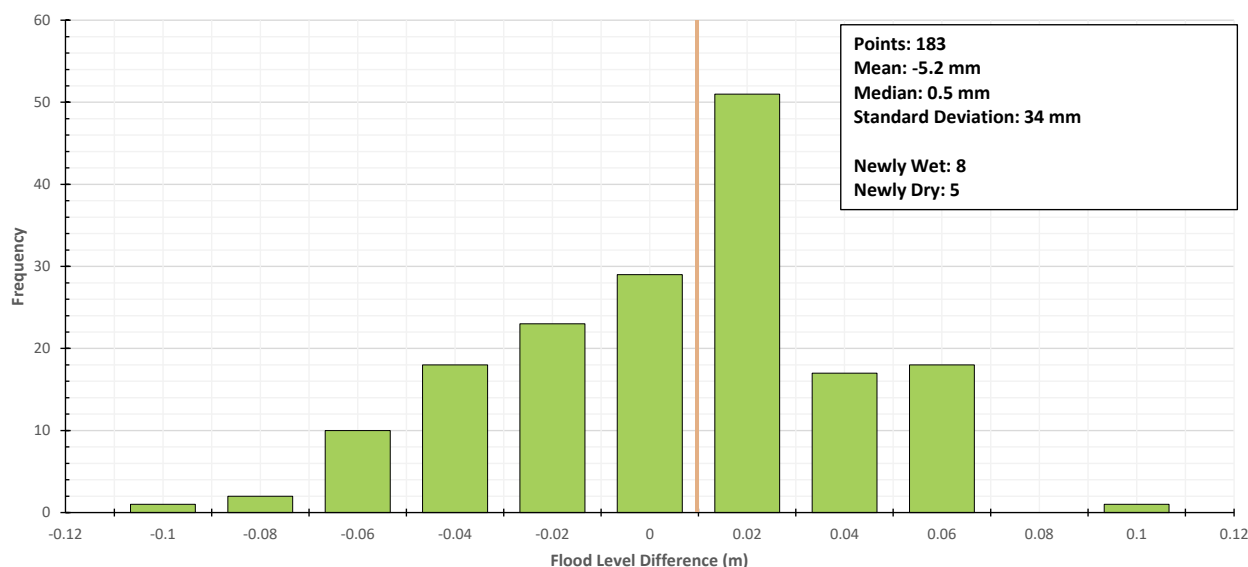


Figure 7.63 Condamine River—Crop variability effects on flood sensitive receptors

The results of the sensitivity tests show that adopting a uniform representative roughness value generates a robust average prediction on the Condamine River floodplain, and improves model confidence in predicting impacts in future conditions subject to spatial and seasonal variations in cropping.

7.5.5.5 Back Creek interactions

Back Creek drains the 106 km² catchment upstream of Millmerran into the Condamine River. In frequent, smaller flood events, Back Creek meanders to the Condamine approximately 16 km downstream of the Gore Highway; however, in larger events, Back Creek breaks its banks earlier and floodwaters flow more directly to the Condamine River at around the Centenary Bridge.

As Back Creek's catchment size is significantly smaller than that of the Condamine River, the timing of its flood peak is much shorter. The peak 1% AEP local flood event in Back Creek is a 9-hour event, whereas the peak 1% AEP regional flood event is 2 to 4 days. This difference in timing implies that modelling the Condamine River solely from a regional perspective could overlook potential localised impacts from a Back Creek flood event.

Therefore, to better understand this behaviour and any potential impacts of differences in flood peak timing, five sensitivity analyses were undertaken, as shown in Table 7.49.

Table 7.49 Sensitivity tests for interactions between Back Creek and Condamine River

Scenario	Condamine River inflow	Inclusion of internal Back Creek inflows within the Condamine River hydraulic model	URBS model used to generate external Back Creek inflows into Condamine River hydraulic model
1	Yes	No	N/A
2	Yes	Yes	Condamine River regional model
3	Yes	Yes	Back Creek local model
4	No	Yes	Condamine River regional model
5	No	Yes	Back Creek local model

All of the above scenarios were analysed the 1% AEP event in the regional Condamine River hydraulic model domain using three durations: 2,880, 4,320 and 5,760 minutes. Inflows downstream of the Project alignment and Millmerran were not adjusted as they were already included in the Condamine River regional flood model.

The flood extents produced by Scenario 1 above are shown in Figure 7.64. The peak Condamine River flood level, where Back Creek joins, is approximately 379.9 mAHD. The creek bed level where the Gore Highway passes over Back Creek is approximately 394.8 mAHD. These two locations are roughly 7 to 8 km apart; therefore, flooding in Condamine River is expected to produce small-to-negligible backwater effects in the Back Creek channel around Millmerran.

As the confluence of Back Creek and the Condamine River is downstream of the Project alignment, the distribution of flows from Back Creek are not expected to have any material impact on the current design of the proposed embankments, culverts or bridges of the Project.

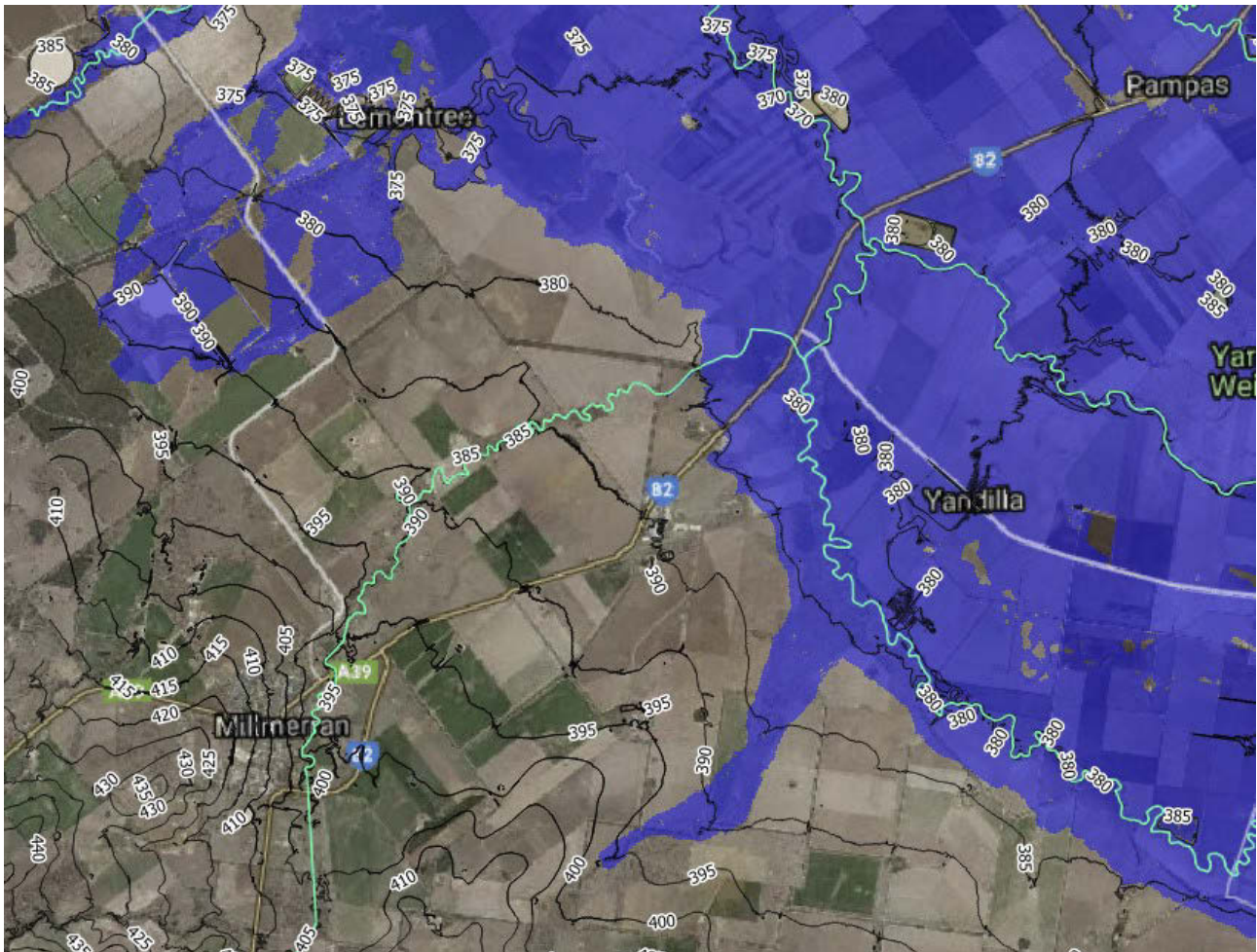


Figure 7.64 Regional flood event extents (blue hatch) without Back Creek contribution (Scenario 1) (1% AEP event)

In Scenario 3, the coincident upstream Back Creek inflow was reassessed using the Back Creek local hydrologic model (Section 8.2) and applying the regional ARF. This results in peak flow increases in the order of 50 to 100 per cent over Scenario 2, as summarised in Table 7.50.

Table 7.50 Back Creek inflow sensitivity to areal reduction factor

1% AEP event duration (minutes)	Peak Back Creek Inflow in Scenario 3 (m ³ /s)	Peak Back Creek Inflow in Scenario 2 (m ³ /s)
2,880	156.9	103.0
4,320	153.0	75.0
5,760	127.3	86.0

The flood level sensitivities to adopting the local Back Creek model inflows in the regional model are shown in Figure 7.65. The larger peak flows in the Back Creek channel produce higher peak water levels downstream of Millmerran, although flood extents are largely unaffected. Minor flood level decreases (up to -10 mm) were identified in areas along Condamine River, due in part to the changed timing of the flood contribution from Back Creek.

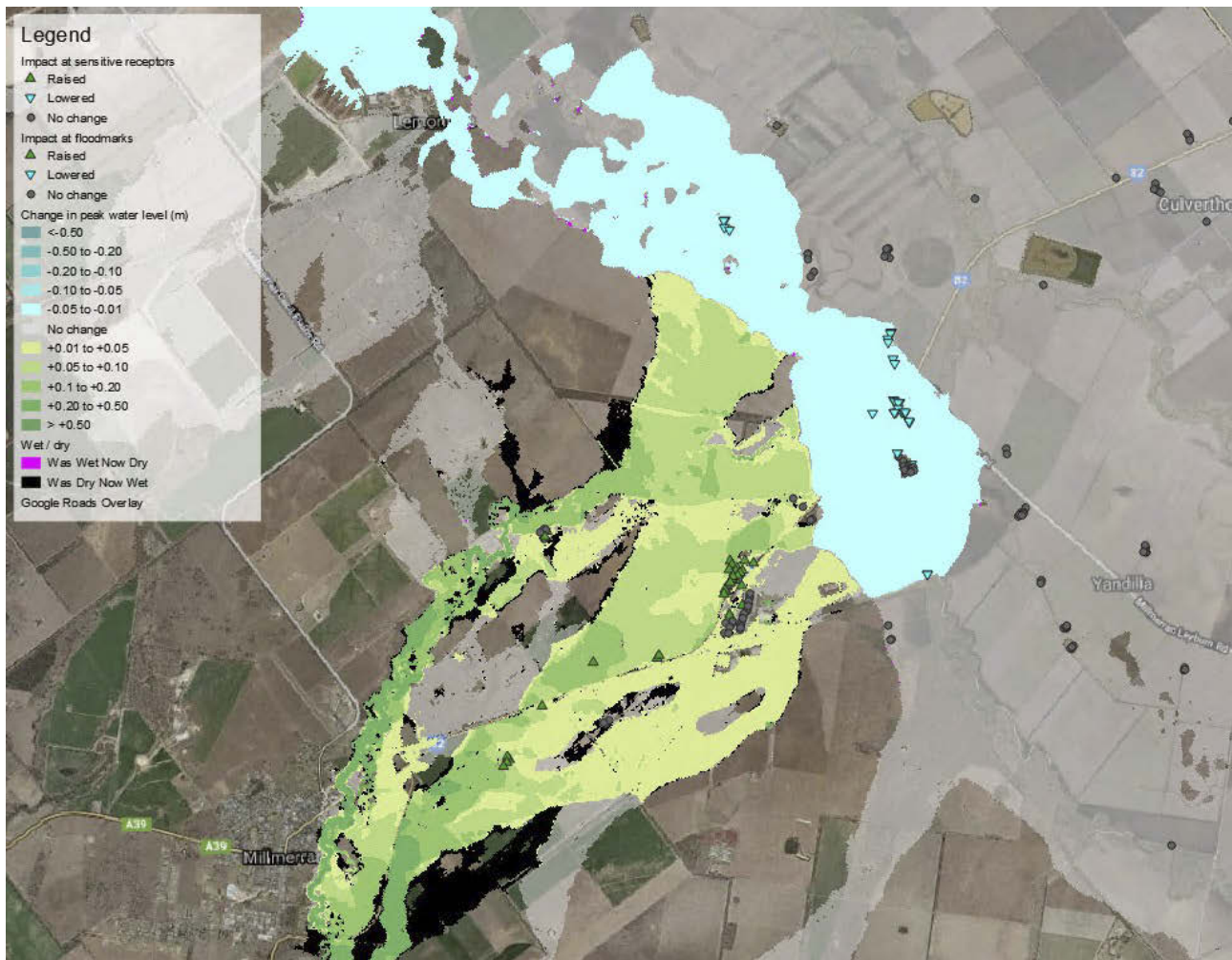


Figure 7.65 Influence of local Back Creek model inflows to peak flood levels—assessing the difference of Scenarios 3 and 1 and the difference of Scenarios 2 and 1 (1% AEP event)

The flood impact sensitivities to Back Creek inflows were further assessed at 286 FSRs and 50 historic flood marker locations on the Condamine River floodplain. The impact of adopting Back Creek inflows analysed as a local catchment is summarised in Figure 7.66, which shows:

- Peak flood levels are unaffected at most FSR locations (235), whereas 22 FSRs would have lower flood levels and 29 FSRs would have higher flood levels, with a typical variation of 17 mm.
- Peak flood levels are unaffected at most historic flood markers (34), whereas 10 would have lower flood levels and 6 would have higher flood levels with a typical variation of 16 mm.

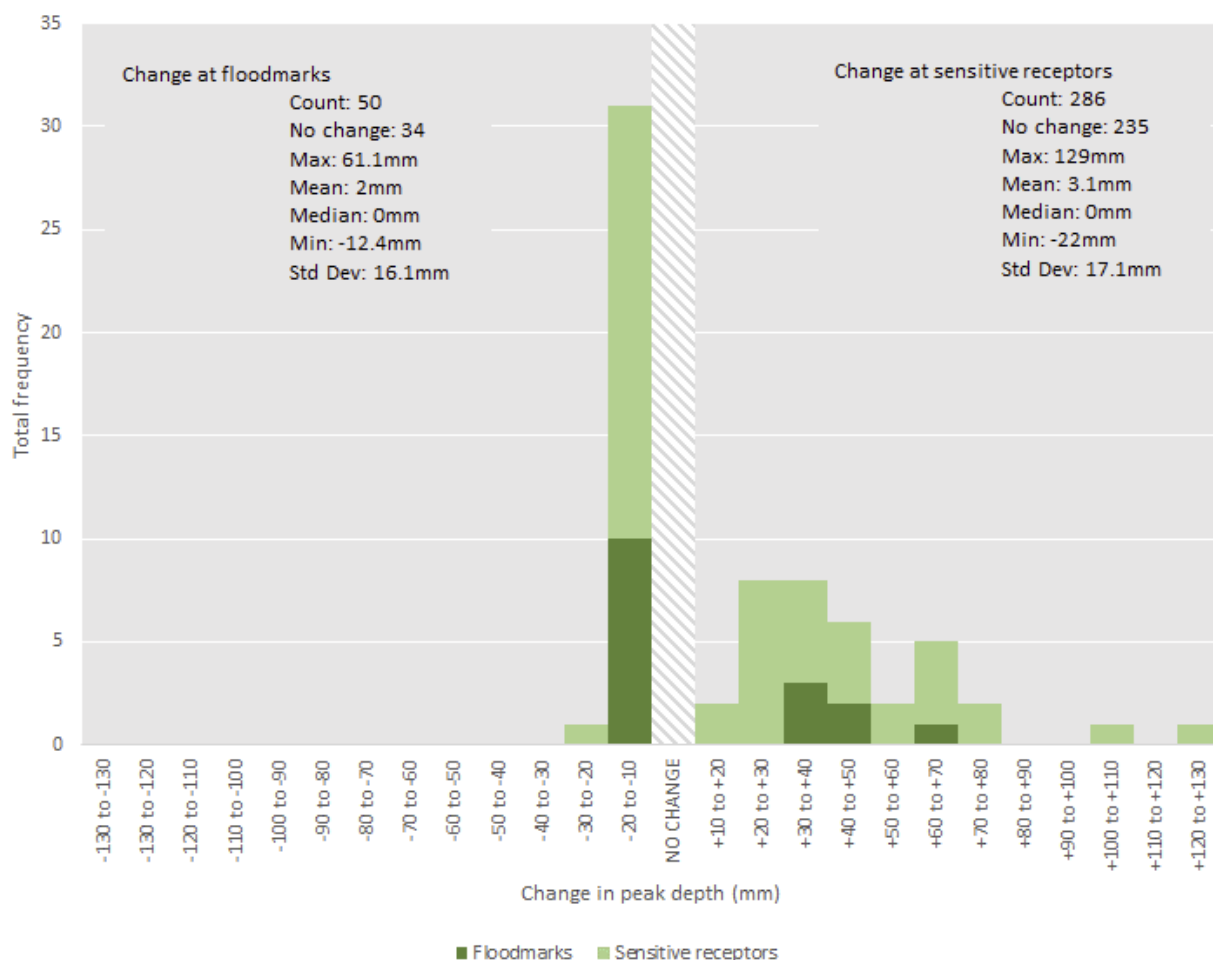


Figure 7.66 Impact of adopting Back Creek inflows—Frequency distribution of peak flood level changes at flood markers and sensitive receptors (1% AEP event)

The sensitivity analyses indicate that Back Creek has a negligible influence on the regional flood behaviour along the Condamine River, despite some peak flood level uncertainties in the upstream Back Creek channel outside the limit of the regional floodplain.

7.6 Additional modelling items to be considered in detailed design

The following points summarise additional items to be included in the flood models for detailed design, as per agreement with the Expert Flood Panel:

- An additional longer critical storm duration will be included in the hydraulic assessment for events smaller than the 1% AEP event
- Blockage factors will be reviewed in line with the final design and local catchment conditions
- Further consideration to climate change design provisions will be given in line with the regulator approved Project conditions or limits
- Refine hydraulic model definition of land use and topography using up-to-date survey data, including: 1) refinement of roughness definition along channels; 2) review and inclusion of relevant missing culverts under Gore Highway; 3) improving definitions of cross-drainage, structural controls including weirs, bridges, roads and railways where required.
- Review and implement the most appropriate bridge loss modelling approach
- Where practicable, a review of mapping cut-offs will be undertaken to potentially improve flood impact representation

- Review and refine river roughness application to ensure global consistency
- Review and refine definitions of road and railway controls where the flood levels may be sensitive
- Review and refine definition of rail embankment
- Review model boundary definition for extreme events and update as required
- Review and refine existing bridge definitions around Cecil Weir and the downstream model boundary
- A review of the latest TUFLOW version will be undertaken at the commencement of detailed design to determine whether updates of the TUFLOW version from 2020-10-AD-iSP-w64 are beneficial to modelling.

Further details of mitigation measures can be found in Section 22. A review of proposed mitigation measures will be completed during the detailed design stage of the Project after hydraulic models have been updated to capture the above changes.

7.7 Planned and future roads

Consultation with the DTMR has not identified any planned or future road upgrades within the Condamine River catchment. ARTC will continue to consult and engage with DTMR to determine any future or planned road projects that may need to be considered as part of the detailed design hydraulic modelling.

TRC identified timber bridge replacement projects along Melrose Road and Branchview Road being currently planned within the Condamine River catchment in 2023 and 2024, respectively. These bridge locations are approximately 10 km downstream of the Project alignment; therefore, it is not anticipated that these projects will influence the hydraulic modelling for the Project alignment. ARTC will continue to consult and engage with TRC to determine any future or planned road projects that may need to be considered as part of the detailed design hydraulic modelling.

8 Back Creek

The Project alignment crosses the study area via Back Creek as well as a tributary to the south of Millmerran. Back Creek is a narrow and meandering system which in the 1% AEP event has flood depths of up to 5.4 m in its main channel. The flood depths along the alignment range from 0.5 m to 1.5 m through the floodplain crossings with a depth of 4.7 m at the Project alignment crossing of the Back Creek channel. The Existing Case 1% AEP inundated floodplain varies between 400 m and 2.3 km wide.

Commodore Mine is approximately 2 km upstream of the proposed crossing on Back Creek. Future mine development may involve a diversion of Back Creek itself, which in turn may require additional flood modelling to ascertain any potential effects the diversion could have on the Project alignment.

The location of the Project alignment in relation to Back Creek is shown in Figure D1-A in Volume 2 – Appendix A.

8.1 Data collection and review—Back Creek

Available background information including existing hydrologic and hydraulic models, survey, streamflow data, available calibration information and anecdotal flood data was gathered. Data was sourced from a wide range of stakeholders, including:

- TRC—existing flood studies
- BoM—rainfall data
- DTMR—existing infrastructure details.

8.1.1 Previous studies

A number of previous hydrology and hydraulic studies were sourced as part of this assessment. A review of each study was undertaken to determine suitability for use on the Project, as documented in the following sections.

8.1.1.1 Work Package 3, Historical study for Millmerran Final Report, DHI/WRM 2014c

This study focused on a small reach of Back Creek extending approximately from the southern Millmerran-Inglewood Road bridge to Yandilla. The study involved the development of a coupled 1D/2D MIKE FLOOD hydraulic model. The model attempted to recreate the anecdotally observed flood extents of the January 2011 event by scaling steady state inflows. The model could not be used for the Project's hydrology and flooding assessment because of the unsuitability of the hydrology underpinning the hydraulic model.

8.1.2 Survey

ARTC provided LiDAR data from 2015 as 1 m grid DEM tiles. Using GIS software, a DEM was generated with a 1 m grid resolution for use in the Project, based on the 2015 dataset. This was used for modelling within the permanent footprint and up to the full extent of the 2015 LiDAR where relevant.

In areas that were not covered by the LiDAR provided by ARTC, LiDAR tiles were sourced from Geoscience Australia. The DEM datasets used for modelling were based on surveys flown between 2009 and 2015. SRTM data was used for catchment delineation where no LiDAR data could be sourced to inform the hydrologic modelling.

The survey data sources and DEM developed for Back Creek are shown in Figure D1-B in Volume 2 – Appendix D.

8.1.3 Aerial imagery

Aerial imagery of the Back Creek study area was provided by ARTC and was used to identify and confirm topographic and vegetative characteristics of the study area. Aerial imagery captured in 2015 was made available. Additional imagery outside the study area was sourced from QGIS imagery in an open-source format.

8.1.4 Existing drainage structure data

No available DTMR as-constructed drawings were available for culvert and bridge details.

Additional information will be refined as local survey is progressively completed during detailed design.

8.1.5 Stream gauge data

No streamflow gauges exist within the Back Creek catchment study area.

8.1.6 Rainfall data

A number of daily and sub-daily rainfall stations are located in and around the Back Creek catchment; however, since there are no streamflow gauges to use for model calibration, no historical rainfall data was sought.

8.1.7 Anecdotal and observed flood data

Several local landowners have commented on the rapid catchment response to large rainfall events and the velocity of associated flood waters. One landowner was able to identify an aerial image where flood levels reached during the 2011 and 2013 events on his property adjacent to Back Creek. Additional consultation with landowners may be necessary to discuss veracity of modelling results.

8.1.8 Site inspection

A site inspection was undertaken during February 2018. During the site inspection, all major waterway crossings were visited and inspected with photographs taken and details recorded of the crossing, existing drainage structures and surrounding catchment. An assessment of the relative roughness and blockage potential was undertaken during the site inspection.

8.2 Hydrologic model development—Back Creek

8.2.1 Model setup

A calibrated hydrologic model for the Upper Condamine River was established by FFJV, as outlined in Section 7.2. The results from this model reflect the large-scale, regional characteristics of longer duration floods and tend to underestimate the expected flood flows arising from a local storm event in Back Creek. Due to this, a local URBS hydrological model was established for the Back Creek catchment.

The URBS hydrologic model covers approximately 151 km² of the Back Creek catchment up to the Grasstree Creek confluence. The proposed rail bridge (310-BR38) was selected as the focal point of hydrological analysis given its location immediately upstream of the Millmerran township. The combined sub-catchment area draining to the focal point is 92.1 km².

The Back Creek catchment was delineated into 29 sub-catchments, with 12 sub-catchments within the focal area. Approximately 3.5 km² of Back Creek catchment reporting to Commodore Mine was excluded from the hydrologic analysis.

The URBS hydrologic model extent and sub-catchment boundaries are presented in Figure D1-C in Volume 2 – Appendix D.

The URBS model setup details are summarised in Table 8.1.

Table 8.1 Summary of Back Creek URBS model details

Input parameter	Details
URBS model type	Basic
Routing variables	Catchment area, stream lengths
Channel lag parameter, α	0.85
Catchment non-linearity parameter, m	0.8

Note that default values were adopted for all other URBS parameters.

8.2.2 Sub-catchments

The Back Creek catchment was delineated into 29 sub-catchments, covering an area of 151 km², with 12 sub-catchments within the focal area. Approximately 3.5 km² of Back Creek catchment reporting to Commodore Mine was excluded from the hydrologic analysis.

The hydrologic model extent and sub-catchment map is presented in Figure D1-C in Volume 2 – Appendix D.

8.2.3 Design event parameters

Hydrologic information to assist estimation of design event flows was sourced from the ARR 2019 Data Hub, as summarised in Table 8.2.

Table 8.2 Back Creek—Summary of URBS model design event inputs

Input parameter	Remarks
Design rainfall	Intensity-Frequency-Durations for each sub-catchment were downloaded from the BoM's Data Hub to account for variation in rainfall across the catchment.
Extreme event rainfall	Probable Maximum Precipitation depths for durations up to 6 hours (for use in modelling the PMF event) were obtained using the method presented in the Bulletin 53 (BoM, 2003). The rainfall depths for the 1 in 10,000 AEP event were estimated using the interpolation method presented in ARR 2019 Book 8 Section 3.5.
Losses	The losses were initially adopted from the ARR Data Hub (Initial loss – 34 mm, Continuing loss – 0.7 mm/hr). Adjustments were made to reconcile URBS flows to QRT estimates as per Expert Flood Panel recommendation. Zero losses were applied under the 1:10,000 AEP and PMP.
Areal reduction factor	Parameters were adopted for the semi-arid inland Queensland region. The catchment area upstream of the hydrological focal point is 92.1 km ² , which yields an ARF of between 69.4% and 96.8% depending on design storm event AEP and duration. No areal reduction was applied for design events rarer than the 1 in 2,000 AEP event.
Ensemble TPs	Central Slopes regions. However, as the study catchment area exceeds 75 km ² , the standard ensemble rainfall patterns from ARR 2019 do not apply to this catchment for storm durations longer than 12h. These were replaced with the areal TPs for the Central Slopes region.
Pre-burst depths	Median pre-burst depths were downloaded from the ARR 2019 Data Hub for each sub-catchment. Pre-burst depths vary by design storm event AEP and duration. Pre-burst depths were applied to the model by reducing the initial losses for each storm event.

8.2.3.1 Spatial distribution

The ARR 2019 design event methodology was adopted for this assessment. ARR 2019 recommends adopting the spatially varying method for catchments larger than 20 km². The Back Creek catchment is approximately 151 km²; therefore, each sub-catchment was assigned an IFD derived from the BoM website at the sub-catchment centroid.

8.2.3.2 Rainfall depth estimation

Design rainfall depth for 50% to 1 in 2,000 AEP events were obtained from BoM website for each sub-catchment. As an example, the IFD representing the catchment centroid of Back Creek URBS model is provided in Table 8.3.

Due to the size of the model area, there is minimal difference in rainfall depth across the catchment.

Table 8.3 Rainfall depth (mm) used for the Back Creek catchment centroid

Duration (hr)	Annual Exceedance Probability (% AEP)						
	50	20	10	5	2	1	0.05
0.25	17.5	24.5	29.2	34.0	40.3	45.3	70.8
0.50	23.9	33.4	40.1	46.7	55.7	62.8	98.0
1.00	30.2	42.2	50.7	59.2	70.9	80.1	125.0
2.00	36.5	50.5	60.5	70.7	84.7	95.8	149.0
3.00	40.3	55.4	66.2	77.2	92.3	104.0	162.0
4.50	44.6	60.7	72.2	83.8	100.0	113.0	176.0
6.00	47.9	64.9	76.9	89.0	106.0	119.0	186.0
12.00	57.5	77.1	90.7	104.0	124.0	139.0	216.0
24.00	69.7	93.5	110.0	126.0	149.0	168.0	262.0
48.00	83.8	114.0	135.0	157.0	186.0	210.0	336.0
72.00	92.1	127.0	151.0	176.0	210.0	238.0	375.0

8.2.3.3 Areal reduction factor

The ARF was estimated using the coefficients for semi-arid inland Queensland presented in Table 8.4 and was applied to all IFD areas. The URBS software directly downloaded these coefficients from ARR 2019 Data Hub to estimate the ARF using Equation 2.4.1 of ARR 2019 Book 2.

The estimated ARF for Back Creek catchment vary between 69.4 per cent and 96.8 per cent depending on design storm event AEP and duration. No areal reduction was applied for design events rarer than the 1 in 2,000 AEP event.

Table 8.4 Back Creek—Areal reduction factor coefficients from ARR 2019 Data Hub

a	b	c	d	e	f	g	h	i
0.159	0.283	0.25	0.308	7.30E-07	1.0	0.039	0.0	0.0

8.2.3.4 Critical duration and temporal patterns

Combinations of 7 storm durations (between 30 min and 72 hours) and 10 TPs (50%, 20%, 10%, 5%, 2%, 1%, 1 in 2,000, 1 in 10,000, PMF) were assessed in the calibrated URBS model to determine the critical storm duration and TP combinations for use in the hydraulic design and impact assessment of the Project.

Temporal patterns for 20% AEP to 1 in 2,000 AEP events were adopted based on 10 burst patterns recommended by ARR 2019. The TPs and pre-burst data were directly downloaded by URBS software from the ARR 2019 Data Hub at the centroid of Back Creek catchment. Temporal patterns for the 1 in 10,000 AEP and PMF design events for up to 6 hours were obtained from *The Estimation of Probable Maximum Precipitation in Australia: Generalised Short-Duration Method* (BoM, 2003).

Among the 10 TPs assessed for each storm duration, the critical TP was selected as that which produced the 5th highest (i.e. one above median) discharge at the focal point of hydrologic analysis (310-BR38). The critical TP is a robust yet conservative characterisation of its corresponding storm duration. The storm duration that produces the highest critical discharge was then selected as the critical duration.

The adopted critical duration and TP combinations for hydraulic modelling are shown in Table 8.5.

Table 8.5 Back Creek—Critical duration and temporal patterns for hydraulic modelling

AEP (%)	Critical duration storm/temporal pattern
20	Pattern 4
10	Pattern 7
5	Pattern 0
2	Pattern 7
1	Pattern 9
1 in 2,000	Pattern 9
1 in 10,000	PMP Temporal Pattern
PMF	PMP Temporal Pattern

8.2.3.5 Pre-burst

Median pre-burst depths for durations from 30 min to 72 hours for a 1% AEP design event are between 0 mm and 15.7 mm (ARR 2019 Data Hub). Due to the small depth of pre-burst in this area, the impact of pre-burst condition is considered to be minimal in this catchment. The adopted pre-burst depths are summarised in Table 8.6.

Table 8.6 Back Creek—Adopted pre-burst depths

Duration (hours)	Pre-burst depth (mm)					
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP
0.5	0.6	0.8	0.9	1	2.2	3.1
0.75	0.6	0.8	0.9	1	2.2	3.1
1.0	0.6	0.8	0.9	1	2.2	3.1
1.5	0.2	0.3	0.4	0.4	1.6	2.5
2.0	0.2	0.2	0.3	0.3	2.1	3.4
3.0	0	0	0.1	0.1	3.5	6
6.0	0.2	0.3	0.4	0.4	0.9	1.2
9.0	0	0.1	0.2	0.2	5.3	9.1
12.0	0	0.1	0.2	0.2	5.3	9.1
18.0	0	0.5	0.9	1.2	9.5	15.7
24.0	0	0.7	1.2	1.7	7	11
48.0	0	0	0	0	0.1	0.2
72.0	0	0	0	0	0	0

8.2.3.6 Design rainfall losses

The constant initial and continuing losses initially adopted in the URBS model were 34 mm and 0.7 mm/hr, respectively. The losses were iteratively adjusted to reconcile the modelled peak flows with QRT as detailed in Section 8.2.4.

The adopted design event losses are provided in Table 8.7.

Table 8.7 Back Creek—Adopted design event losses

AEP	Initial storm loss (mm)	Continuing loss (mm/hr)
20%	63.0	1.26
10%	60.6	1.21
5%	42.0	0.84
2%	28.0	0.56
1%	21.0	0.42

8.2.3.7 Extreme event rainfall

Probable Maximum Precipitation depths for durations up to 6 hours (for use in modelling the PMF event) were obtained using the methodology presented in *The Estimation of Probable Maximum Precipitation in Australia: Generalised Short-Duration Method* (BoM, 2003). Once the PMP was estimated, rainfall for events between 1 in 2,000 AEP and the PMF was then interpolated. For the 1 in 10,000 AEP event, ARF were applied based on interpolation methodology presented in ARR 2019 Section 3.5 in Book 8 and the rainfall depths were estimated accordingly.

The parameters adopted for PMP rainfall estimation are presented in Table 8.8. The adopted extreme rainfall depths are provided in Table 8.9.

Probable Maximum Precipitation calculations will be reviewed and refined during detailed design.

Table 8.8 Back Creek—Input data for Probable Maximum Precipitation estimation

Factor	Value
EAF	1
MAF	0.8
Portion of rough catchment	100%
Location	(-27.889, 151.275)
Catchment area (km ²)	106

Table 8.9 Back Creek—Adopted extreme event rainfall depths (mm)

Duration (hour)	1 in 10,000 AEP rainfall depth (mm)	PMP rainfall depth (mm)
0.25	86	142
0.5	120	208
0.75	140	264
1	154	315
1.5	171	407
2	184	470
2.5	193	528
3	201	574
4	215	650

Duration (hour)	1 in 10,000 AEP rainfall depth (mm)	PMP rainfall depth (mm)
5	225	711
6	232	756

8.2.4 Model calibration

In the absence of historic streamflow data to calibrate the Back Creek URBS catchment, the URBS model peak flows have been adjusted to align with those produced by the DTMR QRT method as per Expert Flood Panel recommendations.

Iterative adjustments of the URBS routing parameter and ARR Data Hub catchment losses were undertaken until the URBS design flood quantiles reproduced the QRT estimates under AEPs between 20% and 1%. The calibration results are summarised in Table 8.10.

A tabulation of the adopted URBS design parameters is provided in Table 8.2.

Table 8.10 Back Creek—Comparison of URBS and QRT peak flows

AEP	QRT estimated discharge (m ³ /s)	URBS model discharge (m ³ /s)	Difference between URBS and QRT estimates
20%	66.3	65.8	-0.8%
10%	109.4	110.2	0.7%
5%	169.4	170.3	0.6%
2%	260.0	261.4	0.6%
1%	339.8	340.5	0.2%

8.2.5 Hydrologic model validation

8.2.5.1 Area-scaled FFA at Canal Creek

In the absence of historic streamflow data to validate the Back Creek catchment, the URBS model peak flows were compared to an area-scaled FFA of an analogous catchment.

Canal Creek was chosen for this exercise due to its proximity to Back Creek and its relatively similar topography and catchment area. The catchment area of Canal Creek is 395 km² to the stream gauge at Leyburn (422338A), which is approximately 430 per cent of the size of the Back Creek focal area (92.1 km²).

The area-scaled FFA was not used for calibration as there was no suitable topographic data to verify the rating curve on Canal Creek, which has only been gauged up to approximately the 10% AEP event.

The Canal Creek FFA discharge quantiles were scaled using the following formula:

$$\frac{Q_{Back\ Creek}}{Q_{Canal\ Creek}} = \left(\frac{Catchment\ Area_{Back\ Creek}}{Catchment\ Area_{Canal\ Creek}} \right)^n$$

An n-value of 0.7 was adopted (typically 0.6 < n < 0.8). The scaling calculation predicts a ratio of 36.1 per cent between discharge quantiles of Back Creek and those of Canal Creek.

Table 8.11 provides a comparison of the FFA peak flows at Canal Creek, the scaled peak flows at Back Creek, and the peak flows produced using the calibrated URBS model. A good correspondence is observed between the URBS design flows and scaled FFA flows, despite general uncertainties associated with the latter approach, demonstrating that the adopted URBS design flows are robust for rail design and impact assessment purposes.

Table 8.11 Canal Creek peak flow estimated (LP3) and scaled flow at focal point (310-BR38)

AEP (%)	FFA peak flow (m ³ /s)	Scaled peak flow at Back Creek (m ³ /s)	URBS design flow (m ³ /s)	Difference between URBS design flow and scaled Back Creek flow
20	190	69	66	-4.3%
10	338	122	110	-9.8%
5	493	178	170	-4.5%
2	693	250	261	4.4%
1	830	300	341	13.7%

8.2.5.2 Regional Flood Frequency Estimate

The Regional Flood Frequency Estimate (RFFE) from ARR 2019 was used as an additional validation of peak flows produced by the Back Creek URBS model. The RFFE technique is generally subject to much greater uncertainties than an at-site FFA, as it essentially represents a transfer function that produces flood quantile estimates using a limited set of predictor variables (e.g. catchment area and design rainfall intensity), which do not always sufficiently describe the flood-producing characteristics in the target catchment.

Table 8.12 compares the RFFE peak flows and URBS design flows at the focal point of the hydrological analysis (310-BR38), indicating that the URBS flows are within the RFFE confidence limits but, on average, 40 per cent below the median RFFE flows.

The RFFE predictions were influenced by the 15 nearest gauges to the catchment outlet, most of which are located along the Great Dividing Range, which could be sensitive to intra-basin flood variabilities. A bias correction of -0.477 was reported at the nearest gauge on Canal Creek at Leyburn, implying that the RFFE overestimates the flood peaks compared to the at-site observations that were used to develop the area-scaled FFA (Section 8.2.5.1) by approximately 60 per cent. The comparison between URBS and RFFE is considered reasonable and within the typical relative error (i.e. difference between RFFE and FFA flood quantile estimates) of the RFFE technique (generally around ± 50 per cent).

Table 8.12 Back Creek—Comparison between Regional Flood Frequency Estimation and URBS results at focal point

AEP (%)	RFFE—5% confidence limit (m ³ /s)	RFFE—estimate of flow (m ³ /s)	RFFE—95% confidence limit (m ³ /s)	URBS design flow (m ³ /s)	Difference between URBS design flow and RFFE
20	42	104	257	66	-37%
10	66	171	451	110	-36%
5	90	259	735	170	-34%
2	127	418	1,360	261	-38%
1	158	577	2,080	341	-41%

8.3 Hydraulic model development—Back Creek

A 2D modelling approach was adopted to appropriately simulate flood mechanisms around the Project alignment at Back Creek. The platform used for hydraulic modelling is TUFLOW HPC software package (2020-10-AD) with SGS and QT approach. The processes and assumptions adopted throughout the development of the hydraulic model are described in the following sections.

8.3.1 Model setup

The setup of the TUFLOW model is summarised in Table 8.13.

Table 8.13 Back Creek—Hydraulic model summary

Parameter	Information
Completion date	June 2022
AEPs assessed	20%, 10%, 5%, 2%, 1%, 1 in 2,000, 1 in 10,000 AEP and PMF
Hydraulic model build	TUFLOW HPC GPU – version 2020-10-AD-iSP-w64
Model extent	Refer to Figure D1-D in Volume 2 – Appendix D
Grid size	5 m
SGS size	1 m
QT grid size	2.5 m, 2 nd Order
DEM (year flown)	ARTC dataset (2015) and Queensland LiDAR Data – Brookstead dataset (2012).
Roughness	Spatially varying roughness values compliant with industry norms.
Eddy viscosity	Wu (default)
Model validation	2010 event
D/S model boundary	Height-Discharge (HQ) Boundary with normal slope approximated based upon topography dataset.
Hydraulic model timestep	Adaptive Timestep
Modelled scenarios	Existing Case, Developed Case
Sensitivity analysis	Blockage – 0%, 50% Climate change

The hydraulic model extent and the spatial distribution of land use in the 2D model domain is presented in Figure D1-D and Figure D1-E in Volume 2 – Appendix D.

8.3.2 Model topography and grid size

The topography within the TUFLOW hydraulic model was based on a 1 m DEM data collected during 2015, which was supplied by ARTC, supplemented by the 1 m Queensland LiDAR Data – Brookstead dataset, flown as part of the Inland Town Stage 4 project in 2012. A 5 m x 5 m grid size was selected to allow the features of the channels and floodplain to be represented with sufficient accuracy, while maintaining efficient model run times. The 5 m x 5 m grid size is considered adequate to represent floodplain topography and is consistent with the approach typically adopted in similar flood studies. The 2D model topography was modified adjacent to the culverts so that the existing drainage flows, existing road/rail crowns and levee banks were represented accurately in the model. For the Developed Case, the topography was modified to include the Project rail embankment, drainage structures and associated infrastructure.

8.3.2.1 Sub-grid sampling and Quadtree mesh refinement

The Back Creek hydraulic model was established using TUFLOW's 2020-10-AD model version, including SGS and QT add-on modules. The below points provide a brief description of each module, along with the key parameters adopted for this assessment.

- Traditional grid-mesh based 2D hydraulic models sample the underlying DEM by taking a single elevation at the cell centres and cell faces. Sub-grid sampling allows for increased resolution by sampling a greater number of points of the underlying DEM to establish a water surface elevation vs volume relationship for each grid, along with a water surface elevation vs width relationship (wetted perimeter) at cell sides. The adoption of SGS allows increased representation of storage and conveyance between cells. A SGS distance of 1 m was adopted for this assessment.

- Quadtree mesh is constructed by subdividing the base mesh-grid size into four smaller square cells. Cells can be further sub-divided into four, and so on, allowing for additional resolution in location of interest. The base hydraulic model grid cell size (5 m x 5 m) was revised to include additional spatial resolution of approximately 2.5 m, to better represent model velocity at the entry and exit of culverts. Quadtree meshes were applied in the model covering minimum 25 m upstream and 100 m downstream of the design culvert locations, and culverts within the permanent footprint.

8.3.3 Roughness

The hydraulic roughness generally reflects the types of development and ground cover that exists within the hydraulic model extent. The distribution of roughness categories adopted for this assessment were based on the information supplied in the land-use layer from Queensland Globe database and aerial imagery, and confirmed during the site inspection.

The Back Creek hydraulic model adopts the calibrated losses in the Condamine River regional model as reproduced in Table 8.14.

Table 8.14 Back Creek catchment—Adopted roughness values

Category	Manning's 'n' value
River	0.035
Sealed roads	0.015
Unsealed roads	0.025
Rural residential	0.09
Urban residential	0.10
Commercial	0.06
Open space—Minimum vegetation	0.045
Open space—Moderate vegetation	0.055
Open space—Heavy vegetation	0.065
Open water	0.025
Rail	0.025
Riparian	0.04
Crops ¹	0.1085

Table note:

- 1 A uniform value was adopted for the crop areas, due to the seasonal and spatial variations in vegetation distribution and the uncertainty this brings to the design event approach

8.3.4 Hydraulic structures

There are no details of existing hydraulic structures situated within the extents of the Back Creek hydraulic model. A further review of existing structures will be completed at the detailed design stage.

8.3.5 Boundary conditions

The following boundary conditions were applied within the hydraulic model:

- Flow versus time model boundaries were applied for all inflow hydrographs, with local and total hydrographs sourced from the hydrologic models
- Delineated sub-catchments in the hydrologic model were used as internal flow boundaries within the hydraulic model extent (SA polygons). The distributing inflow along known conveyance paths allows for the consideration of structural responses to local inflows without having to re-split the catchment at the alignment.

- A height-discharge boundary (HQ) was applied for the downstream outflow model boundary. The downstream boundary is located at approximately 10 km downstream of the Back Creek and Grasstree Creek confluence past Lemontree, which was deemed sufficient to eliminate any impact of water level uncertainties at the boundary on modelling results at the Project alignment and FSR affected by Back Creek flooding.

8.3.6 Model calibration

Model calibration could not be undertaken for the Back Creek hydraulic model in absence of streamflow gauges on Back Creek.

8.3.7 Model validation

As per Expert Flood Panel recommendations, the original Back Creek URBS and TUFLOW models adopted for the draft EIS were extended a further 8 km and 15 km downstream, respectively into the Condamine River floodplain. The model extension overlaps with 12 flood markers of the December 2010 flood event, of which six were surveyed by FFJV and the other six were supplied via the Expert Flood Panel. These flood markers, shown in Figure 8.1, are located between 4 and 9 km downstream of the Project alignment and between 1 and 5 km upstream of the flood extents of the regional Condamine River flood; as such, they would be solely influenced by local flooding in Back Creek during the 2010 event. An analysis using these extended models has been completed below on these flood markers.

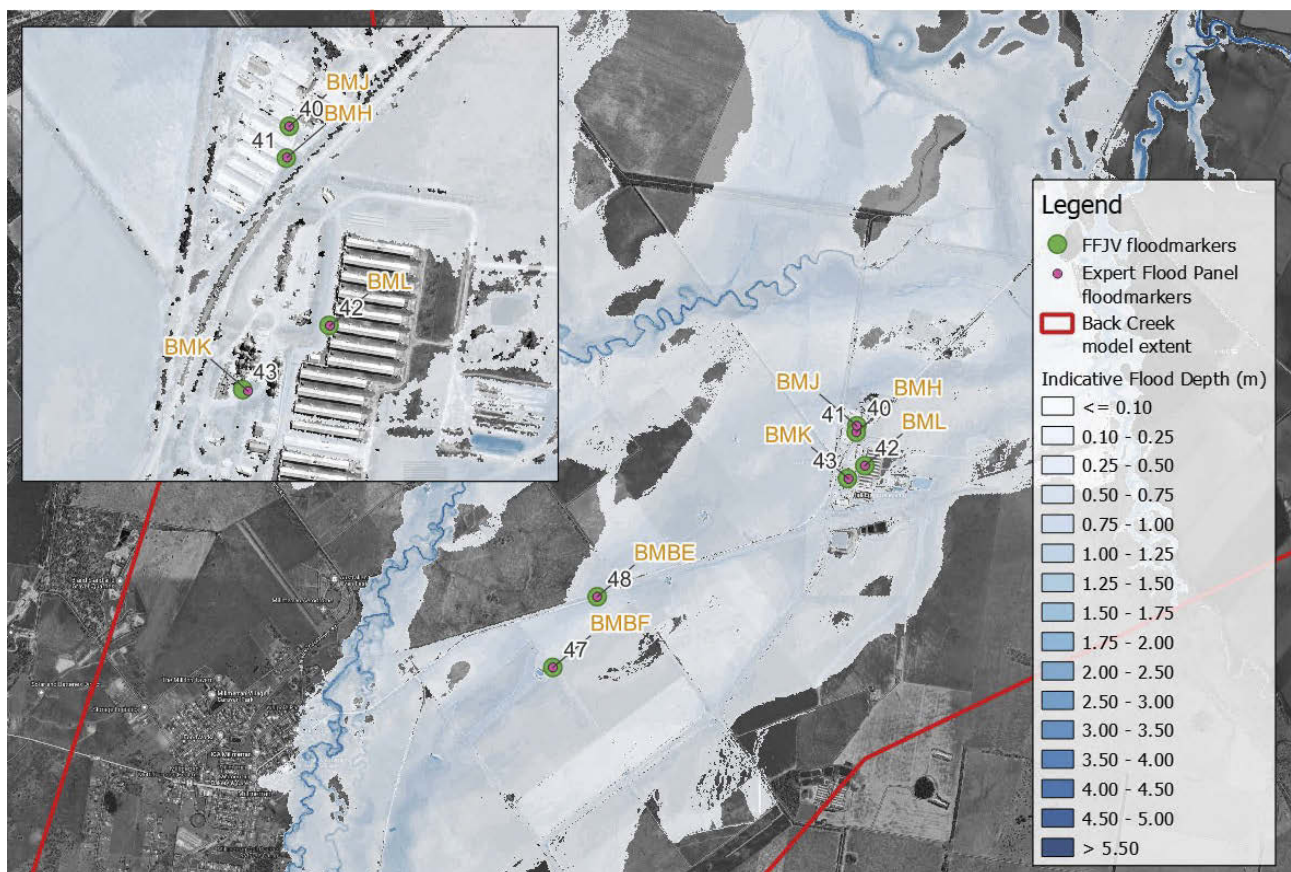


Figure 8.1 Flood markers within Back Creek catchment in 2010 flood event

In the absence of rainfall gauges within the Back Creek catchment, daily rainfall totals for the 2010 flood event in Back Creek were derived from gridded rainfall data available from Queensland Government's Scientific Information for Landowners platform. Sub-daily rainfall definition from the closest pluviograph at Leyburn Post Office (GS: 41063) was adopted for the Back Creek catchment. A comparison of daily rainfall totals between gauge and gridded rainfall data sources is summarised in Table 8.15. Discrepancies between data sources are unavoidable since the rain gauge data is taken at a point source compared to Scientific Information for Landowners data, which is aggregated data across a 5 km x 5 km grid. Nevertheless, there is a strong correlation in daily totals for the event.

Table 8.15 Back Creek catchment—December 2010 flood event cumulative rainfall totals

Event time	Leyburn Station (GS: 41063)	Scientific Information for Landowners gridded rainfall data
26/12/2010 9:00 am	4.8 mm	6.8 mm
27/12/2010 9:00 am	64.4 mm	68.3 mm
28/12/2010 9:00 am	109.6 mm	118.2 mm

In the absence of available stream-gauge data to inform flood hydrographs and event losses, the 2010 flood event hydrograph was produced using the design URBS model based on the recommended ARR Data Hub initial and continuing losses (34 mm and 0.7 mm/hr). Figure 8.2 shows the modelled 2010 flood event hydrograph, predicting an event peak at 331 m³/s at just past midnight on the 27 December 2010. The modelled hydrograph is considered a good approximation of the actual 2010 event, given that the predicted peak is comparable to the 1% AEP QRT estimate at the proposed rail crossing (Section 8.2.4).

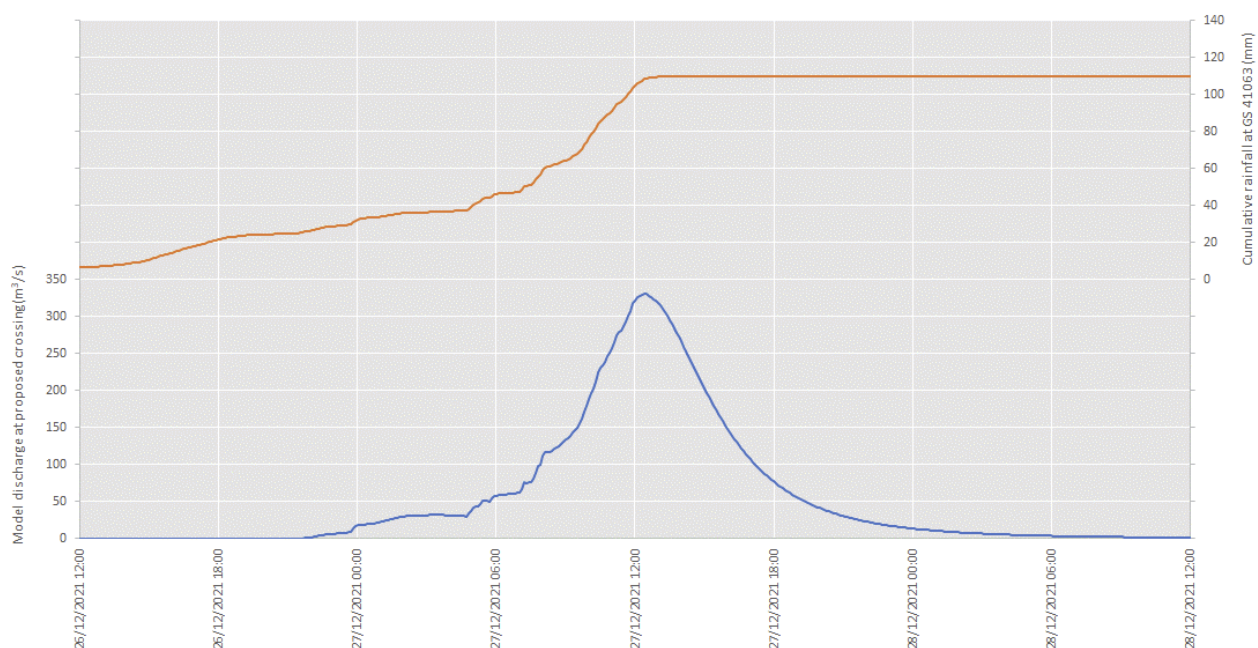


Figure 8.2 Back Creek—December 2010 flood event

Table 8.16 presents a comparison of the modelled flood levels and surveyed flood markers in the 2010 event. In accordance with industry best practice, flood markers matched to within ± 200 mm in model validation are considered to be a good match, with matches within ± 300 mm considered to be an acceptable match. These targeted tolerances were developed to account for typical topographic and modelling uncertainties, and had previously been adopted for other large infrastructure projects such as the Yeppen floodplain crossing and industry benchmark flood studies such as the Brisbane River Flood Study.

The model reproduced the 2010 event with a good or acceptable match achieved at all 12 flood markers considered. The modelled peak flood levels are, on average, 41 mm higher at the FFJV flood markers, and 20 mm higher at the additional flood markers provided by the Expert Flood Panel. The 2010 peak flood level reproductions are considered excellent, in that the average error to both sets of flood markers is well within the typical vertical error of airborne topographic surveys (i.e. ± 100 mm). Noting that the FFJV and Expert Flood Panel flood markers were likely duplicates of each other (Figure 8.1 and coordinates in Table 8.16), the difference in average errors (i.e. +41 mm at FFJV flood markers and +20 mm at Expert Flood Panel flood markers) is believed to reflect a systematic bias due to independent surveys undertaken.

The results in Table 8.16 further show that Back Creek flood peaked between approximately 3:00 pm and 5:00 pm on the 27 December 2010, which correlates well to the photographs of the event supplied by local residents, indicating that the flood in Back Creek peaked mid-to-late afternoon on the 27 December 2010, the day before the regional Condamine River flood peaked.

Table 8.16 Back Creek— Flood model validation to 2010 event flood markers

Flood marker ID	Easting	Northing	2010 peak water level (mAHD)	Modelled peak water level (mAHD)	Difference in peak water level (mm)	Classification	Time of modelled peak water level	Comments
40	335486	6918651	384.088	384.021	-67	Good	27/12/2010 17:15	Suspect duplicate of BMJ
41	335481	6918589	384.238	384.132	-106	Good	27/12/2010 17:15	Suspect duplicate of BMH
42	335569	6918265	384.812	384.854	42	Good	27/12/2010 17:10	Suspect duplicate of BML
43	335402	6918138	385.425	385.692	267	Acceptable	27/12/2010 17:05	Suspect duplicate of BMK
47	332543	6916262	393.9414	394.001	60	Good	27/12/2010 14:50	Duplicate of BMBF
48	332968	6916955	392.07	392.121	51	Good	27/12/2010 15:00	Duplicate of BMBE
BMBE	332968	6916955	392.07	392.121	51	Good	27/12/2010 15:00	Duplicate of #48
BMBF	332543	6916262	393.927	394.001	74	Good	27/12/2010 14:50	Duplicate of #47
BMH	335483	6918590	384.283	384.132	-151	Good	27/12/2010 17:15	Suspect duplicate of #41
BMJ	335486	6918651	384.106	384.021	-85	Good	27/12/2010 17:15	Suspect duplicate of #40
BMK	335412	6918135	385.467	385.677	210	Acceptable	27/12/2010 17:00	Suspect duplicate of #43
BML	335571	6918265	384.835	384.854	19	Good	27/12/2010 17:10	Suspect duplicate of #42

8.4 Existing Case modelling results—Back Creek

8.4.1 Hydrologic model peak flow assessment

A critical duration assessment was undertaken to determine which storm duration(s) produced peak flood flows where the Back Creek intersects the Project alignment (310-BR38).

To assess the critical storm duration, the following methodology was adopted:

- The critical durations were modelled for a range of AEP events:
 - Each AEP was modelled for a range of durations
 - Each duration was modelled for each of the 10 associated TPs
- A critical duration assessment was undertaken at the locations mentioned above to determine which duration produced the highest median flow of the 10 TPs for each event.

The critical duration and adopted TP for each design event AEP is provided in Table 8.17. Table 8.17 outlines the estimated peak flow for the 1% AEP event based on the adopted critical TP for a number of key locations (Figure D1-D in Volume 2 – Appendix D).

Table 8.17 Back Creek—Peak flow at key locations as applied in the hydraulic model

AEP (%)	Peak flow (m ³ /s)	Critical storm
20	68	48 hour – Pattern 4
10	118	48 hour – Pattern 7
5	187	48 hour – Pattern 0
2	255	9 hour – Pattern 7
1	301	9 hour – Pattern 9
1 in 2,000	485	4.5 hour – Pattern 9
1 in 10,000	841	3 hour – PMP TP Generalised Short Duration Method (GSDM)
PMF	3,648	4 hour – PMP TP (GSDM)

Table 8.18 Back Creek—1% AEP event peak inflow at key locations as applied in the hydraulic model

Sub-catchment name	1% AEP event peak flow (m ³ /s)	Critical storm (hrs) duration and temporal pattern
06T	215.2	9 hour – Pattern 9
07L	50.8	9 hour – Pattern 9
11L	29.8	9 hour – Pattern 9
10L	8.1	9 hour – Pattern 9
08L	68.4	9 hour – Pattern 9
09La	37.9	9 hour – Pattern 9
09Lb	4.4	9 hour – Pattern 9
12L	34.6	9 hour – Pattern 9
12T	340.5	9 hour – Pattern 9
14L	13.0	9 hour – Pattern 9
13L	23.1	9 hour – Pattern 9
15L	21.6	9 hour – Pattern 9
15T	346.4	9 hour – Pattern 9
16L	34.0	9 hour – Pattern 9

Sub-catchment name	1% AEP event peak flow (m ³ /s)	Critical storm (hrs) duration and temporal pattern
17L	22.4	9 hour – Pattern 9
18L	17.3	9 hour – Pattern 9
19L	39.3	9 hour – Pattern 9
20L	18.4	9 hour – Pattern 9
21L	24.3	9 hour – Pattern 9
22L	17.9	9 hour – Pattern 9
23L	18.8	9 hour – Pattern 9
24L	24.5	9 hour – Pattern 9
25L	9.9	9 hour – Pattern 9
26L	24.6	9 hour – Pattern 9
27L	8.3	9 hour – Pattern 9
28L	7.9	9 hour – Pattern 9
29L	9.8	9 hour – Pattern 9

8.4.2 Existing Case flood maps

Existing conditions flood maps are presented in Volume 2 – Appendix D and via an [Online Digital Portal](#). Appendix D contains the Existing Case 20% and 1% AEP flood maps, whilst the [Online Digital Portal](#) contains the remainder of the modelled events. The location of each map set is summarised below:

- 20% AEP peak levels: Figure D2-A
- 1% AEP peak levels: Figure D2-B
- 20% AEP peak velocity: Figure D2-C
- 1% AEP peak velocity: Figure D2-D
- 20% AEP peak hazard: Figure D2-E
- 1% AEP peak hazard: Figure D2-F
- 20% AEP peak time of inundation: Figure D2-G
- 1% AEP peak time of inundation: Figure D2-H
- 20% AEP, 10% AEP, 5% AEP, 2% AEP, 1% AEP, 1 in 2,000 AEP, 1 in 10,000 AEP and PMF Existing Case results: [Online Digital Portal](#)
- For higher resolution mapping refer to the [Online Digital Portal](#).

8.4.3 Existing Case flood inundation extent and flood levels

The Back Creek floodplain is general characterised by the main channel and secondary tributaries joining upstream of the Millmerran township.

During minor events (20% AEP) there are two breakout areas located upstream and downstream of Millmerran-Inglewood Road. For larger events (1% AEP) the north-eastern part of Millmerran is flooded, along with overtopping of the Gore Highway and Millmerran-Inglewood Road.

In the Existing Case 1% AEP event, the flood extent within in the vicinity of the Project alignment is approximately 300 m, before widening to the east near Millmerran (approximately 2.3 km wide) at the breakout locations. The peak 1% AEP event depth at the Project alignment is approximately 4.6 m.

Figure D2-B in Volume 2 – Appendix D shows the 1% AEP indicative flood extent and peak water levels within the Back Creek floodplain.

8.4.4 Flood immunity of existing infrastructure

Table 8.19 presents a summary of overtopping depths for key roads near the Project alignment under a range of design events. Modelling results show that Millmerran-Inglewood Road has an existing low flood immunity in the areas close to the Project alignment.

Table 8.19 Back Creek—Existing Case: Overtopping depths of key infrastructure

Infrastructure location	Location	Overtopping depth (m)							
		PMF	1 in 10,000 AEP	1 in 2,000 AEP	1% AEP	2% AEP	5% AEP	10% AEP	20% AEP
Millmerran-Inglewood Road	Approximately 1,400 m D/S of proposed alignment	2.35	1.50	1.24	0.98	0.95	0.85	0.71	0.57
Gore Highway	Approximately 3 km D/S of proposed alignment	1.93	1.18	0.92	0.62	0.57	0.48	1.88	0.30

8.4.5 Existing Case velocities

In the Existing Case 1% AEP event, the model predicts typical velocities between 1 m/s and 2 m/s in the Back Creek channel. The average velocity across the floodplain is approximately 0.6 m/s. Typical values remain below 0.5 m/s in shallower areas of the floodplain, and from 0.8 m/s to 1 m/s in minor flow channels (minor channels).

Figure D2-D in Volume 2 – Appendix D shows the 1% AEP peak flood velocities within the Back Creek floodplain for the Existing Case.

8.4.6 Existing Case flood hazard

In the Existing Case 1% AEP event, high flood hazard zones in excess of 2 m²/s occur within the main Back Creek channel and secondary tributaries. Peak hazard values in the vicinity of the Project exceed 4 m²/s. Flood hazard values outside of the main channels are typically less than 0.5 m²/s.

Figure D2-F in Volume 2 – Appendix D shows the 1% AEP peak flood hazard within the Back Creek floodplain for the Existing Case.

8.4.7 Existing Case flood duration of inundation

For the Existing Case 1% AEP event, modelling demonstrates that the majority of the Back Creek floodplain is inundated for greater than 10 hours. Time of inundation along the floodplain fringes (low flow areas) are typically inundated for less than 5 hours.

Figure D2-H in Volume 2 – Appendix D shows the 1% AEP flood duration of inundation within the Back Creek floodplain for the Existing Case.

8.5 Developed Case modelling results—Back Creek

8.5.1 Drainage structures

The hydraulic design of the major drainage structures was undertaken using the TUFLOW hydraulic model (1D and 2D approach).

The Back Creek hydraulic model includes the following regional cross-drainage structures (or regional structures):

- Two waterway bridges
- Two RCP locations (a total of 24 cells).

Design bridge details are summarised in Table 8.20, and details of the proposed local and regional culvert structures are summarised in the Cross-Drainage Register located in Appendix B. The location of proposed cross-drainage features is presented in Figure D1-F in Volume 2 – Appendix D.

Local drainage structures are presented in Appendix B. These structures were sized through the local drainage design and, depending on their interaction with the Gowrie Creek floodplain, were incorporated in the hydraulic model.

The location of proposed drainage structures is shown in Figure D1-F in Volume 2 – Appendix D.

Table 8.20 Back Creek—Proposed bridge location and details

Chainage (km)	Structure ID	Approximate span (m)	Deck width (m)	Soffit level (mAHD)	Deck superstructure type	Deck depth (mm)
126.97	310-BR37	167	3.97	417.36	Type D1	2,000
128.06	310-BR38	230	3.97	407.41	Type D1	2,000

All bridges were modelled as an opening in the rail embankment. The optimisation of bridge lengths was balanced between minimising the changes to the hydraulic regime, primarily afflux and velocities, and the cost of replacing bridge spans by large earth embankments. As agreed with the Expert Flood Panel, bridge locations will be updated in detailed design to be modelled utilising LFC shapefiles.

Blockage of hydraulic structures was assessed in accordance with ARR 2019. Despite the catchment being vegetated, ARR guidelines determined that the likelihood of significant amounts of debris accumulating against the piers of the bridge is low; therefore a zero blockage factor has been applied to bridges.

8.5.2 Hydraulic design criteria outcomes

The hydraulic model was run for the Developed Case for the range of events (20%, 10%, 5%, 2%, 1% AEP events and 1 in 2,000 AEP, 1 in 10,000 AEP and PMF events) with the outcomes presented in the following sections.

8.5.2.1 Rail immunity and structures results

Results of the Back Creek model indicate the freeboard to formation design requirement has been achieved for the 1% AEP event. There is over 2.3 m freeboard to the soffit level of the bridges, noting that the formation levels were generally dictated by other engineering criteria and constraints.

Appendix B summarises the peak 1% AEP water levels on the upstream side of the proposed alignment.

8.5.3 Flood Impact Objectives outcomes—Back Creek

The impact of the Developed Case was assessed through inclusion of the proposed rail design and comparison of model results against the Existing Case results.

Changes to flooding behaviour that were assessed, and are reported in the following sections, include potential changes in peak water levels, time of inundation, velocity, hazard, and peak flows.

- Changes in 20% AEP and 1% AEP peak water levels are presented in Figures D4-A and D4-B in Volume 2 – Appendix D
- Changes in 20% AEP and 1% AEP velocities are presented in Figures D4-C and D4-D in Volume 2 – Appendix D
- Changes in 20% AEP and 1% AEP hazard are presented in Figures D4-E and D4-F in Volume 2 – Appendix D
- Changes in 20% AEP and 1% AEP time of inundation are presented in Figures D4-G and D4-H in Volume 2 – Appendix D
- Changes in 20% AEP and 1% AEP velocities and percentage change in velocities with cut-offs applied are presented in Figures D5-A to D5-F in Volume 2 – Appendix D
- Percentage changes in 20% AEP and 1% AEP in hazard are presented in Figures D6-A and D6-B in Volume 2 – Appendix D
- Percentage changes in 20% AEP and 1% AEP in time of inundation and with cut-offs applied are presented in Figures D7-A and D7-D in Volume 2 – Appendix D
- Refer to the [Online Digital Portal](#) for other modelled AEP results and higher resolution mapping.

One-on-one consultation with landowners who are expected to experience changes in flooding behaviour on their property was conducted by ARTC supported by FFJV. Refer to EIS Chapter 14: Flooding and Geomorphology and EIS Appendix E: Consultation Report for further details.

The Project design outcomes relative to the FIOs (Table 4.2) are presented in the following sections.

Flood sensitive receptors referred to in this section are shown in Figures P1 (1 to 59) in Volume 2 – Appendix P.

8.5.3.1 Flood impacts on flood sensitive receptors

Based on the available aerial imagery, no buildings or critical infrastructure are located within the area impacted by afflux and change in flood hazard in the Back Creek floodplain for all modelled events up to the 1 in 2,000 AEP event.

8.5.3.2 Flood impacts on State-controlled roads

The extent of the hydraulic model developed for Back Creek is shown in Figure 8.3. Within the extents there are three State-controlled roads that are influenced by the Project including, Millmerran-Inglewood Road, Charles Street, and the Gore Highway. Table 8.21 shows seven locations of potential FIO exceedances. The details for each type of exceedance including afflux, velocity, time of inundation and hazard exceedances is included within Table 8.21 with justification and proposed mitigation measures also detailed.

- Millmerran-Inglewood Road has low immunity, with the Existing Case model results indicating that sections of the road are overtopped in a 20% AEP event. During design conditions, three locations (road section IDs #1, #3, and #8) were identified as having increase in afflux, with ID #3 also showing increases in the duration of inundation, velocity, and hazard. Peak impacts across Millmerran-Inglewood Road at ID #3 show an increase in water level by 190 mm, flood inundation duration of 490 per cent (increasing from 0.26 hours to 1.53 hours), velocity increase of 45 per cent and 0.67 m/s, and a flood hazard increase of 151 per cent (increasing from 0.07 m²/s to 0.16 m²/s). IDs #1 and #8 have afflux increases of approximately 39 mm and 62 mm respectively. The extent of impact is also relatively minor with a maximum impact length of 7.5 m (ID #1) and 12.5 m (ID #8).

Due to the rail design constraints, particularly at ID #3, ongoing model strengthening, including additional survey and cross-drainage design, will be investigated at the detailed design stage.

- Existing Case model results show the Gore Highway is overtopped in numerous locations during the 20% AEP flood event. For design conditions, three separate locations were identified as showing exceedances. IDs #233 and #520 show afflux and change of time of inundation. The afflux exceedance was 26 mm (ID #233) and 23 mm (ID #520), and the maximum time of inundation exceedance at both locations was 167 per cent (increasing from 0.15 hours to 0.41 hours) and 25 per cent (increasing from 0.23 hours to 0.26 hours), respectively. ID #232 recorded a maximum increase in time of inundation of 33 per cent (increasing from 0.38 hours to 0.5 hours). In all instances the length of exceedance along the Gore Highway is minor, with the largest exceedance registering as 10 m.

The Gore Highway impacts are minor—additional survey will be explored as a potential mitigation solution at exceedance locations.

- Charles Street at road section ID #111 shows an increase in flood inundation duration of approximately 19 per cent (increasing from 1.51 hours to 1.80 hours). This impact is at the limit of model accuracy with a total length of exceedance recorded as 2.5 m (half the model cell size).

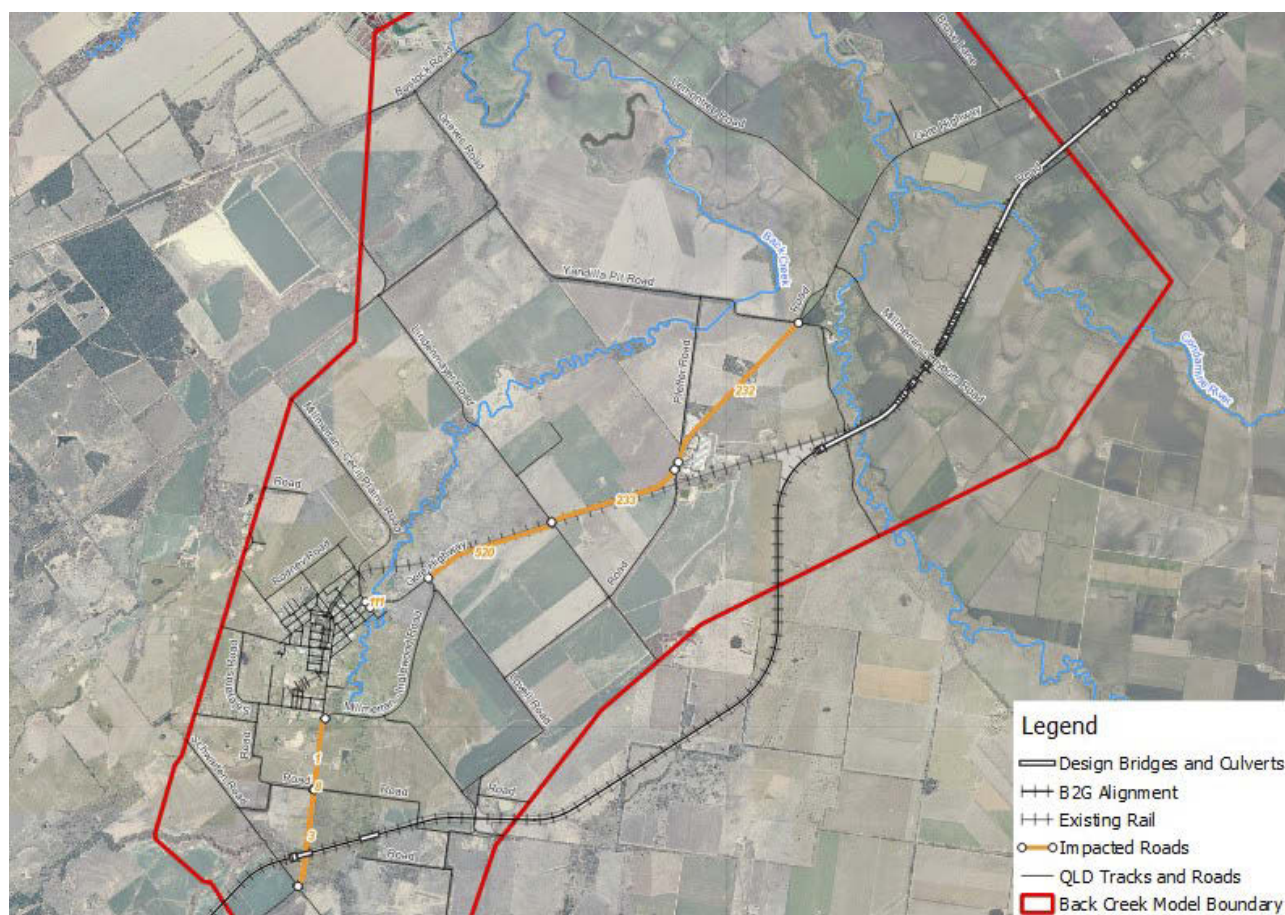


Figure 8.3 Back Creek—Potentially impacted State-controlled roads

Table 8.21 Back Creek - Summary of impacts on State-controlled roads

[illegible]

Table Notes

1. Time of Submergence has been measured across the full length of the road segment and does not correspond to the exact location of exceedance within that segment
2. Change in water level (afflux) is calculated based on the maximum water surface level difference between the existing and design case results, and does not consider the underlying topography. Depth values presented are based on the respective underlying topographies and in some cases do not align with afflux calculations. This can be attributed to interpolation calculations of results performed within the hydraulic model.
3. Percentage increases should be read in conjunction with existing and design case results. The FfOs require that change be presented in percentage change for time of inundation, velocity and hazard and can sometimes produce large increases when existing case results are significantly small. Some locations were identified to experience significant percentage increases however the "absolute" difference between Existing and Design results are relatively small
4. The above table should be read in conjunction with change maps provided as part of Volume II and the Digital Platform.
5. The symbol "<" denotes that no change above Flood Impact Objectives has been determined and therefore no result has been presented.
6. Changes recorded above may only occur under a small fraction of model simulation time.
7. Three decimal places do not infer absolute accuracy in modelling results, but it is necessary to identify small changes.
8. Design optimizations that could be implemented (but not limited to) include adjustment of culvert spacing, realignment of culverts to existing footpaths when more detailed survey is available, modifying the grade of culverts, road height and alignment adjustments, removal of bridge abutments from waterway extents etc

8.5.3.3 Flood impacts on local public roads

Table 8.22 presents a list of local public roads within the Back Creek hydraulic model extent that have potential FIO exceedances due to the permanent footprint, with justification and mitigation measures identified for each. The impacted road sections are shown in Figure 8.4.

A summary of key impacts, exceedance justification and mitigation measures is documented below, with additional detail provided in Table 8.22:

- A total of 11 local roads show potential FIO exceedances during design conditions. The majority of exceedances are due to changes in peak water levels (afflux) and increases in time of inundation. Sporadic increases in velocity and hazard are noted for a number of roads.
- Larger afflux exceedances occur nearer the permanent footprint in road section IDs # 2 (Unidentified Road), #4 (Unidentified Road) and #5 (Schwarten Road). The maximum increase at ID #2 occurs during the 5% AEP event, with a corresponding water level increase of 184 mm. Afflux at IDs #4 and #5 do not exceed 100 mm. This is consistent with other afflux exceedances for the remaining roads.
- Duration of inundation changes are reported for all locations, excluding ID #4 (Unidentified Road). The maximum percentage change occurs at ID #5 (Schwarten Road), with an approximate flood inundation increase of 390 per cent (increasing from 0.74 hours to 3.64 hours). The unidentified road at ID #2 and Hall Road at Location 675 have exceedances greater than 100 per cent. The impacts are likely caused by local flow changes around the proposed road designs and cross-drainage features (i.e. culverts and bridges). Roads further away from the alignment show smaller exceedances in flood duration with an average increase of 30 per cent.
- Changes in peak velocity were noted for an Unidentified Road at ID #2, Schwarten Road at ID #5, Hall Road at ID #521, and Pfeffer Road at ID #58. The maximum increase of 0.6 m/s occurred at Schwarten Road during the 5% AEP event. Velocity exceedances in this location are a result constricted flow through the proposed bridge location. There were no recorded changes exceeding 1 m/s across the impacted roads.
- Flood hazard exceedance were recorded along Schwarten Road (ID #5). The maximum exceedance was recorded for the 1% AEP design flood, with an increase of approximately 22.3 per cent.
- Changes in peak flood conditions are likely the result of channelised flood water through cross-drainage infrastructure and design alignment limitations (i.e. displacement and redirection of flood water). Further cross-drainage and bridge design investigation, along with ongoing model strengthening are required during detailed design.
- Specifically, for Schwarten Road (ID #5) model improvements such as inclusion of proposed local cross-drainage are proposed as mitigation for exceedances along this segment to reduce flow being directed along the upstream toe of the proposed embankment. It is expected a reduction in exceedance can be obtained through this.

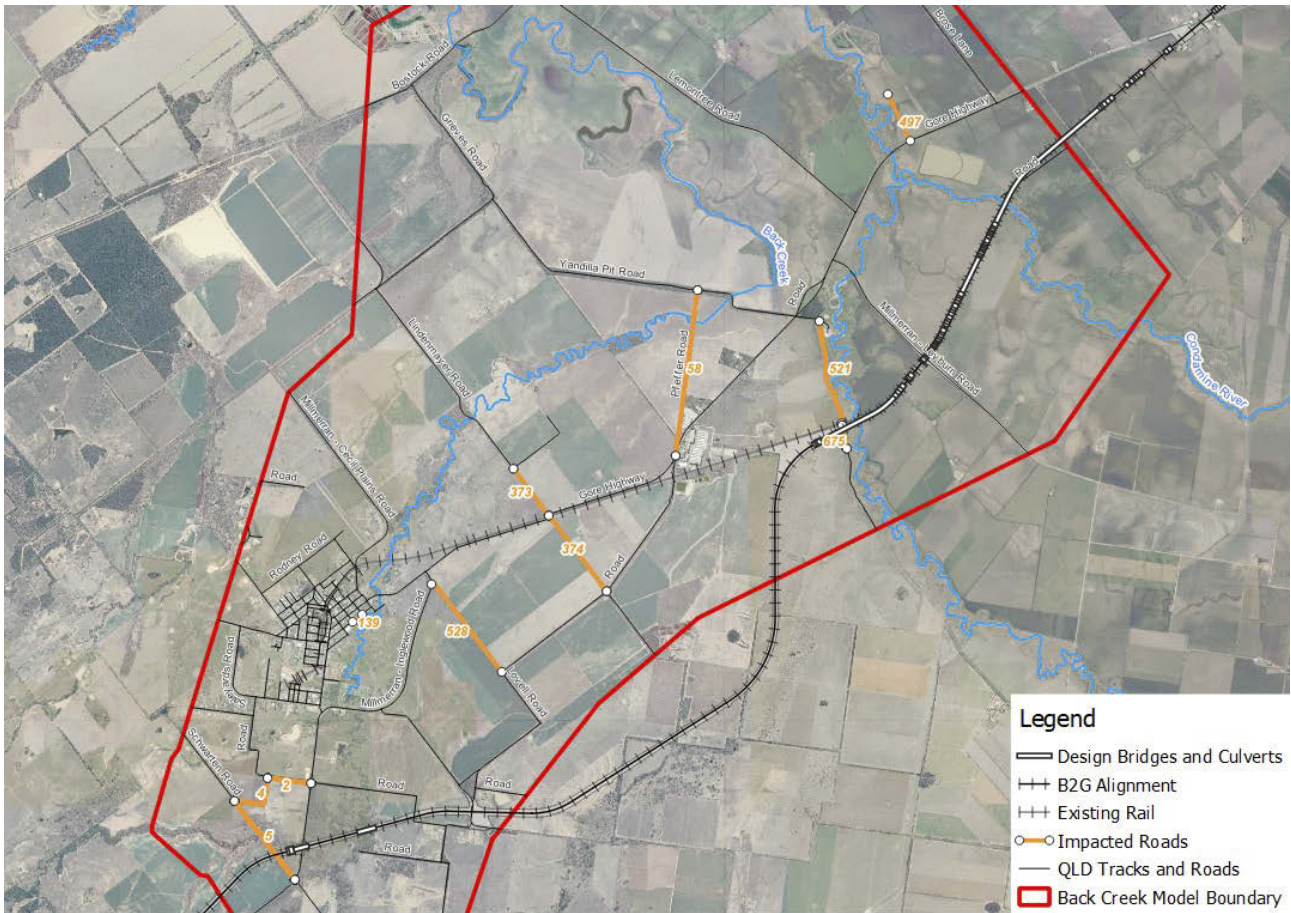


Figure 8.4 Back Creek—Potentially impacted local public roads

Table 8.22 Back Creek - Summary of impacts on local public roads

[illegible]

Table Notes

1. Time of Submergence has been measured across the full length of the road segment and does not correspond to the exact location of exceedance within that segment.
2. Change in water level (afflux) is calculated based on the maximum water surface level difference between the existing and design case results, and does not consider the underlying topography. Depth values presented are based on the respective underlying topographies and in some cases do not align with afflux calculations. This can be attributed to interpolation calculations of results performed within the hydraulic model.
3. Percentage increases should be read in conjunction with existing and design case results. The FIOs require that change be presented in percentage change for time of inundation, velocity and hazard and can sometimes produce large increases when existing case results are significantly small. Some locations were identified to experience significant percentage increases however the "absolute" difference between Existing and Design results are relatively small.
4. The above table should be read in conjunction with change maps provided as part of Volume II and the Digital Platform.
5. The symbol "-" denotes that no change above Flood Impact Objectives has been determined and therefore no result has been presented.
6. Changes recorded above may only occur during a small fraction of model simulation time.
7. Three decimal places do not infer absolute accuracy in modelling results, but it is necessary to identify small changes.
8. Design optimisations that could be implemented (but not limited to) include adjustment of culvert spacing, realignment of culverts to existing footpaths when more detailed survey is available, modifying the grade of culverts, road height and alignment adjustments, removal of bridge abutments from waterway extents etc.

Table 8.23 presents a list of rail infrastructure with the Back Creek hydraulic model extents that have potential FIO exceedances due to the permanent footprint along with exceedance justification and mitigation measures. The impacted rail sections are shown in Figure 8.5.

- Rail section ID #3384 shows potential FIO exceedances for afflux, duration of inundation, and velocity. Afflux exceedances range from 694 mm for the 20% AEP and progressively increase to 773 mm for the 1% AEP event. Duration increases are noted for the 5% AEP with an increase of 20.9 per cent (from 0.52 hours to 0.63 hours). Velocity changes range from 0.65 m/s to 0.69 m/s for the 10% and 5% AEP events, respectively.

- Rail section ID #12035 has a minor increase in the time of inundation. The increase is approximately 11 per cent (0.15 hours) but occurs over a single grid cell (i.e. 2.5 m).



Table 8.23 Back Creek - Summary of flood impacts on existing rail infrastructure

Route Name	Section Name	Status	Flood Impact Objective Target					AEP	Maximum FIO Exceedance					Length (m)				Exceedance Justification	Proposed Mitigation Measures ²			Depth (m) ²		Time of Inundation in hours (related to % exceedance)		Time of Submergence (hours) ²		Velocity in m/s (related to % exceedance)		Velocity in m/s (related to actual change exceedance)		Hazard in m ² /s (related to % exceedance)			
			Change in Peak Water Level (mm)	Change in Time of Inundation (%)	Change in Velocity (%)	Change in Velocity (m/s)	Change in Hazard (%)		Change in Peak Water Level (mm)	Change in Time of Inundation (%) ²	Change in Velocity (%) ¹	Change in Velocity (m/s)	Change in Hazard (%) ²	Afflux	Time on Inundation	Velocity	Hazard					Existing Case	Design Case	Existing	Design	Existing	Design	Existing	Design	Existing	Design	Existing	Design	Existing	Design
3384	Millmeran Branch	Abandoned	10	10	10	0.5	10	20%	694	-	-	-	-	60.00	-	-	-	> Maturity of current rail embankment design is limited by available information (e.g. detailed survey). > The Flood Impact Objective exceedance has been identified due to a change in flood extent and a cell which was dry in the existing case and now flood-affected in the developed case.	> Further road and/or rail design optimization investigations are required during detailed design. > Stakeholder Consultation (Landowner/ Government).																
								10%	713	-	-	0.65	-	60.00	-	5.00	-																		
								5%	756	21	-	0.69	-	57.50	2.50	10.00	-																		
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12035	Millmeran Branch	Abandoned	10	10	10	0.5	10	20%	-	11	-	-	-	2.50	-	-	-	> Rail design constraints alter the existing flow regime.	> Stakeholder Consultation (Landowner/ Government).																
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								5%	-	-	-	-	-	-	-	-	-																		

8.5.3.5 Flood impacts on private land outside the permanent footprint

Table 8.24 presents the predicted changes in flood conditions associated with the permanent footprint on private land outside the permanent footprint. A summary of key impacts is provided in the below points, with exceedance justification and mitigation measure provided in Table 8.24:

- A total of 30 private lots were identified as having a potential impact that exceed the Project FIO targets. The property types vary from 'Grains', 'Cattle Breeding and Fattening', 'Cotton', 'Dwelling – Large House Site', 'Pigs', 'Reservoir (Dams, Bores)', 'Sheep Grazing-Dry', 'Subdivided Land – Discounted by LG', 'Vacant – Large House Site', and 'Vacant Rural Land (Excl 01& 04)'.
- Potential time of inundation was identified as the primary exceedance type for Back Creek. A total of 28 properties were recorded. Visual inspection of the results shows a number of properties have larger areas of exceedance upstream and downstream of bridge 310-BR37 and 310-BR38. The primary cause of these exceedances are due to the redirection and constriction of flood water through the structures, which in turn causes additional ponding upstream and prolonged flooding downstream. This occurrence is particularly evident at Lot Plan 2SP260880. Lot 2SP26088 has a flood increase in duration of approximately 12 hours.

Sporadic duration exceedance is also noted further downstream of the Back Creek Project crossings, approximately 1.5 km to +8 km (extending to the Condamine floodplain). These exceedances are located along the flood fringes, and shallower section of the flood extent. Duration exceedance of this nature, represent a small proportion of the total lot size, and are expected to be of minor consequence. The average increase in duration of flooding across the Back Creek floodplain for properties is approximately 1.4 hours.

- Afflux exceedances are noted for three Lots (2RP233820, 2SP260880, and 140DER34129). The maximum increase in peak water level was recorded at Lot Plan 2SP260880, with a value of approximately 320 mm. Lots showing increases in peak water level are located immediately upstream of both bridge locations.
- Velocity increases are most prominent nearer the cross-drainage structures. This is particularly evident for bridge 310-BR38. Velocity increases are noted along the upstream left embankment, and immediately downstream of the bridge opening. Peak velocity exceedances in this area range from <0.1 m/s to 1.39 m/s across all events. Note that the maximum increase has been identified due to a change in flood extent, and a previously dry cell is now reporting a velocity increase of 1.39 m/s on Lot Plan 2DY1006.

A number of properties with exceedances for velocity are also located much further downstream, these locations have a reduced area of impact and likely occur in shallower locations in the floodplain, thus having a lower consequence.

- Hazard impacts are isolated to properties upstream and downstream of bridges 310-BR37 and 310BR38. The maximum change in peak hazard is approximately 1,326 per cent (an approximate change of 0.14 m²/s) on Lot Plan 2SP260880.
- In most instances, exceedances are a result of the maturity of the cross-drainage and bridge design, and rail design constraints impacting the existing flow regime. In order to address these exceedances additional cross-drainage and bridge optimisation investigations are required.

Table 8.24 Back Creek- Summary of flood impacts on private land outside the permanent footprint

ID	ID (Lupinus ID)*SR ID (Culvert ID)	Sensitive Receptor Type	Catchment Type	Landuse ¹⁴	Lot Area (m2)	Flood Impact Objective Target						AEP	Maximum FO Exceedance					Area (ha) ¹⁵			Exceedance Justification	Proposed Mitigation Measures ¹⁶		
						Change in Peak Water Level (mm)	Change in Time of inundation (%)	Change in Time of inundation (hours)	Change in Velocity (%)	Change in Velocity (m/s)	Change in Hazard (%)		Change in Peak Water Level (mm)	Change in Time of inundation (% Change) ¹⁷	Change in Time of inundation (hours)	Change in Velocity (m/s) ¹⁸	Change in Velocity (m/s)	Change in Hazard (%) ¹⁹	Afflux	Time on inundation			Velocity	Hazard
1	3RP23820	Lot Type Parcel	Regional	GRAINS	859,792	200	10	10	1	10	0.5	20	20%	-	475	0.97	25	0.76	40	0.56	0.10	0.02	Maturity of current cross drainage design is limited by available information (e.g. detailed survey) Maturity of current bridge design is limited by available information (e.g. detailed survey) The Flood Impact Objective exceedance has been identified due to a change in flood extent and a cell which was dry in the existing case and now flood-affected in the developed case.	Further cross drainage optimization investigations are required during detailed design. Further road and/or rail design optimization investigations are required during detailed design.
													10%	-	238	1.72	23	0.88	30	0.23	0.04	-		
													5%	-	224	1.98	27	0.99	46	1.04	0.28	-		
													2%	-	193	0.91	43	0.88	167	1.13	0.33	-		
													0.05%	-	238	2.83	39	1.03	183	1.08	0.36	-		
2	14XDER34129	Lot Type Parcel	Regional	VACANT RURAL LAND (EXCL G1 & G4)	95,901	200	10	1	10	0.5	20	20%	-	464	6.13	-	0.22	109	0.32	0.09	0.13	Maturity of current bridge design is limited by available information (e.g. detailed survey) Further road and/or rail design optimization investigations are required during detailed design.	Further road and/or rail design optimization investigations are required during detailed design.	
												10%	-	90	1.29	43	0.43	105	0.43	0.84	0.62			
												5%	-	142	1.57	46	0.77	185	0.18	2.46	8.22			
												2%	-	99	1.35	60	0.32	154	0.32	3.31	5.01			
												0.05%	-	242	231	64	0.34	168	0.88	0.28	3.43			5.29
3	37UDER34129	Lot Type Parcel	Regional	GRAINS	608,550	200	10	1	10	0.5	20	20%	-	242	-	-	0.19	-	0.17	0.003	-	Maturity of current bridge design is limited by available information (e.g. detailed survey) Further road and/or rail design optimization investigations are required during detailed design.	Further road and/or rail design optimization investigations are required during detailed design.	
												10%	-	180	1.92	35	-	35	0.39	0.01	0.02			
												5%	-	240	1.67	28	0.03	86	0.14	0.02	0.21			
												2%	-	138	1.80	25	0.10	208	0.88	0.05	0.02			
												0.05%	-	287	2.63	25	0.12	279	0.70	-	0.71			
4	2DV1036	Lot Type Parcel	Regional	GRAINS	2,204,365	200	10	1	10	0.5	20	20%	-	566	-	25	1.30	0.33	0.01	-	Maturity of current cross drainage design is limited by available information (e.g. detailed survey) Maturity of current bridge design is limited by available information (e.g. detailed survey) The Flood Impact Objective exceedance has been identified due to a change in flood extent and a cell which was dry in the existing case and now flood-affected in the developed case.	Further cross drainage optimization investigations are required during detailed design. Further road and/or rail design optimization investigations are required during detailed design.		
												10%	-	840	3.90	14	1.39	0.45	0.02	-			-	
												5%	-	1,005	13.49	10	0.59	21	0.66	0.01			0.003	
												2%	-	1,047	9.30	19	0.69	35	0.72	0.04			0.03	
												0.05%	-	689	7.74	21	0.70	35	0.71	0.04			0.03	
5	31XDER3484	Lot Type Parcel	Regional	GRAINS	305,171	200	10	1	10	0.5	20	20%	-	56	0.57	-	0.03	-	0.15	0.10	-	Maturity of current bridge design is limited by available information (e.g. detailed survey) Further road and/or rail design optimization investigations are required during detailed design.	Further road and/or rail design optimization investigations are required during detailed design.	
												10%	-	82	0.78	10	0.05	24	1.13	0.21	0.01			
												5%	-	49	1.21	14	0.08	38	1.07	0.26	0.13			
												2%	-	139	1.76	14	0.08	37	1.17	0.26	0.22			
												0.05%	-	-	-	-	-	-	-	-	-			
6	2SP26080	Lot Type Parcel	Regional	GRAINS	4,585,960	200	10	1	10	0.5	20	20%	-	289	2.90	14	0.66	-	1.32	0.06	-	Exceedances experienced on this Lot are due to the embankment impeding flow towards the north. In the existing case, an overland (steep) valley (to the west of proposed bridge structure 310-BP38) which is blocked under design conditions and is redirected towards proposed cross drainage structures. This results in upstream ponding and time of inundation increases on this Lot. Flow is directed towards bridge 310-BP38 in the design case where it enters the main Back Creek Channel towards the Condamine River. In addition, an increase in time of inundation also occurs to the north west corner of this Lot due to the Schwarten Road diversion. Increased cross flow due to the representation of road drainage within the hydraulic model, with further model refinements to be undertaken in detailed design. The exceedance remains due to the current design being at a reference design stage and optimization of bridge length and cross drainage structures under the alignment not being fully completed. Further iteration of cross drainage design and placement needs to be undertaken at detailed design stage. It is anticipated that a reduction in FPO exceedance can occur through mitigation, although some areas of exceedance may remain due to the presence of the project.	It is anticipated exceedances on this Lot can be reduced through a review and optimization of the Bridge 310-BP38 and cross drainage improvements through the embankment within the vicinity. Mitigation could include increasing the bridge opening of 310-BP38 or installation of relief culverts through the road embankment. In addition, the hydraulic model improvements will be undertaken in the detailed design stage to better represent road drainage around the Schwarten Road/Minnamang Road interface.	
												10%	-	395	4.30	13	0.58	-	3.04	0.40	-			
												5%	-	1,159	12.56	50	0.91	1326	3.50	1.78	1.65			
												2%	-	276	1.21	76	0.62	532	3.61	2.65	2.4			
												0.05%	-	320	189	2.01	79	1.31	489	1.30	2.60			2.90
7	2RP23820	Lot Type Parcel	Regional	DWELLING - LARGE HOUSESITE	18,777	10	5	1	10	0.5	10	20%	-	52	-	-	-	0.15	0.05	-	-	Exceedances on this Lot occur due to the proposed rail and road embankments constraining the existing flow paths and reducing the available storage lot downstream of the property. The bridge abutment encroaches into the existing flow path causing a reduction in the waterway area and directing the runoff towards the bridge opening. This results in an increase in water level (afflux), time of inundation and hazard upstream from the rail alignment. Due to the maturity of the current rail and bridge design being at a reference design stage, these exceedance remain as optimization of bridge length and abutment placement is still to be completed. Further review of the bridge and rail embankment design will be undertaken in the detailed design phase.	To mitigate these exceedances, a review and further optimization of the rail and road embankment design is required. The bridge abutment encroaches into the existing flow path causing a reduction in the waterway area and directing the runoff towards the bridge opening. This results in an increase in water level (afflux), time of inundation and hazard upstream from the rail alignment. Stakeholder Consultation (Landowner/Government) will be undertaken to discuss any residual impacts once the above mitigation measures have been implemented in the design.	
												10%	-	162	106	1.70	-	0.31	0.08	-	-			
												5%	-	165	88	5.39	21	0.30	0.19	0.04	-			
												2%	-	148	72	1.99	34	0.56	0.24	0.04	-			
												0.05%	-	129	224	3.82	29	0.55	0.20	-	-			
8	5RP16063	Lot Type Parcel	Regional	GRAINS	291,418	200	10	1	10	0.5	20	20%	-	234	2.17	-	0.04	-	0.63	0.09	-	Maturity of current cross drainage design is limited by available information (e.g. detailed survey) Further road and/or rail design optimization investigations are required during detailed design.	Further cross drainage optimization investigations are required during detailed design.	
												10%	-	86	0.91	1.02	-	0.01	-	-	-			
												5%	-	39	-	-	0.03	-	0.51	0.003	-			
												2%	-	54	-	-	0.04	-	0.29	0.01	-			
												0.05%	-	-	-	-	-	-	-	-	-			
9	5RP16058	Lot Type Parcel	Regional	CATTLE BREEDING & FATTENING	93,694	200	10	1	10	0.5	20	20%	-	144	-	-	0.01	-	0.01	0.003	-	Rail design constraints alter the existing flow regime. Further cross drainage optimization investigations are required during detailed design.	Stakeholder Consultation (Landowner/Government). Further cross drainage optimization investigations are required during detailed design.	
												10%	-	39	0.66	-	-	0.01	-	-	-			
												5%	-	76	-	0.003	-	-	-	-	-			
												2%	-	206	0.21	-	-	0.01	-	-	-			
												0.05%	-	55	-	-	-	0.01	-	-	-			
10	28UDER3476	Lot Type Parcel	Regional	GRAINS	323,659	200	10	1	10	0.5	20	20%	-	55	0.84	-	0.01	-	0.88	0.02	-	Maturity of current cross drainage design is limited by available information (e.g. detailed survey) Further road and/or rail design optimization investigations are required during detailed design.	Further cross drainage optimization investigations are required during detailed design.	
												10%	-	67	0.54	-	0.02	-	4.52	0.02	-			
												5%	-	27	0.37	-	-	36	-	-	-			
												2%	-	71	0.92	-	-	1.21	-	-	-			
												0.05%	-	-	-	-	-	-	-	-	-			
11	5SP124139	Lot Type Parcel	Regional	CATTLE BREEDING & FATTENING	1,138,967	200	10	1	10	0.5	20	20%	-	-	-	-	-	-	-	0.02	-	Maturity of current cross drainage design is limited by available information (e.g. detailed survey) Maturity of current bridge design is limited by available information (e.g. detailed survey) Rail design constraints alter the existing flow regime.	Ongoing flood and flood awareness management. Stakeholder Consultation (Landowner/Government). Further cross drainage optimization investigations are required during detailed design.	
												10%	-	-	-	-	-	-	-	-	-			
												5%	-	-	-	-	-	-	-	-	-			
												2%	-	-	-	-	-	-	-	-	-			
												0.05%	-	-	-	-	-	-	-	-	-			
12	279XDER3476	Lot Type Parcel	Regional	GRAINS	321,732	200	10	1	10	0.5	20	20%	-	43	0.70	-	0.003	-	0.69	0.003	-	Maturity of current cross drainage design is limited by available information (e.g. detailed survey) Further road and/or rail design optimization investigations are required during detailed design.	Further cross drainage optimization investigations are required during detailed design.	
												10%	-	66	0.24	-	-	2.32	-	-	-			
												5%	-	21	0.58	-	-	0.17	-	-	-			
												2%	-	34	0.26	-	-	-	-	-	-			
												0.05%	-	-	-	-	-	-	-	-	-			
13	14UDER3476	Lot Type Parcel	Regional	VACANT RURAL LAND (EXCL G1 & G4)	34,237	200	10	1	10	0.5	20	20%	-	-	-	-	-	-	-	0.02	-	Maturity of current cross drainage design is limited by available information (e.g. detailed survey) Further road and/or rail design optimization investigations are required during detailed design.	Ongoing flood and flood awareness management. Stakeholder Consultation (Landowner/Government). Further cross drainage optimization investigations are required during detailed design.	
												10%	-	-	-	-	-	-	-	-	-			
												5%	-	-	-	-	-	-	-	-	-			
												2%	-	-	-	-	-	-	-	-	-			
												0.05%	-	-	-	-	-	-	-	-	-			
14	3RP94591	Lot Type Parcel	Regional	SHEEP GRAZING DRY	3,770,663	200	10	1	10	0.5	20	20%	-	24	-	-	-	8.03	-	-	-	Maturity of current cross drainage design is limited by available information (e.g. detailed survey) Maturity of current bridge design is limited by available information (e.g. detailed survey) Rail design constraints alter the existing flow regime.	Further cross drainage optimization investigations are required during detailed design. Further road and/or rail design optimization investigations are required during detailed design.	
												10%	-	23	0.24	-	-	0.04	-	-	-			
												5%	-	34	0.12	-	0.001	-	0.24	0.003	-			
												2%	-	-	-	-	-	-	-	-	-			
												0.05%	-	-	-	-	-	0.06	-	0.004	-			
15	5SP133098	Lot Type Parcel	Regional	GRAINS	2,772,035	200	10	1	10	0.5	20	20%	-	164	0.62	-	0.006	-	0.006	-	-	Maturity of current cross drainage design is limited by available information (e.g. detailed survey) Maturity of current bridge design is limited by available information (e.g. detailed survey) Rail design constraints alter the existing flow regime.	Ongoing flood and flood awareness management. Stakeholder Consultation (Landowner/Government). Further cross drainage optimization investigations are required during detailed design.	
												10%	-	207	0.20	-	-	0.04	-	-	-			
												5%	-	53	-	-	0.001	-	0.02	0.003	-			
												2%	-	40	2.38	-	-	0.04	-	-	-			
												0.05%	-	-	-	-	-	-	-	-	-			
16	6SP128162	Lot Type Parcel	Regional	GRAINS	1,576,101	200	10	1	10	0.5	20	20%	-	77	0.15	-	0.003	-	0.20	-	-	Maturity of current cross drainage design is limited by available information (e.g. detailed survey) Maturity of current bridge design is limited by available information (e.g. detailed survey) Rail design constraints alter the existing flow regime.	Further cross drainage optimization investigations are required during detailed design. Further road and/or rail design optimization investigations are required during detailed design.	
												10%	-	-	-	-	-	-	-	-	-			
												5%	-	-	-	-	-	-	-	-	-			
												2%	-	-	-	-	-	-	-	-	-			
												0.05%	-	-	-	-	-	-	-	-	-			
17	1RP107935	Lot Type Parcel	Regional	SHEEP GRAZING DRY	1,787,848	200	10	1	10	0.5	20	20%	-	47	0.58	-	0.06	-	0.06	-	-	Maturity of current cross drainage design is limited by available information (e.g. detailed survey) Maturity of current bridge design is limited by available information (e.g. detailed survey) Rail design constraints alter the existing flow regime.	Further cross drainage optimization investigations are required during detailed design. Further road and/or rail design optimization investigations are required during detailed design.	
												10%	-	35	0.08	-	-	0.01	-	-	-			
												5%	-	36	0.24	-	-	0.37	-	-	-			
												2%	-	-	-	-	-	-	-	-	-			
												0.05%	-	-	-	-	-	-	-	-	-			
18	3RP16300	Lot Type Parcel	Regional	GRAINS	1,564,968	200	10	1	10	0.5	20	20%	-	17	-	-	-	0.003	-	-	-	Maturity of current cross drainage design is limited by available information (e.g. detailed survey) Maturity of current bridge design is limited by available information (e.g. detailed survey) Rail design constraints alter the existing flow regime.	Ongoing flood and flood awareness management. Stakeholder Consultation (Landowner/Government). Further cross drainage optimization investigations are required during detailed design.	
												10%	-	-	0.17	-	-	0.006	-	-	-			
												5%	-	42	0.06	-	-	0.07	-	-	-			
												2%	-	-	0.16	-	-	0.003	-	-	-			
												0.05%	-	-	-	-	-	-	-	-	-			
19	3RP16058	Lot Type Parcel	Regional	CATTLE BREEDING & FATTENING	177,711	200	10	1	10	0.5	20	20%	-	275	5.56	-	0.27	-	0.14	-	-	Maturity of current cross drainage design is limited by available information (e.g. detailed survey) Maturity of current bridge design is limited by available information (e.g. detailed survey) Rail design constraints alter the existing flow regime.	Further cross drainage optimization investigations are required during detailed design. Further road and/or rail design optimization investigations are required during detailed design.	
												10%	-	250	1.18	-	-	0.27	-	-	-			
												5%	-	359	4.65	-	-	0.21	-	-	-			
												2%	-	263	0.37	-	-	0.17	-	-	-			
												0.05%	-	263	4.30	-	-	0.21	-	-	-			
20	43P217407	Lot Type Parcel	Regional	SUBDIVIDED LAND - DISCOUNTED BY LG	48,160	10	5	1	10	0.5	10	20%	-	15	-	-	-	0.003	-	-	-	Rail design constraints alter the existing flow regime. Stakeholder Consultation (Landowner/Government).	Ongoing flood and flood awareness management. Stakeholder Consultation (Landowner/Government).	
												10%	-	-	-	-	-	-	-	-	-			
												5%	-	-	-	-	-	-	-	-	-			
												2%	-	-	-	-	-	-	-	-	-			
												0.05%	-	-	-	-	-	-	-	-	-			
21	26RP425672	Lot Type Parcel	Regional	VACANT - LARGE HOUSESITE	375,870	10	5	1	10	0.5	10	20%	-	-	-	-	-	-	0.04	-	-	Maturity of current cross drainage design is limited by available information (e.g. detailed survey) Maturity of current bridge design is limited by available information (e.g. detailed survey) Rail design constraints alter the existing flow regime.	Further cross drainage optimization investigations are required during detailed design. Further road and/or rail design optimization investigations are required during detailed design.	

Depth (m)		Time of installation in hours (related to actual change exceedance)		Time of foundation in hours (related to % exceedance)		Velocity in m/s (related to % exceedance)		Velocity in m/s (related to actual change exceedance)		Heard in m/s (related to % exceedance)	
Existing Case	Design Case	Existing	Design	Existing	Design	Existing	Design	Existing	Design	Existing	Design
-	-	1.14	6.54	0.34	1.31	0.50	0.63	-	0.718	0.12	0.16
-	-	1.03	3.49	0.40	2.11	0.69	0.85	-	0.877	0.15	0.15
-	-	7.95	8.48	0.95	2.92	0.90	0.64	-	0.927	0.12	0.25
-	-	3.76	11.60	0.72	1.64	0.52	0.75	-	0.882	0.10	0.27
-	-	1.11	3.74	0.54	3.36	0.53	0.73	-	1.034	0.09	0.25
-	-	1.19	6.79	0.30	6.43	-	-	0.442	0.664	0.08	0.16
-	-	1.10	2.09	0.77	2.06	0.87	0.97	0.498	0.478	0.08	0.17
-	-	1.88	2.73	0.93	2.46	0.85	1.26	0.383	0.603	0.09	0.25
-	-	1.92	2.93	0.36	1.65	0.50	0.51	0.425	0.742	0.17	0.44
1.490	1.782	1.01	3.35	-	-	0.50	0.83	0.470	0.808	0.08	0.15
-	-	1.79	6.14	-	-	-	-	0.338	0.508	-	-
-	-	1.10	3.08	0.46	2.38	0.55	0.74	-	-	0.27	0.36
-	-	1.00	4.41	0.81	2.47	0.78	1.02	0.491	0.520	0.08	0.15
-	-	1.07	2.54	0.59	2.39	0.92	1.15	0.400	0.504	0.05	0.15
-	-	1.10	4.20	0.73	3.56	0.96	1.19	0.377	0.502	0.04	0.15
-	-	1.95	10.30	-	-	0.58	0.73	-	1.590	-	-
-	-	2.87	27.00	0.70	4.67	0.50	0.58	0.558	1.286	-	-
-	-	1.41	16.08	0.24	13.72	0.65	0.77	-	0.980	0.13	0.16
-	-	1.60	18.41	0.82	6.12	0.57	0.67	-	0.695	0.13	0.16
-	-	1.01	7.82	0.34	8.90	0.56	0.66	-	0.702	0.12	0.17
-	-	1.02	1.59	0.50	1.11	-	-	0.470	0.501	-	-
-	-	1.85	3.36	0.96	1.74	0.50	0.50	0.446	0.501	0.12	0.15
-	-	1.01	1.93	0.13	1.36	0.51	0.58	0.429	0.510	0.11	0.16
-	-	1.00	2.27	0.11	1.88	0.50	0.57	0.434	0.510	0.12	0.16
-	-	1.01	3.94	0.87	3.78	0.53	0.60	-	0.656	-	-
-	-	1.08	5.34	0.73	5.03	0.51	0.57	-	0.587	-	-
-	-	1.05	13.19	0.76	13.32	0.50	0.75	-	0.912	0.01	0.15
-	-	2.42	9.11	0.34	1.55	0.50	0.88	-	0.631	0.05	0.33
0.333	0.653	1.02	2.94	0.75	2.76	0.50	0.90	-	1.307	0.07	0.38
-	-	0.092	1.53	2.30	-	-	-	-	-	-	-
-	-	0.162	1.01	2.68	1.70	-	-	-	-	-	-
-	-	0.185	1.18	2.19	5.39	-	-	-	-	0.14	0.17
-	-	0.148	1.02	1.76	1.99	-	-	-	-	0.11	0.15
-	-	0.120	1.00	3.24	3.82	-	-	-	-	0.14	0.18
-	-	1.20	4.01	0.18	2.95	-	-	0.463	0.506	-	-
-	-	21.18	39.15	0.88	1.39	0.498	0.504	-	-	-	-
-	-	1.28	2.14	-	-	-	-	0.480	0.500	-	-
-	-	15.65	21.70	-	-	-	-	0.493	0.524	-	-
-	-	2.95	4.55	-	-	-	-	0.485	0.534	-	-
-	-	1.36	3.32	-	-	-	-	0.490	0.502	-	-
-	-	3.33	4.64	0.68	1.34	-	-	-	-	-	-
-	-	10.54	33.30	-	-	-	-	-	-	-	-
-	-	1.02	3.12	0.94	1.15	-	-	-	-	-	-
-	-	6.13	9.49	-	-	-	-	-	-	-	-
-	-	1.00	1.55	0.26	1.10	-	-	0.499	0.509	-	-
-	-	1.99	3.33	0.59	1.13	-	-	0.488	0.505	-	-
-	-	1.88	1.31	0.68	1.02	-	-	-	-	-	-
-	-	1.95	1.89	0.14	1.06	-	-	-	-	-	-
-	-	-	-	-	-	-	-	0.496	0.501	-	-
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
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ID	ID (Logplan ID)SR ID Culvert ID)	Sensitive Receptor Type	Catchment Type	Landuse ¹¹	Lot Area (m2)	Flood Impact Objective Target						AEP	Maximum FIO Exceedance					Area (m ²) ³				Exceedance Justification	Proposed Mitigation Measures ¹⁰																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																
						Change in Peak Water Level (mm)	Change in Time of Inundation (%)	Change in Time of Inundation (hours)	Change in Velocity (%)	Change in Velocity (m/s)	Change in Hazard (%)		Change in Peak Water Level (mm)	Change in Time of Inundation (% Change) ⁴	Change in Time of Inundation (hours)	Change in Velocity (m/s) ⁵	Change in Velocity (m/s)	Change in Hazard (%) ⁶	Afflux	Time on Inundation	Velocity			Hazard																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																															
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Table Notes

1. Floodplains are dynamic in nature and therefore model results vary from cell to cell. The "Area" is therefore defined as the maximum area over which change above the Flood Impact Objective occurs and does not relate directly to the maximum FIO exceedance recorded.

2. Change in water level (afflux) is calculated based on the maximum water surface level difference between the existing and design case results, and does not consider the underlying topography. Depth values presented are based on the respective underlying topographies and in some cases do not align with afflux calculations. This can be attributed to interpolation calculations of results performed within the hydraulic model.

3. Percentage increases should be read in conjunction with existing and design case results. The FIOs require that change be

8.5.3.6 Flow distribution

A key landowner concern is the potential change in flow distribution. Maximum velocity-depth modelling results were used to create change in unit flow maps between Existing and Design conditions to determine the change in flow distribution. Unit flow is a measure of discharge that is commonly used for assessing peak flow along cross-sections, or to identify the efficient flow corridors within a floodplain.

The presented flow distribution maps were filtered with a velocity x depth product cut-off of 0.15 m²/s. The cut-off is in line with FIO requirements and was used to identify main channels (high flow zones), and shallow lower flood zones (i.e. outer extents of the flood surface).

The following points summarise the change in flow distribution for Back Creek and should be considered in conjunction with flow distribution figures shown in Figure Q4-1 to Figure Q4-8 of Volume 2 – Appendix Q.

- Minor flow decreases (upstream) and increases (downstream) are localised to cross-drainage infrastructure. This behaviour is expected due to the constriction of the floodway through the culvert and bridge openings. This is particularly evident for all flood events up to the 1% AEP event, where the Project alignment is not overtopped.
- While the Project alignment does impact flow conditions immediately upstream and downstream of the Project alignment, regional conditions remain relatively unchanged.
- In the rarer events (1 in 2,000, 1 in 10,000 AEP and PMF events) some reduction in flow is experienced downstream extending towards the Condamine River in the 1 in 10,000 AEP and PMF events. This can be attributed to the alignment restricting flow at both bridge crossings in these larger events. Some positive change (>20 per cent) occurs upstream of the alignment in all three of these rare events.

Ongoing mitigation measures to reduce changes in flow distribution are likely to occur during future design stages of the Project, and may include further refinement of the bridge abutments and additional survey where necessary.

8.5.4 Extreme events

8.5.4.1 Extreme event immunity

The risk of overtopping of the Project alignment was assessed for a range of extreme events (1 in 2,000 AEP, 1 in 10,000 AEP and PMF) with Table 8.25 presenting the depth of water above the formation level and over the top of rail at each structure. It is noted that the function of the floodplain culverts is to balance flood levels on the upstream and downstream sides of the Project alignment. As such, overtopping of the rail is not predicted to result in significant excessive flows or velocities as would occur in a dam embankment overtopping scenario. Further analysis of the overtopping risk to FSRs can be found in Section 21.

As can be seen below, the Project alignment is not expected to overtop at each of the listed structures in extreme events. Some overtopping may occur further along the alignment in lower sections.

Table 8.25 Back Creek—Extreme events: Estimated depth of water above formation and top of rail levels

Chainage (km)	Structure ID	Depth of water above formation level (m)			Depth of water above top of rail (m)		
		1 in 2,000 AEP	1 in 10,000 AEP	PMF	1 in 2,000 AEP	1 in 10,000 AEP	PMF
126.77	C126.77	-	-	-	-	-	-
126.81	C126.81	-	-	-	-	-	-
127.00	310-BR37	-	-	-	-	-	-
128.00	310-BR38	-	-	-	-	-	-

8.5.4.2 Impacts during extreme events

Flood immunity of FSRs sensitive receptors is discussed in Section 8.5.3.1, and maps demonstrating the impacts during extreme events are shown on the [Online Digital Portal](#).

In accordance with the FIOs, FSRs are considered impacted by extreme flood events if the change in water level is greater than 250 mm in the 1 in 2,000 AEP event, subject to the receptor classification (i.e. at existing habitable dwellings and/or commercial and industrial buildings/premises). In addition, a stricter afflux objective of 10 mm in the 1 in 500 AEP event applies to sites with existing sensitive infrastructure, uses involving vulnerable people and/or any critical road network that was designed to be immune to flooding in any extreme event.

Review of the 1 in 2,000 AEP event results for the Back Creek floodplain against the FIOs show that flood level changes at all FSRs were limited to 250 mm, and no identified existing sensitive infrastructure or uses involving vulnerable people are expected to experience a more than 10 mm increase in flood level. A separate assessment of the 1 in 500 AEP event is not required as they are conservatively represented by the 1 in 2,000 AEP event results. The flood impacts in extreme events due to the Project are therefore compliant against the stipulated FIO limits for extreme events.

Table 8.26 details extreme event results, although it should be noted that FIO targets do not apply to the 1 in 10,000 AEP and PMF events.

Table 8.26 Back Creek—Summary of extreme event impacts at flood sensitive receptors

FSR ID	1 in 2,000 AEP event		1 in 10,000 AEP event		PMF event	
	Change in peak water level (mm)	Existing Case flood depth (m)	Change in peak water level (mm)	Existing Case flood depth (m)	Change in peak water level (mm)	Existing Case flood depth (m)
CON_ID_85	-	-	-	-	-10	0.51
CON_ID_86	-	-	-2	0.05	-10	0.65
CON_ID_83	-1	0.11	-2	0.25	-10	0.84
CON_ID_84	-1	0.08	-2	0.22	-10	0.80
CON_ID_65	-	-	-	-	-7	0.12
CON_ID_66	-	-	-	-	-8	0.20
CON_ID_67	-	-	-	-	-7	0.19
CON_ID_70	-	-	-	-	-11	0.32
CON_ID_71	-	-	-	-	-11	0.38
CON_ID_63	-	-	-	-	-7	0.23
CON_ID_64	-	-	-	-	-8	0.36
CON_ID_69	-	-	-	-	-11	0.45
CON_ID_73	-	-	-	-	-11	0.43
CON_ID_72	-	-	-	-	-12	0.61
CON_ID_127	-	-	-	-	-9	0.52
CON_ID_128	-	-	-	-	-9	0.65
CON_ID_129	-	-	-	-	-9	0.52
CON_ID_130	-	-	-	-	-9	0.59
CON_ID_126	-	-	-3	0.04	-9	0.80
CON_ID_125	-	-	-	-	-9	0.55
CON_ID_122	-	-	-	-	-9	0.41
CON_ID_123	-	-	-	-	-9	0.53
CON_ID_124	-	-	-	-	-9	0.67

FSR ID	1 in 2,000 AEP event		1 in 10,000 AEP event		PMF event	
	Change in peak water level (mm)	Existing Case flood depth (m)	Change in peak water level (mm)	Existing Case flood depth (m)	Change in peak water level (mm)	Existing Case flood depth (m)
CON_ID_145	-	0.27	-3	0.38	-8	0.99
CON_ID_74	-	-	-	-	-8	0.98
CON_ID_75	-	-	-	-	-8	0.91
CON_ID_76	-	-	-	-	-8	0.98
CON_ID_77	-	-	-	-	-8	1.01
CON_ID_87	-1	0.15	-4	0.28	-9	0.94
CON_ID_121	-	-	-	-	-10	0.41
CON_ID_88	-	-	-	-	-8	0.44
CON_ID_27	-1	0.15	-	0.30	-19	0.99
CON_ID_28	-	-	-	0.01	-20	0.69
CON_ID_107	-	0.01	-9	0.06	-21	1.06
CON_ID_108	-	-	-9	0.03	-21	1.03
CON_ID_54	-10	0.06	-24	0.31	-15	0.99
CON_ID_55	-	-	-21	0.24	-16	0.93
CON_ID_56	-3	0.02	-17	0.19	-16	0.87
CON_ID_57	-2	0.02	-18	0.16	-16	0.88
CON_ID_53	-9	0.08	-26	0.35	-15	1.05
CON_ID_58	-3	0.09	-20	0.28	-15	1.01
CON_ID_59	-4	0.07	-21	0.29	-15	1.01
CON_ID_29	-2	0.31	-8	0.46	-22	1.43
CON_ID_30	-1	0.08	-6	0.16	-18	0.88
CON_ID_31	-2	0.24	-8	0.38	-19	1.28
CON_ID_40	-1	0.08	-8	0.19	-21	1.12
CON_ID_44	-	-	-7	0.09	-21	0.99
CON_ID_45	-2	0.21	-9	0.37	-22	1.34
CON_ID_38	-1	0.07	-6	0.15	-20	0.93
CON_ID_39	-1	0.12	-6	0.21	-20	1.03
CON_ID_41	-1	0.06	-2	0.09	-19	0.83
CON_ID_42	-	-	-	-	-18	0.45
CON_ID_43	-1	0.05	-7	0.15	-20	0.96
CON_ID_46	-1	0.19	-5	0.26	-18	0.93
CON_ID_35	-1	0.08	-5	0.14	-19	0.84
CON_ID_36	-1	0.04	-6	0.10	-19	0.84
CON_ID_37	-1	0.05	-6	0.13	-19	0.89
CON_ID_32	-	-	-	-	-12	0.26
CON_ID_33	-1	0.01	-3	0.06	-19	0.66
CON_ID_34	-1	0.17	-5	0.24	-19	0.92
CON_ID_109	-	-	-	-	-16	0.53
CON_ID_47	-1	0.26	-6	0.35	-14	0.86

FSR ID	1 in 2,000 AEP event		1 in 10,000 AEP event		PMF event	
	Change in peak water level (mm)	Existing Case flood depth (m)	Change in peak water level (mm)	Existing Case flood depth (m)	Change in peak water level (mm)	Existing Case flood depth (m)
CON_ID_49	-	-	-	-	-15	0.51
CON_ID_48	-1	0.10	-6	0.18	-14	0.67
CON_ID_115	-	-	-	-	-11	0.36
CON_ID_116	-	-	-	-	-10	0.36
CON_ID_117	-	-	-	-	-7	0.09
CON_ID_113	-	-	-	-	-10	0.21
CON_ID_114	-	-	-	-	-12	0.17
CON_ID_110	-	-	-	-	-6	0.12
CON_ID_111	-	-	-	-	-6	0.10
CON_ID_112	-	-	-	-	-11	0.25
CON_ID_143	-	-	-	-	-9	0.36
CON_ID_144	-	-	-	-	-8	0.33
CON_ID_78	-	-	-	-	-	0.04
CON_ID_131	-	-	-5	0.01	-12	0.52
CON_ID_132	-2	0.02	-7	0.12	-12	0.74
CON_ID_136	-	-	-6	0.12	-12	0.68
CON_ID_139	-	-	-	-	-12	0.29
CON_ID_140	-2	0.07	-6	0.15	-12	0.70
CON_ID_141	-	-	-5	0.02	-11	0.52
CON_ID_142	-	-	-	-	-10	0.41
CON_ID_135	-2	0.06	-6	0.15	-13	0.77
CON_ID_137	-	-	-	-	-12	0.66
CON_ID_138	-	-	-	-	-12	0.36
CON_ID_134	-	-	-	-	-14	0.27
CON_ID_68	-	-	-	-	-12	0.04
CON_ID_50	-2	0.23	-7	0.33	-17	1.16
CON_ID_51	-2	0.19	-6	0.28	-17	1.07
CON_ID_52	-3	0.42	-8	0.54	-20	1.39
CON_ID_60	-	-	-	-	-13	0.67
CON_ID_61	-	-	-9	0.06	-13	0.88
CON_ID_62	-	0.02	-8	0.07	-13	0.87
CON_ID_1	-2	0.23	-7	0.29	-5	0.58
CON_ID_3	-2	0.24	-8	0.31	-6	0.67
CON_ID_2	-3	0.49	-11	0.60	-6	1.02
BAC_ID_1	-	-	-	-	+33	0.34
BAC_ID_2	-	-	+2	0.07	+32	0.48
BAC_ID_3	+4	0.09	+20	0.18	+170	0.63
BAC_ID_4	+5	0.18	+19	0.29	+128	0.75
BAC_ID_5	+6	0.23	+20	0.36	+124	0.85

FSR ID	1 in 2,000 AEP event		1 in 10,000 AEP event		PMF event	
	Change in peak water level (mm)	Existing Case flood depth (m)	Change in peak water level (mm)	Existing Case flood depth (m)	Change in peak water level (mm)	Existing Case flood depth (m)
BAC_ID_8	-	-	-	-	-	0.12
BAC_ID_9	-	-	-	-	-	0.11
BAC_ID_10	-	-	-	-	-	0.12
BAC_ID_13	-	-	-	-	-	0.06
BAC_ID_14	-	-	-	-	-	0.08
BAC_ID_18	-	-	-	-	-41	0.74
BAC_ID_19	-	-	-	-	-42	0.65
CON_ID_371	-	-	-	-	-	0.04
CON_ID_372	-	-	-6	0.26	-7	1.38
BAC_ID_21	-	1.19	-	1.62	-	2.97
CON_ID_379	-6	0.55	-20	0.83	-14	1.76

8.5.5 Sensitivity analysis—Back Creek

The sensitivity of the model to various parameters was assessed using the following two scenarios:

- An increase in rainfall intensity (i.e. to reflect climate change scenario)
- An increase and decrease in blockage of culverts from 25 to 50 per cent and 0 per cent.

8.5.5.1 Climate change

The potential impacts of climate change in the Back Creek floodplain were assessed for the 1% AEP design event to determine the sensitivity of the Project to the potential long-term changes in climate. The assessment was undertaken in accordance with ARR 2019.

The Representative Concentration Pathways 8.5 (2090 horizon) climate change scenario has been adopted for the Project with an associated increase in rainfall intensity of 23.9 per cent across the catchment area. The increase in rainfall intensity was applied to both the Existing Case and Developed Case scenarios.

The Project is expected to retain a 1% AEP flood immunity, with at least 2.2 m of freeboard to formation level under the climate change scenario.

The climate change sensitivity result indicate there are no predicted changes in peak water levels for the 1% AEP climate change event, as shown in Table 8.27. In addition to Table 8.27, peak water levels for the climate change scenario are presented in Figure D8-C in Volume 2 – Appendix D.

Table 8.27 Back Creek—1% AEP event with Representative Concentration Pathways 8.5 condition Climate Change assessment

FSR ID ¹	FSR type	1% AEP climate change event	
		Change in peak water level (mm)	Existing Case flood depth (m)
CON_ID_83	Shed	-	0.07
CON_ID_84	Shed	-	0.04
CON_ID_145	Shed	-	0.23
CON_ID_87	House	-	0.11
CON_ID_27	Shed	-	0.11
CON_ID_107	House	-	0.01
CON_ID_54	Shed	-	0.00

FSR ID ¹	FSR type	1% AEP climate change event	
		Change in peak water level (mm)	Existing Case flood depth (m)
CON_ID_56	Shed	-	0.00
CON_ID_58	House	-	0.04
CON_ID_59	Shed	-	0.01
CON_ID_29	Silos	-	0.24
CON_ID_30	House	-	0.05
CON_ID_31	Shed	-	0.19
CON_ID_40	Commercial	-	0.05
CON_ID_45	Commercial	-	0.14
CON_ID_38	Commercial	-	0.04
CON_ID_39	Commercial	-	0.08
CON_ID_41	Commercial	-	0.05
CON_ID_43	Commercial	-	0.01
CON_ID_46	Commercial	-	0.16
CON_ID_35	Commercial	-	0.05
CON_ID_36	Commercial	-	0.02
CON_ID_37	Commercial	-	0.03
CON_ID_34	Commercial	-	0.14
CON_ID_47	House	-	0.22
CON_ID_48	House	-	0.07
CON_ID_132	Commercial	-	0.00
CON_ID_140	Commercial	-	0.03
CON_ID_135	House	-	0.02
CON_ID_50	House	-	0.16
CON_ID_51	Shed	-	0.14
CON_ID_52	House	-	0.34
CON_ID_1	House	-	0.18
CON_ID_3	Shed	-	0.20
CON_ID_2	Shed	-	0.42
BAC_ID_5	Shed	-	0.02

Table note:

- 1 Only FSR locations that recorded a flood depth during pre-Project climate change conditions, and post Project climate change conditions have been recorded in the above table.

8.5.5.2 Blockage

Blockage was assessed in accordance with ARR 2019. The blockage assessment was undertaken and resulted in a blockage factor of 25 per cent adopted for culverts. A minimum culvert size of 900 mm diameter was adopted to reduce potential for blockage and maintenance. A significant community concern is the potential impacts to flood conditions should the proposed culverts become blocked with debris. The primary concern is the blockage of culverts which is likely to drive flood levels higher, particularly upstream of the culverts, and divert more flow through residences, across access roads and other infrastructure. A sensitivity analysis was undertaken with 0 and 50 per cent blockage on culverts.

Notably, there are no predicted impacts on FSRs due to the blockage assessment.

During detailed design the blockage factors will be reviewed in line with the final design and local catchment conditions. This may result in a varied and/or lower blockage factors being applied along the Project alignment. Should impacts be determined due to a change in blockage factor, additional mitigation measures will be considered in detailed design.

Maps demonstrating the effects of blockage are shown in Figures D8-A (0 per cent) and D8-B (50 per cent) in Volume 2 – Appendix D.

8.6 Additional modelling items to be considered in detailed design

The following points summarise additional items to be included in the flood models for detailed design as per agreement with the Expert Flood Panel:

- Undertake additional model strengthening by improving the representation of cross-drainage infrastructure, third-party infrastructure, noise barriers, survey data, and bridge structures within the hydraulic model. Specifically, the modelling of bridges will be included as LFC shape files within the hydraulic model.
- A further review of existing structures will be completed once detailed survey has been completed
- A review of sub-catchment delineation at the Project alignment to better reflect upstream and downstream catchments will be undertaken and where necessary updated
- A review of bridge losses will be undertaken to verify against alternative methods
- A revised impact assessment on roads will be completed and appropriate mitigation measures included in the design
- Velocities and scour protection measures will be reviewed as part of an impact assessment at detailed design to minimise scour as a result of the Project
- A review of critical durations and TPs will be completed
- Where flood exceedances have been identified additional analysis of cross-drainage infrastructure, and other mitigation options will be investigated to reduce impacts. Land and stakeholder consultation will also form part of the mitigation process.
- A review of the latest TUFLOW version will be undertaken at the commencement of detailed design to determine whether updates of the TUFLOW version from 2020-10-AD-iSP-w64 are beneficial to modelling.

Further details of mitigation measures can be found in Section 22. A review of proposed mitigation measures will be completed during the detailed design stage of the Project after hydraulic models have been updated to capture the above changes.

8.7 Planned and future roads

Consultation with the DTMR has not identified any planned or future road upgrades within the Back Creek catchment. ARTC will continue to consult and engage with DTMR to determine any future or planned road projects that may need to be considered as part of the detailed design hydraulic modelling.

TRC has not identified any planned or future roads within the Back Creek hydraulic model domain. ARTC will continue to consult and engage with TRC to determine any future or planned road projects that may need to be considered as part of the detailed design hydraulic modelling.

9 Nicol Creek

The Project alignment crosses Nicol Creek Study area approximately 350 m east of Millmerran-Inglewood Road. Nicol Creek is a narrow and meandering system with floodwater depths in the main channel under the Existing Case 1% AEP event of between 2 and 4 m, and between 0.3 and 1 m in breakout areas. At the Project alignment the peak flood depth is approximately 1.8 m.

Nicol Creek is a defined creek in terms of channel depths and banks and the flood extents under the 1% AEP event remain in close proximity to the creek alignment. The Existing Case 1% AEP inundated floodplain varies between 50 m and 200 m wide.

The location of the Project alignment in relation to Nicol Creek is shown in Figure E-1A in Volume 2 – Appendix E.

9.1 Data collection and review—Nicol Creek

Available background information including existing hydrologic and hydraulic models, survey, streamflow data, available calibration information and anecdotal flood data has been gathered. Data was sourced from a wide range of stakeholders including:

- Goondiwindi Regional Council (GRC)—existing flood studies
- BoM—rainfall gauging data
- DTMR—existing infrastructure details.

9.1.1 Previous studies

A number of previous hydrology and hydraulic studies were sourced as part of this assessment. A review of each study was undertaken to determine suitability for use on the Project as documented in the following sections.

9.1.1.1 Inglewood Flood Study, Engeny, 2015

Engeny was commissioned by GRC to undertake a flood study of Inglewood. The study objectives were ‘to define the nature, extent and risks of flooding in Inglewood in order to inform disaster management planning and response, as well as control future development’ (Engeny, 2015). An URBS hydrologic model and TUFLOW (1D/2D) hydraulic model were developed as part of this study.

The Inglewood Flood Study was used extensively for the development of the Nicol Creek, Bringalily Creek (Section 10), Native Dog Creek (Section 11) and Cattle Creek (Section 12) hydrology models, which all form part of the wider Canning Creek catchment.

9.1.2 Survey

ARTC provided LiDAR data from 2015 as 1 m grid DEM tiles. Using GIS software, a DEM was generated with a 1 m grid resolution for use in the Project based on the 2015 dataset. This was used for modelling within the permanent footprint and up to the full extent of the 2015 LiDAR where relevant.

In areas that were not covered by the LiDAR provided by ARTC, additional LiDAR tiles were sourced from Geoscience Australia. The DEM datasets utilised for modelling were based on survey flown between 2009 and 2015. SRTM data was used for catchment delineation where no LiDAR data could be sourced, to inform the hydrologic modelling.

The survey data sources, and DEM developed for Nicol Creek are shown in Figure E1-B of Volume 2 – Appendix E.

9.1.3 Aerial imagery

Aerial imagery of the Nicol Creek Study area was provided by ARTC and was used to identify and confirm topographic and vegetative characteristics of the study area. Aerial imagery captured in 2015 was made available. Additional imagery outside the study area was sourced from QGIS imagery in an open-source format.

9.1.4 Existing drainage structure data

DTMR as-constructed drawings were also sourced for culvert and bridge details. This information will be refined as the local survey is progressively completed as part of detailed design.

Details on existing drainage infrastructure included within the hydraulic models are presented in Section 9.3.4.

9.1.5 Stream gauge data

No streamflow gauges exist within the Nicol Creek catchment study area.

Refer to Section 9.2.1.1 for details on the Macintyre Brook gauge at Inglewood, and the FFA that was undertaken to validate the hydrologic model design flow estimates for the Canning Creek system, using the Macintyre Brook at Inglewood gauge (GS 416402C).

9.1.6 Rainfall data

Several daily and sub-daily rainfall stations are located in and around the Nicol Creek catchment; however, since there are no streamflow gauges to use for model calibration, no historical rainfall data was sought.

9.1.7 Anecdotal and observed flood data

No anecdotal or observed flood data was available for this area of Nicol Creek.

9.1.8 Site inspection

A site inspection was undertaken during February 2018. During the site inspection, all major waterway crossings were visited and inspected with photographs taken and details recorded of the crossing, existing drainage structures and surrounding catchment. An assessment of the relative roughness and blockage potential was undertaken during the site inspection.

9.2 Hydrologic model development—Nicol Creek

9.2.1 Unified Canning Creek model setup

Nicol Creek is one of four major waterways that cross the Project alignment before flowing into Canning Creek. The other waterways are Bringalily Creek, Native Dog Creek, and Cattle Creek. Due to their proximity to one another, all four waterways were modelled in a unified Canning Creek Model.

The hydrology of the Canning Creek catchment was previously modelled in URBS as part of the Inglewood Flood Study (Engeny, 2015). Relevant sections of the existing hydrology model were deemed suitable for use with results validated against the 1976 historical flood event at multiple stream-gauge locations (416402B/C, 416415A).

For this assessment the existing Inglewood Flood Study hydrology model was refined to ensure it was suitable for use by incorporating:

- Updated hydrologic inputs to ensure design rainfall intensities, losses and TPs conformed to relevant ARR 2019 standards
- A cutdown version of the larger URBS model was established, including the Nicol Creek, Bringalily Creek, Native Dog Creek, and Cattle Creek catchments. As previously highlighted, due to their proximity to one another, all four waterways were modelled in a unified model, herein referred to as the 'Canning Creek model'.
- The Canning Creek model was validated against the Inglewood Flood Study hydrology model to ensure consistency, and an additional FFA was also performed to ensure consistent and appropriate hydrology parameters were adopted for all local models
- Sub-delineation of the Canning Creek model was undertaken to extract estimated peak flows at key locations and Project alignment crossings for Nicol Creek, Bringalily Creek, Native Dog Creek, and Cattle Creek.

As the Canning Creek model was used extensively for the above catchments the sections below outline additional analysis including, model updates, adopted model parameters, validation, and results.

Further details of the Nicol Creek hydrologic model including model updates, validation, and results are also presented in the below sections.

9.2.1.1 Canning Creek hydrologic model

The unified Canning Creek model covers approximately 1,202 km² of the Canning Creek catchment upstream of Inglewood. The catchment comprises 35 sub-catchments to capture the variability of rainfall and to better represent the network of creeks and streams within the catchment.

The hydrologic set-up including sub-catchment delineation is presented in Figure E1-C of Volume II – Appendix E.

Adopted URBS model parameters

The Canning Creek model was developed using a default URBS alpha value of 1.2. The default value is smaller than the adopted value in the *Inglewood Flood Study* (Engeny, 2015). The previous study adopted a uniform alpha value of 1.7 as an average representation of the total catchment area reporting to the gauge; however, this value is expected to change between sub-catchments due to the topographic variability and presence of control structures. For example, to the east of Canning Creek approximately 50 per cent of the gauged Macintyre Brook catchment (1,760 km²) reports to Coolmunda Dam, which has a storage capacity of 69 GL, and a spillway capacity of 6,860 m³/s; therefore, flows in Macintyre Brook have been heavily regulated by the storage and discharge of the dam.

A sensitivity test was conducted for the Canning Creek model by adopting the original alpha parameter of 1.7 and comparing the peak flows. The test demonstrated a 23 per cent reduction in the 1% AEP peak flow at the Canning Creek model outlet. As such, the default value of 1.2 was adopted for this investigation leading to a conservative estimate when compared to the *Inglewood Flood Study*.

The adopted Canning Creek model details are summarised in Table 9.1. The values are consistent for the four waterways assessed within Canning Creek, including Nicol Creek, Bringalily Creek, Native Dog Creek, and Cattle Creek.

Table 9.1 Canning Creek—Summary of URBS model inputs

Input parameter	Remarks
URBS model type	Basic
Routing variables	Catchment area, stream lengths
Channel lag parameter, α	1.20
Catchment non-linearity parameter, m	0.8

Initial and continuing losses

The Canning Creek model losses are shown in Table 9.2. In conjunction with other hydrologic parameters, loss values were adjusted to achieve a satisfactory match to an area-scaled FFA of the Macintyre Brook at Inglewood gauge (GS 416402C).

Table 9.2 Canning Creek—Adopted initial and continuing losses

AEP	Initial storm loss (mm)	Continuing loss (mm/hr)
20%	42.8	2.8
10%	38.6	2.6
5%	32.6	2.2
2%	22.3	1.5
1%	6.4	0.4

Three key locations were used to validate the results of the Canning Creek model against the results from the *Inglewood Flood Study* (Engeny, 2015) model for the 1% AEP event. The DTMR's QRT flow estimation method was also carried out to provide a further validation of the URBS hydrologic model results. This approach provides a procedure for calculating design floods for ungauged catchments that relate the design flood discharges to readily available catchment characteristics, such as catchment area and 2% AEP 72-hour rainfall intensity. This method is commonly used for ungauged catchments in Queensland.

The results of this comparison are presented in Table 9.3.

Table 9.3 Canning Creek—Comparison of flows at key locations—1% AEP event

Inglewood Flood Study URBS catchment ID	FFJV URBS catchment ID	Revised Inglewood Flood Study URBS discharge estimate (m ³ /s)	FFJV URBS flow estimate (m ³ /s)	QRT estimate (m ³ /s)
S_2	08T	561	632	703
S_3	14T	579	618	707
S_5	28T	980	1,408	1,322

Canning Creek design model parameters

Hydrologic information to assist estimation of design event flows was sourced from the ARR 2019 Data Hub, as summarised in Table 9.4. Details of the adopted hydrologic model parameters are also presented below.

Table 9.4 Canning Creek—Summary of URBS model design event inputs

Input parameter	Remarks
Design rainfall	Intensity-Frequency-Durations for each sub-catchment were downloaded from the BoM's website to account for variation in rainfall across the catchment.
Extreme event rainfall	Probable Maximum Precipitation depths for durations up to 6 hours (for use in modelling the PMF event) were obtained using the method presented in the Bulletin 53 (BoM, 2003). The rainfall depths for the 1 in 10,000 AEP event were estimated using the interpolation method presented in ARR 2019 Book 8 Section 3.5.

Input parameter	Remarks
Losses	The losses were initially adopted from the <i>Inglewood Flood Study</i> (Engeny, 2015) URBS model (Initial loss – 15 mm, Continuing loss – 1 mm/hr). Adjustments were made to reconcile URBS flows to area-scaled FFA.
Areal reduction factor	Parameters were adopted for the semi-arid inland Queensland region. The catchment area of Brungalily Creek is approximately 138 km ² , which yields an ARF between 52.2% and 93.5% depending on design storm event AEP and duration.
Ensemble TPs	Central Slopes regions. However, as the study catchment area exceeds 75 km ² , the standard ensemble rainfall patterns from ARR 2019 do not apply to this catchment for storm durations longer than 12h. These were replaced with the areal TPs for the Central Slopes region.
Pre-burst depths	Median pre-burst depths were downloaded from the ARR 2019 Data Hub for each sub-catchment. Pre-burst depths vary by design storm event AEP and duration. Pre-burst depths were applied to the model by reducing the initial losses for each storm event.

Canning Creek model calibration

Insufficient data was available to calibrate historical flood events in the Canning Creek catchment. Design peak flow estimates were validated against FFA estimates, as documented in the section below.

Canning Creek flood frequency analysis

A FFA was undertaken to validate the hydrologic model design flow estimates for Canning Creek. The FFA from the *Inglewood Flood Study* (Engeny, 2015) was adopted and updated to include additional flood data post-2015. The exercise was undertaken using the Macintyre Brook at Inglewood gauge (GS 416402C).

Due to the difference in catchment areas reporting to the Inglewood gauge (Canning Creek (1,202 km²), and Macintyre Brook (3,430 km²)), an area-scaled adjustment of the estimated peak flow values was undertaken. The downscaled adjustment for the Canning Creek model outlet was estimated based on the ratio of catchment areas to the power of 0.7. The FFA was undertaken using a LPIII distribution.

The FFA and scaled flow results at the Canning Creek model outlet are summarised in Table 9.5.

Table 9.5 Macintyre Brook at Inglewood (416402) peak flow estimated (LP3) and scaled flow at Canning Creek catchment outlet

AEP (%)	FFA peak flow (m ³ /s)	Scaled flow at Canning Creek model outlet (m ³ /s)
10	910	440
5	1,600	770
2	2,950	1,420
1	4,390	2,110
0.2	9,540	4,580

To validate the Canning Creek model, the initial and continuing losses were adjusted until there was a reasonable match between the modelled flows and those from the scaled FFAs. Emphasis was placed on matching flows to those specified in the Inglewood Flood Study. It was noted that the flows in the Inglewood Flood Study's FFA were routinely 20 per cent higher than the FFA generated by FFJV.

The QRT and RFFE flow estimation methods were included as additional points of comparison. All URBS hydrologic model results between the 20% AEP and 1% AEP show a fair match with QRT flow estimates. For events between the 5% and 1% AEP, all flow estimates are within the 90 per cent confidence limits of the RFFE estimates; however, it is noted that for events less than the 5% AEP, URBS model results are outside of the lower bound RFFE estimates.

The URBS model flows for the 1% AEP design event corroborate well with the scaled FFA and QRT estimates. The estimated flood flows at the Canning Creek model outlet are presented in Figure 9.1 and summarised in Table 9.6.

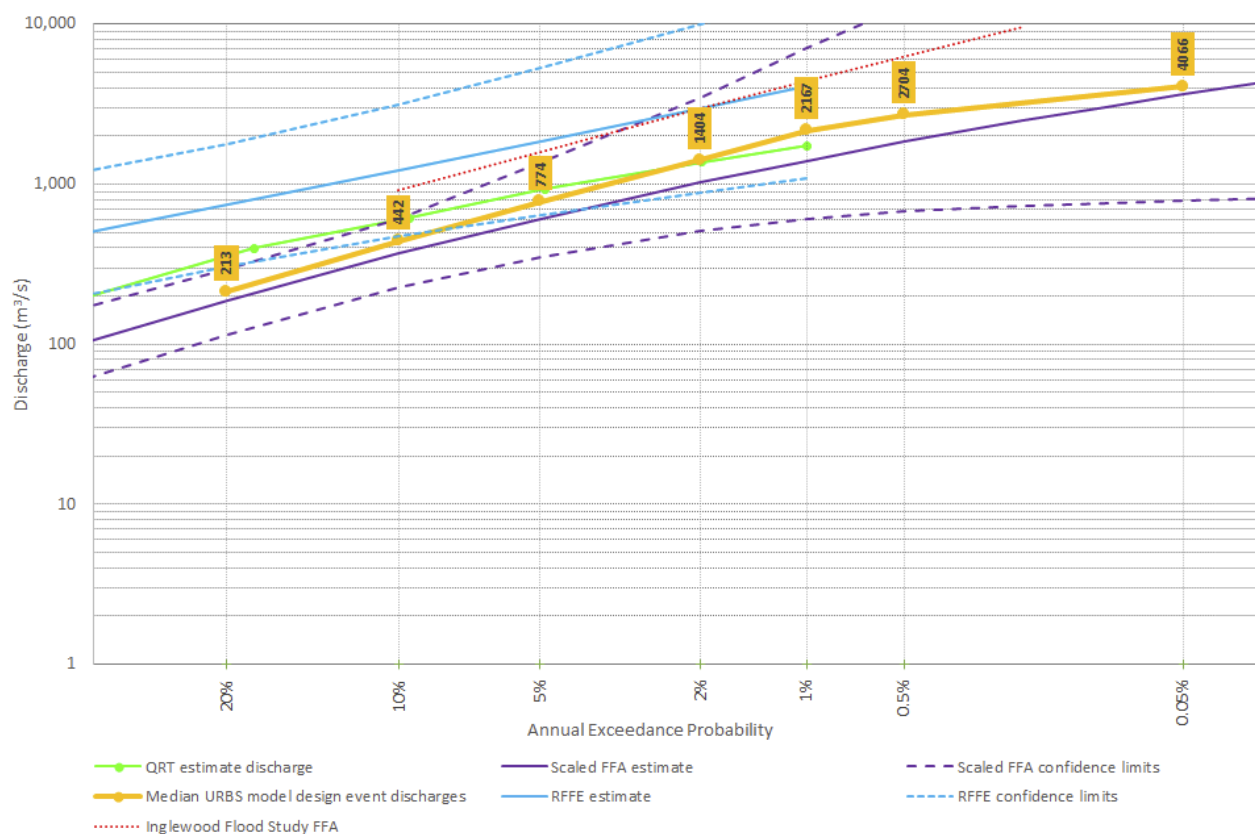


Figure 9.1 Estimate of flows at the Canning Creek model outlet

Table 9.6 Estimate of flows at the Canning Creek model outlet

AEP (%)	RFFE – lower bound 90% confidence level (m³/s)	RFFE – estimate of flow (m³/s)	RFFE – upper bound 90% confidence level (m³/s)	QRT (m³/s) ¹	URBS model flows (m³/s)
20	309	740	1,790	395	213
10	472	1,220	3,160	611	442
5	644	1,850	5,280	927	774
2	891	2,980	9,870	1,366	1,404
1	1,090	4,090	15,200	1,734	2,167
1 in 2,000	-	-	-	-	4,066

Table note:

¹ The QRT method estimates the 39.3%, 18.1%, 9.5% and 4.9% AEP instead of 50%, 20% and 10% and 5%, respectively

Canning Creek—Individual catchment peak flow assessment

Further investigations were undertaken to determine the suitability of design flow estimates for Nicol Creek, Bringalily Creek, Native Dog Creek, and Cattle Creek. Design flow rates were informed by the scaled FFA, including the adoption of design flow losses for each catchment.

Table 9.7 summarises peak flows estimated using both URBS and the QRT method for all watercourses in the Canning Creek system, noting that the same URBS parameters were applied to assess all watercourses. The adopted URBS design flows are already conservative for Bringalily Creek and Native Dog Creek as well as the overall Canning Creek system. The URBS design flows are lower than the QRT estimates for Nicol Creek and Cattle Creek.

Table 9.7 Comparison between URBS and QRT flows for Canning Creek and tributaries

Watercourse	Catchment area (km ²)	1% AEP Peak Flow (m ³ /s)		
		URBS	QRT	Difference (%)
Nicol Creek	38	133	193	-45
Bringalily Creek	188	807	534	+51
Native Dog Creek	27	173	154	+12
Cattle Creek	65	213	271	-21
Canning Creek	1,202	2,167	1,735	+25

Table note:

1 Peak flows reported at individual model outlets

9.2.2 Nicol Creek hydrologic model

The Nicol Creek hydrologic model consists of 13 sub-catchments upstream of the Project alignment, with a total area of approximately 38.4 km². As previously explained, the Nicol Creek catchment is part of the larger Canning Creek model, that has been further sub-delineated to allow for increase hydrologic definition and extraction of results.

Figure E1-C in Volume 2 – Appendix E shows the sub-catchment delineation for Nicol Creek.

9.2.2.1 Design model parameters

Hydrologic information to assist estimation of design event flows was sourced from the ARR 2019 Data Hub as summarised in Table 9.8.

Table 9.8 Nicol Creek—Summary of URBS model design event inputs

Input parameter	Remarks
Design rainfall	A single IFD was downloaded from the BoM's website at the centroid of the Nicol Creek catchment.
Extreme event rainfall	Probable Maximum Precipitation depths for durations up to 6 hours (for use in modelling the PMF event) were obtained using the method presented in the Bulletin 53 (BoM, 2003). The rainfall depths for the 1 in 10,000 AEP event were estimated using the interpolation method presented in ARR 2019 Book 8 Section 3.5.
Losses	The losses were initially adopted from the <i>Inglewood Flood Study</i> (Engeny, 2015) URBS model (Initial loss – 15 mm, Continuing loss – 1 mm/hr). Adjustments were made to reconcile URBS flows to area-scaled FFA.
Areal reduction factor	Parameters were adopted for the semi-arid inland Queensland region. The catchment area of Nicole Creek is approximately 38 km ² , which yields an ARF between 79% and 97% depending on design storm event AEP and duration.
Ensemble TPs	Central Slopes region.
Pre-burst depths	Median pre-burst depths were downloaded from the ARR 2019 Data Hub for each sub-catchment. Pre-burst depths vary by design storm event AEP and duration. Pre-burst depths were applied to the model by reducing the initial losses for each storm event.

Spatial distribution

The ARR 2019 design event methodology was adopted for this assessment. ARR 2019 recommends adopting the spatially varying method for catchments larger than 20 km². The Nicol Creek catchment is approximately 38 km². Given the minor difference, a single IFD was adopted for Nicol Creek.

Rainfall depth estimation

Design rainfall depth for 20% to 1 in 2,000 AEP were manually obtained from BoM website for each sub-catchment. As an example, the IFD representing the centroid of Nicol Creek URBS model is provided in Table 9.9.

Table 9.9 Rainfall depth (mm) used for the Nicol Creek catchment centroid

Duration (hr)	Annual Exceedance Probability (% AEP)						
	50	20	10	5	2	1	1 in 2,000
0.25	22.2	30.9	37	43.1	51.4	57.8	90.6
0.50	23.8	33.3	39.9	46.5	55.4	62.4	97.9
1.00	30.1	42	50.4	58.9	70.3	79.4	125
2.00	36.4	50.3	60.2	70.3	83.9	94.8	148
3.00	40.3	55.3	65.9	76.7	91.5	103	162
4.50	44.5	60.5	71.9	83.4	99.3	112	175
6.00	47.8	64.7	76.6	88.6	105	119	185
12.00	57.3	76.7	90.4	104	123	139	217
24.00	69.3	92.9	109	126	149	168	263
48.00	83.1	113	135	156	186	210	341
72.00	91.3	126	150	175	210	237	377

Areal reduction factor

The ARF was estimated using the coefficients for semi-arid inland Queensland presented in Table 9.10 and was applied to all IFD areas. The URBS software directly downloaded these coefficients from ARR 2019 Data Hub to estimate the ARF using Equation 2.4.1 of ARR 2019 Book 2.

The estimated ARF for Nicol Creek catchment vary between 79 and 97 per cent depending on design storm event AEP and duration.

Table 9.10 Nicol Creek—Areal reduction factor coefficients from ARR 2019 Data Hub

a	b	c	d	e	f	g	h	i
0.159	0.283	0.25	0.308	7.30E-07	1.0	0.039	0.0	0.0

Temporal patterns

Temporal patterns for 20% AEP to 1 in 2,000 AEP events were adopted based on the 10-burst ensemble approach presented in the ARR 2019. The TPs and pre-burst data were directly downloaded by URBS software from the ARR 2019 Data Hub at the centroid of Nicol Creek catchment.

Temporal patterns for the 1 in 10,000 AEP and PMF design events for up to 6 hours were obtained from *The Estimation of Probable Maximum Precipitation in Australia: Generalised Short-Duration Method* (BoM, 2003).

The adopted temporal patterns for all modelled events are shown in Table 9.11.

Table 9.11 Nicol Creek—Adopted temporal patterns

AEP (%)	Critical duration storm/temporal pattern
20	12 hour - Pattern 4
10	12 hour - Pattern 0
5	12 hour - Pattern 6
2	9 hour - Pattern 6
1	9 hour - Pattern 9
1 in 2,000	9 hour - Pattern 9
1 in 10,000	6 hour - PMP temporal pattern
PMF	6 hour - PMP temporal pattern

Pre-burst

Median pre-burst depths for standard durations from 0.5 hour to 72 hours for a 1% AEP design event are between 1.1 mm and 13.1 mm (Data Hub accessed November 2018). Due to the small depth of pre-burst in this area, the impact of pre-burst condition is considered to be minimal in this catchment. The adopted pre-burst depths are summarised in Table 9.12.

Table 9.12 Nicol Creek—Adopted pre-burst depths

Duration	Pre-burst depth (mm)					
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP
0.5	0.6	0.8	0.9	1.1	1.7	2.3
0.75	0.6	0.8	0.9	1.1	1.7	2.3
1.0	0.6	0.8	0.9	1.1	1.7	2.3
1.5	0.2	0.3	0.4	0.5	1.6	2.4
2.0	0.2	0.3	0.3	0.3	1.7	2.8
3.0	0.1	0.0	0.0	0.0	2.8	5.0
6.0	0.2	0.6	0.8	1.1	1.1	1.1
12.0	0.0	0.2	0.2	0.3	5.2	8.8
18.0	0.0	0.2	0.2	0.3	5.2	8.8
24.0	0.0	0.3	0.5	0.7	7.8	13.1
48.0	0.0	0.0	0.0	0.0	0.1	0.2
72.0	0.0	0.0	0.0	0.0	0.0	0.0

Design rainfall losses

In this assessment, an initial and continuing loss model was applied, where the lost rainfall depth does not contribute to the catchment runoff. In the loss model, it was assumed that there is an initial loss of rainfall depth followed by a constant continuing loss (mm/hr) throughout the rainfall event.

A range of design rainfall losses were trialled for the design events. Initial and continuing losses in the URBS model were adjusted until there was a reasonable match between the modelled flows and those from the scaled FFAs.

The adopted design event losses are provided in Table 9.13.

Table 9.13 Nicol Creek—Adopted design event losses

AEP	Initial storm loss (mm)	Continuing loss (mm/hr)
20%	42.3	2.8
10%	38.6	2.6
5%	32.6	2.2
2%	22.3	1.5
1%	6.4	0.4

Extreme event rainfall

Probable Maximum Precipitation depths for durations up to 6 hours (for use in modelling the PMF event) were obtained using the methodology presented in *The Estimation of Probable Maximum Precipitation in Australia: Generalised Short- Duration Method* (BoM, 2003). Once the PMP was estimated, rainfall for events between 1 in 2,000 AEP and the PMF was then interpolated.

For the 1 in 10,000 AEP event, ARF were applied based on interpolation methodology presented in ARR 2019 Section 3.5 in Book 8 and the rainfall depths were estimated accordingly. The parameters adopted for PMP rainfall estimation are presented in Table 9.14 , and the adopted extreme rainfall depths are provided in Table 9.15.

Table 9.14 Nicol Creek—Input data for Probable Maximum Precipitation estimation

Factor	Value
EAF	1
MAF	0.8
Portion of rough catchment	100%
Nicol Creek location and catchment area (km ²)	(-28.034, 151.201) and 38.4 km ²

Table 9.15 Nicol Creek—Adopted extreme event rainfall depths (mm)

Duration (hour)	Nicol Creek	
	1 in 10,000 AEP rainfall depth (mm)	PMP rainfall depth (mm)
0.25	87	154
0.5	121	226
0.75	143	287
1	157	341
1.5	174	438
2	184	509
2.5	195	567
3	204	616
4	216	700
5	225	770
6	231	820

9.2.2.2 Model calibration

Insufficient data was available to calibrate the model to historical flood events in the Nicol Creek catchment.

9.2.2.3 Model validation

The RFFE flow estimation method was used for the validation of the hydrologic model of the Nicol Creek catchment.

The RFFE online tool was used to obtain an estimation of catchment peak flow at the outlet of Nicol Creek catchment. The RFFE inputs are summarised in Table 9.16, and outputs are summarised in Figure 9.2 and Table 9.17.

Table 9.16 Nicol Creek—Model inputs for the Regional Flood Frequency Estimation

Model parameter	Input value
Region name	East Coast
Region version	RFFE Model 2016 V1
Latitude at catchment outlet (degree)	-28.09
Longitude at catchment outlet (degree)	151.20
Latitude at catchment centroid (degree)	-28.03
Longitude at catchment centroid (degree)	151.20
Distance of the nearest gauged catchment in the database (km)	38.8
Catchment area (km ²)	38.4
Design rainfall intensity, 50% AEP and 6-hr duration (mm/hr)	7.97
Design rainfall intensity, 2% AEP and 6-hr duration (mm/hr)	17.52

Additional peak flow validation was undertaken using the QRT flow estimation method. The estimation of peak flow using the QRT method for the Nicol Creek catchment is based on the total catchment area of 38.4 km² and a 2% AEP 72-hour rainfall intensity of 2.92 mm/hr. The estimated peak flows at the Nicol Creek catchment outlet using QRT is provided in Table 9.17.

The flood flows in Nicol Creek were estimated from the Canning Creek model with additional catchment definition, and ARF amended to suit the Nicol Creek catchment. All flows produced by the URBS between the 20% AEP and 1% AEP events reside within the 90 per cent confidence limits of the RFFE and show a close match with QRT. The estimated flood flows at the proposed Nicol Creek crossing are presented in Figure 9.2 and Table 9.17.

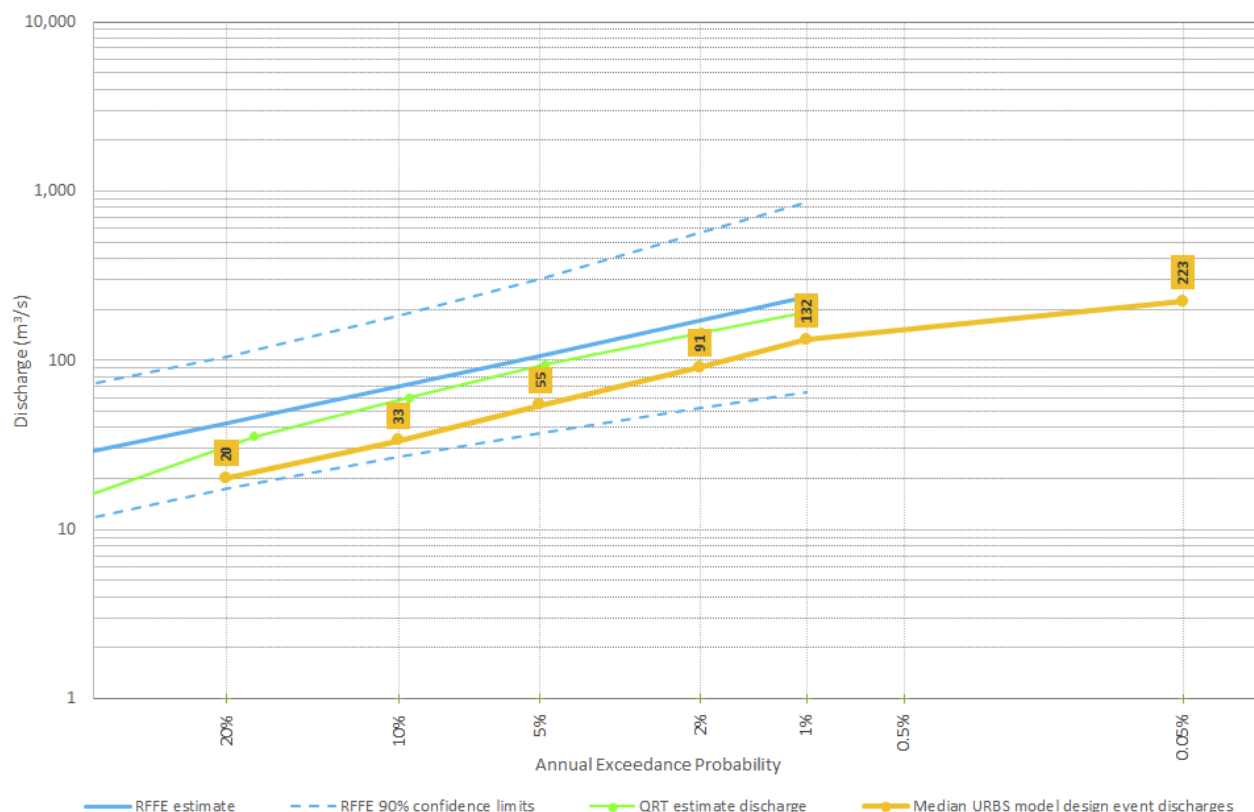


Figure 9.2 Estimate of flows at the Nicol Creek Project crossing

Table 9.17 Estimate of flows at the Nicol Creek Project crossing

AEP (%)	RFFE – lower bound 90% confidence level (m³/s)	RFFE – estimate of peak flow (m³/s)	RFFE – upper bound 90% confidence level (m³/s)	QRT – estimate of peak flow (m³/s) ¹	URBS model flows (m³/s)
20	18	43	105	34	20
10	27	70	184	60	33
5	37	107	303	94	55
2	52	172	563	146	91
1	65	237	860	193	132
1 in 2,000	-	-	-	-	223
1 in 10,000	-	-	-	-	394
PMF	-	-	-	-	1,884

Table note:

1 The QRT method estimates the 39.3%, 18.1%, 9.5% and 4.9% AEP instead of 50%, 20% and 10% and 5%, respectively

9.3 Hydraulic model development—Nicol Creek

A 2D modelling approach was adopted to simulate the flood regime around the proposed rail crossing at Nicol Creek. The platform used for hydraulic modelling was TUFLOW HPC software package (version 2020-10-AD) with SGS and QT approach. The processes and assumptions adopted throughout the development of the hydraulic model are described in the following sections.

9.3.1 Model setup

The setup of the TUFLOW model is summarised in Table 9.18.

The hydraulic model extent and the spatial distribution of land use in the 2D model domain is presented in Figure E1-D and Figure E1-E in Volume 2 – Appendix E, respectively.

Table 9.18 Nicol Creek hydraulic model summary

Parameter	Information
Completion date	August 2022
AEPs assessed	20%, 10%, 5%, 2%, 1%, 1 in 2,000, 1 in 10,000 AEP and PMF
Hydraulic model build	TUFLOW HPC GPU – version 2020-10-AD-iSP-w64
Model extent	Refer to Figure E-1D in Volume 2 – Appendix E
Grid size	5 m
SGS distance	1 m
QT grid size	2.5 m, 2 nd Order
DEM (year flown)	ARTC dataset (2015).
Roughness	Spatially varying roughness values compliant with industry norms.
Eddy viscosity	Wu (default)
Model calibration	N/A
D/S model boundary	Height-discharge (HQ) boundary with normal slope approximated based upon topography dataset.
Hydraulic model timestep	Adaptive timestep
Modelled scenarios	Existing Case, Developed Case
Sensitivity analysis	Climate change and Blockage (0% and 50% Blockage on culverts)

9.3.2 Model topography and grid size

The topography within the hydraulic model was based on 1 m DEM data collected during 2015. A 5 m x 5 m grid size was selected to allow the features of the channels and floodplain to be represented with sufficient accuracy while maintaining efficient model run times. The 5 m x 5 m grid size is considered adequate to represent floodplain topography and is consistent with the approach typically adopted in similar flood studies.

The 2D model topography was modified adjacent to the culverts so that the existing drainage flows, existing road/rail crowns and levee banks were represented accurately in the model. For the Developed Case, the topography was modified to include the Project rail embankment, drainage structures and associated infrastructure (Section 9.5).

9.3.2.1 Sub-grid sampling and Quadtree mesh refinement

The hydraulic model was established using TUFLOW's 2020-10-AD model version, including SGS and QT add-on modules. The below points provide a brief description of each module, along with the key parameters adopted for this assessment.

- Traditional grid-mesh based 2D hydraulic models sample the underlying DEM by taking a single elevation at the cell centres and cell faces. Sub-grid sampling allows for increased resolution by sampling a greater number of points of the underlying DEM to establish a water surface elevation versus volume relationship for each grid, along with a water surface elevation versus width relationship (wetted perimeter) at cell sides. The adoption of SGS allows increased representation of storage and conveyance between cells. A SGS distance of 1 m was adopted for this assessment.

- Quadtree mesh is constructed by subdividing the base mesh-grid size into four smaller square cells. Cells can be further sub-divided into four, and so on, allowing for additional resolution in location of interest. The base hydraulic model grid cell size (5 m x 5 m) was revised to include additional spatial resolution of approximately 2.5 m, to better represent model velocity at the entry and exit of culverts. Quadtree meshes were applied in the model covering minimum 25 m upstream and 100 m downstream of the design culvert locations, and culverts within the permanent footprint.

9.3.3 Roughness

The hydraulic roughness generally reflects the types of development and ground cover that exists within the hydraulic model extent. The distribution of roughness categories adopted for this assessment was based on the information supplied in the land-use layer from Queensland Globe database and aerial imagery, and confirmed during the site inspection. Specific roughness values applied to the model are detailed in Table 9.19.

Table 9.19 Nicol Creek—Adopted roughness values

Category	Manning's 'n' value
River	0.035
Sealed roads	0.015
Unsealed roads	0.025
Rural residential	0.09
Urban residential	0.10
Commercial	0.06
Open space—Minimum vegetation	0.045
Open Space—Moderate vegetation	0.055
Open Space—Heavy vegetation	0.065
Open water	0.025
Rail	0.025
Riparian	0.04
Crops ¹	0.1085

Table note:

- 1 A uniform value was adopted for the crop areas, due to the seasonal and spatial variations in vegetation distribution and the uncertainty this brings to the design event approach

9.3.4 Hydraulic structures

There are no details of existing hydraulic structures situated within the extents of the hydraulic model.

9.3.5 Boundary conditions

The following boundary conditions were applied within the hydraulic model:

- Flow versus time model boundaries were applied for all inflow hydrographs, with local and total hydrographs sourced from the hydrologic models
- Delineated sub-catchments in the hydrologic model were used as internal flow boundaries within the hydraulic model extent (SA polygons). The modelled flows in the hydrologic model were applied to the lowest points of each SA polygon.
- Where the Project alignment intersected the sub-catchment, the inflows were divided accordingly based on catchments upstream and downstream of the alignment

- A height-discharge boundary (HQ) was applied for the downstream outflow model boundary. It was confirmed that it was a sufficient distance away from the area of interest so as not to impact on modelling results.

9.4 Existing Case modelling results—Nicol Creek

9.4.1 Hydrologic model peak flow assessment

A critical duration analysis was performed to identify the appropriate design storm(s) and consequent peak flood flows at identified locations of interest. The locations of interest for this assessment include the Project's major Nicol Creek crossing (Bridge 310-BR11), and the outlet of the hydrologic model (Figure E1-C in Volume 2 – Appendix E).

To assess the critical storm duration(s), the following methodology was adopted:

- Modelling was completed for a range of AEP events:
 - Each AEP was modelled for a range of durations from 30 minutes to 144 hours
 - Each duration was modelled for the 10 associated TPs.
- For each duration, peak flood flows were extracted at the location of interest for the highest median TP (rank 6). The duration and TP resulting in the highest rank 6 peak flow estimate at the location of interest was adopted as an inflow into the hydraulic model.

The critical duration and adopted TP for each design event AEP is provided in Table 9.20. Table 9.21 outlines the estimated peak flow for the 1% AEP event based on the adopted critical TP for a number of key locations (Figure E1-D in Volume 2 – Appendix E).

Table 9.20 Nicol Creek—Peak flow at key locations as applied in the hydraulic model

AEP (%)	Peak flow (m ³ /s)	Critical duration storm/temporal pattern
20	22	12 hour – Pattern 4
10	36	12 hour – Pattern 0
5	55	12 hour – Pattern 6
2	93	9 hour – Pattern 6
1	133	9 hour – Pattern 9
1 in 2,000	223	9 hour – Pattern 9
1 in 10,000	394	6 hour – PMP temporal pattern
PMF	1,884	6 hour – PMP temporal pattern

Table 9.21 Nicol Creek—1% AEP event estimated peak flow at key locations as applied in the hydraulic model

Sub-catchment name	1% AEP event peak flow (m ³ /s)	Critical storm (hrs) duration and temporal pattern
04T	45.8	9 hrs T9
05L	6.6	9 hrs T9
07L	19.9	9 hrs T9
06T	31.2	9 hrs T9
07T	89.2	9 hrs T9
08L	17.8	9 hrs T9
09L	17.2	9 hrs T9
10L	31.9	9 hrs T9

Sub-catchment name	1% AEP event peak flow (m ³ /s)	Critical storm (hrs) duration and temporal pattern
11L	17.1	9 hrs T9
12L	22.8	9 hrs T9
NIC	132.4	9 hrs T9

9.4.2 Existing Case flood maps

Existing conditions flood maps are presented in Volume 2 – Appendix E and via an [Online Digital Portal](#). Appendix E contains the Existing Case 20% and 1% AEP flood maps, while the [Online Digital Portal](#) contains the remainder of the modelled events. The location of each map set is summarised below:

- 20% AEP Peak Levels: Figure E2-A
- 1% AEP peak levels: Figure E2-B
- 20% AEP peak velocity: Figure E2-C
- 1% AEP peak velocity: Figure E2-D
- 20% AEP peak hazard: Figure E2-E
- 1% AEP peak hazard: Figure E2-F
- 20% AEP peak time of inundation: Figure E2-G
- 1% AEP peak time of inundation: Figure E2-H
- 20% AEP, 10% AEP, 5% AEP, 2% AEP, 1% AEP, 1 in 2,000 AEP, 1 in 10,000 AEP and PMF Existing Case results: [Online Digital Portal](#)
- For higher resolution mapping, refer to the [Online Digital Portal](#).

9.4.3 Existing Case flood inundation extent and flood levels

The peak 1% AEP event flood depth is about 2 to 4 m in the Nicol Creek channel. This depth reduces to an average flood depth of approximately 0.6 m in other areas of the floodplain, notably between Millmerran-Inglewood Road and the Project alignment.

The maximum 1% AEP flood depth on the floodplain is estimated to be up to 1.7 m (353.1 mAHD) where the Project alignment crosses the floodplain.

Figure E2-B in Volume 2 – Appendix E shows the 1% AEP indicative flood extent and peak water levels within the Nicol Creek floodplain for the Existing Case.

9.4.4 Existing Case flood immunity of existing infrastructure

Table 9.22 presents a summary of estimated overtopping depths for key roads near the Project alignment under a range of design events. Modelling results show that Millmerran-Inglewood Road has an existing low flood immunity in the areas close to the Project alignment.

Table 9.22 Nicol Creek—Existing Case: Overtopping depths of key infrastructure

Infrastructure	Overtopping depth (m)							
	PMF	1 in 10,000 AEP	1 in 2,000 AEP	1% AEP	2% AEP	5% AEP	10% AEP	20% AEP
Millmerran-Inglewood Road	5.18	3.48	2.63	2.33	2.16	1.97	1.92	1.46

9.4.5 Existing Case flood velocities

Existing peak 1% AEP event flood velocities are predicted to reach up to 2.8 m/s upstream of the proposed bridge crossing, and within Nicol Creek. For the 1% AEP flood event, flood water breaks out of the main channel downstream of Millmerran-Inglewood Road and enters the broader floodplain. The average velocity across the floodplain is approximately 0.25 m/s, refer E2-D in Volume 2 – Appendix E.

9.4.6 Existing Case flood hazard

For the 1% AEP event, a velocity-depth product of approximately 6 to 7 m²/s is recorded within Nicol Creek, approximately 200 m upstream of the proposed bridge crossing.

Excluding the channelised high-flow zones, flow along the floodplain north of Nicol Creek immediately upstream of the Project alignment has an average velocity-depth product of less than 0.5 m²/s, as shown in Figure E2-F in Volume 2 – Appendix E.

9.4.7 Existing Case flood duration of inundation

For the 1% AEP event, existing flood duration of inundation ranges from 5 to 11 hours for the 20% to 0.5% AEP flood events. Longer flood inundation is evident south of Paton Road, where flood water ponds against local bunds and depressions.

Due to the low flood immunity of Millmerran-Inglewood Road, significant ponding of water upstream of the road embankment is not evident, refer Figure E2-H in Volume 2 – Appendix E.

9.5 Developed Case modelling results—Nicol Creek

9.5.1 Drainage structures

The hydraulic design of the major drainage structures was undertaken using the TUFLOW model (1D and 2D approach).

In the Nicol Creek floodplain, the Project includes the following floodplain (or regional structures):

- One waterway bridge
- Four RCP locations (a total of 36 cells).

Design bridge details are summarised in Table 9.23, and details of the proposed culvert structures are summarised in the Cross-Drainage Register located in Appendix B. The location of proposed cross-drainage features is presented in Figure E1-F in Volume 2 – Appendix E.

Table 9.23 Nicol Creek—Proposed bridge location and details

Chainage (km)	Structure ID	Approximate span (m)	Deck width (m)	Soffit level (mAHD)	Deck superstructure type	Deck depth (mm)
104.39	310-BR11	92	3.97	353.90	Type D1	2,000

All bridges were modelled as an opening in the rail embankment. The optimisation of bridge lengths was balanced between minimising the changes to the hydraulic regime, primarily afflux and velocities, and the cost of replacing bridge spans by large earth embankments. As agreed with the Expert Flood Panel, bridge locations will be updated in detailed design to be modelled utilising LFC shapefiles.

Blockage of hydraulic structures was assessed in accordance with ARR 2019. The predominant vegetation in the catchment is considered heavily forested; however, given the appreciable size of the bridge spans, and the relative size of upstream debris, a blockage factor was not applied to bridges. Significant amounts of debris accumulating against the piers of the bridge are considered low.

9.5.2 Hydraulic design criteria outcomes

The hydraulic model was run for the Developed Case for the range of events (20%, 10%, 5%, 2%, 1% AEP events and 1 in 2,000 AEP, 1 in 10,000 AEP and PMF events) with the outcomes presented in the following sections.

9.5.2.1 Rail immunity and structures results

Appendix B summarises the peak 1% AEP water levels on the upstream side of the Project alignment.

The results of flood modelling indicate that a 1% AEP event flood immunity to the proposed rail formation is achieved for the Project alignment across the Nicol Creek floodplain, and that 1% AEP peak water levels remain below the proposed rail formation level. There is over 0.7 m freeboard to the rail formation level for the 1% AEP event, noting that the formation levels were generally dictated by other engineering criteria and constraints.

9.5.3 Flood Impact Objectives outcomes—Nicol Creek

The impact of the Developed Case was assessed through inclusion of the Project's revised reference design and comparison of model results against the Existing Case results.

Changes to flooding behaviour that were assessed, and are reported in the following sections, include potential changes in peak water levels, time of inundation, velocity, hazard, and peak flows.

- Changes in 20% AEP and 1% AEP peak water levels are presented in Figures E4-A and E4-B in Volume 2 – Appendix E
- Changes in 20% AEP and 1% AEP velocities are presented in Figures E4-C and E4-D in Volume 2 – Appendix E
- Changes in 20% AEP and 1% AEP hazard are presented in Figures E4-E and E4-F in Volume 2 – Appendix E
- Changes in 20% AEP and 1% AEP time of inundation are presented in Figures E4-G and E4-H in Volume 2 – Appendix E
- Changes in 20% AEP and 1% AEP velocities and percentage change in velocities with cut-offs applied are presented in Figures E5-A to E5-F in Volume 2 – Appendix E
- Percentage changes in 20% AEP and 1% AEP in hazard are presented in Figures E6-A and E6-B in Volume 2 – Appendix E
- Percentage changes in 20% AEP and 1% AEP in time of inundation and with cut-offs applied are presented in Figures E7-A and E7-D in Volume 2 – Appendix E
- Refer to the [Online Digital Portal](#) for other modelled AEP results and higher resolution mapping.

One-on-one consultation with landowners who are expected to experience changes in flooding behaviour on their property was conducted by ARTC, supported by FFJV. Refer to EIS Chapter 14: Flooding and Geomorphology and EIS Appendix E: Consultation Report for further details.

The Project design outcomes relative to the FIOs (Table 4.2) are presented in the following sections.

Flood sensitive receptors referred to in this section are shown in Figures P1 (1 to 59) in Volume 2 – Appendix P.

9.5.3.1 Flood impacts on flood sensitive receptors

Based on the available aerial imagery, no buildings or critical infrastructure are located within the area impacted by afflux and change in flood hazard in the Nicol Creek floodplain for all modelled events up to the 1 in 2,000 AEP event.

9.5.3.2 Flood impacts on State-controlled roads

The extent of the hydraulic model developed for the Nicol Creek catchment is shown in Figures E1-D in Volume 2 – Appendix E. Within the extent, the only State-controlled road which is influenced by flooding and the Project footprint is Millmerran-Inglewood Road.

Developed Case flooding from the Nicol Creek catchment and comparison against the FIOs has demonstrated that no adverse impacts on Millmerran-Inglewood Road are expected due to the presence of the Project for all modelled events up to and including the 1% AEP event.

9.5.3.3 Flood impacts on local public roads

The change in peak water levels, velocity, time of inundation and flood hazard (velocity-depth) for all modelled events up to the 1% AEP were evaluated on local public roads within the hydraulic model domain.

Developed Case flooding from the Nicol Creek catchment and comparison against the FIOs has demonstrated that no adverse impacts on local public roads are expected due to the presence of the Project, for all modelled events up to and including the 1% AEP event.

9.5.3.4 Flood impacts on existing rail infrastructure

No third-party rail infrastructure is located within the Nicol Creek hydraulic model boundary.

9.5.3.5 Flood impacts on private land outside the permanent footprint

In accordance with the FIOs, Table 9.24 presents the modelled changes and potential impacts on third-party property outside of the Project's permanent footprint. A summary of key impacts, exceedance justifications, and mitigation measures are provided below:

- Two properties were identified as having a potential impact exceeding the Project FIO criteria
- The exceedances are due to increases in the time of inundation duration (two properties), velocities and hazard (one property)
- For the 1% AEP flood event the following exceedances are noted:
 - Velocity increased by approximately 1.17 m/s
 - Time of inundation increased from 0.33 hours to 1.70 hours
 - Flood hazard increased by approximately 0.04 m²/s.
- Flood Impact Objective exceedances are localised to upstream and downstream of the Project alignment (within 500 m), and cross-drainage infrastructure. Exceedances are likely a result of the maturity of current cross-drainage design and limited available information (i.e. survey).
- Potential mitigation measures include further cross-drainage optimisation investigations and ongoing stakeholder communication.

Table 9.24 Nicol Creek – Summary of flood impacts on private land outside the permanent footprint

ID	ID (Lotplan ID/ ¹ FSR ID/ Culvert ID)	Sensitive Receptor Type	Catchment Type	Landuse ¹¹	Lot Area (m ²)	Flood Impact Objective Target							AEP	Maximum FIO Exceedance							Area (ha) ³				Exceedance Justification	Proposed Mitigation Measures ⁹																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																							
						Change in Peak Water Level (m)	Change in Peak Water Level (mm)	Change in Time of Inundation (%)	Change in Time of Inundation (Hours)	Change in Velocity (%)	Change in Velocity (m/s)	Change in Hazard (%)		Change in Peak Water Level (mm)	Change in Time of Inundation (%) ²	Change in Time of Inundation (hours)	Change in Velocity (%) ³	Change in Velocity (m/s)	Change in Hazard (%) ³	Afflux	Time on Inundation	Velocity	Hazard																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																										
1	2RP197967	Lot Type Parcel	Regional	GRANS	1,145,330	0.20	200	10	1	10	0.5	20	20%	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-

Table Note:

1. Floodplains are dynamic in nature and therefore model results vary from cell to cell. The "Area" is therefore defined as the maximum area over which change above the Flood Impact Objective occurs and does not relate directly to the maximum FIO exceedance recorded.

2. Change in water level (afflux) is calculated based on the maximum water surface level difference between the existing and design case results, and does not consider the underlying topography. Depth values presented are based on the respective underlying topographies and in some cases do not align with afflux calculations. This can be attributed to interpolation calculations of results performed within the hydraulic model.

3. Percentage increases should be read in conjunction with existing and design case results. The FIOs require that change be presented in percentage change for time of inundation, velocity and hazard and can sometimes produce large increases when existing case results are significantly small. Some locations were identified to experience significant percentage increases however the "absolute" difference between Existing and Design results are relatively small.

4. The Flood Impact Objective exceedance has been identified due to a change in flood extent and a cell which was dry in the existing case and now flood-affected in the developed case.

5. The above table should be read in conjunction with change maps provided as part of Volume II and the Digital Platform.

6. The symbol "-" denotes that no change above Flood Impact Objectives has been determined and therefore no result has been presented.

7. Changes recorded above may only occur during a small fraction of model simulation time.

8. Three decimal places do not infer absolute accuracy in modelling results, but it is necessary to identify small changes.

9. Flood impact objectives are assessed based on landuse applied to the entire lot and exceedances identified do not necessarily indicate an impact to a dwelling.

10. Design optimisations that could be implemented (but not limited to) include adjustment of culvert spacing, realignment of culverts to existing flowpaths when more detailed survey is available, modifying the grade of culverts, road height and alignment adjustments, removal of bridge abutments from waterway extents etc.

11. Landuse data, dated 2016, has been used for this assessment. Due to the changing nature of this dataset, this will be updated to the latest available information in Detailed Design.

Depth (m)		Time of Inundation in hours (related to % exceedance)		Time of Inundation in hours (related to actual change exceedance)		Velocity in m/s (related to % exceedance)		Velocity in m/s (related to actual change exceedance)		Hazard in m ³ /s (related to % exceedance)	
Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	0.98	1.07	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	1.10	1.66	-	-	-	-	-	-	-
-	-	1.33	2.29	0.72	1.13	-	-	-	1.17	0.13	0.17
-	-	-	-	-	-	-	-	-	-	-	-
-	-	1.18	2.09	0.72	1.10	-	-	-	-	-	-
-	-	4.49	8.58	0.59	2.09	-	-	-	-	-	-
-	-	1.03	5.05	0.88	4.42	-	-	-	-	-	-
-	-	1.03	4.48	0.92	3.29	-	-	-	-	-	-
-	-	1.08	6.06	0.33	1.70	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-

9.5.3.6 Flow distribution

A key landowner concern is the potential change in flow distribution. Maximum velocity-depth modelling results were used to create change in unit flow maps between Existing and Design Case conditions to determine the change in flow distribution. Unit flow is a measure of discharge that is commonly used for assessing peak flow along crossing-sections or to identify the efficient flow corridors within a floodplain.

The presented flow distribution maps were filtered with a velocity x depth product cut-off of 0.15 m²/s. The cut-off is in line with FIO requirements and was used to identify main channels (high-flow zones), and shallow lower flood zones (i.e. outer extents of the flood surface).

The following points summarise the change in flow distribution for Nicol Creek and should be considered in conjunction with flow distribution figures supplied in Figure Q5-1 to Figure Q5-8 of Volume 2 – Appendix Q:

- Minor flow decreases (upstream) and increases (downstream) are localised to cross-drainage infrastructure. This behaviour is expected due to the constriction of the floodway through the culvert and bridge openings. This is particularly evident for all flood events up to the 1% AEP event, where the Project alignment is not overtopped.
- While the Project does impact flow conditions immediately upstream and downstream of the alignment, regional conditions remain relatively unchanged.

Ongoing mitigation measures to reduce changes in flow distribution are likely to occur during future design stages of the Project, and may include further refinement of the bridge abutments and additional survey where necessary.

9.5.4 Extreme events

9.5.4.1 Extreme event immunity

The risk of overtopping of the Project alignment was assessed for a range of extreme events (1 in 2,000 AEP, 1 in 10,000 AEP and PMF). Table 9.25 presents the estimated depth of water above the rail formation level and over the top of rail at each structure. It is noted that the function of the floodplain culverts is to balance flood levels on the upstream and downstream sides of the alignment. As such, overtopping of the rail is not predicted to result in significant excessive flows or velocities as would occur in a dam embankment overtopping scenario. Further analysis of the overtopping risk to sensitive receptors can be found in Section 21.

Table 9.25 Nicol Creek—Extreme events: Depth of water above formation level and over top or rail level

Chainage (km)	Structure ID	Depth of water above formation level (m)			Depth of water above top of rail (m)		
		1 in 2,000 AEP	1 in 10,000 AEP	PMF	1 in 2,000 AEP	1 in 10,000 AEP	PMF
104.4	310-BR11	-	-	1.08	-	-	0.38
104.94	C104.94	-	0.17	1.42	-	-	0.72
105.09	C105.09	-	-	0.75	-	-	0.05
105.11	C105.11	-	-	0.50	-	-	-
105.13	C105.13	-	-	0.50	-	-	-

9.5.4.2 Impacts during extreme events

Flood immunity of FSRs is discussed in Section 9.5.3.1, and maps demonstrating the impacts during extreme events are shown on the [Online Digital Portal](#).

In accordance with the FIOs, FSRs are considered impacted by extreme flood events if the change in water level is greater than 250 mm in the 1 in 2,000 AEP event, subject to the receptor classification (i.e. at existing habitable dwellings and/or commercial and industrial buildings/premises). In addition, a stricter afflux objective of 10 mm in the 1 in 500 AEP event applies to sites with existing sensitive infrastructure, uses involving vulnerable people and/or any critical road network that was designed to be immune to flooding in any extreme event.

Review of the 1 in 2,000 AEP event results for the Nicol Creek floodplain against the FIOs show that flood level changes at all FSRs were limited to 250 mm, and no identified existing sensitive infrastructure or uses involving vulnerable people are expected to experience a more than 10 mm increase in flood level. A separate assessment of the 1 in 500 AEP event is not required as they are conservatively represented by the 1 in 2,000 AEP event results. The flood impacts in extreme events due to the Project are therefore compliant against the stipulated FIO limits for extreme events.

Table 9.26 details extreme event results, although it should be noted that FIO targets do not apply to the 1 in 10,000 AEP and PMF events.

Table 9.26 Nicol Creek—Extreme events: Impacts at flood sensitive receptors

FSR ID	FSR type	1 in 2,000 AEP event		1 in 10,000 AEP event		PMF event	
		Change in peak water level (mm)	Existing Case flood depth (m)	Change in peak water level (mm)	Existing Case flood depth (m)	Change in peak water level (mm)	Existing Case flood depth (m)
NIC_ID_1	House					-	0.11
NIC_ID_2	Shed					-	0.16
NIC_ID_3	Shed					-	0.46
NIC_ID_4	Shed					-	1.09
NIC_ID_5	Shed					-	0.72
NIC_ID_9	Shed			-	0.60	-	2.09
NIC_ID_10	Shed			-	0.61	+10	2.09
NIC_ID_11	House			-	0.32	+10	1.75
NIC_ID_12	Shed	-	0.01	+10	0.66	+190	2.07
NIC_ID_13	Shed	-	0.50	+20	1.17	+200	2.64
NIC_ID_14	Shed			+10	0.68	+110	2.22
NIC_ID_15	Shed					+80	0.28
NIC_ID_16	Silos			-	0.31	+10	1.76
NIC_ID_17	Shed					-	0.45
NIC_ID_18	Shed					-	1.08
NIC_ID_19	Silos					-	0.71
NIC_ID_21	Silos					-	0.57
NIC_ID_20	House			-	0.26	-	1.48
NIC_ID_22	Shed					-	0.86
NIC_ID_23	Shed					-	0.88
NIC_ID_24	House					-	0.79
NIC_ID_25	Shed			-	0.22	-	1.38
NIC_ID_26	House			-	0.22	-	1.19
NIC_ID_27	Shed	-	0.12	-	0.35	-	1.13
NIC_ID_28	Shed	-	0.06	-	0.23	-	1.53
NIC_ID_29	Silos	-	0.09	-	0.29	-	1.31

FSR ID	FSR type	1 in 2,000 AEP event		1 in 10,000 AEP event		PMF event	
		Change in peak water level (mm)	Existing Case flood depth (m)	Change in peak water level (mm)	Existing Case flood depth (m)	Change in peak water level (mm)	Existing Case flood depth (m)
NIC_ID_30	House					-	0.72
NIC_ID_32	Private Access	-	0.61	-	1.40	-	2.80
NIC_ID_33	Shed					-	0.24
NIC_ID_35	House			-	0.07	-	0.37
NIC_ID_46	Shed					-	0.43

9.5.5 Sensitivity analysis—Nicol Creek

The sensitivity of the model to various parameters was assessed using the following two scenarios:

- An increase in rainfall intensity (i.e. to reflect climate change scenario)
- An increase and decrease in blockage of culverts from 25 to 50 per cent and 0 per cent.

9.5.5.1 Climate change

The potential impacts on FSRs locations were assessed for the 1% AEP climate change event, to determine the Project's long-term hydraulic performance and sensitivity to changes in climate. The assessment was undertaken in accordance with ARR 2019.

The Representative Concentration Pathways 8.5 (2090 horizon) climate change scenario was adopted for the Project, with an associated increase in rainfall intensity of 23.9 per cent across the catchment area.

The Project is expected to retain a 1% AEP flood immunity, with over 0.5 m of freeboard to formation level under the climate change scenario.

The climate change sensitivity result indicates there have been no changes in peak water level for the 1% AEP climate change event, as shown in Table 9.27. Note only FSR locations that recorded a flood depth during design and existing conditions were analysed as part of this sensitivity assessment.

Table 9.27 Nicol Creek—1% AEP event with Representative Concentration Pathways 8.5 conditions: Structure performance

FSR ID	FSR ID type	1% AEP climate change event	
		Change in peak water level (mm)	Existing Case flood depth (m)
NIC_ID_13	Shed	-	0.38
NIC_ID_27	Shed	-	0.08
NIC_ID_29	Silos	-	0.06
NIC_ID_30	Private access road	-	0.50

For the 1% AEP event, the change in peak water levels for the Representative Concentration Pathways 8.5 climate change scenario is presented in Figure E8-C in Volume 2 – Appendix E.

9.5.5.2 Blockage

Blockage was assessed in accordance with ARR 2019. The blockage assessment was undertaken and resulted in a blockage factor of 25 per cent adopted for culverts. A minimum culvert size of 900 mm diameter was adopted to reduce potential for blockage and maintenance. A significant community concern is the potential impacts to flood conditions should the proposed culverts become blocked with debris. The primary concern is the blockage of culverts, which is likely to drive flood levels higher, particularly upstream of the culverts, and divert more flow through residences, across access roads and other infrastructure. A sensitivity analysis was undertaken with 0 and 50 per cent blockage on culverts.

In addition, the culvert blockage sensitivity assessment results indicated that FSRs have not been impacted by blockage, as detailed in Table 9.28.

During detailed design, the blockage factors will be reviewed in line with the final design and local catchment conditions. This may result in a varied and/or lower blockage factors being applied along the Project alignment. Should impacts be determined due to a change in blockage factor, additional mitigation measures will be considered in detailed design.

Table 9.28 Nicol Creek—1% AEP event: Culvert blockage assessment

FSR ID	FSR type	Existing Case flood depth (m)	1% AEP afflux (mm)		
			0% blockage	Developed Case (25% blockage)	50% blockage
NIC_ID_13	Shed	0.26	-	-	-
NIC_ID_27	Shed	0.06	-	-	-
NIC_ID_29	Shed	0.04	-	-	-
NIC_ID_32	Shed	0.41	-	-	-

For the 1% AEP event, the change in peak water levels for the 0 and 50 per cent blockage scenarios are presented in Figures E8-A and E8-B in Volume 2 – Appendix E.

9.6 Additional modelling items to be considered in detailed design

The following points summarise additional items to be included in the flood models for detailed design as per agreement with the Expert Flood Panel:

- Section 9.2.1 shows that the peak flow estimates are approximately 20 per cent less than the FFA estimates. The Nicol Creek URBS models will be reviewed and updated as appropriate at detailed design to better align with FFA estimates.
- Undertake additional model strengthening by improving the representation of cross-drainage infrastructure, third-party infrastructure, noise barriers, survey data, and bridge structures within the hydraulic model. Specifically, the modelling of bridges will be included as LFC shape files within the hydraulic model.
- A review of bridge losses will be undertaken to verify against alternative methods
- Where flood exceedances have been identified, additional analysis of cross-drainage infrastructure, and other mitigation options may be investigated to reduce impacts. Land and stakeholder consultation will also form part of the mitigation process.
- A review of the latest TUFLOW version will be undertaken at the commencement of detailed design to determine whether updates of the TUFLOW version from 2020-10-AD-iSP-w64 are beneficial to modelling.

Further details of mitigation measures can be found in Section 22. A review of proposed mitigation measures will be completed during the detailed design stage of the Project after hydraulic models have been updated to capture the above changes.

9.7 Planned and future roads

Consultation with the DTMR has not identified any planned or future road upgrades within the Nicol Creek catchment. ARTC will continue to consult and engage with DTMR to determine any future or planned road projects that may need to be considered as part of the detailed design hydraulic modelling.

TRC has not identified any planned or future roads within the Nicol Creek hydraulic model domain. ARTC will continue to consult and engage with TRC to determine any future or planned road projects that may need to be considered as part of the detailed design hydraulic modelling.

10 Bringalily Creek

Bringalily Creek is a well-defined water course with high sinuosity and is an upstream tributary of Canning Creek. Under the Existing Case 1% AEP event, the flood depth in Bringalily Creek channel is up to approximately 7.5 m. On the floodplain, in the vicinity of the Project alignment, flood depths range from 2 to 5 m. The Bringalily Creek flood inundation extent, under the Existing Case 1% AEP event, varies between 500 m and 1.2 km wide.

The location of the Project alignment in relation to Bringalily Creek Study area is shown in Figure F1-A in Volume 2 – Appendix F.

10.1 Data collation and review—Bringalily Creek

Available background information including existing hydrologic and hydraulic models, survey, streamflow data, available calibration information and anecdotal flood data has been gathered. Data was sourced from a range of stakeholders, including:

- GRC—existing flood studies
- BoM—rainfall gauging data
- DTMR—existing infrastructure details.

10.1.1 Previous studies

Previous hydrology and hydraulic studies were sourced as part of this assessment. A review of each study was undertaken to determine suitability for use on the Project, as documented in the following sections.

10.1.1.1 Inglewood Flood Study, Engeny, 2015

Engeny was commissioned by GRC to undertake a flood study of Inglewood. The study objectives were ‘to define the nature, extent and risks of flooding in Inglewood in order to inform disaster management planning and response, as well as control future development.’ (Engeny, 2015). An URBS hydrologic model and TUFLOW (1D/2D) hydraulic model were developed.

10.1.2 Survey

ARTC provided LiDAR data from 2015 as 1 m grid DEM tiles. Using GIS software, a DEM was generated with a 1 m grid resolution for use in the Project based on the 2015 dataset. This was used for modelling within the permanent footprint and up to the full extent of the 2015 LiDAR where relevant. In areas that were not covered by the LiDAR provided by ARTC, LiDAR tiles were sourced from Geoscience Australia. The DEM datasets used for modelling were based on surveys flown between 2009 and 2015, with preference given to the most recent data available. SRTM data was used for catchment delineation where no LiDAR data could be sourced, to inform the hydrologic modelling.

The survey data sources, and DEM developed for Nicol Creek are shown in Figure F1-B of Volume 2 – Appendix F.

10.1.3 Aerial imagery

Aerial imagery of the Bringalily Creek Study area was provided by ARTC and was used to identify and confirm topographic and vegetative characteristics of the study area. Aerial imagery captured in 2015 was made available. Additional imagery outside the study area was sourced from QGIS imagery in an open-source format.

10.1.4 Existing drainage structure

No available DTMR as-constructed drawings were available for culvert and bridge details.

Additional information will be refined as local survey is progressively completed during detailed design. Details of infrastructure included within the Bringalily Creek flood assessment are in Section 10.3.4.

10.1.5 Stream gauge data

There are no stream gauges within the Bringalily Creek catchment study area assessed as part of this investigation.

10.1.6 Rainfall data

Several daily and sub-daily rainfall stations are located in and around Bringalily Creek catchment. However, since there are no streamflow gauges to use for model calibration, no historical rainfall data was sought.

10.1.7 Anecdotal and observed flood data

No anecdotal or observed flood data was available for this section of the Bringalily Creek Catchment.

10.1.8 Site inspection

A site inspection was undertaken during February 2018. During the site inspection, all major waterway crossings were visited and inspected with photographs taken and details recorded of the crossing, existing drainage structures and surrounding catchment. An assessment of the relative roughness and blockage potential was undertaken during the site inspection.

10.2 Hydrologic model development—Bringalily Creek

10.2.1 Model setup

Bringalily Creek is one of four major waterways, which cross the Project alignment before flowing into Canning Creek system. The other waterways include Nicol Creek, Native Dog Creek and Cattle Creek. Due to their proximity to one another, all four waterways were modelled in a unified Canning Creek model.

The Project alignment crosses two major watercourses in the vicinity of Bringalily Creek. The catchment area upstream of the main Bringalily Creek crossing (310-BR10) is approximately 188 km². The catchment area upstream of the Project alignment crossing (310-BR08) of Bringalily Creek's minor tributary is approximately 14.5 km². This tributary is TW affected by flooding on Bringalily Creek generated by approximately 85 km² of upstream catchment.

In addition to the below sections, refer to Section 9.2 (Nicol Creek Catchment) for further details on the development of the hydrologic model and associated modelling parameters.

10.2.2 Sub-catchments

The Bringalily catchment is comprised of approximately 10 sub-catchments and an approximate catchment area of 289 km².

The hydrologic model extent and sub-catchment map is presented in Figure F1-C in Volume 2 – Appendix F.

10.2.3 Design model parameters

Hydrologic information to assist estimation of design event flows was sourced from the ARR 2019 Data Hub as summarised in Table 10.1. Details of the adopted hydrologic model parameters are presented below.

Table 10.1 Bringalily Creek—Summary of URBS model design event inputs

Input parameter	Remarks
Design rainfall	Intensity-Frequency-Durations for each sub-catchment were downloaded from the BoM's website to account for variation in rainfall across the catchment.
Extreme event rainfall	Probable Maximum Precipitation depths for durations up to 6 hours (for use in modelling the PMF event) were obtained using the method presented in the Bulletin 53 (BoM, 2003). The rainfall depths for the 1 in 10,000 AEP event were estimated using the interpolation method presented in ARR 2019 Book 8 Section 3.5.
Losses	The losses were initially adopted from the <i>Inglewood Flood Study</i> (Engeny, 2015) URBS model (Initial loss – 15 mm, Continuing loss – 1 mm/hr). Adjustments were made to reconcile URBS flows to area-scaled FFA.
Areal reduction factor	Parameters were adopted for the semi-arid inland Queensland region. The catchment area of Bringalily Creek reporting to the Project alignment is approximately 188 km ² (total catchment is 289 km ²), which yields an ARF between 62% to 94% depending on design storm event AEP and duration.
Ensemble TPs	Central Slopes regions. However, as the study catchment area exceeds 75 km ² , the standard ensemble rainfall patterns from ARR 2019 do not apply to this catchment for storm durations longer than 12h. These were replaced with the areal TPs for the Central Slopes region.
Pre-burst depths	Median pre-burst depths were downloaded from the ARR 2019 Data Hub for each sub-catchment. Pre-burst depths vary by design storm event AEP and duration. Pre-burst depths were applied to the model by reducing the initial losses for each storm event.

10.2.3.1 Spatial distribution

The ARR 2019 design event methodology was adopted for this assessment. ARR 2019 recommends adopting the spatially varying method for catchments larger than 20 km². The Bringalily Creek catchment is approximately 289 km². A total of eight sub-catchment were assigned an IFD derived from BoM website.

10.2.3.2 Rainfall depth estimation

Design rainfall depth for 20% to 1 in 2,000 AEP were manually obtained from BoM website for 8 of the 10 Bringalily sub-catchments. As an example, the IFD representing the catchment centroid of Bringalily Creek URBS model is provided in Table 10.2. Due to the size of the model area, there is minimal difference in rainfall depth across the catchment.

Table 10.2 Rainfall depth (mm) used for the Bringalily Creek catchment centroid

Duration (hr)	Annual Exceedance Probability (% AEP)						
	50	20	10	5	2	1	0.05
0.25	17.6	24.5	29.2	33.8	40.1	44.9	70.4
0.50	24	33.5	40	46.5	55.2	62	97.3
1.00	30.4	42.3	50.6	58.9	70.1	78.8	124
2.00	36.7	50.7	60.5	70.4	83.8	94.3	148
3.00	40.6	55.6	66.3	77	91.6	103	161
4.50	44.9	61	72.3	83.8	99.6	112	175
6.00	48.2	65.1	77.1	89.1	106	119	186
12.00	57.6	77.1	90.9	105	124	140	218
24.00	69.4	92.9	110	126	150	170	266

Duration (hr)	Annual Exceedance Probability (% AEP)						
	50	20	10	5	2	1	0.05
48.00	83	112	134	155	186	210	346
72.00	91	124	149	174	208	235	378

10.2.3.3 Areal reduction factor

The ARF was estimated using the coefficients for semi-arid inland Queensland presented in Table 10.3 and was applied to all IFD areas.

The URBS software directly downloaded these coefficients from ARR 2019 Data Hub to estimate the ARF using Equation 2.4.1 of ARR 2019 Book 2.

The estimated ARF for Bringalily Creek catchment vary between 62 to 94 per cent depending on design storm event AEP and duration.

Table 10.3 Bringalily Creek—Areal reduction factor coefficients from ARR 2019 Data Hub

a	b	c	d	e	f	g	h	i
0.159	0.283	0.25	0.308	7.30E-07	1.0	0.039	0.0	0.0

10.2.3.4 Temporal patterns

Temporal patterns for 20% AEP to 1 in 2,000 AEP events were adopted based on 10 burst ensemble approach presented in the ARR 2019. The TPs and pre-burst data were directly downloaded by URBS software from the ARR 2019 Data Hub for the Bringalily Creek catchment.

Temporal patterns for the 1 in 10,000 AEP and PMF design events for duration up to 6 hours were obtained from *The Estimation of Probable Maximum Precipitation in Australia: Generalised Short-Duration Method* (BoM, 2003).

The adopted TPs for all modelled events are shown in Table 10.4.

Table 10.4 Bringalily Creek – Adopted temporal patterns

Design Event	Adopted temporal pattern
20% AEP	Pattern 4
10% AEP	Pattern 6
5% AEP	Pattern 6
2% AEP	Pattern 0
1% AEP	Pattern 9
1 in 2,000 year (0.05%) AEP	Pattern 9
1 in 10,000 year (0.01%) AEP	PMP Temporal Pattern
PMF	PMP Temporal Pattern

10.2.3.5 Pre-burst

Median pre-burst depths for standard durations from 0.5 hour to 72 hours for a 1% AEP design event are between 1.1 mm and 13.1 mm (ARR 2019 Data Hub). Due to the small depth of pre-burst in this area, the impact of pre-burst condition is considered to be minimal in this catchment. The adopted pre-burst depths are summarised in Table 10.5.

Table 10.5 Bringalily Creek—Adopted pre-burst depths

Duration (hrs)	Pre-burst depth (mm)					
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP
0.5	0.6	0.8	0.9	1.1	1.7	2.3
0.75	0.6	0.8	0.9	1.1	1.7	2.3
1.0	0.6	0.8	0.9	1.1	1.7	2.3
1.5	0.2	0.3	0.4	0.5	1.6	2.4
2.0	0.2	0.3	0.3	0.3	1.7	2.8
3.0	0.1	0.0	0.0	0.0	2.8	5.0
6.0	0.2	0.6	0.8	1.1	1.1	1.1
12.0	0.0	0.2	0.2	0.3	5.2	8.8
18.0	0.0	0.2	0.2	0.3	5.2	8.8
24.0	0.0	0.3	0.5	0.7	7.8	13.1
48.0	0.0	0.0	0.0	0.0	0.1	0.2
72.0	0.0	0.0	0.0	0.0	0.0	0.0

10.2.3.6 Design rainfall losses

In this assessment, an initial and continuing loss model was applied, where the lost rainfall depth does not contribute to the catchment runoff. In the loss model, it was assumed that there is an initial loss of rainfall depth followed by a constant continuing loss (mm/hr) throughout the rainfall event

A range of design rainfall losses were trialled for the design events. Initial and continuing losses in the URBS model were adjusted until there was a reasonable match between the modelled flows and those from the scaled FFA. The adjustment of rainfall losses are detailed in Section 9.2.1.1

The adopted design event losses are provided in Table 10.6.

Table 10.6 Bringalily Creek—Adopted design event losses

AEP	Initial storm loss (mm)	Continuing loss (mm/hr)
20%	42.3	2.8
10%	38.6	2.6
5%	32.6	2.2
2%	22.3	1.5
1%	6.4	0.4

10.2.3.7 Extreme event rainfall

Probable Maximum Precipitation depths for durations up to 6 hours (for use in modelling the PMF event) were obtained using the methodology presented in *The Estimation of Probable Maximum Precipitation in Australia: Generalised Short-Duration Method* (BoM, 2003). Once the PMP was estimated, rainfall for events between 1 in 2,000 AEP and the PMF events was then interpolated. For the 1 in 10,000 AEP event, ARF were applied based on interpolation methodology presented in ARR 2019 Section 3.5 in Book 8 and the rainfall depths were estimated accordingly. The parameters adopted for PMP rainfall estimation are presented in Table 10.7. The adopted extreme rainfall depths are provided in Table 10.8.

Table 10.7 Bringalily Creek—Input data for Probable Maximum Precipitation estimation

Factor	Value
EAF	1
MAF	0.8
Portion of rough catchment	100%
Bringalily Creek location and Catchment Area (km ²)	(-28.080, 151.258) and 188.1 km ²

Table 10.8 Bringalily Creek—Adopted extreme event rainfall depths (mm)

Duration (hour)	Bringalily Creek	
	1 in 10,000 AEP rainfall depth (mm)	PMP rainfall depth (mm)
0.25	87	125
0.5	120	184
0.75	142	234
1	152	281
1.5	172	365
2	182	421
2.5	193	475
3	201	515
4	211	586
5	221	634
6	232	675

10.2.4 Event selection for calibration

Insufficient data was available to calibrate historical flood events in the Bringalily Creek catchment.

10.2.5 Model calibration

Insufficient data was available to calibrate historical flood events in the Bringalily Creek catchment.

10.2.6 Model validation

The flood flows in Bringalily Creek were estimated from the Canning Creek model with the ARF amended to suit the Bringalily Creek catchment.

All flows produced by the URBS model between the 20% AEP and 1% AEP events reside within the 90 per cent confidence limits of the RFFE and show a close match with QRT estimates. The estimated flood flows at the proposed Bringalily Creek crossing (310-BR10) are presented in Figure 10.1 and Table 10.9.

Further detail of model validation is provided in the hydrologic model validation for Nicol Creek catchment (Section 9.2.2.3).

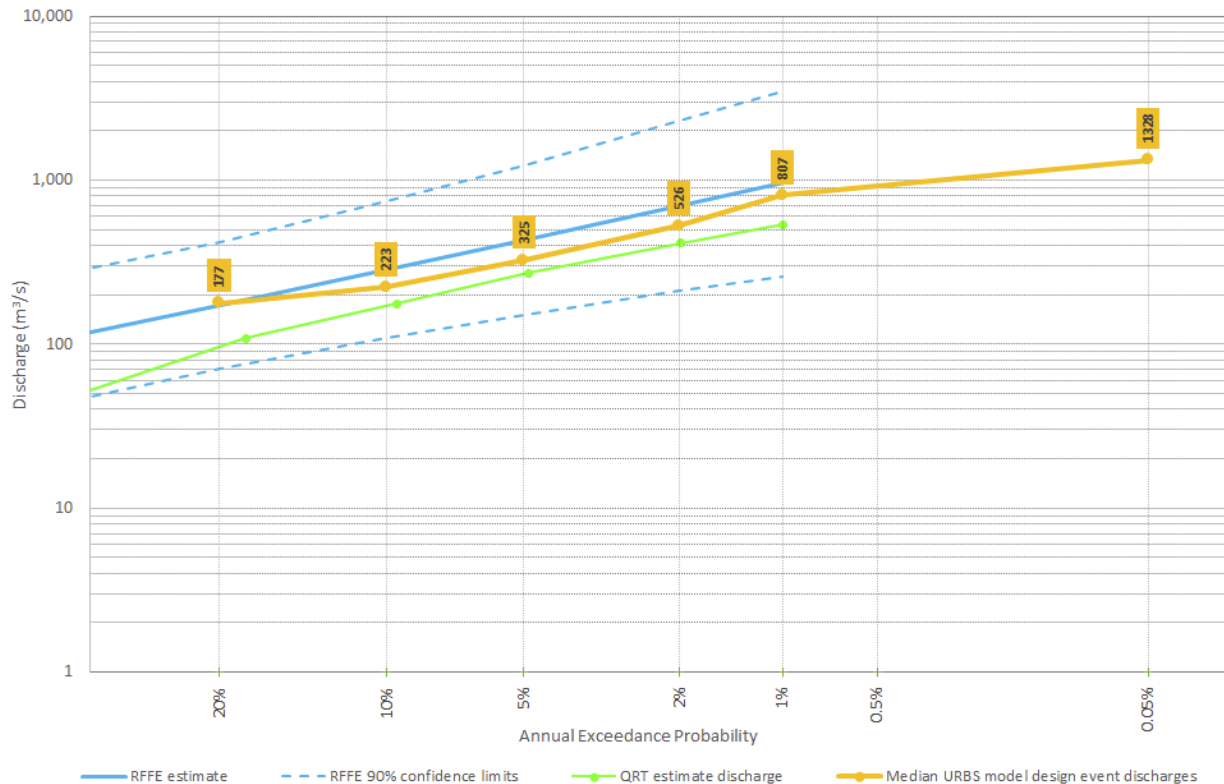


Figure 10.1 Estimate of flows at the Bringalily Creek crossing

Table 10.9 Estimate of flows at the Bringalily Creek crossing

AEP (%)	RFFE – lower bound 90% confidence level (m³/s)	RFFE – estimate of flow (m³/s)	RFFE – upper bound 90% confidence level (m³/s)	DTMR quantile regression technique (m³/s) ¹	URBS model flows (m³/s)
20	71	172	422	109	177
10	109	284	741	176	223
5	150	431	1,220	271	325
2	210	694	2,280	411	526
1	259	957	3,490	534	807
1 in 2,000	-	-	-	-	1,328
1 in 10,000	-	-	-	-	2,678
PMF	-	-	-	-	8,552

Table note:

1 The QRT method estimates the 39.3%, 18.1%, 9.5% and 4.9% AEP instead of 50%, 20% and 10% and 5%, respectively

10.3 Hydraulic model development—Bringalily Creek

A 2D modelling approach was adopted to appropriately simulate flood mechanisms around the Project alignment crossing on Bringalily Creek. The platform used for hydraulic modelling is the TUFLOW HPC software package. The processes and assumptions adopted throughout the development of the hydraulic model are described in the following sections.

10.3.1 Model setup

The setup of the TUFLOW model is summarised in Table 10.10.

Table 10.10 Bringalily Creek—hydraulic model summary

Parameter	Information
Completion date	August 2022
AEPs assessed	20%, 10%, 5%, 2%, 1%, 1 in 2,000, 1 in 10,000 AEP and PMF
Hydraulic model build	TUFLOW HPC GPU – version 2020-10-AD-iSP-w64
Model extent	Refer to Figure F1-D in Volume 2 – Appendix F
Grid size	5 m
SGS distance	1 m
QT grid size	2.5 m, 2 nd Order
DEM (year flown)	ARTC dataset (2015)
Roughness	Spatially varying roughness values compliant with industry norms.
Eddy viscosity	Wu (default)
Model calibration	N/A
D/S model boundary	Height-Discharge (HQ) Boundary with normal slope approximated based upon topography dataset.
Hydraulic model timestep	Adaptive Timestep
Modelled scenarios	Existing Case, Developed Case
Sensitivity analysis	Climate change and blockage (0% and 50% Blockage on culverts)

The hydraulic model extent and the spatial distribution of land use in the 2D model domain is presented in Figure F1-D and F1-E in Volume 2 – Appendix F, respectively.

10.3.2 Model topography and grid size

The topography within the TUFLOW hydraulic model was based on 1 m DEM data collected during 2015, which was supplied by ARTC. A 5 m x 5 m grid size was selected to allow the features of the channels and floodplain to be represented with sufficient accuracy while maintaining efficient model run times. The 5 m x 5 m grid size is considered adequate to represent floodplain topography and is consistent with the approach typically adopted in similar flood studies. The 2D model topography was modified adjacent to the culverts so that the existing drainage flows, existing road/rail crowns and levee banks were represented accurately in the model. For the Developed Case, the topography was modified to include the Project rail embankment, drainage structures and associated infrastructure.

10.3.2.1 Sub-grid sampling and Quadtree mesh refinement

The Bringalily Creek hydraulic model was established using TUFLOW's 2020-10-AD model version, including SGS and QT add-on modules. The below points provide a brief description of each module, along with the key parameters adopted for this assessment.

- Traditional grid-mesh based 2D hydraulic models sample the underlying DEM by taking a single elevation at the cell centres and cell faces. Sub-grid sampling allows for increased resolution by sampling a greater number of points of the underlying DEM to establish a water surface elevation versus volume relationship for each grid, along with a water surface elevation versus width relationship (wetted perimeter) at cell sides. The adoption of SGS allows increased representation of storage and conveyance between cells. A SGS distance of 1 m was adopted for this assessment.

- Quadtree mesh is constructed by subdividing the base mesh-grid size into four smaller square cells. Cells can be further sub-divided into four, and so on, allowing for additional resolution in location of interest. The base hydraulic model grid cell size (5 m x 5 m) was revised to include additional spatial resolution of approximately 2.5 m, to better represent model velocity at the entry and exit of culverts. Quadtree meshes were applied in the model covering minimum 25 m upstream and 100 m downstream of the design culvert locations, and culverts within the permanent footprint.

10.3.3 Roughness

The hydraulic roughness generally reflects the types of development and ground cover that exists within the hydraulic model extent. The distribution of roughness categories adopted for this assessment was based on the information supplied in the land use layer from Queensland Globe database, aerial imagery and confirmed during the site inspection. Specific roughness values applied to the model are detailed in Table 10.11.

Table 10.11 Bringalily Creek catchment —Adopted roughness values

Category	Manning's 'n' value
River	0.035
Sealed roads	0.015
Unsealed roads	0.025
Rural residential	0.09
Urban residential	0.10
Commercial	0.06
Open space—Minimum vegetation	0.045
Open space—Moderate vegetation	0.055
Open space—Heavy vegetation	0.065
Open water	0.025
Rail	0.025
Riparian	0.04
Crops ¹	0.1085

Table note:

- 1 A uniform value was adopted for the crop areas, due to the seasonal and spatial variations in vegetation distribution and the uncertainty this brings to the design event approach

10.3.4 Hydraulic structures

The following existing structures were incorporated into the hydraulic model:

- One existing bridge on Millmerran-Inglewood Road (DTMR bridge 246) modelled as an opening
- Eight existing culverts on Millmerran-Inglewood Road.

In the absence of as-constructed data, the dimensions and number of culvert barrels were estimated from aerial photography. Details of existing culverts are summarised in Table 10.12.

Table 10.12 Bringalily Creek—Existing culverts within the hydraulic model extent

Structure ID	Type	Cells	Width (mm)	Height (mm)	Length (m)	U/S invert (mAHD)	D/S invert (mAHD)
TMR_001	RCBC	6	1800	600	8.0	329.22	329.01
TMR_002	RCBC	3	1200	600	8.4	331.78	331.64
TMR_003	RCBC	1	1800	600	14.4	331.67	331.52

Structure ID	Type	Cells	Width (mm)	Height (mm)	Length (m)	U/S invert (mAHD)	D/S invert (mAHD)
TMR_004	RCBC	2	1200	450	9.0	332.13	332.08
TMR_005	RCBC	2	1200	450	8.7	332.19	331.99

Approximated details of DTMR bridge are included in Table 10.13.

Table 10.13 Bringalily Creek—Existing bridge

Bridge ID	Approximate span (m)	Deck width (m)	Deck level (mAHD)	Deck superstructure type	Deck depth (mm)
TMR 246	30	8.60 ¹	332.9	H20S16	1,050

Table note:

¹ Estimated from aerial imagery

10.4 Existing Case modelling results—Bringalily Creek

10.4.1 Hydrological peak flow assessment

A critical duration assessment was undertaken to determine which storm duration(s) produced peak flood flows where the major waterways are intersected by the Project alignment and at the downstream outlet of the model. To assess the critical storm duration the following methodology was adopted:

- Modelling was completed for a range of AEP events:
 - Each AEP was modelled for a range of durations
 - Each duration was modelled for each of the 10 associated TPs.
- A critical duration assessment was undertaken at the locations mentioned above to determine which duration produced the highest median flow of the 10 TPs for each event. The critical duration and adopted TP for each design event AEP is provided in Table 10.14. Table 10.15 also demonstrates the peak flow for the 1% AEP event.

Table 10.14 Bringalily Creek—Critical flood durations and temporal patterns

Design Event	Critical storm (hrs) duration and temporal pattern
20% AEP	12 hours – Pattern 4
10% AEP	12 hours – Pattern 6
5% AEP	12 hours – Pattern 6
2% AEP	12 hours – Pattern 0
1% AEP	9 hours – Pattern 9
1 in 2,000 year (0.05%) AEP	9 hours – Pattern 9
1 in 10,000 year (0.01%) AEP	6 hours – PMP Temporal Pattern
PMF	6 hours – PMP Temporal Pattern

Table 10.15 Bringalily Creek—1% AEP event peak flow at key locations as applied in the hydraulic model

Sub-catchment name	1% AEP event peak flow (m ³ /s)	Critical storm (hrs) duration and temporal pattern
03T	640.2	9 hours – Pattern 9
06L	128.5	9 hours – Pattern 9
07L	187.0	9 hours – Pattern 9
04L	21.3	9 hours – Pattern 9
05L	4.9	9 hours – Pattern 9

Sub-catchment name	1% AEP event peak flow (m ³ /s)	Critical storm (hrs) duration and temporal pattern
09L	66.6	9 hours – Pattern 9
10L	9.0	9 hours – Pattern 9
08L	78.8	9 hours – Pattern 9
08T	698.4	9 hours – Pattern 9

10.4.2 Existing Case flood maps

Existing conditions flood maps are presented in Volume 2 – Appendix F and via an [Online Digital Portal](#). Appendix F contains the Existing Case 20% and 1% AEP flood maps, whilst the [Online Digital Portal](#) contains the remainder of the modelled events. The location of each map set is summarised below:

- 20% AEP peak levels: Figure F2-A
- 1% AEP peak levels: Figure F2-B
- 20% AEP peak velocity: Figure F2-C
- 1% AEP peak velocity: Figure F2-D
- 20% AEP peak hazard: Figure F2-E
- 1% AEP peak hazard: Figure F2-F
- 20% AEP peak time of inundation: Figure F2-G
- 1% AEP peak time of inundation: Figure F2-H
- 20% AEP, 10% AEP, 5% AEP, 2% AEP, 1% AEP, 1 in 2,000 AEP, 1 in 10,000 AEP and PMF Existing Case results: [Online Digital Portal](#)
- For higher resolution mapping refer to the [Online Digital Portal](#).

10.4.3 Existing Case flood inundation extent and flood levels

The peak 1% AEP event modelled flood depth is approximately 7.5 m within the main Bringalily Creek channel. This depth reduces to an average modelled flood depth of around 1.0 m in other areas of the floodplain. The peak flood depth on the floodplain is estimated to be up to 5.5 m (333.8 mAHD) and 4.1 m (328.6 mAHD) where the Project bridges 310-BR10 and 310-BR08 cross the floodplain, respectively.

Figure F2-B in Volume 2– Appendix F shows the 1% AEP event indicative flood extent and peak water levels within the Bringalily Creek floodplain for the Existing Case.

10.4.4 Flood immunity of existing infrastructure

Table 10.16 presents a summary of overtopping depths for key roads near the Project alignment under a range of design events. Modelling results show that Millmerran-Inglewood Road along with Heckels Road and Forestry Road have a low existing flood immunity in the areas close to the Project alignment and overtops in five key locations.

Table 10.16 Bringalily Creek—Existing Case: Overtopping depths of key infrastructure

Infrastructure location	Overtopping depth (m)							
	PMF	1 in 10,000 AEP	1 in 2,000 AEP	1% AEP	2% AEP	5% AEP	10% AEP	20% AEP
Millmerran-Inglewood Road – middle	5.53	2.77	1.48	0.92	0.81	0.27	0.16	0.11
Millmerran-Inglewood Road – south	3.86	1.18	0.14	0.05	0.07	0.04	0.03	0.03
Millmerran-Inglewood Road – north	5.75	3.02	1.77	1.24	1.14	0.57	0.27	0.00
Heckels Road	4.95	2.31	1.20	0.75	0.70	0.54	0.45	0.39
Forestry Road	3.42	0.80	0.31	0.26	0.34	0.23	0.21	0.20

10.4.5 Existing Case flood velocities

Peak 1% AEP event flood velocities are predicted to reach 5.0 m/s in localised areas (and confined within the main channel) with average velocities on the floodplain of approximately 0.6 m/s, as shown in Figure F2-D in Volume 2 – Appendix F.

10.4.6 Existing Case flood hazard

For the 1% AEP event a velocity-depth product of greater than 9 m²/s is achieved within the existing bridge crossing. Excluding the channelised high-flow zones, sheet flow in the floodplains west of Millmerran-Inglewood Road have an average velocity-depth product of less than 0.5 m²/s as shown in Figure F2-F in Volume 2 – Appendix F.

10.4.7 Existing Case flood duration of inundation

Results for the 1% AEP event highlights flood water inundating areas upstream of Millmerran-Inglewood Road for periods greater than 10 hours. The profile also highlights ponding areas created by farming activities and existing road infrastructure as shown in Figure F2-H in Volume 2 – Appendix F.

10.5 Developed Case modelling results—Bringalily Creek

10.5.1 Drainage structures

The hydraulic design of the major drainage structures was undertaken using the TUFLOW model (1D and 2D approach).

In the Bringalily Creek floodplain, the Project includes the following floodplain (or regional structures):

- Three waterway bridges
- Eight RCP locations (a total of 82 cells).

Local drainage structures are presented in Appendix B. These structures were sized through the local drainage design and depending on their interaction with the Bringalily Creek floodplain were incorporated in the hydraulic model. The 1% AEP event flood levels at each drainage structure are presented in Appendix B.

Design bridge details have been summarised in Table 10.17 and details of the proposed culvert structures are summarised in the Cross-Drainage Register located in Appendix B. The location of cross-drainage features has been presented in Figure F1-F in Volume 2 – Appendix F.

Table 10.17 Bringalily Creek—Proposed bridge locations and details

Chainage (km)	Structure ID	Span (m)	Deck width (m)	Soffit level (mAHD)	Deck superstructure type	Deck depth (mm)
93.93	310-BR07 ¹	195	3.97	325.42	Type D1	2,000
97.58	310-BR08	299	3.97	332.79	Type D1	2,000
100.39	310-BR10	621	3.97	334.54	Type D1	2,000

Table note:

1 Structure 310-BR07 included in the Native Dog Creek hydraulic model.

All bridges were modelled as an opening in the rail embankment. The optimisation of bridge length was balanced between minimising the changes to the hydraulic regime, primarily change in peak water levels and velocities, and the economics of using bridge spans versus earth embankments.

While all three bridges have been included in Bringalily model, the primary focus of the assessment was to assess the hydraulic performance of the design rail between Chainage (Ch) 90 km and Ch 143 km, including bridges 310-BR08 and 310-BR10. Bridge 310-BR07 was specifically assessed in the Native Dog Creek model (Section 11.5.1).

Blockage of hydraulic structures was assessed in accordance with ARR 2019. The predominant vegetation in the catchment is considered heavily forested; however, given the appreciable size of the bridge spans, and the relative size of upstream debris, a blockage factor was not applied to bridges. The risk of significant amounts of debris accumulating against the piers of the bridge is considered low.

10.5.2 Hydraulic design criteria outcomes

The hydraulic model was run for the Developed Case for the range of events (20%, 10%, 5%, 2%, 1% AEP events and 1 in 2,000 AEP, 1 in 10,000 AEP and PMF events) with the outcomes presented in the following sections.

10.5.2.1 Rail immunity and structures results

Appendix B summarises the peak 1% AEP event water levels on the upstream side of the Project alignment.

The results of flood modelling indicate that a 1% AEP event flood immunity to the proposed rail formation is achieved for the Project alignment across the Bringalily Creek floodplain, and that 1% AEP peak water levels remain below the proposed rail formation level. There is over 1.5 m freeboard to the rail formation level for the 1% AEP event, noting that the formation levels were generally dictated by other engineering criteria and constraints.

10.5.3 Flood Impact Objectives outcomes—Bringalily Creek

The impact of the Developed Case was assessed through inclusion of the proposed rail design and comparison of model results against the Existing Case results.

Changes to flooding behaviour that were assessed, and are reported in the following sections, include potential changes in peak water levels, time of inundation, velocity, hazard and peak flows:

- Changes in 20% AEP and 1% AEP peak water levels are presented in Figures F4-A and F4-B in Volume 2 – Appendix F
- Changes in 20% AEP and 1% AEP velocities are presented in Figures F4-C and F4-D in Volume 2 – Appendix F
- Changes in 20% AEP and 1% AEP hazard are presented in Figures F4-E and F4-F in Volume 2 – Appendix F
- Changes in 20% AEP and 1% AEP time of inundation are presented in Figures F4-G and F4-H in Volume 2 – Appendix F

- Changes in 20% AEP and 1% AEP velocities and percentage change in velocities with cut-offs applied are presented in Figures F5-A to F5-F in Volume 2 – Appendix F
- Percentage changes in 20% AEP and 1% AEP in hazard are presented in Figures F6-A and F6-B in Volume 2 – Appendix F
- Percentage changes in 20% AEP and 1% AEP in time of inundation and with cut-offs applied are presented in Figures F7-A and F7-D in Volume 2 – Appendix F
- Refer to the [Online Digital Portal](#) for other modelled AEP results and higher resolution mapping.

One-on-one consultation with landowners who are expected to experience changes in flooding behaviour on the property was conducted by ARTC, supported by FFJV. Refer to EIS Chapter 14: Flooding and Geomorphology and EIS Appendix E: Consultation Report for further details.

The Project design outcomes relative to the FIOs (Table 4.2) are presented in the following sections.

Flood sensitive receptors referred to in this section are shown in Figures P1 (1 to 59) in Volume 2 – Appendix P.

10.5.3.1 Flood impacts on flood sensitive receptors

Based on the available aerial imagery, no buildings or critical structures are located within the area impacted by afflux and change in hazard in the Bringalily Creek floodplain for events up to and including the 1 in 2,000 AEP event.

10.5.3.2 Flood impacts on State-controlled roads

Within the extent of the hydraulic model, the only State-controlled road which is influenced by flooding and the Project alignment is Millmerran-Inglewood Road. Table 10.18 shows four locations of potential FIO exceedances. The details for each type of exceedance including afflux, velocity, time of inundation and hazard exceedances is included within Table 10.18 as well as justification and proposed mitigation measures for each exceedance.

The following provide a summary of the key exceedances:

- Millmerran-Inglewood Road is the only State-controlled road influenced by the Project alignment within the Bringalily catchment.
- Location 26 of the Millmerran-Inglewood Road shows the greatest impacts with a maximum afflux of approximately 95 mm, and for an extent of 187 m for the 1% AEP design flood event. This segment of Millmerran-Inglewood Road also experiences some change in time of inundation and hazard.
- Minor afflux exceedances are noted for locations 25, 28 and 30.
- Exceedances on State-controlled roads is due to the Project's design constrains altering the existing flow regime, proximity of the road in relation to the rail, and the maturity of cross-drainage design.
- During detailed design proposed mitigation measures including site-specific treatments and additional cross-drainage optimisation investigations are likely to be undertaken. Stakeholder communication is also likely to inform potential mitigation options.

The extent of the model, and location of the State-controlled road is shown in Figure 10.2.

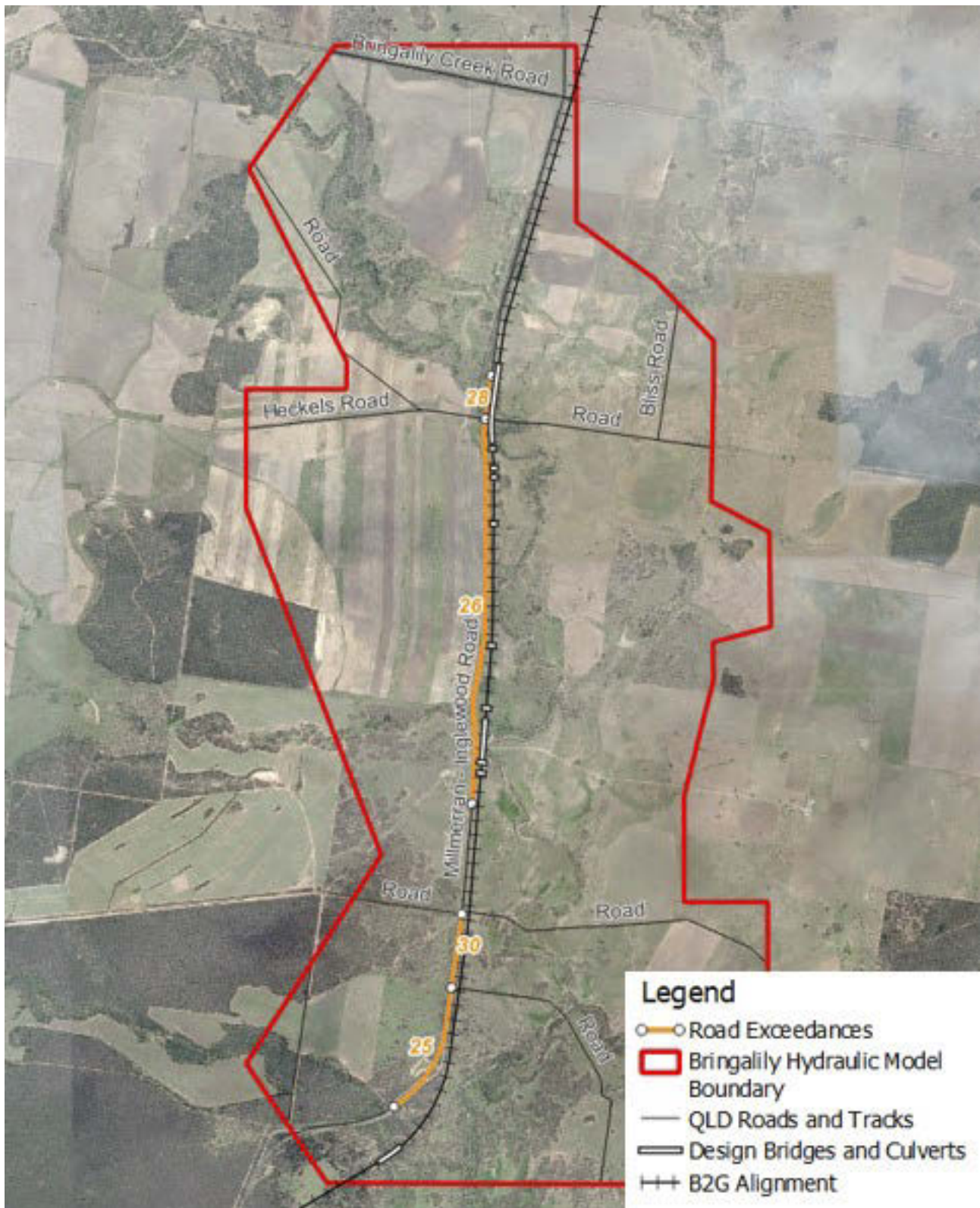


Figure 10.2 Bringality Creek—Impacted State-controlled roads

[illegible]

Table Note:

1. Time of Submergence has been measured across the full length of the road segment and does not correspond to the exact location of exceedance within that segment
2. Change in water level (afflux) is calculated based on the maximum water surface level difference between the existing and design case results, and does not consider the underlying topography. Depth values presented are based on the respective underlying topographies and in some cases do not align with afflux calculations. This can be attributed to interpolation calculations of results performed within the hydraulic model.
3. The above table should be read in conjunction with change maps provided as part of Volume II and the Digital Platform.
4. The symbol "-" denotes that no change Observe Flood Impact Objectives has been determined and therefore no result has been presented.
5. Changes recorded above may only occur during a small fraction of model simulation time.
6. Three decimal places do not infer absolute accuracy in modelling results, but it is necessary to identify small changes.
7. Design optimisations that could be implemented (but not limited to) include adjustment of culvert spacing, realignment of culverts to existing flowpaths when more detailed survey is available, modifying the grade of culverts, road height and alignment adjustments, removal of bridge abutments from waterway extents etc

[illegible]

10.5.3.3 Flood impacts on local public roads

No impacts have been assessed on local roads in the Bringalily Creek hydraulic model boundary.

10.5.3.4 Flood impacts on existing rail infrastructure

No third-party rail infrastructure is located within the Bringalily Creek hydraulic model boundary.

10.5.3.5 Flood impacts on private land outside the permanent footprint

In accordance with the FIOs, Table 10.19 presents the modelled changes and potential impacts on third-party property outside of the Project's permanent footprint. A summary of key impacts, exceedance justification and mitigation measures are provided in the below points, with additional detail provided in Table 10.19:

- Nine separate properties (including one watercourse) with individual land-use categories have been identified as having a potential impact exceeding the Project FIO criteria
- The majority of all potential exceedances occurred in Cattle Breeding and Grains land-use types. There is one instance of exceedances occurring within a watercourse
- Maximum change in water level (afflux) for all properties is less than 0.5 m. It should be noted that most large increases are localised and don't translate to uniform flood level increases across the lot.
- Maximum change in velocity for all properties is approximately 1.1 m/s
- Maximum change in the duration of inundation is <3 hours
- Maximum change in hazard for all properties is <0.2 m²/s
- Exceedances may be a result of the maturity of current cross-drainage (culverts and bridges) design, and limited available information (e.g. detailed survey)
- In order to mitigate potential exceedances, further cross-drainage optimisation investigations, and ongoing model strengthening may occur in detailed design.

Table 10.19 Bringalily Creek – Summary of flood impacts on private land outside the permanent footprint

[illegible]

Table Notes

1. Floodplains are dynamic in nature and therefore model results vary from cell to cell. The "Area" is defined as the maximum area over which change above the Flood Impact Objective occurs and does not relate directly to the maximum Flood exceedance recorded.
2. Change in water level (afflux) is calculated based on the maximum water surface level difference between the existing and design case results, and does not consider the underlying topography. Depth values presented are based on the respective underlying topographies and in some cases do not align with afflux calculations. This can be attributed to interpolation calculations of results performed within the hydraulic model.
3. Percentage increases should be read in conjunction with existing and design case results. The FIOs require that change be presented in percentage change in time of inundation, velocity and hazard and can sometimes produce large increases when existing case results are significantly small. Some locations were identified to experience significant percentage increases however the "absolute" difference between Existing and Design results is small.
4. The Flood Impact Objective has been identified as a change in flood extent and a cell which was dry in the existing case and now flood-affected in the developed case.
5. The above table should be read in conjunction with change maps provided as part of Volume II and the Digital Platform.
6. The symbol "-" denotes that no change above Flood Impact Objectives has been determined and therefore no result has been presented.
7. Changes recorded above may only occur during a small fraction of model simulation time.
8. Three decimal places do not infer absolute accuracy in modelling results, but it is necessary to identify small changes.
9. Flood impact objectives are assessed based on landuse applied to the entire lot and exceedances identified do not necessarily indicate an impact to a dwelling.
10. Design optimisations that (but not be limited to) include adjustment of culvert spacing, realignment of culverts to existing townships when detailed survey is available, modifying the grade of culverts, road height and alignment adjustments, removal of bridge abutments from waterway extents etc.
11. Landuse data, dated 2018, has been used for this assessment. Due to the changing nature of this dataset, this will be updated to the latest available information in Detail Design.

10.5.3.6 Flow distribution

A key landowner concern is the potential change in flow distribution. To determine the change in flow distribution, maximum velocity-depth modelling results were used to produce change in unit flow maps between Existing and Developed Case conditions. Unit flow is the measure of discharge (velocity-depth product) and is often used for assessing peak flow along a cross-section, or to determine the effective flow corridors within a floodplain.

The presented flow distribution maps were filtered with a velocity-depth product cut-off of 0.15 m²/s. The cut-off is in line with FIO requirements and was used to assist in identifying main channels (high flow zones) from shallow lower flood zones (i.e. outer extents of the flood surface).

The following points summarise the potential change in flow distribution for the Bringalily catchment and should be considered in conjunction with flow distribution figures supplied in Figure Q6-1 to Figure Q6-8 of Volume 2 – Appendix Q:

- The hydraulic results for Bringalily Creek show the primary change of flow conditions occurs at bridge 310-BR08, and to a lesser extent at all culvert infrastructure for modelled events up to the 1% AEP event. This is mainly confined to the eastern side of the Project alignment, which results in a positive increase in flow distribution.
- A consistent increase in unit flow is noted downstream of bridge 310-BR07, with localised increases and decreases occurring at culvert locations.
- Flow conditions remain largely unimpacted at bridge 310-BR10, up to the 1% AEP event, of which spans the majority of the existing waterway, thus having minimal effect on existing flow conditions. In more extreme events (1 in 2,000, 1 in 10,000 AEP and PMF events), some positive change in flow distribution occurs on the eastern side of the Project alignment due to the interaction of flood waters with the design.

Ongoing mitigation measures to reduce changes in flow distribution are likely to occur during future design stages of this Project, and may include further refinement of the bridge abutments, culvert skews, and additional survey where necessary.

10.5.4 Extreme events

10.5.4.1 Extreme event immunity

The risk of overtopping of the Project alignment was assessed for a range of extreme events (1 in 2,000 AEP, 1 in 10,000 AEP and PMF), with Table 10.20 presenting the depth of water above the formation level and over the top of rail at each structure. It is noted that the function of the floodplain culverts is to balance flood levels on the upstream and downstream sides of the alignment. As such, overtopping of the rail is not predicted to result in significant excessive flows or velocities as would occur in a dam embankment overtopping scenario. Further analysis of the overtopping risk to sensitive receptors can be found in Section 21.

Table 10.20 Bringalily Creek—Extreme events: Estimated depth of water above formation level and over top of rail level

Chainage (km)	Structure ID	Depth of water above formation level (m)			Depth of water above top of rail (m)		
		1 in 2,000 AEP	1 in 10,000 AEP	PMF	1 in 2,000 AEP	1 in 10,000 AEP	PMF
94.00	310-BR07	-	-	-	-	-	-
94.91	C94.91	-	0.16	2.65	-	-	1.95
95.07	C95.07	-	0.52	3.14	-	-	2.44
96.20	C96.20	-	-	2.49	-	-	1.79

Chainage (km)	Structure ID	Depth of water above formation level (m)			Depth of water above top of rail (m)		
		1 in 2,000 AEP	1 in 10,000 AEP	PMF	1 in 2,000 AEP	1 in 10,000 AEP	PMF
97.01	C97.01	-	-	1.69	-	-	0.99
97.29	C97.29	-	-	-	-	-	-
97.38	C97.38	-	-	-	-	-	-
97.6	310-BR08	-	-	-	-	-	-
97.84	C97.84	-	-	-	-	-	-
98.36	C98.36	-	-	-	-	-	-
98.87	C98.87	-	-	1.26	-	-	0.56
99.37	C99.37	-	0.06	1.96	-	-	1.26
99.77	C99.77	-	-	2.18	-	-	1.48
99.84	C99.84	-	-	2.31	-	-	1.61
100.00	C100.00	-	-	2.35	-	-	1.65
100.6	310-BR10	-	0.50	3.21	-	-	2.51

10.5.4.2 Impacts during extreme events

Flood immunity of FSRs is discussed in Section 10.5.3.1, and maps demonstrating the impacts during extreme events can be found in the [Online Digital Portal](#).

In accordance with the FIOs, FSRs are considered impacted by extreme flood events if the change in water level is greater than 250 mm in the 1 in 2,000 AEP event, subject to the receptor classification (i.e. at existing habitable dwellings and/or commercial and industrial buildings/premises). In addition, a stricter afflux objective of 10 mm in the 1 in 500 AEP event applies to sites with existing sensitive infrastructure, uses involving vulnerable people and/or any critical road network that was designed to be immune to flooding in any extreme event.

Review of the 1 in 2,000 AEP event results for the Bringalily Creek floodplain against the FIOs show that flood level changes at all FSRs were limited to 250 mm, and no identified existing sensitive infrastructure or uses involving vulnerable people are expected to experience a more than 10 mm increase in flood level. A separate assessment of the 1 in 500 AEP event is not required as they are conservatively represented by the 1 in 2,000 AEP event results. The flood impacts in extreme events due to the Project are therefore compliant against the stipulated FIO limits for extreme events.

Table 10.21 details extreme event results, although it should be noted that FIO targets do not apply to the 1 in 10,000 AEP and PMF events.

Table 10.21 Bringalily Creek—Extreme events: Estimated impacts at flood sensitive receptors

FSR ID	FSR type	1 in 2,000 AEP event		1 in 10,000 AEP event		PMF event	
		Change in peak water level (mm)	Existing Case flood depth (m)	Change in peak water level (mm)	Existing Case flood depth (m)	Change in peak water level (mm)	Existing Case flood depth (m)
BRI_ID_1	Shed	-28	0.52	55	1.18	172	3.55
BRI_ID_2	House	1	0.68	71	1.35	197	3.68
BRI_ID_3	Shed	-21	0.56	75	1.17	154	3.60
BRI_ID_4	Shed	-33	0.58	59	1.22	167	3.62
BRI_ID_5	House	-	-	-	-	4	2.48
BRI_ID_6	Shed	-	-	-	-	6	2.05
BRI_ID_7	Shed	-	-	-	-	5	1.42

FSR ID	FSR type	1 in 2,000 AEP event		1 in 10,000 AEP event		PMF event	
		Change in peak water level (mm)	Existing Case flood depth (m)	Change in peak water level (mm)	Existing Case flood depth (m)	Change in peak water level (mm)	Existing Case flood depth (m)
BRI_ID_8	Shed	-	-	-	-	9	1.75
BRI_ID_9	House	-	0.06	1	0.16	35	2.43
BRI_ID_10	Shed	-	-	-	0.15	35	2.26
BRI_ID_11	Shed	-	-	-	0.08	35	1.93
BRI_ID_12	Shed	-	-	-	-	34	1.68
BRI_ID_13	Shed	-	-	-	-	36	2.12
BRI_ID_14	Shed	-	-	-	0.04	36	1.82
BRI_ID_15	Shed	-	-	-	-	42	1.90
BRI_ID_16	Shed	-	-	-	-	47	1.71
BRI_ID_17	Shed	-	-	0	0.39	13	2.40
BRI_ID_18	Shed	16	0.79	14	1.87	32	4.67
BRI_ID_19	Shed	-	-	60	0.31	118	3.09
BRI_ID_20	House	-18	0.44	-13	1.31	3	3.80
BRI_ID_21	Shed	-17	0.44	-12	1.36	9	3.92
BRI_ID_22	Shed	-26	0.39	-20	1.28	2	3.77
BRI_ID_24	Private Access Road	16	1.13	-38	2.45	6	5.36
BRI_ID_25	Private Access Road	-	-	30	0.35	65	2.45
BRI_ID_26	House	-	-	-	-	5	2.57
BRI_ID_27	Shed	-	-	-	0.10	5	2.58
BRI_ID_28	Shed	-	-	-	-	6	2.74
BRI_ID_29	House	-	-	-0	0.10	7	1.55
BRI_ID_30	Shed	-	-	-0	0.08	8	1.71
BRI_ID_31	Shed	1	0.42	-12	1.11	7	3.74

10.5.5 Sensitivity analysis—Bringalily Creek

The sensitivity of the model to various parameters was assessed using the following two scenarios:

- An increase in rainfall intensity (i.e. to reflect climate change scenario)
- An increase and decrease in blockage of culverts from 25 to 50 per cent and 0 per cent.

10.5.5.1 Climate change

The potential impacts of climate change in the Bringalily Creek floodplain were assessed for the 1% AEP design event to determine the sensitivity of the Project to the potential long-term changes in climate. The assessment was undertaken in accordance with ARR 2019.

The Representative Concentration Pathways 8.5 (2090 horizon) climate change scenario has been adopted for the Project with an associated increase in rainfall intensity of 23.9 per cent across the catchment area.

The Project is expected to retain a 1% AEP flood immunity, with at least 1.1 m freeboard to formation level under the climate change scenario.

The climate change sensitivity assessment results for FSRs indicated that two FSRs (BRI_ID_18 and BRI_ID_24) experience an increase in peak water level for a 1% AEP design event with climate change conditions as shown in Table 10.22.

Table 10.22 Bringalily Creek—1% AEP event Representative Concentration Pathways 8.5 conditions: Impacts at flood sensitive receptors

FSR ID	FSR type	1% AEP climate change event	
		Change in peak water level (mm)	Existing Case flood depth (m)
BRI_ID_1	Shed	-30	0.3
BRI_ID_2	House	+2	0.5
BRI_ID_3	Shed	-53	0.3
BRI_ID_4	Shed	-57	0.3
BRI_ID_18	Shed	25	0.6
BRI_ID_20	House	-11	0.2
BRI_ID_21	Shed	-1	0.2
BRI_ID_24	Private Access Road	+6	0.8
BRI_ID_31	Shed	+7	0.3

For the 1% AEP event, the change in peak water levels for the Representative Concentration Pathways 8.5 climate change scenario is presented in Figure F8-C in Volume 2 – Appendix F.

10.5.5.2 Blockage

Blockage was assessed in accordance with ARR 2019. The blockage assessment was undertaken and resulted in a blockage factor of 25 per cent adopted for culverts. A minimum culvert size of 900 mm diameter was adopted to reduce potential for blockage and maintenance. A significant community concern is the potential impacts to flood conditions should the proposed culverts become blocked with debris. The primary concern is the blockage of culverts, which is likely to drive flood levels higher, particularly upstream of the culverts, and divert more flow through residences, across access roads and other infrastructure. A sensitivity analysis was undertaken with 0 and 50 per cent blockage on culverts.

In addition, the culvert blockage sensitivity assessment results indicated that FSRs have not exceeded the FIO's limits as detailed in Table 10.23.

During detailed design the blockage factors will be reviewed in line with the final design and local catchment conditions. This may result in a varied and/or lower blockage factors being applied along the Project alignment. Should impacts be determined due to a change in blockage factor, additional mitigation measures will be considered in detailed design.

Table 10.23 Bringalily Creek—Summary of blockage impacts at flood sensitive receptors

FSR ID	FSR type	Existing Case flood depth (m)	Change in peak water level (mm)		
			0% Blockage	25% Blockage	50% Blockage
BRI_ID_1	Shed	0.2	-30	-35	-32
BRI_ID_2	House	0.4	+2	-3	+1
BRI_ID_3	Shed	0.2	-53	-64	-59
BRI_ID_4	Shed	0.2	-57	-65	-63
BRI_ID_18	Shed	0.4	25	17	+26
BRI_ID_20	House	0.1	-11	-6	-12
BRI_ID_21	Shed	0.1	-1	-6	-2
BRI_ID_22	Shed	0.1	-	-	-

FSR ID	FSR type	Existing Case flood depth (m)	Change in peak water level (mm)		
			0% Blockage	25% Blockage	50% Blockage
BRI_ID_24	Private Access Road	0.6	+6	+6	+6
BRI_ID_31	Shed	0.2	+7	-	+7

For the 1% AEP event, the change in peak water levels for the 0 and 50 per cent blockage scenarios are presented in Figures F8-A and F8-B in Volume 2 – Appendix F.

10.6 Additional modelling items to be considered in detailed design

The following points summarise additional items to be included in the flood models for detailed design as per agreement with the Expert Flood Panel:

- Undertake additional model strengthening by improving the representation of cross-drainage infrastructure, third party infrastructure, survey data, and bridge structures within the hydraulic model. Specifically, the modelling of bridges will be included as LFC shape files within the hydraulic model.
- A review of sub-catchment delineation will be completed to more accurately reflect catchments upstream and downstream of the alignment
- A review of the hydraulic model downstream boundary will be undertaken
- Further investigation of the catchment critical durations and TPs will be undertaken to adopt suitable flows for cross-drainage infrastructure design as well as considering water levels and velocity
- Location of inflow boundaries will be reviewed to ensure a consistent approach
- Where flood exceedances have been identified, additional analysis of cross-drainage infrastructure, and other mitigation options may be investigated to reduce impacts. Land and stakeholder consultation will also form part of the mitigation process.
- A review of the latest TUFLOW version will be undertaken at the commencement of detailed design to determine whether updates of the TUFLOW version from 2020-10-AD-iSP-w64 are beneficial to modelling.

Further details of mitigation measures can be found in Section 22. A review of proposed mitigation measures will be completed during the detailed design stage of the Project after hydraulic models have been updated to capture the above changes.

10.7 Planned and future roads

Consultation with the DTMR has not identified any planned or future road upgrades within the Bringalily Creek catchment. ARTC will continue to consult and engage with DTMR to determine any future or planned road projects that may need to be considered as part of the detailed design hydraulic modelling.

TRC has not identified any planned or future roads within the Bringalily Creek hydraulic model domain. ARTC will continue to consult and engage with TRC to determine any future or planned road projects that may need to be considered as part of the detailed design hydraulic modelling.

11 Native Dog Creek

Native Dog Creek is a well-defined water course with well-vegetated overbank areas. Native Dog Creek crosses Millmerran-Inglewood Road and is an upstream tributary of Canning Creek.

Under the Existing Case 1% AEP event, the estimated flood depth in Native Dog Creek channel is up to approximately 3.2 m with depths of 1 m on the floodplain area. The floodplain inundation extent is approximately 580 m wide.

The location of the Project alignment in relation to Native Dog Creek is shown in Figure G1-A in Volume 2 – Appendix G.

11.1 Data collation and review—Native Dog Creek

Available background information, including existing hydrologic and hydraulic models, survey, streamflow data, available calibration information and anecdotal flood data has been gathered. Data was sourced from a range of stakeholders including:

- GRC—existing flood studies
- BoM—rainfall gauging data
- DTMR—existing infrastructure details.

11.1.1 Previous studies

Previous hydrology and hydraulic studies were sourced as part of this assessment. A review of each study was undertaken to determine suitability for use on the Project, as documented in the following sections.

11.1.1.1 Inglewood Flood Study, Engeny, 2015

Engeny was commissioned by GRC to undertake a flood study of Inglewood. The study objectives were ‘to define the nature, extent and risks of flooding in Inglewood in order to inform disaster management planning and response, as well as control future development.’ (Engeny, 2015). An URBS hydrologic model and TUFLOW (1D/2D) hydraulic model were developed.

11.1.2 Survey

ARTC provided LiDAR data from 2015 as 1 m grid DEM tiles. Using GIS software, a DEM was generated with a 1 m grid resolution for use in the Project based on the 2015 dataset. This was used for modelling within the permanent footprint and up to the full extent of the 2015 LiDAR where relevant. In areas that were not covered by the LiDAR provided by ARTC, LiDAR tiles were sourced from Geoscience Australia. The DEM datasets utilised for modelling were based on surveys flown between 2009 and 2015, with preference given to the most recent data available. SRTM data was used for catchment delineation where no LiDAR data could be sourced, to inform the hydrologic modelling.

The survey data sources, and DEM developed for Native Dog Creek are shown in Figure G1-B of Volume 2 – Appendix G.

11.1.3 Aerial imagery

Aerial imagery of the Native Dog Creek Study area was provided by ARTC and was used to identify and confirm topographic and vegetative characteristics of the Study area. Aerial imagery captured in 2015 was made available. Additional imagery outside the Study area was sourced from QGIS imagery in an open-source format.

11.1.4 Existing drainage structure

No available DTMR as-constructed drawings were available for culvert and bridge details.

Additional information will be refined as local survey is progressively completed during detailed design.

Details of infrastructure included within the Native Dog Creek flood assessment is located in Section 11.3.4.

11.1.5 Stream gauge data

There are no stream gauges within the Native Dog Creek catchment study area assessed as part of this investigation.

11.1.6 Rainfall data

Several daily and sub-daily rainfall stations are located near and around the Native Dog Creek catchment; however, since there are no streamflow gauges to use for model calibration, no historical rainfall data was sought.

11.1.7 Anecdotal and observed flood data

No anecdotal or observed flood data was available for this area of Native Dog Creek.

11.1.8 Site inspection

A site inspection was undertaken during February 2018. During the site inspection, all major waterway crossings were visited and inspected, with photographs taken and details recorded of the crossing, existing drainage structures and surrounding catchment. An assessment of the relative roughness and blockage potential was undertaken during the site inspection.

11.2 Hydrologic model development—Native Dog Creek

11.2.1 Model setup

Native Dog Creek is one of four major waterways that cross the Project alignment before flowing into Canning Creek. The other waterways are Bringalily Creek, Cattle Creek, and Nicol Creek. Due to their proximity to one another, all four waterways were modelled in a unified Canning Creek model (using the URBS modelling platform). In addition to the below sections, refer to Section 9.2 (Nicol Creek Catchment) for further details on the development of the hydrologic model and associated modelling parameters.

The catchment area upstream of the Project alignment crossing of Native Dog Creek is approximately 27 km².

11.2.2 Design model parameters

Hydrologic information to assist estimation of design event flows was sourced from the ARR 2019 Data Hub as summarised in Table 11.1.

Table 11.1 Native Dog Creek—Summary of URBS model design event inputs

Input parameter	Remarks
Design rainfall	A single IFD was downloaded from BoM to simulate design rainfall within the Native Dog Creek catchment.

Input parameter	Remarks
Extreme event rainfall	Probable Maximum Precipitation depths for durations up to 6 hours (for use in modelling the PMF event) were obtained using the method presented in the Bulletin 53 (BoM, 2003). The rainfall depths for the 1 in 10,000 AEP event were estimated using the interpolation method presented in ARR 2019 Book 8 Section 3.5.
Losses	The losses were initially adopted from the <i>Inglewood Flood Study</i> (Engeny, 2015) URBS model (Initial loss – 15 mm, Continuing loss – 1 mm/hr). Adjustments were made to reconcile URBS flows to area-scaled FFA.
Areal reduction factor	Parameters were adopted for the semi-arid inland Queensland region. The catchment area of Cattle Creek is approximately 27 km ² , which yields an ARF between 81.7% and 97% depending on design storm event AEP and duration.
Ensemble TPs	Central Slopes Region.
Pre-burst depths	Median pre-burst depths were downloaded from the ARR 2019 Data Hub for each sub-catchment. Pre-burst depths vary by design storm event AEP and duration. Pre-burst depths were applied to the model by reducing the initial losses for each storm event.

11.2.3 Spatial distribution

The ARR 2019 design event methodology was adopted for this assessment. ARR 2019 recommends adopting the spatially varying method for catchments larger than 20 km². As the Native Dog Creek catchment is marginally larger than 20 km² (approximately 27 km²) a single IFD has been adopted for this catchment.

11.2.4 Rainfall depth estimation

Design rainfall depth for the 20% to 1 in 2,000 AEP were manually obtained from BoM website for each sub-catchment. The IFD adopted for the Native Dog Creek hydrologic model is provided in Table 11.2.

Table 11.2 Rainfall depth (mm) used for the Native Dog Creek catchment centroid

Duration (hr)	Annual Exceedance Probability (% AEP)						
	50	20	10	5	2	1	1 in 2,000
0.25	17.6	24.5	29.1	33.7	39.9	44.7	70.5
0.5	24.0	33.4	39.9	46.3	54.9	61.6	97.2
1	30.4	42.2	50.4	58.6	69.6	78.2	123.0
2	36.8	50.6	60.4	70.2	83.3	93.6	147.0
3	40.7	55.7	66.2	76.8	91.2	103.0	161.0
4.5	45.0	61.1	72.4	83.8	99.5	112.0	176.0
6	48.4	65.3	77.2	89.3	106.0	119.0	187.0
12	57.9	77.4	91.2	105.0	125.0	141.0	221.0
24	69.7	93.2	110.0	127.0	152.0	171.0	270.0
48	83.3	113.0	134.0	156.0	187.0	211.0	352.0
72	91.3	125.0	149.0	174.0	209.0	236.0	381.0

11.2.5 Areal reduction factor

The ARF was estimated using the coefficients for semi-arid inland Queensland presented in Table 11.3 and was applied to all IFD areas. The URBS software directly downloaded these coefficients from ARR 2019 Data Hub to estimate the ARF using Equation 2.4.1 of ARR 2019 Book 2.

The estimated ARF for Native Dog Creek catchment vary between 81.7 to 97 per cent depending on design storm event AEP and duration.

Table 11.3 Native Dog Creek—Areal reduction factor coefficients from ARR 2019 Data Hub

a	b	c	d	e	f	g	h	i
0.16	0.28	0.25	0.31	0.00	1.00	0.04	0.00	0.00

11.2.6 Temporal patterns

Temporal patterns for the 20% AEP to 1 in 2,000 AEP events were adopted based on 10 burst ensemble approach presented in the ARR 2019. The TPs and pre-burst data were directly downloaded by URBS software from the ARR 2019 Data Hub at the centroid of Native Dog Creek catchment.

Temporal patterns for the 1 in 10,000 AEP and PMF design events for duration up to 6 hours were obtained from *The Estimation of Probable Maximum Precipitation in Australia: Generalised Short-Duration Method* (BoM, 2003).

The adopted TPs for all modelled events are shown in Table 11.4.

Table 11.4 Native Dog Creek—Adopted temporal patterns

AEP (%)	Critical duration storm/temporal pattern
20% AEP	Pattern 7
10% AEP	Pattern 2
5% AEP	Pattern 8
2% AEP	Pattern 5
1% AEP	Pattern 7
1 in 2,000 year (0.05%) AEP	Pattern 7
1 in 10,000 year (0.01%) AEP	PMP temporal pattern
PMF	PMP temporal pattern

11.2.7 Pre-burst

Median pre-burst depths for standard durations from 0.5 hour to 72 hours for a 1% AEP design event are between 1.1 mm and 13.1 mm (Data Hub, accessed November 2018). Due to the small depth of pre-burst in this area, the impact of pre-burst condition is considered to be minimal in this catchment. The adopted pre-burst depths are summarised in Table 11.5.

Table 11.5 Native Dog Creek—Adopted pre-burst depths

Duration (hrs)	Pre-burst depth (mm)					
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP
0.5	0.6	0.8	0.9	1.1	1.7	2.3
0.75	0.6	0.8	0.9	1.1	1.7	2.3
1.0	0.6	0.8	0.9	1.1	1.7	2.3
1.5	0.2	0.3	0.4	0.5	1.6	2.4
2.0	0.2	0.3	0.3	0.3	1.7	2.8
3.0	0.1	0	0	0	2.8	5
6.0	0.2	0.6	0.8	1.1	1.1	1.1
12.0	0	0.2	0.2	0.3	5.2	8.8
18.0	0	0.3	0.5	0.7	7.8	13.1

Duration (hrs)	Pre-burst depth (mm)					
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP
24.0	0	0.5	0.9	1.2	6.5	10.4
48.0	0	0	0	0	0.1	0.2
72.0	0	0	0	0	0	0

11.2.8 Design rainfall losses

In this assessment, an initial and continuing loss model was applied, where the lost rainfall depth does not contribute to the catchment runoff. In the loss model, it was assumed that there is an initial loss of rainfall depth followed by a constant continuing loss (mm/hr) throughout the rainfall event.

A range of design rainfall losses were trialled for the design events. Initial and continuing losses in the hydrologic model were adjusted until there was a reasonable match between the modelled flows and those from the scaled FFA estimates. The adopted design event losses are provided in Table 11.6.

Table 11.6 Native Dog Creek – Adopted design event losses

AEP	Initial storm loss (mm)	Continuing loss (mm/hr)
20%	42.3	2.8
10%	38.6	2.6
5%	32.6	2.2
2%	22.3	1.5
1%	6.4	0.4

11.2.9 Extreme event rainfall

Probable Maximum Precipitation depths for durations up to 6 hours (for use in modelling the PMF event) were obtained using the methodology presented in *The Estimation of Probable Maximum Precipitation in Australia: Generalised Short Duration Method* (BoM, 2003). Once the PMP was estimated, rainfall for events between 1 in 2,000 AEP and the PMF was then interpolated. For the 1 in 10,000 AEP event, ARF were applied based on interpolation methodology presented in ARR 2019 Section 3.5 in Book 8 and the rainfall depths were estimated accordingly.

The parameters adopted for PMP rainfall estimation are presented in Table 11.7 and the adopted extreme rainfall depths are provided in Table 11.8.

Table 11.7 Native Dog Creek—Input data for Probable Maximum Precipitation estimation

Factor	Value
EAF	1
MAF	0.8
Portion of rough catchment	100%
Cattle Creek location and catchment area (km ²)	(-28.173, 151.134) and 27.25 km ²

Table 11.8 Native Dog Creek—Adopted extreme event rainfall depths (mm)

Duration (hour)	Native Dog Creek	
	1 in 10,000 AEP rainfall depth (mm)	PMP rainfall depth (mm)
0.25	87	163
0.5	120	238
0.75	142	302

Duration (hour)	Native Dog Creek	
	1 in 10,000 AEP rainfall depth (mm)	PMP rainfall depth (mm)
1	152	356
1.5	174	457
2	183	533
2.5	194	591
3	203	645
4	218	735
5	230	808
6	239	859

11.2.10 Event selection for calibration

Insufficient data was available to calibrate historical flood events in the Native Dog Creek catchment.

11.2.11 Model calibration

Insufficient data was available to calibrate historical flood events in the Native Dog Creek catchment.

11.2.12 Model validation

The ARR 2019 RFFE was used to validate peak flow estimates for the Native Dog Creek hydrologic model. The online tool was used to obtain an estimation of catchment peak flow at the outlet of Native Dog Creek catchment.

RFFE inputs are summarised in Table 11.9, and outputs are summarised in Figure 11.1 and Table 11.10.

Table 11.9 Native Dog Creek catchment —Model inputs for the Regional Flood Frequency Estimation

Model parameter	Input value
Region name	East Coast
Region version	1
Latitude at catchment outlet (degree)	-28.186
Longitude at catchment outlet (degree)	151.178
Latitude at catchment centroid (degree)	-28.173
Longitude at catchment centroid (degree)	151.133
Distance of the nearest gauged catchment in the database (km)	39.41
Catchment area (km ²)	27.246
Design rainfall intensity, 50% AEP and 6 hr duration (mm/hr)	8.066
Design rainfall intensity, 2% AEP and 6 hr duration (mm/hr)	17.631

Additional peak flow validation was undertaken using DTMR's QRT. This approach provides a procedure for calculating design floods for ungauged catchments that relates to the design flood discharges, to readily available catchment characteristics such as catchment area and 2% AEP 72-hour rainfall intensity. This method is commonly used for ungauged catchments in Queensland.

As discussed in Section 9.2.1.1, flood flows for Native Dog Creek were informed from the Canning Creek model with additional catchment definition, and ARF amended to suit the reduced Native Dog Creek catchment. All flows produced by the URBS (all events from the 20% AEP to the 1% AEP) reside within the 90 per cent confidence limits of the RFFE and show a close match with QRT. The estimated flood flows at the proposed Cattle Creek crossing are presented in Figure 11.1 and Table 11.10.

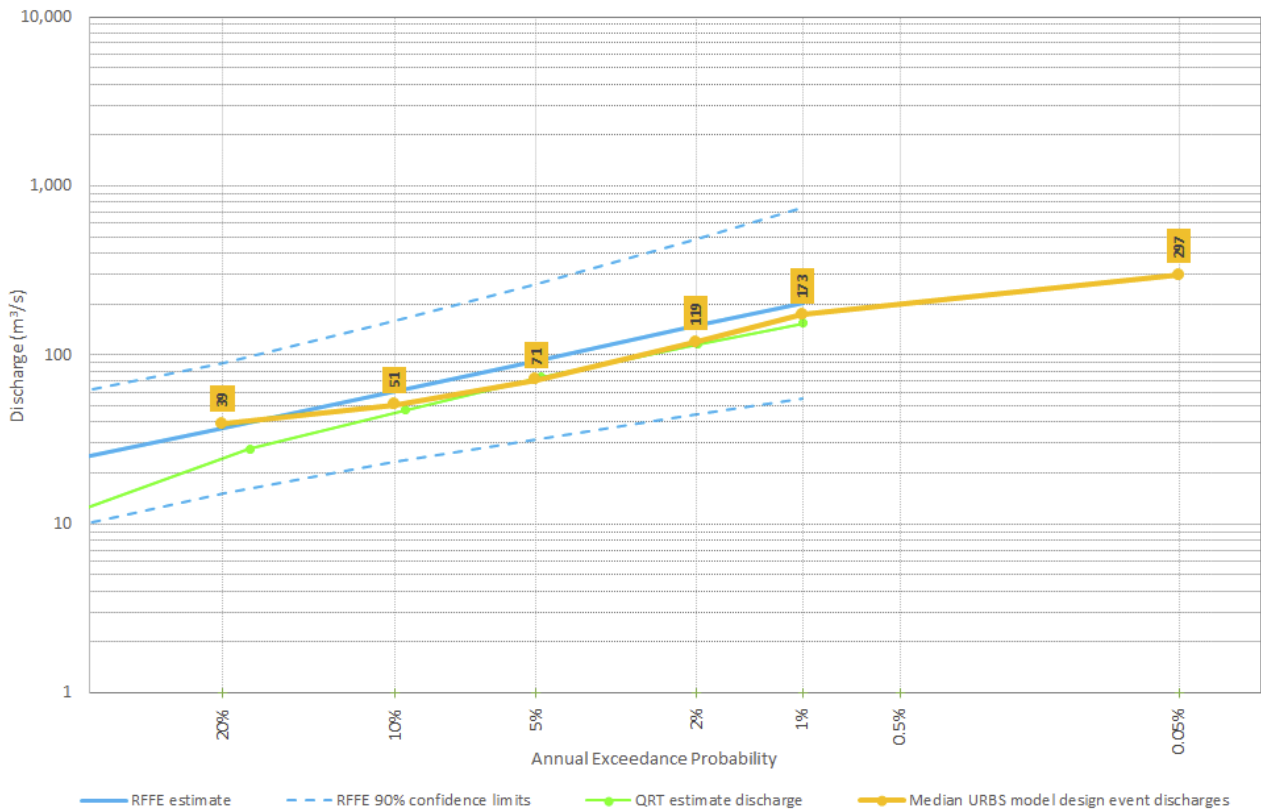


Figure 11.1 Estimate of flows at the Native Dog Creek Project crossing

Table 11.10 Estimate of flows at the Native Dog Creek crossing

AEP (%)	RFFE – lower bound 90% confidence level (m³/s)	RFFE – estimate of flow (m³/s)	RFFE – upper bound 90% confidence level (m³/s)	DTMR quantile regression technique (m³/s) ¹	URBS model flows (m³/s)
20	15	37	90	28	39
10	23	60	158	47	51
5	32	92	261	74	71
2	45	148	486	116	119
1	55	204	745	154	173
1 in 2,000	-	-	-	-	297
1 in 10,000	-	-	-	-	600
PMF	-	-	-	-	2,544

Table note:

1 The QRT method estimates the 39.3%, 18.1%, 9.5% and 4.9% AEP instead of 50%, 20% and 10% and 5%, respectively

11.3 Hydraulic model development—Native Dog Creek

A 2D modelling approach was adopted to appropriately simulate flood mechanisms around the Project rail crossing adjacent Native Dog Creek. The platform used for hydraulic modelling is TUFLOW HPC software package (version 2020-10-AD) with SGS approach. The processes and assumptions adopted throughout the development of the hydraulic model are described in the following sections.

11.3.1 Model setup

The setup of the hydraulic model is summarised in Table 11.11.

Table 11.11 Native Dog Creek— Hydraulic model summary

Parameter	Information
Completion date	June 2022
AEPs assessed	20%, 10%, 5%, 2%, 1%, 1 in 2,000, 1 in 10,000 AEP and PMF
Hydraulic model build	TUFLOW HPC GPU – version 2020-10-AD-iSP-w64
Model extent	Refer to Figure G1-D in Volume 2 – Appendix G
Grid size	5 m
SGS Size	1 m
QT parameters	2.5 m, 2 nd Order
DEM (year flown)	ARTC dataset (2015).
Roughness	Spatially varying roughness values compliant with industry norms.
Eddy viscosity	Wu (default)
Model calibration	N/A
D/S model boundary	Height-Discharge (HQ) Boundary with normal slope approximated based upon topography dataset.
Hydraulic model timestep	Adaptive Timestep
Modelled scenarios	Existing Case, Developed Case
Sensitivity analysis	Climate change

The hydraulic model extent and the spatial distribution of land use in the 2D model domain is presented in Figure G1-D and Figure G1-E in Volume 2 – Appendix G.

11.3.2 Model topography and grid size

The topography within the hydraulic model was based on 1 m DEM data collected during 2015, which was supplied by ARTC. A 5 x 5 m grid size was selected to allow the features of the channels and floodplain to be represented with sufficient accuracy while maintaining efficient model run times. The 5 x 5 m grid size is considered adequate to represent floodplain topography and is consistent with the approach typically adopted in similar flood studies.

The 2D model topography was modified adjacent to the culverts so that the existing drainage flows, existing road/rail crowns and levee banks were represented accurately in the model. For the Developed Case, the topography was modified to include the Project rail embankment, drainage structures and associated infrastructure.

11.3.2.1 Sub-grid sampling

The Native Dog Creek hydraulic model was established using TUFLOW's 2020-10-AD model version, including the SGS add-on module. The below points provide a brief description of the module, along with the key parameters adopted for this assessment.

- Traditional grid-mesh based 2D hydraulic models sample the underlying DEM by taking a single elevation at the cell centres and cell faces. Sub-grid sampling allows for increased resolution by sampling a greater number of points of the underlying DEM to establish a water surface elevation versus volume relationship for each grid, along with a water surface elevation versus width relationship (wetted perimeter) at cell sides. The adoption of SGS allows increased representation of storage and conveyance between cells. A SGS distance of 1 m was adopted for this assessment.

11.3.3 Roughness

The hydraulic roughness generally reflects the types of development and ground cover that exists within the hydraulic model extent. The distribution of roughness categories adopted for this assessment were based on the information supplied in the land use layer from Queensland Globe database, aerial imagery and confirmed during the site inspection. Specific roughness values applied to the model are detailed in Table 11.12.

Table 11.12 Native Dog Creek catchment —Adopted roughness values

Category	Manning's 'n' value
River	0.035
Sealed roads	0.015
Unsealed roads	0.025
Rural residential	0.09
Urban residential	0.10
Commercial	0.06
Open space—Minimum vegetation	0.045
Open space—Moderate vegetation	0.055
Open space—Heavy vegetation	0.065
Open water	0.025
Rail	0.025
Riparian	0.04
Crops ¹	0.1085

Table note:

- 1 A uniform value was adopted for the crop areas, due to the seasonal and spatial variations in vegetation distribution and the uncertainty this brings to the design event approach

11.3.4 Hydraulic structures

The following existing structure was incorporated into the hydraulic model:

- One existing bridge on Millmerran-Inglewood Road (DTMR bridge 24518).

Approximated details of the DTMR bridge are included in Table 11.13. The bridge was modelled as an opening in the hydraulic model. The bridge will be modelled using layered the flow constriction (lfcs) feature during detailed design stage.

Table 11.13 Native Dog Creek—Existing bridge

Bridge ID	Approximate span (m)	Deck width (m)	Deck level (mAHD)	Deck superstructure type	Deck depth (mm)
TMR 24518	56.9	8.60 ¹	326.7	T44	1,050

Table note:

- 1 Estimated from aerial imagery

11.3.5 Boundary conditions

The following boundary conditions were applied within the hydraulic model:

- Delineated sub-catchments in the hydrologic model were used as internal flow boundaries within the hydraulic model extent (SA polygons). The modelled flows in the hydrologic model were applied to the lowest points of each SA polygon.

- A height-discharge boundary (HQ) was applied for the downstream outflow model boundary. It was confirmed that it was a sufficient distance away from the area of interest so as not to impact on modelling results.

11.4 Existing Case modelling results—Native Dog Creek

11.4.1 Hydrologic model peak flow assessment

A critical duration assessment was undertaken to determine which storm duration(s) produced peak flood flows where the major waterways are intersected by the Project alignment and at the downstream outlet of the model. To assess the critical storm duration the following methodology was adopted:

- The models were modelled for a range of AEP events:
 - Each AEP was modelled for a range of durations
 - Each duration was modelled for each of the 10 associated TPs.
- A critical duration assessment was undertaken at the key locations to determine which duration produced the highest median flow of the 10 TPs for each event.

The adopted design loss rates for the hydrologic model are provided in Section 11.2.8.

Table 11.14 presents the critical duration and TPs adopted and Table 11.15 presents estimated peak flow applied to the hydraulic model for a number of key locations (Figure G1-D in Volume 2 – Appendix G).

Table 11.14 Native Dog Creek—Critical durations and temporal patterns

AEP (%)	Critical storm duration/temporal pattern
20	12 hour – Pattern 7
10	9 hour – Pattern 2
5	9 hour – Pattern 8
2	3 hour – Pattern 5
1	2 hour – Pattern 7
1 in 2,000	1.5 hour – Pattern 7
1 in 10,000	6 hour – PMP temporal pattern
PMF	6 hour – PMP temporal pattern

Table 11.15 Native Dog Creek—1% AEP event peak flow at key locations as applied in the hydraulic model

Sub-catchment name	1% AEP event peak flow (m ³ /s)	Critical storm (hrs) duration and temporal pattern
16L	172.7	2 hour – Pattern 7

11.4.2 Existing Case flood maps

Existing conditions flood maps have been presented in Volume 2 – Appendix G and via an [Online Digital Portal](#). Volume 2 – Appendix G contains the existing 20% and 1% AEP flood maps, while the [Online Digital Portal](#) contains the remainder of the modelled events. The location of each map set is summarised below:

- 20% AEP peak levels: Figure G2-A
- 1% AEP peak levels: Figure G2-B
- 20% AEP peak velocity: Figure G2-C
- 1% AEP peak velocity: Figure G2-D
- 20% AEP peak hazard: Figure G2-E

- 1% AEP peak hazard: Figure G2-F
- 20% AEP peak time of inundation: Figure G2-G
- 1% AEP peak time of inundation: Figure G2-H
- 20% AEP, 10% AEP, 5% AEP, 2% AEP, 1% AEP, 1 in 2,000 AEP, 1 in 10,000 AEP and PMF Existing Case results: [Online Digital Portal](#)
- For higher resolution mapping refer to the [Online Digital Portal](#).

11.4.3 Existing Case flood inundation extent and flood levels

The peak Existing Case 1% AEP modelled flood depth is about 3.2 m within the main Native Dog Creek channel. This depth reduces to an average modelled flood depth of around 1 m in other areas of the floodplain. The maximum flood depth is estimated to be up to 1.5 m (321.4 mAHD) where the Project alignment crosses the floodplain.

Figure G2-B in Volume 2 – Appendix G shows the 1% AEP event indicative flood extent and peak water levels within the Native Dog Creek floodplain for the Existing Case.

11.4.4 Flood immunity of existing infrastructure

Table 11.16 presents a summary of overtopping depths for key roads near the Project alignment under a range of design events. Modelling results show that Millmerran-Inglewood Road has an existing low flood immunity in the areas close to the Project alignment.

Table 11.16 Native Dog Creek—Existing Case: Estimated overtopping depths of key infrastructure

Infrastructure location	Overtopping depth (m)							
	PMF	1 in 10,000 AEP	1 in 2,000 AEP	1% AEP	2% AEP	5% AEP	10% AEP	20% AEP
Millmerran-Inglewood Road	5.32	3.25	2.18	1.68	1.43	1.13	0.98	0.88

11.4.5 Existing Case velocities

Peak 1% AEP flood velocities are expected to reach 3.1 m/s in localised areas with the average velocity across the floodplain approximately 0.9 m/s as shown in Figure G2-D in Volume 2 – Appendix G.

11.4.6 Existing Case flood hazard

Peak 1% AEP velocity-depth product within the channels of Native Dog Creek at the Project alignment are in the order of 1.5 m²/s to 2.5 m²/s, and on the floodplain velocity-depth product are generally in the order of 0.3 m²/s to 0.7 m²/s as shown in Figure G2-F in Volume 2 – Appendix G.

11.4.7 Existing Case flood time of inundation

Peak 1% AEP time of inundation within the Native Dog Creek channel are generally in the order of 3 to 4 hours. On the floodplain, time of inundation is generally in the order of 2 to 3 hours as shown in Figure G2-H in Volume 2 – Appendix G.

11.5 Developed Case modelling results—Native Dog Creek

11.5.1 Drainage structures

The hydraulic design of the major drainage structures was undertaken using the hydraulic model (1D and 2D approach).

In the Native Dog Creek floodplain, the Project includes one waterway bridge.

Local drainage structures are presented in Appendix B. These structures were sized through the local drainage design and, depending on their interaction with the Native Dog Creek floodplain, were incorporated in the hydraulic model.

The proposed bridge is summarised in Table 11.17 and shown in Figure G1-F in Volume 2 – Appendix G. The 1% AEP flood level at the proposed bridge is presented in Appendix B.

Table 11.17 Native Dog Creek—Proposed bridge location and details

Chainage (km)	Structure ID	Approximate span (m)	Deck width (m)	Soffit level (mAHD)	Deck superstructure type	Deck depth (mm)
93.93	310-BR07	184	3.97	325.4	Type D1	2,000

Bridges were modelled as an opening in the rail embankment. This approach was adopted due to the high-soffit level of the bridge (approximately 5.9 m above the 1% AEP event peak water surface level) and therefore interaction between the bridge deck and flood waters is unlikely. The optimisation of bridge lengths was balanced between minimising the changes to the hydraulic regime, primarily afflux and velocities, and the cost of replacing bridge spans by large earth embankments.

Blockage of hydraulic structures was assessed in accordance with ARR 2019. The predominant vegetation in the catchment is considered heavily forested; however, given the appreciable size of the bridge spans and the relative size of upstream debris, a blockage factor was not applied to bridges. The chance of significant amounts of debris accumulating against the piers of the bridge are considered low.

11.5.2 Hydraulic design criteria outcomes

The hydraulic model was run for the Developed Case for the range of events (20%, 10%, 5%, 2%, 1% AEP events and 1 in 2,000 AEP, 1 in 10,000 AEP and PMF events) with the outcomes presented in the following sections.

11.5.2.1 Rail immunity and structures results

Appendix B summarises the peak 1% AEP water levels on the upstream side of the Project alignment.

The results of flood modelling indicate that a 1% AEP event flood immunity to the proposed rail formation is achieved for the Project alignment across the Native Dog Creek floodplain, and that peak water levels remain below the proposed rail formation level. There is over 5.9 m freeboard to the bridge soffit, noting that the formation levels were generally dictated by other engineering criteria and constraints.

11.5.3 Flood Impact Objectives outcomes—Native Dog Creek

The Impact of the Developed Case was assessed through inclusion of the proposed rail design and comparison of model results against the Existing Case results.

Changes to flooding behaviour that were assessed, and are reported in the following sections, include potential changes in peak water levels, duration of inundation, velocities, hazard and peak flows within the floodplain:

- Changes in 20% AEP and 1% AEP peak water levels are presented in Figures G4-A and G4-B in Volume 2 – Appendix G
- Changes in 20% AEP and 1% AEP velocities are presented in Figures G4-C and G4-D in Volume 2 – Appendix G
- Changes in 20% AEP and 1% AEP hazard are presented in Figures G4-E and G4-F in Volume 2 – Appendix G
- Changes in 20% AEP and 1% AEP time of inundation are presented in Figures G4-G and G4-H in Volume 2 – Appendix G
- Changes in 20% AEP and 1% AEP velocities and percentage change in velocities with cut-offs applied are presented in Figures G5-A to G5-F in Volume 2 – Appendix G
- Percentage changes in 20% AEP and 1% AEP in hazard are presented in Figures G6-A and G6-B in Volume 2 – Appendix G
- Percentage changes in 20% AEP and 1% AEP in time of inundation and with cut-offs applied are presented in Figures G7-A to G7-D in Volume 2 – Appendix G
- Refer to the [Online Digital Portal](#) for other modelled AEP results and higher resolution mapping.

One-on-one consultation with landowners who are expected to experience changes in flooding behaviour on the property was conducted by ARTC, supported by FFJV. Refer to EIS Chapter 14: Flooding and Geomorphology and EIS Appendix E: Consultation Report for further details.

The Project design outcomes relative to the FIOs (Table 4.2) are presented in the following sections.

Flood sensitive receptors referred to in this section are shown in Figures P1 (1 to 59) in Volume 2 – Appendix P.

11.5.3.1 Flood impacts on flood sensitive receptors

Based on the available aerial imagery, no buildings or critical infrastructure are located within the area affected by afflux and change in flood hazard in the Native Dog Creek floodplain, for modelled events up to and including the 1 in 2,000 AEP event.

11.5.3.2 Flood impacts on State-controlled roads

Within the extent, the only State-controlled road that is influenced by flooding and the Project is the Millmerran-Inglewood Road.

Developed case flooding from the Native Dog Creek catchment and comparison against the FIOs has demonstrated that no adverse impacts on Millmerran-Inglewood Road are expected due to the presence of the Project for all modelled events up to and including the 1% AEP event.

11.5.3.3 Flood impacts on local public roads

The change in peak water levels, velocity, time of inundation and flood hazard (velocity-depth product) for the 1% AEP event were evaluated on local public roads within the hydraulic model domain. Developed case flooding from the Native Dog Creek catchment and comparison against the FIOs has demonstrated that no adverse impacts on local public roads are expected due to the presence of the Project for all modelled events up to and including the 1% AEP event.

11.5.3.4 Flood impacts on existing rail infrastructure

No third-party rail infrastructure is located within the Native Dog Creek hydraulic model boundary.

11.5.3.5 Flood impacts on private land outside the permanent footprint

Developed Case flooding from the Native Dog Creek catchment and comparison against the FIOs has demonstrated that no adverse impacts on private land outside of the permanent footprint is predicted due to the presence of the Project for all modelled events up to and including the 1% AEP event.

11.5.3.6 Flow distribution

A key landowner concern is the potential change in flow distribution. To determine the change in flow distribution maximum velocity-depth modelling results were used to produce change in unit flow maps between Existing and Developed Case conditions. Unit flow is the measure of discharge (velocity-depth product) and is often used for assessing peak flow along a cross-section, or to determine the effective flow corridors within a floodplain.

The presented flow-distribution maps were filtered with a velocity-depth product cut-off of 0.15 m²/s. The cut-off is in line with FIO requirements and was used to assist in identifying main channels (high flow zones) from shallow lower flood zones (i.e. outer extents of the flood surface).

The following points summarise the change in flow distribution for Native Dog Creek and should be considered in conjunction with flow distribution figures supplied in Figure Q7-1 to Figure Q7-8 of Volume 2 – Appendix Q:

- The hydraulic results for Native Dog Creek show negligible changes in global flow distribution patterns for all events up to and including the 1% AEP event
- Some change occurs locally around the bridge opening to the south of the catchment with some negative change occurring adjacent bridge abutments and an increase in flow distribution through the bridge opening in the 1 in 2,000 AEP and 1 in 10,000 AEP events.
- For the PMF event unit flow increases around the bridge and local culvert structures located to the north, approximately 100 m. The increase in flow is balanced by a reduction in the larger floodplain to the southeast of the Project alignment.
- Local increases in unit flow (>20 per cent) are noted immediately upstream and downstream of the rail bridge, and culvert structures. This localised behaviour is expected as flood velocities increase through the cross-drainage infrastructure.
- Reduced overtopping during the PMF event causes a reduction in flow on the eastern/downstream side of the alignment.

Ongoing mitigation measures to reduce changes in flow distribution are likely to occur during future design stages of the Project, and may include further refinement of the bridge abutments and additional survey where necessary.

11.5.4 Extreme events

11.5.4.1 Flood immunity for extreme events

The risk of overtopping of the Project alignment was assessed for a range of extreme events (1 in 2,000 AEP, 1 in 10,000 AEP and PMF) with Table 11.18 presenting the depth of water above the formation level and over the top of rail at each structure. It is noted that the function of the floodplain culverts is to balance flood levels on the upstream and downstream sides of the alignment.

As such, overtopping of the rail is not predicted to result in significant excessive flows or velocities as would occur in a dam embankment overtopping scenario. Further analysis of the overtopping risk to FSRs can be found in Section 21.

Table 11.18 Native Dog Creek—Extreme events: Estimated depth of water above formation and top of rail levels

Chainage (km)	Structure ID	Depth of water above formation level (m)			Depth of water above top of rail (m)		
		1 in 2,000 AEP	1 in 10,000 AEP	PMF	1 in 2,000 AEP	1 in 10,000 AEP	PMF
93.93	310-BR07	-	-	-	-	-	-

Although no overtopping is present at the Native Dog Creek structures, some overtopping is experienced further north along the Project alignment from bridge 310-BR07 where regional structures are not currently proposed.

11.5.4.2 Impacts during extreme events

Flood immunity of FSRs is discussed in Section 11.5.3.1, and maps demonstrating the impacts during extreme events are shown on the [Online Digital Portal](#). In accordance with the Project FIOs, FSRs are considered impacted by extreme flood events if the change in water level is greater than 250 mm in the 1 in 2,000 AEP event, pending the receptor classification (i.e. at existing habitable dwellings and/or commercial and industrial buildings/premises).

As indicated above, based on the available aerial imagery, no buildings or critical infrastructure are located within the hydraulic model domain of Native Dog Creek floodplain and therefore no assessment has been completed.

11.5.5 Sensitivity analysis—Native Dog Creek

The sensitivity of the model to an increase in rainfall intensity (i.e. to reflect climate change scenario was assessed).

11.5.5.1 Climate change

The potential impacts of climate change in the Native Dog Creek floodplain were assessed for the 1% AEP design event to determine the sensitivity of the Project to the potential long-term changes in climate. The assessment was undertaken in accordance with ARR 2019.

The Representative Concentration Pathways 8.5 (2090 horizon) climate change scenario has been adopted for the Project with an associated increase in rainfall intensity of 23.9 per cent across the catchment area.

Due to no FSRs being present with the hydraulic model domain, Table 11.19 presents the structure's performance with Representative Concentration Pathways 8.5 climate change conditions. For the 1% AEP event, the change in peak water levels for the Representative Concentration Pathways 8.5 climate change scenario are presented in Figure G8-A in Volume 2 – Appendix G.

Climate change results are expected to increase peak water levels upstream of the Project alignment by up to 0.1 m at proposed bridge 310-BR07 for the 1% AEP event. The Project alignment is expected to retain 1% AEP flood immunity, with over 5 m freeboard to formation level under the climate change scenario.

Table 11.19 Native Dog Creek—1% AEP event with Representative Concentration Pathways 8.5 conditions: Structure performance

Structure ID	Structure type	1% AEP peak water level (mAHD)	1% AEP +CC peak water level (mAHD)	Difference in peak water level (m)	Freeboard to rail formation with CC (m)
310-BR07	Bridge	321.5	321.6	+0.1	5.9

Table note:

CC = Climate Change

The downstream extents of these impacts are similar to those under the 1% AEP event.

11.5.5.2 Blockage

Blockage of hydraulic structures was assessed in accordance with ARR 2019. Despite the catchment being heavily forested, ARR guidelines determined that the likelihood of significant amounts of debris accumulating against the piers of the bridge is low. Therefore, a zero-blockage factor was applied at the Native Dog Creek bridge. Additionally, there are no regional culverts in the Native Dog Creek floodplain; hence, no sensitivity scenarios were conducted.

During detailed design the blockage factors will be reviewed in line with the final design and local catchment conditions. This may result in a varied and/or lower blockage factors being applied along the Project alignment. Should impacts be determined due to a change in blockage factor, additional mitigation measures will be considered in detailed design.

11.6 Additional modelling items to be considered in detailed design

The following points summarise additional items to be included in the flood models for detailed design as per agreement with the Expert Flood Panel:

- Additional 'as-built' information will be sought from DTMR, where available, to accurately represent existing infrastructure within the modelling domain including DTMR Bridge – TMR 24518. Existing bridges will be modelled using Ifcsh during detailed design.
- A review of the latest TUFLOW version will be undertaken at the commencement of detailed design to determine whether updates of the TUFLOW version from 2020-10-AD-iSP-w64 are beneficial to modelling
- Undertake additional model strengthening by improving the representation of cross-drainage infrastructure, survey data, and bridge structures within the hydraulic model
- Review and update the locations of the inflows to the TUFLOW model
- In order to meet FIOs, additional analysis of cross-drainage infrastructure may be required, along with potential stakeholder consultation.

Further details of mitigation measures can be found in Section 22. A review of proposed mitigation measures will be completed during the detailed design stage of the Project after hydraulic models have been updated to capture the above changes.

11.7 Planned and future roads

Consultation with the DTMR has not identified any planned or future road upgrades within the Native Dog Creek catchment. ARTC will continue to consult and engage with DTMR to determine any future or planned road projects that may need to be considered as part of the detailed design hydraulic modelling.

Consultation with GRC has not identified any planned or future road upgrades within the Native Dog Creek catchment. ARTC will continue to consult and engage with TRC to determine any future or planned road projects that may need to be considered as part of the detailed design hydraulic modelling.

12 Cattle Creek

Cattle Creek is a well-defined watercourse with minor breakout flow paths in the meandering sections of the creek. The creek system has well vegetated overbank areas which assists flow to remain within the main channel rather than breaking into overbank areas.

Under the Existing Case 1% AEP event, the estimated flood depth in Cattle Creek channel is up to 4.5 m with approximately 1.5 m deep water on the floodplain area. The floodplain inundated extent is approximately 100 m wide.

The location of the Project alignment in relation to Cattle Creek is shown in Figure H1-A in Volume 2 – Appendix H.

12.1 Data collection and review—Cattle Creek

Available background information including existing hydrologic and hydraulic models, survey, streamflow data, available calibration information and anecdotal flood data has been gathered. Data was sourced from a wide range of stakeholders including:

- GRC—existing flood studies
- BoM—rainfall gauging data
- DTMR—existing infrastructure details.

12.1.1 Previous studies

A number of previous hydrology and hydraulic studies were sourced as part of this assessment. A review of each study was undertaken to determine suitability for use on the Project as documented in the following sections.

12.1.1.1 Inglewood Flood Study, Engeny, 2015

Engeny was commissioned by GRC to undertake a flood study of Inglewood. The study objectives were ‘to define the nature, extent and risks of flooding in Inglewood in order to inform disaster management planning and response, as well as control future development.’ (Engeny, 2015). An URBS hydrologic model and TUFLOW (1D/2D) hydraulic model were developed.

The *Inglewood Flood Study* was used extensively for the development of the Nicol Creek, Bringalily Creek, Native Dog Creek, and Cattle Creek hydrology models.

12.1.2 Survey

ARTC provided LiDAR data from 2015 as 1 m grid DEM tiles. Using GIS software, a DEM was generated with a 1 m grid resolution for use in the Project, based on the 2015 dataset. This was used for modelling within the permanent footprint and up to the full extent of the 2015 LiDAR where relevant.

In areas that were not covered by the LiDAR provided by ARTC, LiDAR tiles were sourced from Geoscience Australia. The DEM datasets utilised for modelling were based on survey flown between 2009 and 2015. SRTM data was used for catchment delineation, where no LiDAR data could be sourced, to inform the hydrologic modelling.

The survey data sources and DEM developed for Cattle Creek are shown in Figure H1-B in Volume 2 – Appendix B.

12.1.3 Aerial imagery

Aerial imagery of the Cattle Creek study area was provided by ARTC and was used to identify and confirm topographic and vegetative characteristics of the study area. Aerial imagery captured in 2015 was made available. Additional imagery outside the study area was sourced from QGIS imagery in an open-source format.

12.1.4 Existing drainage structure data

No available DTMR as-constructed drawings were available for culvert and bridge structures.

Additional information will be refined as local survey is progressively completed during detailed design. Details of infrastructure included within the Cattle Creek flood assessment is in Section 12.3.2.

12.1.5 Stream gauge data

No streamflow gauges exist within the Cattle Creek catchment study area.

12.1.6 Rainfall data

Several daily and sub-daily rainfall stations are located near and around the Cattle Creek catchment; however, since there are no streamflow gauges to use for model calibration, no historical rainfall data was sought.

12.1.7 Anecdotal and observed flood data

No anecdotal or observed flood data was available for this area of Cattle Creek.

12.1.8 Site inspection

A site inspection was undertaken during February 2018. During the site inspection, all major waterway crossings were visited and inspected with photographs taken and details recorded of the crossing, existing drainage structures and surrounding catchment. An assessment of the relative roughness and blockage potential was undertaken during the site inspection.

12.2 Hydrologic model development—Cattle Creek

12.2.1 Model setup

Cattle Creek is one of four major waterways that cross the Project alignment before flowing into Canning Creek. The other waterways are Bringalily Creek, Native Dog Creek, and Nicol Creek. Due to their proximity to one another, all four waterways were modelled in a unified Canning Creek model (using the URBS modelling platform). In addition to the below sections, refer to Section 9.2 (Nicol Creek Catchment) for further details on the development of the hydrologic model and associated modelling parameters.

The Project alignment crosses two major watercourses in the vicinity of Cattle Creek. The catchment area upstream of the main Cattle Creek crossing is approximately 65 km².

12.2.2 Design model parameters

Hydrologic information to assist estimation of design event flows was sourced from the ARR 2019 Data Hub, as summarised in Table 12.1.

Table 12.1 Cattle Creek—Summary of URBS model design event inputs

Input parameter	Remarks
Design rainfall	Four IFDs were downloaded from the BoM's website to represent the spatial rainfall variability in the Cattle Creek catchment.
Extreme event rainfall	Probable Maximum Precipitation depths for durations up to 6 hours (for use in modelling the PMF event) were obtained using the method presented in the Bulletin 53 (BoM, 2003). The rainfall depths for the 1 in 10,000 AEP event were estimated using the interpolation method presented in ARR 2019 Book 8 Section 3.5.
Losses	The losses were initially adopted from the <i>Inglewood Flood Study</i> (Engeny, 2015) URBS model (Initial loss – 15 mm, Continuing loss – 1 mm/hr). Adjustments were made to reconcile URBS flows to area-scaled FFA.
Areal reduction factor	Parameters were adopted for the semi-arid inland Queensland region. The catchment area of Cattle Creek is approximately 64.8 km ² , which yields an ARF between 79.6% and 97.7% depending on design storm event AEP and duration.
Ensemble TPs	Central Slopes Region.
Pre-burst depths	Median pre-burst depths were downloaded from the ARR 2019 Data Hub for each sub-catchment. Pre-burst depths vary by design storm event AEP and duration. Pre-burst depths were applied to the model by reducing the initial losses for each storm event.

12.2.2.1 Spatial distribution

The ARR 2019 design event methodology was adopted for this assessment. ARR 2019 recommends adopting the spatially varying method for catchments larger than 20 km². The Cattle Creek catchment is approximately 65 km². A total of four IFDs were adopted for this model.

12.2.2.2 Rainfall depth estimation

Design rainfall depth for 20% to 1 in 2,000 AEP event were manually obtained from BoM website for each sub-catchment. As an example, the sample IFD extracted for the Cattle Creek hydrologic model is provided in Table 12.2.

Table 12.2 Rainfall depth (mm) used for the Cattle Creek catchment centroid

Duration (hr)	Annual Exceedance Probability (% AEP)						
	50	20	10	5	2	1	1 in 2,000
0.25	17.6	24.5	29.1	33.6	39.7	44.4	70.6
0.50	24.1	33.4	39.8	46.1	54.5	61.0	97.1
1.00	30.5	42.2	50.3	58.3	69.0	77.2	123.0
2.00	37.0	50.8	60.4	69.9	82.8	92.7	147.0
3.00	41.0	55.9	66.3	76.8	90.9	102.0	162.0
4.50	45.4	61.4	72.7	84.0	99.5	112.0	177.0
6.00	48.8	65.7	77.6	89.7	106.0	120.0	189.0
12.00	58.3	77.9	91.9	106.0	126.0	143.0	226.0
24.00	70.0	93.6	111.0	128.0	153.0	174.0	277.0
48.00	83.4	112.0	134.0	156.0	188.0	213.0	360.0
72.00	91.3	124.0	148.0	174.0	209.0	237.0	385.0

12.2.2.3 Areal reduction factor

The ARF was estimated using the coefficients for semi-arid inland Queensland presented in Table 12.3 and was applied to all IFD areas. The URBS software directly downloaded these coefficients from ARR 2019 Data Hub to estimate the ARF using Equation 2.4.1 of ARR 2019 Book 2.

The estimated ARF for Cattle Creek catchment vary between 79.6 to 97.7 per cent depending on design storm event AEP and duration.

Table 12.3 Cattle Creek—Areal reduction factor coefficients from ARR 2019 Data Hub

a	b	c	d	e	f	g	h	i
0.16	0.28	0.25	0.31	0.00	1.00	0.04	0.00	0.00

12.2.2.4 Temporal patterns

Temporal patterns for 20% AEP to 1 in 2,000 AEP events were adopted based on the 10-burst ensemble approach presented in the ARR 2019. The TPs and pre-burst data were directly downloaded by URBS software from the ARR 2019 Data Hub at the centroid of Cattle Creek catchment.

Temporal patterns for the 1 in 10,000 AEP and PMF design events for duration up to 6 hours were obtained from *The Estimation of Probable Maximum Precipitation in Australia: Generalised Short-Duration Method* (BoM, 2003).

The adopted TPs for all modelled events are shown in Table 12.4.

Table 12.4 Cattle Creek—Adopted temporal patterns

AEP (%)	Critical duration storm/temporal pattern
20% AEP	Pattern 4
10% AEP	Pattern 6
5% AEP	Pattern 4
2% AEP	Pattern 6
1% AEP	Pattern 9
1 in 2,000 year (0.05%) AEP	Pattern 9
1 in 10,000 year (0.01%) AEP	PMP temporal pattern
PMF	PMP temporal pattern

12.2.2.5 Pre-burst

Median pre-burst depths for standard durations from 0.5 hours to 72 hours for a 1% AEP design event are between 1.1 mm and 13.1 mm (Data Hub accessed November 2018). Due to the small depth of pre-burst in this area, the impact of pre-burst condition is considered to be minimal in this catchment. The adopted pre-burst depths are summarised in Table 12.5.

Table 12.5 Cattle Creek—Adopted pre-burst depths

Duration (hrs)	Pre-burst depth (mm)	Pre-burst depth (mm)	Pre-burst depth (mm)	Pre-burst depth (mm)	Pre-burst depth (mm)	Pre-burst depth (mm)
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP
0.5	0.6	0.8	0.9	1.1	1.7	2.3
0.75	0.6	0.8	0.9	1.1	1.7	2.3
1.0	0.6	0.8	0.9	1.1	1.7	2.3
1.5	0.2	0.3	0.4	0.5	1.6	2.4

Duration (hrs)	Pre-burst depth (mm)	Pre-burst depth (mm)	Pre-burst depth (mm)	Pre-burst depth (mm)	Pre-burst depth (mm)	Pre-burst depth (mm)
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP
2.0	0.2	0.3	0.3	0.3	1.7	2.8
3.0	0.1	0	0	0	2.8	5
6.0	0.2	0.6	0.8	1.1	1.1	1.1
12.0	0	0.2	0.2	0.3	5.2	8.8
18.0	0	0.2	0.2	0.3	5.2	8.8
24.0	0	0.3	0.5	0.7	7.8	13.1
48.0	0	0.5	0.9	1.2	6.5	10.4
72.0	0	0	0	0.1	0.3	0.4

12.2.2.6 Design rainfall losses

In this assessment, an initial and continuing loss model was applied, where the lost rainfall depth does not contribute to the catchment runoff. In the loss model, it was assumed that there is an initial loss of rainfall depth followed by a constant continuing loss (mm/hr) throughout the rainfall event.

A range of design rainfall losses were trialled for the design events. Initial and continuing losses in the URBS model were adjusted until there was a reasonable match between the modelled flows and those from the scaled FFAs. The adopted design event losses are provided in Table 12.6.

Table 12.6 Cattle Creek—Adopted design event losses

AEP	Initial storm loss (mm)	Continuing loss (mm/hr)
20%	42.3	2.8
10%	38.6	2.6
5%	32.6	2.2
2%	22.3	1.5
1%	6.4	0.4

12.2.2.7 Extreme event rainfall

Probable Maximum Precipitation depths for durations up to 6 hours (for use in modelling the PMF event) were obtained using the methodology presented in *The Estimation of Probable Maximum Precipitation in Australia: Generalised Short-Duration Method* (BoM, 2003). Once the PMP was estimated, rainfall for events between 1 in 2,000 AEP and the PMF was then interpolated.

For the 1 in 10,000 AEP event, ARF were applied based on interpolation methodology presented in ARR 2019 Section 3.5 in Book 8 and the rainfall depths were estimated accordingly. The parameters adopted for PMP rainfall estimation are presented in Table 12.7 and the adopted extreme rainfall depths are provided in Table 12.8.

Table 12.7 Cattle Creek—Input data for Probable Maximum Precipitation estimation

Factor	Value
EAF	1
MAF	0.8
Portion of rough catchment	100%
Cattle Creek location and catchment Area (km ²)	(-28.187, 151.095) and 64.8 km ²

Table 12.8 Cattle Creek—Adopted extreme event rainfall depths (mm)

Duration (hour)	Cattle Creek	
	1 in 10,000 AEP rainfall depth (mm)	PMP rainfall depth (mm)
0.25	87	146
0.5	120	214
0.75	141	271
1	152	325
1.5	173	419
2	186	484
2.5	196	543
3	204	588
4	216	666
5	227	732
6	236	780

12.2.3 Event selection for calibration

Insufficient data was available to calibrate historical flood events in the Cattle Creek catchment.

12.2.4 Model calibration

Insufficient data was available to calibrate historical flood events in the Cattle Creek catchment.

12.2.5 Model validation

The RFFE was used to validate peak flow estimates for the Cattle Creek hydrologic model. The online tool was used to obtain an estimation of catchment peak flow at the outlet of Cattle Creek catchment. Input parameters have been provided in Table 12.9.

Table 12.9 Cattle Creek catchment—Model inputs for the Regional Flood Frequency Estimation

Model parameter	Input value
Region name	East Coast
Region version	1
Latitude at catchment outlet (degree)	-28.23
Longitude at catchment outlet (degree)	151.155
Latitude at catchment centroid (degree)	-28.165
Longitude at catchment centroid (degree)	151.074
Distance of the nearest gauged catchment in the database (km)	37.97
Catchment area (km ²)	64.8
Design rainfall intensity, 50% AEP and 6 hr duration (mm/hr)	8.106
Design rainfall intensity, 2% AEP and 6 hr duration (mm/hr)	17.745

In addition to the RFFE analysis, further validation using DTMR's QRT was undertaken. This approach provides an estimate of peak flow by using readily available catchment characteristics such as catchment area, and the 2% AEP 72-hour rainfall intensity.

The estimated flood flows at the proposed Cattle Creek crossing are presented in Figure 12.1 and Table 12.10. Review of the results show the modelled peak flow estimates are lower than the mean estimates produced by the RFFE method. Despite this, all URBS design quantiles are within the 95 per cent confidence limits of the RFFE. It should also be noted that there is reduced confidence in the RFFE outputs, as the unusual shape of the Cattle Creek catchments is outside of the RFFE's reliable range. A closer match is noted when comparing URBS peak flow estimates to QRT estimates. As the URBS flows are consistently lower than the RFFE and QRT estimates, further review of the Cattle Creek hydrology will be undertaken during detailed design.

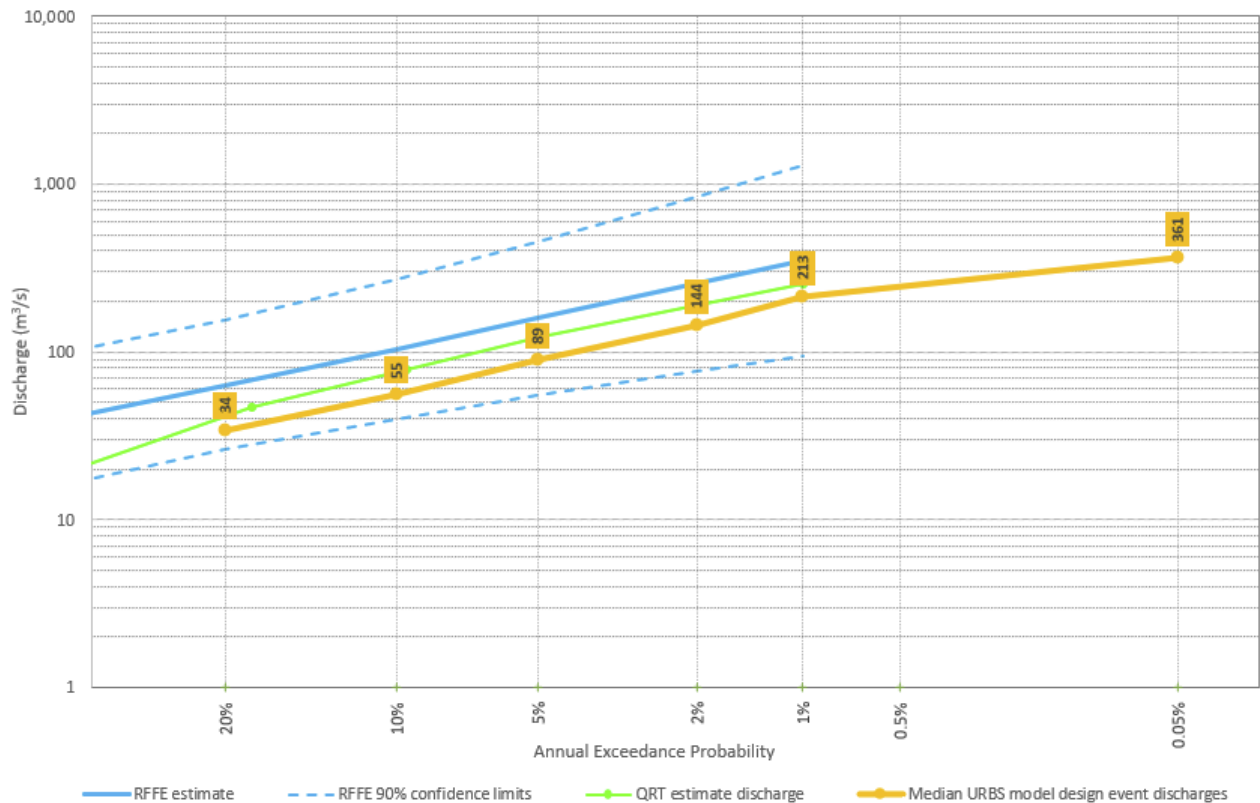


Figure 12.1 Estimate of flows at the Cattle Creek crossing

Table 12.10 Estimate of flows at the Cattle Creek crossing

AEP (%)	RFFE – lower bound 95% confidence level (m³/s)	RFFE – estimate of flow (m³/s)	RFFE – upper bound 95% confidence level (m³/s)	Quantile Regression Technique (m³/s) ¹	URBS model flows (m³/s)
20	26.1	63.1	154	46.8	34
10	40.0	104	271	78.7	55
5	54.7	158	449	123.1	89
2	76.2	254	837	191.6	144
1	93.8	349	1280	252.9	213
1 in 2,000	-	-	-	-	361
1 in 10,000	-	-	-	-	941
PMF	-	-	-	-	3,770

Table note:

1 The QRT method estimates the 39.3%, 18.1%, 9.5% and 4.9% AEP instead of 50%, 20% and 10% and 5%, respectively

12.3 Hydraulic model development—Cattle Creek

A 2D modelling approach was adopted to simulate the flood regime around the proposed rail crossing at Cattle Creek. The platform used for hydraulic modelling is TUFLOW HPC software package (2020-10-AD) with SGS and QT approach. The processes and assumptions adopted throughout the development of the hydraulic model are described in the following sections.

12.3.1 Model setup

The setup of the hydraulic model is summarised in Table 12.11.

Table 12.11 Cattle Creek— Hydraulic model summary

Parameter	Information
Completion date	June 2022
AEPs assessed	20%, 10%, 5%, 2%, 1%, 1 in 2,000, 1 in 10,000 AEP and PMF
Hydraulic model build	TUFLOW HPC GPU – version 2020-10-AD-iSP-w64
Model extent	Refer to Figure H1-D in Volume 2 – Appendix H
Grid size	5 m
SGS Size	1 m
QT Parameters	2.5 m, 2 nd Order
DEM (year flown)	ARTC dataset (2015)
Roughness	Spatially varying roughness values compliant with industry norms.
Eddy viscosity	Wu (default)
Model calibration	N/A
D/S model boundary	Height-Discharge (HQ) Boundary with normal slope approximated based upon topography dataset.
Hydraulic model timestep	Adaptive Timestep
Modelled scenarios	Existing Case, Developed Case
Sensitivity analysis	Blockage – 0%, 50% Climate change

The hydraulic model extent and the spatial distribution of land use in the 2D model domain is presented in Figure H1-D and Figure H1-E in Volume 2 – Appendix H.

12.3.2 Model topography and grid size

The topography within the TUFLOW hydraulic model was based on 1 m DEM data collected during 2015, which was supplied by ARTC. A 5 m x 5 m grid size was selected to allow the features of the channels and floodplain to be represented with sufficient accuracy while maintaining efficient model run times. The 5 m x 5 m grid size is considered adequate to represent floodplain topography and is consistent with the approach typically adopted in similar flood studies.

The 2D model topography was modified adjacent to the culverts so that the existing drainage flows, existing road/rail crowns and levee banks were represented accurately in the model. For the Developed Case, the topography was modified to include the Project rail embankment, drainage structures and associated infrastructure.

12.3.2.1 Sub-grid Sampling and Quadtree mesh refinement

The Cattle Creek hydraulic model was established using TUFLOW's 2020-10-AD model version, including SGS and QT add-on modules. The below points provide a brief description of each module, along with the key parameters adopted for this assessment.

- Traditional grid-mesh based 2D hydraulic models sample the underlying DEM by taking a single elevation at the cell centres and cell faces. Sub-grid sampling allows for increased resolution by sampling a greater number of points of the underlying DEM to establish a water surface elevation versus volume relationship for each grid, along with a water surface elevation versus width relationship (wetted perimeter) at cell sides. The adoption of SGS allows increased representation of storage and conveyance between cells. A SGS distance of 1 m was adopted for this assessment.
- Quadtree mesh is constructed by subdividing the base mesh-grid size into four smaller square cells. Cells can be further sub-divided into four, and so on, allowing for additional resolution in location of interest. The base hydraulic model grid cell size (5 m x 5 m) was revised to include additional spatial resolution of approximately 2.5 m, to better represent model velocity at the entry and exit of culverts. Quadtree meshes were applied in the model covering minimum 25 m upstream and 100 m downstream of the high-risk areas (e.g. culverts).

12.3.3 Roughness

The hydraulic roughness generally reflects the types of development and ground cover that exists within the hydraulic model extent. The distribution of roughness categories adopted for this assessment were based on the information supplied in the land-use layer from Queensland Globe database and aerial imagery, and confirmed during the site inspection. Specific roughness values applied to the model are detailed in Table 12.12.

Table 12.12 Cattle Creek catchment—Adopted roughness values

Category	Manning's 'n' value
River	0.035
Sealed roads	0.015
Unsealed roads	0.025
Rural residential	0.09
Urban residential	0.10
Commercial	0.06
Open space—Minimum vegetation	0.045
Open space—Moderate vegetation	0.055
Open space—Heavy vegetation	0.065
Open water	0.025
Rail	0.025
Riparian	0.04
Crops ¹	0.1085

Table note:

- 1 A uniform value was adopted for the crop areas, due to the seasonal and spatial variations in vegetation distribution and the uncertainty this brings to the design event approach

12.3.4 Hydraulic structures

The following existing structures are located in the hydraulic model domain:

- One existing bridge on Millmerran-Inglewood Road.

Assumed details of the DTMR bridge are included in Table 12.13. The bridge was modelled as an opening in the hydraulic model. The bridge will be modelled using lfcsh feature during the detailed design stage.

Table 12.13 Cattle Creek—Existing bridge

Bridge ID	Approximate span (m)	Deck width (m)	Deck level (mAHD)	Deck superstructure type	Deck depth (mm)
TMR 24519	55	9.20 ¹	324.6	T44	1,050

Table note:

¹ Estimated from aerial imagery

12.3.5 Boundary conditions

The following boundary conditions were applied within the hydraulic model:

- Delineated sub-catchments in the hydrologic model were used as internal flow boundaries within the hydraulic model extent (SA polygons). The modelled flows in the hydrologic model were applied to the lowest points of each SA polygon.
- A height-discharge boundary (HQ) was applied for the downstream outflow model boundary. It was confirmed that it was a sufficient distance away from the area of interest so as not to impact on modelling results.

12.4 Existing Case modelling results—Cattle Creek

12.4.1 Hydrologic model peak flow assessment

A critical duration assessment was undertaken to determine which storm duration(s) produced peak flood flows where the major waterways are intersected by the Project alignment and at the downstream outlet of the model. To assess the critical storm duration the following methodology was adopted:

- The models were modelled for a range of AEP events:
 - Each AEP was modelled for a range of durations
 - Each duration was modelled for each of the 10 associated TPs.
- A critical duration assessment was undertaken at the locations mentioned above to determine which duration produced the highest median flow of the 10 TPs for each event.

The adopted design loss rates for the hydrologic model are provided in Section 12.2.2.6.

Table 12.14 presents the critical duration and TPs adopted and Table 12.15 presents estimated peak flow applied to the hydraulic model for a number of key locations (Figure H1-D in Volume 2 – Appendix H).

Table 12.14 Cattle Creek—Critical flood durations and temporal patterns

AEP (%)	Critical storm duration/temporal pattern
20	12 hour – Pattern 4
10	12 hour – Pattern 6
5	12 hour – Pattern 4
2	12 hour – Pattern 6
1	9 hour – Pattern 9
1 in 2,000	9 hour – Pattern 9
1 in 10,000	6 hour – PMP temporal pattern
PMF	6 hour – PMP temporal pattern

Table 12.15 Cattle Creek—1% AEP event peak flow at key locations as applied in the hydraulic model

Sub-catchment name	1% AEP event peak flow (m ³ /s)	Critical storm (hrs) duration and temporal pattern
20T	246.6	9 hour – Pattern 9
21L	20.7	9 hour – Pattern 9
22L	2.9	9 hour – Pattern 9
CAT	213.2	9 hour – Pattern 9

12.4.2 Existing Case flood maps

Existing conditions flood maps have been presented in Volume 2 – Appendix H and via an [Online Digital Portal](#). Volume 2 – Appendix H contains the existing 20% and 1% AEP flood maps, while the [Online Digital Portal](#) contains the remainder of the modelled events. The location of each map set is summarised below:

- 20% AEP peak levels: Figure H2-A
- 1% AEP peak levels: Figure H2-B
- 20% AEP peak velocity: Figure H2-C
- 1% AEP peak velocity: Figure H2-D
- 20% AEP peak hazard: Figure H2-E
- 1% AEP peak hazard: Figure H2-F
- 20% AEP peak time of inundation: Figure H2-G
- 1% AEP peak time of inundation: Figure H2-H
- 20% AEP, 10% AEP, 5% AEP, 2% AEP, 1% AEP, 1 in 2,000 AEP, 1 in 10,000 AEP and PMF Existing Case results: [Online Digital Portal](#)
- For higher resolution mapping refer to the [Online Digital Portal](#).

12.4.3 Existing Case flood inundation extent and flood levels

The peak modelled flood depth for the Existing Case 1% AEP event is approximately 4.5 m within the main Cattle Creek channel. This depth reduces to an average modelled flood depth of around 1.5 m in other areas of the floodplain. The peak flood depth is estimated to be up to 3.0 m (323.7 mAHD) where the Project alignment crosses the floodplain.

Figure H2-B in Volume 2 – Appendix H shows the 1% AEP flood extent and peak water levels within the Cattle Creek floodplain for the Existing Case.

12.4.4 Flood immunity of existing infrastructure

Table 12.16 presents a summary of overtopping depths for key roads near the Project alignment under a range of design events. Modelling results show that Millmerran-Inglewood Road has an existing low flood immunity in the areas close to the Project alignment.

Table 12.16 Cattle Creek—Existing Case: Estimated overtopping depths of key infrastructure

Infrastructure	Overtopping depth (m)							
	PMF	1 in 10,000 AEP	1 in 2,000 AEP	1% AEP	2% AEP	5% AEP	10% AEP	20% AEP
Millmerran-Inglewood Road	6.19	4.07	3.13	2.51	2.15	1.80	1.57	1.37

12.4.5 Existing Case velocities

Peak Existing Case 1% AEP flood velocities are expected to reach approximately 3.5 m/s in localised areas of the main creek channel, whereas the average velocity across the floodplain is approximately 0.9 m/s, as shown in Figure H2-D in Volume 2 – Appendix H.

12.4.6 Existing Case flood hazard

For the Existing Case 1% AEP event, a peak velocity-depth product of approximately 4 m²/s to 4.5 m²/s is recorded within Cattle Creek, and generally in the order of 0.3 m²/s to 0.6 m²/s on the floodplain, as shown in Figure H2-F in Volume 2 – Appendix H.

12.4.7 Existing Case flood time of inundation

Peak 1% AEP time of inundation within the Cattle Creek channel is generally in the order of 10 to 12 hours. On the floodplain, time of inundation is generally in the order of 5 to 9 hours as shown in Figure H2-H in Volume 2 – Appendix H.

12.5 Developed Case modelling results—Cattle Creek

12.5.1 Drainage structures

The hydraulic design of the major drainage structures was undertaken using the hydraulic model (1D and 2D approach).

In the Cattle Creek floodplain, the Project includes the following floodplain (or regional structures):

- One waterway bridge
- One RCP location (a total of six cells)
- One RCBC location (a total of 15 cells).

Local drainage structures are presented in Appendix B. These structures were sized through the local drainage design and, depending on their interaction with the Cattle Creek floodplain, were incorporated in the hydraulic model.

Design bridge details have been summarised in Table 12.17 and details of the proposed culvert structures are summarised in the Cross-Drainage Register located in Appendix B. The location of cross-drainage features has been presented in Figure H1-F in Volume 2 – Appendix H.

Table 12.17 Cattle Creek—Proposed bridge locations and details

Chainage (km)	Structure ID	Approximate span (m)	Deck width (m)	Soffit level (mAHD)	Deck superstructure type	Deck depth (mm)
88.28	310-BR06	138	3.97	327.38	Type D1	2,000

Bridges were modelled as an opening in the rail embankment. This approach was adopted due to the high soffit level of the bridge (approximately 5 m above the 1% AEP event peak water surface level); therefore, interaction between the bridge deck and flood waters is unlikely. The optimisation of bridge lengths was balanced between minimising the changes to the hydraulic regime, primarily afflux and velocities, and the cost of replacing bridge spans by large earth embankments. Bridge locations will be updated in detailed design to be modelled utilising LFC shapefiles.

Blockage of hydraulic structures was assessed in accordance with ARR 2019. Despite the predominant vegetation in the catchment being heavily forested, no allowance for blockage at bridges was made, as the likelihood of significant amounts of debris accumulating against the piers of the bridge is considered low. Therefore, a zero-blockage factor was applied at the Cattle Creek bridge.

12.5.2 Hydraulic design criteria outcomes

The hydraulic model was run for the Developed Case for the range of events (20%, 10%, 5%, 2%, 1% AEP events and 1 in 2,000 AEP, 1 in 10,000 AEP and PMF events) with the outcomes presented in the following sections.

12.5.2.1 Rail immunity and structures results

Appendix B summarises the peak 1% AEP water levels at structures, on the upstream side of the Project alignment.

The results of flood modelling indicate that a 1% AEP event flood immunity to the proposed rail formation is achieved for the Project alignment across the Cattle Creek floodplain, and that peak water levels remain below the proposed rail formation level. There is 3.3 m freeboard to the formation level at culvert locations, noting that the formation levels were generally dictated by other engineering criteria and constraints.

12.5.3 Flood Impact Objectives outcomes—Cattle Creek

The impact of the Developed Case was assessed through inclusion of the proposed rail design and comparison of model results against the Existing Case results.

Changes to flooding behaviour that were assessed, and are reported in the following sections, include potential changes in peak water levels, duration of inundation, velocities, hazard and peak flows within the floodplain:

- Changes in 20% AEP and 1% AEP peak water levels are presented in Figures H4-A and H4-B in Volume 2 – Appendix H
- Changes in 20% AEP and 1% AEP velocities are presented in Figures H4-C and H4-D in Volume 2 – Appendix H
- Changes in 20% AEP and 1% AEP hazard are presented in Figures H4-E and H4-F in Volume 2 – Appendix H
- Changes in 20% AEP and 1% AEP time of inundation are presented in Figures H4-G and H4-H in Volume 2 – Appendix H
- Changes in 20% AEP and 1% AEP velocities and percentage change in velocities with cut-offs applied are presented in Figures H5-A to H5-F in Volume 2 – Appendix H
- Percentage changes in 20% AEP and 1% AEP in hazard are presented in Figures H6-A and H6-B in Volume 2 – Appendix H
- Percentage changes in 20% AEP and 1% AEP in time of inundation and with cut-offs applied are presented in Figures H7-A to H7-D in Volume 2 – Appendix H
- Refer to the [Online Digital Portal](#) for other modelled AEP results and higher resolution mapping.

One-on-one consultation with landowners who are expected to experience changes in flooding behaviour on the property was conducted by ARTC, supported by FFJV. Refer to EIS Chapter 14: Flooding and Geomorphology and EIS Appendix E: Consultation Report for further details.

The Project design outcomes relative to the FIOs (Table 4.2) are presented in the following sections.

Flood sensitive receptors referred to in this section are shown in Figures P1 (1 to 59) in Volume 2 – Appendix P.

12.5.3.1 Flood impacts on flood sensitive receptors

Based on the available aerial imagery, no buildings or critical infrastructure are located within the area impacted by afflux and change in flood hazard in the Cattle Creek floodplain for all modelled events up to and including to the 1 in 2,000 AEP event.

12.5.3.2 Flood impacts on State-controlled roads

Within the extent of the hydraulic model, the only State-controlled road that is influenced by flooding and the Project is Millmerran-Inglewood Road.

Developed Case flooding from the Cattle Creek catchment and comparison against the FIOs has demonstrated that no adverse impacts on Millmerran-Inglewood Road are expected due to the presence of the Project for all modelled events up to and including the 1% AEP event.

12.5.3.3 Flood impacts on local public roads

The change in peak water levels, flood hazard (velocity-depth product), time of inundation and velocity up to the 1% AEP event were evaluated on local public roads within the hydraulic model domain. One local track (two segments) has been identified to be impacted by afflux, a change in time of inundation, velocity and hazard due to the introduction of the Project. Figure 12.2 identifies the location of these exceedances. Table 12.18 identifies FIO exceedances for this track. Exceedance justification and mitigation measures have been detailed in Table 12.18.

Due to the alignment of the Project, this track will be removed as part of construction and therefore the identified FIO exceedances are not deemed significant.

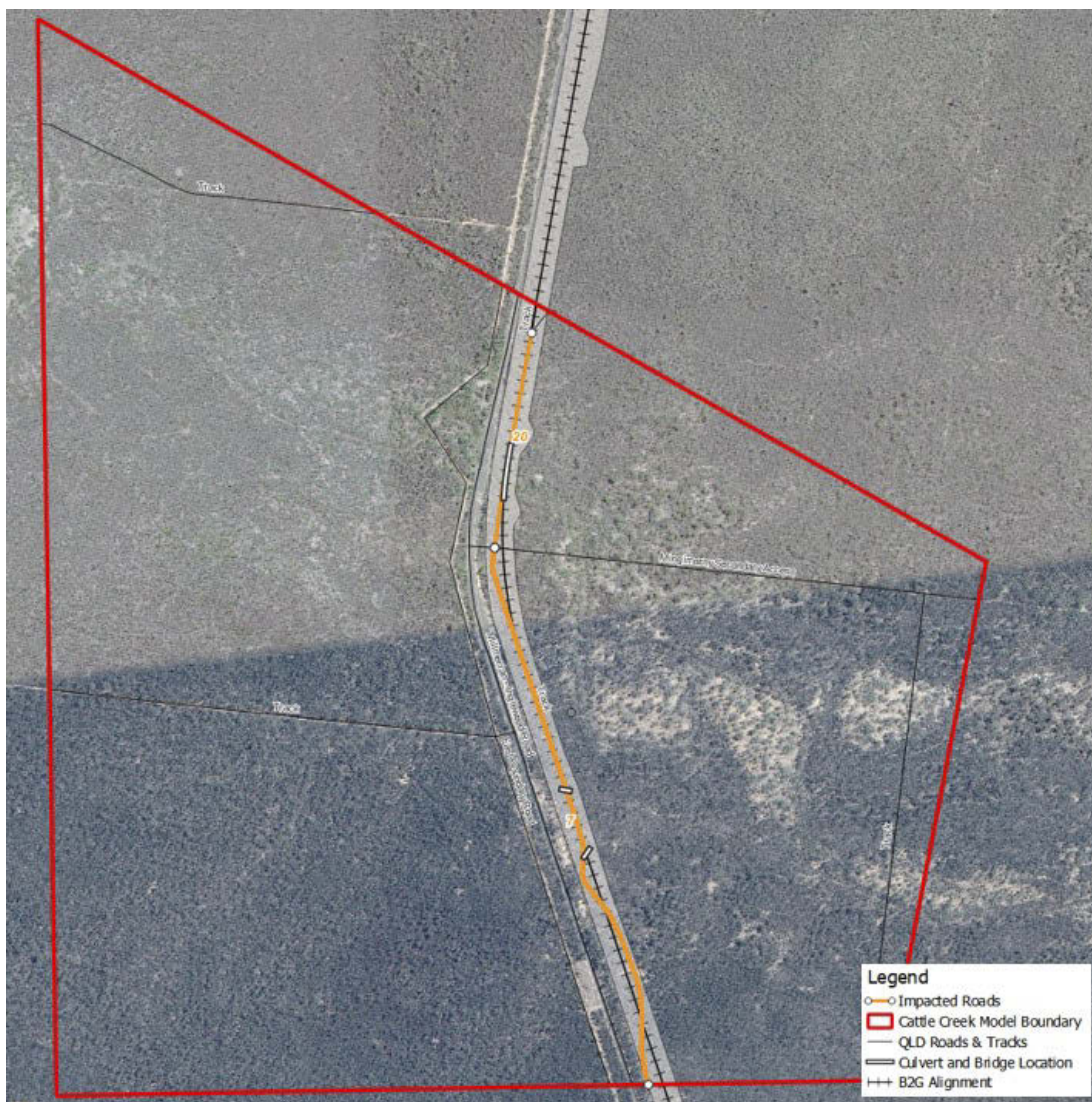


Figure 12.2 Cattle Creek—Impacted local public roads

Table 12.18 Cattle Creek – Summary of impacts on local public roads

Road Object ID	Road Segment	Road Name	Surface Type	Flood Impact Objective Target					AEP	Maximum FIO Exceedance					Length (m)				Exceedance Justification	Proposed Mitigation Measures ²
				Change in Peak Water Level (mm)	Change in Time of Inundation (%)	Change in Velocity (%)	Change in Velocity (m/s)	Change in Hazard (%)		Change in Peak Water Level (mm)	Change in Time of Inundation (%) ³	Change in Velocity (%) ³	Change in Velocity (m/s)	Change in Hazard (%) ³	Afflux	Time on Inundation	Velocity	Hazard		
7	1601510399000489579	Track 0	Unsealed	10	10	10	0.5	10	20%	-	-	-	-	-	-	-	-	-	This track is situated inside the Project corridor and will be decommissioned as part of the Project. The Project alignment will be built over this track and therefore, exceedances have not been mitigated and the operational capacity of the road will be comprised irrespective of flooding.	Consultation with the appropriate Road Manager will be undertaken with respect to this Local Road to discuss any mitigation measures due to the overall impact on the Track.
									10%	72	-	-	-	-	15.00	-	-	-		
									5%	179	-	-	0.03	-	17.50	-	2.50	-		
									2%	213	2708	25	0.52	213	5.00	12.50	5.00	7.50		
									1%	358	419	12	0.58	177	12.50	17.50	5.00	15.00		
									0.05%	-	-	-	-	-	-	-	-	-		
									0.01%	-	-	-	-	-	-	-	-	-		
20	1601510399000489585	Track 0	Unsealed	10	10	10	0.5	10	20%	-	-	-	-	-	-	-	-	-	This track is situated inside the Project corridor and will be decommissioned as part of the Project. The Project alignment will be built over this track and therefore, exceedances have not been mitigated and the operational capacity of the road will be comprised irrespective of flooding.	Consultation with the appropriate Road Manager will be undertaken with respect to this Local Road to discuss any mitigation measures due to the overall impact on the Track.
									10%	-	-	-	-	-	-	-	-	-		
									5%	-	-	-	-	-	-	-	-	-		
									2%	-	-	-	-	-	-	-	-	-		
									1%	-	30	-	-	-	-	2.50	-	-		
									0.05%	-	-	-	-	-	-	-	-	-		
									0.01%	-	-	-	-	-	-	-	-	-		

Table Notes

1. Time of Submergence has been measured across the full length of the road segment and does not correspond to the exact location of exceedance within that segment

2. Change in water level (afflux) is calculated based on the maximum water surface level difference between the existing and design case results, and does not consider the underlying topography. Depth values presented are based on the respective underlying topographies and in some cases do not align with afflux calculations. This can be attributed to interpolation calculations of results performed within the hydraulic model.

3. Percentage increases should be read in conjunction with existing and design case results. The FIOs require that change be presented in percentage change for time of inundation, velocity and hazard and can sometimes produce large increases when existing case results are significantly small. Some locations were identified to experience significant percentage increases however the "absolute" difference between Existing and Design results are relatively small.

4. The Flood Impact Objective exceedance has been identified due to a change in flood extent and a cell which was dry in the existing case and now flood-affected in the developed case.

5. The above table should be read in conjunction with change maps provided as part of Volume II and the Digital Platform.

6. The symbol "-" denotes that no change above Flood Impact Objectives has been determined and therefore no result has been presented.

7. Changes recorded above may only occur during a small fraction of model simulation time.

8. Three decimal places do not infer absolute accuracy in modelling results, but it is necessary to identify small changes.

9. Design optimisations that could be implemented (but not limited to) include adjustment of culvert spacing, realignment of culverts to existing flowpaths when more detailed survey is available, modifying the grade of culverts, road height and alignment adjustments, removal of bridge abutments from waterway extents etc

Depth (m) ²		Time of Inundation in hours (related to % exceedance)		Time of Submergence (hours) ²		Velocity in m/s (related to % exceedance)		Velocity in m/s (related to actual change exceedance)		Hazard in m ² /s (related to % exceedance)	
Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case
-	-	-	-	-	-	-	-	-	-	-	-
-	0.072	-	-	-	-	-	-	-	-	-	-
-	0.179	-	-	-	-	-	-	0.485	0.512	-	-
-	0.213	0.06	1.51	14.52	2.30	0.581	0.724	-	0.525	0.06	0.16
-	0.358	0.29	1.52	11.53	2.30	0.548	0.614	-	0.578	0.08	0.21
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
-	-	0.74	0.89	11.04	11.05	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-

12.5.3.4 Flood impacts on existing rail infrastructure

No third-party rail infrastructure is located within the Cattle Creek hydraulic model boundary.

12.5.3.5 Flood impacts on private land outside the permanent footprint

Developed Case flooding from the Cattle Creek catchment, and comparison against the FIOs, has demonstrated that no adverse impacts on private land outside of the permanent footprint is expected due to the presence of the Project for all modelled events up to and including the 1% AEP event.

12.5.3.6 Flow distribution

A key landowner concern is the potential change in flow distribution. To determine the change in flow distribution maximum velocity-depth modelling results were used to produce change in unit flow maps between Existing and Developed Case conditions. Unit flow is the measure of discharge (velocity-depth product) and is often used for assessing peak flow along a cross-section, or to determine the effective flow corridors within a floodplain.

The presented flow distribution maps were filtered with a velocity-depth product cut-off of $0.15 \text{ m}^2/\text{s}$. The cut-off is in line with FIO requirements and was used to assist in identifying main channels (high flow zones) from shallow lower flood zones (i.e. outer extents of the flood surface).

The following points summarise the potential change in flow distribution for the Cattle Creek catchment and should be considered in conjunction with flow distribution figures supplied in Figure Q8-1 to Figure Q8-8 of Volume 2 – Appendix Q:

- For all events up to the 1% AEP event, the hydraulic results demonstrate minimal impact to the existing flow patterns outside of the permanent footprint
- For the 1 in 2,000 AEP event, a minor reduction in unit flow is noted on the downstream side of the Project alignment
- For the 1 in 10,000 AEP and PMF flood events, a unit flow increase is noted upstream of the Project alignment, and downstream of all cross-drainage structures (Culvert 87.18 and C87.37). The increase in flow is a result of flood waters running parallel to the Project alignment due to limited overtopping/weir overflow.
- Ongoing mitigation measures to reduce changes in flow distribution are likely to occur during future design stages of this Project and may include further refinement of the bridge abutments and additional survey where necessary.

12.5.4 Extreme events

12.5.4.1 Flood immunity for extreme events

The risk of overtopping of the Project alignment was assessed for a range of extreme events (1 in 2,000 AEP, 1 in 10,000 AEP and PMF events) with Table 12.19 presenting the depth of water above the formation level and over the top of rail at each structure. It is noted that the function of the floodplain culverts is to balance flood levels on the upstream and downstream sides of the Project alignment.

As such, overtopping of the rail is not predicted to result in significant excessive flows or velocities as would occur in a dam embankment overtopping scenario. Further analysis of the overtopping risk to FSRs can be found in Section 21.

Table 12.19 Cattle Creek—Extreme events: Estimated depth of water above formation and top of rail levels

Chainage (km)	Structure ID	Depth of water above formation level (m)			Depth of water over top of rail (m)		
		1 in 2,000 AEP	1 in 10,000 AEP	PMF	1 in 2,000 AEP	1 in 10,000 AEP	PMF
87.18	C87.18	-	-	1.78			
87.37	C87.37	-	0.18	1.09	-	-	0.39
88.2	310-BR06	-	-	-	-	-	-

12.5.4.2 Impacts during extreme events

Flood immunity of FSRs is discussed in Section 12.5.3.1, and maps demonstrating the impacts during extreme events are shown on the [Online Digital Portal](#). In accordance with the Project FIOs, FSRs are considered impacted by extreme flood events if the change in water level is greater than 250 mm in the 1 in 2,000 AEP event, pending the receptor classification (i.e. at existing habitable dwellings and/or commercial and industrial buildings/premises).

As indicated above, based on the available aerial imagery, no buildings or critical infrastructure are located within the hydraulic model domain of Cattle Creek floodplain and therefore no assessment has been completed.

12.5.5 Sensitivity analysis—Cattle Creek

The sensitivity of the model to various parameters was assessed using the following two scenarios:

- An increase in rainfall intensity (i.e. to reflect climate change scenario)
- Increase in blockage of culverts from 25 to 0 per cent and 50 per cent.

12.5.5.1 Climate change

The potential impacts of climate change in the Cattle Creek floodplain were assessed for the 1% AEP design event to determine the sensitivity of the Project to the potential long-term changes in climate. The assessment was undertaken in accordance with ARR 2019.

The Representative Concentration Pathways 8.5 (2090 horizon) climate change scenario has been adopted for the Project with an associated increase in rainfall intensity of 23.9 per cent across the catchment area.

Due to no FSRs being present with the hydraulic model domain, Table 12.20 presents the structures performance with Representative Concentration Pathways 8.5 climate change conditions. For the 1% AEP event, the change in peak water levels for the Representative Concentration Pathways 8.5 climate change scenario is presented in Figure H8-C in Volume 2 – Appendix H.

Climate change results are expected to increase peak water levels upstream of the Project alignment by up to 0.2 m at structure locations for the 1% AEP event. The Project alignment is expected to retain 1% AEP flood immunity with 3.1 m freeboard to formation level under the climate change scenario.

Table 12.20 Cattle Creek—1% AEP event with Representative Concentration Pathways 8.5 conditions: Structure performance

Structure ID	Structure type	1% AEP peak water level (mAHD)	1% AEP +CC peak water level (mAHD)	Difference in peak water level (m)	Freeboard to rail formation with CC (m)
310-BR06	Bridge	323.9	324.1	+0.2	5.3
C87.37	RCP	321.9	322.0	+0.1	3.4
C87.18	RCBC	323.6	323.8	+0.2	3.1

12.5.5.2 Blockage

Blockage was assessed in accordance with ARR 2019. The blockage assessment was undertaken, and resulted in a blockage factor of 25 per cent adopted for culverts. A minimum culvert size of 900 mm diameter was adopted to reduce potential for blockage and maintenance. A significant community concern is the potential impacts to flood conditions should the proposed culverts become blocked with debris. The primary concern is the blockage of culverts, which is likely to drive flood levels higher, particularly upstream of the culverts, and divert more flow through residences, across access roads and other infrastructure. A sensitivity analysis was undertaken with 0 and 50 per cent blockage on culverts.

Results of the blockage sensitivity analysis are shown in Figure H8-A and H8-B in Volume 2 – Appendix H, respectively. There is little difference between results with zero blockage and baseline blockage, but peak blockage increases with 50 per cent blockage.

Due to no FSRs being present with the hydraulic model domain, a sensitivity analysis was only completed on structures performance. Table 12.21 provides a summary of 1% AEP peak flood levels at cross-drainage structures for the blockage scenarios.

Table 12.21 Cattle Creek—1% AEP event: Culvert blockage assessment

Structure ID	Structure type	1% AEP Peak water levels (mAHD)			Increase from Developed Case to 50% blockage scenario (mm)
		0% blockage	Developed Case (25% blockage)	50% blockage	
C87.37	RCP	321.8	321.9	322.1	+23
C87.19	RCBC	323.6	323.6	323.7	+4

During detailed design, the blockage factors will be reviewed in line with the final design and local catchment conditions. This may result in a varied and/or lower blockage factors being applied along the Project alignment. Should impacts be determined due to a change in blockage factor, additional mitigation measures will be considered in detailed design.

12.6 Additional modelling items to be considered in detailed design

The following points summarise additional items to be included in the flood models for detailed design as per agreement with the Expert Flood Panel:

- Additional 'as-built' information will be sought from DTMR, where available, to accurately represent existing infrastructure within the modelling domain including DTMR Bridge – TMR 24519. Existing and design bridges will be modelled using Ifcsh.
- Review and, if necessary, update the Cattle Creek URBS model to ensure higher design flow estimates, and consequently design velocities, are considered
- A review of the latest TUFLOW version will be undertaken at the commencement of detailed design to determine whether updates of the TUFLOW version from 2020-10-AD-iSP-w64 are beneficial to modelling
- Undertake additional model strengthening by improving the representation of cross-drainage infrastructure, survey data, and bridge structures within the hydraulic model
- Review and update the locations of the inflows to the TUFLOW model
- In order to meet FIOs, additional analysis of cross-drainage infrastructure may be required, along with potential stakeholder consultation.

Further details of mitigation measures can be found in Section 22. A review of proposed mitigation measures will be completed during the detailed design stage of the Project after hydraulic models have been updated to capture the above changes.

12.7 Planned and future roads

Consultation with the DTMR has not identified any planned or future road upgrades within the Cattle Creek catchment. ARTC will continue to consult and engage with DTMR to determine any future or planned road projects that may need to be considered as part of the detailed design hydraulic modelling.

Consultation with GRC has not identified any planned or future road upgrades within the Cattle Creek catchment. ARTC will continue to consult and engage with GRC to determine any future or planned road projects that may need to be considered as part of the detailed design hydraulic modelling.

13 Pariagara Creek

Portions of Pariagara Creek and smaller tributaries are well defined channels, conveying runoff from the adjacent hills; however, closer to the Project alignment, the terrain flattens out and, consequently, more overland flow occurs. This is particularly prevalent between the Project alignment and Millmerran Inglewood Road, where the topography is relatively flat and less vegetated.

Under the Existing Case 1% AEP event, the flood depth in the Pariagara Creek channel is up to approximately 6 m. The Existing Case 1% AEP floodplain inundated extent is approximately 2.7 km wide where the Project alignment crosses.

The location of the Project alignment in relation to Pariagara Creek is shown in Figure I1-A in Volume 2 – Appendix I.

13.1 Data collection and review—Pariagara Creek

Available background information including existing hydrologic and hydraulic models, survey, streamflow data, available calibration information and anecdotal flood data was gathered. Data was sourced from a wide range of stakeholders including:

- GRC—existing flood studies
- BoM—rainfall gauging data.

13.1.1 Previous studies

A number of previous hydrology and hydraulic studies were sourced as part of this assessment. A review of each study was undertaken to determine suitability for use on the Project as documented in the following sections.

13.1.1.1 Inglewood Flood Study, Engeny, 2015

Engeny was commissioned by GRC to undertake a flood study of Inglewood. The study objectives were ‘*to define the nature, extent and risks of flooding in Inglewood in order to inform disaster management planning and response, as well as control future development*’ (Engeny, 2015). An URBS hydrologic model and TUFLOW (1D/2D) hydraulic model were developed as part of this study.

13.1.2 Survey

ARTC provided LiDAR data from 2015 as 1 m grid DEM tiles. Using GIS software, a DEM was generated with a 1 m grid resolution for use in the Project based on the 2015 dataset. This was used for modelling within the permanent footprint and up to the full extent of the 2015 LiDAR where relevant.

In areas that were not covered by the LiDAR provided by ARTC, LiDAR tiles were sourced from Geoscience Australia. The DEM datasets utilised for modelling were based on surveys flown between 2009 and 2015. SRTM data was used for catchment delineation where no LiDAR data could be sourced, to inform the hydrologic modelling.

The survey data sources and DEM developed for Pariagara Creek are shown in Figure I1-B in Volume 2 – Appendix I.

13.1.3 Aerial imagery

Aerial imagery of the Pariagara Creek Study area was provided by ARTC and was used to identify and confirm topographic and vegetative characteristics of the study area. Aerial imagery captured in 2015 was made available. Additional imagery outside the study area was sourced from QGIS imagery in an open-source format.

13.1.4 Existing drainage structure data

DTMR as-built drawings were sought for culvert and bridge details. No information for Pariagara Creek study area was readily available during model development. This information will be refined as the local survey is progressively completed.

13.1.5 Stream gauge data

No streamflow gauges exist within the Pariagara Creek catchment study area.

13.1.6 Rainfall data

A number of daily and sub-daily rainfall stations are located in and around the Pariagara Creek catchment; however, since there are no streamflow gauges to use for model calibration, no historical rainfall data was sought.

13.1.7 Anecdotal and observed flood data

No anecdotal or observed flood data was available for this area of Pariagara Creek.

13.1.8 Site inspection

A site inspection was undertaken during February 2018. During the site inspection, all major waterway crossings were visited and inspected with photographs taken and details recorded of the crossing, existing drainage structures and surrounding catchment. An assessment of the relative roughness and blockage potential was undertaken during the site inspection.

13.2 Hydrologic model development—Pariagara Creek

13.2.1 Model setup

A hydrologic model of Pariagara Creek was established in URBS using the latest procedures detailed in ARR 2019 and the available rainfall data from BoM. The 2019 ARR procedures cover revisions made to hydrologic parameters such as losses, pre-burst depths, TPs and ARF.

The hydrologic model covers approximately 245 km² of the Pariagara Creek catchment upstream of the Project alignment. The catchment was delineated into 28 sub-catchments to capture the variability of rainfall and to better represent the network of creeks and streams within the catchment.

The hydrologic model setup including extent and sub-catchment map is presented in Figure I1-C in Volume 2 – Appendix I.

Model inputs are summarised in Table 13.1.

Table 13.1 Pariagara Creek—Summary of URBS model inputs

Input parameter	Remarks
URBS model type	Basic
Routing variables	Catchment area, stream lengths
Channel lag parameter, α	1.20
Catchment non-linearity parameter, m	0.8

Note that the default values were adopted for all other URBS parameters.

13.2.2 Sub-catchments

The Pariagara catchment is comprised of approximately 28 sub-catchments with an approximate total catchment area of 245 km². Sub-catchment details adopted for the Pariagara hydrologic model are outlined in Table 13.2.

Table 13.2 Pariagara Creek sub-catchment data

Sub-catchment name	Area (km ²)	Longitude	Latitude
1	7.83	151.002	-28.232
2	14.85	151.044	-28.215
3	11.55	151.023	-28.249
4	9.12	151.010	-28.273
5	7.38	151.040	-28.273
6	20.25	151.078	-28.231
7	7.30	151.063	-28.254
8	10.01	151.033	-28.296
9	3.67	151.055	-28.300
10	9.35	151.071	-28.279
11	6.69	151.092	-28.273
12	7.52	151.084	-28.303
13	8.90	151.097	-28.323
14	5.05	151.108	-28.258
15	10.55	151.133	-28.265
16	23.17	151.127	-28.298
17	10.25	151.133	-28.334
18	10.31	151.107	-28.349
19	4.60	151.111	-28.367
20	6.80	150.999	-28.319
21	7.63	151.025	-28.322
22	4.19	151.035	-28.340
23	4.81	151.054	-28.349
24	13.46	151.061	-28.327
25	4.87	151.075	-28.355
26	4.00	151.083	-28.369
27	4.80	151.036	-28.366
28	6.37	151.063	-28.373

The hydrologic model extent and sub-catchment map is presented in Figure I1-C in Volume 2 – Appendix I.

13.2.3 Design event parameters

Hydrologic information to assist estimation of design event flows was sourced from the ARR 2019 Data Hub as summarised in Table 13.3. Details of the adopted hydrologic model parameters are presented below.

Table 13.3 Pariagara Creek—Summary of URBS model design event inputs

Input parameter	Remarks
Design rainfall	Intensity-Frequency-Durations for each sub-catchment were downloaded from the BoM's website to account for variation in rainfall across the catchment.
Extreme event rainfall	Probable Maximum Precipitation depths for durations up to 6 hours (for use in modelling the PMF event) were obtained using the method presented in the Bulletin 53 (BoM, 2003). The rainfall depths for the 1 in 10,000 AEP event were estimated using the interpolation method presented in ARR 2019 Book 8 Section 3.5.
Losses	The losses were adopted from the <i>Inglewood Flood Study</i> (Engeny, 2015) URBS model. Losses do not vary with AEP. Initial loss – 15 mm Continuing loss – 1 mm/hr
Areal reduction factor	Parameters were adopted for the semi-arid inland Queensland region. The catchment area U/S of the proposed rail crossing on Pariagara Creek is approximately 245 km ² , which yields an ARF between 64.5% and 93.3% depending on design storm event AEP and duration.
Ensemble TPs	Central Slopes regions. However, as the study catchment area exceeds 75 km ² , the standard ensemble rainfall patterns from ARR 2019 do not apply to this catchment for storm durations longer than 12h. These were replaced with the areal TPs for the Central Slopes region.
Pre-burst depths	Median preburst depths were downloaded from the ARR 2019 Data Hub for each sub-catchment. Preburst depths vary by design storm event AEP and duration. Preburst depths were applied to the model by reducing the initial losses for each storm event.

13.2.3.1 Spatial distribution

The ARR 2019 design event methodology was adopted for this assessment. ARR 2019 recommends adopting the spatially varying method for catchments larger than 20 km². The Pariagara Creek catchment is approximately 245 km²; therefore, each sub-catchment was assigned an IFD derived from the BoM website at the sub-catchment centroid.

13.2.3.2 Rainfall depth estimation

Design rainfall depth for the 20% to the 1 in 2,000 AEP were manually obtained from BoM website for each sub-catchment. As an example, the IFD representing the catchment centroid of Pariagara Creek hydrologic model located within sub-catchment #10 is provided in Table 13.4.

Table 13.4 Rainfall depth (mm) used for the Pariagara Creek catchment centroid

Duration (hr)	AEP						
	50%	20%	10%	5%	2%	1%	1 in 2,000
0.25	17.8	24.6	29.2	33.8	39.8	44.5	70.9
0.50	24.2	33.5	39.9	46.2	54.6	61.0	97.4
1.00	30.6	42.4	50.5	58.5	69.1	77.3	123.0
2.00	37.2	51.0	60.6	70.2	83.1	93.0	148.0
3.00	41.2	56.2	66.7	77.2	91.4	102.0	163.0
4.50	45.7	61.9	73.3	84.7	100.0	113.0	178.0
6.00	49.1	66.3	78.3	90.5	107.0	121.0	191.0

Duration (hr)	AEP						
	50%	20%	10%	5%	2%	1%	1 in 2,000
12.00	58.8	78.6	92.8	107.0	128.0	144.0	228.0
24.00	70.7	94.3	112.0	129.0	155.0	176.0	280.0
48.00	84.1	113.0	134.0	157.0	188.0	214.0	365.0
72.00	92.0	124.0	148.0	173.0	209.0	237.0	387.0

13.2.3.3 Areal reduction factor

The ARF was estimated using the coefficients for semi-arid inland Queensland, presented in Table 13.5, and was applied to all IFD areas. The URBS software directly downloaded these coefficients from ARR 2019 Data Hub to estimate the ARF using Equation 2.4.1 of ARR 2019 Book 2. The estimated ARF for Pariagara Creek catchment vary between 64.5 to 93.3 per cent depending on design storm event AEP and duration.

Table 13.5 Pariagara Creek—Areal reduction factor coefficients from ARR 2019 Data Hub

a	b	c	d	e	f	g	h	i
0.159	0.283	0.25	0.308	7.30E-07	1.0	0.039	0.0	0.0

13.2.3.4 Temporal patterns

Temporal patterns for 20% AEP to 1 in 2,000 AEP events were adopted based on the 10-burst ensemble approach presented in the ARR 2019. The TPs and pre-burst data were directly downloaded by URBS software from the ARR 2019 Data Hub at the centroid of Pariagara Creek catchment.

Temporal patterns for the 1 in 10,000 AEP and PMF design events for duration up to 6 hours were obtained from *The Estimation of Probable Maximum Precipitation in Australia: Generalised Short-Duration Method* (BoM, 2003).

The adopted TPs for all modelled events are shown in Table 13.6.

Table 13.6 Pariagara Creek—Adopted temporal patterns

Design Event	Critical storm (hrs) duration and temporal pattern
20% AEP	Pattern 2
10% AEP	Pattern 5
5% AEP	Pattern 4
2% AEP	Pattern 1
1% AEP	Pattern 4
1 in 2,000 AEP	Pattern 9
1 in 10,000 AEP	PMP Pattern
PMF	PMP Pattern

13.2.3.5 Pre-burst

Median pre-burst depths for standard durations from 0.5 hour to 72 hours for a 1% AEP design event are between 1.1 mm and 13.1 mm (ARR 2019 Data Hub). Due to the small depth of pre-burst in this area, the impact of pre-burst condition is considered to be minimal in this catchment. The adopted pre-burst depths are summarised in Table 13.7.

Table 13.7 Pariagara Creek—Adopted median pre-burst depths

Duration	Pre-burst depth (mm)					
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP
0.5	0.6	0.9	1.1	1.2	1.5	1.8
0.75	0.6	0.9	1.1	1.2	1.5	1.8
1.0	0.6	0.9	1.1	1.2	1.5	1.8
1.5	0.4	0.4	0.5	0.5	1.5	2.2
2	0.2	0.3	0.3	0.4	1.7	2.6
3.0	0.1	0	0	0	2.9	5.1
6.0	0.3	0.7	1	1.2	1.2	1.1
12.0	0	0.1	0.2	0.3	5.7	9.7
18.0	0	0.1	0.2	0.3	5.7	9.7
24.0	0	0.3	0.5	0.7	7	11.8
48.0	0	0	0	0	0.1	0.2
72.0	0	0	0	0	0	0

13.2.3.6 Design rainfall losses

The adopted constant initial and continuing losses in the URBS model are 15 mm and 1 mm/hr respectively. The losses were adopted from the *Inglewood Flood Study* (Engeny, 2015) URBS model and validated by comparing the modelled flows against QRT and RFFE estimates. The URBS model validation is detailed in Section 13.2.5.

13.2.3.7 Extreme event rainfall

Probable Maximum Precipitation depths for durations up to 6 hours (for use in modelling the PMF event) were obtained using the methodology presented in *The Estimation of Probable Maximum Precipitation in Australia: Generalised Short-Duration Method* (BoM, 2003). Once the PMP was estimated, rainfall for events between 1 in 2,000 AEP and the PMF was then interpolated. For the 1 in 10,000 AEP event, ARF were applied based on interpolation methodology presented in ARR 2019 Section 3.5 in Book 8 and the rainfall depths were estimated accordingly. The parameters adopted for PMP rainfall estimation are presented in Table 13.8. The adopted extreme rainfall depths are provided in Table 13.9.

Table 13.8 Pariagara Creek—Input data for Probable Maximum Precipitation estimation

Factor	Value
EAF	1
MAF	0.8
Portion of rough catchment	100%
Catchment area (km ²)	251.5 ¹
Location	(-28.293, 151.083)

Table note:

- 1 The total catchment area was included for the PMP calculation, this includes a small proportion of area downstream of the Project alignment

Table 13.9 Pariagara Creek—Adopted extreme event rainfall depths (mm)

Duration (hrs)	1 in 10,000 AEP rainfall depth (mm)	PMP rainfall depth (mm)
0.25	87	126
0.5	121	185
0.75	141	236
1	154	283
1.5	173	368
2	186	425
2.5	197	478
3	207	519
4	222	591
5	233	639
6	243	681

13.2.4 Model calibration

Insufficient data was available to calibrate historical flood events in the Pariagara Creek catchment.

13.2.5 Hydrologic model validation

The hydrologic model was validated against the DTMR QRT and RFFE method estimates. The routing parameter α was adjusted until there was a reasonable match between the URBS model flows and those derived using QRT (Section 13.2.5.1) and RFFE (Section 13.2.5.2) methods at the proposed Pariagara Creek crossing.

13.2.5.1 Quantile regression technique

This approach provides a procedure for calculating design floods for ungauged catchments that relates the design flood discharges to readily available catchment characteristics such as catchment area and 2% AEP 72-hour rainfall intensity. This method is commonly used for ungauged catchments in Queensland. The estimation of peak using QRT for Pariagara Creek catchment is based on the total catchment area and 2% AEP 72-hour rainfall intensity of 245 km² and 2.89 mm/hr, respectively.

The estimated peak flows at the Pariagara Creek catchment outlet using QRT are provided in Table 13.10.

Table 13.10 Estimate of flows at the Pariagara Creek catchment outlet using QRT

AEP	Estimated peak flow (m ³ /s)
20%	130
10%	210
5%	323
2%	488
1%	631

13.2.5.2 Regional Flood Frequency Estimation

The RFFE online tool was used to obtain an estimation of catchment peak flow at the outlet of Pariagara Creek catchment (i.e. sub-catchment #28). The RFFE inputs and outputs are summarised in Table 13.11 and Table 13.12, respectively.

Table 13.11 Pariagara Creek catchment—Model inputs for the Regional Flood Frequency Estimation

Model parameter	Input value
Region name	East Coast
Region version	RFFE Model 2016 V1
Latitude at catchment outlet (degree)	-28.39
Longitude at catchment outlet (degree)	151.08
Latitude at catchment centroid (degree)	-28.29
Longitude at catchment centroid (degree)	151.07
Distance of the nearest gauged catchment in the database (km)	34.4
Catchment area (km ²)	245
Design rainfall intensity, 50% AEP and 6 hr duration (mm/hr)	8.19
Design rainfall intensity, 2% AEP and 6 hr duration (mm/hr)	17.86

Table 13.12 Pariagara Creek catchment —Regional Flood Frequency Estimation results obtained from ARR 2019 website

AEP (%)	RFFE – lower bound 90% confidence level (m ³ /s)	RFFE – estimate of flow (m ³ /s)	RFFE – upper bound 90% confidence level (m ³ /s)
50	35	86	213
20	89	213	512
10	136	351	906
5	185	532	1,520
2	255	855	2,850
1	311	1,180	4,400

13.2.5.3 Summary

All URBS model results between the 50% AEP and 1% AEP events reside within the 90 per cent confidence limits of the RFFE and show a close match with QRT. The estimated flood flows at the Project alignment crossing of Pariagara Creek are presented in Figure 13.1 and Table 13.13.

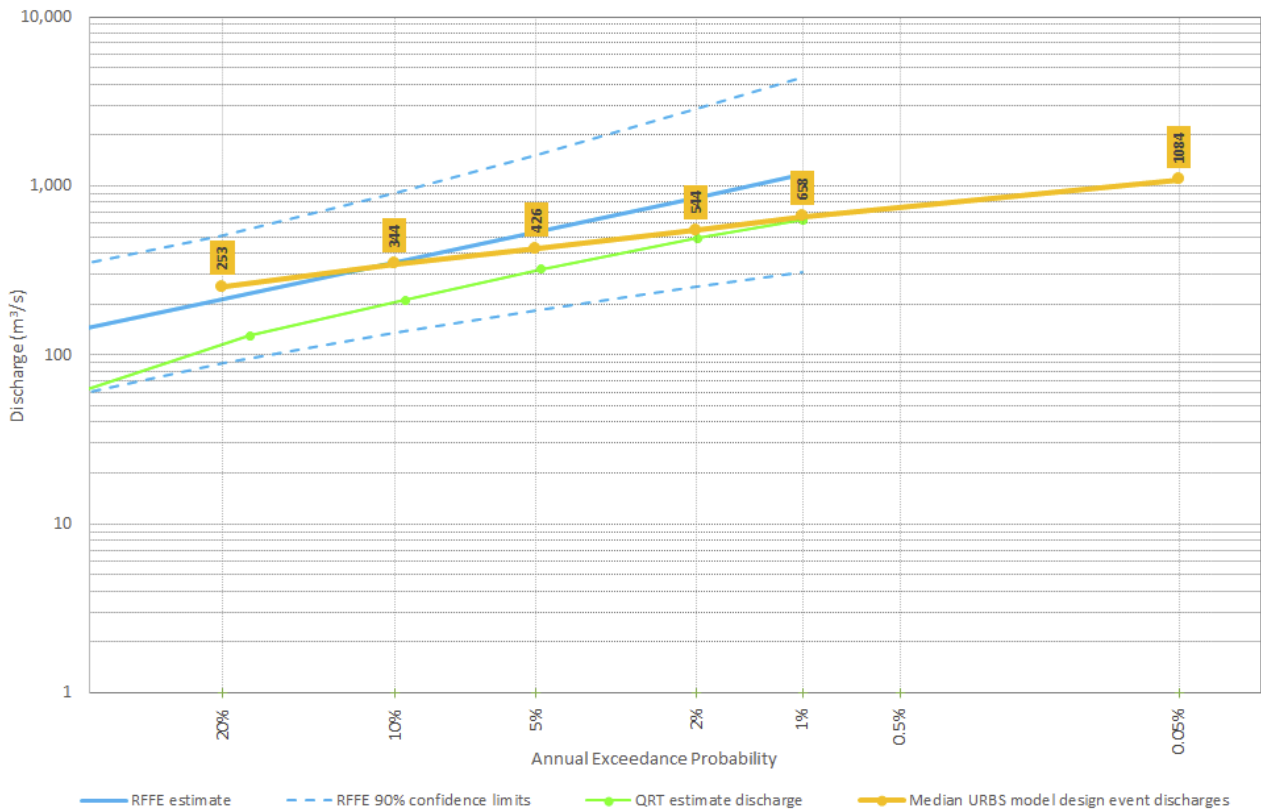


Figure 13.1 Estimate of flows at the Pariagara Creek crossing

Table 13.13 Estimate of flows at the Pariagara Creek crossing

AEP (%)	RFFE – lower bound 90% confidence level (m³/s)	RFFE – estimate of flow (m³/s)	RFFE – upper bound 90% confidence level (m³/s)	DTMR quantile regression technique (m³/s) ¹	URBS model flows (m³/s)
20	89	213	512	130	253
10	136	351	906	210	344
5	185	532	1,520	323	426
2	255	855	2,850	488	544
1	311	1,180	4,400	631	658
1 in 2,000	-	-	-	-	1,084
1 in 10,000	-	-	-	-	2,247
PMF	-	-	-	-	8,055

Table note:

¹ The QRT method estimates the 39.3%, 18.1%, 9.5% and 4.9% AEP instead of 50%, 20% and 10% and 5%, respectively

13.3 Hydraulic model development—Pariagara Creek

A 2D modelling approach was adopted to appropriately simulate flood mechanisms around the Project alignment at Pariagara Creek. The platform used for hydraulic modelling is TUFLOW HPC software package (2020-10-AD) with SGS and QT approach. The processes and assumptions adopted throughout the development of the hydraulic model are described in the following sections.

13.3.1 Model setup

The setup of the hydraulic model is summarised in Table 13.14.

Table 13.14 Pariagara Creek— Hydraulic model summary

Parameter	Information
Completion date	June 2022
AEPs assessed	20%, 10%, 5%, 2%, 1%, 1 in 2,000, 1 in 10,000 AEP and PMF
Hydraulic model build	TUFLOW HPC GPU – version 2020-10-AD-iSP-w64
Model extent	Refer to Figure I1-D in Volume 2 – Appendix I
Grid size	5 m
SGS size	1 m
QT grid size	2 nd Order; 2.5 m
DEM (year flown)	ARTC dataset (2015).
Roughness	Spatially varying roughness values compliant with industry norms.
Eddy viscosity	Wu (default)
Model calibration	N/A
D/S model boundary	Height-Discharge (HQ) Boundary with normal slope approximated based upon topography dataset.
Hydraulic model timestep	Adaptive timestep
Modelled scenarios	Existing Case, Developed Case
Sensitivity analysis	Blockage – 0%, 50% Climate change

The hydraulic model extent and the spatial distribution of land use in the 2D model domain are presented in Figure I1-D and Figure I1-E in Volume 2 – Appendix I.

13.3.2 Model topography and grid size

The topography within the hydraulic model was based on 1 m DEM data collected during 2015, which was supplied by ARTC. A 5 x 5 m grid size was selected to allow the features of the channels and floodplain to be represented with sufficient accuracy while maintaining efficient model run times. The 5 x 5 m grid size is considered adequate to represent floodplain topography and is consistent with the approach typically adopted in similar flood studies.

The 2D model topography was modified adjacent to the culverts so that the existing drainage flows, existing road/rail crowns and levee banks were represented accurately in the model. For the Developed Case, the topography was modified to include the Project rail embankment, drainage structures and associated infrastructure.

13.3.2.1 Sub-grid sampling and Quadtree mesh refinement

The Pariagara Creek hydraulic model was established using TUFLOW's 2020-10-AD model version, including SGS and QT add-on modules. The below points provide a brief description of each module, along with the key parameters adopted for this assessment.

- Traditional grid-mesh based 2D hydraulic models sample the underlying DEM by taking a single elevation at the cell centres and cell faces. Sub-grid sampling allows for increased resolution by sampling a greater number of points of the underlying DEM to establish a water surface elevation versus volume relationship for each grid, along with a water surface elevation versus width relationship (wetted perimeter) at cell sides. The adoption of SGS allows increased representation of storage and conveyance between cells. A SGS distance of 1 m was adopted for this assessment.

- Quadtree mesh is constructed by subdividing the base mesh-grid size into four smaller square cells. Cells can be further sub-divided into four, and so on, allowing for additional resolution in location of interest. The base hydraulic model grid cell size (5 x 5 m) was revised to include additional spatial resolution of approximately 2.5 m, to better represent model velocity at the entry and exit of culverts. Quadtree meshes were applied in the model covering minimum 25 m upstream and 100 m downstream of the design culvert locations, and culverts within the permanent footprint.

13.3.3 Roughness

The hydraulic roughness generally reflects the types of development and ground cover that exists within the hydraulic model extent. The distribution of roughness categories adopted for this assessment was based on the information supplied in the land use layer from Queensland Globe database, aerial imagery and confirmed during the site inspection. Specific roughness values applied to the model are detailed in Table 13.15.

Table 13.15 Pariagara Creek catchment—Adopted roughness values

Category	Manning's 'n' value
River	0.035
Sealed roads	0.015
Unsealed roads	0.025
Rural residential	0.09
Urban residential	0.10
Commercial	0.06
Open space—Minimum vegetation	0.045
Open space—Moderate vegetation	0.055
Open space—Heavy vegetation	0.065
Open water	0.025
Rail	0.025
Riparian	0.04
Crops ¹	0.1085

Table note:

- ¹ A uniform value was adopted for the crop areas, due to the seasonal and spatial variations in vegetation distribution and the uncertainty this brings to the design event approach

13.3.4 Hydraulic structures

No existing hydraulic structures have been included within the extent of the hydraulic model. A further review of existing structures will be completed at the detailed design stage.

13.3.5 Boundary conditions

The following boundary conditions were applied within the hydraulic model:

- Flow versus time model boundaries were applied for all inflow hydrographs with local and total hydrographs sourced from the hydrologic models.
- Delineated sub-catchments in the hydrologic model were used as internal flow boundaries within the hydraulic model extent (SA polygons). The modelled flows in the hydrologic model were applied to the lowest points of each SA polygon.
- Where the Project alignment intersected the sub-catchment, the inflows were proportioned accordingly based on catchments upstream and downstream of the alignment.

- A height-discharge boundary (HQ) was applied for the downstream outflow model boundary. It was confirmed that it was a sufficient distance away from the area of interest so as not to impact on modelling results.

13.4 Existing Case modelling results—Pariagara Creek

13.4.1 Hydrologic model peak flow assessment

A critical duration assessment was undertaken to determine which storm duration(s) produced peak flood flows where the major waterways are intersected by the Project alignment and at the downstream outlet of the hydraulic model. To assess the critical storm duration the following methodology based on the approach described in ARR 2019 Book 2 Chapter 5 was adopted:

- The models were modelled for a range of AEP events:
 - Each AEP was modelled for a range of durations
 - Each duration was modelled for each of the 10 associated TPs.
- A critical duration assessment was undertaken at the locations mentioned above to determine which duration produced the highest median flow of the 10 TPs for each event.

The critical duration and adopted TP for each design event AEP is provided in Table 13.16. Table 13.17 also demonstrates the peak flow for the 1% AEP event.

Table 13.16 Pariagara Creek—Critical flood durations and temporal patterns

Design event	Critical storm (hrs) duration and temporal pattern
20% AEP	9 hours - Pattern 2
10% AEP	12 hours – Pattern 5
5% AEP	12 hours – Pattern 4
2% AEP	12 hours – Pattern 1
1% AEP	12 hours - Pattern 4
1 in 2,000 AEP	9 hours – Pattern 9
1 in 10,000 AEP	6 hours – Pattern 0
PMF	6 hours – Pattern 0

Table 13.17 Pariagara Creek—1% AEP event peak flow at key locations as applied in the hydraulic model

Sub-catchment name	1% AEP event peak flow (m ³ /s)	Critical storm (hrs) duration and temporal pattern
18T	520.1	12 hours - Pattern 4
19L	23.6	12 hours - Pattern 4
25T	153.2	12 hours - Pattern 4
26L	19.7	12 hours - Pattern 4
26T	641.1	12 hours - Pattern 4
28T	43.4	12 hours - Pattern 4

13.4.1.1 Design storm sensitivity assessment

As noted in Table 13.16 a single storm duration was simulated in the hydraulic model for each AEP. The critical storm durations were selected as those that produced the highest peak flows in the hydrologic model. To address the Expert Flood Panel comment regarding the selection of a single design storm, a sensitivity assessment was undertaken in the hydraulic model to assess the effects of different storm durations on the flood level. The 1% AEP event was re-run with storm durations ranging from 360-minutes to 1,440-minutes.

The storm durations that produced the maximum flood levels at various parts of the floodplain are illustrated in Figure 13.2. It was found that the 720-minute storm (12 hours) that was adopted in the draft EIS is critical over most of the floodplain, with the 540-minute storm (9 hours) generating marginally higher flood levels in some headwater areas.

It is further noted that the URBS peak flows for the 540-minute and 720-minute storm events only differ by 7 m³/s (1.1 per cent difference).

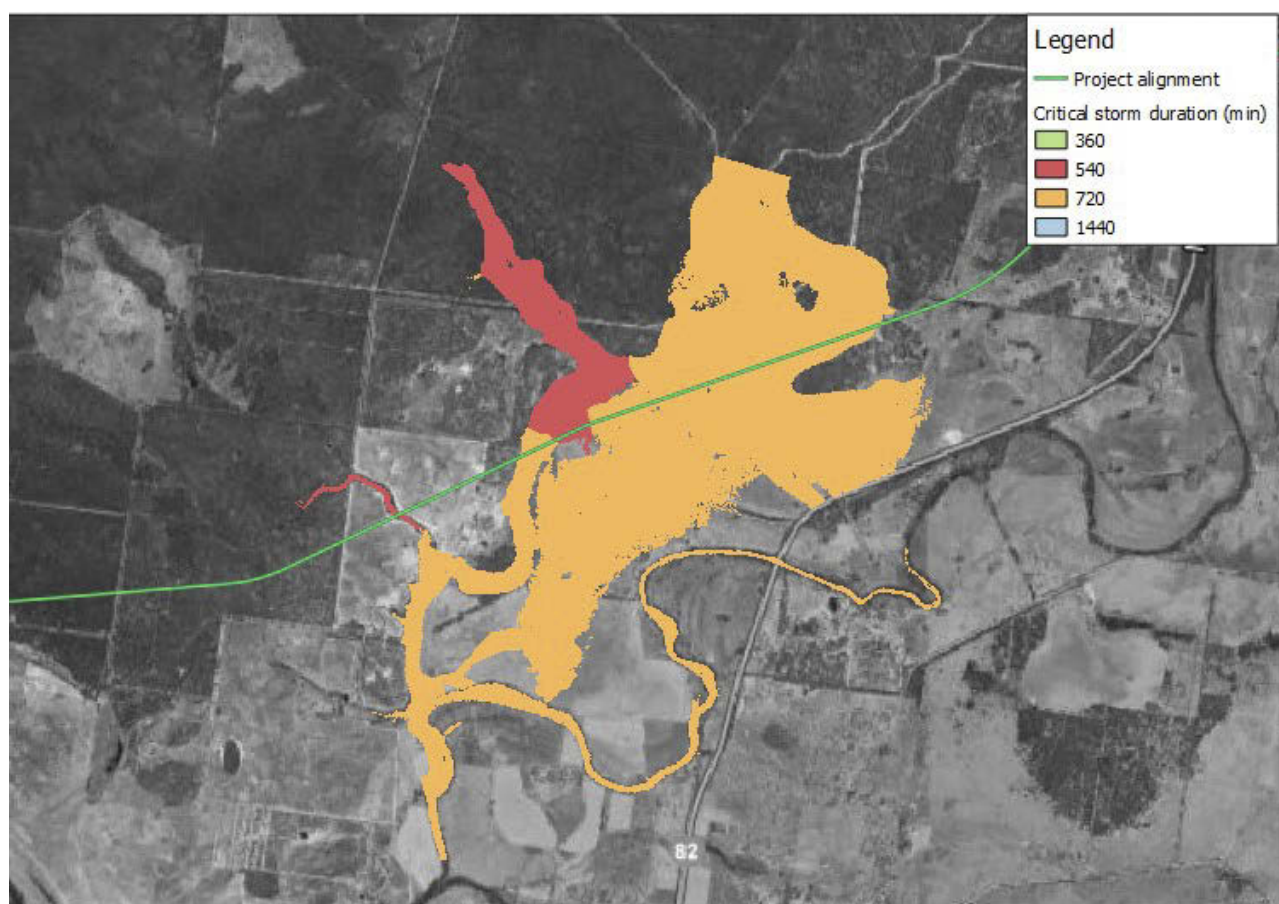


Figure 13.2 Critical storm durations in Pariagara Creek flood model

Differences between the ensemble maxima flood levels and those produced by the adopted single storm duration (720-minute) are shown in Figure 13.3. The comparison reveals negligible flood level change along the Project alignment and on most of the floodplain. Isolated differences up to 20 to 30 mm are observed along headwater tributaries, which impact neither the alignment nor any FSRs.

The adopted single critical storm duration is therefore considered appropriate for this assessment.

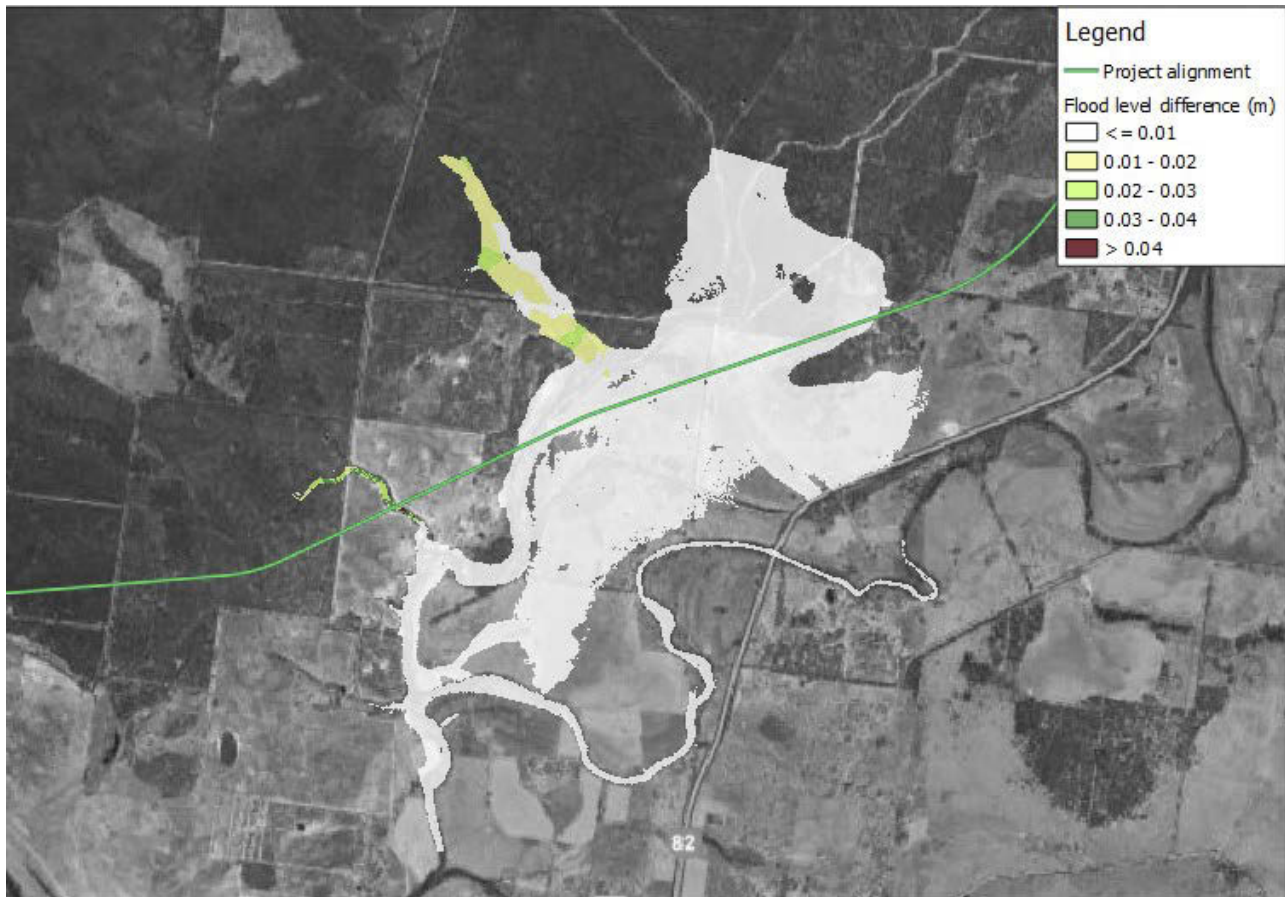


Figure 13.3 Pariagara Creek—Flood level sensitivity to storm duration—1% AEP: Ensemble maximum scenario minus single critical storm scenario

For each storm duration, a single TP was selected as that which produced the rank 6 (one above the median) peak flow at the hydrologic model outlet. Although the critical TP can vary across the floodplain, only a single TP per storm duration was assessed in the hydraulic model.

To assess the flood level sensitivity to TPs, all 10 design TPs were simulated in the hydraulic model for the 1% AEP critical storm duration (720-minute). Figure 13.4 shows that a single TP (temporal pattern four) is critical on most of the floodplain, which is consistent to the hydrologic model prediction.

Figure 13.5 demonstrates that the flood levels generated using a formal ensemble approach are mostly identical to those produced by the adopted critical temporal pattern. Variations up to ± 30 mm are identified in small, localised areas which do not impact the Project rail design.

Lastly, to simulate 10 TPs for each storm duration in the hydraulic model was not considered a practical approach, which would have resulted in an order of magnitude increase in simulation requirements. The selection of a single TP critical duration for the purpose of minimising simulation time is also corroborated in Book 2, Chapter 5.9.2 of ARR 2019.

The adopted single critical TP approach is therefore considered appropriate for this assessment.

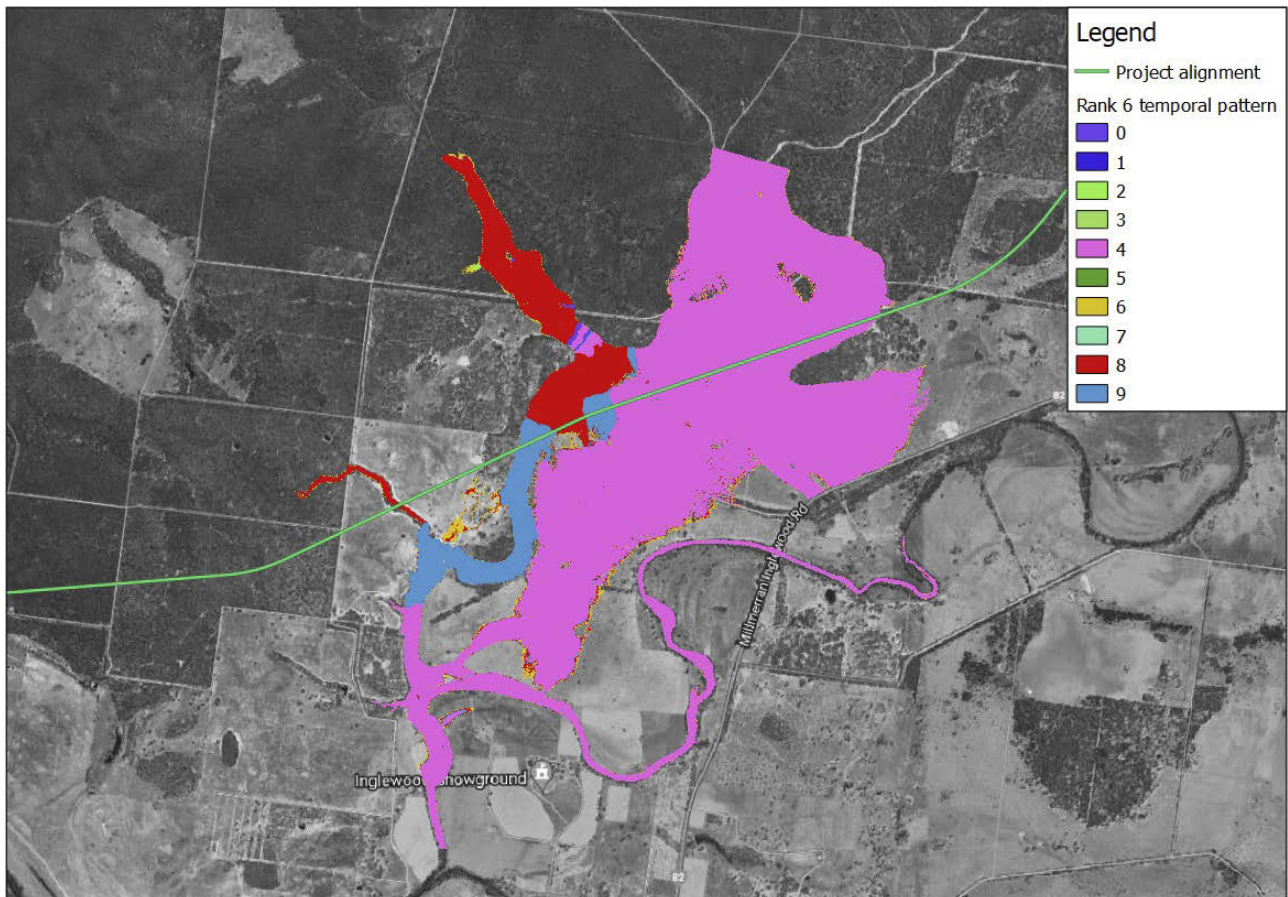


Figure 13.4 Rank 6 temporal patterns on Pariagara Creek floodplain

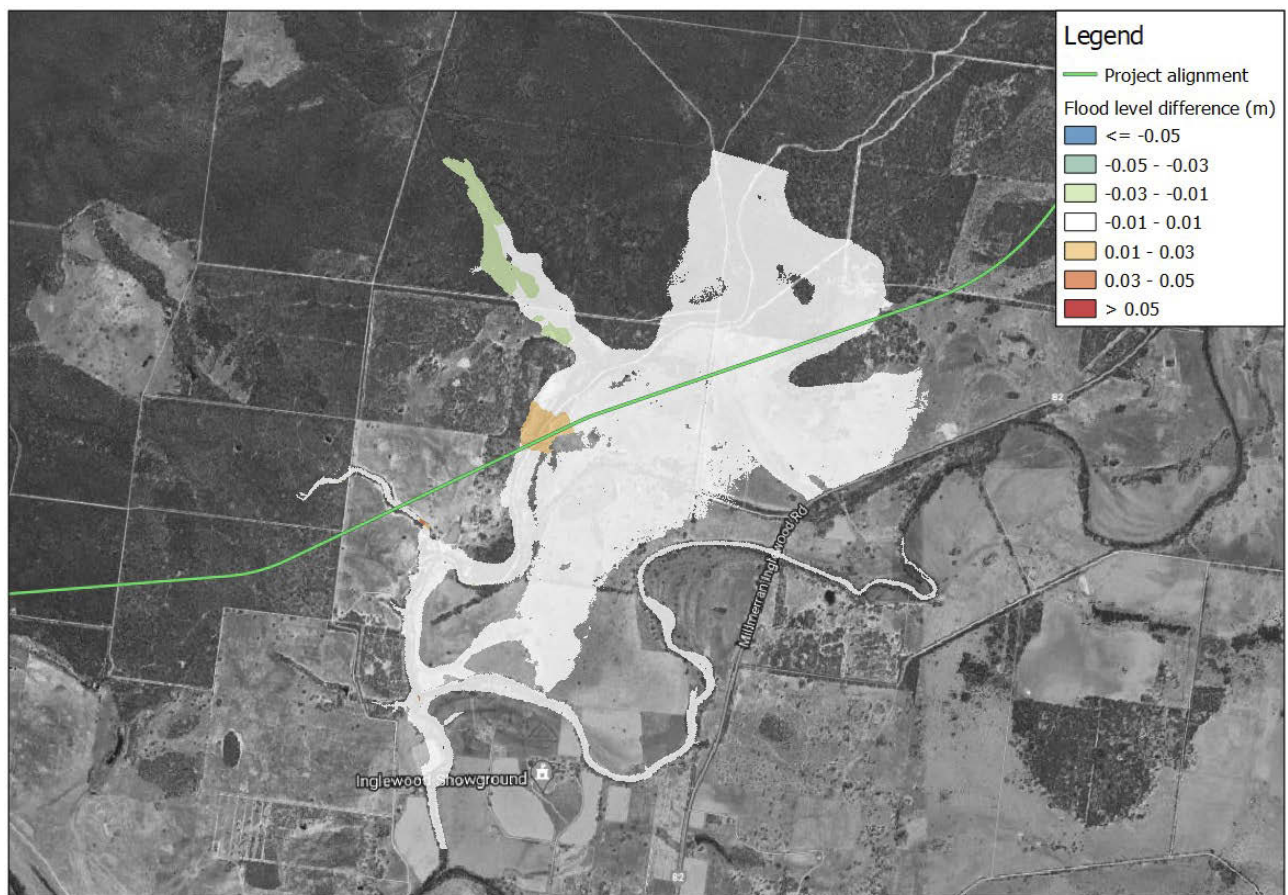


Figure 13.5 Pariagara Creek—Flood level sensitivity to temporal patterns—1% AEP 720-minute: Ensemble rank 6 temporal pattern minus adopted single temporal pattern

The western channel at Ch 66.23 km has a much smaller catchment area upstream of the Project alignment (10 km²) compared to that of the main Pariagara Creek channel (245 km²). Given their apparent mutual independence, the peak flows and critical durations of the western channel have been reassessed in the hydrologic model, with results for the 1% AEP event summarised in Table 13.18. The reassessment indicates approximately 13 per cent flow increase if the critical duration of the western channel was selected based on its local catchment, and a 44 per cent flow increase if the ARF and critical duration were both selected based on the western channel catchment.

The western channel inflow in the hydraulic model was increased based on the local catchment assessment (i.e. Case #2, Table 13.18) and tested for the 1% AEP event to determine its implications on the Project rail design and adjacent area. It is noted that the scenario combines coincident critical flooding from both regional and local catchments and as such has a much rarer exceedance probability than the nominal 1% AEP.

Figure 13.6 presents the sensitivity test results, showing minor increases in afflux and wet extent upstream of the Project alignment in association with the increased western channel inflow. The flood levels outside the Project footprint are up to 300 mm higher than the draft EIS estimates. The impacts are fully contained between the channel high banks and does not propagate to adjacent land or impact the rail immunity.

The present assessment demonstrates that an increase in the western channel inflow does not result in changes to the revised reference design, or associated impact assessment outcomes. Noting the weak hydraulic dependence between Pariagara Creek and the western channel, the western channel inflow will be refined with consideration to its local catchment characteristics in detailed design.

Table 13.18 Reassessment of peak flow and critical duration at Ch 66.23 km

AEP	Case 1. ARF and critical duration based on Pariagara Creek catchment (adopted)		Case 2. ARF and critical duration based on western channel catchment		Case 3. ARF based on Pariagara Creek catchment and critical duration based on western channel catchment	
	Peak flow (m ³ /s)	Critical duration (h)	Peak flow (m ³ /s)	Critical duration (h)	Peak flow (m ³ /s)	Critical duration (h)
1%	42.8	12	61.8 (+44%)	3	48.2 (+13%)	3

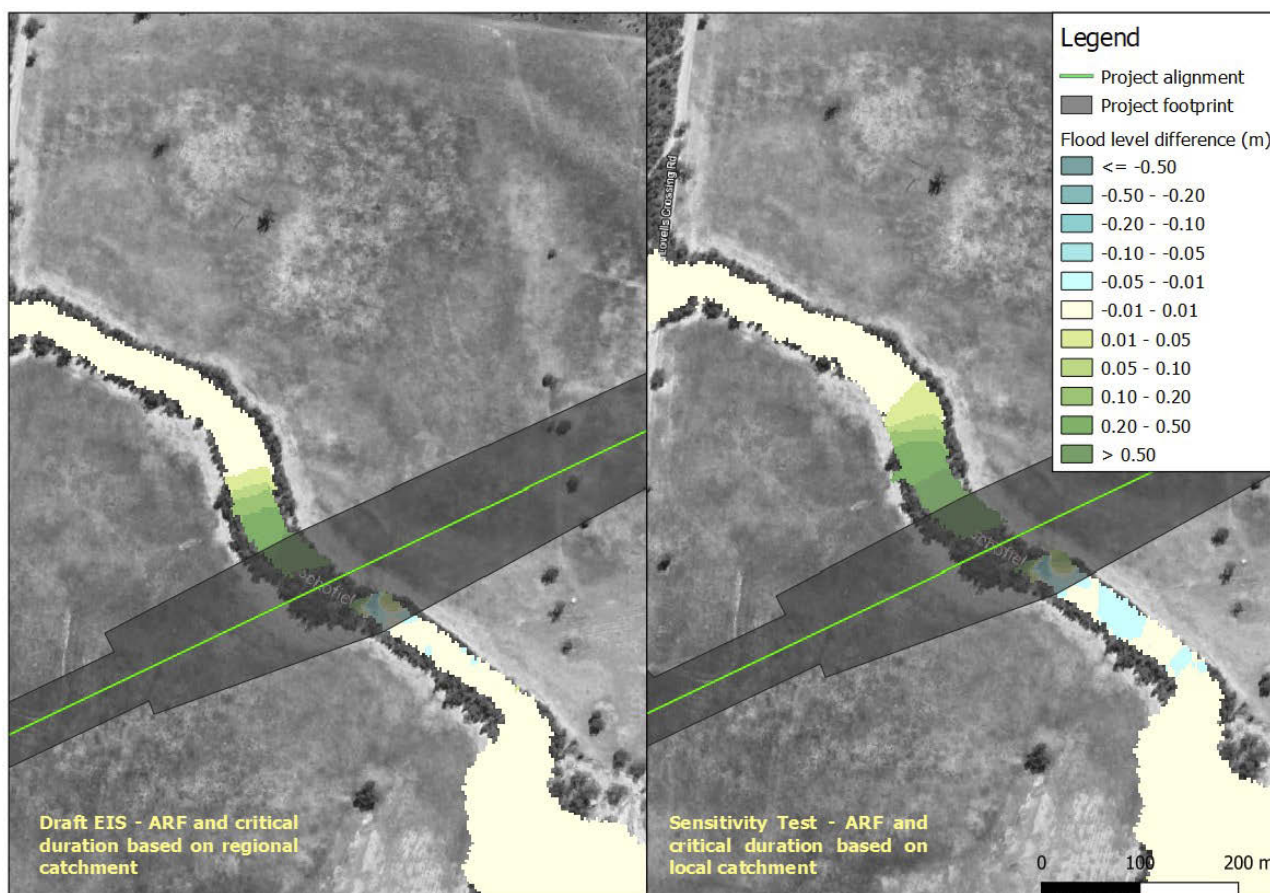


Figure 13.6 1% AEP afflux sensitivity at Ch 66.23 km—Comparison between draft EIS case (left) and sensitivity test case (right)

13.4.2 Existing Case flood maps

Existing conditions flood maps have been presented in Volume 2 – Appendix I and via an [Online Digital Portal](#). Appendix I contains the existing 20% and 1% AEP flood maps, whilst the [Online Digital Portal](#) contains the remainder of the modelled events. The location of each map set is summarised below:

- 20% AEP peak levels: Figure I2-A
- 1% AEP peak levels: Figure I2-B
- 20% AEP peak velocity: Figure I2-C
- 1% AEP peak velocity: Figure I2-D
- 20% AEP peak hazard: Figure I2-E
- 1% AEP peak hazard: Figure I2-F
- 20% AEP peak time of inundation: Figure I2-G
- 1% AEP peak time of inundation: Figure I2-H
- 20% AEP, 10% AEP, 5% AEP, 2% AEP, 1% AEP, 1 in 2,000 year AEP, 1 in 10,000 year AEP and PMF Existing Case results: [Online Digital Portal](#)
- For higher resolution mapping refer to the [Online Digital Portal](#).

13.4.3 Existing Case flood inundation extent and flood levels

The peak 1% AEP flood depth is approximately 6 m within the Pariagara Creek channel. The Project alignment crosses approximately 2.7 km of floodplain adjacent to Pariagara Creek. The peak flood depths on the floodplain are estimated to be an average of 1.9 m, and up to 4.6 m (285.2 mAHD) at the proposed bridge crossing.

Figure I2-B in Volume 2 – Appendix I shows the 1% AEP flood extent and peak water levels within the Pariagara Creek floodplain for the Existing Case.

13.4.4 Flood immunity of existing infrastructure

Table 13.19 presents a summary of overtopping depth for key infrastructure near the Project alignment under a range of design events. Modelling results show that Millmerran-Inglewood Road and Thornton Road have an existing low flood immunity in the areas close to the Project alignment.

Table 13.19 Pariagara Creek—Existing Case: Estimated overtopping depths of key infrastructure

Infrastructure	Overtopping depth (m)							
	PMF	1 in 10,000 AEP	1 in 2,000 AEP	1% AEP	2% AEP	5% AEP	10% AEP	20% AEP
Millmerran-Inglewood Road	1.35-	0.50	0.14	-	-	-	-	-
Thornton Road	6.54	4.94	4.35	4.06	3.95	3.76	3.58	3.36

13.4.5 Existing Case velocities

Peak 1% AEP flood velocities are expected to reach up to 4.9 m/s in localised areas (and confined within the main channel) with the average velocity across the floodplain approximately 0.5 m/s, as shown in Figure I2-D in Volume 2 – Appendix I.

13.4.6 Existing Case flood hazard

For the 1% AEP event a velocity x depth product of less than nine m²/s is achieved within proximity to the bridge crossing site and generally confined to the main channel. Excluding the channelised high flow zones, sheet flow in the floodplains have an average velocity x depth produce of less than 0.5 m²/s as shown in Figure I2-F in Volume 2 – Appendix I.

13.4.7 Existing Case flood duration of inundation

Results for the 1% AEP event highlights flood water inundating areas upstream of Millmerran-Inglewood Road for a period greater than 10 hours. The profile also highlights ponding areas created by farming activities and existing road infrastructure as shown in Figure I2-H in Volume 2 – Appendix I.

The model indicates that the time of inundation across the floodplain during the critical 1% AEP design flood is between 10 to 18 hours.

13.5 Developed Case modelling results—Pariagara Creek

13.5.1 Drainage structures

The hydraulic design of the major drainage structures was undertaken using the hydraulic model (1D and 2D approach). In the Pariagara Creek floodplain, the Project includes the following floodplain (or regional structures):

- One waterway bridge
- Eighteen RCP locations (a total of 176 cells)
- One RCBC locations (a total of eight cells).

Local drainage structures are presented in Appendix B. These structures were sized through the local drainage design and depending on their interaction with the Pariagara Creek floodplain were incorporated in the hydraulic model.

Design bridge details have been summarised in Table 13.20, and details of the proposed culvert structures are summarised in the Cross-Drainage Register located in Appendix B. The location of cross-drainage features has been presented in Figure I1-F in Volume 2 – Appendix I.

Table 13.20 Pariagara Creek—Proposed bridge location and details

Chainage (km)	Structure ID	Approximate span (m)	Deck width (m)	Deck level (mAHD)	Deck superstructure type	Deck depth (mm)
67.35	310-BR05	345	3.97	287.7	Type D1	2,000

All bridges were modelled as an opening in the rail embankment. The optimisation of bridge lengths was balanced between minimising the changes to the hydraulic regime, primarily afflux and velocities, and the cost of replacing bridge spans by large earth embankments.

Blockage of hydraulic structures was assessed in accordance with ARR 2019. The predominant vegetation in the catchment is considered heavily forested, however, given the appreciable size of the bridge spans, and the relative size of upstream debris, a blockage factor was not applied to bridges. Significant amounts of debris accumulating against the piers of the bridge are considered low.

13.5.2 Hydraulic design criteria outcomes

The hydraulic model was run for the Developed Case for the range of events (20%, 10%, 5%, 2%, 1% AEP events and 1 in 2,000 AEP, 1 in 10,000 AEP and PMF events) with the outcomes presented in the following sections.

13.5.2.1 Rail immunity and structures results

Appendix B summarises the peak 1% AEP water levels on the upstream side of the Project alignment.

The results of flood modelling indicate that a 1% AEP event flood immunity to the rail formation is achieved for the Project alignment across the Pariagara Creek floodplain, and that peak water levels remain below the proposed rail formation level. There is over 1.2 m freeboard to the rail formation level for the 1% AEP event, noting that the formation levels were generally dictated by other engineering criteria and constraints.

13.5.3 Flood Impact Objectives outcomes—Pariagara Creek

The impact of the Developed Case was assessed through inclusion of the Project's revised reference design and comparison of model results against the Existing Case results.

Changes to flooding behaviour that were assessed, and are reported in the following sections, include potential changes in peak water levels, time of inundation, velocity, hazard and peak flows within the floodplain:

- Changes in 20% AEP and 1% AEP peak water levels are presented in Figures I4-A and I4-B in Volume 2 – Appendix I
- Changes in 20% AEP and 1% AEP velocities are presented in Figures I4-C and I4-D in Volume 2 – Appendix I
- Changes in 20% AEP and 1% AEP hazard are presented in Figures I4-E and I4-F in Volume 2 – Appendix I
- Changes in 20% AEP and 1% AEP time of inundation are presented in Figures I4-G and I4-H in Volume 2 – Appendix I
- Changes in 20% AEP and 1% AEP velocities and percentage change in velocities with cut-offs applied are presented in Figures I5-A to I5-F in Volume 2 – Appendix I
- Percentage changes in 20% AEP and 1% AEP in hazard are presented in Figures I6-A and I6-B in Volume 2 – Appendix I
- Percentage changes in 20% AEP and 1% AEP in time of inundation and with cut-offs applied are presented in Figures I7-A and I7-D in Volume 2 – Appendix I
- Refer to the [Online Digital Portal](#) for other modelled AEP results and higher resolution mapping.

One-on-one consultation with landowners who are expected to experience changes in flooding behaviour on the property was conducted by ARTC, supported by FFJV. Refer to EIS Chapter 14: Flooding and Geomorphology and EIS Appendix E: Consultation Report for further details.

The Project design outcomes relative to the FIOs (Table 4.2) are presented in the following sections.

Flood sensitive receptors referred to in this section are shown in Figures P1 (1 to 59) in Volume 2 – Appendix P.

13.5.3.1 Flood impacts on flood sensitive receptors

Based on the available aerial imagery, no buildings or critical structures are located within the area impacted by afflux and change in hazard in the Pariagara Creek floodplain for events up to and including the 1 in 2,000 AEP event.

13.5.3.2 Flood impacts on State-controlled roads

Within the extent of the hydraulic model, the only State-controlled road which is influenced by flooding and the Project is Millmerran-Inglewood Road. Table 13.21 identifies the location of potential FIO exceedances and the details for each exceedance, including afflux, velocity, time of inundation and hazard. Justification and proposed mitigation measures are also detailed within Table 13.21.

The following provide a summary of the key exceedances:

- Millmerran-Inglewood Road (road section ID #7) is the only State-controlled road within the hydraulic model extents. The road is subject to changes in peak water level and time of inundation duration. Exceedance values have been listed below:
 - During the 5% AEP event, peak water levels are expected to increase by approximately 13 mm, and for an extent of 30 m
 - For the 10% AEP, the maximum percentage change in the duration of inundation is approximately 70.4 per cent (increasing from 0.66 hours to 1.12 hours), for an extent of 5 m.

- Potential exceedances at ID #7 are likely a result of rail design constraints altering the existing flow conditions (i.e. volume and hydrograph timing). Additionally, it is expected that the maturity of the cross-drainage design is limited by available survey.
- Further mitigation measures including further cross-drainage optimisation, and stakeholder consultation will be undertaken during detailed design.

The extent of the model, and location of the State-controlled road is shown in Figure 13.7.

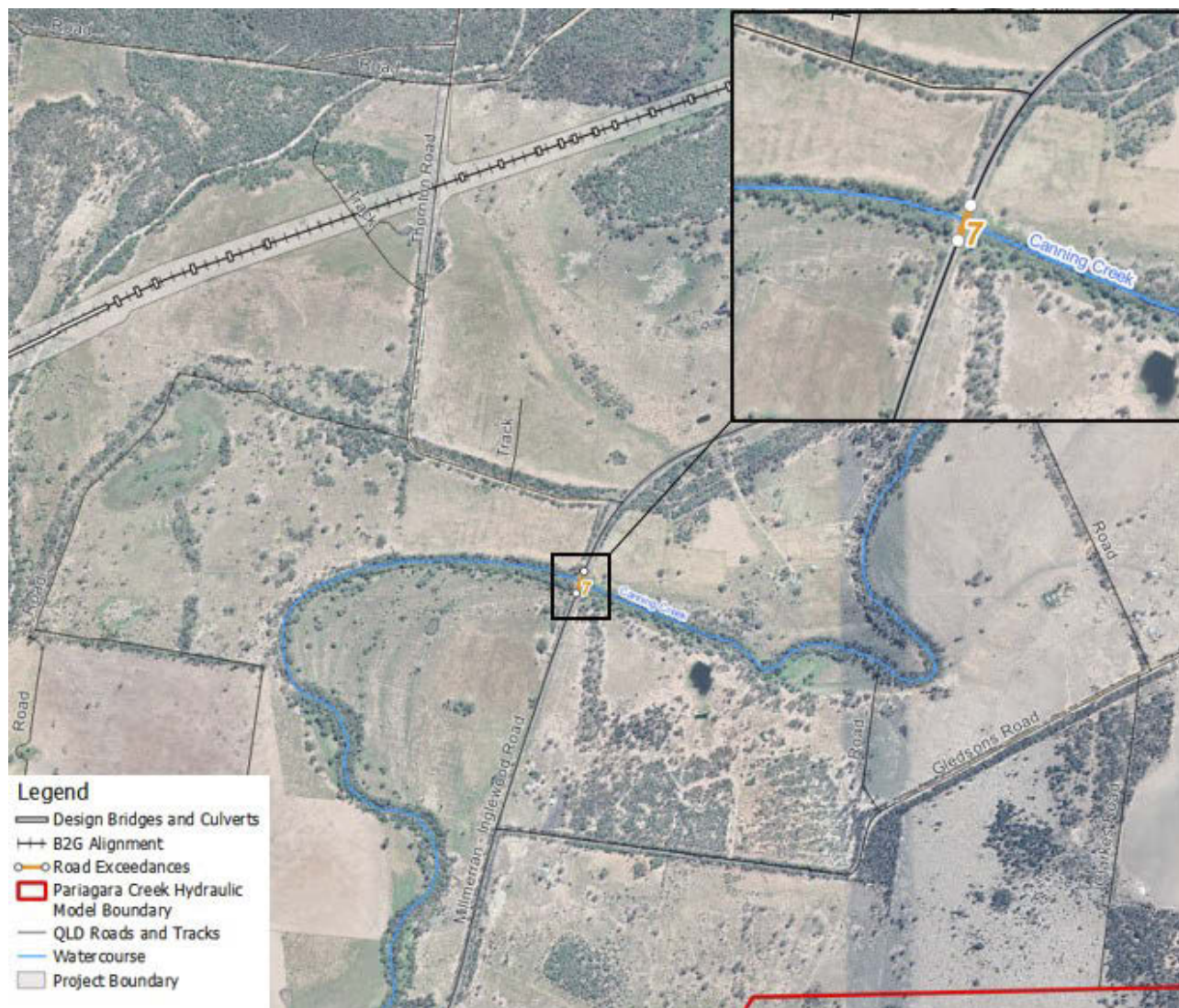


Figure 13.7 Pariagara Creek—Potentially impacted State-controlled Roads

Table 13.21 Pariagara Creek – Summary of flood impacts on State-controlled roads

Road Object ID	Road Segment	Road Name	Surface Type	Flood Impact Objective Target					AEP	Maximum FIO Exceedance					Length (m)				Exceedance Justification	Proposed Mitigation Measures ⁷
				Change in Peak Water Level (mm)	Change in Time of Inundation (%)	Change in Velocity (%)	Change in Velocity (m/s)	Change in Hazard (%)		Change in Peak Water Level (mm)	Change in Time of Inundation (% Change)	Change in Velocity (%)	Change in Velocity (m/s)	Change in Hazard (%)	Afflux	Time on Inundation	Velocity	Hazard		
7	16015103990000232644	Millnerian - Inglewood Road	Sealed	10	10	20	1	10		20%	-	-	-	-	-	-	-	-	- Maturity of current cross-drainage design (culverts and bridges) is limited by available information (e.g. detailed survey). - Rail design constraints alter the existing flow regime. - Stakeholder Consultation (Landowner/ Government).	- Further cross-drainage optimization investigations are required during detailed design.
										10%	70	-	-	-	-	5.00	-	-		
										5%	13	-	-	-	30.00	-	-	-		
										2%	-	-	-	-	-	-	-	-		
										1%	-	-	-	-	-	-	-	-		
										0.05%	-	-	-	-	-	-	-	-		

Table Notes:

1. Time of Submergence has been measured across the full length of the road segment and does not correspond to the exact location of exceedance within that segment

2. Change in water level (afflux) is calculated based on the maximum water surface level difference between the existing and design case results, and does not consider the underlying topography. Depth values presented are based on the respective underlying topographies and in some cases do not align with afflux calculations. This can be attributed to interpolation calculations of results performed within the hydraulic model.

3. The above table should be read in conjunction with change maps provided as part of Volume II and the Digital Platform.

4. The symbol "-" denotes that no change above Flood Impact Objectives has been determined and therefore no result has been presented.

5. Changes recorded above may only occur during a small fraction of model simulation time.

6. Three decimal places do not infer absolute accuracy in modelling results, but it is necessary to identify small changes.

7. Design optimisations that could be implemented (but not limited to) include adjustment of culvert spacing, realignment of culverts to existing flowpaths when more detailed survey is available, modifying the grade of culverts, road height and alignment adjustments, removal of bridge abutments from waterway extents etc

Depth (m) ²		Time of Inundation in hours (related to % exceedance)		Time of Submergence (hours) ¹		Velocity in m/s (related to % exceedance)		Velocity in m/s (related to actual change exceedance)		Hazard in m ² /s (related to % exceedance)	
Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case
-	-	-	-	-	-	-	-	-	-	-	-
-	-	0.66	1.12	5.48	5.48	-	-	-	-	-	-
0.621	0.635	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-

13.5.3.3 Flood impacts on local public roads

Within the extent of the hydraulic model, 11 local public roads are expected to be influenced by the Project alignment. Table 13.22 identifies the location of potential FIO exceedances and the details for each exceedance, including afflux, velocity, time of inundation and hazard. Justification and proposed mitigation measures are also detailed.

The following provide a summary of the key exceedances.

- Ten local roads show potential FIO exceedances due to afflux. Most impacted are located upstream of the Project alignment. The unsealed track (road section ID #35) shows the largest recorded value for afflux with a maximum value of approximately 228 mm, occurring during the 1% AEP event. The corresponding length of impact is approximately 372 m.
- Changes in the duration of inundation may occur for a total of 10 roads. The maximum percentage change in flood inundation was recorded at Sandy Creek Boundary Road (approximately 194 per cent) during the 5% AEP flood event. The percentage increase equates to a duration increase of approximately 1.3 hours (from 0.66 hours to 1.94 hours).
- Velocity exceedances are noted for eight roads. This includes percentage change and 'absolute' exceedances. The maximum percentage change was recorded at ID #47 during a 5% AEP flood event, with a value 121 per cent (approximately 0.77 m/s). A maximum 'absolute' exceedance of approximately 0.84 m/s was recorded at ID #44 during the 2% AEP storm event. This increase can be attributed to a newly flood-affected cell demonstrating an increase in flood inundation in this area.
- Changes in flood hazard are noted for seven local roads and tracks. The maximum increase of 124 per cent occurs along an unsealed track at Location 35. The large percentage increase is a result of existing hazard values increasing from 0.09 m²/s to 0.21 m²/s during the 1% AEP event.
- The majority of impacts over local roads is driven by the maturity of the cross-drainage design, and limited available information (i.e. detailed survey). Further to this, it is noted that the design constraints of the Project alter the existing flow regime.
- To mitigate potential FIO exceedances, further cross-drainage optimisation along with stakeholder consultation are required during detailed design.

The extent of the model, and location of the public local roads is shown in Figure 13.8.

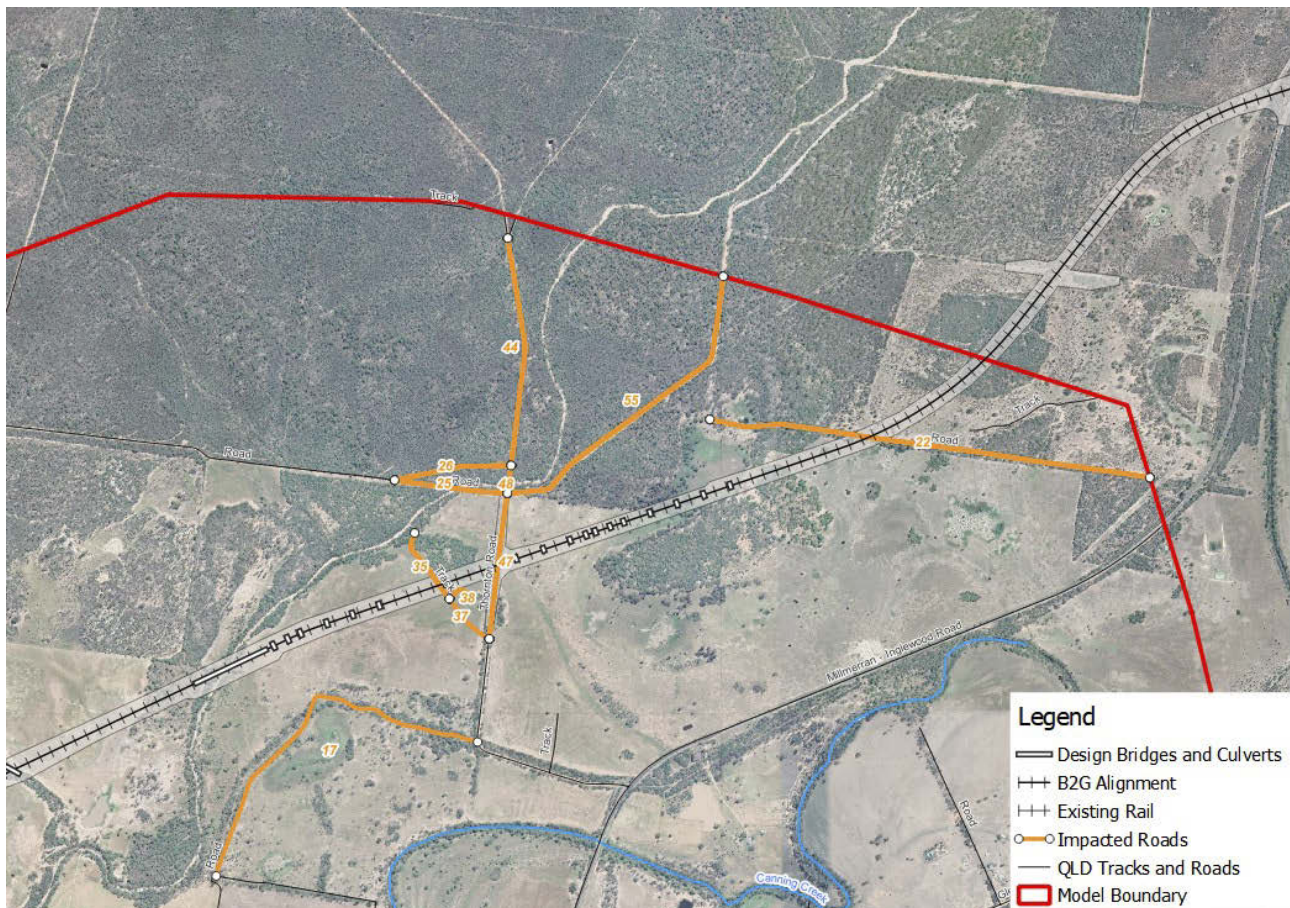


Figure 13.8 Pariagara Creek—Potentially impacted local public roads

Table 13.22 Parigara Creek - Summary of impacts on local public roads

Road Object ID	Road Segment	Road Name	Surface Type	Flood Impact Objective Target					AEP	Maximum FIO Exceedance					Length (m)				Exceedance Justification	Proposed Mitigation Measures ^a
				Change in Peak Water Level (mm)	Change in Time of Inundation (%)	Change in Velocity (%)	Change in Velocity (m/s)	Change in Hazard (%)		Change in Peak Water Level (mm)	Change in Time of Inundation (% Change)	Change in Velocity (%)	Change in Velocity (m/s)	Change in Hazard (%)	Afflux	Time on Inundation	Velocity	Hazard		
22	16015103990000433694	Road 0	Unsealed	10	10	10	0.5	10	20%	73	39	-	-	-	307.50	5.00	-	-	Inspection of the aerial imagery indicates that this road does not currently exist and is a potential future road corridor. This local road is impacted due to the presence of the rail embankment, restricting flow downstream and leading to an increase in upstream water level. Within the vicinity of this road 11 culvert crossings are proposed across the embankment ranging from 2 cells through to 40 cells. Currently due to the maturity of the design and hydraulic modelling, these culverts have been grouped into banks. This grouping can result in a restriction of flow crossing the alignment which can create upstream impacts. The exceedances remain due to the current design being at a reference design level and culvert placement and spacing being grouped (rather than spaced across the embankment), resulting in concentrated flow. Further iteration of cross drainage and placement needs to be undertaken at detailed design stage. As design progresses and modelling updates are made based on the recommendations of the Expert Flood Panel, it is envisaged upstream impacts on this road can be reduced. This same justification applies to Road IDs 25, 26, 44, 47, 48 and 55.	In detailed design, improvements to the hydraulic model will be undertaken to capture detailed survey and address outstanding comments made by the Expert Flood Panel. In addition, a review and optimisation of the cross drainage design at this location will be undertaken to spread culverts along the embankment and improve the conveyance downstream of the alignment. It is anticipated some reduction in exceedances can occur along this road segment. In addition, consultation with the appropriate Road Manager will be undertaken in accordance with the Flood Impact Objectives, to discuss impacts to this road and agree on any site specific mitigation measures.
										10%	50	50	-	-	-	312.50	10.00	-		
										5%	62	43	-	-	-	317.50	7.50	-		
										2%	102	70	-	-	-	330.00	22.50	-		
										1%	108	81	-	-	-	330.00	15.00	-		
										0.05%	-	-	-	-	-	-	-	-		
25	16015103990000433318	Road 0	Unsealed	10	10	10	0.5	10	20%	28	-	-	-	-	407.50	-	-	75.00	Similar to Road ID 22, this local road is impacted due to the presence of the rail embankment, restricting flow downstream and leading to an increase in upstream water level. In addition, a road upgrade is also proposed on Thornton Road which additionally contributes to this exceedance. Within the vicinity of this road 11 culvert crossings are proposed across the embankment ranging from 2 cells through to 40 cells. Currently due to the maturity of the design and hydraulic modelling, these culverts have been grouped into banks. This grouping can result in a restriction of flow crossing the alignment which can create upstream impacts. The exceedances remain due to the current design being at a reference design level and culvert placement and spacing being grouped (rather than spaced across the embankment), resulting in concentrated flow. Further iteration of cross drainage and placement needs to be undertaken at detailed design stage. As design progresses and modelling updates are made based on the recommendations of the Expert Flood Panel, it is envisaged upstream impacts on this road can be reduced.	In detailed design, improvements to the hydraulic model will be undertaken to capture detailed survey and address outstanding comments made by the Expert Flood Panel. In addition, a review and optimisation of the cross drainage design at this location will be undertaken to spread culverts along the embankment and improve the conveyance downstream of the alignment. It is anticipated some reduction in exceedances can occur along this road segment. In addition, consultation with the appropriate Road Manager will be undertaken in accordance with the Flood Impact Objectives, to discuss impacts to this road and agree on any site specific mitigation measures.
										10%	29	11	11	0.05	13	70.00	2.50	17.50		
										5%	28	-	12	0.05	12	70.00	-	45.00		
										2%	42	-	12	0.06	15	432.50	-	137.50		
										1%	59	10	15	0.02	19	450.00	2.50	180.00		
										0.05%	-	-	-	-	-	-	-	-		
26	16015103990000433319	Road 0	Unsealed	10	10	10	0.5	10	20%	64	64	-	-	0.03	15	355.00	12.50	12.50	Similar to Road ID 22, this local road is impacted due to the presence of the rail embankment, restricting flow downstream and leading to an increase in upstream water level. In addition, a road upgrade is also proposed on Thornton Road which additionally contributes to this exceedance. Within the vicinity of this road 11 culvert crossings are proposed across the embankment ranging from 2 cells through to 40 cells. Currently due to the maturity of the design and hydraulic modelling, these culverts have been grouped into banks. This grouping can result in a restriction of flow crossing the alignment which can create upstream impacts. The exceedances remain due to the current design being at a reference design level and culvert placement and spacing being grouped (rather than spaced across the embankment), resulting in concentrated flow. Further iteration of cross drainage and placement needs to be undertaken at detailed design stage. As design progresses and modelling updates are made based on the recommendations of the Expert Flood Panel, it is envisaged upstream impacts on this road can be reduced.	In detailed design, improvements to the hydraulic model will be undertaken to capture detailed survey and address outstanding comments made by the Expert Flood Panel. In addition, a review and optimisation of the cross drainage design at this location will be undertaken to spread culverts along the embankment and improve the conveyance downstream of the alignment. It is anticipated some reduction in exceedances can occur along this road segment. In addition, consultation with the appropriate Road Manager will be undertaken in accordance with the Flood Impact Objectives, to discuss impacts to this road and agree on any site specific mitigation measures.
										10%	22	-	-	0.02	-	102.50	-	17.50		
										5%	26	-	-	0.02	-	145.00	-	20.00		
										2%	42	-	-	0.04	12	440.00	-	35.00		
										1%	60	31	-	0.05	15	445.00	5.00	32.50		
										0.05%	-	-	-	-	-	-	-	-		
35	16015103990000488752	Track 0	Unsealed	10	10	10	0.5	10	20%	22	-	-	-	0.01	-	57.50	-	2.50	This local road is split by the alignment with its operational capacity impacted due to the construction of the Project embankment. In the existing case, flood waters overflow flow across this track, south towards Carrington Creek. In the design case, these floodwaters are concentrated to one 20 cell bank of culverts concentrating flows and encouraging ponding on the upstream side. This ponding leads to the identified exceedances of an increase in water level and time of inundation. The exceedances remain due to the current design being at a reference design level and culvert placement and spacing being grouped (rather than spaced across the embankment), resulting in concentrated flow. Further iteration of cross drainage and placement needs to be undertaken at detailed design stage. As design progresses and modelling updates are made based on the recommendations of the Expert Flood Panel, it is envisaged upstream impacts on this road can be reduced.	In detailed design, improvements to the hydraulic model will be undertaken to better represent the existing flow regime and spread culverts along the embankment and improve the conveyance downstream of the alignment. It is anticipated some reduction in exceedances can occur along this road segment. In addition, consultation with the appropriate Road Manager will be undertaken in accordance with the Flood Impact Objectives, to discuss the future operational capacity of this road, any impacts and agree on any site specific mitigation measures.
										10%	-	-	-	-	-	-	-	-		
										5%	100	194	-	-	-	20.00	12.50	-		
										2%	175	16	-	-	-	225.00	5.00	-		
										1%	228	38	13	0.11	125	372.50	15.00	120.00		
										0.05%	-	-	-	-	-	-	-	-		
44	16015103990000363275	Thornton Road	Unsealed	10	10	10	0.5	10	20%	21	-	-	-	-	-	72.50	-	-	Similar to Road ID 22, this local road is impacted due to the presence of the rail embankment, restricting flow downstream and leading to an increase in upstream water level. In addition, a road upgrade is also proposed on Thornton Road which additionally contributes to this exceedance. Within the vicinity of this road 11 culvert crossings are proposed across the embankment ranging from 2 cells through to 40 cells. Currently due to the maturity of the design and hydraulic modelling, these culverts have been grouped into banks. This grouping can result in a restriction of flow crossing the alignment which can create upstream impacts. The exceedances remain due to the current design being at a reference design level and culvert placement and spacing being grouped (rather than spaced across the embankment), resulting in concentrated flow. Further iteration of cross drainage and placement needs to be undertaken at detailed design stage. As design progresses and modelling updates are made based on the recommendations of the Expert Flood Panel, it is envisaged upstream impacts on this road can be reduced.	In detailed design, improvements to the hydraulic model will be undertaken to capture detailed survey and address outstanding comments made by the Expert Flood Panel. In addition, a review and optimisation of the cross drainage design at this location will be undertaken to spread culverts along the embankment and improve the conveyance downstream of the alignment. It is anticipated some reduction in exceedances can occur along this road segment. In addition, consultation with the appropriate Road Manager will be undertaken in accordance with the Flood Impact Objectives, to discuss impacts to this road and agree on any site specific mitigation measures.
										10%	25	27	-	-	12	92.50	10.00	-		
										5%	105	99	-	0.01	14	112.50	12.50	2.50		
										2%	130	79	-	0.84	22	252.50	62.50	30.00		
										1%	70	35	-	0.05	36	417.50	25.00	22.50		
										0.05%	-	-	-	-	-	-	-	-		
47	16015103990000363278	Thornton Road	Unsealed	10	10	10	0.5	10	20%	75	-	-	-	0.39	-	95.00	-	30.00	Similar to Road ID 22, Thornton Road is impacted due to the presence of the rail embankment, restricting flow downstream and leading to an increase in upstream water level. Although an upgrade to Thornton Road is proposed, the implications of the embankment restricting flow downstream still lead to propagation of impacts on the northern end of the upgraded section. Within the vicinity of this road 11 culvert crossings are proposed across the embankment ranging from 2 cells through to 40 cells. Currently due to the maturity of the design and hydraulic modelling, these culverts have been grouped into banks. This grouping can result in a restriction of flow crossing the alignment which can create upstream impacts. The exceedances remain due to the current design being at a reference design level and culvert placement and spacing being grouped (rather than spaced across the embankment), resulting in concentrated flow. Further iteration of cross drainage and placement needs to be undertaken at detailed design stage. As design progresses and modelling updates are made based on the recommendations of the Expert Flood Panel, it is envisaged upstream impacts on this road can be reduced.	In detailed design, improvements to the hydraulic model will be undertaken to capture detailed survey and address outstanding comments made by the Expert Flood Panel. In addition, a review and optimisation of the cross drainage design at this location will be undertaken to spread culverts along the embankment and improve the conveyance downstream of the alignment. It is anticipated some reduction in exceedances can occur along this road segment. In addition, consultation with the appropriate Road Manager will be undertaken in accordance with the Flood Impact Objectives, to discuss impacts to this road and agree on any site specific mitigation measures.
										10%	39	26	117	0.14	63	105.00	2.50	87.50		
										5%	32	15	122	0.16	63	110.00	2.50	77.50		
										2%	42	-	119	-	49	120.00	-	132.50		
										1%	59	-	114	-	44	127.50	-	137.50		
										0.05%	-	-	-	-	-	-	-	-		

Depth (m) ²		Time of Inundation in hours (related to % exceedance)		Time of Submergence (hours) ³		Velocity in m/s (related to % exceedance)		Velocity in m/s (related to actual change exceedance)		Hazard in m/s ² (related to % exceedance)	
Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case
-	0.073	1.22	1.70	4.57	4.70	-	-	-	-	-	-
0.786	0.836	1.85	2.77	8.34	8.43	-	-	-	-	-	-
0.940	1.002	2.13	3.05	9.26	9.35	-	-	-	-	-	-
-	0.102	1.95	3.32	7.89	8.68	-	-	-	-	-	-
1.269	1.377	2.00	3.63	13.66	13.73	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
0.183	0.210	-	-	-	-	-	-	-	-	-	-
0.294	0.323	5.05	5.60	13.85	13.85	0.502	0.558	0.461	0.514	0.22	0.25
0.570	0.598	-	-	-	-	0.545	0.609	0.489	0.537	0.38	0.43
0.761	0.802	-	-	-	-	0.653	0.734	0.500	0.559	0.58	0.67
0.902	0.960	3.69	4.07	17.28	17.27	0.564	0.647	0.500	0.519	0.55	0.65
-	-	-	-	-	-	-	-	-	-	-	-
-	0.064	0.96	1.58	6.50	6.50	-	-	0.471	0.500	0.13	0.15
0.765	0.787	-	-	-	-	-	-	0.497	0.519	-	-
0.939	0.965	-	-	-	-	-	-	0.482	0.506	-	-
1.128	1.170	-	-	-	-	-	-	0.466	0.504	0.52	0.59
1.236	1.296	2.02	2.85	14.80	14.80	-	-	0.493	0.540	0.62	0.71
-	-	-	-	-	-	-	-	-	-	-	-
1.210	1.232	-	-	-	-	-	-	0.498	0.509	-	-
-	-	-	-	-	-	-	-	-	-	-	-
-	0.100	0.52	1.53	12.51	12.51	-	-	-	-	-	-
0.138	0.168	3.39	3.94	16.00	16.00	-	-	-	-	-	-
0.218	0.278	6.88	9.47	15.78	15.77	0.506	0.574	0.422	0.531	0.09	0.21
-	-	-	-	-	-	-	-	-	-	-	-
0.220	0.241	-	-	-	-	-	-	-	-	-	-
0.122	0.147	2.69	3.42	11.26	11.26	-	-	-	-	0.14	0.16
-	0.105	0.28	0.56	12.01	12.00	-	-	0.495	0.502	0.14	0.16
-	0.130	0.65	1.16	14.92	14.92	-	-	-	0.839	0.14	0.17
0.170	0.241	2.08	2.81	15.56	15.56	-	-	0.499	0.546	0.14	0.19
-	-	-	-	-	-	-	-	-	-	-	-
-	0.075	-	-	-	-	-	-	0.414	0.807	-	-
0.304	0.343	3.40	4.30	8.57	8.58	0.589	1.280	0.500	0.643	0.17	0.28
0.479	0.511	4.95	5.68	9.52	9.53	0.631	1.401	0.361	0.524	0.28	0.45
0.670	0.712	-	-	-	-	0.642	1.403	-	-	0.42	0.63
0.748	0.807	-	-	-	-	0.670	1.433	-	-	0.52	0.75
-	-	-	-	-	-	-	-	-	-	-	-

Road Object ID	Road Segment	Road Name	Surface Type	Flood Impact Objective Target					AEP	Maximum FIO Exceedance					Length (m)				Exceedance Justification	Proposed Mitigation Measures*
				Change in Peak Water Level (mm)	Change in Time of Inundation (%)	Change in Velocity (%)	Change in Velocity (m/s)	Change in Hazard (%)		Change in Peak Water Level (mm)	Change in Time of Inundation (% Change)	Change in Velocity (%)	Change in Velocity (m/s)	Change in Hazard (%)	Afflux	Time on Inundation	Velocity	Hazard		
48	16015103990000363279	Thornton Road	Unsealed	10	10	10	0.5	10		20%	22	-	-	-	-	142.50	-	-	Similar to Road ID 22, this local road is impacted due to the presence of the rail embankment, restricting flow downstream and leading to an increase in upstream water level. In addition, a road upgrade is also proposed on Thornton Road which additionally contributes to this exceedance. Within the vicinity of this road 11 culvert crossings are proposed across the embankment ranging from 2 cells through to 40 cells. Currently due to the maturity of the design and hydraulic modelling, these culverts have been grouped into banks. This grouping can result in a restriction of flow crossing the alignment which can create upstream impacts.	In detailed design, improvements to the hydraulic model will be undertaken to capture detailed survey and address outstanding comments made by the Expert Flood Panel. In addition, a review and optimisation of the cross drainage design at this location will be undertaken to spread culverts along the embankment and improve the conveyance downstream of the alignment. It is anticipated some reduction in exceedances can occur along this road segment. In addition, consultation with the appropriate Road Manager will be undertaken in accordance with the Flood Impact Objectives, to discuss impacts to this road and agree on any site specific mitigation measures.
										10%	23	-	0.03	11	147.50	-	2.50	12.50		
										5%	27	-	-	15	147.50	-	-	10.00		
										2%	43	-	-	0.04	15	147.50	-	10.00		
										1%	61	-	10	0.05	17	147.50	-	15.00		
										0.05%	-	-	-	-	-	-	-	-		
55	16015103990000327348	Sandy Creek Boundary Road	Unsealed	10	10	10	0.5	10		20%	44	43	12	0.06	-	997.50	35.00	10.00	Similar to Road ID 22, this local road is impacted due to the presence of the rail embankment, restricting flow downstream and leading to an increase in upstream water level. In addition, a road upgrade is also proposed on Thornton Road which additionally contributes to this exceedance. Within the vicinity of this road 11 culvert crossings are proposed across the embankment ranging from 2 cells through to 40 cells. Currently due to the maturity of the design and hydraulic modelling, these culverts have been grouped into banks. This grouping can result in a restriction of flow crossing the alignment which can create upstream impacts.	In detailed design, improvements to the hydraulic model will be undertaken to capture detailed survey and address outstanding comments made by the Expert Flood Panel. In addition, a review and optimisation of the cross drainage design at this location will be undertaken to spread culverts along the embankment and improve the conveyance downstream of the alignment. It is anticipated some reduction in exceedances can occur along this road segment. In addition, consultation with the appropriate Road Manager will be undertaken in accordance with the Flood Impact Objectives, to discuss impacts to this road and agree on any site specific mitigation measures.
										10%	79	121	-	0.01	21	1112.50	37.50	7.50		
										5%	168	194	-	0.03	27	1365.00	232.50	2.50		
										2%	153	112	-	-	28	1482.50	80.00	-		
										1%	180	184	-	0.01	28	1555.00	122.50	2.50		
										0.05%	-	-	-	-	-	-	-	-		
37	16015103990000488750	Track 0	Unsealed	10	10	10	0.5	10		20%	-	-	-	-	-	-	-	-	Exceedances experienced on both this Track and Track ID38, occur due to the constriction of flow through the embankment via culvert C08.41. In the existing case, floodwaters overflow flow across this track. In the design case the inclusion of the rail embankment and discharge location of culvert C08.41, which is directly upstream of this Track, results in concentrated runoff overtopping this track and increasing water level and time of inundation.	In detailed design, improvements to the hydraulic model will be undertaken to better represent the existing flow regime and spread culverts along the embankment and improve the conveyance downstream of the alignment. It is anticipated some reduction in exceedances can occur along this road segment. In addition, consultation with the appropriate Road Manager will be undertaken in accordance with the Flood Impact Objectives, to discuss the future operational capacity of this road, any impacts and agree on any site specific mitigation measures.
										10%	-	-	-	-	-	-	-	-		
										5%	138	183	-	-	-	142.50	52.50	-		
										2%	-	-	-	-	-	-	-	-		
										1%	-	-	-	-	-	-	-	-		
										0.05%	-	-	-	-	-	-	-	-		
38	16015103990000488751	Track 0	Unsealed	10	10	10	0.5	10		20%	-	-	-	-	-	-	-	-	Similar to ID37, exceedances experienced on this Track occur due to the constriction of flow through the embankment via culvert C08.41. In the existing case, floodwaters overflow flow across this track. In the design case the inclusion of the rail embankment and discharge location of culvert C08.41, which is directly upstream of this Track, results in concentrated runoff overtopping this track and increasing water level and time of inundation.	In detailed design, improvements to the hydraulic model will be undertaken to better represent the existing flow regime and spread culverts along the embankment and improve the conveyance downstream of the alignment. It is anticipated some reduction in exceedances can occur along this road segment. In addition, consultation with the appropriate Road Manager will be undertaken in accordance with the Flood Impact Objectives, to discuss the future operational capacity of this road, any impacts and agree on any site specific mitigation measures.
										10%	-	-	-	-	-	-	-	-		
										5%	66	40	-	-	-	32.50	15.00	-		
										2%	11	62	-	0.04	-	20.00	40.00	2.50		
										1%	-	14	-	-	-	-	40.00	-		
										0.05%	-	-	-	-	-	-	-	-		
17	16015103990000433394	Road 0	Unsealed	10	10	10	0.5	10		20%	-	-	-	-	-	-	-	-	This exceedance occurs downstream of the Project alignment and on the fringes of the flood extent. Upstream constriction of flow through Bridge 310-BR05 results in increases in water level and change in flood extent downstream. This change results in line of foundation increases on the fringes of the flowpath which ultimately impacts this Track. It should be noted that approximately 2.5m of impact has been recorded for this exceedance and therefore due to the extremely localised increase, it is not anticipated that this change will have an impact on the operational capacity of this track.	Due to the constriction of flow upstream, it is proposed to undertake optimisation of the placement of bridge 310-BR05 abutments to reduce the downstream impact. In addition, consultation with the appropriate Road Manager will be undertaken in accordance with the Flood Impact Objectives, to discuss the future operational capacity of this road, any impacts and agree on any site specific mitigation measures.
										10%	-	-	-	-	-	-	-	-		
										5%	-	-	-	-	-	-	-	-		
										2%	-	-	-	-	-	-	-	-		
										1%	-	17	-	-	-	-	2.50	-		
										0.05%	-	-	-	-	-	-	-	-		

Table Notes:
1. Time of Submergence has been measured across the full length of the road segment and does not correspond to the exact location of exceedance within that segment.
2. Change in water level (afflux) is calculated based on the maximum water surface level difference between the existing and design case results, and does not consider the underlying topography. Depth values presented are based on the respective underlying topographies and in some cases do not align with afflux calculations. This can be attributed to interpolation calculations of results performed within the hydraulic model.
3. The Flood Impact Objective exceedance has been identified due to a change in flood extent and a cell which was dry in the existing case and now flood-affected in the developed case.
4. The above table should be read in conjunction with change maps provided as part of Volume II and the Digital Platform.
5. The symbol “-” denotes that no change above Flood Impact Objectives has been determined and therefore no result has been presented.
6. Changes recorded above may only occur during a small fraction of model simulation time.
7. Three decimal places do not infer absolute accuracy in modelling results, but it is necessary to identify small changes.
8. Design optimisations that could be implemented (but not limited to) include adjustment of culvert spacing, realignment of culverts to existing flowpaths when more detailed survey is available, modifying the grade of culverts, road height and alignment adjustments, removal of bridge abutments from waterway extents etc

Depth (m) ²		Time of Inundation in hours (related to % exceedance)		Time of Submergence (hours) ³		Velocity in m/s (related to % exceedance)		Velocity in m/s (related to actual change exceedance)		Hazard in m/s ² (related to % exceedance)	
Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case
2.416	2.438	-	-	-	-	-	-	-	-	-	-
2.309	2.333	-	-	-	-	-	-	0.483	0.509	0.50	0.56
2.487	2.515	-	-	-	-	-	-	-	-	0.37	0.43
2.276	2.319	-	-	-	-	-	-	0.471	0.508	0.63	0.73
2.384	2.445	-	-	-	-	0.897	0.989	0.457	0.506	0.78	0.91
-	-	-	-	-	-	-	-	-	-	-	-
0.301	0.345	0.64	0.92	8.74	8.74	0.506	0.565	0.460	0.522	-	-
-	0.073	1.19	2.83	13.00	13.00	-	-	0.497	0.511	0.17	0.21
-	0.168	0.66	1.94	14.07	14.07	-	-	0.479	0.509	0.22	0.28
-	0.153	0.94	1.99	17.13	17.13	-	-	-	-	0.33	0.43
-	0.180	1.08	3.06	18.53	16.52	-	-	0.497	0.510	0.42	0.54
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
0.244	0.310	1.94	2.71	3.64	4.49	-	-	-	-	-	-
0.206	0.218	1.12	1.81	3.92	3.96	-	-	0.488	0.525	-	-
-	-	5.95	6.77	10.46	10.49	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
-	-	2.35	2.75	10.62	10.46	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-

13.5.3.4 Flood impacts on existing rail infrastructure

No third-party rail infrastructure is located within the Pariagara Creek hydraulic model boundary.

13.5.3.5 Flood impacts on private land outside the permanent footprint

In accordance with the FIOs, Table 13.23 presents the modelled changes and potential impacts on third party property outside of the Project's permanent footprint. A summary of key impacts, exceedance justification and mitigation measures are provided in the below points with additional details for each exceedance provided in Table 13.23:

- 27 lot type parcels have been identified as having potential impacts exceeding FIO targets. The majority of all potential exceedances occurred in 'Cattle Breeding and Fattening' and 'Grains' land use types. Other recorded land use types include 'Forestry and Logs', 'Cattle Grazing and Breeding', 'Vacant – Large House Site', 'Dwelling – Large House Site', 'Small Crops and Fodder Irrigated', and 'Noxious/ Offensive Industry (Including Abattoir)'. Two watercourse parcels were also identified with exceedances as part of this investigation:
 - A number of impacted properties have multiple land use designations, or potentially incorrect land use designation assigned. Confirmation of assigned land use will take place during detailed design.
- A total of 15 parcels recorded potential changes in peak water levels. The majority of these changes are driven by a change in flood extent results in model cells becoming newly wet and therefore generating a large increase. These have been identified in orange in Table 13.23. The maximum increase for all lots is approximately 0.9 m on Lot 71MA3479, recorded in the 1% AEP flood event. It should be noted that most large increases are localised and don't translate to uniform flood level increases across the lot.
- Time of inundation exceedances occur in all listed lot type parcels. The maximum change in the time of inundation is 9.1 hours (Lot Plan 87MH16) and is attributed to a change in flood extent and newly flood-affected cell. The largest percentage change in flood duration was 455 per cent on Lot Plan 87MH16, which represents a flood inundation increase of 4.61 hours during the 1% AEP flood event.
- Velocity increases ('absolute' and percentage change) were recorded for 15 lots. The maximum percentage change in velocity for all properties is approximately 72 per cent on Lot Plan 1RP156957 (increasing from 0.50 m/s to 0.87 m/s), and the maximum 'absolute' change is approximately 2.1 m/s on Lot Plan 87MH16. The 'absolute' change can be attributed to newly flood-affected cells and a change in flood extent.
- 12 parcel types, including two designated watercourses, recorded FIO exceedances against flood hazard criteria. Maximum change in hazard for all properties is <1 m²/s. This implies that all exceedances occurred in slow moving water, or in shallower section of the floodplain.

Recorded exceedances are likely a result of the maturity of current cross-drainage (culverts and bridges) design, and limited available information (e.g. detailed survey). In order to mitigate potential exceedances, further cross-drainage optimisation investigations, and ongoing model strengthening may occur in detailed design.

Table 13.23 Pariagara Creek – Summary of flood impacts on private land outside the permanent footprint

ID	ID (Lotplan ID/FSR ID/ Culvert ID)	Sensitive Receptor Type	Catchment Type	Landuse ¹¹	Lot Area (m ²)	Flood Impact Objective Target						AEP	Maximum FIO Exceedance						Area (ha) ¹				Exceedance Justification	Proposed Mitigation Measures ¹²	
						Change in Peak Water Level (mm)	Change in Time of Inundation (Hours)	Change in Time of Inundation (Hours)	Change in Velocity (%)	Change in Velocity (m/s)	Change in Hazard (%)		Change in Peak Water Level (mm)	Change in Time of Inundation (%)	Change in Time of Inundation (hours)	Change in Velocity (%)	Change in Velocity (m/s)	Change in Hazard (%)	Afflux	Time on Inundation	Velocity	Hazard			
1	341FTY1881	Lot Type Parcel	Regional	FORESTRY & LOGS	721,413	200	10	1	10	0.5	20	20%	-	97	1.28	-	-	122	-	1.23	-	0.01	- Maturity of current cross-drainage design is limited by available information (e.g. detailed survey) - Identified property has multiple land use designations, or potentially incorrect land use designations assigned, resulting in a conservative FIO assessment. - The Flood Impact Objective exceedance has been identified due to a change in Flood extent and a cell which was dry in the existing case and now flood-affected in the developed case.	- Further cross-drainage optimization investigations are required during detailed design. - Confirmation of the assigned landuse.	
												10%	-	234	2.77	-	-	93	-	1.42	-	0.13			
												5%	799	246	5.79	-	0.03	54	0.05	2.63	0.004	0.18			
												2%	-	223	2.54	-	0.07	37	-	3.21	0.05	0.12			
												1%	-	389	3.74	15	0.08	35	-	4.22	0.04	0.48			
												0.05%	-	-	-	-	-	-	-	-	-	-			
												20%	-	97	1.28	-	-	122	-	1.23	-	0.01			
2	ASP274206	Lot Type Parcel	Regional	CATTLE BREEDING & FATTENING	721,413	200	10	1	10	0.5	20	10%	-	234	2.77	-	-	93	-	1.42	-	0.13	- Maturity of current cross-drainage design is limited by available information (e.g. detailed survey) - Identified property has multiple land use designations, or potentially incorrect land use designations assigned, resulting in a conservative FIO assessment. - The Flood Impact Objective exceedance has been identified due to a change in Flood extent and a cell which was dry in the existing case and now flood-affected in the developed case.	- Further cross-drainage optimization investigations are required during detailed design. - Confirmation of the assigned landuse.	
												5%	799	246	5.79	-	0.03	54	0.05	2.63	0.004	0.18			
												2%	-	223	2.54	-	0.07	37	-	3.21	0.05	0.12			
												1%	-	389	3.74	15	0.08	35	-	4.22	0.04	0.48			
												0.05%	-	-	-	-	-	-	-	-	-	-			
												20%	-	97	1.28	-	-	122	-	1.23	-	0.01			
												10%	-	234	2.77	-	-	93	-	1.42	-	0.13			
3	ASP274206	Lot Type Parcel	Regional	CATTLE GRAZING & BREEDING	721,413	200	10	1	10	0.5	20	5%	799	246	5.79	-	0.03	54	0.05	2.63	0.004	0.18	- Maturity of current cross-drainage design is limited by available information (e.g. detailed survey) - Identified property has multiple land use designations, or potentially incorrect land use designations assigned, resulting in a conservative FIO assessment.	- Further cross-drainage optimization investigations are required during detailed design. - Confirmation of the assigned landuse.	
												2%	-	223	2.54	-	0.07	37	-	3.21	0.05	0.12			
												1%	-	389	3.74	15	0.08	35	-	4.22	0.04	0.48			
												0.05%	-	-	-	-	-	-	-	-	-	-			
												20%	-	97	1.28	-	-	122	-	1.23	-	0.01			
												10%	-	234	2.77	-	-	93	-	1.42	-	0.13			
												5%	799	246	5.79	-	0.03	54	0.05	2.63	0.004	0.18			
4	25P109525	Lot Type Parcel	Regional	CATTLE BREEDING & FATTENING	873,028	200	10	1	10	0.5	20	10%	-	94	1.25	-	-	-	-	0.44	-	-	- Maturity of current cross-drainage design is limited by available information (e.g. detailed survey) - Identified property has multiple land use designations, or potentially incorrect land use designations assigned, resulting in a conservative FIO assessment. - The Flood Impact Objective exceedance has been identified due to a change in Flood extent and a cell which was dry in the existing case and now flood-affected in the developed case.	- Further cross-drainage optimization investigations are required during detailed design. - Stakeholder Consultation (Landowner/ Government).	
												5%	-	217	2.45	-	-	-	-	0.46	-	-			
												2%	-	209	2.66	-	-	-	-	0.61	-	-			
												1%	-	203	2.42	-	-	-	-	0.74	-	-			
												0.05%	-	270	3.16	-	-	-	-	0.96	-	-			
												20%	-	126	1.16	-	0.02	-	-	0.62	0.06	-			
												10%	-	38	-	23	0.13	25	-	0.49	0.69	0.05			
5	97MH38	Lot Type Parcel	Regional	CATTLE BREEDING & FATTENING	1,166,919	200	10	1	10	0.5	20	5%	512	232	4.70	24	0.68	25	0.18	1.40	0.86	0.05	- Maturity of current cross-drainage design is limited by available information (e.g. detailed survey) - Maturity of current bridge design is limited by available information (e.g. detailed survey) - The Flood Impact Objective exceedance has been recorded due to a change in flood extent resulting in flood water filling a depression in the topography. The total depth of the water within the depression has been recorded as the change in WL (afflux).	- Further cross-drainage optimization investigations are required during detailed design. - Further road and/or rail design optimization investigations are required during detailed design. - Stakeholder Consultation (Landowner/ Government).	
												2%	-	221	2.59	50	0.76	146	-	3.59	1.66	1.32			
												1%	-	311	3.47	43	0.31	323	-	6.06	3.69	5.89			
												0.05%	-	-	-	-	-	-	-	-	-	-			
												20%	-	165	1.97	23	0.12	187	-	0.84	0.14	1.05			
												10%	-	378	3.00	33	0.65	230	-	0.31	0.45	3.11			
												5%	-	175	-	44	0.65	237	-	0.35	1.58	5.24			
6	1RP156957	Lot Type Parcel	Regional	CATTLE BREEDING & FATTENING	958,327	200	10	1	10	0.5	20	2%	-	61	-	54	0.58	124	-	0.37	3.64	6.17	- Maturity of current cross-drainage design is limited by available information (e.g. detailed survey) - The Flood Impact Objective exceedance has been identified due to a change in Flood extent and a cell which was dry in the existing case and now flood-affected in the developed case.	- Further cross-drainage optimization investigations are required during detailed design.	
												1%	-	60	-	73	0.78	134	-	0.10	5.70	6.00			
												0.05%	-	-	-	-	-	-	-	-	-	-			
												20%	-	90	1.07	-	-	-	-	0.33	-	-			
												10%	-	182	-	-	-	133	-	0.20	-	0.06			
												5%	-	160	-	-	-	80	-	0.22	-	0.10			
												2%	-	220	-	-	-	84	-	0.24	-	0.12			
7	2RP156957	Lot Type Parcel	Regional	CATTLE BREEDING & FATTENING	752,806	200	10	1	10	0.5	20	1%	-	311	3.21	-	-	62	-	0.27	-	0.06	- Maturity of current cross-drainage design is limited by available information (e.g. detailed survey)	- Further cross-drainage optimization investigations are required during detailed design.	
												0.05%	-	-	-	-	-	-	-	-	-	-			
												20%	-	158	36	-	-	-	-	0.00	0.06	-			-
												10%	-	177	46	4.20	-	-	-	0.00	0.03	-			-
												5%	-	214	40	-	-	-	-	0.01	0.05	-			-
												2%	-	350	19	2.25	-	-	-	4.69	0.02	-			-
												1%	-	901	30	-	-	-	-	0.75	0.07	-			-
8	71MA3479	Lot Type Parcel	Regional	VACANT - LARGE HOUSESITE	138,759	10	5	1	10	0.5	10	0.05%	-	-	-	-	-	-	-	-	-	-	The rail embankment, located upstream in the catchment relative to the Lot, prevents runoff from spreading across the available floodplain. Flow is constricted by the presence of the bridge structure (310-BR05) and culvert C06.23, which leads to propagation of increased water levels downstream. The large maximum exceedances identified generally occur on the floodplain fringes and are attributed to newly flood-affected cells. A review of modelling results indicates that the exceedances occur due to the gridded nature of the hydraulic modelling and the underlying topography where the underlying topography is sloped and the water level in the existing case has not flood-affected the adjacent gridded cell and subsequently does not reach ground level. This causes the existing case water level to stop with the adjacent cell completely dry. In the design case sufficient flood-affecting of the adjacent cell occurs for the water level to meet the adjacent ground level. In reality, this would not occur and the water level would reach the underlying topography. The afflux measurement detailed in the exceedance table is based on the depth change associated with the dry cell in the existing case becoming flood-affected in the design case. Generally increases in water level across this Lot range anywhere from 10 to 12 mm in the 2% and 1% AEP Events. Due to the maturity of the current rail, bridge and cross drainage design being at a reference design stage, these exceedance remain as optimisation of bridge length, abutment placement and culvert spread and dimensions are still to be completed.	To mitigate these exceedances, a review and further optimisation of cross drainage (bridge and culverts) upstream of the Lot will be undertaken including bridge width and abutment placement, possible culvert inclusions, spread and/or number of barrels changed to balance the impacts associated with the presence of the embankment.	
												20%	-	75	0.99	-	0.03	-	-	0.23	0.01	-			
												10%	-	111	0.91	-	0.04	-	-	0.44	0.02	-			
												5%	-	154	0.59	-	0.04	-	-	0.65	0.07	-			
												2%	-	119	1.03	10	0.84	25	-	0.93	0.41	0.15			
												1%	-	173	1.52	12	0.06	46	-	1.19	1.03	0.75			
												0.05%	-	-	-	-	-	-	-	-	-	-			
9	341FTY1881	Lot Type Parcel	Regional	FORESTRY & LOGS	2,964,171	200	10	1	10	0.5	20	20%	-	75	0.99	-	0.03	-	-	0.23	0.01	-	- Maturity of current cross-drainage design is limited by available information (e.g. detailed survey) - Identified property has multiple land use designations, or potentially incorrect land use designations assigned, resulting in a conservative FIO assessment. - The Flood Impact Objective exceedance has been identified due to a change in Flood extent and a cell which was dry in the existing case and now flood-affected in the developed case.	- Further cross-drainage optimization investigations are required during detailed design. - Confirmation of the assigned landuse.	
												10%	-	111	0.91	-	0.04	-	-	0.44	0.02	-			
												5%	-	154	0.59	-	0.04	-	-	0.65	0.07	-			
												2%	-	119	1.03	10	0.84	25	-	0.93	0.41	0.15			
												1%	-	173	1.52	12	0.06	46	-	1.19	1.03	0.75			
												0.05%	-	-	-	-	-	-	-	-	-	-			
												20%	-	75	0.99	-	0.03	-	-	0.23	0.01	-			
10	ASP274206	Lot Type Parcel	Regional	CATTLE BREEDING & FATTENING	2,964,171	200	10	1	10	0.5	20	10%	-	111	0.91	-	0.04	-	-	0.44	0.02	-	- Maturity of current cross-drainage design is limited by available information (e.g. detailed survey) - Identified property has multiple land use designations, or potentially incorrect land use designations assigned, resulting in a conservative FIO assessment. - The Flood Impact Objective exceedance has been identified due to a change in Flood extent and a cell which was dry in the existing case and now flood-affected in the developed case.	- Further cross-drainage optimization investigations are required during detailed design. - Confirmation of the assigned landuse.	
												5%	-	154	0.59	-	0.04	-	-	0.65	0.07	-			
												2%	-	119	1.03	10	0.84	25	-	0.93	0.41	0.15			
												1%	-	173	1.52	12	0.06	46	-	1.19	1.03	0.75			
												0.05%	-	-	-	-	-	-	-	-	-	-			
												20%	-	75	0.99	-	0.03	-	-	0.23	0.01	-			
												10%	-	111	0.91	-	0.04	-	-	0.44	0.02	-			
11	ASP274206	Lot Type Parcel	Regional	CATTLE GRAZING & BREEDING	2,964,171	200	10	1	10	0.5	20	5%	-	154	0.59	-	0.04	-	-	0.65	0.07	-	- Maturity of current cross-drainage design is limited by available information (e.g. detailed survey) - Identified property has multiple land use designations, or potentially incorrect land use designations assigned, resulting in a conservative FIO assessment. - The Flood Impact Objective exceedance has been identified due to a change in Flood extent and a cell which was dry in the existing case and now flood-affected in the developed case.	- Further cross-drainage optimization investigations are required during detailed design. - Confirmation of the assigned landuse.	
												2%	-	119	1.03	10	0.84	25	-	0.93	0.41	0.15			
												1%	-	173	1.52	12	0.06	46	-	1.19	1.03	0.75			
												0.05%	-	-	-	-	-	-	-	-	-	-			
												20%	-	75	0.99	-	0.03	-	-	0.23	0.01	-			
												10%	-	111	0.91	-	0.04	-	-	0.44	0.02	-			
												5%	-	154	0.59	-	0.04	-	-	0.65	0.07	-			
12	25MH070	Lot Type Parcel	Regional	CATTLE GRAZING & BREEDING	479,331	200	10	1	10	0.5	20	10%	-	48	0.54	-	-	-	-	0.11	-	-	- Maturity of current cross-drainage design is limited by available information (e.g. detailed survey) - Maturity of current bridge design is limited by available information (e.g. detailed survey) - The Flood Impact Objective exceedance has been identified due to a change in Flood extent and a cell which was dry in the existing case and now flood-affected in the developed case.	- Further cross-drainage optimization investigations are required during detailed design. - Further road and/or rail design optimization investigations are required during detailed design. - Stakeholder Consultation (Landowner/ Government).	
												5%	-	19	0.09	-	-	-	-	0.02	-	-			
												2%	-	40	0.76	-	-	-	-	0.04	-	-			
												1%	-	172	2.39	-	1.23	-	-	0.85	0.01	-			
												0.05%	-	-	-	-	-	-	-	-	-	-			
												20%	-	46	0.54	-	-	-	-	0.11	-	-			
												10%	-	48	0.54	-	-	-	-	0.10	-	-			
13	91MH79	Lot Type Parcel	Regional	CATTLE BREEDING & FATTENING	1,009,027	200	10	1	10	0.5	20	5%	-	189	2.39	-	-	-	-	0.05	-	-	- Maturity of current cross-drainage design is limited by available information (e.g. detailed survey)	- Further cross-drainage optimization investigations are required during detailed design.	
												2%	-	202	2.33	-	-	-	-	0.12	-	-			
												1%	-	297	3.34	-	-	-	-	0.18	-	-			
												0.05%	-	-	-	-	-	-	-	-	-	-			
												20%	-	33	0.35	-	-	-	-	0.004	-	-			
												10%	-	102	1.47	-	-	-	-	0.03	-	-			
												5%	-	189	2.39	-	-	-	-	0.05	-	-			

ID	ID (Lotplan ID*/FSR ID/ Culvert ID)	Sensitive Receptor Type	Catchment Type	Landuse ¹¹	Lot Area (m ²)	Flood Impact Objective Target						AEP	Maximum FIO Exceedance						Area (ha) ¹				Exceedance Justification	Proposed Mitigation Measures ¹²
						Change in Peak Water Level (mm)	Change in Time of Inundation (%)	Change in Time of Inundation (Hours)	Change in Velocity (%)	Change in Velocity (m/s)	Change in Hazard (%)		Change in Peak Water Level (mm)	Change in Time of Inundation (%)	Change in Time of Inundation (hours)	Change in Velocity (%)	Change in Velocity (m/s)	Change in Hazard (%)	Afflux	Time on Inundation	Velocity	Hazard		
14	87MH16	Lot Type Parcel	Regional	VACANT - LARGE HOUSESITE	1,070,628	-	5	1	10	0.5	10	20%	115	79	2.48	-	-	190	0.14	0.11	-	0.02	Lot 87MH16 is split by the rail alignment with exceedances occurring upstream of Culvert C66.23 and along the fringes of the main flowpath downstream of the alignment. Upstream impacts are due to the alignment preventing runoff discharging towards the main creek channel and being constricted to one discharge location at C66.23 which results in upstream ponding. As per commitments made to the Independent Flood Panel, a review of the representation of the inflow for this western channel will be undertaken in detailed design along with cross drainage optimisation. For downstream impacts, flow is constricted by the presence of the bridge structure (310-BR05), which leads to propagation of increased water levels downstream. The large maximum exceedances identified generally occur on the floodplain fringes and are attributed to newly flood-affected cells. A review of modelling results indicates that the exceedances occur due to the gridded nature of the hydraulic modelling and the underlying topography where the topography is sloped and the water level in the existing case has not flood-affected the adjacent gridded cell and subsequently does not reach ground level. This causes the existing case water level to stop with the adjacent cell completely dry. In the design case sufficient flood-affecting of the adjacent cell occurs for the water level to meet the adjacent ground level. In reality, this would not occur and the water level would reach the underlying topography. The afflux measurement detailed in the exceedance table is based on the depth change associated with the dry cell in the existing case becoming flood-affected in the design case. Due to the maturity of the current rail, bridge and cross drainage design being at a reference design stage, these exceedance remain as optimisation of bridge length, abutment placement and culvert spread and dimensions are still to be completed.	As detailed in the justification column, a review of the inflow representation of the Western Channel will be undertaken in Detailed Design to increase model reliability. In addition, it is proposed to undertake optimisation of cross drainage culvert C66.23 as well as the placement of bridge 310-BR05 abutments. It is anticipated that some residual exceedances will remain on this Lot after optimisation of the design. In the event this is the case, landowner consultation will be undertaken to discuss residual impacts and where appropriate agree site specific mitigation measures.
												10%	35	172	0.65	-	-	50	0.14	0.11	-	0.01		
												5%	159	455	4.78	-	-	563	0.21	0.22	-	0.03		
												2%	553	159	9.08	-	2.08	1575	7.80	0.51	0.004	0.09		
												1%	329	356	6.41	-	1.04	2495	2.60	0.59	0.01	0.05		
												0.05%	840	-	-	-	-	-	0.93	-	-	-		
15	71MA3479	Lot Type Parcel	Regional	VACANT - LARGE HOUSESITE	459,228	10	5	1	10	0.5	10	20%	296	32	-	-	-	0.003	0.05	-	-	The rail embankment, located upstream in the catchment relative to the Lot, prevents runoff from spreading across the available floodplain. Flow is constricted by the presence of the bridge structure (310-BR05) and culvert C66.23, which leads to propagation of increased water levels downstream. The large maximum exceedances identified generally occur on the floodplain fringes and are attributed to newly flood-affected cells. A review of modelling results indicates that the exceedances occur due to the gridded nature of the hydraulic modelling and the underlying topography where the underlying topography is sloped and the water level in the existing case has not flood-affected the adjacent gridded cell and subsequently does not reach ground level. This causes the existing case water level to stop with the adjacent cell completely dry. In the design case sufficient flood-affecting of the adjacent cell occurs for the water level to meet the adjacent ground level. In reality, this would not occur and the water level would reach the underlying topography. The afflux measurement detailed in the exceedance table is based on the depth change associated with the dry cell in the existing case becoming flood-affected in the design case. Generally increases in water level across this Lot range anywhere from 10 to 12 mm in the 2% and 1% AEP Events. Due to the maturity of the current rail, bridge and cross drainage design being at a reference design stage, these exceedance remain as optimisation of bridge length, abutment placement and culvert spread and dimensions are still to be completed.	To mitigate these exceedances, a review and further optimisation of cross drainage (bridge and culverts) upstream of the Lot will be undertaken including bridge width and abutment placement, possible culvert incisions, spread and/or number of barrels changed to balance the impacts associated with the presence of the embankment.	
												10%	210	56	-	-	-	0.01	0.13	-	-			
												5%	199	38	3.44	-	-	-	1.21	0.27	-			-
												2%	200	85	-	-	-	72	0.22	0.40	-			0.003
												1%	-	12	0.17	-	-	-	-	0.01	-			-
												0.05%	-	-	-	-	-	-	-	-	-			-
16	79MH94	Lot Type Parcel	Regional	VACANT - LARGE HOUSESITE	202,540	10	5	1	10	0.5	10	20%	466	29	-	-	-	0.003	0.03	-	-	- Rail design constraints alter the existing flow regime. - the Flood Impact Objective exceedance has been identified due to a change in Flood extent and a cell which was dry in the existing case and now flood-affected in the developed case. - Further cross-drainage optimization investigations are required during detailed design. - Stakeholder Consultation (Landowner/ Government).		
												10%	24	36	-	-	-	0.004	0.02	-	-			
												5%	14	21	-	-	-	0.15	0.01	-	-			
												2%	-	94	-	-	-	-	0.02	-	-			
												1%	-	42	-	25	-	-	0.01	0.003	-			-
												0.05%	-	-	-	-	-	-	-	-	-			-
17	59BNT527	Lot Type Parcel	Regional	VACANT - LARGE HOUSESITE	211,707	10	5	1	10	0.5	10	20%	-	-	-	-	-	-	-	-	-	- Rail design constraints alter the existing flow regime. - the Flood Impact Objective exceedance has been identified due to a change in Flood extent and a cell which was dry in the existing case and now flood-affected in the developed case. - Further cross-drainage optimization investigations are required during detailed design. - Stakeholder Consultation (Landowner/ Government).		
												10%	-	-	-	-	-	-	-	-	-			-
												5%	37	32	-	-	-	-	0.02	0.01	-			-
												2%	155	31	-	-	-	1.52	0.004	-	-			
												1%	-	84	-	71	-	77	-	0.03	0.004			0.003
												0.05%	-	67	-	57	-	56	-	0.004	0.004			0.003
18	0	Watercourse	Regional	0	96,236	-	-	-	10	0.5	-	20%	-	-	-	-	-	-	-	-	-	- Lack of available data for model validation and ongoing model strengthening. - Stakeholder Consultation (Landowner/ Government).		
												10%	-	-	-	-	-	-	-	-	-			-
												5%	-	-	-	-	-	-	-	-	-			-
												2%	-	-	-	-	0.02	-	-	-	0.01			-
												1%	-	-	-	-	0.02	-	-	-	0.01			-
												0.05%	-	-	-	-	-	-	-	-	-			-
19	4RP156020	Lot Type Parcel	Regional	DWELLING - LARGE HOUSESITE	643,134	10	5	1	10	0.5	10	20%	-	-	-	-	-	-	-	-	-	- Rail design constraints alter the existing flow regime. - the Flood Impact Objective exceedance has been identified due to a change in Flood extent and a cell which was dry in the existing case and now flood-affected in the developed case. - Further cross-drainage optimization investigations are required during detailed design. - Stakeholder Consultation (Landowner/ Government).		
												10%	335	40	2.75	-	-	-	0.01	0.02	-			-
												5%	385	26	-	-	-	-	1.43	0.08	-			-
												2%	-	39	-	-	0.08	-	-	0.02	0.003			-
												1%	-	35	-	33	-	-	-	0.004	0.004			-
												0.05%	-	-	-	-	-	-	-	-	-			-
20	0	Watercourse	Regional	0	159,100	-	-	-	10	0.5	-	20%	-	-	-	-	-	-	-	-	-	- Lack of available data for model validation and ongoing model strengthening. - Stakeholder Consultation (Landowner/ Government).		
												10%	-	-	-	-	-	-	-	-	-			-
												5%	-	-	-	-	-	-	-	-	-			-
												2%	-	-	-	-	-	-	-	-	-			-
												1%	-	-	-	-	0.06	-	-	-	0.003			-
												0.05%	-	-	-	-	-	-	-	-	-			-
21	2RP202134	Lot Type Parcel	Regional	DWELLING - LARGE HOUSESITE	181,720	10	5	1	10	0.5	10	20%	-	34	-	-	-	-	0.003	-	-	- Rail design constraints alter the existing flow regime. - the Flood Impact Objective exceedance has been identified due to a change in Flood extent and a cell which was dry in the existing case and now flood-affected in the developed case. - Further cross-drainage optimization investigations are required during detailed design. - Stakeholder Consultation (Landowner/ Government).		
												10%	-	22	0.35	-	-	-	-	0.01	-			-
												5%	707	13	-	-	-	-	0.58	0.01	-			-
												2%	-	37	-	-	-	-	-	0.01	-			-
												1%	-	-	-	-	-	-	-	-	-			-
												0.05%	-	-	-	-	-	-	-	-	-			-
22	221723	Lot Type Parcel	Regional	SMALL CROPS & FODDER - IRRIGATED	69,629	0	10	1	10	0.5	20	20%	-	38	7.06	-	-	-	0.04	-	-	- Rail design constraints alter the existing flow regime. - Further cross-drainage optimization investigations are required during detailed design. - Stakeholder Consultation (Landowner/ Government).		
												10%	-	40	0.38	-	-	-	-	0.02	-			-
												5%	-	56	-	-	-	-	-	0.01	-			-
												2%	-	-	-	-	-	-	-	-	-			-
												1%	-	-	-	-	-	-	-	-	-			-
												0.05%	-	-	-	-	-	-	-	-	-			-
23	58BNT527	Lot Type Parcel	Regional	VACANT - LARGE HOUSESITE	204,486	10	5	1	10	0.5	10	20%	-	280	23	-	-	-	0.37	0.01	-	- Rail design constraints alter the existing flow regime. - the Flood Impact Objective exceedance has been identified due to a change in Flood extent and a cell which was dry in the existing case and now flood-affected in the developed case. - Further cross-drainage optimization investigations are required during detailed design. - Stakeholder Consultation (Landowner/ Government).		
												10%	774	36	6.18	-	-	-	0.01	0.01	-			-
												5%	14	21	-	-	-	-	2.14	0.01	-			-
												2%	-	32	-	-	-	-	-	0.02	-			-
												1%	-	-	-	-	-	-	-	-	-			-
												0.05%	-	-	-	-	-	-	-	-	-			-
24	211723	Lot Type Parcel	Regional	SMALL CROPS & FODDER - IRRIGATED	57,382	200	10	1	10	0.5	20	20%	-	33	0.36	-	-	-	0.01	-	-	- Further cross-drainage optimization investigations are required during detailed design. - Stakeholder Consultation (Landowner/ Government).		
												10%	-	42	0.56	-	-	-	-	0.03	-			-
												5%	-	-	-	-	-	-	-	-	-			-
												2%	-	27	-	-	-	-	-	0.003	-			-
												1%	-	-	-	-	-	-	-	-	-			-
												0.05%	-	-	-	-	-	-	-	-	-			-

ID	ID (Lotplan ID ¹ /FSR ID) Culvert ID	Sensitive Receptor Type	Catchment Type	Landuse ¹¹	Lot Area (m ²)	Flood Impact Objective Target						AEP	Maximum FIO Exceedance						Area (ha) ¹				Exceedance Justification	Proposed Mitigation Measures ¹²
						Change in Peak Water Level (mm)	Change in Time of Inundation (%)	Change in Time of Inundation (Hours)	Change in Velocity (%)	Change in Velocity (m/s)	Change in Hazard (%)		Change in Peak Water Level (mm)	Change in Time of Inundation (%) ²	Change in Time of Inundation (hours)	Change in Velocity (%) ³	Change in Velocity (m/s)	Change in Hazard (%) ³	Afflux	Time on Inundation	Velocity	Hazard		
25	129MH95	Lot Type Parcel	Regional	NOXIOUS/OFFENSIVE INDUSTRY (INCLUDING ABBATOIR)	8,100	10	10	1	10	0.5	20	20%	-	-	-	-	-	-	-	-	-	-	Rail design constraints after the existing flow regime.	- Further cross-drainage optimization investigations are required during detailed design. - Stakeholder Consultation (Landowner/ Government).
												10%	-	30	-	-	-	-	-	0.003	-	-		
												5%	11	-	-	-	-	-	0.01	-	-	-		
												2%	-	-	-	-	-	-	-	-	-	-		
												1%	-	-	-	-	-	-	-	-	-	-		
												0.05%	-	-	-	-	-	-	-	-	-	-		
												20%	217	-	-	-	-	-	0.01	-	-	-		
26	57BNT527	Lot Type Parcel	Regional	VACANT - LARGE HOUSESITE	200,539	10	5	1	10	0.5	10	10%	-	44	0.46	-	-	-	-	0.02	-	-	Rail design constraints after the existing flow regime. - the Flood Impact Objective exceedance has been identified due to a change in Flood extent and a cell which was dry in the existing case and now flood-affected in the developed case.	- Further cross-drainage optimization investigations are required during detailed design. - Stakeholder Consultation (Landowner/ Government).
												5%	452	28	-	-	-	-	1.26	0.04	-	-		
												2%	-	28	-	-	-	-	0.45	-	-	-		
												1%	-	-	0.30	-	-	-	0.003	-	-	-		
												0.05%	-	-	-	-	-	-	-	-	-	-		
												20%	-	-	-	-	-	-	-	-	-	-		
												10%	-	-	-	-	-	-	-	-	-	-		
27	90MH63	Lot Type Parcel	Regional	DWELLING - LARGE HOUSESITE	438,405	10	5	1	10	0.5	10	5%	13	-	-	-	-	-	0.03	-	-	-	Rail design constraints after the existing flow regime.	- Further cross-drainage optimization investigations are required during detailed design. - Stakeholder Consultation (Landowner/ Government).
												2%	-	68	0.85	-	-	-	-	0.02	-	-		
												1%	-	-	-	-	-	-	-	-	-	-		
												0.05%	-	-	-	-	-	-	-	-	-	-		
												20%	-	-	-	-	-	-	-	-	-	-		

Table Notes:

1. Floodplains are dynamic in nature and therefore model results vary from cell to cell. The "Area" is therefore defined as the maximum area over which change above the Flood Impact Objective occurs and does not relate directly to the maximum FIO exceedance recorded.

2. Change in water level (afflux) is calculated based on the maximum water surface level difference between the existing and design case results, and does not consider the underlying topography. Depth values presented are based on the respective underlying topographies and in some cases do not align with afflux calculations. This can be attributed to interpolation calculations of results performed within the hydraulic model.

3. Percentage increases should be read in conjunction with existing and design case results. The FIOs require that change be presented in percentage change for time of inundation, velocity and hazard and can sometimes produce large increases when existing case results are significantly small. Some locations were identified to experience significant percentage increases however the "absolute" difference between Existing and Design results are relatively small.

4. The Flood Impact Objective exceedance has been identified due to a change in flood extent and a cell which was dry in the existing case and now flood-affected in the developed case.

5. The above table should be read in conjunction with change maps provided as part of Volume II and the Digital Platform.

6. The symbol "-" denotes that no change above Flood Impact Objectives has been determined and therefore no result has been presented.

7. Changes recorded above may only occur during a small fraction of model simulation time.

8. Three decimal places do not infer absolute accuracy in modelling results, but it is necessary to identify small changes.

9. Flood impact objectives are assessed based on landuse applied to the entire lot and exceedances identified do not necessarily indicate an impact to a dwelling.

10. Design optimisations that could be implemented (but not limited to) include adjustment of culvert spacing, realignment of culverts to existing flowpaths when more detailed survey is available, modifying the grade of culverts, road height and alignment adjustments, removal of bridge abutments from waterway extents etc

11. Landuse data, dated 2018, has been used for this assessment. Due to the changing nature of this dataset, this will be updated to the latest available information in Detailed Design.

Depth (m) ²		Time of inundation in hours (related to % exceedance)		Time of inundation in hours (related to actual change exceedance)		Velocity in m/s (related to % exceedance)		Velocity in m/s (related to actual change exceedance)		Hazard in m ² /s (related to % exceedance)	
Existing Case	Design Case	Existing	Design	Existing	Design	Existing	Design	Existing	Design	Existing	Design
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	1.76	2.29	-	-	-	-	-	-	-
0.275	0.283	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
-	0.217	-	-	-	-	-	-	-	-	-	-
-	-	1.09	1.57	0.85	1.31	-	-	-	-	-	-
-	0.452	1.02	1.31	-	-	-	-	-	-	-	-
-	-	1.07	1.37	-	-	-	-	-	-	-	-
-	-	-	-	0.90	1.20	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
0.670	0.683	-	-	-	-	-	-	-	-	-	-
-	-	1.04	1.75	0.75	1.60	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-

13.5.3.6 Flow distribution

A key landowner concern is the potential change in flow distribution. To determine the change in flow distribution the maximum velocity x depth product results were used to produce change in unit flow maps between existing and design conditions. Unit flow is the measure of discharge (velocity x depth product) and is often used for assessing peak flow along a cross-section, or to determine the effective flow corridors within a floodplain.

The presented flow distribution maps have been filtered with a velocity x depth product cut-off of 0.15 m²/s. The cut-off is in line with Project FIO requirements and has been used to assist in identifying main channels (high flow zones) and shallow lower flood zones (i.e. outer extents of the flood surface).

The following points summarise the potential change in flow distribution for Pariagara Creek and should be considered in conjunction with flow distribution figures supplied in Figure Q9-1 to Figure Q9-8 of Volume 2 – Appendix Q:

- For all modelled events, the results show increases and decreases in flow distribution upstream and downstream of the Project alignment, respectively
- For events up to the 1% AEP, the majority of these changes occur in lower flow zones (<0.15 m³/s), resulting in a reduced flood risk
- In the rarer events (1 in 2,000 AEP, 1 in 10,000 AEP and PMF events) a reduction in flow distribution can be seen on the downstream side of the Project alignment and positive change on the upstream side. Due to the presence of the Project alignment, flows are conveyed along the alignment to the west through the proposed bridge opening 310-BR05. This results in a positive change in flow distribution in the western portion of the Pariagara Creek floodplain.

Ongoing mitigation measures to reduce changes in flow distribution are likely to occur during future design stages of this Project, and may include further refinement of the bridge abutments, culvert skews, and additional survey where necessary.

13.5.4 Extreme events

13.5.4.1 Extreme event immunity

The risk of overtopping of the Project alignment was assessed for a range of extreme events (1 in 2,000 AEP, 1 in 10,000 AEP and PMF), with Table 13.24 presenting the depth of water above the formation level and over the top of rail at each structure. It is noted that the function of the floodplain culverts is to balance flood levels on the upstream and downstream sides of the alignment.

As such, overtopping of the rail is not predicted to result in significant excessive flows or velocities as would occur in a dam embankment overtopping scenario. Further analysis of the overtopping risk to sensitive receptors can be found in Section 21.

Table 13.24 Pariagara Creek—Extreme events: Depth of water above formation and top of rail levels

Chainage (km)	Structure ID	Depth of water above formation level (m)			Depth of water above top of rail (m)		
		1 in 2,000 AEP	1 in 10,000 AEP	PMF	1 in 2,000 AEP	1 in 10,000 AEP	PMF
67.55	310-BR05	-	-	1.19	-	-	0.49
66.23	C66.23	-	-	0.25	-	-	-
67.57	C67.57	-	-	0.69	-	-	-
67.64	C67.64	-	-	0.60	-	-	-
67.70	C67.70	-	-	0.81	-	-	0.11
67.83	C67.83	-	-	0.92	-	-	0.22

Chainage (km)	Structure ID	Depth of water above formation level (m)			Depth of water above top of rail (m)		
		1 in 2,000 AEP	1 in 10,000 AEP	PMF	1 in 2,000 AEP	1 in 10,000 AEP	PMF
67.96	C67.96	-	-	0.97	-	-	0.27
68.09	C68.09	-	-	0.79	-	-	0.09
68.41	C68.41	-	-	0.73	-	-	0.03
68.76	C68.76	-	-	0.84	-	-	0.14
68.89	C68.89	-	-	0.84	-	-	0.14
69.02	C69.02	-	-	0.99	-	-	0.29
69.10	C69.10	-	-	0.87	-	-	0.17
69.14	C69.14	-	-	1.00	-	-	0.30
69.21	C69.21	-	-	0.92	-	-	0.22
69.28	C69.28	-	-	0.99	-	-	0.29

13.5.4.2 Impacts during extreme events

Flood immunity of FSRs is discussed in Section 13.5.3.1, and maps demonstrating the impacts during extreme events can be found in the [Online Digital Portal](#).

In accordance with the FIOs, FSRs are considered impacted by extreme flood events if the change in water level is greater than 250 mm in the 1 in 2,000 AEP event, subject to the receptor classification (i.e. at existing habitable dwellings and/or commercial and industrial buildings/premises). In addition, a stricter afflux objective of 10 mm in the 1 in 500 AEP event applies to sites with existing sensitive infrastructure, uses involving vulnerable people and/or any critical road network that was designed to be immune to flooding in any extreme event.

Review of the 1 in 2,000 AEP event results for the Pariagara Creek floodplain against the FIOs show that flood level changes at all FSRs were limited to 250 mm, and no identified existing sensitive infrastructure or uses involving vulnerable people are expected to experience a more than 10 mm increase in flood level. A separate assessment of the 1 in 500 AEP event is not required as they are conservatively represented by the 1 in 2,000 AEP event results. The flood impacts in extreme events due to the Project are therefore compliant against the stipulated FIO limits for extreme events.

Table 13.25 details extreme event results, although it should be noted that FIO targets do not apply to the 1 in 10,000 AEP and PMF events. Maps demonstrating the impacts during extreme events are shown in the [Online Digital Portal](#).

Table 13.25 Pariagara Creek—Summary of extreme event impacts at flood sensitive receptors

FSR ID	FSR type	1 in 2,000 AEP event		1 in 10,000 AEP event		PMF event	
		Change in peak water level (mm)	Existing Case flood depth (m)	Change in peak water level (mm)	Existing Case flood depth (m)	Change in peak water level (mm)	Existing Case flood depth (m)
PAR_ID_1	Shed	-	-	-	-	+2	1.12
PAR_ID_2	Shed	-	-	-	-	+1	1.09
PAR_ID_5	Shed	-	-	-	-	+1	1.10
PAR_ID_6	Shed	-	-	-	-	+1	1.22
PAR_ID_7	Shed	-	-	-	-	+1	1.15
PAR_ID_8	Shed	-	-	-	-	+2	0.94
PAR_ID_9	Shed	-	-	-	-	+1	0.18
PAR_ID_10	Shed	-	-	-	-	+2	0.95
PAR_ID_11	Shed	-	-	-	-	+2	0.96

FSR ID	FSR type	1 in 2,000 AEP event		1 in 10,000 AEP event		PMF event	
		Change in peak water level (mm)	Existing Case flood depth (m)	Change in peak water level (mm)	Existing Case flood depth (m)	Change in peak water level (mm)	Existing Case flood depth (m)
PAR_ID_12	Shed	-	-	-	-	+2	0.96
PAR_ID_13	Shed	-	-	-	-	+2	0.98
PAR_ID_14	Shed	-	-	-	-	+3	1.11
PAR_ID_15	Shed	-	-	-	-	+4	1.38
PAR_ID_16	Shed	-	-	-	-	+5	1.37
PAR_ID_17	Shed	-	-	-	-	+5	0.64
PAR_ID_18	Shed	-	-	-	-	+7	1.39
PAR_ID_19	Shed	-	-	-	-	+6	1.76
PAR_ID_20	Shed	-	-	-	-	+7	1.40
PAR_ID_21	Shed	-	-	+9	0.37	+5	2.03
PAR_ID_22	Shed	-	-	+8	0.25	+5	1.87
PAR_ID_23	Shed	-	-	+9	0.38	+5	2.00
PAR_ID_24	Shed	-	-	-	-	-4	1.40
PAR_ID_25	House	-	-	-	-	+3	1.15
PAR_ID_26	Shed	-	-	-	-	+3	1.22
PAR_ID_27	Shed	-	-	-	-	-3	0.51
PAR_ID_28	House	-	-	-	-	-5	0.51
PAR_ID_30	Shed	-	-	-	-	-5	0.13
PAR_ID_31	Shed	-	-	-	-	-8	1.42
PAR_ID_32	Shed	-	-	-	-	-8	1.35
PAR_ID_33	Shed	-	-	-	-	-4	1.33
PAR_ID_35	House	-	-	-	-	-22	0.43
PAR_ID_36	Shed	-	-	-	-	+3	1.24
PAR_ID_38	Shed	-	-	-	-	0	1.15
PAR_ID_39	Shed	-	-	-	-	+4	1.52
PAR_ID_40	Shed	-	-	+8	0.34	+5	1.97
PAR_ID_41	House	-	-	-	-	-5	1.29

13.5.5 Sensitivity analysis—Pariagara Creek

The sensitivity of the model to various parameters was assessed using the following two scenarios:

- An increase in rainfall intensity (i.e. to reflect climate change scenario)
- An increase and decrease in blockage of culverts from 25 to 50 per cent and 0 per cent.

13.5.5.1 Climate change

The potential impacts of climate change in the Pariagara Creek floodplain were assessed for the 1% AEP design event to determine the sensitivity of the Project to the potential long-term changes in climate. The assessment was undertaken in accordance with ARR 2019.

The Representative Concentration Pathways 8.5 (2090 horizon) climate change scenario has been adopted for the Project with an associated increase in rainfall intensity of 23.9 per cent across the catchment area.

The Project is expected to retain a 1% AEP flood immunity to formation level under the climate change scenario with over 1 m of freeboard to formation level.

Due to no FSRs being present with the hydraulic model domain, Table 13.26 presents the structures performance with Representative Concentration Pathways 8.5 climate change conditions. For the 1% AEP event, the change in peak water levels for the Representative Concentration Pathways 8.5 climate change scenario is presented in Figure I8-C in Volume 2 – Appendix I.

Table 13.26 Pariagara Creek—1% AEP Event Climate Change Assessment

Structure ID	Structure type	1% AEP peak water level (mAHD)	1% AEP +CC peak water level (mAHD)	Difference in peak water level (m)
310-BR05	Bridge	285.26	285.50	+0.24
C66.23	RCBC	285.35	285.61	+0.26
C67.57	RCP	285.34	285.58	+0.23
C67.64	RCP	285.45	285.67	+0.23
C67.70	RCP	285.49	285.73	+0.24
C67.83	RCP	285.6	285.83	+0.23
C67.96	RCP	285.68	285.93	+0.24
C68.09	RCP	285.78	286.04	+0.26
C68.41	RCP	286.41	286.59	+0.18
C68.76	RCP	286.99	287.12	+0.13
C68.89	RCP	287.11	287.28	+0.17
C69.02	RCP	287.25	287.44	+0.19
C69.10	RCP	287.29	287.48	+0.20
C69.14	RCP	287.31	287.51	+0.20
C69.21	RCP	287.33	287.54	+0.21
C69.28	RCP	287.34	287.55	+0.21
C69.41	RCP	287.35	287.57	+0.22
C69.54	RCP	287.36	287.58	+0.22
C69.67	RCP	287.38	287.60	+0.23
C69.80	RCP	287.38	287.61	+0.23

13.5.5.2 Blockage

Blockage was assessed in accordance with ARR 2019. The blockage assessment was undertaken and resulted in a blockage factor of 25 per cent adopted for culverts. A minimum culvert size of 900 mm diameter was adopted to reduce potential for blockage and maintenance. A significant community concern is the potential impacts to flood conditions should the proposed culverts become blocked with debris. The primary concern is the blockage of culverts, which is likely to drive flood levels higher, particularly upstream of the culverts, and divert more flow through residences, across access roads and other infrastructure. A sensitivity analysis was undertaken with 0 and 50 per cent blockage on culverts.

Results of the blockage, 0 and 50 per cent, sensitivity analysis are shown in Figure I8-A and I8-B in Volume 2 – Appendix I, respectively. There is little difference between results with zero blockage and baseline blockage, but peak water levels increase with 50 per cent blockage.

Due to no FSRs being present with the hydraulic model domain, a sensitivity analysis was only completed on structures performance. Table 13.27 provides a summary of 1% AEP peak flood levels at cross-drainage structures for the blockage scenarios.

During detailed design the blockage factors will be reviewed in line with the final design and local catchment conditions. This may result in a varied and/or lower blockage factors being applied along the Project alignment. Should impacts be determined due to a change in blockage factor, additional mitigation measures will be considered in detailed design.

Table 13.27 Pariagara Creek—1% AEP Event: Culvert blockage assessment

Structure ID	Structure type	1% AEP Peak water levels (mAHD)			Increase from Developed Case to 50% blockage scenario (mm)
		0% blockage	Developed Case (25% blockage)	50% blockage	
C66.23	RCBC	285.08	285.35	285.92	+0.57
C67.57	RCP	285.34	285.34	285.35	+0.00
C67.64	RCP	285.44	285.45	285.45	+0.00
C67.70	RCP	285.49	285.49	285.5	+0.01
C67.83	RCP	285.59	285.6	285.61	+0.01
C67.96	RCP	285.67	285.68	285.7	+0.02
C68.09	RCP	285.77	285.78	285.8	+0.02
C68.41	RCP	286.4	286.41	286.42	+0.01
C68.76	RCP	286.99	286.99	287.00	+0.01
C68.89	RCP	287.1	287.11	287.13	+0.02
C69.02	RCP	287.23	287.25	287.27	+0.02
C69.10	RCP	287.27	287.29	287.31	+0.03
C69.14	RCP	287.3	287.31	287.34	+0.03
C69.21	RCP	287.32	287.33	287.36	+0.03
C69.28	RCP	287.33	287.34	287.37	+0.03
C69.41	RCP	287.33	287.35	287.38	+0.03
C69.54	RCP	287.35	287.36	287.4	+0.03
C69.67	RCP	287.36	287.38	287.41	+0.03
C69.80	RCP	287.37	287.38	287.42	+0.03

13.6 Additional modelling items to be considered in detailed design

The following points summarise additional items to be included in the flood models for detailed design as per agreement with the Expert Flood Panel:

- Additional 'as-built' information will be sought for existing infrastructure, where available, to accurately represent the infrastructure within the modelling domain. Existing bridges will be modelled using Ifcsh
- A review of bridge losses will be undertaken to verify against alternative methods
- A review of the western channel inflow application will be undertaken to better represent the conveyance in this area
- Inflow application within the hydraulic model is to be reviewed
- Tailwater application to be reviewed to account for the effects of the Macintyre Brook

- A review of the latest TUFLOW version will be undertaken at the commencement of detailed design to determine whether updates of the TUFLOW version from 2020-10-AD-iSP-w64 are beneficial to modelling
- Undertake additional model strengthening by improving the representation of cross-drainage infrastructure, survey data, and bridge structures within the hydraulic model
- In order to meet FIOs, additional analysis of cross-drainage infrastructure may be required, along with potential stakeholder consultation.

Further details of mitigation measures can be found in Section 22. A review of proposed mitigation measures will be completed during the detailed design stage of the Project after hydraulic models have been updated to capture the above changes.

13.7 Planned and future roads

Consultation with the DTMR has not identified any planned or future road upgrades within the Pariagara Creek catchment. ARTC will continue to consult and engage with DTMR to determine any future or planned road projects that may need to be considered as part of the detailed design hydraulic modelling.

Consultation with GRC has not identified any planned or future road upgrades within the Pariagara Creek catchment. ARTC will continue to consult and engage with GRC to determine any future or planned road projects that may need to be considered as part of the detailed design hydraulic modelling.

14 Macintyre Brook—Yelarbon to Inglewood

Macintyre Brook Study area runs in a westerly direction through Inglewood and to the south of Yelarbon. The Macintyre Brook is fed by several creek systems as it flows from the east of Inglewood westwards towards Yelarbon, including Mosquito Creek and Canning Creek. Coolmunda Dam is situated on Macintyre Brook upstream of Inglewood. Kippenbung Creek runs from east to west along the southern side of Yelarbon flowing into the Dumaresq River approximately 24 km downstream of the Macintyre Brook confluence with the Dumaresq River. Brigalow Creek, a tributary of the Weir River, runs from east to west to the north of Yelarbon. The hydraulic model for this assessment covers from the area immediately east of Inglewood to downstream of Yelarbon. Key locations covered in the impact assessment include:

- Inglewood
- Whetstone
- Millmerran-Inglewood Road
- Yelarbon.

The location of the Project alignment in relation to Macintyre Brook at Yelarbon to Inglewood is shown in Figure J1-A in Volume 2 – Appendix J.

14.1 Data collection and review: Macintyre Brook—Yelarbon to Inglewood

Available background information including existing hydrologic and hydraulic models, survey, streamflow data, available calibration information and anecdotal flood data has been gathered. Data was sourced from a wide range of stakeholders including:

- GRC—existing flood studies
- BoM—rainfall gauging data
- DTMR—existing infrastructure details
- DRDMW—stream gauging data
- Queensland Government—Flood Mapping Program
- QR—existing rail infrastructure details.

14.1.1 Previous studies

Several previous hydrology and hydraulic studies were sourced as part of this assessment. A review of each study was undertaken to determine suitability for use on the Project as documented in the following sections.

There are a number of key studies that cover the Macintyre Brook system that were considered in this assessment. These include:

- Flood hazard mapping – Yelarbon (Bundle 8) (SKM, 2013b)
- *Inglewood Flood Study* (Engeny, 2015)
- *Melbourne-Brisbane Inland Rail – 2017 Phase 2 Preparatory Alignment Assessment Report North Star to Yelarbon* (WSP, 2017)
- *Floodplain Management Plan for the Borders River Valley Floodplain 2020* (BRVFMP) (NSW Department of Planning and Environment (DPE), 2020)
- North Star to NSW/QLD Border EIS – *Hydrology and Flooding Technical Report* (FFJV, 2020)
- North Star to NSW/QLD Border: *Preferred Infrastructure Report – Hydrology and Flooding* (PIR) (FFJV, 2021).

14.1.1.1 Flood hazard mapping—Yelarbon (Bundle 8), SKM, 2013b

A flood hazard study for Yelarbon was undertaken in 2013 as part of the Queensland Reconstruction Authority Queensland Flood Mapping Program. The assessment considered the impact of local catchment flooding.

The study included development of a TUFLOW model, simulation of the 1956 flood event, validation of the model and design assessment of the 2%, 1% and 0.2% AEP events. Flood extent, flood hazard, depth and velocity were presented for the three events.

A flood frequency assessment was utilised to predict flows for Kippenbung Creek. The assessment and modelling did not include Macintyre Brook flood flows.

The model predicted that during a 1% AEP event, flood depths between 0.8 and 2.0 m occurred in the town centre. Velocities of up to 2 m/s were experienced in the town centre, with velocities of 3.9 m/s along the Cunningham Highway just outside of the town.

As this model only considered local catchments (not Macintyre Brook), it was not considered further for use in this assessment. The assessment has utilised for comparative purposes only.

14.1.1.2 Inglewood Flood Study, Engeny, 2015

In 2015, Engeny was commissioned by GRC to undertake the *Inglewood Flood Study*. The study objectives were *‘to define the nature, extent and risks of flooding in Inglewood in order to inform disaster management planning and response, as well as control future development’* (Engeny, 2015).

An URBS hydrologic model and TUFLOW (1D/2D) hydraulic model were developed and an assessment of the following undertaken:

- 1976 historical event for model validation
- 10%, 5%, 1% and 0.5% AEP events
- Sensitivity analysis for Coolmunda Dam storage capacity.

Findings from the study were:

- While Coolmunda Dam does not serve as a flood mitigation measure for Inglewood, the availability of storage within Coolmunda Dam does have a significant influence on flood events up to the 1% AEP event
- For flood events greater than 10% AEP, the obstruction caused by the existing railway line causes flows in excess of bankfull capacity to be diverted towards Brook Street in a westerly direction
- The Macintyre Brook and Canning Creek channel banks have a 20% AEP to 10% AEP capacity
- The flood hazard within the Macintyre Brook and Canning Creek channel banks is classified at Extreme whilst the broader floodplain including the Inglewood Township is mostly classified as Significant
- The Inglewood Hospital is estimated to have a flood immunity of approximately 0.5% AEP
- The main evacuation route for Inglewood is via the Cunningham Highway Bridge in an easterly direction
- The Cunningham Highway Bridge is predicted to become flooded and closed after 12 hours and 8.5 hours in the 10% and 1% AEP events, respectively
- The closure duration for the Cunningham Highway Bridge is approximately 8.5 hours and 26 hours in the 10% and 1% AEP events, respectively.

14.1.1.3 Melbourne-Brisbane Inland Rail – 2017 Phase 2 Preparatory Alignment Assessment Report North Star to Yelarbon, WSP, 2017

The Phase 2 Preparatory Alignment Report documents the assessment of changes to the alignment, with updates to the models documented in North Star to NSW/QLD Border, Hydrologic and Hydraulic Modelling – Illabo to Stockinbingal and North Star to Yelarbon (SMEC, 2016a).

14.1.1.4 Floodplain Management Plan for the Borders River Valley Floodplain 2020, NSW DPE, 2020

The Borders River Valley Floodplain management plan provides a framework for coordinating and assessing development works on a whole of valley basis.

As part of the BRVFMP hydrologic and hydraulic models (URBS, RAFTS and TUFLOW) were established for the assessment of development impacts on flood characteristics within the floodplain. The hydrology uses previously established models from the *Border Rivers Floodplain Hydraulic Analysis* (Lawson and Treloar 1998). The URBS models were originally developed by the BoM for the Weir River and Macintyre Brook. The hydrologic models were not modified for the Floodplain Management Plan, 2018. Details of the Lawson and Treloar (1998) models are provided in Appendix 6 of the Floodplain Management Plan.

The NSW DPE TUFLOW hydraulic model uses current conditions including existing and approved development in floodplain, with small (1996 flood event) and large (1976 flood event) historical rainfall events to assess flood conditions and development impacts. Under the plan, development in the floodplain requires assessment using the NSW DPE hydraulic model to determine if the development meets nominated criteria in terms of changes to flood characteristics (i.e. changes in peak flood levels, changes in flowpaths, flow rates and velocities).

A TUFLOW GPU hydraulic model was developed by NSW DPE for the BRVFMP. The TUFLOW model covers an area of approximately 1.1 million ha extending from approximately 50 km upstream of Boggabilla to 40 km downstream of Mungindi. The main watercourses within the model are the Macintyre River, Weir River, Boomi River and Barwon River.

The topography in the TUFLOW model is defined using a high-resolution DEM. The DEM was created from a variety of LiDAR datasets including Macintyre 2013 and Gwydir 2013 datasets and supplemented to the north with Queensland LiDAR datasets. LiDAR was available for the majority of the model area. Where data was not available, SRTM1-second (approximately 30 m) resolution elevation data was used.

The TUFLOW model grid size is 30 m. Topography modifiers were incorporated into the model to ensure that topographic features such as roads, rail and levee banks are correctly represented. There are no drainage structures included in the TUFLOW model (culverts/bridges).

The NSW DPE hydraulic model was calibrated to the 1996 historical event and verified with the 1976 historical event (noting that the topographic conditions were difficult to replicate for the 1976 conditions). For the 1976 event topographic features (roads, rail, farm levees, farm channels etc.) known not to be in place in 1976 were removed from the 1976 calibration hydraulic model.

DPE is the custodian of the models and provided the models to ARTC for review and use.

14.1.1.5 North Star to NSW/QLD Border EIS – Hydrology and Flooding Technical Report (FFJV, 2020)

For the EIS assessment, NSW DPE provided the following existing information developed for the *Border Rivers Valley Floodplain Management Plan* (NSW DPE, 2020):

- Existing 1976 and 1996 URBS hydrologic models for the Macintyre River, Dumaresq River and Macintyre Brook catchments
- Existing 1976 and 1996 RAFTS hydrologic models for Ottleys Creek catchment
- Existing 1976 and 1996 TUFLOW hydraulic models for the Border Rivers Valley floodplain.

For the EIS, hydrologic and hydraulic modelling was undertaken using the provided information with the following amendments:

- Creation of a TUFLOW hydraulic sub-model based on the provided BRVFMP hydraulic model which covers an area of approximately 1.1 million ha extending from approximately 50 km upstream of Boggabilla to 40 km downstream of Mungindi. The TUFLOW hydraulic sub-model has the same upstream boundary and extends to 18 km downstream of Goondiwindi

- Inclusion of an additional historical, more recent, calibration event being the January 2011 event
- Sourcing and inclusion of current LiDAR survey data across the floodplain to obtain details of current ground levels and levee heights/locations to support assessment of project. This information was included in the hydraulic sub-model based on community feedback that the BRVFMP levee layer was not representative of current topographic conditions.
- Inclusion of Goondiwindi town levee heights based on information provided by GRC to replace the unlimited heights used in the BRVFMP model.

The performance of the hydraulic sub-model was validated against a range of information collected for the three historical flood events (1976, 1996 and 2011) including:

- Stream gauge level and flow hydrographs
- Flood markers
- Flood event photography from landholders
- Aerial flood photography
- Landowner and community observations of the flood events.

This base modelling was used to develop the revised reference design and assess the impacts on peak water levels, flood flow distribution, velocities and duration of inundation. The impacts of the refined design were documented in the NS2B EIS Chapter 13: Surface Water and Hydrology and associated Appendix H: Hydrology and Flooding Technical Report.

The impact assessment was carried out primarily against the 1% AEP event with the levees based on the 2019 LiDAR data. In addition, modelling of the 1% AEP event with the BRVFMP levees and the 1976 flows with levees based on the 2019 LiDAR data was undertaken and documented in the EIS.

14.1.1.6 North Star to NSW/QLD Border: Preferred Infrastructure Report – Hydrology and Flooding (FFJV, 2021)

The hydrologic and hydraulic modelling undertaken for the EIS is based on the existing BRVFMP hydrologic and hydraulic modelling. An Independent Peer Review was undertaken for GRC which identified that there were a number of potential limitations with the modelling including that it was not reflective of current best practice, including requirements of the recently updated ARR 2019.

The methodology followed for the updated hydrologic and hydraulic modelling to address PIR scope and the issues raised in the GRC Independent Peer Review included:

- Compliance to ARR 2019 in relation to hydrologic and hydraulic modelling guidelines for major infrastructure.
- Extension of the existing URBS hydrologic models to form one linked hydrologic model that includes:
 - The four major upstream catchments to within the hydraulic model boundary—Macintyre River, Macintyre Brook, Dumaresq River and Ottleys Creek
 - The wider floodplain area down to Goondiwindi
 - The southern tributaries catchments—Mobbindry Creek, Back Creek, Strayleaves Creek and Forest Creek
 - The Brigalow Creek catchment.
- Review and update of available rainfall and streamflow data used to represent the three historical calibration events (1976, 1996 and 2011)
- Refinement of major upstream catchments URBS models to use common modelling parameters for each catchment (apart from loss rates and dam details relevant to date of historical flood event)
- Refinement of southern tributaries URBS modelling

- Update of the TUFLOW hydraulic model used for the EIS to include the following:
 - Revised flows from the updated URBS hydrologic model with no factoring of flows for historical events
 - Refinement of upstream inflow locations into the hydraulic model
 - Review and update of existing drainage structures under the non-operational Camurra-Boggabilla rail line, Bruxner Way and North Star Road using available information.
- Joint calibration of the hydrologic model and hydraulic sub-model including:
 - Validation against the available all discoverable recorded stream gauge, recorded flood markers and anecdotal data for the 1976, 1996 and 2011 historical flood events
 - Comparison against the BRVFMP calibration outcomes.
- 1% AEP design event modelling and reconciliation to FFA of key stream gauges in the upper catchments and at Boggabilla
- Modelling of the Existing Case and Developed Case (including the Project's revised reference design) for the 1976 flows scenario for the following cases, being:
 - Verified 2019 levees and validated 1976 flows
 - BRVFMP levees and validated 1976 flows
 - BRVFMP levees and factored flows
 - Verified 2019 levees and 1% AEP flows
 - BRVFMP levees and 1% AEP flows.
- Determination of critical event for crossing locations (i.e. either 1976 flow scenario or 1% AEP flows)
- Assessment of the performance of the revised reference design against the Floodplain Management Objectives.

The PIR hydrologic and hydraulic modelling has been adopted for this assessment as the basis of impact assessment for the Project alignment. Full details of the updates are provided in the PIR and detailed in Section 17.

Adoption of models

To be consistent with the latest modelling for NS2B and the requirements of the review process, the NS2B PIR hydrologic modelling has been adopted for the Macintyre Brook impact assessment. The hydrologic and hydraulic modelling methodology is outlined in further sections and covered in detail in the Macintyre River technical chapter (Section 17) and the PIR – Hydrology and Flooding (FFJV, 2021). As the NS2B hydraulic modelling did not include Macintyre Brook, a TUFLOW model of this area has been developed (Section 14.3). The *Inglewood Flood Study* (Engeny, 2015) model was utilised for incorporating details around Inglewood.

14.1.2 Survey

ARTC provided LiDAR data from 2015 as 1 m grid DEM tiles over the Macintyre Brook – Yelarbon to Inglewood Study area. Using GIS software, a DEM was generated with a 1 m grid resolution for use for the Project. The DEM was based on the 2015 dataset.

Additional LiDAR data extents were required to appropriately model downstream boundary conditions and facilitate calibration against stream flow gauges. In areas that were not covered by the LiDAR provided by ARTC, LiDAR tiles were sourced from Geoscience Australia. The DEM datasets utilised for modelling were based on surveys flown between 2009 and 2015.

SRTM data was used for catchment delineation where no LiDAR data could be sourced, to inform the hydrologic modelling.

The survey data sources, and DEM developed for Macintyre Brook is shown in Figure J1-B in Volume 2 – Appendix J.

14.1.3 Aerial imagery

Aerial imagery of the Macintyre Brook – Yelarbon to Inglewood study area captured in 2015 was provided by ARTC and was used to identify and confirm topographic and vegetative characteristics of the study area. Additional imagery outside the study area was sourced from QGIS in an open-source format.

14.1.4 Reservoirs

One large reservoir is located in the upstream catchment and has been included in the hydrologic model being Coolmunda Dam on Macintyre Brook upstream of Inglewood. Coolmunda Dam was constructed between 1969 to 1972. Reservoir details were obtained from previous studies and Sunwater. Further details are provided in Section 17.2.5.4.

14.1.5 Existing drainage structure data

Drainage structure geometry information was obtained from the following sources:

- Previous studies
- Site inspection
- Topographic survey (Section 14.1.2)
- QR.

Structure geometry information contained within the previous hydraulic models was used in this assessment. Queensland Rail as-constructed drawings were also sourced for culvert sizes along the rail where no other information was available. Existing floodplain infrastructure includes:

- Bybera Road Bridge
- Cunningham Highway
- Millmerran-Inglewood Road
- Local roads
- Existing QR rail line
- Levees and dams associated with farming practices.

The Bybera Road Bridge crosses the Macintyre Brook approximately 5 km downstream of Inglewood. The Bybera Road Bridge is approximately 30 m long and has less than 10% AEP flood immunity (Engeny, 2015). A 40 km stretch of the Cunningham Highway runs between Inglewood and Yelarbon and is a low-level road that includes two bridges and minor drainage structures. One of these bridges is the Cunningham Highway Bridge at Inglewood. The *Inglewood Flood Study* (Engeny, 2015) predicts this bridge has an immunity less than a 2% AEP event and reports that this bridge is a main evacuation route.

Millmerran-Inglewood Road crosses Canning Creek approximately 4 km northeast of Inglewood on a bridge structure. This bridge is 60 m long and has a flood immunity above the 1% AEP event (Engeny, 2015).

Key features were incorporated into the model topography, including the existing QR rail line and the Cunningham Highway to the east of Yelarbon.

Ground survey of the existing QR rail line was not available for this assessment. Determination of the formation level of the existing rail was through inspection of the available topographic data (LiDAR). This includes the existing rail line through Yelarbon.

14.1.6 Stream gauge data

Stream gauges are used to provide a record of observed stream levels in the study area. These were originally manually recorded staff levels (typically recorded on a daily basis with more frequent records during flood events) with modern gauges providing a continuous automated record.

Although levels may be adequate for flood warning services, hydrologic investigations are usually more interested in streamflow. A rating curve is required to convert recorded levels into an equivalent stream flow. The most reliable source of data for deriving a rating curve are actual instream flow measurements taken during flood events. These are often difficult/dangerous to obtain during major flood events unless the gauge site is located near an appropriate structure spanning the waterway (e.g. a high-level bridge), and so are often only available for low to moderate flows. The rating must therefore be extrapolated to higher flows. This is often based on simple power-law best fit through the available data; however, ideally, the extrapolation is based on more reliable means, such as a hydraulic model calibrated to the reliable part of the rating curve.

Other factors can also influence the short- and long-term reliability of the rating curve. Changes to channel bed or roughness, either long-term or during a flood event, can change the hydraulic properties and hence the rating curve. Gauges are preferably located at a hydraulic control, either natural or artificial, (e.g. a weir), or where the bed material has low erodibility. The gauge location may also not produce a singular relationship between flow and level. This may occur in areas where there is significant floodplain storage, and hence the level is dependent on the duration and rate of change of the flow, or the gauge location may be affected by backwater from a downstream tributary.

There are seven gauges within the Macintyre Brook catchment that were selected for the hydrologic assessment. The locations of the adopted gauges are presented in Figure J1-C in Volume 2 – Appendix J and the gauge details are outlined in Table 14.1. Of these, there are two gauges within the Macintyre Brook hydraulic model area with suitable flow and level data for the hydraulic assessment, being Inglewood and Booba Sands.

Table 14.1 Macintyre Brook catchment—Summary of key stream gauges used in the model calibration

Gauge name	Gauge No.	Area (km ²)	Description
Macintyre Brook @ Barongarook	DRDMW 416410A	114	Small catchment upstream of Coolmunda Dam. Relatively reliable rating. Site ceased in 2002.
Bracker Creek @ Terraine	DRDMW 416404C	685	Small catchment upstream of Coolmunda Dam. Relatively reliable rating. Site ceased operation in 2002.
Canning Creek @ Woodspring	DRDMW 415404A	1,245	Rating appears reasonable but based on limited flow gauging. Site ceased operation in 1980.
Macintyre Brook @ Coolmunda T/W	DRDMW 416416A	1,760	Immediately downstream of Coolmunda Dam. Unknown rating quality. Site ceased operation in 2002. Coolmunda Dam commissioned 1968
Macintyre Brook @ Inglewood CBM	Unknown	~1,800	Upstream of the confluence of Macintyre Brook and Canning Creek. No information about this gauge is currently available.
Macintyre Brook @ Inglewood	DRDMW 416402B/C	3,430	Downstream of the confluence of Macintyre Brook and Canning Creek. Site changed from 'B' to 'C' in 1981, keeping the same location but a modification of the control weir and rating.
Macintyre Brook @ Ben Dor Weir	DRDMW 416406A	4,000	Operational between 1954 and 1988. Highest gauged flow is 41.9 m ³ /s. Significant uncertainty in the high flow rating and inconsistent with Inglewood.
Macintyre Brook @ Booba Sands	DRDMW 416415A	4,092	Relatively reliable rating but site has only been operational since 1987

14.1.7 Rainfall data

Rainfall data was sourced from the BoM as well as WaterNSW and DoR. Detailed pluviograph data (rainfall distribution with a timestep less than 1 hour) is typically available at a relatively limited number of gauges across the catchment. This data is supplemented by daily (24-hour) rainfall data.

The number of rainfall gauges used for each calibration event is summarised in Table 14.2, noting that many of these gauges are located outside the Macintyre Brook catchment boundaries (including Macintyre River and Dumaresq River and surrounds). The gauges are also not located uniformly across the catchment, and some areas of the catchment have significantly higher detail in terms of both spatial and temporal distribution of the rainfall. Rainfall gauge locations available for the three calibration events are shown in Figure C2 to C4 in Appendix C, together with the total rainfall depth of the event.

The gauge pluviograph data for each event was reviewed to identify suspect or missing data and also for consistency with nearby gauges. A number of cases were identified where several days of rainfall were accumulated in a single record or where a gauge had the similar depths to surrounding gauges but offset by a day, most likely the result of errors in the transposition of manual record data.

Table 14.2 Macintyre Brook catchment—Number of rainfall gauges used in the model calibration

Event	Detailed pluviograph gauges			BoM Daily Gauges
	BoM	DoR (Qld)	WaterNSW	
February 1976	15	-	-	202
January 1996	18	4	2	165
January 2011	16	6	23	162

14.1.8 Anecdotal and observed flood data

Anecdotal flood data for the historical flood events was collected from many sources including:

- Previous studies
- Landowners and stakeholders, including GRC.

Anecdotal data includes information obtained from a wide range of sources and, as such, it is of varying levels of accuracy and reliability. The anecdotal data has been used to assess the performance of the hydraulic model to replicate historical flood conditions.

14.1.9 Site inspection

A site inspection was undertaken on 17 October 2018. During the site inspection, all major waterway crossings were visited and inspected, with photographs taken and details recorded of the crossing, existing drainage structures and surrounding catchment. An assessment of the relative roughness and blockage potential was undertaken during the site inspection.

Additional inspections have been undertaken associated with stakeholder meetings. Footage of the late 2021 flood event was also collected by helicopter.

14.2 Hydrologic model development: Macintyre Brook—Yelarbon to Inglewood

The hydrologic modelling for the Macintyre Brook catchment was included in the Macintyre River hydrologic modelling completed to support the PIR for the NS2B project. The background to the hydrologic and hydraulic modelling, including calibration and design event modelling, for this work is detailed in Section 17.2.

In addition to the model calibration documented in Section 17.2, the December 2021 flood event was modelled as a validation event for the Macintyre Brook hydrologic and hydraulic models. This validation event is documented in the following sections.

14.2.1 Hydrologic modelling

Hydrologic modelling to determine design flows for the Macintyre River system has been undertaken using a Design Event Approach (DEA) based on the Ensemble Event methodology described in ARR, 2019.

The URBS hydrological models for the Macintyre Brook catchment were used to estimate the design hydrographs for the 20%, 10%, 5%, 2%, 1%, 1 in 2,000, 1 in 10,000 AEP and PMF events. A climate change scenario was also considered for the 1% AEP event based on Representative Concentration Pathways 8.5 scenario for 2090. Details of the adopted methodology is provided in Section 14.5.6.1. Details of the adopted hydrologic model parameters specific to the Macintyre Brook catchment (i.e. focal points) are presented in the following sections.

For the Macintyre Brook modelling, three hydrologic models were used (Figure J1-C in EIS Appendix T2: Hydrology and Flooding Technical Report - Volume 2):

- Macintyre Brook to Booba Sands Catchment Model
- Yelarbon Local Model (encompassing Kippenbung and Bringalow catchments)
- Lower Macintyre River Model including inflows from Dumaresq (Section 17.2).

The local catchments of Kippenbung Creek to the south of Yelarbon were assessed to calculate the local catchment flows through Yelarbon. This is pertinent in the smaller AEP events where the Macintyre Brook breakouts are not the dominant source of flooding at Yelarbon.

Application of the ensemble design event flows into the hydraulic model is complicated in that each set of hydrologic simulations is relevant only to a particular point in the catchment.

- The rainfall ARF and ensemble of TPs used in the simulation are based on the catchment area to the particular location of interest.
- The ensemble of TPs used in the simulation is also based on the catchment area to the particular location of interest.
- A pattern selected as being representative of flows at a particular location (e.g. median or 'Rank 6') may be a different rank at other locations and may therefore produce flows higher (or lower) than the median at that location.

The catchment flooding therefore cannot be represented using a single representative design event. Hydrologic and hydraulic simulations were therefore conducted for a number of focal locations, with the design flood surface taken as the envelope of the critical design event at each focal location. This process involved:

- Assessing and adopting a design flow appropriate to each location of interest based on the representative ARF and pattern ensemble applicable to that location, from which a characteristic (Rank 6) pattern was selected.
- Reviewing the corresponding peak flows that this event produced at downstream locations. If this flow exceeded the adopted design flow at a downstream location due to the issues discussed above (inappropriate ARF, different ensemble or pattern rank) and thereby represented a more extreme event in the downstream catchment, the rainfall/inflows applied to the catchment downstream of that particular area of interest were reduced until the flow no longer exceeded the equivalent AEP design flow at the downstream location (this reduction does not affect the catchment around the area of interest, but only acts to prevent inconsistent/inappropriate flows/levels further downstream. Areas upstream and downstream are subject to their own individual characteristic design events).
- Simulating the characteristic design event for each focal location, then compiling the results to produce a combined flood surface elevation.

14.2.2 Model validation

The December 2021 event was used to validate the Macintyre Brook hydrologic model developed through calibration, as discussed in Section 17.2. This model covers the Macintyre Brook catchment, encompassing both Woodspring and Inglewood to Booba Sands, just upstream from Glenarvon and Yelarvon. Validation of the model was undertaken to test the robustness of the hydrologic model and confirm that both the developed hydrologic and hydraulic models can replicate the recorded catchment response through comparison against flows and flood levels. It is important to note the reliability of the results is dependent upon the accuracy of the modelled flows, which in turn is dependent on the accuracy of the recorded rainfall. Discussion of the hydraulic model validation using the 2021 event is discussed in Section 14.3.8

14.2.3 Flood frequency analysis

Flood frequency analysis has been undertaken for the Macintyre Brook catchment to validate the design event flows. The methodology and results are documented in Section 17.2.7.

14.2.4 Data extraction for 2021 event

14.2.4.1 Rainfall data

Rainfall data for the December 2021 event was sourced directly from BoM, who provided data for rainfall gauges, spanning a radius of 150 km from Inglewood for the period between 1 November 2021 to 31 December 2021. Furthermore, the gauges provided were of varying recording frequency ranging from 6-minutes up to 24-hour data. A thorough review of the data provided was conducted to determine a series of gauges which were suitable for use and remove any suspect/missing data. During the review, it was found that a number of key gauges which recorded 6-minute data within the Macintyre Brook catchment were not operational at the time of the event. As such, where these gauges had corresponding hourly data, this was used within the validation or alternatively a suitable nearby gauge was then sourced from the BoM's Water Data Online Portal. The hourly rainfall data was also supplemented with hourly flood network gauge data and the daily rainfall data. The final number of rainfall gauges used in the assessment was 191.

The suitable rainfall gauges were reviewed to ensure corroboration between hourly and daily rainfall gauges. A number of cases were identified where several days of rainfall were accumulated in a single record or where a gauge had the similar depths to surrounding gauges but offset by a day, most likely the result of errors in the transposition of manual record data. These gauges were reviewed and adjusted accordingly.

14.2.4.2 Stream gauge and rating data

Stream gauge data was sourced from the BoM's Water Data Online Portal. Rating Curves for Barongarook, Bracker Creek at Terraine, Coolmunda TW, Macintyre Brook at Inglewood and Macintyre Brook at Booba Sands were reviewed and compared against the previous rating curves used in the hydrologic model. It was found that previous rating curves had a good match with the recent rating curve information for all gauges except at Coolmunda TW. The rating curve for Coolmunda TW gauge was therefore updated using the latest information provided by Sunwater.

14.2.5 Validation model parameters

The Macintyre Brook hydrologic model developed through calibration was utilised to model the December 2021 event. The rainfall data was applied to the model and the losses were varied spatially across the catchment to simulate a reasonable catchment response. By changing the loss parameters, it was possible to account for the unique antecedent conditions the catchment experienced at the time of the December 2021 event. Moreover, ensuring the routing parameters are kept consistent means the catchment characteristics remain the same (i.e. keeping it consistent with previous historical events). Table 14.3 shows the loss values adopted across the catchment for the December 2021 validation event.

Table 14.3 Macintyre Brook catchment—Rainfall losses applied for December 2021 validation event

Location	Loss applied	
	Initial loss (mm)	Continuous loss (mm/hr)
Woodspring	65	1.0
Barongarook	50	0.5
Terraine	45	1.2
Coolmunda	50	1.8
Inglewood	65	1.0
Booba Sands	100	1.5

14.2.6 Validation event model results

Figure 14.1 to Figure 14.5 presents the December 2021 URBS results at Barongarook, Bracker Creek at Terraine, Coolmunda TW, Macintyre Brook at Inglewood and Macintyre Brook at Booba Sands.

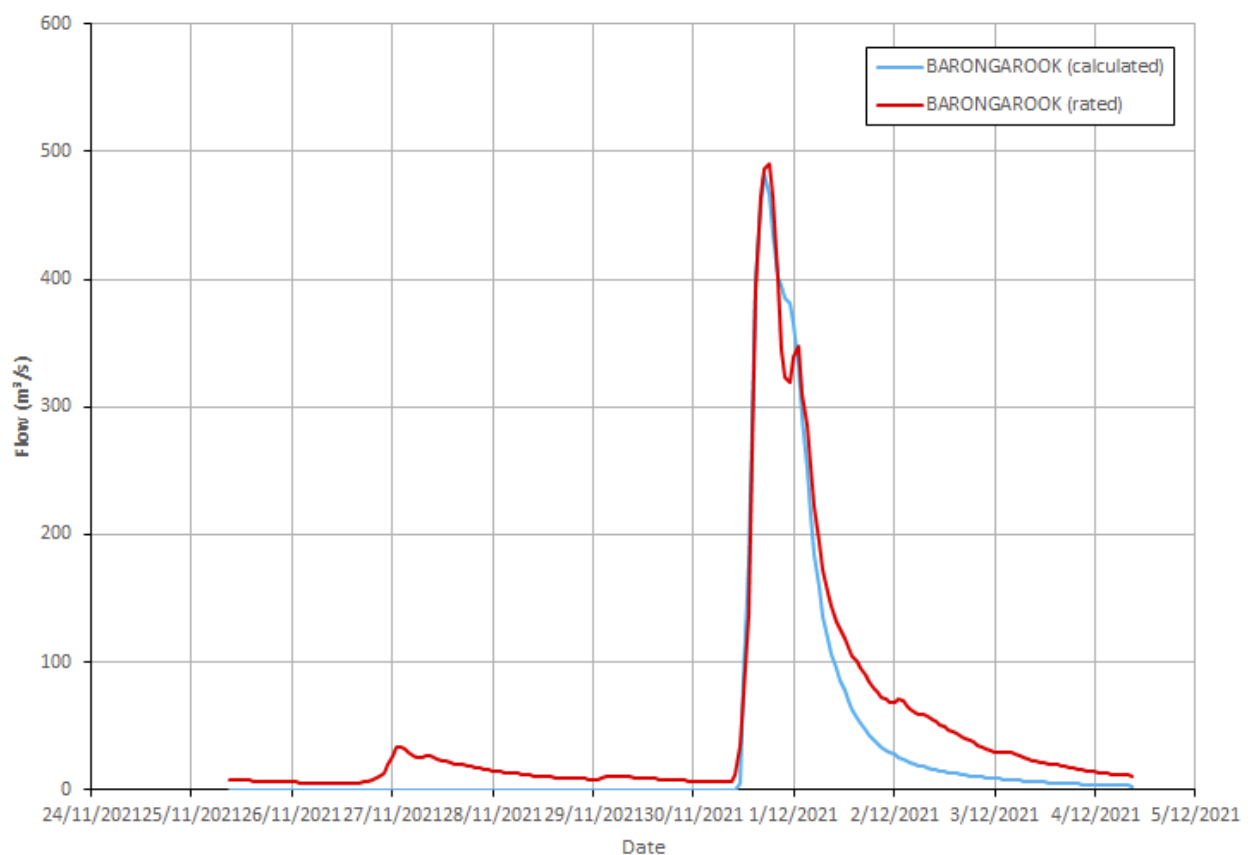


Figure 14.1 December 2021 calculated (blue) and rated (red) hydrographs—Barongarook

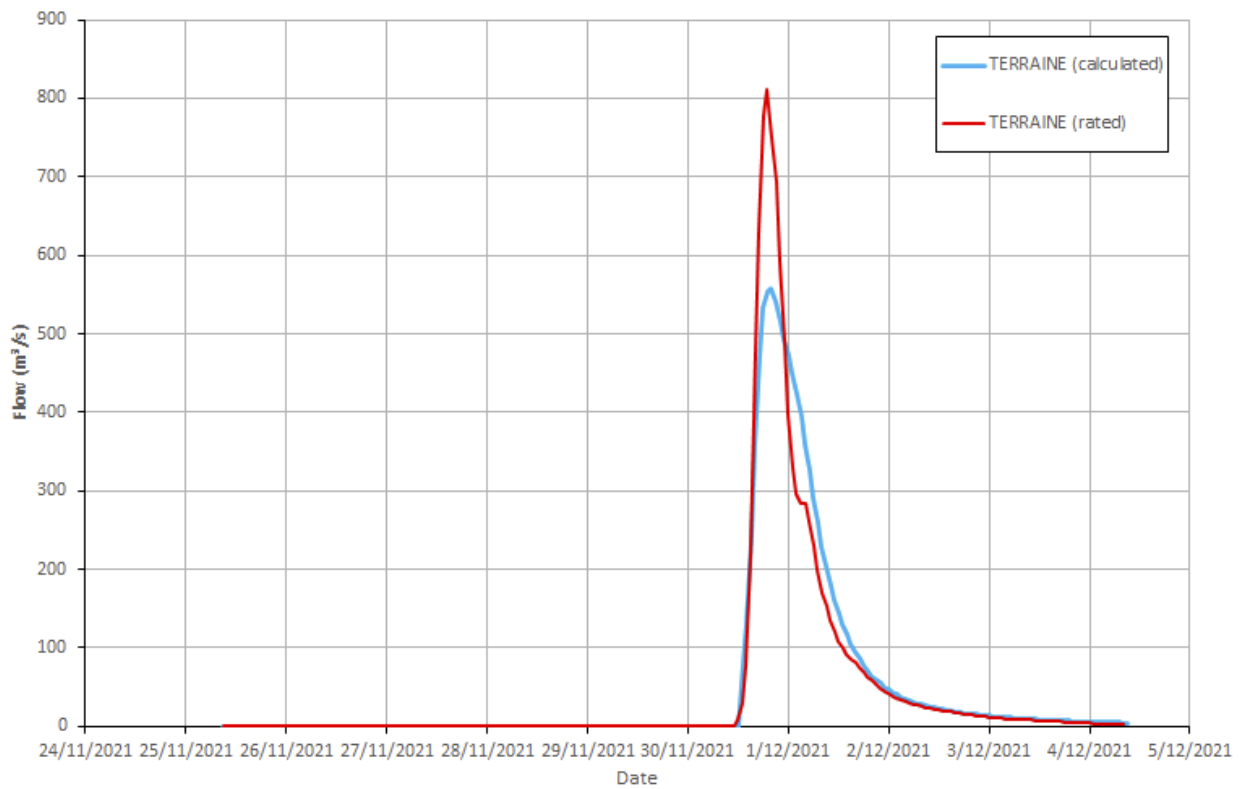


Figure 14.2 December 2021 calculated (blue) and rated (red) hydrographs—Terraine

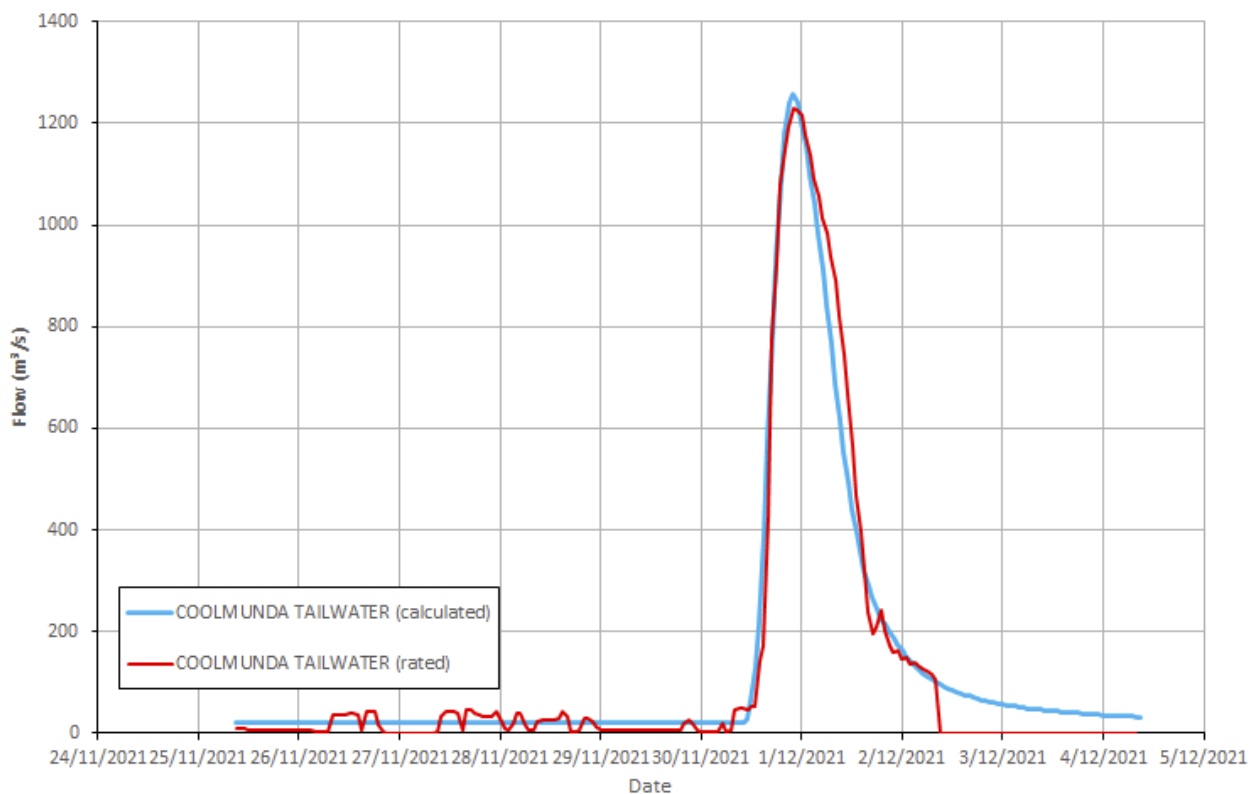


Figure 14.3 December 2021 calculated (blue) and rated (red) hydrographs—Coolmunda

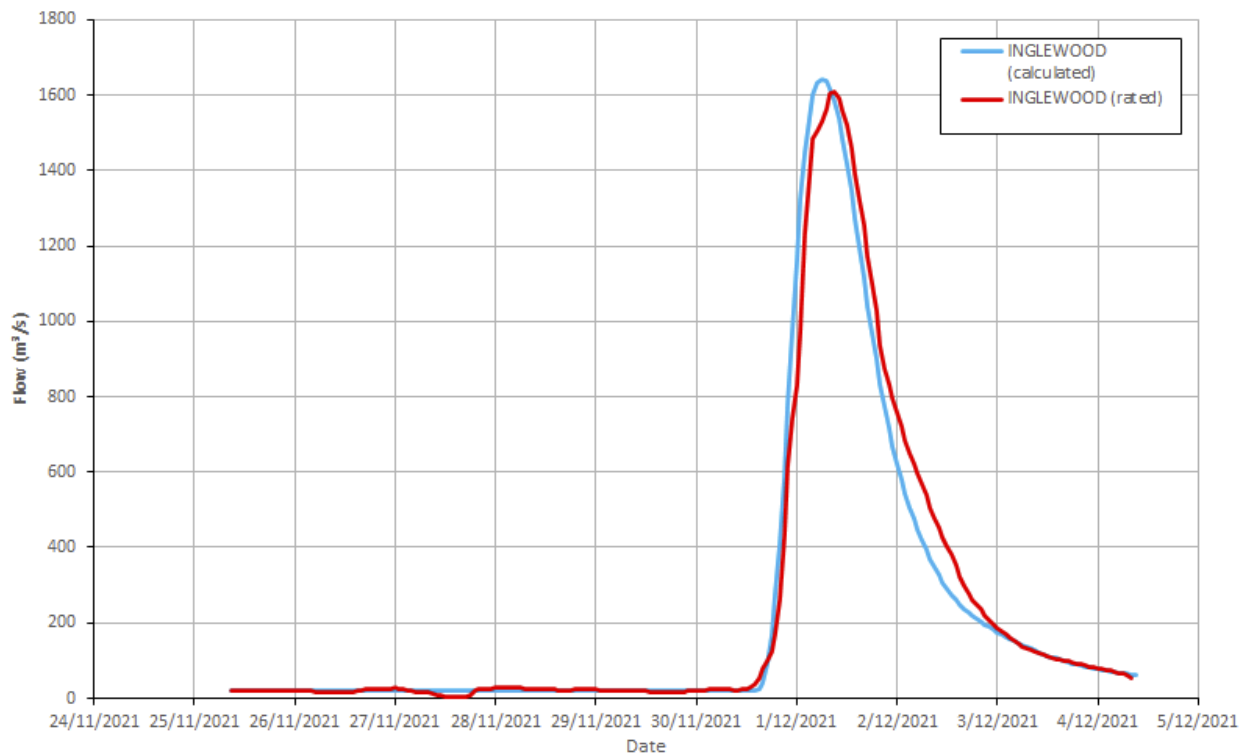


Figure 14.4 December 2021 calculated (blue) and rated (red) hydrographs—Inglewood

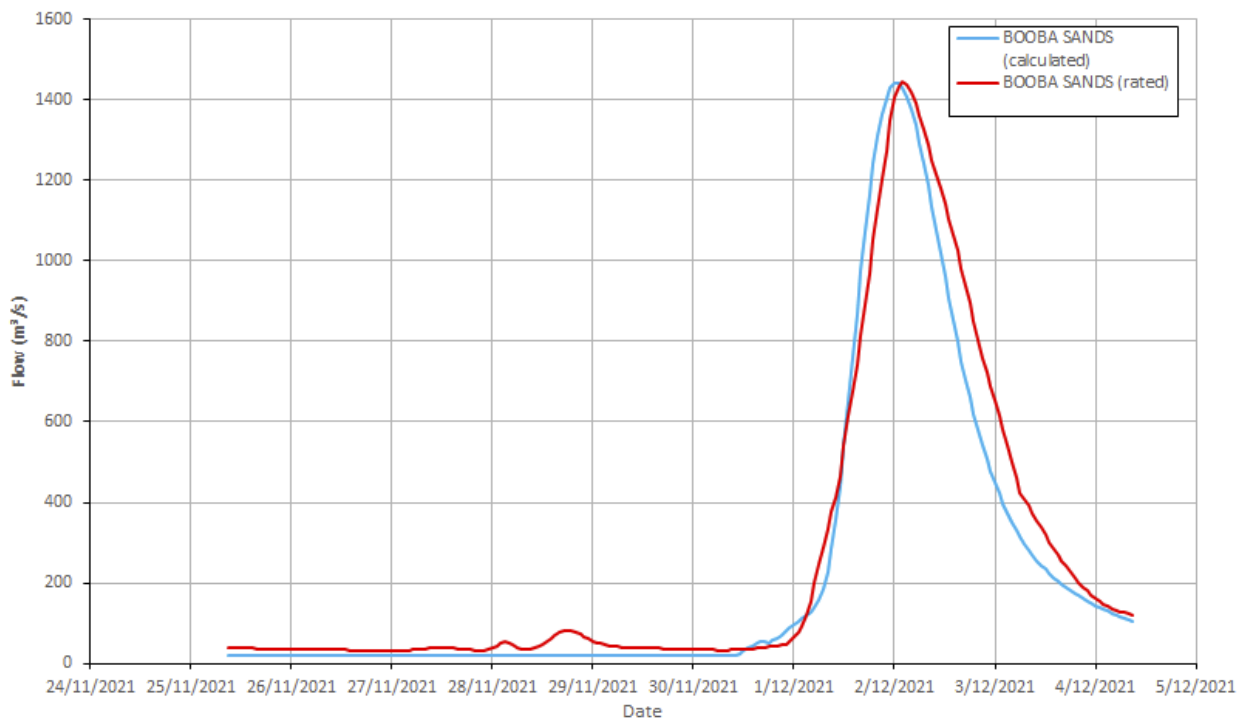


Figure 14.5 December 2021 calculated (blue) and rated (red) hydrographs—Booba Sands

The comparison between calculated (modelled) and rated hydrographs show that the model is predicting flows throughout the catchment, with the rising and falling limbs of the hydrograph matching well with what was recorded at the gauge. Greater emphasis was made on matching the shape of the hydrograph than the peak due to uncertainty of the ratings at higher flows. It should be noted that there is uncertainty in the flow estimates coming from the Canning Creek catchment upstream of Inglewood as the gauge at Woodspring is no longer operated. Nevertheless, a good match at Inglewood and Booba Sands was achieved.

14.3 Hydraulic model development: Macintyre Brook—Yelarbon to Inglewood

Two relevant hydraulic models were developed across the Macintyre Brook—Yelarbon to Inglewood Study area and provided to support this assessment. These are:

- Inland Rail: North Star to Border, Macintyre River model. This model covers the section of the study area from downstream of Yelarbon to the NSW/QLD Border and is covered in Section 17.
- *Inglewood Flood Study* (Engeny, 2015) TUFLOW model. This model covers the section of the study area in Inglewood.

Neither of these models cover the full extent of the study area (from the Border to upstream of Inglewood); as such, a hydraulic model was developed using data from both *Inglewood Flood Study* (Engeny, 2015) TUFLOW model and the NS2B Macintyre River TUFLOW model. This new model extends from immediately downstream of Yelarbon to upstream of Inglewood (Macintyre Brook hydraulic model). The following sections provide details on the Macintyre Brook hydraulic model, referred to as 'TUFLOW hydraulic model' herein.

14.3.1 Model setup

The Macintyre Brook TUFLOW hydraulic model was set up on a 15 m grid and developed in TUFLOW HPC. The model covers an area of 610 km². The hydraulic model extent and adopted land use are presented in Figure J1-D and Figure J1-E in Volume 2 – Appendix J.

The model setup details are presented in Table 14.4.

Table 14.4 Macintyre Brook—hydraulic model setup overview

Parameter	Information
Completion date	August 2022
AEP's assessed	20%, 10%, 5%, 2%, 1%, 1 in 2,000, 1 in 10,000 AEP and PMF events
Hydrologic modelling approach	URBS: Unified River Basin Simulation runoff-routing models which converts rainfall excess into flow
Hydraulic modelling approach	TUFLOW: 2-Dimensional numerical model (Version 2020-10-AD-iSP-w64)
Model extent	Refer to Figure J1-D in Volume 2 – Appendix J
Grid size	15 m
SGS size	1.67 m (for 15 m grid)
QT parameters	3.75 m (Medium and Low Risk)/1.875 m (High Risk) ¹ At Yelarbon and Whetstone the QT extents cover 100 m upstream and downstream of the structures, whilst at Inglewood the QT extents cover 20 m upstream and 100 m downstream of the structures.
DEM (year flown)	ARTC dataset (2015)
Roughness	Spatially varying roughness values compliant with industry norms (Table 14.6.)
Eddy viscosity	Wu (default)
Model calibration	1976, 1996, 2011, 2021
U/S model boundary	Flow verse Time/SA
D/S model boundary	Height-Discharge (HQ) Boundary with normal slope approximated based upon topography dataset
Hydraulic model timestep	Adaptive timestep
Modelled scenarios	Existing Case, Developed Case

Table note:

- 1 QT resolution was determined based on culvert crossing Geomorphology risk category. Refer to EIS Appendix H: Geomorphology Report

14.3.2 Model topography and grid size

The topography within the TUFLOW hydraulic model was based on 1 m DEM data collected during 2015, which was supplied by ARTC. A 15 m x 15 m grid size was selected to allow the features of the channels and floodplain to be represented with sufficient accuracy while maintaining efficient model run times. The 15 m x 15 m grid size is considered adequate to represent floodplain topography and is consistent with the approach typically adopted in similar flood studies.

Where necessary, the 2D model topography was modified so that the existing drainage flowpaths, existing road/rail crowns and levee banks were represented accurately in the model. Road crests were enforced in the TUFLOW hydraulic model through digitising lines to represent the road crests for key road infrastructure.

For the Developed Case, the topography was modified to include the Project rail embankment, drainage structures and associated infrastructure.

14.3.3 Sub-grid sampling and Quadtree mesh refinement

The hydraulic model was established using TUFLOW's 2020-10-AD model version, including SGS and QT add-on modules. The below points provide a brief description of each module, along with the key parameters adopted for this assessment.

- Traditional grid-mesh based 2D hydraulic models sample the underlying DEM by taking a single elevation at the cell centres and cell faces. Sub-grid sampling allows for increased resolution by sampling a greater number of points of the underlying DEM to establish a water surface elevation versus volume relationship for each grid, along with a water surface elevation versus width relationship (wetted perimeter) at cell sides. The adoption of SGS allows increased representation of storage and conveyance between cells. Sub-grid sampling has been applied to the model at a 1.67 m sample distance for the base grid.
- Quadtree mesh is constructed by subdividing the base mesh-grid size into four smaller square cells. Cells can be further sub-divided into four, and so on, allowing for additional resolution in location of interest. The base hydraulic model grid cell size (15 m x 15 m) was revised to include additional spatial resolution of approximately 3.75 m, to better represent model velocity at the entry and exit of culverts. Areas defined as high risk of erosion (as defined within EIS Appendix H: Geomorphology Report) were modelled with QT at 1.875 m. Quadtree meshes were applied in the model covering minimum 100 m upstream and downstream of design culvert locations at Yelarbon and Whetstone, and 20 m upstream and 100 m downstream of design culvert locations at Inglewood.
- The schematisation of 2d sx connection lines for design culverts was updated to ensure that the sx lines still represented the approximate flow width of the design culvert where refined grid resolution was used. To accurately represent conveyance of flow across the alignment through large culvert banks, culverts were split into several banks where appropriate.

14.3.4 Existing hydraulic structures

Major structures were included in the TUFLOW hydraulic model. Bridge details were sourced from the *Inglewood Flood Study* (Engeny, 2015) TUFLOW Model for Inglewood and surrounds. Outside of these areas, details were obtained from site inspection, QR, and LiDAR inspection. Table 14.5 presents the existing structures included in the hydraulic model.

Table 14.5 Macintyre Brook—Existing structure details

Structure modelling ID	Structure type	Road deck height (mAHD)	Representation in TUFLOW
Bybera Road Bridge	Bridge	270.5	lfcsh
Cunning Highway Bridge (Inglewood)	Bridge	282.7	lfcsh
Millmerran-Inglewood Road Bridge	Bridge	285.0	lfcsh

Structure modelling ID	Structure type	Road deck height (mAHD)	Representation in TUFLOW
Potters Road Culvert	Culvert: 1 x 1.5 m (h) x 1.8 m (w) RCBC	272.2	1D Network
Lovells Crossing Road Causeway	Causeway	273.3	Z Line
QR Existing Rail Line	Railway embankment	N/A	Z Line
Yelarbon Levee	Levee	N/A	Z Line
Cunningham Highway Culverts (Yelarbon)	Culverts: 1 x 0.6 m (h) x 1.2 m (w) RCBC and 1 x 0.9 m RCP	N/A	1D Network
QR Existing Rail Culverts (Yelarbon)	Culverts (QR IDs): SW 155.820 km 1 x 0.3 m (h) x 0.6 m (w) RCBC SW 155.360 km 1 x 1.2 m (h) x 1.2 m (w) RCBC SW 155.040 km 6 x 0.6 m (h) x 1.2 m (w) RCBC SW 154.250 km 1 x 0.5 m (h) x 3.0 m (w) RCBC SW 154.230 km 1 x 0.5 m (h) x 3.0 m (w) RCBC SW 148.810 km 2 x 1.20 m RCP SW 148.770 km 4 x 1.35 m RCP SW 154.150 km 1 x 0.6 m (h) x 1.2 m (w) RCBC SW 153.480 km 1 x 0.3 m RCP SW 149.610 km 1 x 1.2 m (h) x 1.2 m (w) RCBC	N/A	1D Network

It should be noted that not all existing structures have been represented within the hydraulic model due to limited “as-built” information being available at the time of model development. A review of existing structures will be undertaken in the detailed design stage when more detailed survey has been undertaken to support the Project maturity as well as ongoing consultation with existing structure owners (e.g. QR and DTMR). Refer to Section 14.6 for further details.

14.3.5 Roughness

The hydraulic roughness reflects the types of development and ground cover that exists within the hydraulic model extents. The distribution of roughness categories adopted for this assessment were based on the parameters used in the Macintyre River model, the *Inglewood Flood Study* (Engeny, 2015) model, aerial imagery and confirmed during site inspection. Specific roughness values applied to the model are detailed in Table 14.6.

Table 14.6 Macintyre Brook—Manning's n values

Land use	Manning's n
River channel	0.03
Floodplain area	0.06
Floodplain area 2	0.08
Dense vegetation	0.08
Pasture and sparsely vegetated areas	0.06
Buildings	0.40

Figure J1-E in Volume 2 – Appendix J shows the spatial breakdown of land use in the 2D model domain.

14.3.6 Boundary conditions

14.3.6.1 External and internal boundary conditions

The inflow boundary and internal source boundaries were based on results of the hydrologic modelling. A normal depth boundary was applied for the downstream outflow model boundary. Source Area polygons were applied as inflow boundaries from upstream tributaries. A sensitivity assessment updating the SA inflows at the boundaries to discharge-time boundaries was undertaken. The analysis showed that the difference in levels between the boundaries dissipated a short distance from the boundary, and it was confirmed that the boundaries where a sufficient distance away from the area of interest so as not to impact on modelling results. Furthermore, the SA boundaries were shown to be more stable with the Macintyre Brook flood flows.

Delineated sub-catchments in the hydrologic model were used as internal inflow boundaries within the hydraulic model extent. The modelled flows from the hydrologic model were applied as SA polygons within each sub-catchment in the hydraulic model. Figure J1-D in Volume 2 – Appendix J shows the hydrologic inflow locations to the hydraulic model.

14.3.7 Calibration

Hydrologic models are based on simplistic empirical runoff routing equations using coefficients determined primarily by calibration to a specific point of interest. By contrast, hydraulic models are more physically based, providing a (relatively) realistic representation of the catchment geometry and solving equations of motion within the model domain. Some differences between the hydrologic and hydraulic routing must realistically be expected. Nevertheless, the hydraulic model should closely replicate the flow characteristics (attenuation, timing, etc.) that in the hydrologic model have been validated by calibration to historical flood events.

The hydraulic model must also produce flood levels consistent with the flows. This can be confirmed by comparison with flood levels recorded during historical flood events, although the reliability is dependent on the accuracy of the modelled flows, which are in turn dependent on the accuracy of the recorded rainfall. Further validation across a wide range of flows can be achieved by comparison of the modelled level-flow relationships at the stream gauge sites with the gauge ratings, which allows the level-flow relationship to be confirmed without necessarily having to exactly match a specific flow.

The TUFLOW hydraulic model has been validated using historical events. The primary objectives of the calibration process have been to confirm:

- The hydraulic model roughness factors required to match level-flow relationships at the stream gauges, particularly those where the ratings are well defined by in-streamflow measurements
- That the flood routing through the TUFLOW hydraulic model reasonably matches the hydrologic model (TUFLOW physically represents storage and other catchment characteristics that are represented in hydrology software by empirical coefficients) and that the adopted roughness parameters do not adversely affect the timing or attenuation of the flood routing.

The historical flood events were selected to represent a range of magnitudes and duration. The hydraulic model was run using the updated inflows from the hydrologic model, as detailed in Section 17.2, for each of the three historical calibration events (1976, 1996 and 2011). An iterative joint calibration process was undertaken between the hydrologic and hydraulic models to achieve the best overall match to the available historical flood data across the three historical flood events.

A number of the hydraulic model parameters were adjusted as part of the calibration process including:

- Roughness values
- Definition of the waterway channel invert levels to improve in-channel conveyance
- Use of SGS to improve representation of conveyance
- Inclusion of flows from lower floodplain area.

The following datasets have been used to assess the calibration of the hydraulic model:

- Data from Inglewood (416402B/C) and Booba Sands (416415A) stream gauges, including:
 - Recorded peak water levels and estimated flows
 - Match in shape and timing of recorded flood hydrographs
 - Comparison against the level-flow relationships at the stream gauges
 - Recorded flood markers across the floodplain—these markers come from a range of sources and have been collated from previous studies and/or survey of property, available for the 1976 flood event only.

Figures J2-A to J2-C in Volume 2 – Appendix J present the 1976, 1996 and 2011 flood events, respectively. The following sections detail the outcomes of the hydraulic model calibration for each of the historical flood events.

The updated hydrology for Macintyre Brook includes calibration to three historical events—1976, 1996, 2011. For Macintyre Brook, there is only gauge data for 1996 and 2011 (Inglewood Gauge and Booba Sands); and gauge (Inglewood gauge only) and anecdotal flood data for Inglewood (surveyed flood levels) for 1976.

The updated hydrologic flows have been included as inflows to the TUFLOW hydraulic model and simulated for the three historical events. Table 14.7 presents the recorded flood levels at the gauge and the comparison to the TUFLOW modelled results.

Table 14.7 Macintyre Brook catchment—Historical floods gauge comparison—Updated modelling

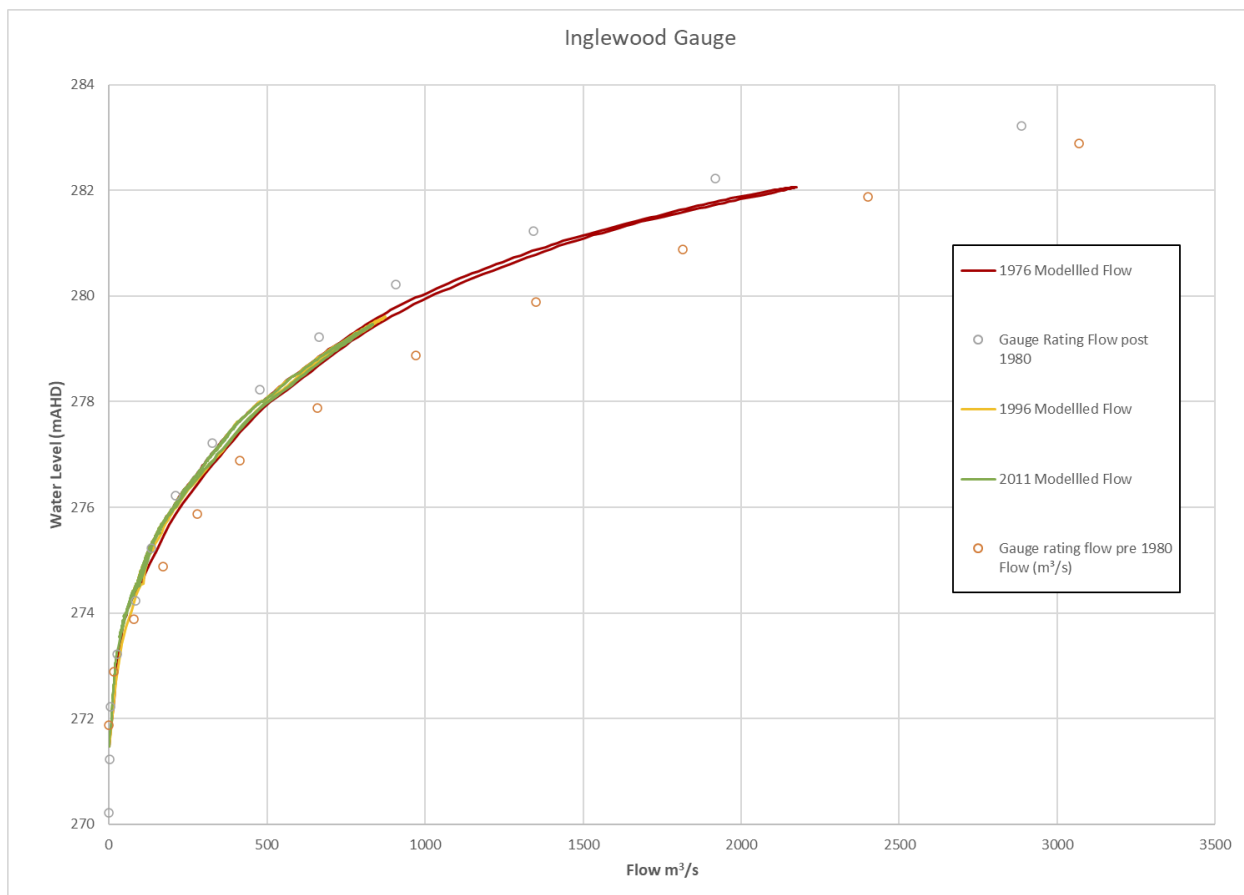
Gauge name	Recorded gauge levels (mAHD)			Updated modelled gauge levels (mAHD)			Difference (m)		
	1976	1996	2011	1976	1996	2011	1976	1996	2011
Inglewood	282.03	280.04	279.90	282.32	279.96	279.83	+0.29	-0.08	-0.07
Booba Sands	- ¹	251.71	251.61	254.09	251.76	251.67	-	+0.05	+0.06

Table note:

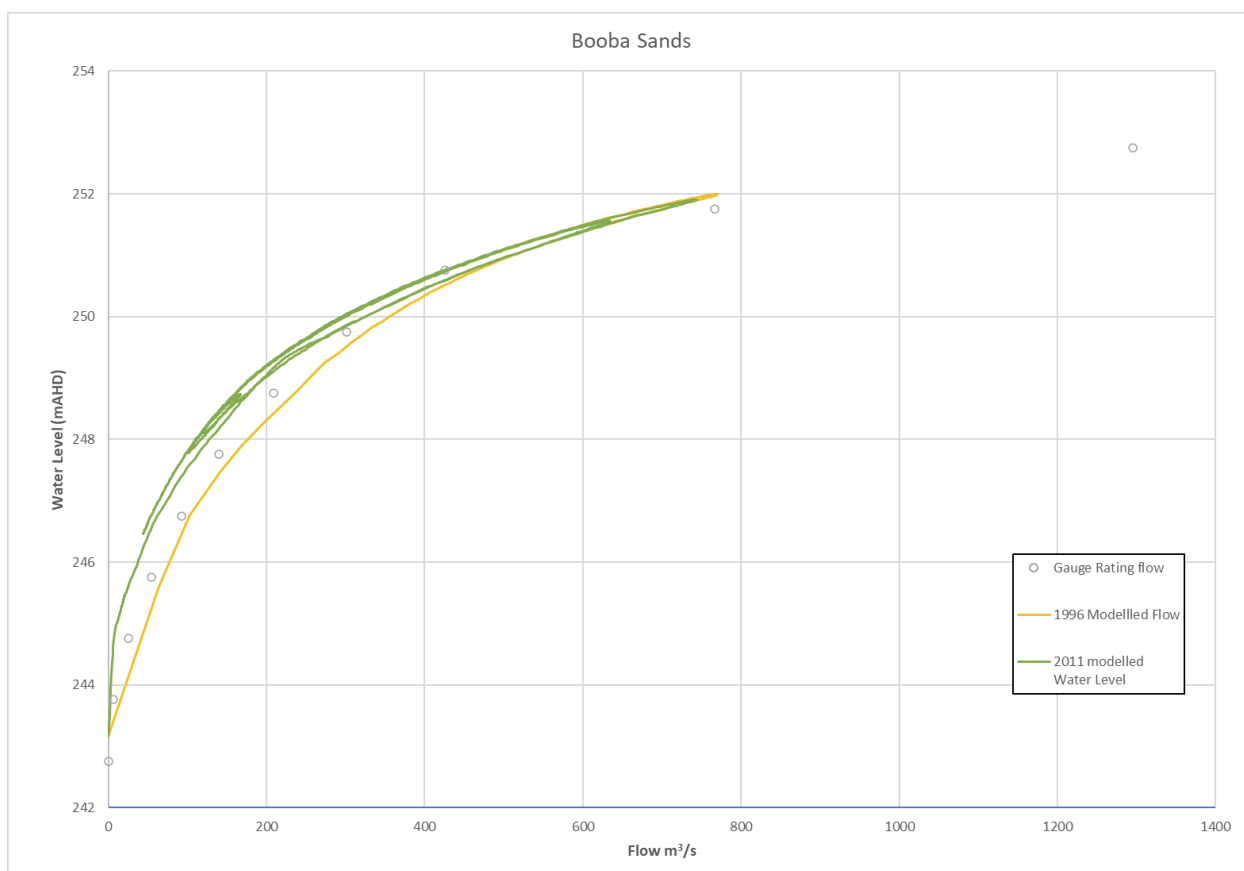
¹ No data exists for this gauge for the 1976 event

The peak flood levels from the TUFLOW model compare well with the historical gauge levels, with the modelled levels for both gauges within ± 0.3 m for all events. It is noted that the 1976 results are outside the targeted ± 0.2 m for stream flow gauges, although as this event occurred in 1976 (over 45 years ago) the +0.29 m match is considered acceptable.

Figure 14.6 presents a comparison of the modelled flows and level information at Inglewood and Booba Sands for each of the three historical events against the rated data from the stream gauges (i.e. measured during events).



a) Inglewood



b) Booba Sands

Figure 14.6 Inglewood and Booba Sands—Comparison of recorded and modelled gauge ratings

For all three historical events, the hydraulic model results give a good representation of conditions at the stream gauges based on the ratings. Figure 14.7 to Figure 14.9 present the comparison of the recorded water levels against the modelled water levels over the duration of the three historical events at both gauges.

The model replicates the hydrograph shape and timing of the flood at these gauge locations for the three historical events.

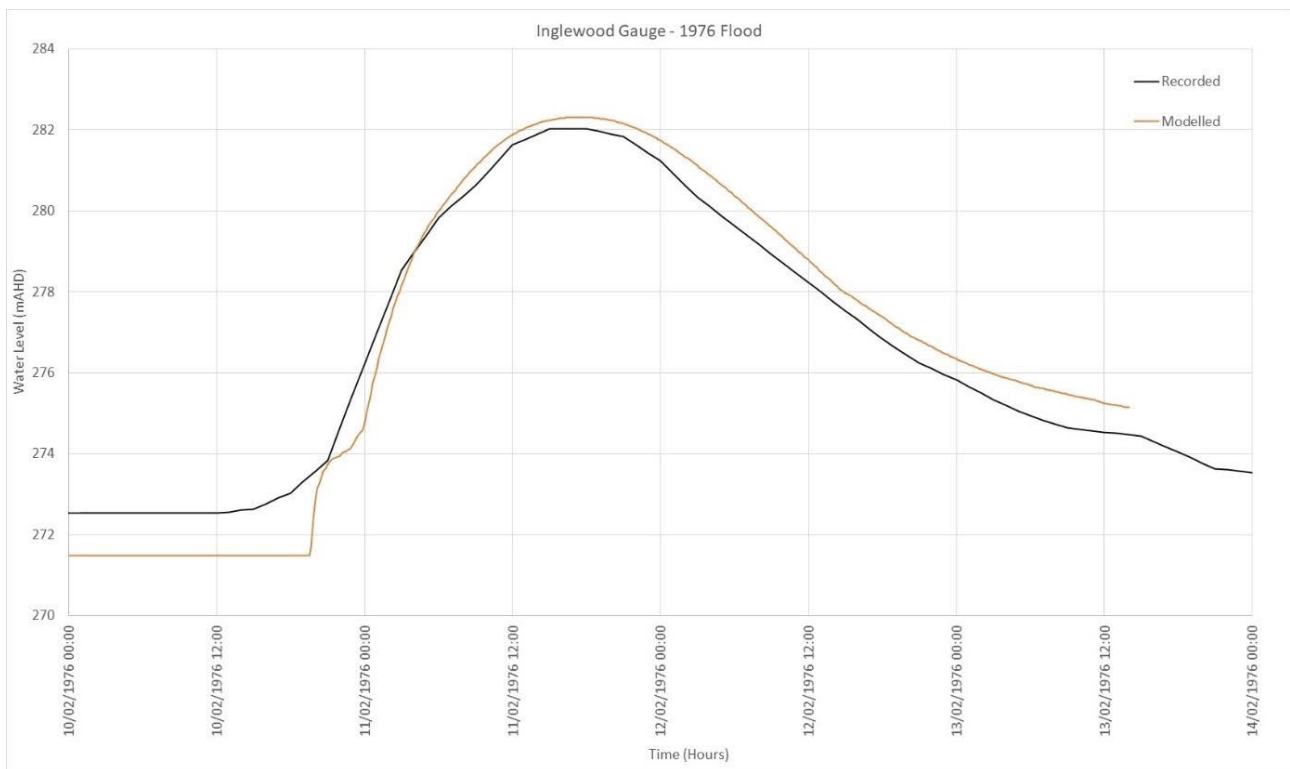
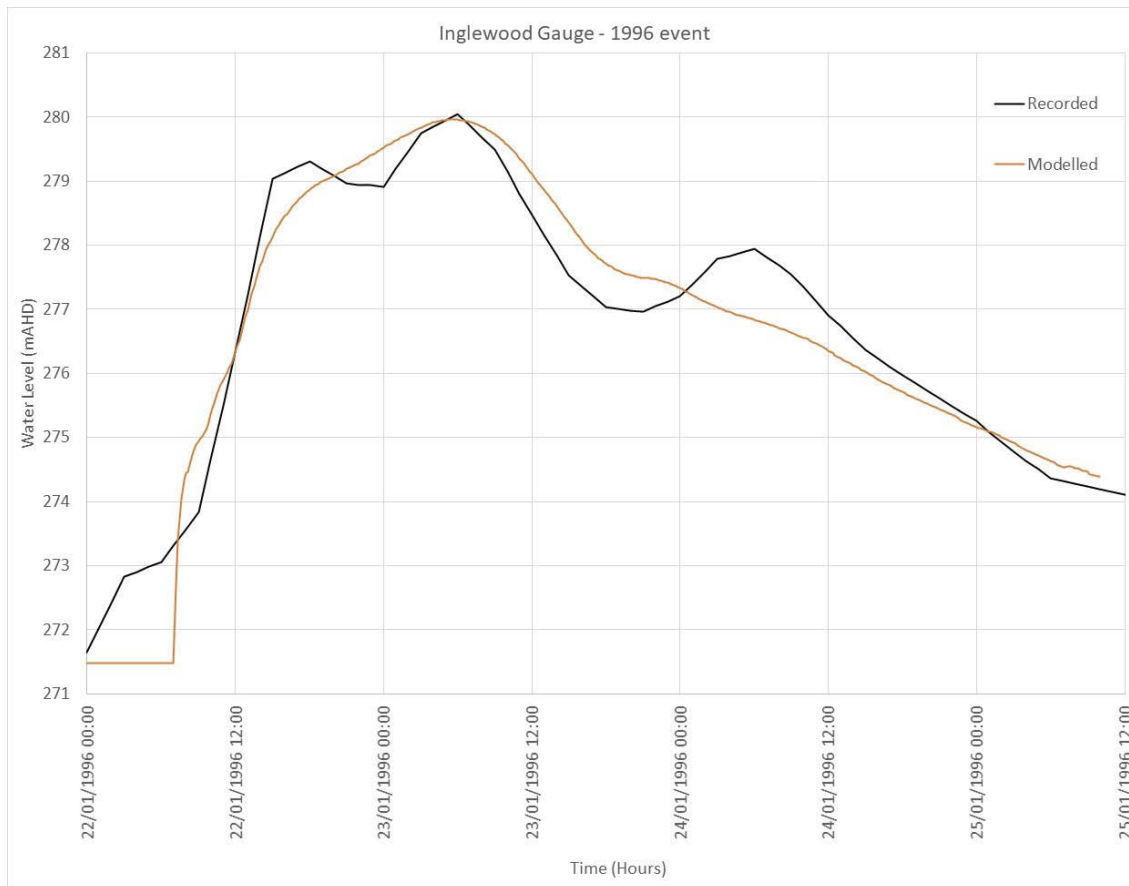
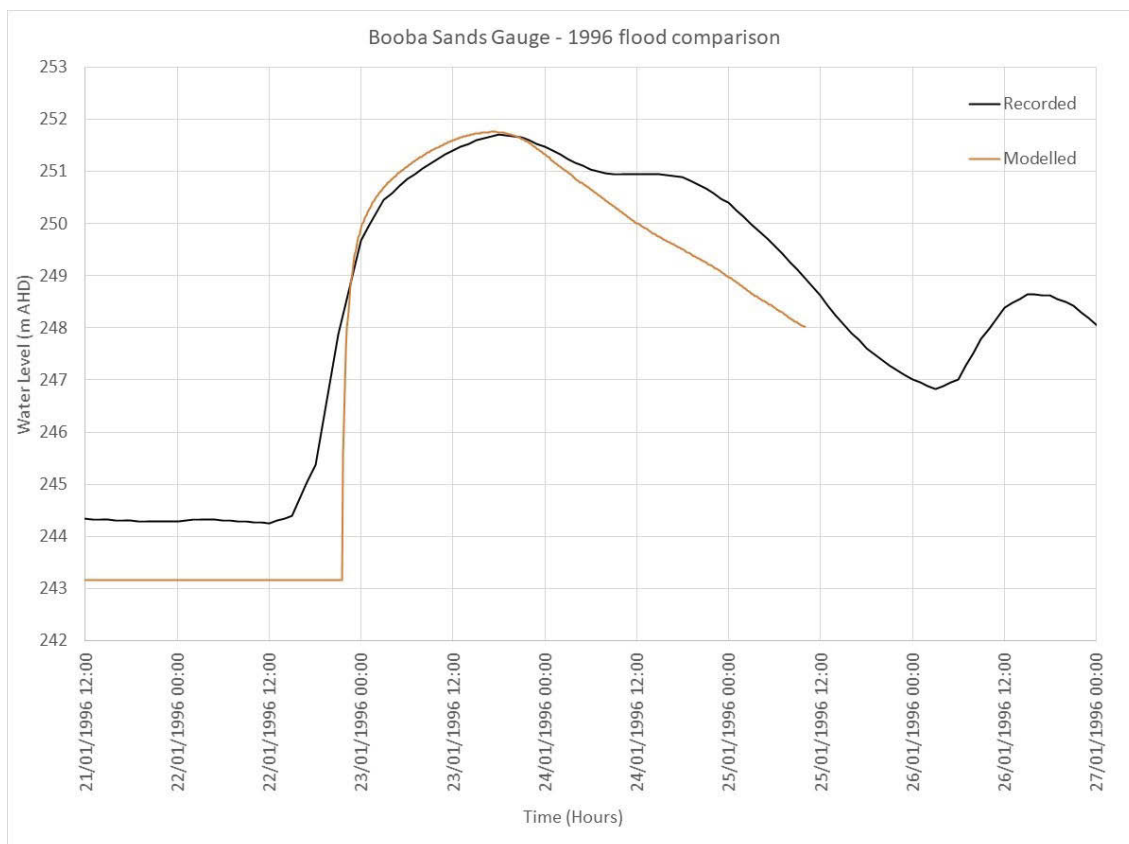


Figure 14.7 1976 Flood recorded and modelled gauge levels at Inglewood

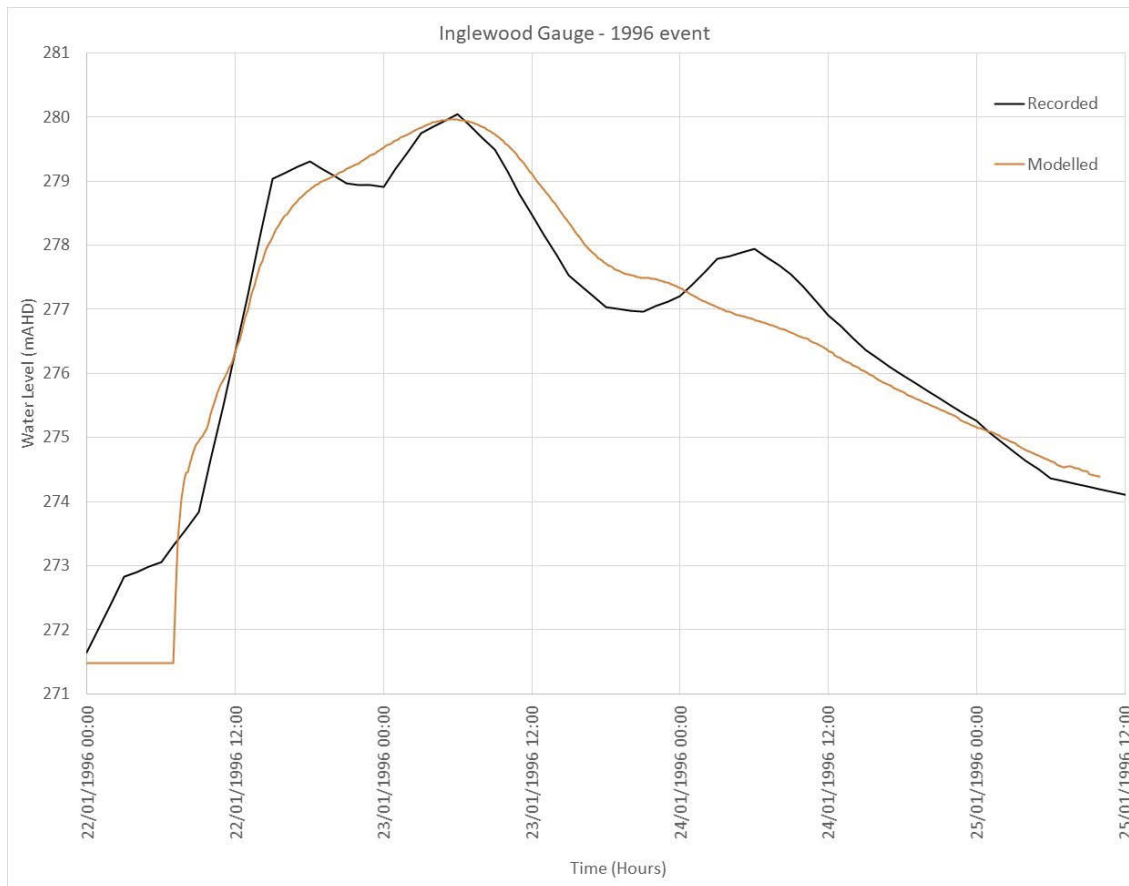


a) Inglewood

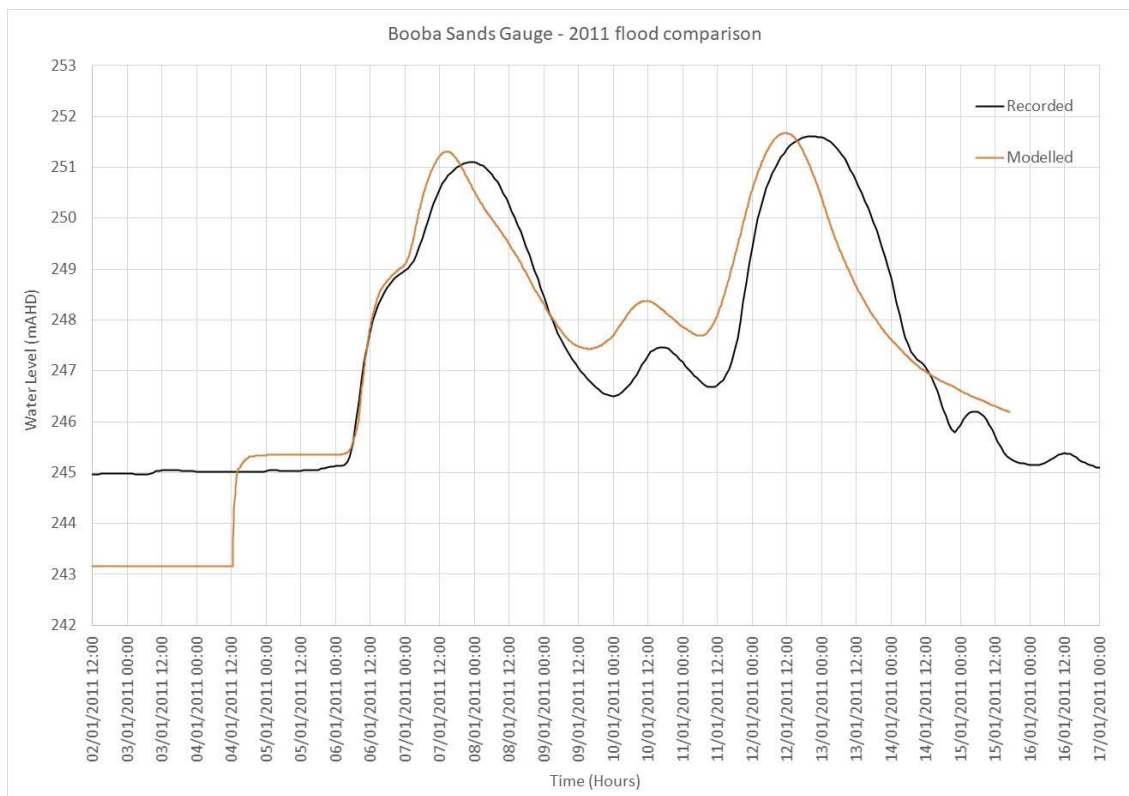


b) Booba Sands

Figure 14.8 Inglewood and Booba Sands—1996 Flood recorded and modelled gauge levels



a) Inglewood



b) Booba Sands

Figure 14.9 Inglewood and Booba Sands—2011 Flood recorded and modelled gauge levels

For the 1976 flood event, there are 19 recorded surveyed floodmarks available in Inglewood to inform this study. These are presented in Table 14.8. The updated hydraulic model flood levels generally compare well to the recorded flood heights, with 15 of the 19 flood markers within ± 0.2 m of the surveyed heights. There was no surveyed flood information available for the 1996 and 2011 flood events as these were predominately contained in channel for the Macintyre Brook.

Table 14.8 Macintyre Brook—1976 Surveyed flood level comparison

ID	Recorded level (mAHD)	Modelled level (mAHD)	Difference (m)
1	283.67	283.59	-0.08
2	283.60	283.78	+0.18
3	283.72	283.89	+0.17
4	283.61	283.69	+0.08
5	284.04	284.16	+0.12
6	284.15	284.16	0.00
7	283.87	283.89	+0.02
8	283.69	283.58	-0.11
9	283.72	283.77	+0.04
10	283.94	284.03	+0.09
11	284.45	284.38	-0.07
12	284.13	283.60	-0.53
13	284.83	284.38	-0.45
14	283.87	283.86	-0.01
15	283.18	283.07	-0.11
16	285.14	284.67	-0.47
17	285.02	283.94	-1.08
18	282.98	282.81	-0.18
19	284.17	284.23	+0.06

It has been demonstrated that the hydrology and hydraulic models suitably represent the historical flood events, and no bias is evident in the outcomes.

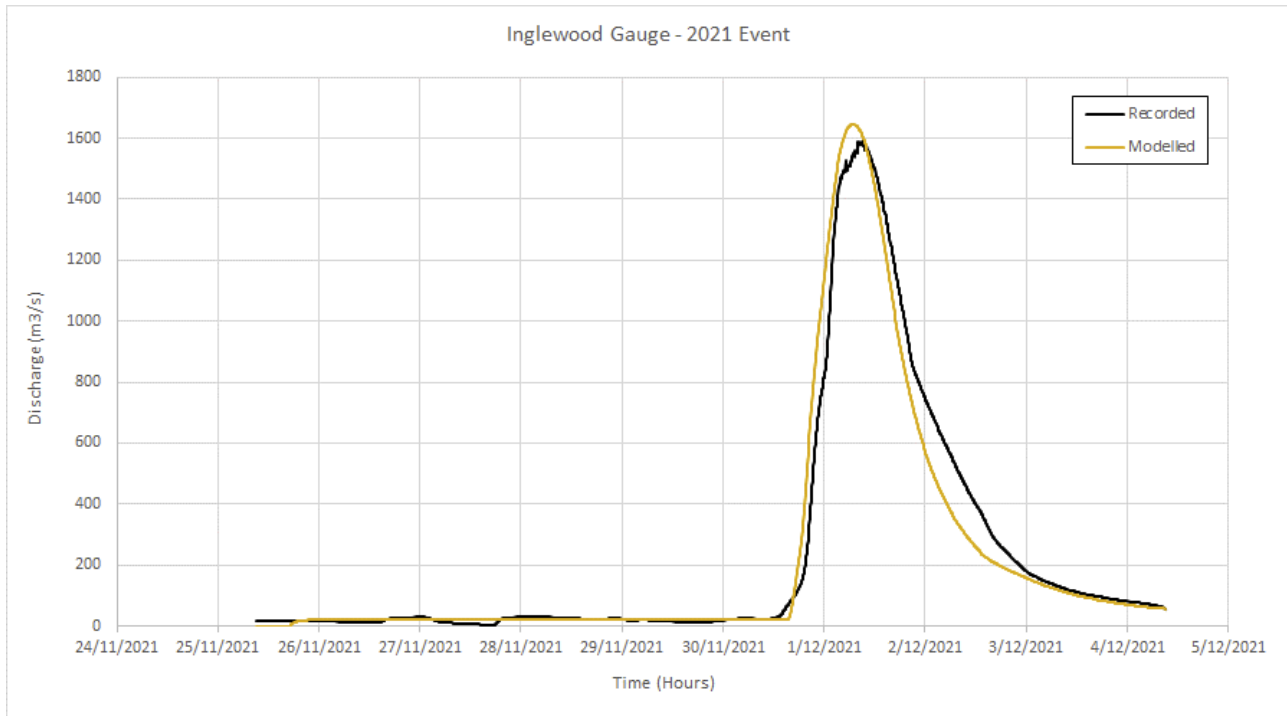
14.3.8 Validation

In addition to being used to validate the hydrologic model, the December 2021 flood event has been used to validate the TUFLOW hydraulic model. No flood markers were available for the 2021 event, so flood levels and flows from stream gauge data on the Macintyre Brook at Inglewood and Booba Sands were compared against the TUFLOW model outputs at these two locations.

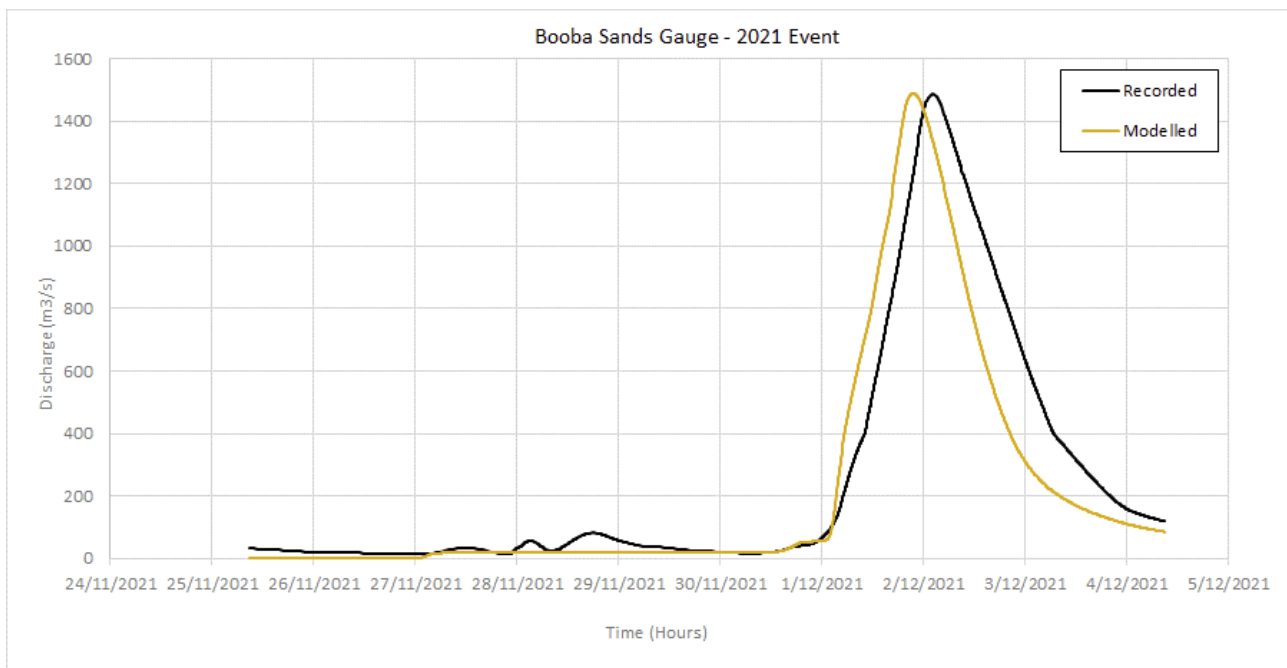
Figure 14.10 presents the flow comparisons between the URBS hydrograph, the recorded hydrograph and the TUFLOW hydrograph for the two gauges. The graph shows there is a good correlation between the modelled and predicted flood levels.

The timing of the hydrograph at Inglewood is slightly early (1 to 2 hours), which is consistent with the hydrology inputs; however, the hydrograph at Booba Sands is approximately 5 to 6 hours earlier than that recorded, indicating that the flow is travelling faster than the recorded hydrology inputs. It should be noted that a similar timing shift is observed in the 2011 calibration modelling, particularly the second peak of the flood (Section 14.3.7). This could potentially be related to the roughness of the floodplain, which could change seasonally or could be affected by the flood itself.

Nevertheless, it can be concluded that the both the hydrologic and hydraulic model are capable of replicating reasonable results with good correlation between predicted and recorded results. Investigating increasing the roughness parameters could assist in slowing down the response of the predicted hydrograph and, in turn, potentially result in a better match between recorded and calculated hydrographs.



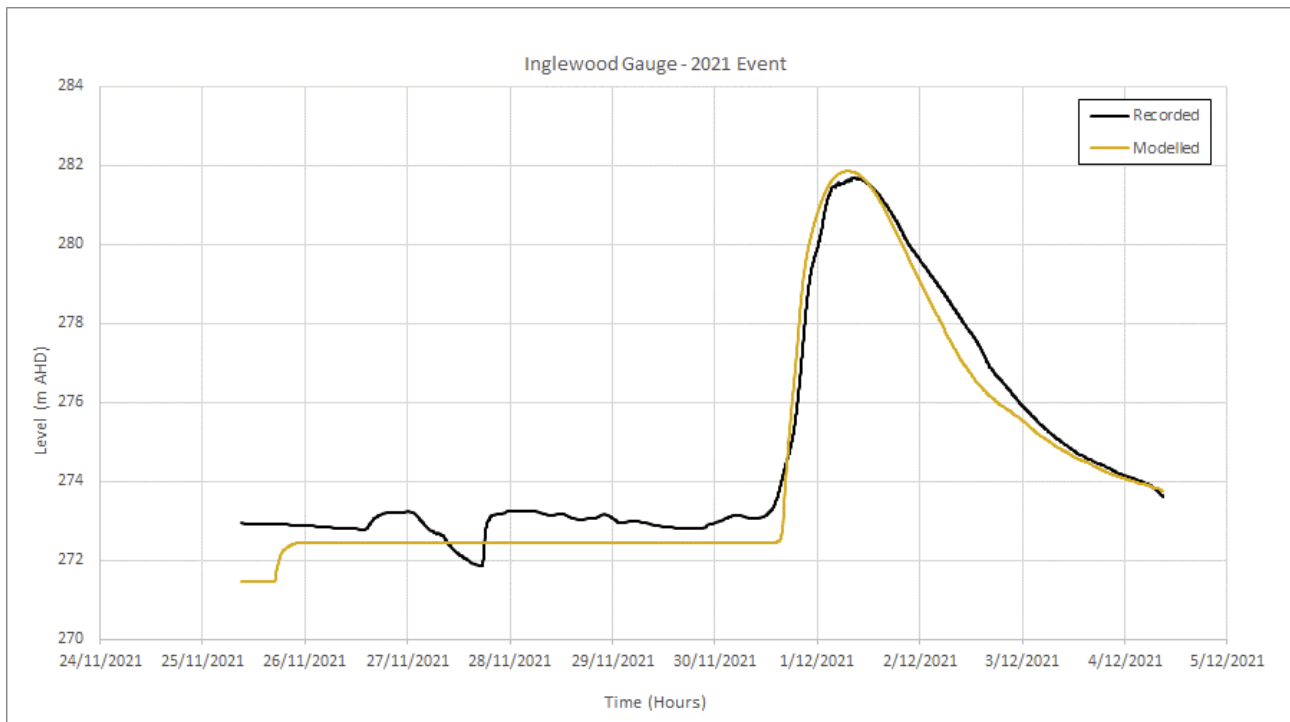
a) Inglewood



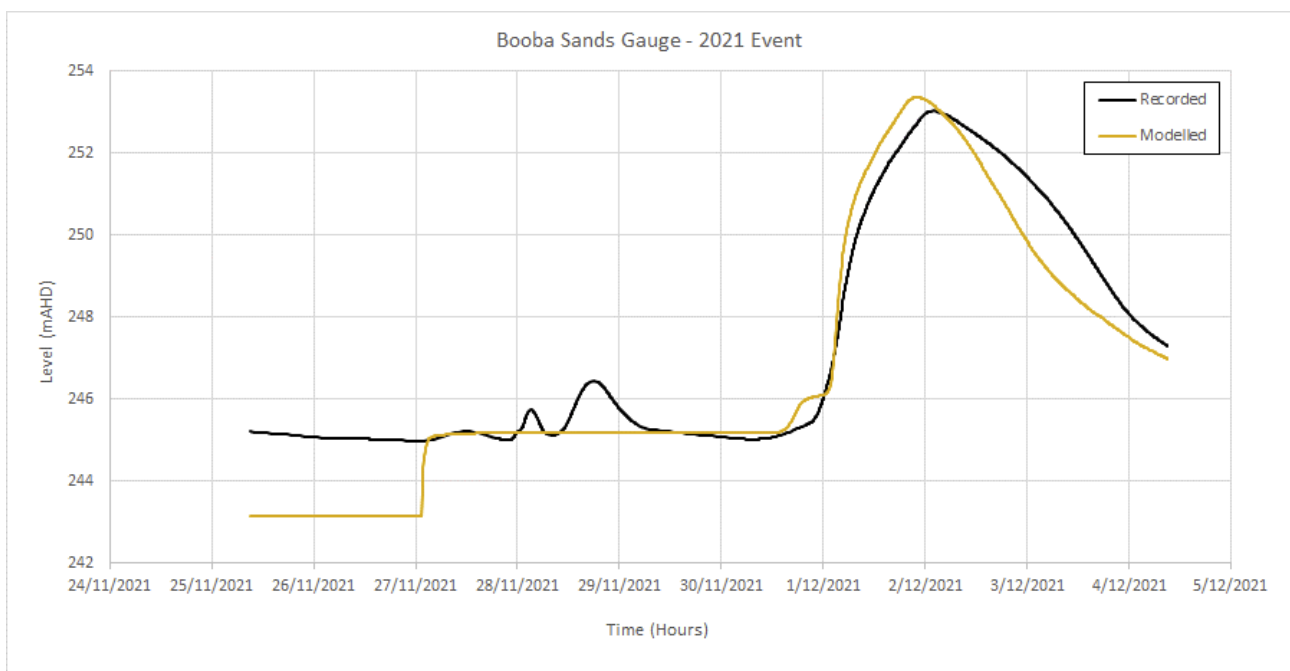
b) Booba Sands

Figure 14.10 Inglewood and Booba Sands: 2021 Stream gauge—model results compared to rated data

Level comparisons are provided in Figure 14.11. The graphs show that there is a good correlation between the modelled and predicted flood levels. Similar to the flow hydrographs presented above, there is a timing shift between the recorded peak flow gauge level and the modelled peak gauge level. Overall, the TUFLOW model gives a reasonable representation of conditions at the stream gauges based on the ratings.



a) Inglewood



b) Booba Sands

Figure 14.11 Inglewood and Booba Sands—2021 flood recorded and modelled gauge levels

14.3.9 Calibration summary

Calibration of the hydrologic and hydraulic modelling of the Macintyre Brook catchment is based on the least reliable data of the three major tributaries of the Border River Catchment, with relatively little detailed rainfall data and issues identified at a number of the stream gauges. The original NSW DPE calibration of the February 1976 flood event (NSW DPE, 2020) achieved a good match of the rated peak flow at Inglewood (although the actual value of this rated peak is questionable) but did not match the overall shape of the hydrograph particularly well. A reasonable match of the hydrographs at the downstream gauges at Inglewood and Booba Sands was achieved for the 1996 event; however, the match at some of the upstream gauges was not as good.

The recalibration has achieved a good representation of the shape of the hydrograph at the primary calibration gauge at Inglewood for the February 1976 flood (although there is some ambiguity over the magnitude due to uncertainty in the gauge rating for this event) as well as a good match of the upstream gauges. Good matches were also obtained for almost all of the available stream gauges for both the 1996 and 2011 flood events. The model is therefore considered to be well calibrated throughout the catchment.

The December 2021 validation event has shown the model provides a good representation of the recent event based on the calibrated floodplain parameters.

14.4 Existing Case modelling results: Macintyre Brook—Yelarbon to Inglewood

14.4.1 Hydrologic model peak flow assessment

A critical duration assessment was undertaken to determine which storm duration(s) produced peak flood flows at the stream flow gauges locations and where the major waterways are intersected by the Project alignment, in particular:

- Canning Creek at Woodspring (just upstream of confluence with Macintyre Brook at Inglewood)
- Macintyre Brook at Inglewood (including Canning Creek)
- Macintyre Brook at Booba Sands (just upstream of Glenarbon and Yelarbon)
- Kippenbung Creek at Yelarbon.

To assess the critical storm duration the following methodology was adopted:

- The models were simulated for a range of AEP events: 20%, 10%, 5%, 2%, 1%, 1 in 2,000, 1 in 10,000 AEP and PMF:
 - Each AEP was modelled for a range of durations
 - Each duration was modelled for each of the 10 associated TPs.
- A critical duration assessment was undertaken at the locations mentioned above to determine which duration produced the highest median flow of the 10 TPs for each event.

Table 14.9 presents the critical duration and TP for all modelled events. It is noted that Yelarbon only has critical durations up to the 1% AEP event. For events larger than the 1% AEP event the Macintyre Brook becomes the dominant flow through Yelarbon.

Table 14.9 Macintyre Brook—Critical durations for varying AEP events

Event	Focal point	Critical duration (hrs)	Temporal pattern
20% AEP	Inglewood	24	Pattern 1 (ID #3795)
	Booba Sands	24	Pattern 1 (ID #3795)
	Woodspring	24	Pattern 5 (ID #3779)
	Yelarbon	12	Pattern 5 (ID #2442)
10% AEP	Inglewood	24	Pattern 1 (ID #3795)
	Booba Sands	24	Pattern 1 (ID #3795)
	Woodspring	24	Pattern 2 (ID #3776)
	Yelarbon	9	Pattern 8 (ID #2401)
5% AEP	Inglewood	24	Pattern 1 (ID #3795)
	Booba Sands	24	Pattern 1 (ID #3795)
	Woodspring	48	Pattern 8 (ID #3962)
	Yelarbon	12	Pattern 5 (ID #2429)

Event	Focal point	Critical duration (hrs)	Temporal pattern
2% AEP	Inglewood	24	Pattern 10 (ID #3804)
	Booba Sands	24	Pattern 3 (ID #3797)
	Woodspring	24	Pattern 9 (ID #3783)
	Yelarbon	12	Pattern 9 (ID #2419)
1% AEP	Inglewood	24	Pattern 10 (ID #3804)
	Booba Sands	24	Pattern 3 (ID #3797)
	Woodspring	24	Pattern 9 (ID #3783)
	Yelarbon	12	Pattern 9 (ID #2419)
1 in 2,000 year	Inglewood	36	Pattern 6 (ID #3890)
	Booba Sands	36	Pattern 6 (ID #3890)
	Woodspring	24	Pattern 8 (ID #3782)
1 in 10,000 year	Inglewood	24	Pattern 4 ¹
	Booba Sands	24	Pattern 5 ¹
	Woodspring	18	Pattern 5 ¹
PMF	Inglewood	24	Pattern 9 ¹
	Booba Sands	36	Pattern 3 ¹
	Woodspring	12	Pattern 9 ¹

Table note:

1 Consistent with ARR (2019) recommendations and methodologies, very rare/extreme events have been assessed using an ensemble of Top-10 TPs used for developing the GTSMR AVM as compiled by Don Carroll

Table 14.10 presents the estimated peak flow applied to the hydraulic model at outlets of several sub-catchments found to produce the critical flows across the model for the 1% AEP event. These are Woodspring, Inglewood and Booba Sands.

Table 14.10 Macintyre Brook—1% AEP event peak flow at key locations as applied in the hydraulic model

Focal points	1% AEP event peak flow (m ³ /s)	Critical storm (hrs) duration and temporal pattern
Woodspring	1,284	24 hour, A=1,000 km ² pattern 9 (ID #3783)
Inglewood	3,370	24 hour, A=5,000 km ² pattern 10 (ID #3804)
Booba Sands	2,688	24 hour, A=5,000 km ² pattern 3 (ID #3797)
Yelarbon	121	12 hour, A=1,000 km ² pattern 9 (ID #2419)

14.4.2 Existing Case flood maps

Existing conditions flood maps are presented in Volume 2 – Appendix J and via an [Online Digital Portal](#). Appendix J contains the Existing Case 20% and 1% AEP flood maps, while the [Online Digital Portal](#) contains the remainder of the modelled events. The location of each map set is summarised below:

- 20% AEP peak levels: Figure J3-A
- 1% AEP peak levels: Figure J3-B
- 20% AEP peak velocity: Figure J3-C
- 1% AEP peak velocity: Figure J3-D
- 20% AEP peak hazard: Figure J3-E
- 1% AEP peak hazard: Figure J3-F
- 20% AEP peak time of inundation: Figure J3-G

- 1% AEP peak time of inundation: Figure J3-H
- 20% AEP, 10% AEP, 5% AEP, 2% AEP, 1% AEP, 1 in 2,000 AEP, 1 in 10,000 AEP and PMF Existing Case results: [Online Digital Portal](#)
- For higher resolution mapping refer to the [Online Digital Portal](#).

14.4.3 Existing Case flood inundation extent and flood levels

The Project hydraulic model predicts Existing Case 1% AEP event depths of up to 11 m will occur in the Macintyre Brook main channel through Inglewood. On the floodplain, depths averaging approximately 1.5 m are predicted across the town of Inglewood. Flow remains mainly in the Macintyre Brook up to the 20% AEP event through Inglewood and breakouts occur between a 20% AEP and 10% AEP events.

At Yelarbon, flows are predicted to be mostly contained in the Macintyre Brook up to a 20% AEP event. Flow is predicted to breakout between a 20% AEP and 10% AEP events and flow towards Yelarbon. Under the 1% AEP event the flood depths at Yelarbon are predicted to be around 2 m deep.

Modelling results were presented to GRC during February 2019, and although Council were unaware of any available calibration information for Yelarbon, the modelling results were accepted as reasonable.

Figure J3-B in Volume 2 – Appendix J shows the 1% AEP event indicative flood extent and peak water levels within the Macintyre Brook floodplain for the Existing Case.

14.4.4 Existing Case flood immunity of existing infrastructure

The overtopping depths for all flood events are presented in Table 14.11. The Cunningham Highway is overtopped up to 1 m in the Existing Case 1% AEP event in Yelarbon. There is up to 2.3 m deep inundation predicted on the Cunningham Highway in Inglewood in the 1% AEP event.

Table 14.11 Macintyre Brook—Existing Case: Estimated overtopping depths of key infrastructure

Infrastructure	Location	Maximum overtopping depth (m)							
		PMF	1 in 10,000 AEP	1 in 2,000 AEP	1% AEP	2% AEP	5% AEP	10% AEP	20% AEP
Cunningham Highway	Yelarbon (North)	3.18	1.35	0.97	0.46	0.43	0.35	0.40	0.39
Cunningham Highway	Yelarbon (adjacent levee)	3.75	1.98	1.63	1.00	0.82	0.69	0.62	0.08
Adjacent Existing Road Bridge	Cunningham Highway, Inglewood	9.41	5.06	4.25	2.30	1.79	0.69	0.02	Dry
Existing Road Bridge	Millmerran-Inglewood Road	7.87	3.61	2.92	2.05	1.86	1.32	0.77	Dry

14.4.5 Existing Case velocities

At Inglewood, velocities of approximately 2.0 m/s are predicted across the floodplain area under the 1% AEP event with higher velocities in the Macintyre Brook of up to 2.4 m/s. At Yelarbon, velocities are predicted to be up to 2.0 m/s at the peak of the flood during a 1% AEP event. Generally, the average velocity across the floodplain is approximately 0.5 m/s as shown in Figure J3-D in Volume 2 – Appendix J.

14.4.6 Existing Case hazard

At Inglewood, the average velocity-depth product across the floodplain is 1.25 m²/s in the 1% AEP event. At Yelarbon the average velocity depth across the floodplain is 0.16 m²/s as shown in Figure J3-F in Volume 2 – Appendix J.

14.4.7 Existing Case time of inundation

At Inglewood, the average time of inundation across the floodplain is 25 hours in the 1% AEP event, with some areas on and adjacent to the Macintyre Brook experiencing longer inundation times of up to 4 days. At Yelarbon, the average time of inundation across the floodplain is 26 hours in the 1% AEP event. Furthermore, there are some localised low spots around the town where water appears to pond, including behind the levee, which then extends across to Yelarbon-Keetah Road. These localised areas experience a time of inundation of up to 4 days in the 1% AEP event, as shown in Figure J3-H in Volume 2 – Appendix J.

14.5 Developed Case modelling results: Macintyre Brook—Yelarbon to Inglewood

14.5.1 Drainage structures

The hydraulic design of the major drainage structures was undertaken using the TUFLOW model (1D and 2D approach).

In the Macintyre Brook floodplain from Yelarbon to Inglewood, the Project includes the following floodplain structures (or regional structures):

- 23 RCP locations (a total of 563 cells)
- 13 RCBC locations (a total of 174 cells).

Local drainage structures are presented in Appendix B. These structures were sized through the local drainage design and depending on their interaction with the Macintyre Brook floodplain were incorporated in the hydraulic model.

The proposed bridges are summarised in Table 14.12 and shown in Figure J1-F to J1-H in Volume 2 – Appendix J.

Table 14.12 Macintyre Brook—Proposed rail bridges and detail

Chainage (km)	Structure ID	Approximate span (m)	Deck width (m)	Soffit level (mAHD)	Deck superstructure type	Deck depth (mm)
25.60	310-BR01	104	16.00	253.49	1100 deep deck units + 220 slab	1,750
52.48	310-BR03	207	3.97	276.02	Type D1	2,000
55.45	310-BR04	207	3.97	286.96	Type D1	2,000
60.53	310-BR51	75	3.97	308.36	Type D1	4,450
67.20	310-BR05	345	3.97	284.44	Type D1	2,000

Table note:

- 1 Details regarding cross-drainage structures for Pariagara Creek (a tributary of Macintyre Brook) are reported in Section 13.5.1, and bridges at Bybera Road and Cremascos Road in Sections 15.5.1 and 16.5.1, respectively

Bridges were represented within the TUFLOW model through use of lfcsh. Each bridge within the model has had a flow constriction coefficient applied to represent obstruction of waterway area due to the piers.

Noise barriers were incorporated into the hydraulic model through Yelarbon from approximately Wondalli Street to the west and East Sawmill Street to the east of town. The noise barriers have been hydraulically modelled as impermeable barriers 4 m high on the top of the formation. This has been applied as a topographic modifier or 'z shape' layer in the TUFLOW model.

Blockage of hydraulic structures was assessed in accordance with ARR 2019. No additional blockage was applied to the waterway area. It is recommended that these parameters be revisited during detailed design.

14.5.2 Proposed levee at Yelarbon

The flood impact assessment at Yelarbon has considered Existing Case regional flooding, Developed Case flood impacts and potential mitigation options at Yelarbon. In other locations along the Project alignment, a traditional cross-drainage approach was used to mitigate impact from raising the rail embankment. This approach is not practical through Yelarbon due to the interface with the existing Graincorp silos and rail siding that are proposed to remain operational at the existing ground levels.

Therefore, to minimise the use of cross-drainage culverts, the proposed mitigation option involves a combination of raising the existing Yelarbon flood levee and cross-drainage through the proposed rail embankment (where the interface with the existing Graincorp infrastructure is not an issue and where FSRs are not impacted by cross-drainage flows).

In order to maintain the existing flood immunity, additional works are proposed to the Cunningham Highway to tie in with the proposed levee. The existing cross-drainage structure through the Cunningham Highway at Yelarbon will be maintained in the proposed works.

A number of different levee extents and heights were investigated for the proposed levee at Yelarbon. The two extents which were investigated were 1.3 km (East Levee only) and 1.8 km (East and West Levee connected). For these two extents, raises between 200 mm and to higher than the 1% AEP event flood level (approximately 1.5 m) were investigated. The investigation focused on the 1% AEP event impacts to the Yelarbon Township and the trafficability of the Cunningham Highway but also considered reducing impacts under more frequent events.

The proposed levee extent and raise height is outlined in Figure 14.12, and shown in the impact maps (Figures J5-A to J9-C in Volume 2 – Appendix J).



Figure 14.12 Yelarbon levee—Proposed raise heights

14.5.3 Hydraulic design criteria outcomes

The hydraulic model was run for the Developed Case for the range of events (20%, 10%, 5%, 2%, 1% AEP events and 1 in 2,000 AEP, 1 in 10,000 AEP and PMF events) with the outcomes presented in the following sections.

14.5.3.1 Rail immunity and structures results

Appendix B summarises the peak 1% AEP event water levels at structures, on the upstream side of the Project alignment.

The results of flood modelling indicate that a 1% AEP event flood immunity to the proposed rail formation is achieved for the Project alignment across the Macintyre Brook floodplain. There is over 0.7 m freeboard to the rail formation level for the 1% AEP event, noting that the formation levels were generally dictated by other engineering criteria and constraints.

14.5.4 Flood Impact Objectives outcomes—Macintyre Brook Yelarbon to Inglewood

The impact of the Developed Case was assessed through inclusion of the proposed rail design and comparison of model results against the Existing Case results.

Changes to flooding behaviour that were assessed, and are reported in the following sections, include potential changes in peak water levels, duration of inundation, velocities, hazard and peak flows within the floodplain.

- Changes in 20% AEP and 1% AEP peak water levels are presented in Figures J5-A and J5-B in Volume 2 – Appendix J

- Changes in 20% AEP and 1% AEP velocities are presented in Figures J5-C and J5-D in Volume 2 – Appendix J
- Changes in 20% AEP and 1% AEP hazard are presented in Figures J5-E and J5-F in Volume 2 – Appendix J
- Changes in 20% AEP and 1% AEP duration of inundation are presented in Figures J5-G and J5-H in Volume 2 – Appendix J
- Changes in 20% AEP and 1% AEP velocities and percentage change in velocities with cut-offs applied are presented in Figures J6-A and J6-F in Volume 2 – Appendix J
- Percentage changes in 20% AEP and 1% AEP in hazard with cut-offs applied are presented in Figures J7-A and J7-B in Volume 2 – Appendix J
- Percentage changes in 20% AEP and 1% AEP in time of inundation and with cut-offs applied are presented in Figures J8-A to J8-D in Volume 2 – Appendix J
- Refer to the [Online Digital Portal](#) for other modelled AEP results and higher resolution mapping.

One-on-one consultation with landowners who are expected to experience changes in flooding behaviour on the property was conducted by ARTC supported by FFJV. Refer to EIS Chapter 14: Flooding and Geomorphology and EIS Appendix E: Consultation Report for further details.

The Project design outcomes relative to the FIOs are presented in the following sections.

Flood sensitive receptors referred to in this section are shown in Figures P1 (1 to 59) in Volume 2 – Appendix P.

14.5.4.1 Flood impacts on flood sensitive receptors

Table 14.13 presents the FSR exceedances for up to and including the 1 in 2,000 AEP event. Based on the available aerial imagery, there are 11 houses predicted to be impacted by changes in peak water levels on the Macintyre Brook floodplain up to and including the 1% AEP event. There are three newly wet FSRs in the 2% AEP event, with MCB_ID_507 exceeding the FIO target.

Under the 1 in 2,000 AEP event there is an additional habitable building and five sheds with predicted afflux above 250 mm and one newly wet shed.

Table 14.13 Macintyre Brook—flood sensitive receptors impacted by afflux

FSR ID	FSR type	FIO target (up to 1% AEP) (mm)	FIO target (1 in 2,000 year) (mm)	Afflux (mm)						Exceedance justification	Mitigation
				20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2,000		
MCB_ID_241	House	10	250	-	-	-	-	+20	-	MCB_ID_241, MCB_ID_242 and MCB_ID_243 are all adjacent one another on the southern side (upstream) of the Yelarbon levee. The increases in peak water levels at these FSRs occurs as a result of flow redistribution caused by the raising and the extension of the Yelarbon levee in the proposed design. The levee provides increased flood immunity to a significant portion of Yelarbon, with floodwaters being diverted south west. The raising and extension of the levee at Yelarbon was required to minimise the impact of the Project on the township. As described in Section 14.5.2 of the Technical Report, several configurations of the levee were investigated for the 1% AEP event to minimise impacts on the township as well as maintaining the trafficability of the Cunningham Highway. Exceedances remain at these FSRs due the ability to extend the levee further to the west being constrained due to a number of habitable dwellings that are located south of the levee (including this house). The further west the levee is extended, the greater the adverse impact experienced at these habitable dwellings. A balance between the extent of the levee and increases in water levels was considered. It should be noted the MCB_ID_242 has been identified as an exceedance even though it does not exceed the FIO target of 10 mm due to house becoming newly wet in the design case, which was otherwise dry in the existing case.	Further work on the Yelarbon Levee and its influence on flow behaviour, and impacts on lots within the vicinity of Yelarbon will continue to be investigated at detailed design to determine the optimal solution. This will be undertaken in consultation with Goondiwindi Regional Council to ensure the wider protection of the township is achieved and to minimise impacts on these FSRs. Ongoing consultation will be undertaken with the impacted landholder, to discuss this exceedance on the upstream portion of the lot and agree any site-specific mitigation measures to further reduce impact.
MCB_ID_242	House	10	250	-	-	-	-	+2 ¹	-		
MCB_ID_243	House	10	250	-	-	-	-	+99	-		
MCB_ID_278	House	10	250	-	-	-	-	+20	-	MCB_ID_278, MCB_ID_420, MCB_ID_426 and MCB_ID_469 are all FSRs located adjacent each other in a cluster of lots on the northern side of the proposed levee upgrade. The increase in peak water levels at these FSRs is a result of flow redistribution caused by the raising and the extension of the Yelarbon levee in the proposed design. The levee provides increased flood immunity to a significant portion of Yelarbon with floodwaters being diverted south west. The raising and extension of the levee at Yelarbon was required to minimise the impact of the Project on the township. As described in Section 14.5.2 of the Technical Report, several configurations of the levee were investigated for the 1% AEP event to minimise impacts on the township as well as maintaining the trafficability of the Cunningham Highway. Exceedances remain at these FSRs due the ability to extend the levee further to the west being constrained due to a number of habitable dwellings that are located south of the levee (including this lot). The further west the levee is extended, the greater the adverse impact experienced at these habitable dwellings. A balance between the extent of the levee and increases in water levels was considered. It should be noted the MCB_ID_469 has been identified as an exceedance even though it does not exceed the FIO target of 10 mm due to house becoming newly wet in the design case, which was otherwise dry in the existing case.	Further work on the Yelarbon Levee and its influence on flow behaviour and impacts on lots within the vicinity of Yelarbon will continue to be investigated at detailed design to determine the optimal solution. This will be undertaken in consultation with Goondiwindi Regional Council to ensure the wider protection of the township is achieved and to minimise impacts on these FSRs. Ongoing consultation will be undertaken with the impacted landholder, to discuss this exceedance on the upstream portion of the lot and agree any site-specific mitigation measures to further reduce impact.
MCB_ID_420	House	10	250	-	-	-	-	+31	-		
MCB_ID_426	House	10	250	-	-	-	-	+17	-		
MCB_ID_469	House	10	250	-	-	-	+1 ¹	-	-		
MCB_ID_507	House	10	250	-	-	-	+36 ¹	+24	-	MCB_ID_278, MCB_ID_420, MCB_ID_426 and MCB_ID_469 are all FSRs located adjacent each other in a cluster of Lots to the west of the proposed levee upgrade. The increase in peak water levels at this FSR occurs as a result of flow redistribution caused by the raising and the extension of the Yelarbon levee in the proposed design. The levee provides increased flood immunity to a significant portion of Yelarbon with floodwaters being diverted south west. The raising and extension of the levee at Yelarbon was required to minimise the impact of the Project on the township. As described in Section 14.5.2 of the Technical Report, several configurations of the levee were investigated for the 1% AEP event to minimise impacts on the township as well as maintaining the trafficability of the Cunningham Highway. Exceedances remain at these FSRs due the ability to extend the levee further to the west being constrained due to a number of habitable dwellings that are located south of the levee. The further west the levee is extended, the greater the adverse impact experienced at these habitable dwellings. A balance between the extent of the levee and increases in water levels was considered.	Further work on the Yelarbon Levee and its influence on flow behaviour and impacts on lots within the vicinity of Yelarbon will continue to be investigated at detailed design to determine the optimal solution. This will be undertaken in consultation with Goondiwindi Regional Council to ensure the wider protection of the township is achieved and to minimise impacts on these FSRs. Ongoing consultation will be undertaken with the impacted landowner, to discuss this exceedance on the upstream portion of the Lot and agree any site-specific mitigation measures to further reduce impact.
MCB_ID_508	Shed	100	250	-	-	-	+18 ¹	-	-		
MCB_ID_509	House	10	250	-	-	-	-	+28	-		
MCB_ID_510	House	10	250	-	-	-	-	+29	-		
MCB_ID_519	Shed	100	250	-	-	-	-	-	+103 ¹	This shed is located on a high island upstream of Yelarbon and the proposed levee. The increase in peak water levels at this FSR occurs as a result of flow redistribution caused by the raising and the extension of the Yelarbon levee in the proposed design. The levee provides increased flood immunity to a significant portion of Yelarbon with floodwaters being diverted south west. The raising and extension of the levee at Yelarbon was required to minimise the impact of the Project on the township. As described in Section 14.5.2 of the Technical Report, several configurations of the levee were investigated for the 1% AEP event to minimise impacts on the township as well as maintaining the trafficability of the Cunningham Highway. This shed has been identified as an exceedance due to it becoming newly wet in the 1 in 2,000-year AEP design event. Exceedances remain at this FSR due the ability to extend the levee further to the west being constrained due to a number of habitable dwellings that are located south of the levee. The further west the levee is extended, the greater the adverse impact experienced at these habitable dwellings. A balance between the extent of the levee and increases in water levels was considered.	Further work on the Yelarbon Levee and its influence on flow behaviour and impacts on lots within the vicinity of Yelarbon will continue to be investigated at detailed design to determine the optimal solution. This will be undertaken in consultation with Goondiwindi Regional Council to ensure the wider protection of the township is achieved and to minimise impacts on this FSR. Ongoing consultation will be undertaken with the impacted landowner, to discuss this exceedance on the upstream portion of the Lot and agree any site-specific mitigation measures to further reduce impact.

FSR ID	FSR type	FIO target (up to 1% AEP) (mm)	FIO target (1 in 2,000 year) (mm)	Afflux (mm)						Exceedance justification	Mitigation
				20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2,000		
MCB_ID_583	Shed	100	250	-	-	-	-	-	+548	In the existing case under the 1% AEP event, the house is dry and not impacted by floodwaters. In the existing case 1 in 2,000 AEP event, the house and surrounding area becomes inundated. Similarly, during the developed case 1 in 2,000 AEP event, the FSR becomes inundated and experiences an increase in water level greater than the 250 mm FIO target. The increase in water levels at this FSR is due to limited drainage infrastructure currently proposed in the Project's revised reference design in the vicinity of this FSR. This results in ponding of water on the upstream side of the alignment (causing an increase in water levels at the house). The increase in water levels remain at this FSR due to limited drainage infrastructure currently proposed in the Project design to convey the 1 in 2,000-year AEP event. Currently, proposed cross-drainage is sized to the 1% AEP event and, as stated above, this portion of the alignment is not impacted under the 1% AEP event and therefore no drainage structures have been proposed at this location. A review of cross-drainage design will be undertaken at this location to potentially increase the conveyance under the Project alignment.	A review of the cross-drainage design at this location will be undertaken to potentially increase the conveyance under the Project alignment. Ongoing consultation will be undertaken with the impacted landowner to discuss this exceedance.
MCB_ID_584	House	10	250	-	-	-	-	-	+548	This cluster of 5 FSRs are impacted due to the presence of the rail embankment and Yelarbon Kurumbul Road upgrade creating upstream ponding and an increase in afflux during the 1 in 2,000-year AEP event. In the existing case, under the 1% AEP event, the house and sheds are dry and not impacted by floodwaters. In the existing case 1 in 2,000 AEP event, the house and surrounding area becomes inundated. Similarly, during the developed case 1 in 2,000 AEP event, the FSR becomes inundated and experiences an increase in water level greater than the 250 mm FIO target. The increase in water levels at this FSR is due to limited drainage infrastructure currently proposed in the Project's revised reference design in the vicinity of this FSR. This results in ponding of water on the upstream side of the alignment (causing an increase in water levels at the FSRs). The increase in water levels remain at these FSRs due to limited drainage infrastructure currently proposed in the Project's revised reference design to convey the 1 in 2,000-year AEP event. Currently, proposed cross-drainage is sized to the 1% AEP event and, as stated above, this portion of the alignment is not impacted under the 1% AEP event and therefore no drainage structures have been proposed at this location. A review of cross-drainage design will be undertaken at this location to potentially increase the conveyance under the Project alignment.	A review of the cross-drainage design at this location will be undertaken to potentially increase the conveyance under the Project alignment. Ongoing consultation will be undertaken with the impacted landholder to discuss this exceedance.
MCB_ID_585	Shed	100	250	-	-	-	-	-	+512		
MCB_ID_586	Shed	100	250	-	-	-	-	-	+480		
MCB_ID_587	Shed	100	250	-	-	-	-	-	+398		
MCB_ID_588	Shed	100	250	-	-	-	-	-	+421		
MCB_ID_604	House	10	250	-	-	-	-	+13	-	This FSR is located adjacent to the Cunningham Highway, upstream of the Yelarbon levee. The increase in peak water levels at this FSR is a result of flow redistribution caused by the raising and the extension of Yelarbon levee in the proposed design. In the existing case 1% AEP event, the property and house are inundated by approximately 400 mm. In the developed case 1% AEP event there is an increase in water levels at the house of 13 mm which exceeds the afflux FIO target of 10 mm. Exceedances remain at this FSR due to a balance trying to be obtained between protecting the Yelarbon community through a levee raise and increasing impacts to receptors outside (and upstream of the levee). Due to the design of the levee only being at a reference design stage, further optimisation of the levee raise is required to be complete in detailed design.	Further work on the Yelarbon Levee and its influence on flow behaviour and impacts on lots within the vicinity of Yelarbon will continue to be investigated at detailed design to determine the optimal solution. This will be undertaken in consultation with Goondiwindi Regional Council to ensure the wider protection of the township is achieved and to minimise impacts on this FSR. Ongoing consultation will be undertaken with the impacted landowner, to discuss this exceedance on the upstream portion of the Lot and agree any site-specific mitigation measures to further reduce impact.

Table note:

1 Newly wet FSR

Table 14.14 provides a summary of the FSRs where exceedances in hazard are predicted above the FIO target for all events up to and including the 1% AEP event. There are two sheds where impacts above the FIO target are predicted.

Table 14.14 Macintyre Brook—flood sensitive receptors impacted by an increase in hazard

FSR ID	FSR type	FIO target	Change in hazard (%)				
			20% AEP	10% AEP	5% AEP	2% AEP	1% AEP
MCB_ID_505	Shed	10%	-	-	-	-	15%
MCB_ID_511	Shed	10%	-	-	-	-	18%

The change in duration of inundation has not been assessed for the FSRs as details of floor levels need to be confirmed through survey.

Mitigation of impacts on FSRs is individual to each landowner and will require ongoing consultation with those impacted to agree on appropriate mitigation specific to each site. ARTC will continue through the next stages of design to engage with impacted landowners to negotiate site-specific mitigation measures.

14.5.4.2 Flood impacts on State-controlled roads

Within the extent of the hydraulic model, several State-controlled roads have been identified to be influenced by the Project and flooding up to and including the 1% AEP event. The State-controlled roads influenced by flooding in the Macintyre Brook catchment are:

- Taloom Street
- Wondalli Street
- Cunningham Highway
- Millmerran—Inglewood Road
- Yelarbon—Keetah Road
- Kildonan Road.

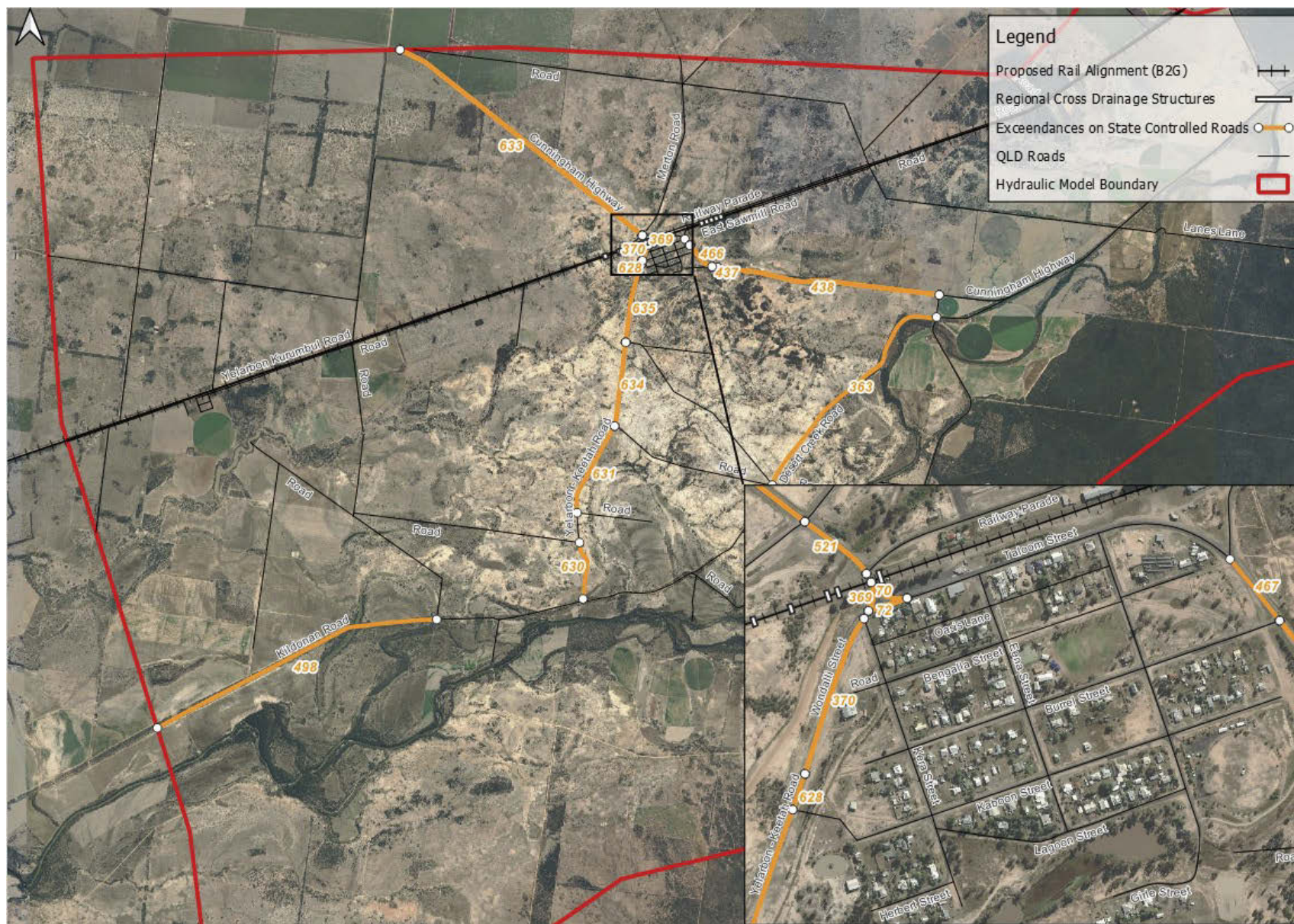
Figure 14.13 and Figure 14.14 presents the locations of potential FIO exceedances for State-controlled roads.

The details for each type of exceedance, including afflux, velocity, time of inundation and hazard, is included in Table 14.15 along with justification and proposed mitigation measures against road segment.

The following provide a summary of the key exceedances:

- Wondalli Street (road section ID #521) has a maximum impact of 640 mm for an extent of 30 m in the 1% AEP event design flood, along with the largest increase in hazard (velocity-depth product) of 36,500 per cent from 0.00 m²/s in the Existing Case to 0.22 m²/s in the Developed Case. Due to the realignment of the Cunningham Highway/Wondalli Street/Taloom St into Yelarbon, this section will be partially realigned and the remainder be closed. The realignment of this section is above the 1% AEP event level.
- Wondalli Street (ID #369) also experiences the largest percentage change of time of inundation increasing from 1.47 hours in the Existing Case to 31.30 hours in the Developed Case (2,033 per cent increase). This portion of Wondalli Street is being upgraded as part of the Project and therefore this impact will be reviewed in detailed design along with proposed road designs.
- Taloom Street (ID #70) has a maximum impact of 490 mm for an extent of 38 m in the 2% AEP design flood. Due to the proposed realignment of the Cunningham Highway/Wondalli Street/Taloom St into Yelarbon, this section will be closed.
- The Cunningham Highway (ID #466) has a maximum impact of 280 mm for an extent of 500 m in the 1% AEP design flood event. At this location the road has been raised to tie into the proposed Yelarbon town levee changes. The developed depth is less than existing for the 1% AEP event. It is noted that at this location the existing depth is 410 mm in the 1% AEP event.

- Yelarbon—Keetah Road (ID #631) has a maximum impact of 300 mm for an extent of 8 m in the 1% AEP design flood event.
- No velocity impacts have been identified for State-controlled roads within the Macintyre Brook modelling domain.



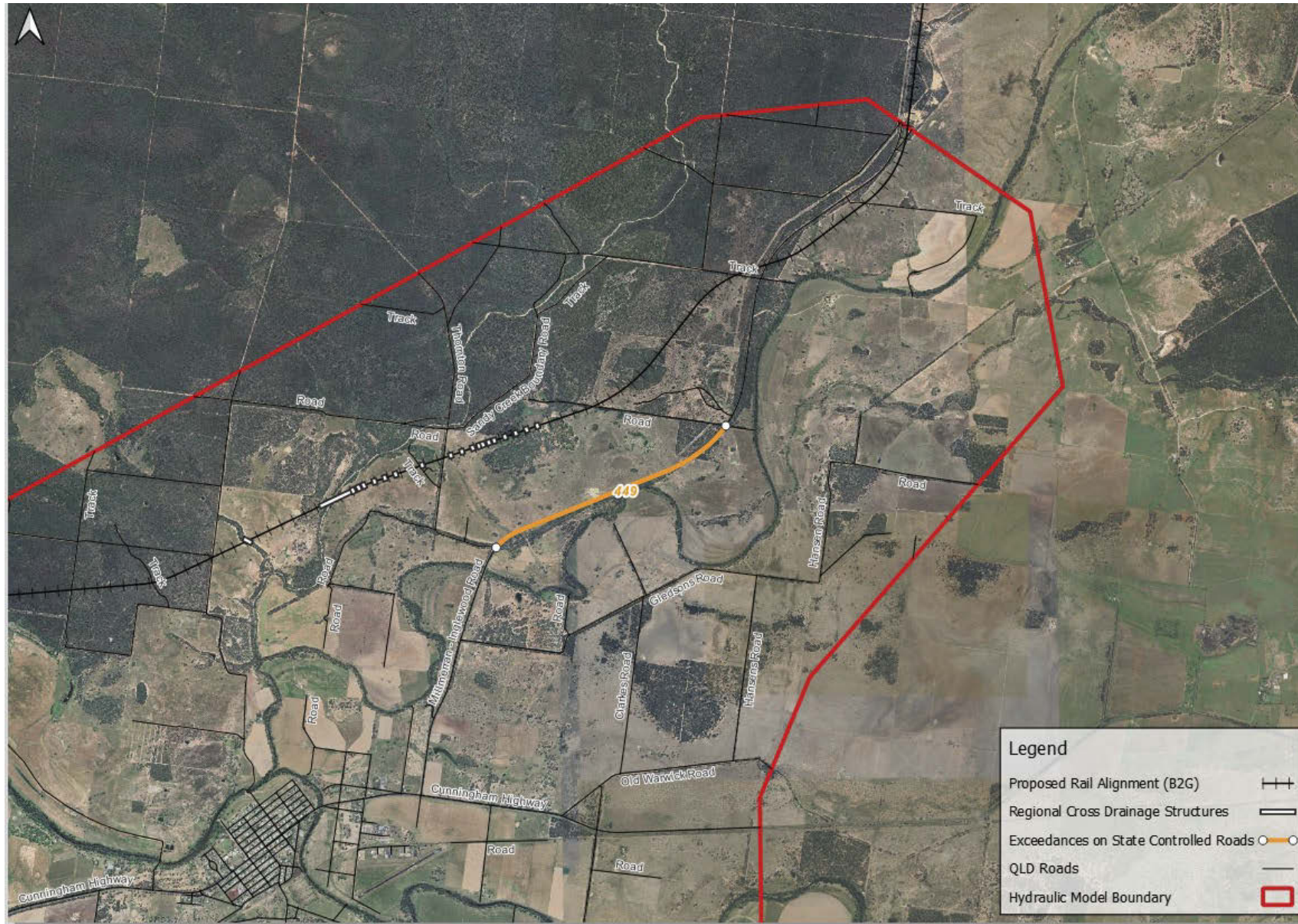


Figure 14.14 Macintyre Brook—Impacted State-controlled roads at Inglewood

Table 14.15 Macintyre Brook - Summary of impacts on State-controlled roads

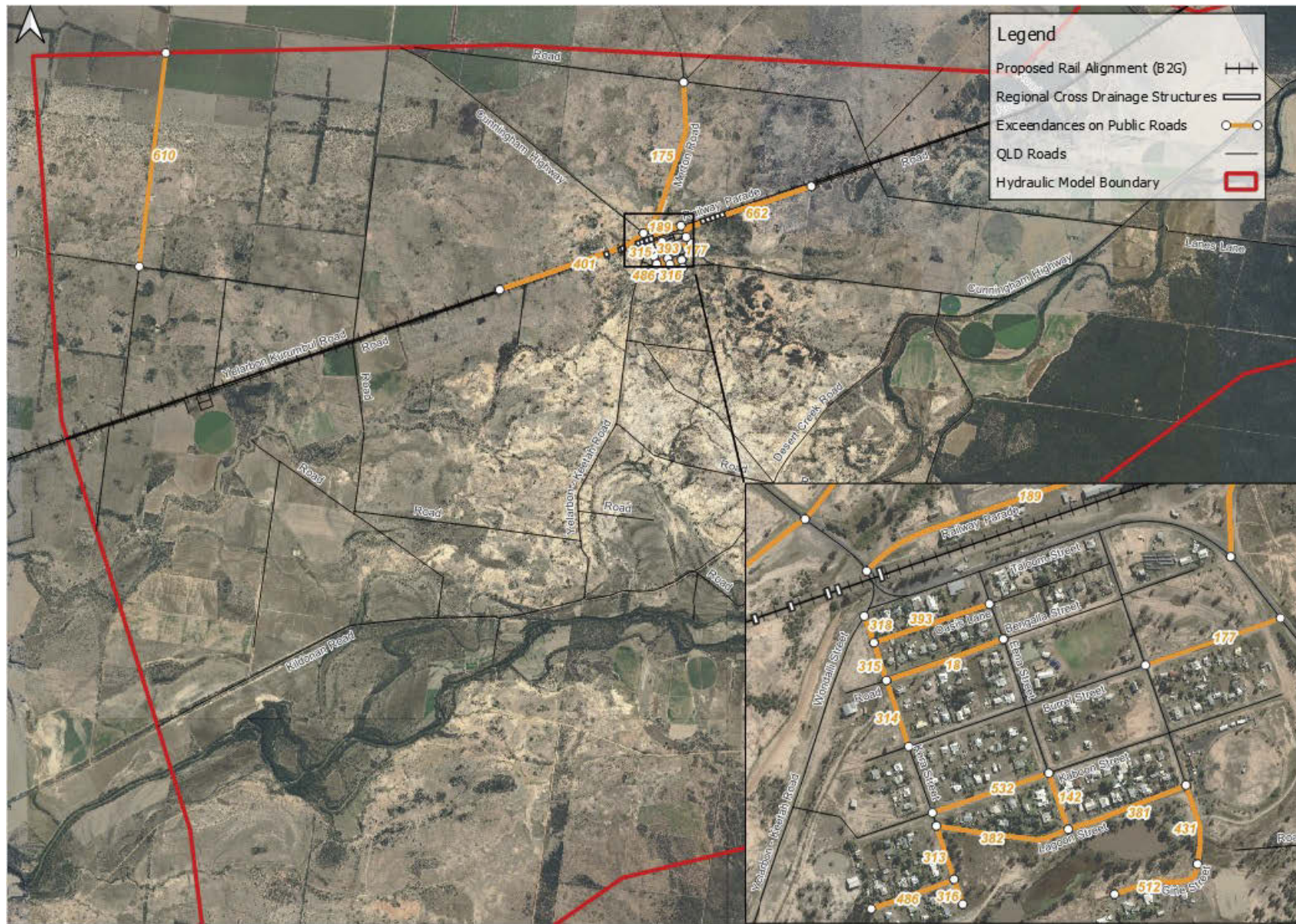
Road Object ID	Road Segment	Road Name	Surface Type	Flood Impact Objective Target					AEP	Maximum FIO Exceedance					Length (m)				Exceedance Justification	Proposed Mitigation Measures ⁹
				Change in Peak Water Level (mm)	Change in Time of inundation (%)	Change in Velocity (%)	Change in Velocity (m/s)	Change in Hazard (%)		Change in Peak Water Level (mm)	Change in Time of inundation (%) Change	Change in Velocity (%)	Change in Velocity (m/s)	Change in Hazard (%)	Afflux	Time on inundation	Velocity	Hazard		
70	1601510399000039048	Taborn Street	Sealed	10	10	20	1	10	20%	451	-	-	-	-	45.00	-	-	-	This segment of Taborn Street is located at the existing Cunningham Highway/Wondall Street/Taborn St intersection which interfaces with the existing rail level crossing. As part of the Project works, significant diversion of this road occurs to remove the level crossing and create an overpass of the rail alignment. As such this segment of Taborn Street becomes redundant and will not be used under developed case conditions. Although an increase in afflux and time of inundation is experienced on this segment, a review of the time of submergence demonstrates a slight improvement. Due to this segment becoming redundant under developed case conditions, the impact is deemed not significant.	Further optimisation of the road/rail interface will be undertaken at this location in the detailed design phase. Therefore as part of the ongoing design optimisation some change in exceedance may occur from that presented in the proposed draft EIS. Further consultation with the State Road Authority will be undertaken to discuss this impact and agree if further mitigation is required.
									5%	473	-	-	-	-	37.50	-	-	-		
									2%	487	-	-	-	-	37.50	-	-	-		
									1%	274	918	-	-	-	15.00	30.00	-	-		
									0.05%	-	-	-	-	-	-	-	-	-		
									20%	152	-	-	-	-	37.50	7.50	-	-		
									10%	173	56	-	-	-	45.00	7.50	-	-		
									5%	187	45	-	-	-	45.00	7.50	-	-		
									2%	117	178	-	-	-	52.50	15.00	-	-		
									1%	-	50	-	-	-	-	30.00	-	-		
72	1601510399000039050	Taborn Street	Sealed	10	10	20	1	10	20%	152	-	-	-	-	37.50	7.50	-	This portion of Taborn Street is proposed to be upgraded as part of the Project with the design varying in height by approximately +/- 100mm from the existing case scenario. The impacts on this road segment are due to the introduction of the new road upgrade within the area having a localised increase in water level from the existing case, due to the redistribution of flood waters. Although there may be some localised increases in afflux and time of inundation, a review of the time of submergence results for all modelled events from the 20% to 1% AEP events, shows that with the proposed design, a significant reduction, and improvement, on the operational capacity of this segment can be achieved through the proposed upgrades.	It is expected that reduction in exceedances on this segment of road can be achieved in the detailed design phase through further optimisation of road cross drainage structures (with particular focus on optimising culvert location, culvert spread, culvert invert levels and grades). Ongoing consultation with the State Road Authority will be undertaken to discuss the road design, associated flood impacts and any site specific mitigations required to minimise the impact the operational capacity of the road.	
									5%	187	-	-	-	-	7.50	-	-			-
									2%	117	178	-	-	-	52.50	15.00	-			-
									1%	-	50	-	-	-	-	30.00	-			-
									0.05%	-	-	-	-	-	-	-	-			-
									20%	52	11	-	-	-	15.00	7.50	-			-
									10%	173	-	-	-	-	7.50	-	-			-
									5%	187	-	-	-	-	7.50	-	-			-
									2%	88	178	-	-	-	22.50	-	-			-
									1%	-	42	-	-	-	-	22.50	-			-
366	1601510399000039093	Wondall Street	Sealed	10	10	20	1	10	20%	213	1467	-	-	-	52.50	7.50	-	This portion of Wondall Street is proposed to be upgraded as part of the Project with the design varying in height by approximately +/- 100mm from the existing case scenario. The impacts on this road segment are due to the introduction of the new road upgrade within the area having a localised increase in water level from the existing case, due to the redistribution of flood waters. The increase in road height, plus overlapping, results in a greater increase in water level on the segment.	It is expected that reduction in exceedances on this segment of road can be achieved in the detailed design phase through further optimisation of road cross drainage structures (with particular focus on optimising culvert location, culvert spread, culvert invert levels and grades). Ongoing consultation with the State Road Authority will be undertaken to discuss the road design, associated flood impacts and any site specific mitigations required to minimise the impact the operational capacity of the road.	
									10%	233	2032	-	-	-	52.50	22.50	-			-
									5%	235	1194	-	-	-	52.50	22.50	-			-
									2%	249	816	-	-	-	52.50	30.00	-			-
									1%	259	239	-	-	-	7.50	37.50	-			-
									0.05%	-	-	-	-	-	-	-	-			-
									20%	343	2009	-	-	-	67.50	127.50	-			-
									10%	366	1969	-	-	-	75.00	187.50	-			-
									5%	376	1076	-	-	-	232.50	210.00	-			-
									2%	473	-	-	-	-	157.50	-	-			-
370	1601510399000039091	Wondall Street	Sealed	10	10	20	1	10	20%	213	-	-	-	-	187.50	-	-	This segment of Wondall Street is located at the existing Cunningham Highway/Wondall Street/Taborn St intersection which interfaces with the existing rail level crossing. As part of the Project works, significant diversion of this road occurs to remove the level crossing and create an overpass of the rail alignment. As such this segment of Wondall Street becomes redundant and will not be used under developed case conditions. Due to this segment becoming redundant under developed case conditions, the impact is deemed not significant.	Further optimisation of the road/rail interface will be undertaken at this location in the detailed design phase. Therefore as part of the ongoing design optimisation some change in exceedance may occur from that presented in the proposed draft EIS. Further consultation with the State Road Authority will be undertaken to discuss this impact and agree if further mitigation is required.	
									1%	-	213	-	-	-	-	187.50	-			-
									0.05%	-	-	-	-	-	-	-	-			-
									20%	97	-	-	-	-	30.00	-	-			-
									10%	26	22	-	-	-	75.00	7.50	-			-
									5%	64	99	-	-	-	150.00	67.50	-			-
									2%	150	91	-	-	-	107	315.00	187.50			-
									1%	280	136	-	-	-	88	517.50	232.50			-
									0.05%	-	-	-	-	-	-	-	-			-
									466	1601510399000039094	Cunningham Highway	Sealed	10	10	20	1	10			20%
10%	366	1969	-	-	-	75.00	187.50	-										-		
5%	376	1076	-	-	-	232.50	210.00	-										-		
2%	473	-	-	-	-	157.50	-	-										-		
1%	-	213	-	-	-	-	187.50	-										-		
0.05%	-	-	-	-	-	-	-	-										-		
20%	97	-	-	-	-	30.00	-	-										-		
10%	26	22	-	-	-	75.00	7.50	-										-		
5%	64	99	-	-	-	150.00	67.50	-										-		
2%	150	91	-	-	-	107	315.00	187.50										-		
521	1601510399000039092	Wondall Street	Sealed	10	10	20	1	10	20%	343	2009	-	-	-	67.50	127.50	-	This portion of Wondall Street is located at the existing Cunningham Highway/Wondall Street/Taborn St intersection which interfaces with the existing rail level crossing. As part of the Project works, significant diversion of this road occurs to remove the level crossing and create an overpass of the rail alignment. As such this segment of Wondall Street becomes redundant and will not be used under developed case conditions. Due to this segment becoming redundant under developed case conditions, the impact is deemed not significant.	Further optimisation of the road/rail interface will be undertaken at this location in the detailed design phase. Therefore as part of the ongoing design optimisation some change in exceedance may occur from that presented in the proposed draft EIS. Further consultation with the State Road Authority will be undertaken to discuss this impact and agree if further mitigation is required.	
									10%	366	1969	-	-	-	75.00	187.50	-			-
									5%	376	1076	-	-	-	232.50	210.00	-			-
									2%	473	-	-	-	-	157.50	-	-			-
									1%	-	213	-	-	-	-	187.50	-			-
									0.05%	-	-	-	-	-	-	-	-			-
									20%	97	-	-	-	-	30.00	-	-			-
									10%	26	22	-	-	-	75.00	7.50	-			-
									5%	64	99	-	-	-	150.00	67.50	-			-
									2%	150	91	-	-	-	107	315.00	187.50			-
633	1601510399000039093	Cunningham Highway	Sealed	10	10	20	1	10	20%	343	2009	-	-	-	67.50	127.50	-	This portion of Wondall St (ID 370) is partially proposed to be upgraded as part of the Project. Exceedances recorded are based on the centreline of the existing road, although the new centreline of the upgrade is slightly offset east by approximately 20m. This offset of centrelines results in the current centreline lying along the toe of the embankment of the proposed upgrade. This area experiences a localised increase in water level and time of inundation due to the presence of the embankment and hence the recorded exceedances. It is not anticipated the exceedances recorded will impact the new-upgraded road and therefore are not deemed significant.	Further optimisation of road cross drainage structures (with particular focus on optimising culvert location, culvert spread, culvert invert levels and grades) will be undertaken in the Detailed Design to mitigate the exceedances experienced along this segment. Further consultation with the State Road Authority will be undertaken to discuss this impact and agree if further mitigation is required.	
									10%	366	1969	-	-	-	75.00	187.50	-			-
									5%	376	1076	-	-	-	232.50	210.00	-			-
									2%	473	-	-	-	-	157.50	-	-			-
									1%	-	213	-	-	-	-	187.50	-			-
									0.05%	-	-	-	-	-	-	-	-			-
									20%	97	-	-	-	-	30.00	-	-			-
									10%	26	22	-	-	-	75.00	7.50	-			-
									5%	64	99	-	-	-	150.00	67.50	-			-
									2%	150	91	-	-	-	107	315.00	187.50			-
639	1601510399000039093	Cunningham Highway	Sealed	10	10	20	1	10	20%	343	2009	-	-	-	67.50	127.50	-	This portion of Wondall St (ID 521) is partially upgraded and partially redundant as part of the proposed road upgrade works. In the developed case the redundant portion of this section of Wondall Street (adjacent to existing Railway Fife intersection) is located and a flow path is created. The reported afflux exceedances occur in this location due to the introduction of the drainage channel and culvert that were dry in the existing case becoming newly flood affected in the developed case. The increase in time of inundation is due to a decrease in road height in the design case at the intersection of Wondall Street and Yelarbon Kurumbi Road. This increase in time of inundation increases the time of submergence of the segment by 10 hours in the 1% AEP event. It is not anticipated that the increase is significant and detrimental to access around Yelarbon due to both Cunningham Highway to the north-west and Yelarbon Kurumbi Road to the south-west having greater time of inundation in the existing case than the predicted value of this segment in the design case.	Further optimisation of the road design and road cross drainage structures (with particular focus on optimising culvert location, culvert spread, culvert invert levels and grades) will be undertaken in the Detailed Design to mitigate the exceedances experienced along this segment. Further consultation with the State Road Authority will be undertaken to discuss this impact and agree if further mitigation is required.	
									10%	401	-	-	-	-	45.00	-	-			-
									5%	405	-	-	-	-	45.00	-	-			-
									2%	461	-	-	-	-	30.00	-	-			-
									1%	636	546	-	-	-	30.00	45.00	-			-
									0.05%	-	-	-	-	-	-	-	-			-
									20%	26	14	-	-	-	7.50	7.50	-			-
									10%	19	29	-	-	-	7.50	22.50	-			-
									5%	79	-	-	-	-	37.50	-	-			-
									2%	98	-	-	-	-	217.50	-	-			-
438	1601510399000039092	Cunningham Highway	Sealed	10	10	20	1	10	20%	26	14	-	-	-	7.50	7.50	-	This section of the Cunningham Highway (ID 633) is partially upgraded and realigned with a portion of the existing highway becoming redundant as part of the proposed road upgrade works. The main area of exceedances are clustered around the realignment works, to the north-west of Yelarbon which experiences an increase in time of inundation. This can be attributed to the change in flood dynamics in this area through the realignment of the new road embankments. A review of the time of submergence for this segment of road, demonstrates that the exceedances identified have little impact on the operational capacity of the overall segment with change time of submergence being negligible. Therefore it is not deemed that the increase in time of inundation at individual cells significantly impact the road.	Through optimisation of road cross drainage structures (with particular focus on optimising culvert location, culvert spread, culvert invert levels and grades) it is anticipated some minor localised exceedance will remain in this area. Further consultation with the State Road Authority will be undertaken to discuss this impact and agree if further mitigation is required.	
									10%	19	29	-	-	-	7.50	22.50	-			-
									5%	79	-	-	-	-	37.50	-	-			-
									2%	98	-	-	-	-	217.50	-	-			-
									1%	87	-	-	-	-	75.00	-	-			-
									0.05%	-	-	-	-	-	-	-	-			-
									20%	-	-	-	-	-	-	-	-			-
									10%	16	-	-	-	-	15.00	-	-			-
									5%	171	-	-	-	-	30.00	-	-			-
									2%	134	158	-	-	-	240.00	60.00	-			-
628	1601510399000039478	Yelarbon - Keelah Road	Sealed	10	10	20	1	10	20%	31	45	-	-	-	45.00	45.00	-	This section of Yelarbon Keelah Road (ID 628) is impacted due to upgrades on Wondall Street and the Cunningham Highway diversion. Floodwaters move from south to north, as they approach these road upgrades they are obstructed by the proposed embankments which causes afflux and increases in time of inundation. These impacts propagate upstream ultimately impacting Yelarbon Keelah Road. A review of time of submergence results across this segment demonstrate that an increase of approximately 12 hours occurs from the existing case. Although this localised increase may be observed, a wider review of the Yelarbon Keelah Road as a whole, demonstrates that further upstream the road is in fact improved for a much greater duration (approximately 30 hours) in the existing case. Therefore the localised increase experienced due to the presence of the Project is not deemed significant and/or will impact the operational capacity of the road as a whole.	Further optimisation of the road design and road cross drainage structures (with particular focus on optimising culvert location, culvert spread, culvert invert levels and grades) will be undertaken in the Detailed Design to mitigate the exceedances experienced along this segment. Some reduction in exceedance is expected. Further consultation with the State Road Authority will be undertaken to discuss this impact and agree if further mitigation is required.	
									10%	-	-	-	-	-	-	-	-			-
									5%	-	-	-	-	-	-	-	-			-
									2%	-	-	-	-	-	-	-	-			-
									1%	-	-	-	-	-	-	-	-			-
									0.05%	-	-	-	-	-	-	-	-			-
									20%	-	-	-	-	-	-	-	-			-
									10%	-	-	-	-	-	-	-	-			-
									5%	-	-	-	-	-	-	-	-			-
									2%	-	-	-	-	-	-	-	-			-
363	1601510399000010023	Desert Creek Road	Unsealed	10	10	10	0.5	10	20%	-	-	-	-	-	-	-	-	Desert Creek Rd is located over 30m away from the rail alignment. The percentage change in time of inundation exceedance under the 1% AEP only is an isolated cell located away from the main drainage paths. No other exceedances are reported on this road and therefore this exceedance can be considered a modelling artefact and not representative of an actual exceedance. It is deemed that this impact will not be detrimental to the operational capacity of this road.	Further consultation with the State Road Authority will be undertaken to discuss this impact and agree if further mitigation is required.	
									10%	-	-	-	-	-	-	-	-			-
									5%	-	-	-	-	-	-	-	-			-
									2%	-	-	-	-	-	-	-	-			-
									1%	-	-	-	-	-	-	-	-			-
									0.05%	-	-	-	-	-	-	-	-			-
									20%	-	-	-	-	-	-	-	-			-
									10%	-	-	-	-	-	-	-	-			-
									5%	-	-	-	-	-	-	-	-			-
									2%	-	-	-	-	-	-	-	-			-
636	1601510399000039475	Yelarbon - Keelah Road	Sealed	10	10	20	1	10	20%	-	-	-	-	-	-	-	-	This section of Yelarbon Keelah Road (ID 628) is impacted due to upgrades on Wondall Street and the Cunningham Highway diversion. Floodwaters move from south to north, as they approach these road upgrades they are obstructed by the proposed embankments which causes afflux and increases in time of inundation. These impacts propagate upstream ultimately impacting Yelarbon Keelah Road. A review of time of submergence results across this segment demonstrate that an increase of approximately 12 hours occurs from the existing case. Although this localised increase may be observed, a wider review of the Yelarbon Keelah Road as a whole, demonstrates that further upstream the road is in fact improved for a much greater duration (approximately 30 hours) in the existing case. Therefore the localised increase experienced due to the presence of the Project is not deemed significant and/or will impact the operational capacity of the road as a whole.	Further optimisation of the road design and road cross drainage structures (with particular focus on optimising culvert location, culvert spread, culvert invert levels and grades) will be undertaken in the Detailed Design to mitigate the exceedances experienced along this segment. Some reduction in exceedance is expected. Further consultation with the State Road Authority will be undertaken to discuss this impact and agree if further mitigation is required.	
									10%	-	-	-	-	-	-	-	-			-
									5%	-	-	-	-	-	-	-	-			-
									2%	-	-	-	-	-	-	-	-			-
									1%	-	-	-	-	-	-	-	-			-
									0.05%	-	-	-	-	-	-	-	-			-
									20%	-	-	-	-	-	-	-	-			-
									10%	-	-	-	-	-	-	-	-			-
									5%	-	-	-	-	-	-	-	-			-
									2%	-	-	-	-	-	-	-	-			-
437	1601510399000039091	Cunningham Highway	Sealed	10	10	20	1	10	20%	-	-	-	-	-	-	-	-	This section of the Cunningham Highway (ID 633) is partially upgraded and realigned with a portion of the existing highway becoming redundant as part of the proposed road upgrade works. The main area of exceedances are clustered around the realignment works, to the north-west of Yelarbon which experiences an increase in time of inundation. This can be attributed to the change in flood dynamics in this area through the realignment of the new road embankments. A review of the time of submergence for this segment of road, demonstrates that the exceedances identified have little impact on the operational capacity of the overall segment with change time of submergence being negligible. Therefore it is not deemed that the increase in time of inundation at individual cells significantly impact the road.	Through optimisation of road cross drainage structures (with particular focus on optimising culvert location, culvert spread, culvert invert levels and grades	

14.5.4.3 Flood impacts on local public roads

Within the extent of the hydraulic model, 36 local public roads are influenced by flooding. Figure 14.15 to Figure 14.17 present the location of the impacted local public roads. Table 14.16 identifies the location of potential FIO exceedances and the details for each exceedance, including afflux, velocity, time of inundation and hazard along with justification and proposed mitigation measures for each.

The following provides a summary of the key exceedances:

- Railway Parade (road section ID #189) shows the greatest impacts, with a maximum afflux of approximately 610 mm, and for an extent of 8 m for the 1% AEP design flood event. Afflux impacts occur at the Wondalli Street intersection, where Wondalli Street has been realigned. This segment of road also experiences the largest increase in hazard impacts (19,500 per cent in the 2% AEP event), although this percentage change maybe large it is due to the Existing Case hazard results being negligible ($<0.01 \text{ m}^2/\text{s}$) with a jump to $0.15 \text{ m}^2/\text{s}$ in the Developed Case. This impact extends for 37.5 m.
- Yelarbon-Kurumbul Road (ID #401) has a maximum impact of approximately 470 mm, and for an extent of 98 m for the 2% AEP design flood event.
- Kera Street (ID #318) has a maximum impact of approximately 170 mm (depth of 172 mm), and for an extent of 53 m for the 5% AEP design flood. This segment of Kera Street also experiences the largest increase in time of inundation of 4,500 per cent increasing from 0.88 hours in the Existing Case to 40.38 hours in the Developed Case. The Project is proposing to upgrade this portion of Kera Street; therefore, the changes indicated are likely due to changes in the road design, height increases and alignment, contributing to significant increases from the Existing Case. These exceedances will be reviewed further in the detailed design stage.
- Minor afflux and time of inundation exceedances are noted for all other local public roads.



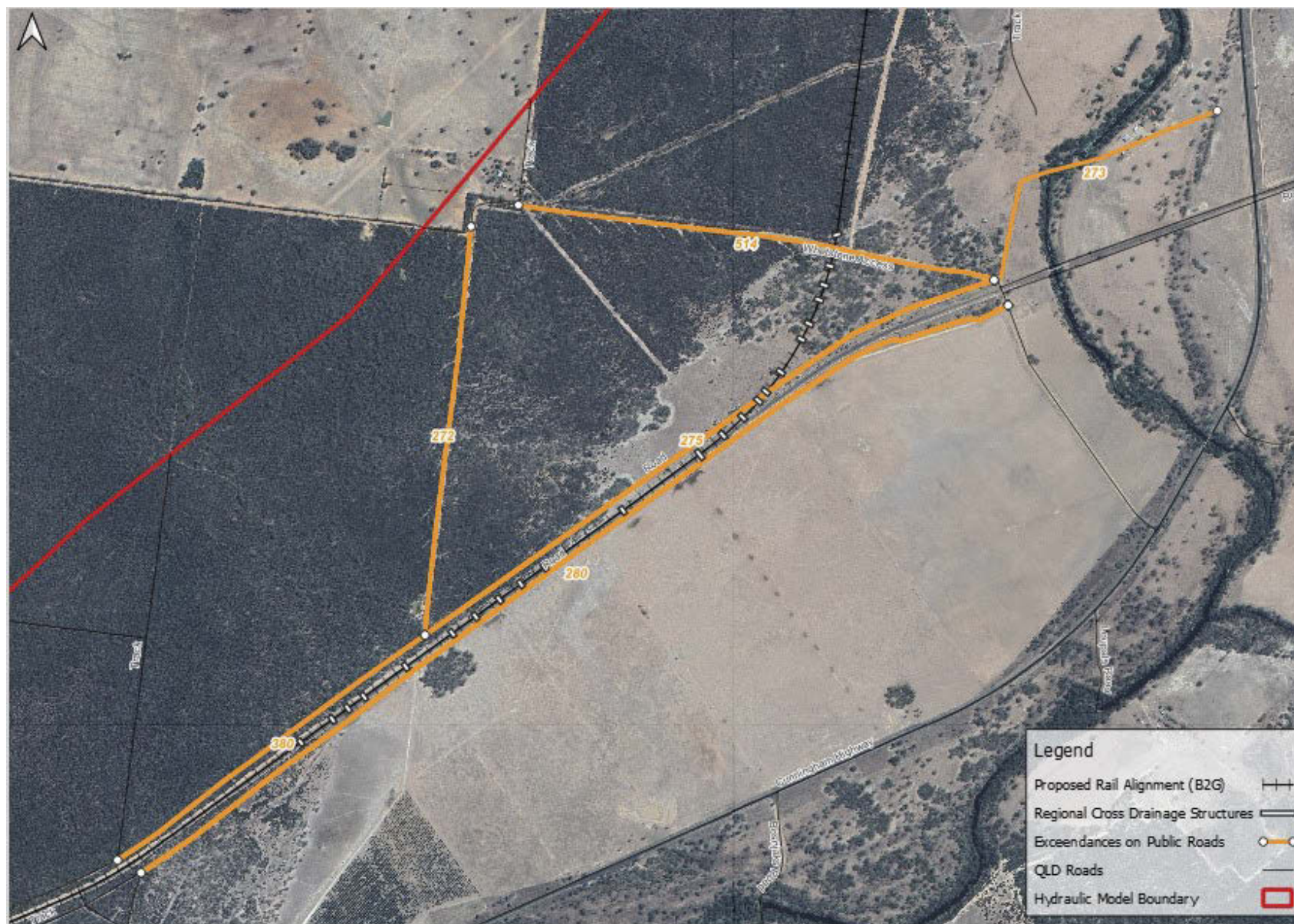


Figure 14.16 Macintyre Brook—Impacted local public roads at Whetstone

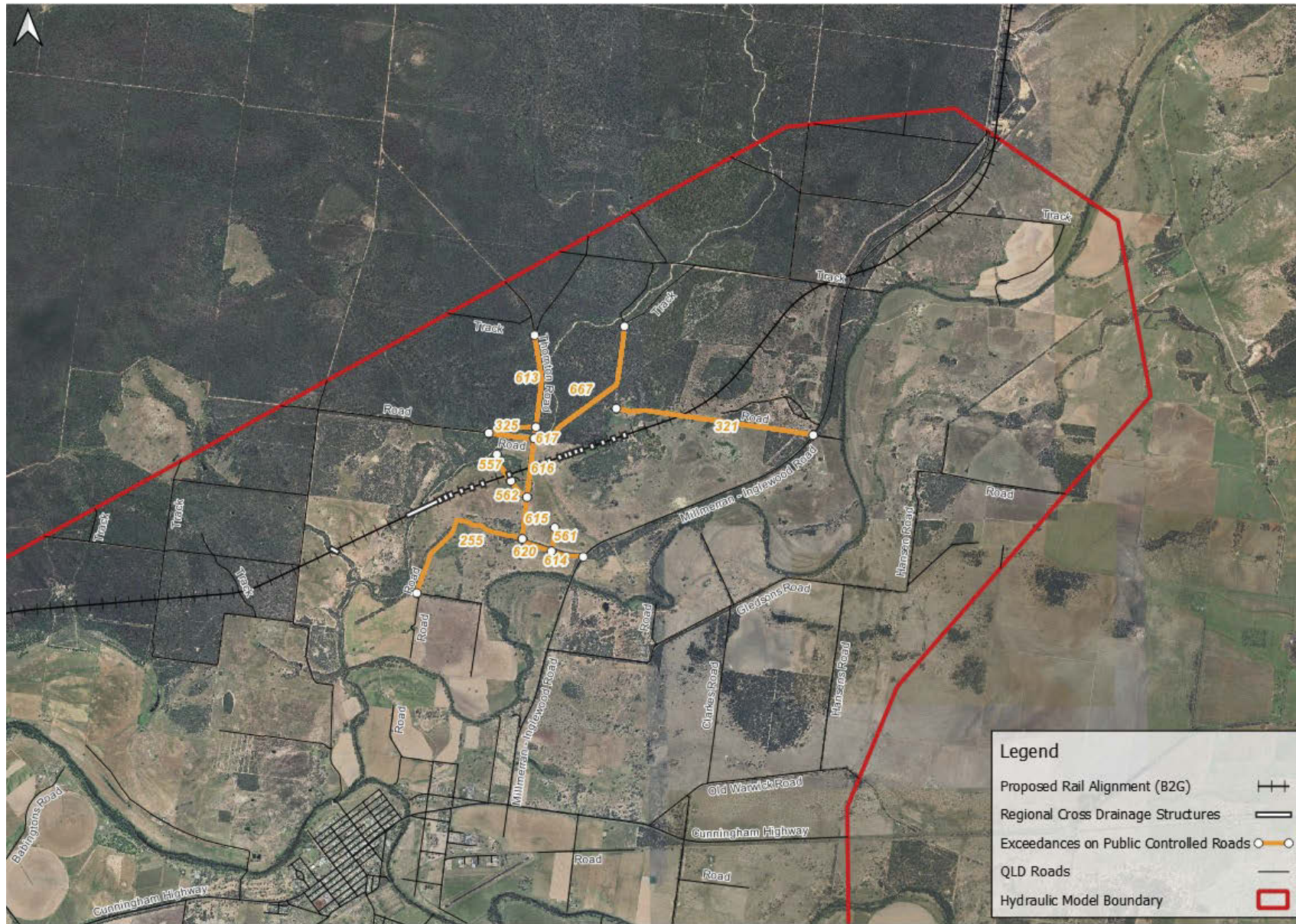


Figure 14.17 Macintyre Brook—Impacted local public roads at Inglewood

Table 14.16 Macintyre Brook – Summary of impacts on local public roads

Road Object ID	Road Segment	Road Name	Surface Type	Flood Impact Objective Target					AEP	Maximum FID Exceedance					Length (m)				Exceedance Justification	Proposed Mitigation Measures ^a	
				Change in Peak Water Level (mm)	Change in Time of Inundation (%)	Change in Velocity (m/s)	Change in Velocity (m/s)	Change in Hazard (%)		Change in Peak Water Level (mm)	Change in Time of Inundation (% Change) ^b	Change in Velocity (m/s) ^c	Change in Velocity (m/s)	Change in Hazard (%) ^d	Afflux	Time on Inundation	Velocity	Hazard			
18	1601510399000029007	Bungalla Street	Sealed	10	10	20	1	10	20%	25	17	-	-	7.50	7.50	-	-	-	-	Small section of impact Near Kera Street junction. Maturity of current cross-drainage design is limited by available information (e.g. detailed survey)	Further road and/or rail design optimization investigations are required during detailed design.
									10%	42	18	-	-	7.50	7.50	-	-	-	-		
									5%	49	17	-	-	7.50	7.50	-	-	-	-		
									2%	-	16	-	-	-	15.00	-	-	-	-		
									1%	-	-	-	-	-	-	-	-	-	-		
189	1601510399000029075	Railway Parade	Sealed	10	10	20	1	10	20%	280	615	-	-	30.00	22.50	-	-	-	-	Railway Parade is subject to realignment at the intersection with the Cunningham Highway resulting in a portion of the existing road becoming redundant. In the developed case the redundant portion of this section of Railway Parade (adjacent to existing Woodville St intersection) is lowered and a flow path is created. The reported afflux exceedances occur in this location due to the introduction of the drainage channel and cuts that were dry in the existing case becoming newly flood affected in the developed case. Time of submergence increases experienced on the road are due to redistribution of flow due to the presence of the alignment and road upgrades. Although increases on some locations have been identified, the overall time of submergence of the road and modelled events has been improved and therefore these increases are not deemed significant.	Further optimisation of road cross drainage structures (with particular focus on culvert location, culvert spread, culvert invert levels and grade) will be undertaken in Detailed Design.
								10%	358	500	-	-	15.00	37.50	-	-	-	-			
									5%	383	375	-	-	15.00	37.50	-	-	-	-		
									2%	423	401	-	-	13992	97.50	37.50	-	-			
									1%	607	203	-	-	4877	7.50	397.50	37.50	-	-		
314	1601510399000018795	Kera Street	Sealed	10	10	20	1	10	20%	18	-	-	-	7.50	-	-	-	-	-	Minor isolated impact (1 cut)	Stakeholder Consultation (Landowner/Government)
									10%	12	-	-	-	7.50	-	-	-	-	-		
									5%	-	-	-	-	-	-	-	-	-	-		
									2%	-	-	-	-	-	-	-	-	-	-		
									1%	-	18	-	-	-	15.00	-	-	-	-		
315	1601510399000018796	Kera Street	Sealed	10	10	20	1	10	20%	76	1299	-	-	15.00	60.00	-	-	-	-	This portion of Kera Street is proposed to be upgraded as part of the Project with the design varying in height by approximately +/- 100mm from the existing case scenario. The impacts on this road segment are due to the introduction of the new road upgrades within the area having a localized increase in water level from the existing case, due to the redistribution of flood waters. The increase in road height, plus overtopping, results in a greater increase in water level on the segment.	Exceedances still remain on this segment due to the maturity of the road and cross drainage design being at a reference design level with further optimisation to be completed. Optimisation of the road design as well as cross drainage design will be completed in the detailed design stage to finalise road height, rail and road cross drainage placement and sizing.
								10%	97	994	-	-	30.00	60.00	-	-	-	-			
									5%	105	580	-	-	75.00	52.50	-	-	-	-		
									2%	-	283	-	-	-	45.00	-	-	-	-		
									1%	-	153	-	-	-	52.50	-	-	-	-		
318	1601510399000018799	Kera Street	Sealed	10	10	20	1	10	20%	120	1244	-	-	15.00	22.50	-	-	-	-	Similar to ID 315, this portion of Kera Street is proposed to be upgraded as part of the Project with the design varying in height by approximately +/- 100mm from the existing case scenario. The impacts on this road segment are due to the introduction of the new road upgrades within the area having a localized increase in water level from the existing case, due to the redistribution of flood waters. The increase in road height, plus overtopping, results in a greater increase in water level on this segment.	Exceedances still remain on this segment due to the maturity of the road and cross drainage design being at a reference design level with further optimisation to be completed. Optimisation of the road design as well as cross drainage design will be completed in the detailed design stage to finalise road height, rail and road cross drainage placement and sizing.
									10%	162	4606	-	-	45.00	37.50	-	-	-	-		
									5%	172	1716	-	-	52.50	37.50	-	-	-	-		
									2%	-	348	-	-	-	67.50	-	-	-	-		
									1%	-	209	-	-	-	60.00	-	-	-	-		
321	1601510399000013094	Road 0	Unsealed	10	10	10	0.5	10	20%	30	275	-	-	300.00	15.00	-	-	-	-	Inspection of the aerial imagery indicates that this road does not currently exist and is a potential future road corridor. This local road is impacted due to the presence of the rail embankment and Thornton Road upgrades, restricting flow within the area and leading to an increase in water level. Within the vicinity of this road 11 culvert crossings are proposed across the embankment ranging from 2 cuts through to 40 cuts. Currently due to the maturity of the design and hydraulic modelling, these culverts have been grouped into banks. This grouping can result in a restriction of flow crossing the alignment which can cause local increases in water level. As design progresses and modelling updates are made based on the recommendations of the Expert Flood Panel, it is envisaged upstream impacts on this road can be reduced. A review of the time of submergence results for this segment shows negligible change between the existing and design case and therefore the identified exceedances are not anticipated to impact the operation capacity of this road.	Further optimisation of cross drainage will be undertaken in the detailed design phase to mitigate these exceedances including allocation to culvert location and spread and potential increase in culvert cell numbers. Ongoing consultation with the road manager will be undertaken to discuss the future operational capacity of this road as well as any residual exceedances. Where appropriate, site specific mitigation measures will be discussed to further reduce impacts on the future road corridor.
									10%	44	-	-	315.00	-	-	-	-	-	-		
									5%	45	-	-	315.00	-	-	-	-	-	-		
									2%	52	143	-	-	337.50	22.50	-	-	-	-		
									1%	84	27	-	-	337.50	7.50	-	-	-	-		
324	1601510399000013318	Road 0	Unsealed	10	10	10	0.5	10	20%	50	-	-	-	30.00	-	-	-	-	-	Maturity of current cross-drainage design is limited by available information (e.g. detailed survey)	Further road and/or rail design optimization investigations are required during detailed design.
									10%	20	11	-	495.00	15.00	7.50	-	-	-	-		
									5%	31	-	-	0.02	15	112.50	-	30.00	45.00	-		
									2%	124	-	-	0.04	14	195.00	-	75.00	-	-		
									1%	28	-	-	0.06	17	37.50	-	67.50	127.50	-		
401	1601510399000056446	Yelarbon Kurumbul Road	Sealed	10	10	20	1	10	20%	396	435	-	-	210.00	52.50	-	-	-	-	This section of the Yelarbon Kurumbul Road is partially upgraded and realigned with a portion of the existing road becoming redundant as a part of the proposed road upgrade works. The main area of exceedances are clustered around the realignment works at the intersection with the Cunningham Highway. These can be attributed to the change in flood dynamics in this area through the introduction of the new road embankments including the Cunningham Highway diversion. The placement of the Cunningham Highway diversion embankment creates afflux directly adjacent Yelarbon Kurumbul Road causing afflux and time of inundation increases on the upgraded area. It is expected that these increases will impact the trafficability of the road during operations and therefore further mitigation is required to reduce / not deteriorate these exceedances. It is anticipated that a reduction in exceedances can occur through road design and cross drainage design optimisations.	Further optimisation of the road designs (including overtopping) in the area will be undertaken in the detailed design stage to balance the road design interfaces with flood impacts. In addition, review of road cross drainage in this area will also be undertaken to balance flow/overflows across proposed embankments (eg Cunningham Highway to the north of Yelarbon). Cross drainage improvements could include increasing culvert sizes and culvert numbers, realignment of culverts to existing flowpaths where possible and spread culverts under embankments to balance flow/overflows and reduce impacts.
									10%	435	2017	-	-	219.00	90.00	-	-	-	-		
									5%	437	951	-	-	225.00	90.00	-	-	-	-		
									2%	469	1315	-	-	97.50	157.50	-	-	-	-		
									1%	-	736	-	-	-	285.00	-	-	-	-		
616	1601510399000036378	Thornton Road	Unsealed	10	10	10	0.5	10	20%	27	102	-	-	97.50	22.50	-	-	-	-	This local road is proposed to be upgraded and raised as part of the project. The increase in height of the road results in upstream afflux which propagates across the northern and southern portions of the upgraded road. These areas also experience time of inundation, velocity and hazard increases. Currently due to the maturity of the road and rail cross drainage design, proposed culverts have been grouped together rather than spread along the embankment under the replication of the existing flow regime. As design progresses and modelling updates are made based on the recommendations of the Expert Flood Panel, it is envisaged upstream impacts on this road can be reduced. A review of the time of submergence of results in the existing and design case demonstrate that although there are localised increases on this road in the design case the overall operational capacity is not impacted with minimal change and in some cases improvement from the existing case.	In detailed design, improvements to the hydraulic model will be undertaken to capture detailed survey and address outstanding comments made by the Expert Flood Panel. In addition, a review and optimisation of the cross drainage design at this location will be undertaken to spread culverts along the embankment and improve the conveyance downstream of the alignment. It is anticipated some reduction in exceedances can occur along this road segment. In addition, consultation with the responsible Road Manager will be undertaken in accordance with the Flood Impact Objectives, to discuss impacts to this road and agree on any site specific mitigation measures.
									10%	57	23	-	-	120.00	52.50	-	-	-	-		
									5%	54	-	-	0.14	63	127.50	-	15.00	120.00	-		
									2%	60	230	-	0.22	47	199.00	-	45.00	-	-		
									1%	175	163	-	0.20	81	217.50	-	45.00	270.00	-		
662	1601510399000010940	East Sawmill Road	Sealed	10	10	20	1	10	20%	148	-	-	-	210.00	-	-	-	-	-	This section of East Sawmill Road is proposed to be realigned 10m to the south as part of the project. In the design case water pond between the new realigned road and rail embankment causing increases in water level and time of inundation. Due to the realignment of this road from its current location, these exceedances remain and are not anticipated to impact the operational capacity of the road as this section will be redundant during the operation phase of the project.	It is not proposed to undertake mitigation for these exceedances due to road segment becoming redundant in the operational phase of the project. Ongoing consultation with the Local Road authority will be undertaken to discuss the road design, associated flood impacts and any site specific mitigations (including overtopping) in the area will be undertaken to balance flow/overflows across proposed embankments (eg Cunningham Highway to the north of Yelarbon). Cross drainage improvements could include increasing culvert sizes and culvert numbers, realignment of culverts to existing flowpaths where possible and spread culverts under embankments to balance flow/overflows and reduce impacts.
									10%	211	182	-	-	262.50	37.50	-	-	-	-		
									5%	79	170	-	-	75.00	120.00	-	-	-	-		
									2%	12	52	-	-	479	7.50	22.50	65.00	-	-		
									1%	108	250	-	-	220	138.00	37.50	270.00	-	-		
393	1601510399000012087	Oxley Lane	Sealed	10	10	20	1	10	20%	-	16	-	-	-	7.50	-	-	-	-	Small section of impact Near Kera Street junction. Maturity of current cross-drainage design is limited by available information (e.g. detailed survey)	Further road and/or rail design optimization investigations are required during detailed design.
									10%	-	17	-	-	-	7.50	-	-	-	-		
									5%	18	15	-	-	15.00	-	-	-	-	-		
									2%	-	20	-	-	-	7.50	-	-	-	-		
									1%	-	-	-	-	-	-	-	-	-	-		
325	1601510399000013319	Road 0	Unsealed	10	10	10	0.5	10	20%	63	10	-	-	150.00	-	-	-	-	-	Maturity of current cross-drainage design is limited by available information (e.g. detailed survey)	Further road and/or rail design optimization investigations are required during detailed design.
									10%	29	-	-	150.00	11	-	-	-	-	-		
									5%	33	10	-	-	285.00	-	-	22.50	-	-		
									2%	39	-	-	0.02	13	105.00	-	22.50	172.50	-		
									1%	-	-	-	-	-	-	-	-	-	-		
613	1601510399000016375	Thornton Road	Unsealed	10	10	10	0.5	10	20%	10	11	-	-	15.00	7.50	-	-	-	-	Maturity of current cross-drainage design is limited by available information (e.g. detailed survey)	Further road and/or rail design optimization investigations are required during detailed design.
									10%	31	11	-	-	105.00	30.00	-	-	-	-		
									5%	110	11	-	-	105.00	21.50	-	-	-	-		
									2%	120	22	-	-	322.50	90.00	-	-	-	-		
									1%	50	33	-	-	-	-	-	-	-	-		
617	1601510399000016379	Thornton Road	Unsealed	10	10	10	0.5	10	20%	16	-	-	-	150.00	-	-	-	-	-	Maturity of current cross-drainage design is limited by available information (e.g. detailed survey)	Further road and/or rail design optimization investigations are required during detailed design.
									10%	30	-	-	0.02	136.00	12	-	-	-	-		
									5%	36	-	-	0.03	150.00	-	-	7.50	7.50	-		
									2%	33	-	-	0.03	150.00	-	-	-	-	-		
									1%	39	-	-	-	13	150.00	-	-	30.00	-		
175	1601510399000013087	Merton Road	Sealed	10	10	20	1	10	20%	-	10	-	-	-	7.50	-	-	-	-	Maturity of current cross-drainage design is limited by available information (e.g. detailed survey)	Further road and/or rail design optimization investigations are required during detailed design.
									10%	-	18	-	-	150.00	-	-	-	-	-		
									5%	-	34	-	-	495.00	-	-	-	-	-		
									2%	-	39	-	-	810.00	-	-	-	-	-		
									1%	-	-	-	-	-	-	-	-	-	-		
273	1601510399000012547	Road 0	Unsealed	10	10	10	0.5	10	20%	-	56	-	-	-	15.00	-	-	-	-	Whetstone road level crossing. Maturity of current cross-drainage design is limited by available information (e.g. detailed survey)	Further road and/or rail design optimization investigations are required during detailed design.
									10%	-	-	-	-	-	-	-	-	-	-		
									5%	-	-	-	-	-	-	-	-	-	-		
									2%	-	-	-	-	-	-	-	-	-	-		
									1%	-	-	-	-	-	-	-	-	-	-		
275	1601510399000012549	Road 0	Unsealed	10	10	10	0.5	10	20%	77	197	-	-	1207.50	625.00	-	-	-	-	Whetstone road level crossing. Maturity of current cross-drainage design is limited by available information (e.g. detailed survey)	Further road and/or rail design optimization investigations are required during detailed design.
									10%	34	261	-	0.52	552.50	231	171.50	7.50	600.00	-		
									5%	102	68	-	0.38	87	2407.50	510.00	30.00	697.50			

Road Object ID	Road Segment	Road Name	Surface Type	Flood Impact Objective Target					AEP	Maximum FID Exceedance					Length (m)				Exceedance Justification	Proposed Mitigation Measures ⁸				
				Change in Peak Water Level (mm)	Change in Time of Inundation (%)	Change in Velocity (%)	Change in Velocity (m/s)	Change in Hazard (%)		Change in Peak Water Level (mm)	Change in Time of Inundation (% Change) ²	Change in Velocity (m/s) ³	Change in Velocity (m/s)	Change in Hazard (m/s) ⁴	Afflux	Time on Inundation	Velocity	Hazard						
382	16015102990000197248	Lagoon Street	Sealed	10	10	20	1	10		20%	-	-	-	-	-	-	-	-	-	Impacted by Yelarbon Levee raising.	Further road and or rail design optimization investigations are required during detailed design			
										10%	-	-	-	-	-	-	-	-	-			-	-	-
										5%	-	-	-	-	-	-	-	-	-			-	-	-
										2%	64	340	-	-	75.00	7.50	-	-	-			-	-	
										1%	50	43	-	-	172.50	225.00	-	7.50	-			-	-	
										0.05%	-	-	-	-	-	-	-	-	-			-	-	
										20%	-	-	-	-	-	-	-	-	-			-	-	
										10%	-	-	-	-	-	-	-	-	-			-	-	
										5%	-	-	-	-	-	-	-	-	-			-	-	
										2%	22	-	-	-	7.50	-	-	-	-			-	-	
486	16015102990000161972	Herbert Street	Sealed	10	10	20	1	10		20%	-	-	-	-	-	-	-	-	-	Impacted by Yelarbon Levee raising.	Further road and or rail design optimization investigations are required during detailed design			
										10%	-	-	-	-	-	-	-	-	-			-	-	
										5%	-	-	-	-	-	-	-	-	-			-	-	
										2%	25	54	-	-	150.00	142.50	-	-	-			-	-	
										1%	-	-	-	-	-	-	-	-	-			-	-	
										0.05%	-	-	-	-	-	-	-	-	-			-	-	
										20%	-	-	-	-	-	-	-	-	-			-	-	
										10%	-	-	-	-	-	-	-	-	-			-	-	
										5%	-	-	-	-	-	-	-	-	-			-	-	
										2%	-	-	-	-	-	-	-	-	-			-	-	
512	16015102990000139645	Coke Street	Sealed	10	10	20	1	10		20%	-	-	-	-	-	-	-	-	-	Impacted by Yelarbon Levee raising.	Further road and or rail design optimization investigations are required during detailed design			
										10%	-	-	-	-	-	-	-	-	-			-	-	
										5%	-	-	-	-	-	-	-	-	-			-	-	
										2%	63	-	-	-	93	112.50	-	-	-			-	-	
										1%	102	41	-	-	50	217.50	97.50	-	-			-	-	
										0.05%	-	-	-	-	-	-	-	-	-			-	-	
										20%	-	-	-	-	-	-	-	-	-			-	-	
										10%	-	-	-	-	-	-	-	-	-			-	-	
										5%	-	-	-	-	-	-	-	-	-			-	-	
										2%	-	-	-	-	-	-	-	-	-			-	-	
532	16015102990000183010	Kuboon Street	Sealed	10	10	20	1	10		20%	-	-	-	-	-	-	-	-	-	Impacted by Yelarbon Levee raising.	Further road and or rail design optimization investigations are required during detailed design			
										10%	-	-	-	-	-	-	-	-	-			-	-	
										5%	-	-	-	-	-	-	-	-	-			-	-	
										2%	86	36	-	-	45.00	22.50	-	-	-			-	-	
										1%	-	-	-	-	-	-	-	-	-			-	-	
										0.05%	-	-	-	-	-	-	-	-	-			-	-	
										20%	-	-	-	-	-	-	-	-	-			-	-	
										10%	-	-	-	-	-	-	-	-	-			-	-	
										5%	-	-	-	-	-	-	-	-	-			-	-	
										2%	-	-	-	-	-	-	-	-	-			-	-	
615	160151029900001963277	Thornton Road	Unsealed	10	10	10	0.5	10		20%	-	-	-	-	-	-	-	-	-	Maturity of current cross-drainage design is limited by available information (e.g. detailed survey)	Further road and or rail design optimization investigations are required during detailed design			
										10%	-	-	-	-	-	-	-	-	-			-	-	
										5%	-	-	-	-	-	-	-	-	-			-	-	
										2%	28	145	-	-	150.00	75.00	-	-	-			-	-	
										1%	82	-	-	-	547.50	-	202.50	547.50	-			-	-	
										0.05%	-	-	-	0.07	45	-	-	-	-			-	-	
										20%	-	-	-	-	-	-	-	-	-			-	-	
										10%	-	-	-	-	-	-	-	-	-			-	-	
										5%	-	-	-	-	-	-	-	-	-			-	-	
										2%	-	-	-	-	-	-	-	-	-			-	-	
177	16015102990000155681	Burrell Street	Sealed	10	10	20	1	10		20%	-	-	-	-	-	-	-	-	-	J cell impacted. Anomaly	Stakeholder Consultation (Landowner/ Government)			
										10%	-	-	-	-	-	-	-	-	-			-	-	
										5%	-	-	-	-	-	-	-	-	-			-	-	
										2%	-	24	-	-	-	-	-	-	-			-	-	
										1%	-	-	-	-	-	-	-	-	-			-	-	
										0.05%	-	-	-	-	-	-	-	-	-			-	-	
										20%	-	-	-	-	-	-	-	-	-			-	-	
										10%	-	-	-	-	-	-	-	-	-			-	-	
										5%	-	-	-	-	-	-	-	-	-			-	-	
										2%	-	-	-	-	-	-	-	-	-			-	-	
272	160151029900001432546	Road 0	Unsealed	10	10	10	0.5	10		20%	-	-	-	-	-	-	-	-	-	Whetstone road level crossing. Maturity of current cross-drainage design is limited by available information (e.g. detailed survey)	Further road and or rail design optimization investigations are required during detailed design			
										10%	-	-	-	-	-	-	-	-	-			-	-	
										5%	-	-	-	-	-	-	-	-	-			-	-	
										2%	-	26	-	-	-	-	-	-	-			-	-	
										1%	21	29	-	-	375.00	15.00	-	-	-			-	-	
										0.05%	-	-	-	-	37.50	37.50	-	-	-			-	-	
										20%	-	-	-	-	-	-	-	-	-			-	-	
										10%	-	-	-	-	-	-	-	-	-			-	-	
										5%	-	-	-	-	-	-	-	-	-			-	-	
										2%	-	-	-	-	-	-	-	-	-			-	-	
514	16015102990000138466	Whetstone Access	Unsealed	10	10	10	0.5	10		20%	-	-	-	-	-	-	-	-	-	Whetstone road level crossing. Maturity of current cross-drainage design is limited by available information (e.g. detailed survey)	Further road and or rail design optimization investigations are required during detailed design			
										10%	-	-	-	-	-	-	-	-	-			-	-	
										5%	-	-	-	-	-	-	-	-	-			-	-	
										2%	-	15	-	-	-	-	-	-	-			-	-	
										1%	140	-	-	-	210.00	7.50	-	-	-			-	-	
										0.05%	-	-	-	-	-	-	-	-	-			-	-	
										20%	-	-	-	-	-	-	-	-	-			-	-	
										10%	-	-	-	-	-	-	-	-	-			-	-	
										5%	-	-	-	-	-	-	-	-	-			-	-	
										2%	-	-	-	-	-	-	-	-	-			-	-	
610	16015102990000188993	Queen Street	Unsealed	10	10	10	0.5	10		20%	-	-	-	-	-	-	-	-	-	Impacted by Yelarbon Levee raising.	Further road and or rail design optimization investigations are required during detailed design			
										10%	-	-	-	-	-	-	-	-	-			-	-	
										5%	-	-	-	-	-	-	-	-	-			-	-	
										2%	-	16	-	-	-	-	-	-	-			-	-	
										1%	-	-	-	-	-	-	-	-	-			-	-	
										0.05%	-	-	-	-	-	-	-	-	-			-	-	
										20%	-	-	-	-	-	-	-	-	-			-	-	
										10%	-	-	-	-	-	-	-	-	-			-	-	
										5%	-	-	-	-	-	-	-	-	-			-	-	
										2%	-	-	-	-	-	-	-	-	-			-	-	
142	16015102990000111737	Eena Street	Sealed	10	10	20	1	10		20%	-	-	-	-	-	-	-	-	-	Impacted by Yelarbon Levee raising.	Further road and or rail design optimization investigations are required during detailed design			
										10%	-	-	-	-	-	-	-	-	-			-	-	
										5%	-	-	-	-	-	-	-	-	-			-	-	
										2%	-	-	-	-	-	-	-	-	-			-	-	
										1%	36	25	-	-	46	37.50	82.50	-	-			-	-	
										0.05%	-	-	-	-	-	-	-	-	-			-	-	
										20%	-	-	-	-	-	-	-	-	-			-	-	
										10%	-	-	-	-	-	-	-	-	-			-	-	
										5%	-	-	-	-	-	-	-	-	-			-	-	
										2%	-	-	-	-	-	-	-	-	-			-	-	
614	160151029900001963276	Thornton Road	Unsealed	10	10	10	0.5	10		20%	-	-	-	-	-	-	-	-	-	Maturity of current cross-drainage design is limited by available information (e.g. detailed survey)	Further road and or rail design optimization investigations are required during detailed design			
										10%	-	-	-	-	-	-	-	-	-			-	-	
										5%	-	-	-	-	-	-	-	-	-			-	-	
										2%	-	17	-	-	-	-	-	-	-			-	-	
										1%	-	-	-	-	-	-	-	-	-			-	-	
										0.05%	-	-	-	-	-	-	-	-	-			-	-	
										20%	-	-	-	-	-	-	-	-	-			-	-	
										10%	-	-	-	-	-	-	-	-	-			-	-	
										5%	-	-	-	-	-	-	-	-	-			-	-	
										2%	-	-	-	-	-	-	-	-	-			-	-	
620	160151029900001963274	Thornton Road	Unsealed	10	10	10	0.5	10		20%	-	-	-	-	-	-	-	-	-	Maturity of current cross-drainage design is limited by available information (e.g. detailed survey)	Further road and or rail design optimization investigations are required during detailed design			
										10%	-	-	-	-	-	-	-	-	-			-	-	
										5%	-	-	-	-	-	-	-	-	-			-	-	
										2%	-	-	-	-	-	-	-	-	-			-	-	
										1%	35	-	-	-	11	480.00	-	-	-			-	-	
										0.05%	-	-	-	-	-	-	-	-	-			-	-	
										20%	-	-	-	-	-	-	-	-	-			-	-	
										10%	-	-	-	-	-	-	-	-	-			-	-	
										5%	-	-	-	-	-	-	-	-	-			-	-	
										2%	-	-	-	-	-	-	-	-	-			-	-	

14.5.4.4 Flood impacts on existing rail infrastructure

Figure 14.18 and Figure 14.19 present the location of the existing rail infrastructure. Table 14.17 presents the exceedances along with associated justification and mitigation measures. There are six exceedance locations predicted on the existing rail infrastructure up to and including the 1% AEP event.

The following provides a summary of the key exceedances:

- There are four sections of existing rail infrastructure predicted to be impacted in Yelarbon (rail section IDs #4911, #7708, #10846 and #16879). Up to a maximum of 100 mm of afflux is predicted in the 10% AEP flood event on rail section ID # 7708. These sections of existing rail are being replaced by the Project. The impacts predicted appear to be at the edges of the existing rail, and due to grid sizes of the hydraulic model, these impacts may be associated with lower elevations outside of the Developed Case embankment. Further consideration of these locations will be undertaken in detailed design.
- Up to 282 mm of afflux in the 5% AEP flood event is predicted on the existing rail at Whetstone (rail section IDs #7261 and #7796). An increase in time of inundation is also predicted from 3 hours to 35 hours in 2% AEP event, although along this segment of rail the maximum time of submergence in the existing and design cases is approximately 55 hours. Therefore, the calculated exceedance will have minimal effect on the operational capability of this segment of the rail line due to other portions already being inundated.



Figure 14.18 Macintyre Brook—Impacted existing rail infrastructure at Yelarbon

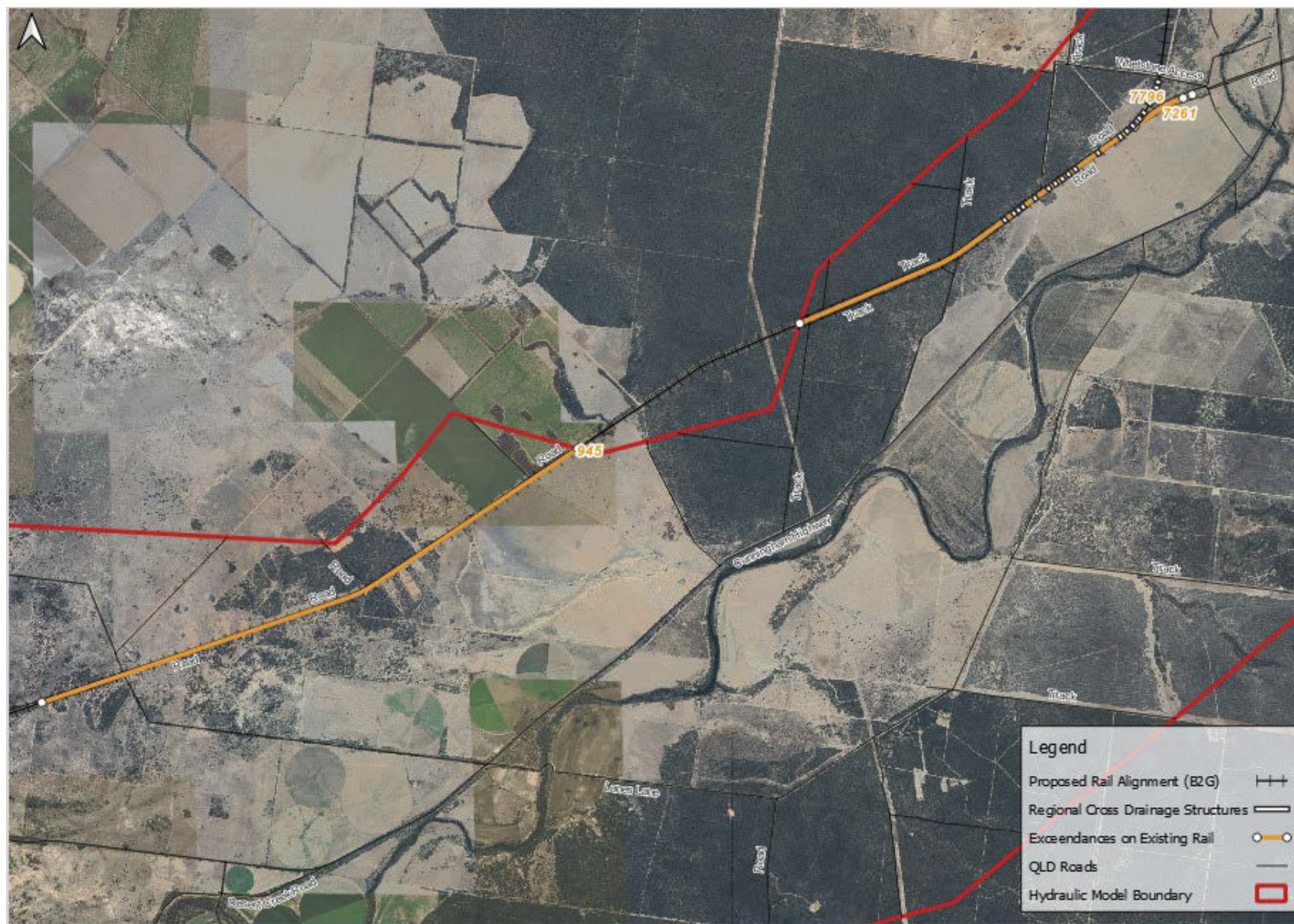


Figure 14.19 Macintyre Brook—Impacted existing rail infrastructure

Table 14.17 Macintyre Brook - Summary of impacts on existing rail infrastructure

Route Name	Section Name	Status	Flood Impact Objective Target					AEP	Maximum FIO Exceedance					Length (m)				Exceedance Justification	Proposed Mitigation Measures ^b
			Change in Peak Water Level (mm)	Change in Time of Inundation (%)	Change in Velocity (%)	Change in Velocity (m/s)	Change in Hazard (%)		Change in Peak Water Level (mm)	Change in Time of Inundation (%) ²	Change in Velocity (%)	Change in Velocity (m/s)	Change in Hazard (%) ²	Afflux	Time on Inundation	Velocity	Hazard		
4911	Toowoomba to Thallon	Operational	10	10	10	0.5	10	20%	57	71	-	-	-	52.5	37.5	-	-	Section through Yelarbon. Maturity of current rail embankment design and cross-drainage design is limited by available information (e.g. detailed survey)	This portion of existing rail is to be removed by the Project.
								10%	86	76	-	-	-	60.0	45.0	-	-		
								5%	81	79	-	-	-	60.0	45.0	-	-		
								2%	90	82	-	-	-	67.5	45.0	-	-		
								1%	-	63	-	-	-	-	67.5	-	-		
								0.05%	-	-	-	-	-	-	-	-	-		
7708	Toowoomba to Thallon	Operational	10	10	10	0.5	10	20%	89	-	-	-	-	15.0	-	-	-	Section through Yelarbon. Maturity of current rail embankment design and cross-drainage design is limited by available information (e.g. detailed survey)	This portion of existing rail is to be removed by the Project.
								10%	99	-	-	-	-	22.5	-	-	-		
								5%	23	47	-	-	-	30.0	22.5	-	-		
								2%	-	11	-	-	-	-	7.5	-	-		
								1%	-	28	-	-	-	-	7.5	-	-		
								0.05%	-	-	-	-	-	-	-	-	-		
16879	Yelarbon Yard	Operational	10	10	10	0.5	10	20%	-	22	-	-	-	-	7.5	-	-	Section through Yelarbon. Maturity of current rail embankment design and cross-drainage design is limited by available information (e.g. detailed survey)	This portion of existing rail is to be removed by the Project.
								10%	-	-	-	-	-	-	-	-	-		
								5%	16	120	-	-	-	7.5	15.0	-	-		
								2%	11	104	-	-	-	7.5	15.0	-	-		
								1%	-	122	-	-	-	-	52.5	-	-		
								0.05%	-	-	-	-	-	-	-	-	-		
945	Toowoomba to Thallon	Operational	10	10	10	0.5	10	20%	-	-	-	-	-	-	-	-	-	Maturity of current rail embankment design and cross-drainage design is limited by available information (e.g. detailed survey)	Further cross-drainage optimization investigations are required during detailed design
								10%	-	-	-	-	-	-	-	-	-		
								5%	352	419	-	-	-	90.0	15.0	-	-		
								2%	154	659	-	-	-	360.0	120.0	-	-		
								1%	101	309	19	0.09	25	435.0	120.0	15.0	30.0		
								0.05%	-	-	-	-	-	-	-	-	-		
7261	Whetstone Siding	Operational	10	10	10	0.5	10	20%	-	-	-	-	-	-	-	-	-	Whetstone. Maturity of current rail embankment design and cross-drainage design is limited by available information (e.g. detailed survey)	Further cross-drainage optimization investigations are required during detailed design
								10%	-	-	-	-	-	-	-	-	-		
								5%	282	281	-	-	-	7.5	7.5	-	-		
								2%	266	1057	-	-	-	90.0	30.0	-	-		
								1%	93	251	-	-	26	450.0	157.5	-	7.5		
								0.05%	-	-	-	-	-	-	-	-	-		
7796	Toowoomba to Thallon	Operational	10	10	10	0.5	10	20%	-	-	-	-	-	-	-	-	-	Whetstone. Maturity of current rail embankment design and cross-drainage design is limited by available information (e.g. detailed survey)	Further cross-drainage optimization investigations are required during detailed design
								10%	-	-	-	-	-	-	-	-	-		
								5%	282	281	-	-	-	37.5	7.5	-	-		
								2%	266	1057	-	-	-	210.0	90.0	-	-		
								1%	136	340	-	0.003	26	525.0	232.5	7.5	22.5		
								0.05%	-	-	-	-	-	-	-	-	-		
10846	Toowoomba to Thallon	Operational	10	10	10	0.5	10	20%	-	-	-	-	-	-	-	-	-	Section through Yelarbon. Maturity of current rail embankment design and cross-drainage design is limited by available information (e.g. detailed survey)	This portion of existing rail is to be removed by the Project.
								10%	-	-	-	-	-	-	-	-	-		
								5%	-	-	-	-	-	-	-	-	-		
								2%	-	-	-	-	-	-	-	-	-		
								1%	-	122	-	-	-	-	7.5	-	-		
								0.05%	-	-	-	-	-	-	-	-	-		

- Table Notes:**
- Time of Submergence has been measured across the full length of the road segment and does not correspond to the exact location of exceedance within that segment
 - Change in water level (afflux) is calculated based on the maximum water surface level difference between the existing and design case results, and does not consider the underlying topography. Depth values presented are based on the respective underlying topographies and in some cases do not align with afflux calculations. This can be attributed to interpolation calculations of results performed within the hydraulic model.
 - Percentage increases should be read in conjunction with existing and design case results. The FIOs require that change be presented in percentage change for time of inundation, velocity and hazard and can sometimes produce large increases when existing case results are significantly small. Some locations were identified to experience significant percentage increases however the "absolute" difference between Existing and Design results are relatively small.
 - The above table should be read in conjunction with change maps provided as part of Volume II and the Digital Platform.
 - The symbol "-" denotes that no change above Flood Impact Objectives has been determined and therefore no result has been presented.
 - Changes recorded above may only occur during a small fraction of model simulation time.
 - Three decimal places do not infer absolute accuracy in modelling results, but it is necessary to identify small changes.
 - Design optimisations that could be implemented (but not limited to) include adjustment of culvert spacing, realignment of culverts to existing flowpaths when more detailed survey is available, modifying the grade of culverts, road height and alignment adjustments, removal of bridge abutments from waterway extents etc

Depth (m) ²		Time of Inundation in hours (related to % exceedance)		Time of Submergence (hours) ²		Velocity in m/s (related to % exceedance)		Velocity in m/s (related to actual change exceedance)		Hazard in m ³ /s (related to % exceedance)	
Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case
0.091	0.124	20.44	35.23	49.53	49.93	-	-	-	-	-	-
0.006	0.017	19.93	34.86	50.94	51.50	-	-	-	-	-	-
0.011	0.019	8.93	16.00	53.89	54.22	-	-	-	-	-	-
0.025	0.038	10.13	18.40	55.71	56.10	-	-	-	-	-	-
-	-	9.50	15.48	67.11	69.77	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
-	0.089	-	-	-	-	-	-	-	-	-	-
-	0.099	-	-	-	-	-	-	-	-	-	-
0.003	0.004	20.83	30.72	37.24	46.13	-	-	-	-	-	-
-	-	25.98	28.83	50.13	47.46	-	-	-	-	-	-
-	-	26.36	33.70	50.91	54.19	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
-	-	3.76	4.58	29.69	27.09	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
0.004	0.004	5.19	11.44	33.19	28.31	-	-	-	-	-	-
-	0.011	8.35	17.06	34.94	28.83	-	-	-	-	-	-
-	-	2.34	5.19	35.63	33.70	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
0.040	0.371	9.80	50.83	53.06	51.70	-	-	-	-	-	-
0.442	0.606	7.49	56.88	70.29	66.76	-	-	-	-	-	-
1.330	0.634	14.91	60.95	73.47	70.02	0.504	0.599	0.480	0.566	0.26	0.32
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
0.015	0.229	8.92	33.98	40.44	40.44	-	-	-	-	-	-
-	0.266	2.98	34.44	55.70	55.62	-	-	-	-	-	-
0.531	0.624	11.77	41.27	59.35	59.33	-	-	-	-	0.26	0.30
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
0.015	0.229	8.92	33.98	50.53	56.63	-	-	-	-	-	-
-	0.266	2.98	34.44	65.99	72.82	-	-	-	-	-	-
-	0.136	15.60	69.59	69.35	75.83	-	0.497	0.500	0.26	0.33	-
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
-	-	2.34	5.19	31.51	5.19	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-

14.5.4.5 Flood impacts on private land outside the permanent footprint

The majority of the area over which impacts are predicted to occur is agricultural land or open land. Table 14.18 presents the predicted changes in flood conditions associated with the Project on private land outside the permanent footprint. A summary of key impacts are provided in the below points, with exceedance justification and mitigation measure provided in Table 14.18:

- 121 separate properties with individual land use categories have been identified as having a potential impact exceeding the Project FIO criteria.
- The majority of all potential exceedances occurred in Cattle Breeding and Fattening and Sheep Grazing-Dry land use types.
- There is one property (41MH778) with a 1% AEP afflux of 381 mm. This property is located adjacent the Yelarbon Levee and downstream of the Cunningham Highway. The increase in water level is a result of diverting the flow around Yelarbon town with the levee. It should be noted that this increase does not translate to uniform flood level increases across the lot. The maximum change in water level (afflux) for all other properties is less than 280 mm in the 1% AEP event.
- Property (22RP53969) has a velocity change of 0.83 m/s in the 1% AEP event. The maximum change in velocity for all other properties is approximately 0.43 m/s.
- Maximum change in the duration of inundation is 35 hours in the 1% AEP event (Lot Plan 89SP140808). This lot is immediately upstream of the Yelarbon levee and the rail embankment.
- Maximum change in hazard for all properties is 0.52 m²/s in the 1% AEP event (Lot Plan 1Y5698).

Table 14.18 Macintyre Brook: Inglewood to Yelarbon – Summary of flood impacts on private land outside the permanent footprint

ID	ID (Luplan ID)*SR ID Culvert ID	Sensitive Receptor Type	Catchment	Catchment Type	Landuse ¹¹	Lot Area (m2)	Flood Impact Objective Target						Maximum FIO Exceedance						Area (ha) ²		Exceedance Justification	Proposed Mitigation Measures ²⁰	Depth (m) ²		Time of inundation in hours (related to % exceedance)		Time of inundation in hours (related to actual change in exceedance)		Velocity in m/s (related to % exceedance)		Velocity in m/s (related to actual change in exceedance)		Hazard in m/s (related to % exceedance)																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																													
							Change in Peak Water Level (mm)	Change in Time of Inundation (Hours)	Change in Velocity (%)	Change in Velocity (m/s)	Change in Hazard (%)	AEP	Change in Peak Water Level (mm)	Change in Time of Inundation (Hours)	Change in Velocity (%)	Change in Velocity (m/s)	Change in Hazard (%)	Afflux	Time on inundation	Velocity			Hazard	Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																											
1	10215691	Lot Type Parcel	Macintyre Brook	Regional	VACANT URBAN LAND	1,008	10	5	1	10	0.5	10	20% 10% 5% 2% 1% 0.05%	64 72 72 - - -	-	-	-	0.03 0.03 0.03 - - -	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-

Future Freight
Integrating Community, Environment and Engineering

ID	ID (Lofplan ID) ¹⁰ PSR ID ¹¹	Sensitive Receptor Type	Catchment	Catchment Type	Landuse ¹¹	Lot Area (m2)	Flood Impact Objective Target						AEP	Maximum FIO Exceedance						Area (ha) ¹²			Exceedance Justification	Proposed Mitigation Measures ⁹				
							Change in Peak Water Level (mm)	Change in Time of Inundation (%)	Change in Time of Inundation (Hours)	Change in Velocity (%)	Change in Velocity (m/s)	Change in Hazard (%)		Change in Peak Water Level (mm)	Change in Time of Inundation (%)	Change in Time of Inundation (Hours)	Change in Velocity (%)	Change in Velocity (m/s)	Change in Hazard (%)	Afflux	Time on Inundation	Velocity			Hazard			
148	01AP13826	Lot Type Parcel	Macintyre Brook	Regional	VACANT - LARGE HOUSESITE	5,463	250	10	1	10	0.5	10	20%	-	-	-	-	-	-	-	-	-	-	A conservative approach to the FIO targets has been applied to this lot due to its potential future landuse of "Vacant Large House site." The FIO target of 250mm afflux in the 1 in 2,000 AEP event applies to "Existing Habitable Dwellings." A review of aerial imagery does not show any existing dwellings on this lot, but the conservative approach of applying the 250mm target on this lot has been taken due to the future potential of the lot being developed. This exceedance remains due to potential the applied FIO target is not warranted and therefore this exceedance is actually not real. This landuse type (and subsequently FIO target) is to be confirmed with the landowner. Further justification of the exceedance is provided below. In the existing case under the 1% AEP event, the lot is dry and not impacted by floodwaters. In the existing case 1 in 2,000 AEP event the lot becomes inundated by approximately 200 to 300mm. During the developed case 1 in 2,000 AEP event, this lot is one of approximately 30 undeveloped adjacent lots that have an increase in water level that exceeds the 250mm afflux target. The increase in water levels is due to limited drainage infrastructure currently proposed in the Project design directly downstream of these lots. This results in ponding of water on the upstream side of the alignment (causing an increase in water levels at the lot). Currently, proposed cross drainage is sized to the 1% AEP event, and as stated above this portion of the alignment is not impacted under the 1% AEP event and therefore no drainage structures have been proposed at this location.	A review of cross drainage design will be undertaken at this location to potentially increase the conveyance under the Project alignment. As mentioned in the justification column, a conservative approach has been applied by using the 250mm afflux target for this lot, even though no existing habitable dwellings are located on the site. In order to confirm the future landuse of this site, and verify whether the FIO target is actually applicable, ongoing consultation will be undertaken with the impacted landholder to discuss this exceedance.			
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Table Notes:

1. Floodplains are dynamic in nature and therefore model results vary from cell to cell. The "Area" is therefore defined as the maximum area over which change above the Flood Impact Objective occurs and does not relate directly to the maximum FIO exceedance recorded.
2. Change in water level (afflux) is calculated based on the maximum water surface level difference between the existing and design case results, and does not consider the underlying topography. Depth values presented are based on the respective underlying topographies and in some cases do not align with afflux calculations. This can be attributed to interpolation calculations of results performed within the hydraulic model.
3. Percentage increases should be read in conjunction with existing and design case results. The FIOs require that change be presented in percentage change for time of inundation, velocity and hazard and can sometimes produce large increases when existing case results are significantly small. Some locations were identified to experience significant percentage increases however the "absolute" difference between Existing and Design results are relatively small.
4. The Flood Impact Objective exceedance has been identified due to a change in flood extent and a cell which was dry in the existing case and now flood-affected in the developed case.
5. The above table should be read in conjunction with change maps provided as part of Volume 11 and the Digital Platform.
6. The symbol "-" denotes that no change above Flood Impact Objectives has been determined and therefore no result has been presented.
7. Changes recorded above may only occur during a small fraction of model simulation time.
8. Three decimal places do not infer absolute accuracy in modelling results, but it is necessary to identify small changes.
9. Flood impact objectives are assessed based on landuse applied to the entire lot and exceedances identified do not necessarily indicate an impact to a dwelling.
10. Design optimisations that could be implemented (but not limited to) include adjustment of culvert spacing, realignment of culverts to existing flowpaths when more detailed survey is available, modifying the grade of culverts, road height and alignment adjustments, removal of bridge abutments from waterway extents etc
11. Landuse data, dated 2018, has been used for this assessment. Due to the changing nature of this dataset, this will be updated to the latest available information in Detailed Design.

Depth (m) ²		Time of Inundation in hours (related to %exceedance)		Time of Inundation in hours (related to actual change)		Velocity in m/s (related to % exceedance)		Velocity in m/s (related to actual change/exceedance)		Hazard in m/s (related to % exceedance)	
Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
0.20	0.70	-	-	-	-	-	-	-	-	-	-

14.5.4.6 Flow distribution

A key landowner concern is the potential change in flow distribution. Maximum velocity-depth modelling results were used to create change in unit flow maps between Existing and Developed Case conditions to determine the change in flow distribution. Unit flow is a measure of discharge that is commonly used for assessing peak flow along cross-sections, or to identify the efficient flow corridors within a floodplain.

The presented flow distribution maps were filtered with a velocity-depth product cut-off of 0.15 m²/s. The cut-off is in line with FIO requirements and was used to identify main channels (high flow zones), and shallow lower flood zones (i.e. outer extents of the flood surface).

The following points summarise the potential change in flow distribution for Macintyre Brook and should be considered in conjunction with flow distribution figures supplied in Figure Q10-1 to Figure Q10-8 of Volume 2 – Appendix Q.

- Changes to unit flow across the model is generally minimal with the main areas of change occurring around Yelarbon due to the presence of the raised levee and at Thornton Road near Millmerran-Inglewood Road
- Up to the 1% AEP event, unit flow changes are generally confined to immediately upstream and downstream of the Project alignment but as flood events become rarer (1 in 2,000, 1 in 10,000 AEP and PMF events) the change becomes more prominent further away from the alignment
- There are two areas where there are increases above 10 per cent to unit flow in the 1% AEP event:
 - To the north and south of the Project alignment at Thornton Road near Millmerran Inglewood. This change is localised as a result of the rail crossing of Thornton Road
 - Along the eastern and southern sides of the Yelarbon levee with increased unit flow of approximately 20 per cent.

Ongoing mitigation measures to reduce changes in flow distribution are likely to occur during future design stages of the Project, and may include further refinement of the bridge abutments, culvert skews, and additional survey where necessary (including existing roads and culvert survey).

14.5.5 Extreme events

14.5.5.1 Extreme event flood immunity

The risk of overtopping of the Project alignment was assessed for a range of extreme events (1 in 2,000 AEP, 1 in 10,000 AEP and PMF) with Table 14.19 presenting the depth of water above the formation level and over the top of rail at each structure. It is noted that the function of the floodplain culverts is to balance flood levels on the upstream and downstream sides of the alignment.

As such, overtopping of the rail is not predicted to result in significant excessive flows or velocities as would occur in a dam embankment overtopping scenario. Further analysis of the overtopping risk to FSRs can be found in Section 21.

Table 14.19 Macintyre Brook: Extreme events—Estimated depth of water above formation level and/or over top of rail level

Chainage (km)	Structure ID	Depth of water above formation level (m)			Depth of water above top of rail (m)		
		1 in 2,000 AEP	1 in 10,000 AEP	PMF	1 in 2,000 AEP	1 in 10,000 AEP	PMF
25.19	C25.19	-	-	1.55	-	-	0.85
25.46	C.25.46	-	0.02	1.69	-	-	0.99
25.50	C.25.50	-	-	1.68	-	-	0.98
25.80	C25.80	-	0.42	2.25	-	-	1.55

Chainage (km)	Structure ID	Depth of water above formation level (m)			Depth of water above top of rail (m)		
		1 in 2,000 AEP	1 in 10,000 AEP	PMF	1 in 2,000 AEP	1 in 10,000 AEP	PMF
25.87	C25.87	-	0.10	1.95	-	-	1.25
25.95	C25.95	-	0.35	2.27	-	-	1.57
25.97	C25.97	-	0.18	2.09	-	-	1.39
26.05	C26.05	-	0.07	2.00	-	-	1.30
27.05	C27.05	-	0.26	1.99	-	-	1.29
27.15	C27.15	-	0.71	2.41	-	0.01	1.71
27.24	C27.24	0.28	0.70	2.45	-	0.00	1.75
27.33	C27.33	-	0.23	1.89	-	-	1.19
27.42	C27.42	-	-	1.52	-	-	0.82
42.87	C42.87	0.23	0.90	4.86	-	0.20	4.16
43.02	C43.02	0.22	0.90	4.87	-	0.20	4.17
43.09	C43.09	0.13	0.91	4.90	-	0.21	4.20
43.16	C43.16	0.28	0.97	4.94	-	0.27	4.24
43.35	C43.35	0.22	0.96	4.95	-	0.26	4.25
43.56	C43.56	0.57	1.39	5.38	-	0.69	4.68
43.66	C43.66	0.23	1.01	5.00	-	0.31	4.30
43.77	C43.77	0.23	1.03	5.01	-	0.33	4.31
43.86	C43.86	0.32	1.13	5.11	-	0.43	4.41
43.97	C43.97	0.31	1.11	5.08	-	0.41	4.38
44.32	C44.32	0.50	1.33	5.30	-	0.63	4.60
44.67	C44.67	0.39	1.23	5.19	-	0.53	4.49
44.78	C44.78	0.23	1.11	5.11	-	0.41	4.41
44.88	C44.88	0.23	1.03	5.03	-	0.33	4.33
44.96	C44.96	0.15	1.00	5.04	-	0.30	4.34
45.01	C45.01	0.25	0.98	4.97	-	0.28	4.27
45.10	C45.10	0.18	0.86	4.84	-	0.16	4.14
45.24	C45.24	0.15	0.80	4.76	-	0.10	4.06
45.30	C45.30	0.08	0.72	4.66	-	0.02	3.96
45.40	C45.40	-	0.47	4.56	-	-	3.86
45.45	C45.45	-	0.41	4.42	-	-	3.72
45.53	C45.53	-	0.39	4.36	-	-	3.66
45.64	C45.64	0.10	0.69	4.38	-	-	3.68

14.5.5.2 Impacts during extreme events

Flood immunity of FSRs is discussed in Section 14.5.4.1, and maps demonstrating the impacts during extreme events are shown on the [Online Digital Portal](#).

In accordance with the FIOs, FSRs are considered impacted by extreme flood events if the change in water level is greater than 250 mm in the 1 in 2,000 AEP event, subject to the receptor classification (i.e. at existing habitable dwellings and/or commercial and industrial buildings/premises). In addition, a stricter afflux objective of 10 mm in the 1 in 500 AEP event applies to sites with existing sensitive infrastructure, uses involving vulnerable people and/or any critical road network that was designed to be immune to flooding in any extreme event.

Review of the 1 in 2,000 AEP event results for the Macintyre Brook floodplain against the FIOs show that flood level changes above the 250 mm limit were observed at one habitable dwelling and six sheds. No identified existing sensitive infrastructure or uses involving vulnerable people are expected to experience a more than 10 mm increase in flood level. A separate assessment of the 1 in 500 AEP event is not required as they are conservatively represented by the 1 in 2,000 AEP event results.

Table 14.20 details extreme event results, although it should be noted that FIO targets do not apply to the 1 in 10,000 AEP and PMF events. Maps demonstrating the impacts during extreme events are shown in the [Online Digital Portal](#).

Table 14.20 Macintyre Brook: Extreme events—Impacts to flood sensitive receptors

FSR ID	FSR type	1 in 2,000 AEP event		1 in 10,000 AEP event		PMF event	
		Change in peak water level (mm)	Existing Case flood depth (m)	Change in peak water level (mm)	Existing Case flood depth (m)	Change in peak water level (mm)	Existing Case flood depth (m)
MCB_ID_519	Shed	103 ¹	-	144 ¹	0.42	55 ¹	2.36
MCB_ID_583	Shed	548	0.36	486	0.61	274	1.39
MCB_ID_584	House	548	0.36	599	0.56	520	0.95
MCB_ID_585	Shed	512	0.45	550	0.67	473	1.09
MCB_ID_586	Shed	480	0.36	500	0.57	419	0.98
MCB_ID_587	Shed	398	0.35	412	0.57	341	0.99
MCB_ID_588	Shed	421	0.57	440	0.80	369	1.22

Table note:

1 FSR is newly wet in the Developed Case

14.5.6 Sensitivity analysis—Macintyre Brook

The sensitivity of the model to various parameters was assessed using the following two scenarios:

- An increase in rainfall intensity (i.e. to reflect climate change scenario)
- An increase and decrease in blockage of culverts from 25 to 50 per cent and 0 per cent.

14.5.6.1 Climate change

The potential impacts of climate change in the Macintyre Brook floodplain were assessed for the 1% AEP climate change event to determine the sensitivity of the Project to the potential long-term changes in climate. The assessment was undertaken in accordance with ARR 2019.

The Representative Concentration Pathways 8.5 (2090 horizon) climate change scenario has been adopted for the Project, with an associated increase in rainfall intensity of 23 per cent across the catchment area.

The predicted flow resulting from a 23 per cent increase in rainfall is 4,826 m³/s in the Macintyre Brook at Inglewood for the 1% AEP event (compared to 3,323 m³/s in existing climate conditions). Therefore, flows are predicted to increase by approximately 45 per cent as a result of the increase in rainfall in the 1% AEP event with climate change at Inglewood.

It may be noted that increasing the rainfall intensity has a disproportionate increase in the peak flow. This is partly due to the influence of the losses subtracted from the rainfall, which remain constant, thereby increasing the effective net rainfall (gross rainfall – loss). Inglewood and Booba Sands are also affected by the initial drawdown of Coolmunda Dam, which was modelled assuming median Volume Below Full (VBF) to achieve reconciliation between the Design Event hydrology and at-site stream gauge FFA (discussed in Section 17.2.7.2).

This disproportionate increase is similarly observed for the 1 in 500 and 1 in 2,000 AEP hydrology with 30 and 60 per cent increase in rainfall intensity leading to 60 and 120 per cent increase in peak flow, respectively.

The model predicts that with an increase in rainfall intensity of 23 per cent across the catchment, the flood levels increase from 282.96 mAHD to 283.44 mAHD at Inglewood.

The Project alignment is not predicted to be overtopped as a result of the 23 per cent increase in rainfall intensity, with water levels predicted to be below formation level.

The climate change sensitivity results at FSRs for the 1% AEP design event are shown in Table 14.21. Note only FSR locations that recorded a flood depth during Developed Case and Existing Case conditions were analysed as part of this sensitivity assessment.

Table 14.21 Macintyre Brook: 1% AEP event Representative Concentration Pathway 8.5 conditions—Impacts at flood sensitive receptors

FSR ID	FSR type	1% AEP climate change event	
		Afflux (mm)	Existing Case flood depth (m)
MCB_ID_75	House	+11	1.06
MCB_ID_76	House	+11	0.40
MCB_ID_82	House	+36	1.26
MCB_ID_83	House	+18	1.22
MCB_ID_228	House	+103	0.69
MCB_ID_229	Shed	+112	0.35
MCB_ID_231	Commercial	+40	0.54
MCB_ID_241	House	+23	0.25
MCB_ID_242	House	+18	0.16
MCB_ID_243	House	+25	0.34
MCB_ID_274	House	+39	0.82
MCB_ID_278	House	+31	0.31
MCB_ID_321	Commercial	+20	0.86
MCB_ID_322	Commercial	+37	0.72
MCB_ID_324	House	+38	1.12
MCB_ID_325	House	+12	0.77
MCB_ID_326	House	+15	0.72
MCB_ID_327	House	+27	0.76
MCB_ID_328	House	+36	0.90
MCB_ID_330	Commercial	+70	0.18
MCB_ID_332	Commercial	+83	0.81
MCB_ID_333	Commercial	+86	0.87
MCB_ID_335	House	+90	0.83
MCB_ID_337	House	+94	0.71
MCB_ID_338	House	+95	0.77

FSR ID	FSR type	1% AEP climate change event	
		Afflux (mm)	Existing Case flood depth (m)
MCB_ID_339	House	+94	0.68
MCB_ID_341	Commercial	+93	0.78
MCB_ID_343	House	+76	0.95
MCB_ID_345	House	+78	0.89
MCB_ID_346	House	+35	1.16
MCB_ID_347	House	+35	1.19
MCB_ID_349	House	+34	1.24
MCB_ID_352	House	+34	1.17
MCB_ID_353	House	+35	1.05
MCB_ID_355	House	+40	1.32
MCB_ID_359	House	+45	1.38
MCB_ID_360	House	+53	1.14
MCB_ID_361	House	+81	1.30
MCB_ID_363	House	+84	1.17
MCB_ID_365	House	+86	1.20
MCB_ID_367	House	+86	1.17
MCB_ID_369	Commercial	+29	1.05
MCB_ID_370	House	+34	1.20
MCB_ID_374	House	+39	1.12
MCB_ID_381	House	+35	0.86
MCB_ID_382	House	+37	0.64
MCB_ID_384	House	+39	0.74
MCB_ID_394	House	+30	1.05
MCB_ID_395	House	+31	1.12
MCB_ID_397	House	+31	1.22
MCB_ID_401	House	+34	0.98
MCB_ID_414	House	+43	0.82
MCB_ID_415	House	+39	0.73
MCB_ID_416	House	+37	0.92
MCB_ID_420	House	+38	0.46
MCB_ID_423	House	+42	0.58
MCB_ID_424	House	+36	0.40
MCB_ID_426	House	+22	0.46
MCB_ID_453	House	+31	1.04
MCB_ID_457	House	+55	1.17
MCB_ID_465	Shed	+111	0.48
MCB_ID_469	House	+28	0.37
MCB_ID_470	House	+58	0.40
MCB_ID_472	House	+105	0.35
MCB_ID_474	House	+121	0.40
MCB_ID_475	House	+137	0.44

FSR ID	FSR type	1% AEP climate change event	
		Afflux (mm)	Existing Case flood depth (m)
MCB_ID_476	Shed	+125	0.28
MCB_ID_477	House	+105	0.38
MCB_ID_478	House	+100	0.47
MCB_ID_479	House	+86	0.38
MCB_ID_480	House	+71	0.32
MCB_ID_481	House	+49	0.40
MCB_ID_482	House	+40	0.43
MCB_ID_483	House	+28	0.52
MCB_ID_484	House	+28	0.46
MCB_ID_494	Electrical Interchange	+106	0.56
MCB_ID_527	Shed	+101	0.83
MCB_ID_528	Shed	+101	0.63
MCB_ID_544	Shed	+114	0.42
MCB_ID_575	House	+35	0.72

The 1% AEP event change in peak water levels for the Representative Concentration Pathways 8.5 climate change scenario is presented in Figure J9-C in Volume 2 – Appendix J.

14.5.6.2 Blockage

Blockage was assessed in accordance with ARR 2019. The blockage assessment was undertaken and resulted in a blockage factor of 25 per cent adopted for culverts. A minimum culvert size of 900 mm diameter was adopted to reduce potential for blockage and maintenance. A significant community concern is the potential impacts to flood conditions should the proposed culverts become blocked with debris. The primary concern is the blockage of culverts which is likely to drive flood levels higher, particularly upstream of the culverts, and divert more flow through residences, across access roads and other infrastructure. A sensitivity analysis was undertaken with 0 and 50 per cent blockage on culverts.

The model predicts that in both the 0 and 50 per cent blockage cases three houses are predicted to experience an increase in peak water levels above 10 mm for the 1% AEP event, as presented in Table 14.22.

During detailed design the blockage factors will be reviewed in line with the final design and local catchment conditions. This may result in a varied and/or lower blockage factors being applied along the Project alignment. Should impacts be determined due to a change in blockage factor, additional mitigation measures will be considered in detailed design.

The results are presented in Figure J9-A and Figure J9-B in Volume 2 – Appendix J for the 0 and 50 per cent blockage respectively.

Table 14.22 Macintyre Brook: 1% AEP event—Blockage assessment

FSR ID	FSR type	Existing Case flood depth (m)	Change in peak water level (mm)		
			0% blockage	25% blockage	50% blockage
MCB_ID_278	House	-	+20	+20	+22
MCB_ID_420	House	-	+31	+31	+32
MCB_ID_426	House	0.15	+14	+14	+16

14.6 Additional modelling items to be considered in detailed design

The following points summarise additional items to be included in the flood models for detailed design as per agreement with the Expert Flood Panel:

- Local catchment inflows to be delineated further.
- Blockage factors will be reviewed in line with the final design and local catchment conditions.
- Significant new developments, with a development approval, that are likely to affect the local hydrology and floodplain behaviour to be constructed prior to the construction of the Project, will be included in the flood modelling. Developments for inclusion will be discussed and agreed with the applicable approval authority.
- During detailed design specific consideration will be given to including fencing, local access roads and safety barriers, on a case-by-case basis, within hydraulic modelling.
- Inclusion of missing existing structures from survey information if available, in the hydraulic modelling.
- Bridge losses to be reviewed, and where appropriate, updated to industry standard.
- A review of Existing and Developed Case roughness factors will be completed, and where appropriate, updated as necessary.
- A review of the latest TUFLOW version will be undertaken to determine whether updates of the TUFLOW version from 2020-10-AD-iSP-w64 are beneficial to modelling.
- Design culvert configurations/sizing to be optimised.
- Drainage diversions.

Further details of mitigation measures can be found in Section 22. A review of proposed mitigation measures will be completed during the detailed design stage of the Project after hydraulic models have been updated to capture the above changes.

14.7 Planned and future roads

Consultation with the DTMR has not identified any planned or future road upgrades within the Macintyre Brook—Yelarbon to Inglewood catchment. ARTC will continue to consult and engage with DTMR to determine any future or planned road projects that may need to be considered as part of the detailed design hydraulic modelling.

Consultation with GRC has identified some upgrade and realignment of substandard curves along Yelarbon-Keetah Road. Yelarbon-Keetah Road is currently represented in the DEM) used in the Macintyre Brook hydraulic model. Due to the minor nature of the road upgrade works identified by GRC, it is not anticipated the works will influence the hydraulic outcomes of the modelling. ARTC will continue to consult and engage with GRC to identify any future works that should be considered as part of the hydraulic modelling during the detailed design stage.

15 Macintyre Brook at Bybera Road

The location of the Project alignment in relation to Macintyre Brook Study area at Bybera Road is shown in Figure K1-A in Volume 2 – Appendix K.

15.1 Data collection and review—Macintyre Brook at Bybera Road

Available background information, including existing hydrologic and hydraulic models, survey, streamflow data, available calibration information and anecdotal flood data was gathered. Data was sourced from a range of stakeholders including:

- GRC—existing flood studies
- BoM—rainfall gauging data
- DTMR—existing infrastructure details.

15.1.1 Previous studies

Previous hydrology and hydraulic studies were sourced as part of this assessment. A review of each study was undertaken to determine suitability for use on the Project, as documented in the following sections.

15.1.1.1 Inglewood Flood Study, Engeny, 2015

Engeny was commissioned by GRC to undertake a flood study of Inglewood. The study objectives were ‘to define the nature, extent and risks of flooding in Inglewood in order to inform disaster management planning and response, as well as control future development’ (Engeny, 2015). An URBS hydrologic model and TUFLOW (1D/2D) hydraulic model were developed.

15.1.1.2 Inland Rail: Phase 2—North Star to NSW/QLD Border – Hydrology and Flooding Calibration Report, FFJV, 2018

The NS2B section of Inland Rail will cross the Macintyre River and its floodplain, which are a part of the Border Rivers catchment. The NS2B alignment runs through Moree Plains LGA, Gwydir LGA and Goondiwindi LGA. To establish the reliability of the models, the 2011 event was included in the calibration process in addition to the 1976 and 1996 events assessed by NSW Department of Planning, Industry and Environment (DPIE). The hydrologic and hydraulic models were found to represent flows and levels across the floodplain consistently compared to the recorded 2011 event.

15.1.2 Survey

ARTC provided LiDAR data from 2015 as 1 m grid DEM tiles. Using GIS software, a DEM was generated with a 1 m grid resolution for use in the Project, based on the 2015 dataset. This was used for modelling within the permanent footprint and up to the full extent of the 2015 LiDAR where relevant. In areas that were not covered by the LiDAR provided by ARTC, LiDAR tiles were sourced from Geoscience Australia. The DEM datasets utilised for modelling were based on surveys flown between 2009 and 2015. SRTM data was used for catchment delineation where no LiDAR data could be sourced, to inform the hydrologic modelling.

The survey data sources and DEM developed for the unnamed creek catchment upstream of Bybera Road is shown in Figure K1-B in Volume 2 - Appendix K.

15.1.3 Aerial imagery

Aerial imagery of the Macintyre Brook at the Bybera Road study area was provided by ARTC and was used to identify and confirm topographic and vegetative characteristics of the study area. Aerial imagery captured in 2015 was made available. Additional imagery outside the study area was sourced from QGIS imagery in an open-source format.

15.1.4 Existing drainage structure data

DTMR as-constructed drawings were also sourced for culvert and bridge details. This information will be refined as local survey is progressively completed. Details of infrastructure included within the Macintyre Brook at Bybera Road flood assessment is located in Section 15.3.4.

15.1.5 Stream gauge data

There are no streamflow gauges within the unnamed creek catchment study area upstream of Bybera Road.

15.1.6 Rainfall data

A number of daily and sub-daily rainfall stations are located in and around the unnamed creek catchment upstream of Bybera Road; however, since there are no streamflow gauges to use for model calibration, no historical rainfall data was sought.

15.1.7 Anecdotal and observed flood data

No anecdotal or observed flood data was available for this area of the Macintyre Brook.

15.1.8 Site inspection

A site inspection was undertaken in February 2018. During the site inspection, all major waterway crossings were visited and inspected, with photographs taken and details recorded of the crossing, existing drainage structures and surrounding catchment. An assessment of the relative roughness and blockage potential was undertaken during the site inspection.

15.2 Hydrologic model development—Macintyre Brook at Bybera Road

15.2.1 Model setup

A hydrologic model of the unnamed creek catchment upstream of Bybera Road was established in URBS using the latest procedures detailed in ARR 2019 and the latest rainfall data from BoM. The new ARR procedures cover revisions made to hydrologic parameters such as losses, pre-burst depths, TPs and ARF.

The URBS model covers approximately 62 km² of the unnamed creek catchment upstream of Bybera Road and the confluence with Macintyre Brook. The catchment was delineated into 20 sub-catchments to capture the variability of rainfall and to better represent the network of creeks and streams within the catchment.

The URBS model was setup as a 'BASIC' model. The hydrologic model setup including extent and sub-catchments are presented in Figure K1-C in Volume 2 – Appendix K.

Model inputs are summarised in Table 15.1.

Table 15.1 Macintyre Brook at Bybera Road—Summary of URBS model inputs

Input parameter	Remarks
URBS model type	Basic
Routing variables	Catchment area, stream lengths
Channel lag parameter, α	1.20
Catchment non-linearity parameter, m	0.8

Note that the default values were adopted for all other URBS parameters.

15.2.2 Sub-catchments

The Macintyre Brook at Bybera Road catchment is comprised of 20 sub-catchments and an approximate catchment area of 62 km².

The hydrologic model extent and sub-catchment map is presented in Figure K1-C in Volume 2 – Appendix K.

15.2.3 Design event parameters

Hydrologic information to assist estimation of design event flows was sourced from the ARR 2019 Data Hub as summarised in Table 15.2.

Table 15.2 Macintyre Brook at Bybera Road—Summary of URBS model design event inputs

Input parameter	Remarks
Design rainfall	Intensity-Frequency-Durations for each sub-catchment were downloaded from the BoM's website to account for variation in rainfall across the catchment.
Extreme event rainfall	Probable Maximum Precipitation depths for durations up to 6 hours (for use in modelling the PMF event) were obtained using the method presented in the Bulletin 53 (BoM, 2003). The rainfall depths for the 1 in 10,000 AEP event were estimated using the interpolation method presented in ARR 2019 Book 8 Section 3.5.
Losses	The losses were adopted from the <i>Inglewood Flood Study</i> (Engeny, 2015) URBS model. Losses do not vary with AEP. Initial loss—15 mm Continuing loss—1 mm/hr
Areal reduction factor	Parameters were adopted for the semi-arid inland Queensland region. The catchment area U/S of the proposed rail crossing adjacent Bybera Road is approximately 62 km ² , which yields an ARF between 76.3% and 95.9% depending on design storm event AEP and duration.
Ensemble temporal patterns	Central Slopes region
Pre-burst depths	Median pre-burst depths were downloaded from the ARR 2019 Data Hub for each sub-catchment. Pre-burst depths vary by design storm event AEP and duration. Pre-burst depths were applied to the model by reducing the initial losses for each storm event.

15.2.3.1 Spatial distribution

The ARR 2019 design event methodology was adopted for this assessment. ARR 2019 recommends adopting the spatially varying method for catchments larger than 20 km². The Macintyre Brook at Bybera Road catchment is approximately 62 km². A total of 20 sub-catchments were assigned an IFD derived from the BoM website.

15.2.3.2 Rainfall depth estimation

Design rainfall depths for 20 per cent to 1 in 2,000 AEP were manually obtained from BoM website for all sub-catchments. As an example, the IFD representing the catchment centroid of Macintyre Brook at Bybera Road URBS model is provided in Table 15.3. Due to the size of the model area, there is minimal difference in rainfall depth across the catchment.

Table 15.3 Rainfall depth (mm) used for the Macintyre Brook at Bybera Road catchment centroid

Duration (hr)	Annual Exceedance Probability (% AEP)						
	50	20	10	5	2	1	0.05
0.25	17.3	24.0	28.6	33.0	39.0	43.7	70.8
0.50	23.6	32.6	38.9	45.1	53.4	59.8	97.1
1.00	29.7	41.1	49.0	56.8	67.3	75.4	123.0
2.00	35.8	49.2	58.6	68.0	80.5	90.3	146.0
3.00	39.7	54.2	64.3	74.6	88.4	99.2	160.0
4.50	43.9	59.5	70.6	81.6	96.8	109.0	175.0
6.00	47.2	63.7	75.4	87.2	103.0	117.0	187.0
12.00	56.7	75.8	89.4	103.0	123.0	139.0	224.0
24.00	68.3	91.1	108.0	125.0	150.0	170.0	275.0
48.00	81.7	110.0	130.0	152.0	183.0	208.0	362.0
72.00	89.6	121.0	144.0	168.0	203.0	231.0	381.0

15.2.3.3 Areal reduction factor

The ARF was estimated using the coefficients for semi-arid inland Queensland presented in Table 15.4 and was applied to all IFD areas.

The URBS software directly downloaded these coefficients from ARR 2019 Data Hub to estimate the ARF using Equation 2.4.1 of ARR 2019 Book 2.

The estimated ARF for Macintyre Brook at Bybera Road catchment vary between 76 and 96 per cent depending on design storm event AEP and duration.

Table 15.4 Macintyre Brook at Bybera Road—Areal reduction factor coefficients from ARR 2019 Data Hub

a	b	c	d	e	f	g	h	i
0.159	0.283	0.25	0.308	7.30E-07	1.0	0.039	0.0	0.0

15.2.3.4 Temporal patterns

Temporal patterns for 20% AEP to 1 in 2,000 AEP events were adopted based on a 10-burst ensemble approach presented in the ARR 2019. The TPs and pre-burst data were directly downloaded by URBS software from the ARR 2019 Data Hub for the Macintyre Brook at Bybera Road catchment.

Temporal patterns for the 1 in 10,000 AEP and PMF design events for durations up to 6 hours were obtained from *The Estimation of Probable Maximum Precipitation in Australia: Generalised Short-Duration Method* (BoM, 2003).

The adopted TPs for all modelled events are shown in Table 15.5.

Table 15.5 Macintyre Brook at Bybera Road—Adopted temporal patterns

AEP (%)	Adopted temporal pattern
20	Pattern 4
10	Pattern 7
5	Pattern 7
2	Pattern 7
1	Pattern 8
1 in 2,000	Pattern 0
1 in 10,000	PMP pattern
PMF	PMP pattern

15.2.3.5 Pre-burst

Median pre-burst depths for durations from 0.5 hour to 72 hours for a 1% AEP design event are between 1.1 mm and 13.1 mm (ARR 2019 Data Hub). Due to the small depth of pre-burst in this area, the impact of pre-burst condition is considered to be minimal in this catchment.

The adopted pre-burst depths are summarised in Table 15.6.

Table 15.6 Macintyre Brook at Bybera Road—Adopted pre-burst depths

Duration (hrs)	Pre-burst depth (mm)					
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP
0.5	0.6	0.9	1.1	1.3	1.4	1.4
0.75	0.6	0.9	1.1	1.3	1.4	1.4
1.0	0.6	0.9	1.1	1.3	1.4	1.4
1.5	0.5	0.5	0.5	0.5	1.6	2.5
2.0	0.2	0.3	0.3	0.3	1.7	2.8
3.0	0.0	0.0	0.0	0.0	2.8	5.0
6.0	0.6	0.6	0.6	0.7	0.9	1.2
9.0	0.6	0.6	0.6	0.7	0.9	1.2
12.0	0.0	0.1	0.2	0.3	5.4	9.1
18.0	0.0	0.1	0.2	0.3	5.4	9.1
24.0	0.0	0.5	0.9	1.2	7.2	11.7
48.0	0.0	0.5	0.9	1.2	6.2	9.9
72.0	0.0	0.0	0.0	0.0	0.6	1.0

15.2.3.6 Design rainfall losses

The adopted constant initial and continuing losses in the URBS model are 15 mm and 1 mm/hr, respectively. The losses were adopted from the *Inglewood Flood Study* (Engeny, 2015) URBS model and validated by comparing the modelled flows and those from the scaled FFA. The URBS model validation is detailed in Section 15.2.6.

15.2.3.7 Extreme event rainfall

Probable Maximum Precipitation depths for durations up to 6 hours (for use in modelling the PMF event) were obtained using the methodology presented in *The Estimation of Probable Maximum Precipitation in Australia: Generalised Short-Duration Method* (BoM, 2003). Once the PMP was estimated, rainfall for events between 1 in 2,000 AEP and the PMF was then interpolated. For the 1 in 10,000 AEP event, ARF were applied based on interpolation methodology presented in ARR 2019 Section 3.5 in Book 8 and the rainfall depths were estimated accordingly.

The parameters adopted for PMP rainfall estimation are presented in Table 15.7, and the adopted extreme rainfall depths are provided in Table 15.8.

Table 15.7 Macintyre Brook at Bybera Road—Input data for Probable Maximum Precipitation estimation

Factor	Value
EAF	1
MAF	0.8
Portion of rough catchment	100%
Bybera Road catchment location and catchment area (km ²)	(-28.375, 150.986) and 62 km ²

Table 15.8 Macintyre Brook at Bybera Road—Adopted extreme event rainfall depths (mm)

Duration (hour)	Macintyre Brook at Bybera Road catchment	
	1 in 10,000 AEP rainfall depth (mm)	PMP rainfall depth (mm)
0.25	88	152
0.5	122	223
0.75	141	283
1	157	337
1.5	175	433
2	184	502
2.5	195	561
3	205	610
4	218	692
5	229	760
6	238	809

15.2.4 Event selection for calibration

Insufficient data was available to calibrate historical flood events in the Macintyre Brook at Bybera Road catchment.

15.2.5 Model calibration

Insufficient data was available to calibrate historical flood events in the Macintyre Brook at Bybera Road catchment.

15.2.6 Model validation

The hydrologic model for the Macintyre Brook at Bybera Road catchment was not calibrated due to unavailability of observed stream gauge data in the catchment; however, the routing parameter α was adjusted until there was a reasonable match between the URBS model flows and those derived using DTMR's QRT and RFFE methods.

All URBS results reside within the 90 per cent confidence limits of the RFFE and show a close match with QRT. Adjusting losses for each AEP may improve the validation against QRT and RFFE and should be investigated at the next stage of reporting. The estimated flood flows at the outlet of the Macintyre Brook at Bybera Road model are presented in Figure 15.1 and Table 15.9. The model outlet lies approximately 250 m downstream of where the unnamed creek crosses the Project alignment.

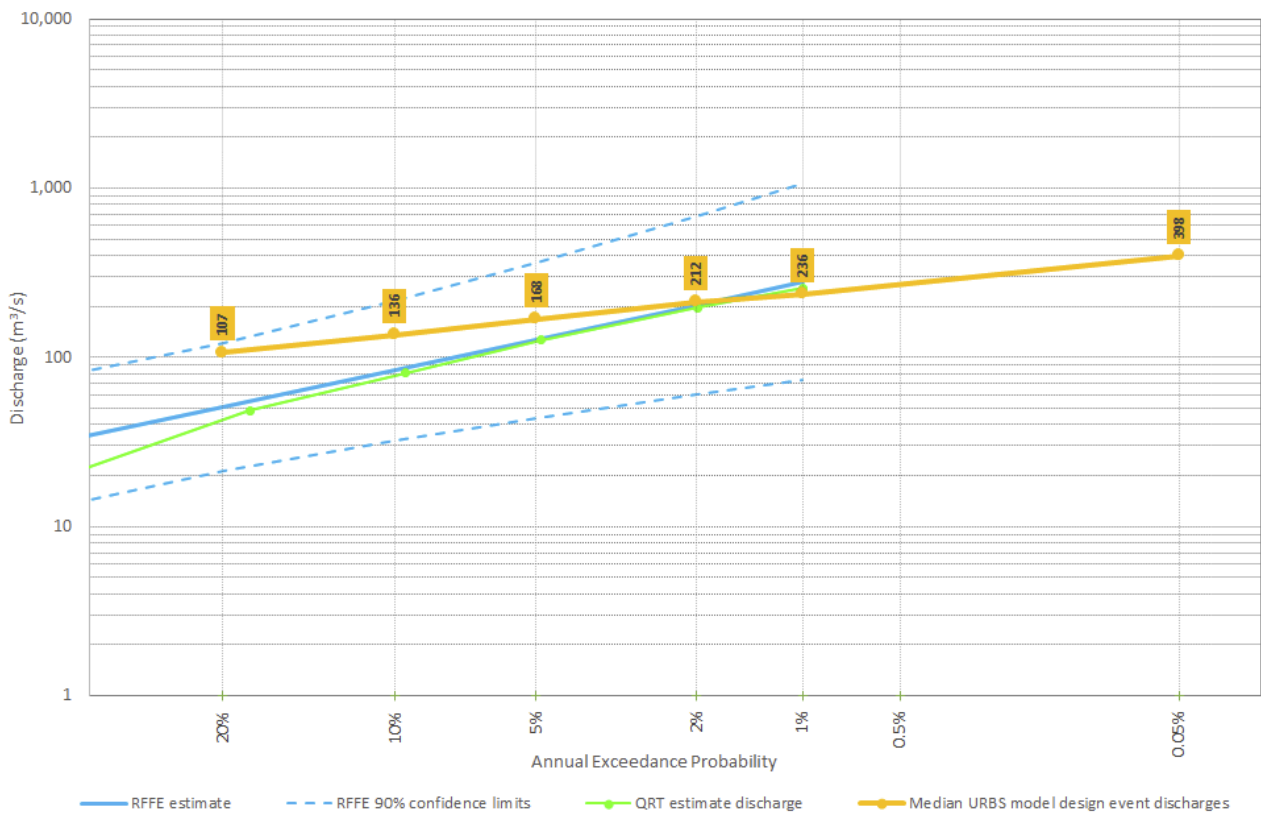


Figure 15.1 Estimate of flows at the outlet of Macintyre Brook at Bybera Road model

Table 15.9 Estimate of flows at the outlet of Macintyre Brook at Bybera Road model

AEP (%)	RFFE—lower bound 90% confidence level (m³/s)	RFFE—estimate of flow (m³/s)	RFFE—upper bound 90% confidence level (m³/s)	DTMR quantile regression technique (m³/s) ¹	URBS model flows (m³/s)
20	21	51	122	48	107
10	32	84	216	81	136
5	44	127	365	126	168
2	61	204	685	195	212
1	74	281	1,060	257	236
1 in 2,000	-	-	-	-	398
1 in 10,000	-	-	-	-	1,010
PMF	-	-	-	-	4,463

Table note:

1 The QRT method estimates the 39.3%, 18.1%, 9.5% and 4.9% AEP instead of 50%, 20% and 10% and 5%, respectively

15.3 Hydraulic model development—Macintyre Brook at Bybera Road

A 2D modelling approach was adopted to appropriately simulate flood mechanisms around the Project rail crossing adjacent Bybera Road. The platform used for hydraulic modelling is TUFLOW HPC software package (version 2020-10-AD) with SGS approach. The processes and assumptions adopted throughout the development of the hydraulic model are described in the following sections.

15.3.1 Model setup

The setup of the TUFLOW model is summarised in Table 15.10.

Table 15.10 Macintyre Brook at Bybera Road—Hydraulic model summary

Parameter	Information
Completion date	August 2022
AEPs assessed	20%, 10%, 5%, 2%, 1%, 1 in 2,000, 1 in 10,000 AEP and PMF
Hydraulic model build	TUFLOW HPC GPU – version 2020-10-AD-iSP-w64
Model extent	Refer to Figure K1-D in Volume 2 – Appendix K
Grid size	5 m
SGS Size	1 m
DEM (year flown)	ARTC dataset (2015)
Roughness	Spatially varying roughness values compliant with industry norms.
Eddy viscosity	Wu (default)
Model calibration	N/A. No stream gauge data available.
D/S model boundary	Height-Discharge (HQ) Boundary with normal slope approximated based upon topography dataset.
Hydraulic model timestep	AdaptiveTimestep
Modelled scenarios	Existing Case, Developed Case
Sensitivity analysis	Climate change

The hydraulic model extent and the spatial distribution of land use in the 2D model domain is presented in Figure K1-D and Figure K1-E in Volume 2 – Appendix K.

15.3.2 Model topography and grid size

The topography within the TUFLOW hydraulic model was based on 1 m DEM data collected during 2015, which was supplied by ARTC. A 5 m x 5 m grid size was selected to allow the features of the channels and floodplain to be represented with sufficient accuracy while maintaining efficient model run times. The 5 m x 5 m grid size is considered adequate to represent floodplain topography and is consistent with the approach typically adopted in similar flood studies.

The 2D model topography was modified so existing road/rail crowns and levee banks were represented accurately in the model. For the Developed Case, the topography was modified to include the Project rail embankment, drainage structures and associated infrastructure.

15.3.2.1 Sub-grid sampling

The Macintyre Brook at Bybera Road hydraulic model was established using TUFLOW's 2020-10-AD model version, including SGS add-on module. The below points provide a brief description of the module, along with the key parameters adopted for this assessment.

- Traditional grid-mesh based 2D hydraulic models sample the underlying DEM by taking a single elevation at the cell centres and cell faces. Sub-grid sampling allows for increased resolution by sampling a greater number of points of the underlying DEM to establish a water surface elevation versus volume relationship for each grid, along with a water surface elevation versus width relationship (wetted perimeter) at cell sides. The adoption of SGS allows increased representation of storage and conveyance between cells. A SGS distance of 1 m was adopted for this assessment.

15.3.3 Roughness

The hydraulic roughness generally reflects the types of development and ground cover that exists within the hydraulic model extent. The distribution of roughness categories adopted for this assessment was based on the information supplied in the land use layer from Queensland Globe database, aerial imagery and confirmed during the site inspection. Specific roughness values applied to the model are detailed in Table 15.11.

Table 15.11 Adopted roughness values for Bringalily Creek catchment

Category	Manning's 'n' value
River	0.035
Sealed roads	0.015
Unsealed roads	0.025
Rural residential	0.09
Urban residential	0.10
Commercial	0.06
Open space—Minimum vegetation	0.045
Open space—Moderate vegetation	0.055
Open space—Heavy vegetation	0.065
Open water	0.025
Rail	0.025
Riparian	0.04
Crops ¹	0.1085

Table note:

- 1 A uniform value was adopted for the crop areas, due to the seasonal and spatial variations in vegetation distribution and the uncertainty this brings to the design event approach

15.3.4 Hydraulic structures

No existing hydraulic structures are situated within the extents of the Bybera Road hydraulic model.

15.3.5 Boundary conditions

The following boundary conditions were applied within the hydraulic model:

- Flow versus time model boundaries were applied for all inflow hydrographs with local and total hydrographs sourced from the hydrologic models

- Delineated sub-catchments in the hydrologic model were used as internal flow boundaries within the hydraulic model extent (SA polygons). The modelled flows in the hydrologic model were applied to the lowest points of each SA polygon.
- Where the Project alignment intersected the sub-catchment, the inflows were divided accordingly based on catchments upstream and downstream of the alignment.

15.4 Existing Case modelling results—Macintyre Brook at Bybera Road

15.4.1 Hydrologic model peak flow assessment

A critical duration assessment was undertaken to determine which storm duration(s) produced peak flood flows where the major waterways are intersected by the Project alignment and at the downstream outlet of the model. To assess the critical storm duration the following methodology was adopted:

- The models were modelled for a range of AEP events:
 - Each AEP was modelled for a range of durations
 - Each duration was modelled for each of the 10 associated TPs.

A critical duration assessment was undertaken at the locations mentioned above to determine which duration produced the highest median flow of the 10 TPs for each event. The critical duration and adopted TP for each design event AEP is provided in Table 15.12. Table 15.13 presents the estimated 1% AEP event peak flow applied to the hydraulic model for a number of key locations (Figure K1-D in Volume 2 – Appendix K).

Table 15.12 Macintyre Brook at Bybera Road—Critical durations for vary AEP events

AEP (%)	Critical storm duration/temporal pattern
20	9 hours – Pattern 4
10	6 hours – Pattern 7
5	6 hours – Pattern 7
2	6 hours – Pattern 7
1	12 hours – Pattern 8
1 in 2,000	6 hours – Pattern 0
1 in 10,000	6 hours – PMP pattern
PMF	6 hours – PMP pattern

Table 15.13 Macintyre Brook at Bybera Road—1% AEP event peak flow at key locations as applied in the hydraulic model

Sub-catchment name	1% AEP peak flow (m ³ /s)	Critical storm duration (hrs) and temporal pattern
15T	182.8	12 hours – Pattern 8
16L	24.6	12 hours – Pattern 8
17L	18.2	12 hours – Pattern 8
18L	19.0	12 hours – Pattern 8
19L	15.8	12 hours – Pattern 8
20L	16.7	12 hours – Pattern 8
20T	235.8	12 hours – Pattern 8

15.4.2 Existing Case flood maps

Existing conditions flood maps are presented in Volume 2 – Appendix K and via an [Online Digital Portal](#). Appendix K contains the Existing Case 20% and 1% AEP flood maps, while the [Online Digital Portal](#) contains the remainder of the modelled events. The location of each map set is summarised below:

- 20% AEP peak levels: Figure K2-A
- 1% AEP peak levels: Figure K2-B
- 20% AEP peak velocity: Figure K2-C
- 1% AEP peak velocity: Figure K2-D
- 20% AEP peak hazard: Figure K2-E
- 1% AEP peak hazard: Figure K2-F
- 20% AEP peak time of inundation: Figure K2-G
- 1% AEP Peak time of inundation: Figure K2-H
- 20% AEP, 10% AEP, 5% AEP, 2% AEP, 1% AEP, 1 in 2,000 AEP, 1 in 10,000 AEP and PMF Existing Case results: [Online Digital Portal](#)
- For higher resolution mapping refer to the [Online Digital Portal](#).

15.4.3 Existing Case flood inundation extent and flood levels

The peak flood depth for the 1% AEP event is approximately 3.4 m within the unnamed creek channel. This depth reduces to a flood depth of around 1.1 m in other areas of the floodplain. The peak flood depth on the floodplain is estimated to be up to 2.6 m (273.7 mAHD) where the Project alignment crosses the floodplain.

Figure K2-B in Volume 2 – Appendix K shows the 1% AEP indicative flood extent and peak water levels within the Macintyre Brook at Bybera Road floodplain for the Existing Case.

15.4.4 Existing Case flood immunity of existing infrastructure

Table 15.14 presents a summary of estimated overtopping depths for key roads near the Project alignment under a range of design events. Modelling results show that Cremascos Road has a low existing flood immunity in the areas close to the Project alignment. The Cunningham Highway, approximately 2 km downstream of the alignment, is demonstrated by modelling results to have an immunity up to and including the 1 in 10,000 AEP event.

Table 15.14 Macintyre Brook at Bybera Road: Existing Case—Overtopping depths of key infrastructure

Infrastructure	Overtopping depth (m)							
	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2,000 AEP	1 in 10,000 AEP	PMF
Cunningham Highway	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.91
Cremascos Road	0.00	0.09	0.17	0.24	0.27	0.40	0.77	1.87

15.4.5 Existing Case velocities

Existing peak 1% AEP event flood velocities are expected to reach above 5.5 m/s in localised areas (and confined within the main channel) with the average velocity across the floodplain approximately 0.3 m/s, as shown in Figure K2-C in Volume 2 – Appendix K.

15.4.6 Existing Case flood hazard

For the 1% AEP event, a velocity x depth of product of $>1.6 \text{ m}^2/\text{s}$ is achieved within the bridge crossing, with the average velocity x depth across the floodplain approximately $0.1 \text{ m}^2/\text{s}$ as shown in Figure K2-E in Volume 2 – Appendix K.

15.4.7 Existing Case flood duration of inundation

Results for the 1% AEP event highlights flood water inundating areas upstream of the bridge crossing for periods great than 15 hours. The profile also highlights ponding areas created by farming activities and existing road infrastructure as shown in Figure K2-G in Volume 2 – Appendix K.

15.5 Developed Case modelling results—Macintyre Brook at Bybera Road

15.5.1 Drainage structures

The hydraulic design of the major drainage structures was undertaken using the TUFLOW model (1d and 2d approach). At the Macintyre Brook at Bybera Road catchment crossing, the Project includes one waterway bridge. No culverts are included within the hydraulic model.

Design bridge details are summarised in Table 15.15 and its proposed location is presented in Figure K1-F in Volume 2 – Appendix K. The 1% AEP flood level at the bridge structure is presented in Appendix B.

Table 15.15 Macintyre Brook at Bybera Road—Proposed bridge location and details

Chainage (km)	Structure ID	Approximate span (m)	Deck width (m)	Soffit level (mAHD)	Deck superstructure type	Deck depth (mm)
55.55	310-BR04	207	3.97	286.96	Type D1	2,000

Bridges were modelled as an opening in the rail embankment. This approach was adopted due to the high soffit level of the bridge (approximately 13 m above the 1% AEP event peak water surface level) and therefore interaction between the bridge deck and flood waters is unlikely. The optimisation of bridge lengths was balanced between minimising the changes to the hydraulic regime, primarily afflux and velocities, and the cost of replacing bridge spans by large earth embankments.

Blockage of hydraulic structures was assessed in accordance with ARR 2019. The predominant vegetation in the catchment is considered heavily forested; however, given the appreciable size of the bridge spans, and the relative size of upstream debris, a blockage factor was not applied to bridges. Significant amounts of debris accumulating against the piers of the bridge are considered low.

15.5.2 Hydraulic design criteria outcomes

The hydraulic model was run for the Developed Case for the range of events (20%, 10%, 5%, 2%, 1% AEP events and 1 in 2,000 AEP, 1 in 10,000 AEP and PMF events) with the outcomes presented in the following sections.

15.5.2.1 Rail immunity and structures results

Appendix B summarises the peak 1% AEP water levels on the upstream side of the Project alignment.

The results of flood modelling indicate that peak water levels remain below the proposed rail formation level, and that in a 1% AEP event, flood immunity to the proposed rail formation is achieved for the Project alignment across the Macintyre Brook at Bybera Road floodplain. There is a 15.1 m freeboard to formation level, noting that the formation levels were generally dictated by other engineering criteria and constraints.

15.5.3 Flood Impact Objectives outcomes—Macintyre Brook at Bybera Road

The impact of the Developed Case was assessed through inclusion of the proposed rail design and comparison of model results against the Existing Case results.

Changes to flooding behaviour that were assessed, and are reported in the following sections, include potential changes in peak water levels, time of inundation, velocity, hazard and peak flows:

- Changes in 20% AEP and 1% AEP peak water levels are presented in Figures K4-A and K4-B in Volume 2 – Appendix K
- Changes in 20% AEP and 1% AEP velocities are presented in Figures K4-C and K4-D in Volume 2 – Appendix K
- Changes in 20% AEP and 1% AEP hazard are presented in Figures K4-E and K4-F in Volume 2 – Appendix K
- Changes in 20% AEP and 1% AEP time of inundation are presented in Figures K4-G and K4-H in Volume 2 – Appendix K
- Changes in 20% AEP and 1% AEP velocities and percentage change in velocities with cut-offs applied are presented in Figures K5-A to K5-F in Volume 2 – Appendix K
- Percentage changes in 20% AEP and 1% AEP in hazard are presented in Figures K6-A and K6-B in Volume 2 – Appendix K
- Percentage changes in 20% AEP and 1% AEP in time of inundation and with cut-offs applied are presented in Figures K7-A to K7-D in Volume 2 – Appendix K
- Refer to the [Online Digital Portal](#) for other modelled AEP results and higher resolution mapping.

One-on-one consultation with landowners who are expected to experience changes in flooding behaviour on their property was conducted by ARTC supported by FFJV. Refer to EIS Chapter 14: Flooding and Geomorphology and EIS Appendix E: Consultation Report for further details.

The Project design outcomes relative to the FIOs (Table 4.2) are presented in the following sections.

Flood sensitive receptors referred to in this section are shown in Figures P1 (1 to 59) in Volume 2 – Appendix P.

15.5.3.1 Flood impacts on flood sensitive receptors

Based on the available aerial imagery, no buildings or critical infrastructure are located within the area impacted by afflux and change in flood hazard in the Macintyre Brook at Bybera Road floodplain for all modelled events up to the 1 in 2,000 AEP event.

15.5.3.2 Flood impacts on State-controlled roads

The extent of the hydraulic model developed for Macintyre Brook at Bybera Road catchment is shown in Figures K1-D in Volume 2 – Appendix K. Within the extent, the only State-controlled road, influenced by the Project and flooding, is the Cunningham Highway.

Developed Case flooding from the Macintyre Brook at Bybera Road catchment and comparison against the FIOs has demonstrated that no adverse impacts on the Cunningham Highway are expected due to the presence of the Project for all modelled events up to and including the 1% AEP event.

15.5.3.3 Flood impacts on local public roads

Within the extent of the hydraulic model, two local public roads are influenced by flooding. Table 15.16 identifies the location of potential FIO exceedances and the details for each exceedance, including afflux, velocity, time of inundation and hazard along with appropriate justification and proposed mitigation measures against each.

The following provide a summary of the key FIO exceedances:

- Cremascos Road (Location 12 in Table 15.16) shows the greatest impacts with a maximum afflux of approximately 0.04 m, and for an extent of 5 m for the 10% AEP design flood
- Minor change in velocity and time of inundation exceedances are noted for an unnamed local public road (Location 7).

The extent of the model, and location of the public local roads that experience an exceedance, are shown in Figure 15.2.

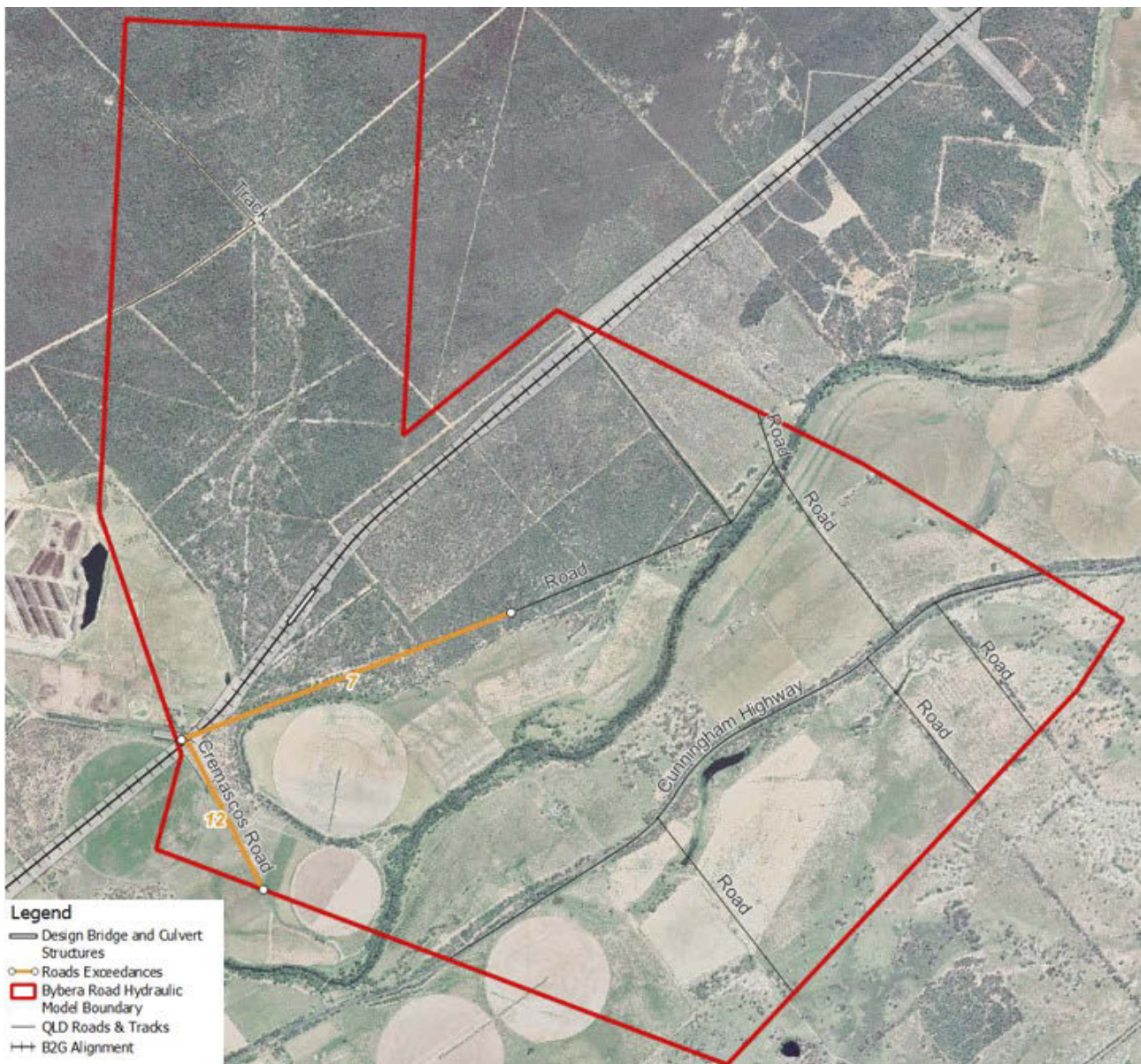


Figure 15.2 Macintyre Brook at Bybera Road— Impacted local public roads

Table 15.16 Macintyre Brook at Bybera Road - Summary of impacts on local public roads

Road Object ID	Road Segment	Road Name	Surface Type	Flood Impact Objective Target					AEP	Maximum FIO Exceedance					Length (m)				Exceedance Justification	Proposed Mitigation Measures*
				Change in Peak Water Level (mm)	Change in Time of Inundation (%)	Change in Velocity (%)	Change in Velocity (m/s)	Change in Hazard (%)		Change in Peak Water Level (mm)	Change in Time of Inundation (%)	Change in Velocity (%)	Change in Velocity (m/s)	Change in Hazard (%)	Afflux	Time on Inundation	Velocity	Hazard		
12	16015103990000087993	Cremascos Road	Sealed	10	10	20	1	10	20%	-	-	-	-	-	-	-	-	Exceedances along Cremascos Road occur on the floodplain fringe and are likely due to some minor redirection of flow due to the presence of the Project. Cremascos Road is impacted in the existing case by a breakout of the main flood which discharges to the Macintyre Brook. The minor increases seen impact only small portions of the road and influence the overall time of submergence by a maximum 0.06 hours (4 minutes). Based on this it is not deemed that the recorded impacts will influence the operational capacity of the road significantly and therefore the impacts are considered minor.	Consultation with the appropriate Road Manager will be undertaken once optimisation of the design has occurred to discuss any residual impacts and agree appropriate mitigation measures if necessary.	
									10%	40	15	-	-	-	5.00	12.50	-			
									5%	-	11	-	-	-	2.50	-	-			
									2%	-	-	-	-	-	-	-	-			
									1%	-	-	-	-	-	-	-	-			
									0.05%	-	-	-	-	-	-	-	-			
7	16015103990000433036	Road 0	Unsealed	10	10	10	0.5	10	20%	-	-	-	0.002	-	-	-	10.00	- Maturity of current bridge design is limited by available information (e.g. detailed survey) optimisation investigations are required during detailed design.		
									10%	0.002	1.50	-	-	-	-	-				
									5%	-	-	-	-	-	-	-	-			
									2%	-	-	-	-	-	-	-	-			
									1%	-	-	-	0.003	-	-	-	10.00			
									0.05%	-	-	-	-	-	-	-	-			

Table Notes

1. Time of Submergence has been measured across the full length of the road segment and does not correspond to the exact location of exceedance within that segment
2. Change in water level (afflux) is calculated based on the maximum water surface level difference between the existing and design case results, and does not consider the underlying topography. Depth values presented are based on the respective underlying topographies and in some cases do not align with afflux calculations. This can be attributed to interpolation calculations of results performed within the hydraulic model.
3. The above table should be read in conjunction with change maps provided as part of Volume II and the Digital Platform.
4. The symbol "-" denotes that no change above Flood Impact Objectives has been determined and therefore no result has been presented.
5. Changes recorded above may only occur during a small fraction of model simulation time.
6. Three decimal places do not infer absolute accuracy in modelling results, but it is necessary to identify small changes.
7. Design optimisations that could be implemented (but not limited to) include adjustment of culvert spacing, realignment of culverts to existing flowpaths when more detailed survey is available, modifying the grade of culverts, road height and alignment adjustments, removal of bridge abutments from roadway extents etc

[illegible]

15.5.3.4 Flood impacts on existing rail infrastructure

No third-party rail infrastructure is located within the Macintyre Brook at Bybera Road hydraulic model boundary.

15.5.3.5 Flood impacts on private land outside the permanent footprint

In accordance with the FIOs, Table 15.17 presents the modelled changes and potential impacts on third-party property outside of the Project's permanent footprint. A summary of key impacts, exceedance justification and mitigation measures are provided in the below points, with further detail provided in Table 15.17:

- Eight separate properties were identified as having a potential impact exceeding the Project FIO criteria
- The majority of all potential exceedances occurred in Cattle Breeding and Animals Special land-use types
- No properties are expected to experience a change in water level (afflux)
- Maximum change in velocity for all properties is approximately 1.3 m/s and 49 per cent
- Maximum change in the duration of inundation is <1.6 hours and 132 per cent
- Maximum change in hazard for all properties is <0.5 m²/s and 53 per cent
- Exceedances may be a result of the maturity of current cross-drainage (culverts and bridges) design, and limited available information (e.g. detailed survey)
- In order to mitigate potential exceedances, further cross-drainage optimisation investigations, and ongoing model strengthening may occur in detailed design.

Table 15.17 Macintyre Brook at Bybera Road – Summary of flood impacts on private land outside the permanent footprint

ID	ID	ID (Lotplan ID/FSR ID/ Culvert ID)	ID (Lotplan ID/FSR ID/ Culvert ID)	Sensitive Receptor Type	Catchment Type	Landuse ¹	Lot Area (m2)	Flood Impact Objective Target						Maximum FIO Exceedance						Area (ha) ²				Exceedance Justification	Proposed Mitigation Measures ^{2b}		
								Change in Peak Water Level (mm)	Change in Time of Inundation (%)	Change in Time of Inundation (Hours)	Change in Velocity (%)	Change in Velocity (m/s)	Change in Hazard (%)	AEP	Change in Peak Water Level (mm)	Change in Time of Inundation (%) ²	Change in Time of Inundation (Hours)	Change in Velocity (%) ²	Change in Velocity (m/s)	Change in Hazard (%) ²	Afflux	Time on Inundation	Velocity			Hazard	
1	1	9119 65MA3454	65MA3454	Lot Type Parcel	Regional	ANIMALS-SPECIAL	655,833	200	10	1	10	0.5	20	20%	-	-	24	0.24	-	0.04	31	-	0.07	0.01	0.09	-Maturity of current bridge design is limited by available information (e.g. detailed survey)	- Further road and or rail design optimization investigations are required during detailed design.
														10%	-	-	30	0.08	16	0.11	36	-	0.01	0.09	0.17		
														5%	-	-	64	0.41	28	0.14	39	-	0.07	0.23	0.75		
														2%	-	-	35	-	32	0.16	44	-	0.01	0.41	0.36		
														1%	-	-	45	0.04	35	0.17	46	-	0.02	0.55	0.45		
														0.05%	-	-	-	-	-	-	-	-	-	-	-		
														20%	-	-	115	1.05	24	0.11	41	-	0.21	0.50	0.79		
2	4	9189 511RP226715	511RP226715	Lot Type Parcel	Regional	ANIMALS-SPECIAL	3,795,347	200	10	1	10	0.5	20	10%	-	-	82	-	30	0.14	42	-	0.26	0.67	0.87	The exceedances experienced on this Lot are due to the concentration of runoff through a bridge structure (310-BR04). The bridge structure reduces the available cross-sectional flow path concentrating flow through the bridge and causing increases in time of inundation, velocity and hazard downstream of the bridge opening.	To mitigate these exceedances, a review and further optimisation of bridge structure will be undertaken in the detailed design phase including a revision of the length of the bridge opening and abutment placement to minimise the constriction of flow in this location. It is anticipated that exceedances experienced on this Lot can be reduced through the proposed mitigation measures. Should residual impacts remain, landowner consultation will be undertaken to discuss any remaining impacts, and agree on appropriate site specific mitigation with the landowner.
														5%	-	-	98	1.33	36	0.21	44	-	0.28	0.76	0.92		
														2%	-	-	134	1.25	43	0.16	47	-	0.32	0.87	1.01		
														1%	-	-	109	1.00	49	0.17	53	-	0.66	1.00	1.12		
														0.05%	-	-	-	-	-	-	-	-	-	-	-		
														20%	-	-	-	-	-	-	-	-	-	-	-		
														10%	-	-	-	-	-	-	-	-	-	-	-		
3	5	9012 66MA3454	66MA3454	Lot Type Parcel	Regional	ANIMALS-SPECIAL	634,157	200	10	1	10	0.5	20	20%	-	-	0.17	-	-	-	-	0.02	-	-	-	-Maturity of current bridge design is limited by available information (e.g. detailed survey) - The Flood Impact Objective exceedance has been identified due to a change in flood extent and a cell which was dry in the existing case and now flood-affected in the developed case.	- Further road and or rail design optimization investigations are required during detailed design.
														10%	-	-	42	1.34	1.70	-	-	0.04	0.01	0.003	-		
														5%	-	-	-	0.08	-	0.004	-	-	0.01	0.003	-		
														2%	-	-	36	0.09	-	0.004	-	-	0.02	0.01	-		
														1%	-	-	-	0.04	-	0.01	-	-	0.01	0.01	-		
														0.05%	-	-	-	-	-	-	-	-	-	-	-		
														20%	-	-	-	-	-	-	-	-	-	-	-		
4	8	9116 67MA3454	67MA3454	Lot Type Parcel	Regional	ANIMALS-SPECIAL	546,219	200	10	1	10	0.5	20	10%	-	-	0.10	-	-	-	-	0.03	-	-	-	-Maturity of current bridge design is limited by available information (e.g. detailed survey)	- Further road and or rail design optimization investigations are required during detailed design.
														5%	-	-	-	0.08	-	-	-	-	0.02	-	-		
														2%	-	-	-	0.06	-	0.002	-	-	0.003	0.03	-		
														1%	-	-	-	0.04	-	0.001	-	-	0.01	0.004	-		
														0.05%	-	-	-	-	-	-	-	-	-	-	-		
														20%	-	-	-	-	-	-	-	-	-	-	-		
														10%	-	-	-	-	-	-	-	-	-	-	-		
5	9	9016 13MH286	13MH286	Lot Type Parcel	Regional	CATTLE BREEDING & FATTENING	113,085	200	10	1	10	0.5	20	20%	-	-	-	-	-	-	-	-	-	-	-	-Rail design constraints alter the existing flow regime.	- Stakeholder Consultation (Landowner/ Government).
														10%	-	-	-	-	-	-	-	-	-	-	-		
														5%	-	-	12	0.14	-	-	-	-	0.01	-	-		
														2%	-	-	22	-	-	-	-	-	0.01	-	-		
														0.05%	-	-	-	-	-	-	-	-	-	-	-		
														20%	-	-	-	-	-	-	-	-	-	-	-		
														10%	-	-	-	-	-	-	-	-	-	-	-		
6	12	9114 9MH291	9MH291	Lot Type Parcel	Regional	ANIMALS-SPECIAL	48,649	200	10	1	10	0.5	20	20%	-	-	0.09	-	-	-	-	0.004	-	-	-	-Maturity of current bridge design is limited by available information (e.g. detailed survey)	- Further road and or rail design optimization investigations are required during detailed design.
														10%	-	-	-	0.07	-	-	-	-	0.01	-	-		
														5%	-	-	-	-	-	-	-	-	-	-	-		
														2%	-	-	-	-	-	-	-	-	-	-	-		
														1%	-	-	-	-	-	-	-	-	-	-	-		
														0.05%	-	-	-	-	-	-	-	-	-	-	-		
														20%	-	-	13	-	-	-	-	-	0.02	-	-		
7	13	9089 8MH286	8MH286	Lot Type Parcel	Regional	ANIMALS-SPECIAL	840,669	200	10	1	10	0.5	20	10%	-	-	34	0.38	-	-	-	0.02	-	-	-	-Maturity of current bridge design is limited by available information (e.g. detailed survey)	- Further road and or rail design optimization investigations are required during detailed design.
														5%	-	-	-	-	-	-	-	-	-	-	-		
														2%	-	-	-	0.08	-	-	-	-	0.003	-	-		
														1%	-	-	-	-	-	-	-	-	-	-	-		
														0.05%	-	-	-	-	-	-	-	-	-	-	-		
														20%	-	-	22	-	-	-	-	-	0.02	-	-		
														10%	-	-	-	-	-	-	-	-	-	-	-		
8	16	9011 1RL1245	1RL1245	Lot Type Parcel	Regional	ANIMALS-SPECIAL	71,843	200	10	1	10	0.5	20	5%	-	-	-	-	-	-	-	-	-	-	-	-Maturity of current bridge design is limited by available information (e.g. detailed survey)	- Further road and or rail design optimization investigations are required during detailed design.
														2%	-	-	-	-	-	-	-	-	-	-	-		
														1%	-	-	-	-	-	-	-	-	-	-	-		
														0.05%	-	-	-	-	-	-	-	-	-	-	-		
														20%	-	-	-	-	-	-	-	-	-	-	-		

Table Notes

1. Floodplains are dynamic in nature and therefore model results vary from cell to cell. The "Area" is therefore defined as the maximum area over which change above the Flood Impact Objective occurs and does not relate directly to the maximum FIO exceedance recorded.
2. Change in water level (afflux) is calculated based on the maximum water surface level difference between the existing and design case results, and does not consider the underlying topography. Depth values presented are based on the respective underlying topographies and in some cases do not align with afflux calculations. This can be attributed to interpolation calculations of results performed within the hydraulic model.
3. Percentage increases should be based on the difference between existing and design case results. The FIOs require that change in water level, velocity and hazard can and sometimes produce large increases when existing case results are significantly small. Some locations were identified to experience significant percentage increases however the "absolute" difference between Existing and Design results are relatively small.
4. The Flood Impact Objective exceedance has been identified due to a change in flood extent and a cell which was dry in the existing case and now flood-affected in the developed case.
5. The above table should be read in conjunction with change maps provided as part of Volume II and the Digital Platform.
6. The symbol "-" denotes that no change in Flood Impact Objectives has been determined and therefore no result has been presented.
7. Changes recorded above may only occur during a small fraction of model simulation time.
8. Three direct absolute places do not infer absolute accuracy in modelling results, but it is necessary to identify small changes.
9. Flood impact objectives are assessed based on landuse applied to the entire cell and exceedances identified do not necessarily indicate an impact to a dwelling.
10. Design optimisations that could be implemented (but not limited to) include adjustment of culvert spacing, realignment of culverts to existing flowpaths when more detailed survey is available, modifying the grade of culverts, road height and alignment adjustments, removal of bridge abutments from waterway extents etc.
11. Landuse data, dated 2018, has been used for this assessment. Due to the changing nature of this dataset, this will be updated to the latest available information in Detailed Design.

[illegible]

15.5.3.6 Flow distribution

A key landowner concern is the potential change in flow distribution. To determine the change in flow distribution, maximum velocity-depth modelling results were used to produce change in unit flow maps between Existing and Developed Case conditions. Unit flow is the measure of discharge (velocity-depth product) and is often used for assessing peak flow along a cross-section, or to determine the effective flow corridors within a floodplain.

The presented flow distribution maps were filtered with a velocity-depth product cut-off of 0.15 m²/s. The cut-off is in line with FIO requirements, and was used to assist in identifying main channels (high flow zones) from shallow lower flood zones (i.e. outer extents of the flood surface).

The following points summarise the potential change in flow distribution for the Macintyre Brook at Bybera Road catchment and should be considered in conjunction with flow distribution figures supplied in Figure Q11-1 to Figure Q11-8 of Volume 2 – Appendix Q.

- Local increases in unit flow (>20 per cent) are noted immediately upstream and downstream of the proposed rail bridge for all events up to the 1 in 10,000 AEP event. This behaviour is expected as flood water is constricted through the rail bridge during Developed Case conditions. The increases in flow are balanced by localised decreases around the abutments of the bridge.
- Negligible change in broader flow-distribution patterns across the floodplain for all events up to the 1 in 10,000 AEP event is observed. A reduction in unit flow is evident for the PMF downstream of the Project alignment. The reduction extends towards the Macintyre Brook and onto the broader floodplain to the east.

Ongoing mitigation measures to reduce changes in flow distribution are likely to occur during future design stages of the Project, and may include further refinement of the bridge abutments and additional survey where necessary.

15.5.4 Extreme events

15.5.4.1 Extreme event immunity

The risk of overtopping of the Project alignment was assessed for a range of extreme events (1 in 2,000 AEP, 1 in 10,000 AEP and PMF). Table 15.18 presents the estimated depth of water above the rail formation level and over the top of rail at each structure. It is noted that the function of the floodplain culverts is to balance flood levels on the upstream and downstream sides of the alignment. As such, overtopping of the rail is not predicted to result in significant excessive flows or velocities as would occur in a dam embankment overtopping scenario. Further analysis of the overtopping risk to sensitive receptors can be found in Section 21.

The rail formation and bridge 310-BR04 at Ch 55.5 km do not overtop in extreme local flood events.

Table 15.18 Macintyre Brook at Bybera Road: Extreme events—depth of water above rail formation and top of rail levels

Chainage (km)	Structure ID	Depth of water above formation level (m)			Depth of water above top of rail (m)		
		1 in 2,000 AEP	1 in 10,000 AEP	PMF	1 in 2,000 AEP	1 in 10,000 AEP	PMF
55.5	310-BR04	-	-	-	-	-	-

15.5.4.2 Impacts during extreme events

Flood immunity of FSRs is discussed in Section 15.5.3.1, and maps demonstrating the impacts during extreme events are shown in the [Online Digital Portal](#).

In accordance with the FIOs, FSRs are considered impacted by extreme flood events if the change in water level is greater than 250 mm in the 1 in 2,000 AEP event, subject to the receptor classification (i.e. at existing habitable dwellings and/or commercial and industrial buildings/premises). In addition, a stricter afflux objective of 10 mm in the 1 in 500 AEP event applies to sites with existing sensitive infrastructure, uses involving vulnerable people and/or any critical road network that was designed to be immune to flooding in any extreme event.

Review of the 1 in 2,000 AEP event results for the Bybera Road catchment against the FIOs show that flood level changes at all FSRs were limited to 250 mm, and no identified existing sensitive infrastructure or uses involving vulnerable people are expected to experience a more than 10 mm increase in flood level. A separate assessment of the 1 in 500 AEP event is not required as they are conservatively represented by the 1 in 2,000 AEP event results. The flood impacts in extreme events due to the Project are therefore compliant against the stipulated FIO limits for extreme events.

Table 15.19 details extreme event results, although it should be noted that FIO targets do not apply to the 1 in 10,000 AEP and PMF events.

Table 15.19 Macintyre Brook at Bybera Road catchment: Extreme events—Impacts at flood sensitive receptors

FSR ID	FSR type	1 in 2,000 AEP event		1 in 10,000 AEP event		PMF event	
		Change in peak water level (mm)	Existing Case flood depth (m)	Change in peak water level (mm)	Existing Case flood depth (m)	Change in peak water level (mm)	Existing Case flood depth (m)
MCB_ID_155	House	-	-	-	-	-17	0.103
MCB_ID_156	Shed	-	-	-	-	-50	0.050
MCB_ID_153	Shed	-	-	-	-	-18	0.141
MCB_ID_151	House	-	-	-	-	-4	0.398
MCB_ID_152	House	-	-	-	-	-24	0.203
MCB_ID_150	Shed	-	-	-	-	-5	0.930
MCB_ID_157	Shed	-	-	-	-	-22	0.148

15.5.5 Sensitivity analysis—Macintyre Brook at Bybera Road

The sensitivity of the model to blockages at hydraulic structures and an increase in rainfall intensity (i.e. to reflect a climate change scenario was assessed).

15.5.5.1 Climate change

The potential impacts of climate change in the Macintyre Brook at Bybera Road floodplain were assessed for the 1% AEP design event to determine the sensitivity of the Project to the potential long-term changes in climate. The assessment was undertaken in accordance with ARR 2019.

The Representative Concentration Pathways 8.5 (2090 horizon) climate change scenario was adopted for the Project with an associated increase in rainfall intensity of 23.9 per cent across the catchment area.

The climate change sensitivity assessment results for FSRs indicated that flood FSRs are located outside the 1% AEP flood extent under the existing and climate change scenario; therefore, no flood impact is achieved under this climate change scenario.

Climate change results are expected to increase peak water levels upstream of the Project alignment by up to 0.2 m at proposed bridge 310-BR04 for the 1% AEP event, as shown in Table 15.20. The Project alignment is expected to retain 1% AEP flood immunity to formation level under the climate change scenario.

Table 15.20 Macintyre Brook at Bybera Road – 1% AEP event with Representative Concentration Pathways 8.5 conditions – structure performance

Structure ID	Structure type	1% AEP peak water level (mAHD)	1% AEP +CC peak water level (mAHD)	Difference in peak water level (m)	Freeboard to rail formation with CC (m)
310-BR04	Bridge	273.8	274.0	+0.2	14.9

For the 1% AEP event, the change in peak water levels for the Representative Concentration Pathways 8.5 climate change scenario is presented in Figure K8-A in Volume 2 – Appendix K.

15.5.5.2 Blockage

Blockage of hydraulic structures was assessed in accordance with ARR 2019. Despite the catchment being vegetated and heavily forested, ARR guidelines determined that the likelihood of significant amounts of debris accumulating against the piers of the bridge is low; therefore, a zero-blockage factor was applied at the Macintyre Brook at Bybera Road bridge. Additionally, there are no regional culverts in the Macintyre Brook at Bybera Road floodplain, hence no sensitivity scenarios were conducted.

During detailed design the blockage factors will be reviewed in line with the final design and local catchment conditions. This may result in a varied and/or lower blockage factors being applied along the Project alignment. Should impacts be determined due to a change in blockage factor, additional mitigation measures will be considered in detailed design.

15.6 Additional modelling items to be considered in detailed design

The following points summarise additional items to be included in the flood models for detailed design as per agreement with the Expert Flood Panel:

- Bridges to be updated to include form loss coefficients
- Sub-catchment delineation and hydrology to be reviewed and, where appropriate, updated
- A review of adopted roughness delineation will be completed and, where appropriate, updated
- The application of SA polygon inflow boundaries is to be reviewed and, where appropriate, updated
- Instabilities in the current hydraulic model are to be reviewed and, where appropriate, the model will be updated to remove these instabilities
- A review of the latest TUFLOW version will be undertaken to determine whether updates of the TUFLOW version from 2020-10-AD-iSP-w64 are beneficial to modelling
- Local culvert C55.06 is to be included within the Macintyre Brook at Bybera Road hydraulic model
- Undertake additional model strengthening by improving the representation of cross-drainage infrastructure, survey data, and bridge structures within the hydraulic model
- In order to meet FIOs, additional analysis of cross-drainage infrastructure may be required, along with potential stakeholder consultation.

Further details of mitigation measures can be found in Section 22. A review of proposed mitigation measures will be completed during the detailed design stage of the Project, after hydraulic models have been updated to capture the above changes.

15.7 Planned and future roads

Consultation with the DTMR has not identified any planned or future road upgrades within the Macintyre Brook—Bybera Road catchment. ARTC will continue to consult and engage with DTMR to determine any future or planned road projects that may need to be considered as part of the detailed design hydraulic modelling.

Consultation with GRC has not identified any planned or future road upgrades within the Macintyre Brook—Bybera Road catchment. ARTC will continue to consult and engage with GRC to determine any future or planned road projects that may need to be considered as part of the detailed design hydraulic modelling.

16 Macintyre Brook at Cremascos Road

The location of the Project alignment in relation to Macintyre Brook Study area at Cremascos Road is shown in Figure L1-A in Volume 2 – Appendix L.

16.1 Data collection and review—Macintyre Brook at Cremascos Road

Available background information, including existing hydrologic and hydraulic models, survey, streamflow data, available calibration information and anecdotal flood data was gathered. Data was sourced from a wide range of stakeholders, including:

- GRC—existing flood studies
- BoM—rainfall gauging data
- DTMR—existing infrastructure details.

16.1.1 Previous studies

Previous hydrology and hydraulic studies were sourced as part of this assessment. A review of each study was undertaken to determine suitability for use on the Project, as documented in the following sections.

16.1.1.1 Inglewood Flood Study, Engeny, 2015

Engeny was commissioned by GRC to undertake a flood study of Inglewood. The study objectives were *‘to define the nature, extent and risks of flooding in Inglewood in order to inform disaster management planning and response, as well as control future development’* (Engeny, 2015). An URBS hydrologic model and TUFLOW (1D/2D) hydraulic model was developed.

16.1.1.2 Inland Rail: Phase 2—North Star to NSW/QLD Border – Hydrology and Flooding Calibration Report, FFJV, 2018

The NS2B section of Inland Rail will cross the Macintyre River and its floodplain, which are a part of the Border Rivers catchment. The NS2B alignment runs through Moree Plains LGA, Gwydir LGA and Goondiwindi LGA. To establish the reliability of the models, the 2011 event was included in the calibration process in addition to the 1976 and 1996 events assessed by DPIE. The hydrologic and hydraulic models were found to consistently represent flows and levels across the floodplain compared to the recorded 2011 event.

16.1.2 Survey

ARTC provided LiDAR data from 2015 as 1 m grid DEM tiles. Using GIS software, a DEM was generated with a 1 m grid resolution for use in the Project, based on the 2015 dataset. This was used for modelling within the permanent footprint and up to the full extent of the 2015 LiDAR where relevant.

In areas that were not covered by the LiDAR provided by ARTC, LiDAR tiles were sourced from Geoscience Australia. The DEM datasets utilised for modelling were based on surveys flown between 2009 and 2015. SRTM data was used for catchment delineation where no LiDAR data could be sourced, to inform the hydrologic modelling.

The survey data sources and DEM developed for the unnamed creek catchment upstream of Cremascos Road are shown in Figure L1-B in Volume 2 – Appendix L.

16.1.3 Aerial imagery

Aerial imagery of the Macintyre Brook study area at Cremascos Road was provided by ARTC and was used to identify and confirm topographic and vegetative characteristics of the study area. Aerial imagery captured in 2015 was made available. Additional imagery outside the study area was sourced from QGIS imagery in an open-source format.

16.1.4 Existing drainage structure data

DTMR as-constructed drawings were also sourced for culvert and bridge details. This information will be refined as the local survey is complete. Details of infrastructure included within the Cremascos Road catchment flood assessment is located in Section 16.3.4.

16.1.5 Stream gauge data

No streamflow gauges exist in the unnamed creek catchment study area upstream of Cremascos Road.

16.1.6 Rainfall data

A number of daily and sub-daily rainfall stations are located in and around the unnamed creek catchment upstream of Cremascos Road; however, since there are no streamflow gauges to use for model calibration, no historical rainfall data was sought.

16.1.7 Anecdotal and observed flood data

No anecdotal or observed flood data was available for this area of the Macintyre Brook.

16.1.8 Site inspection

A site inspection was undertaken during February 2018. During the site inspection, all major waterway crossings were visited and inspected, with photographs taken and details recorded of the crossing, existing drainage structures and surrounding catchment. An assessment of the relative roughness and blockage potential was undertaken during the site inspection.

16.2 Hydrologic model development—Macintyre Brook at Cremascos Road

16.2.1 Model setup

A hydrologic model of the Macintyre Brook at Cremascos Road catchment was established in URBS using the latest procedures detailed in ARR 2019 and the latest rainfall data from BoM. The new ARR procedures cover revisions made to hydrologic parameters such as losses, pre-burst depths, TPs and ARF.

The URBS model covers approximately 57 km² of the unnamed creek catchment upstream of Cremascos Road and the confluence with Macintyre Brook. The catchment was delineated into 14 sub-catchments to capture the variability of rainfall and to better represent the network of creeks and streams within the catchment.

The URBS model was setup as a 'BASIC' model. The hydrologic model setup including extent and sub-catchment map is presented in Figure L1-C in Volume 2 – Appendix L.

Model inputs are summarised in Table 16.1.

Table 16.1 Macintyre Brook at Cremascos Road—Summary of URBS model inputs

Input parameter	Remarks
URBS model type	Basic
Routing variables	Catchment area, stream lengths
Channel lag parameter, α	1.20
Catchment non-linearity parameter, m	0.8

Note that the default values were adopted for all other URBS parameters.

16.2.2 Sub-catchments

The Macintyre Brook at Cremascos Road catchment is comprised of approximately 14 sub-catchments and an approximate catchment area of 57 km².

The hydrologic model extent and sub-catchment map is presented in Figure L-1C in Volume 2 – Appendix L.

16.2.3 Design event parameters

Hydrologic information to assist estimation of design event flows was sourced from the ARR 2019 Data Hub, as summarised in Table 16.2. Details of the adopted hydrologic model parameters are presented below.

Table 16.2 Macintyre Brook at Cremascos Road—Summary of URBS model design event inputs

Input parameter	Remarks
Design rainfall	Intensity-Frequency-Durations for each sub-catchment were downloaded from the BoM's website to account for variation in rainfall across the catchment.
Extreme event rainfall	Probable Maximum Precipitation depths for durations up to 6 hours (for use in modelling the PMF event) were obtained using the method presented in the Bulletin 53 (BoM, 2003). The rainfall depths for the 1 in 10,000 AEP event were estimated using the interpolation method presented in ARR 2019 Book 8 Section 3.5.
Losses	The losses were adopted from the <i>Inglewood Flood Study</i> (Engeny, 2015) URBS model. Losses are fixed and do not vary with AEP: Initial loss—15 mm Continuing loss—1 mm/hr.
Areal reduction factor	Parameters were adopted for the semi-arid inland Queensland region. The catchment area U/S of the proposed rail crossing adjacent Cremascos Road is approximately 57 km ² , which yields an ARF between 77.0% and 96.0% depending on design storm event AEP and duration.
Ensemble temporal patterns	Central Slopes region
Pre-burst depths	Median pre-burst depths were downloaded from the ARR 2019 Data Hub for each sub-catchment. Pre-burst depths vary by design storm event AEP and duration. Pre-burst depths were applied to the model by reducing the initial losses for each storm event.

16.2.3.1 Spatial distribution

The ARR 2019 design event methodology was adopted for this assessment. ARR 2019 recommends adopting the spatially varying method for catchments larger than 20 km². The Cremascos Road catchment is approximately 57 km². A total of 14 sub-catchments were assigned an IFD derived from BoM website.

16.2.3.2 Rainfall depth estimation

Design rainfall depth for 20 per cent to 1 in 2,000 AEP were manually obtained from BoM website for all sub-catchments. As an example, the IFD representing the catchment centroid of Cremascos Road URBS model is provided in Table 16.3. Due to the size of the model area, there is minimal difference in rainfall depth across the catchment.

Table 16.3 Rainfall depth (mm) used for the Cremascos catchment centroid

Duration (hr)	Annual Exceedance Probability (% AEP)						
	50	20	10	5	2	1	0.05
0.25	17.2	23.9	28.4	32.9	38.9	43.6	70.9
0.50	23.4	32.4	38.6	44.8	53.1	59.6	97.2
1.00	29.4	40.7	48.6	56.4	67.0	75.1	123.0
2.00	35.5	48.7	58.1	67.4	80.0	89.8	146.0
3.00	39.3	53.6	63.8	73.9	87.7	98.6	160.0
4.50	43.5	59.0	69.9	80.9	96.1	108.0	175.0
6.00	46.8	63.2	74.7	86.4	103.0	116.0	187.0
12.00	56.3	75.3	88.8	102.0	122.0	138.0	223.0
24.00	68.1	90.8	107.0	124.0	149.0	169.0	275.0
48.00	81.7	109.0	130.0	151.0	182.0	207.0	362.0
72.00	89.8	121.0	144.0	168.0	202.0	230.0	381.0

16.2.3.3 Areal reduction factor

The ARF was estimated using the coefficients for semi-arid inland Queensland, presented in Table 16.4, and was applied to all IFD areas.

The URBS software directly downloaded these coefficients from ARR 2019 Data Hub to estimate the ARF using Equation 2.4.1 of ARR 2019 Book 2.

The estimated ARF for the Cremascos Road catchment varies between 77 and 96 per cent depending on design storm event AEP and duration.

Table 16.4 Macintyre Brook at Cremascos Road—Areal reduction factor coefficients from ARR 2019 Data Hub

a	b	c	d	e	f	g	h	i
0.159	0.283	0.25	0.308	7.30E-07	1.0	0.039	0.0	0.0

16.2.3.4 Temporal patterns

Temporal patterns for 20% AEP to 0.05% AEP events were adopted based on a 10-burst ensemble approach presented in the ARR 2019. The TPs and pre-burst data were directly downloaded by URBS software from the ARR 2019 Data Hub for the Cremascos Road catchment.

Temporal patterns for the 1 in 10,000 AEP and PMF design events for duration up to 6 hours were obtained from *The Estimation of Probable Maximum Precipitation in Australia: Generalised Short-Duration Method* (BoM, 2003).

The adopted TPs for all modelled events are shown in Table 16.5.

Table 16.5 Macintyre Brook at Cremascos Road—Adopted temporal patterns

Design event	Critical storm (hrs) duration and temporal pattern
20% AEP	Pattern 9
10% AEP	Pattern 7
5% AEP	Pattern 7
2% AEP	Pattern 7
1% AEP	Pattern 8

Design event	Critical storm (hrs) duration and temporal pattern
1 in 2,000 AEP (0.05%) AEP	Pattern 8
1 in 10,000 AEP (0.01%) AEP	PMP pattern
PMF	PMP pattern

16.2.3.5 Pre-burst

Median pre-burst depths for durations from 0.5 hour to 72 hours for a 1% AEP design event are between 0 mm and 10.9 mm (ARR 2019 Data Hub). Due to the small depth of pre-burst in this area, the impact of pre-burst condition is considered to be minimal in this catchment. The adopted pre-burst depths are summarised in Table 16.6.

Table 16.6 Macintyre Brook at Cremascos Road—Adopted pre-burst depths

Duration (hrs)	Pre-burst depth (mm)					
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP
0.5	0.7	1.0	1.1	1.3	1.6	1.8
0.75	0.7	1.0	1.1	1.3	1.6	1.8
1.0	0.7	1.0	1.1	1.3	1.6	1.8
1.5	0.3	0.4	0.5	0.5	1.5	2.3
2.0	0.2	0.3	0.3	0.4	1.5	2.3
3.0	0.1	0.0	0.0	0.0	2.5	4.3
6.0	0.5	0.6	0.7	0.7	0.9	1.0
9.0	0.0	0.2	0.3	0.4	5.4	9.2
12.0	0.0	0.2	0.3	0.4	5.4	9.2
18.0	0.0	0.3	0.4	0.6	5.8	9.7
24.0	0.0	0.5	0.8	1.2	6.7	10.9
48.0	0.0	0.0	0.0	0.0	0.1	0.1
72.0	0.0	0.0	0.0	0.0	0.0	0.0

16.2.3.6 Design rainfall losses

The adopted constant initial and continuing losses in the URBS model are 15 mm and 1 mm/hr, respectively. The losses were adopted from the *Inglewood Flood Study* (Engeny, 2015) URBS model, and validated by comparing the modelled flows and those from the RFFE. The URBS model validation is detailed in Section 16.2.6.

16.2.3.7 Extreme event rainfall

Probable maximum precipitation depths for durations up to 6 hours (for use in modelling the PMF event) were obtained using the methodology presented in *The Estimation of Probable Maximum Precipitation in Australia: Generalised Short-Duration Method* (BoM, 2003). Once the PMP was estimated, rainfall for events between 1 in 2,000 AEP and the PMP was then interpolated. For the 1 in 10,000 AEP event, ARF were applied based on interpolation methodology presented in ARR 2019 Section 3.5 in Book 8 and the rainfall depths were estimated accordingly. The parameters adopted for PMP rainfall estimation are presented in Table 16.7. The adopted extreme rainfall depths are provided in Table 16.8.

Table 16.7 Macintyre Brook at Cremascos Road—Input data for Probable Maximum Precipitation estimation

Factor	Value
EAF	1
MAF	0.8
Portion of rough catchment	100%
Bybera Road catchment location and catchment area (km ²)	(-28.410, 150.926) and 57 km ²

Table 16.8 Macintyre Brook at Cremascos Road—Adopted extreme event rainfall depths (mm)

Duration (hour)	Bybera Road catchment	
	1 in 10,000 AEP rainfall depth (mm)	PMP rainfall depth (mm)
0.25	89	153
0.5	122	223
0.75	141	283
1	157	337
1.5	171	434
2	184	503
2.5	195	562
3	205	610
4	219	693
5	231	761
6	241	811

16.2.4 Event selection for calibration

Insufficient data was available to calibrate historical flood events in the Macintyre Brook at the Cremascos Road catchment.

16.2.5 Model calibration

Insufficient data was available to calibrate historical flood events in the Macintyre Brook at the Cremascos Road catchment.

16.2.6 Hydrologic model validation

The hydrologic model for the Macintyre Brook at the Cremascos Road catchment was not calibrated due to unavailability of observed stream gauge data in the catchment; however, the routing parameter α was adjusted until there was a reasonable match between the URBS model flows, and those derived using DTMR's QRT and RFFE methods.

All URBS results reside within the 90 per cent confidence limits of the RFFE and show a close match with QRT. Adjusting losses for each AEP may improve the validation against QRT and RFFE and should be investigated at the next stage of reporting. The estimated flood flows at the outlet of the Macintyre Brook at Cremascos Road model are presented in Figure 16.1 and Table 16.9. The model outlet lies approximately 250 m downstream of where the unnamed creek crosses the Project alignment.

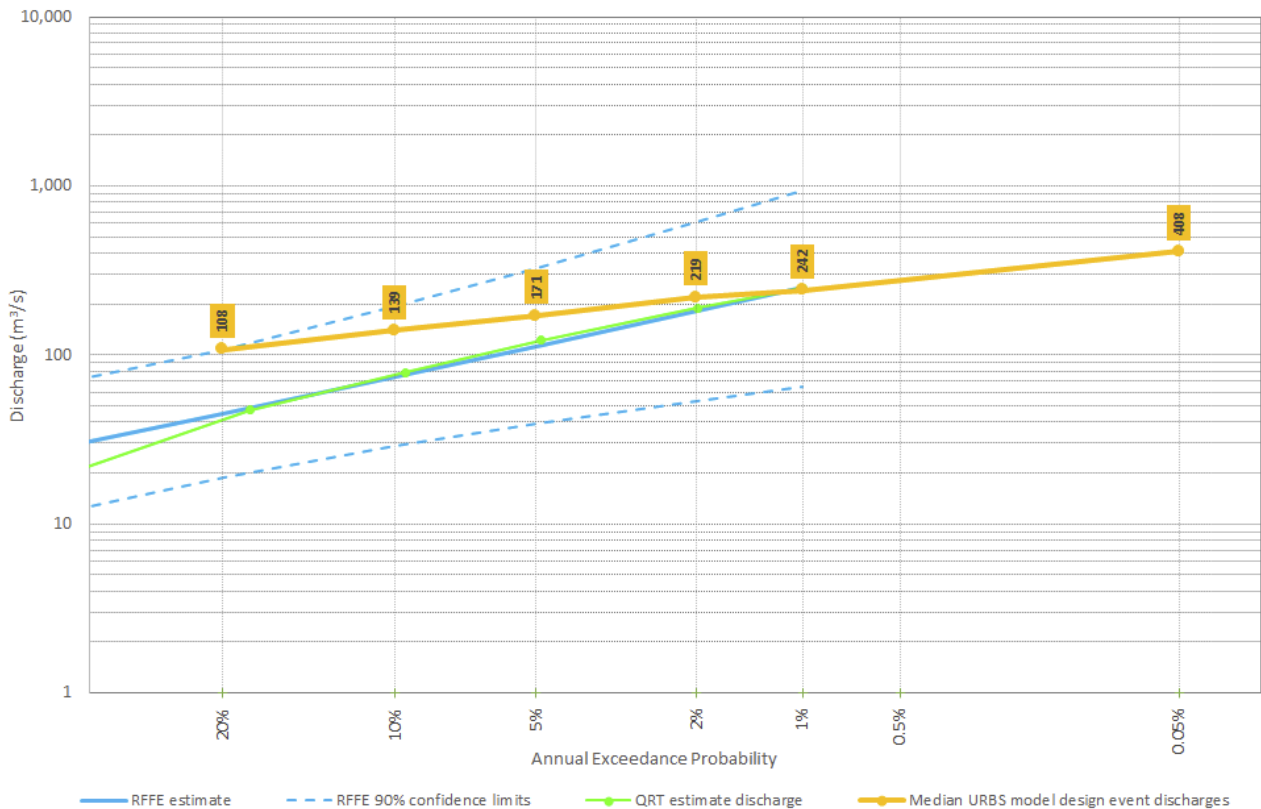


Figure 16.1 Estimate of flows at the outlet of Macintyre Brook at Cremascos Road model

Table 16.9 Estimate of flows at the outlet of Macintyre Brook at Cremascos Road model

AEP (%)	RFFE – lower bound 90% confidence level (m³/s)	RFFE – estimate of flow (m³/s)	RFFE – upper bound 90% confidence level (m³/s)	DTMR quantile regression technique (m³/s) ¹	URBS model flows (m³/s)
20	19	45	108	47	108
10	29	74	192	79	139
5	39	113	324	122	171
2	54	181	609	189	219
1	65	249	941	249	242
1 in 2,000	-	-	-	-	408
1 in 10,000	-	-	-	-	1,116
PMF	-	-	-	-	4,793

Table note:

¹ The QRT method estimates the 39.3%, 18.1%, 9.5% and 4.9% AEP instead of 50%, 20% and 10% and 5%, respectively

16.3 Hydraulic model development—Macintyre Brook at Cremascos Road

A 2D modelling approach was adopted to appropriately simulate flood mechanisms around the Project rail crossing adjacent Cremascos Road. The platform used for hydraulic modelling was TUFLOW HPC software package (2020-10-AD) with SGS approach. The processes and assumptions adopted throughout the development of the hydraulic model are described in the following sections.

16.3.1 Model setup

The setup of the TUFLOW model is summarised in Table 16.10.

The hydraulic model extent and the spatial distribution of land use in the 2D model domain is presented in Figure L1-D in Volume 2 – Appendix L, and landuse classification in Figure L1-E.

Table 16.10 Macintyre Brook at Cremascos Road— Hydraulic model summary

Parameter	Information
Completion date	June 2022
AEPs assessed	20%, 10%, 5%, 2%, 1%, 1 in 2,000, 1 in 10,000 AEP and PMF
Hydraulic model build	TUFLOW HPC GPU – version 2020-10-AD-iSP-w64
Model extent	Refer to Figure L1-D in Volume 2 – Appendix L
Grid size	5 m
SGS distance	1 m
DEM (year flown)	ARTC dataset (2015).
Roughness	Spatially varying roughness values compliant with industry norms.
Eddy viscosity	Wu (default)
Model calibration	N/A
D/S model boundary	Height-Discharge (HQ) Boundary with normal slope approximated based upon topography dataset.
Hydraulic model timestep	Adaptive timestep
Modelled scenarios	Existing Case, Developed Case
Sensitivity analysis	Climate change

16.3.2 Model topography and grid size

The topography within the TUFLOW hydraulic model was based on 1 m DEM data collected during 2015, which was supplied by ARTC. A 5 m x 5 m grid size was selected to allow the features of the channels and floodplain to be represented with sufficient accuracy while maintaining efficient model run times. The 5 m x 5 m grid size is considered adequate to represent floodplain topography and is consistent with the approach typically adopted in similar flood studies.

The 2D model topography was modified adjacent to ensure existing road/rail crowns and levee banks were represented accurately in the model. For the Developed Case, the topography was modified to include the Project rail embankment, drainage structures and associated infrastructure.

16.3.2.1 Sub-grid sampling

The Macintyre Brook at Cremascos Road hydraulic model was established using TUFLOW's 2020-10-AD model version, including SGS add-on module. The below points provide a brief description of the module, along with the key parameters adopted for this assessment.

- Traditional grid-mesh based 2D hydraulic models sample the underlying DEM by taking a single elevation at the cell centres and cell faces. Sub-grid sampling allows for increased resolution by sampling a greater number of points of the underlying DEM to establish a water surface elevation versus volume relationship for each grid, along with a water surface elevation versus width relationship (wetted perimeter) at cell sides. The adoption of SGS allows increased representation of storage and conveyance between cells. A SGS distance of 1 m was adopted for this assessment.

16.3.3 Roughness

The hydraulic roughness generally reflects the types of development and ground cover that exists within the hydraulic model extent. The distribution of roughness categories adopted for this assessment was based on the information supplied in the land use layer from Queensland Globe database, aerial imagery and confirmed during the site inspection. Specific roughness values applied to the model are detailed in Table 16.11.

Table 16.11 Macintyre Brook at Cremascos Road catchment—Adopted roughness values

Category	Manning's 'n' value
River	0.035
Sealed roads	0.015
Unsealed roads	0.025
Rural residential	0.09
Urban residential	0.10
Commercial	0.06
Open space—Minimum vegetation	0.045
Open space – Moderate vegetation	0.055
Open space – Heavy vegetation	0.065
Open water	0.025
Rail	0.025
Riparian	0.04
Crops ¹	0.1085

Table note:

- 1 A uniform value was adopted for the crop areas, due to the seasonal and spatial variations in vegetation distribution and the uncertainty this brings to the design event approach

16.3.4 Hydraulic structures

No existing hydraulic structures are situated within the extents of the Cremascos Road hydraulic model.

16.3.5 Boundary conditions

The following boundary conditions were applied within the hydraulic model:

- Flow versus time model boundaries were applied for all inflow hydrographs, with local and total hydrographs sourced from the hydrologic models
- Delineated sub-catchments in the hydrologic model were used as internal flow boundaries within the hydraulic model extent (SA polygons). The modelled flows in the hydrologic model were applied to the lowest points of each SA polygon.
- Where the Project alignment intersected the sub-catchment, the inflows were divided accordingly based on catchments upstream and downstream of the alignment
- A height-discharge boundary (HQ) was applied for the downstream outflow model boundary. It was confirmed that it was a sufficient distance away from the area of interest so as not to impact on modelling results.

16.4 Existing Case modelling results—Macintyre Brook at Cremascos Road

16.4.1 Hydrologic model peak flow assessment

A critical duration assessment was undertaken to determine which storm duration(s) produced peak flood flows where the major waterways are intersected by the Project alignment and at the downstream outlet of the model. To assess the critical storm duration the following methodology was adopted:

- Modelling was completed for a range of AEP events:
 - Each AEP was modelled for a range of durations
 - Each duration was modelled for each of the 10 associated TPs.

A critical duration assessment was undertaken at the locations mentioned above to determine which duration produced the highest median flow of the 10 TPs for each event. The critical duration and adopted TP for each design event AEP is provided in Table 16.12. Table 16.13 demonstrates the estimated peak flow for the 1% AEP event.

Table 16.12 Macintyre Brook at Cremascos Road—Peak flow at key locations as applied in the hydraulic model

Design Event	Critical storm (hrs) duration and temporal pattern
20% AEP	9 hours - Pattern 9
10% AEP	6 hours - Pattern 7
5% AEP	4.5 hours - Pattern 7
2% AEP	4.5 hours - Pattern 7
1% AEP	4.5 hours - Pattern 8
1 in 2,000 AEP (0.05%) AEP	4.5 hours - Pattern 8
1 in 10,000 AEP (0.01%) AEP	6 hours – PMP pattern
PMF	6 hours – PMP pattern

Table 16.13 Macintyre Brook at Cremascos Road—1% AEP event peak flow at key locations as applied in the hydraulic model

Sub-catchment name	1% AEP event peak flow (m ³ /s)	Critical storm (hrs) duration and temporal pattern
07T	196.0	4.5 hours - Pattern 8
10T	71.5	4.5 hours - Pattern 8
11L	5.7	4.5 hours - Pattern 8
12L	15.4	4.5 hours - Pattern 8
13L	9.5	4.5 hours - Pattern 8
14L	9.4	4.5 hours - Pattern 8
14T	241.9	4.5 hours - Pattern 8

16.4.2 Existing Case flood maps

Existing conditions flood maps are presented in Volume 2 – Appendix L and via an [Online Digital Portal](#). Appendix L contains the Existing Case 20% and 1% AEP flood maps, while the [Online Digital Portal](#) contains the remainder of the modelled events. The location of each map set is summarised below:

- 20% AEP peak levels: Figure L2-A
- 1% AEP peak levels: Figure L2-B

- 20% AEP peak velocity: Figure L2-C
- 1% AEP peak velocity: Figure L2-D
- 20% AEP peak hazard: Figure L2-E
- 1% AEP peak hazard: Figure L2-F
- 20% AEP peak time of inundation: Figure L2-G
- 1% AEP peak time of inundation: Figure L2-H
- 20% AEP, 10% AEP, 5% AEP, 2% AEP, 1% AEP, 1 in 2,000 AEP, 1 in 10,000 AEP and PMF Existing Case results: [Online Digital Portal](#)
- For higher resolution mapping refer to the [Online Digital Portal](#).

16.4.3 Existing Case flood inundation extent and flood levels

The estimated peak flood depth for the 1% AEP event is approximately 5.6 m within the Cremascos Creek channel, with an average flood depth of around 1.0 m in other areas of the floodplain. The flood depth on the floodplain is estimated to be up to 2.8 m (270.6 mAHD) where the Project alignment crosses the floodplain.

Figure L2-B in Volume 2 – Appendix L shows the 1% AEP indicative flood extent and peak water levels within the Macintyre Brook at Cremascos Road floodplain for the Existing Case.

16.4.4 Existing Case flood immunity of existing infrastructure

Table 16.14 presents a summary of estimated overtopping depths for key roads within the hydraulic model domain under a range of design events. Modelling results show that Cremascos Road has a low existing flood immunity in the areas close to the Project alignment. The Cunningham Highway, approximately 1.5 km downstream of the Project alignment, is demonstrated by modelling results to have an immunity up to and including the 1 in 2,000 AEP event.

Table 16.14 Macintyre Brook at Cremascos Road: Existing Case—Overtopping depths of key infrastructure

Infrastructure	Overtopping depth (m)							
	PMF	1 in 10,000 AEP	1 in 2,000 AEP	1% AEP	2% AEP	5% AEP	10% AEP	20% AEP
Cunningham Highway	1.34	0.02	0.00	0.00	0.00	0.00	0.00	0.00
Cremascos Road	7.28	4.64	3.41	2.92	2.83	2.61	2.39	2.26

16.4.5 Existing Case velocities

Existing peak flood velocities for the 1% AEP event are expected to reach 4.7 m/s in localised areas, with the average velocity across the floodplain approximately 0.5 m/s. Velocities within the channel, adjacent to the Project alignment, are in the order of 1.7 m/s as shown in Figure L2-D in Volume 2 – Appendix L.

16.4.6 Existing Case flood hazard

For the 1% AEP event, an existing velocity x depth product of $>4.2 \text{ m}^2/\text{s}$ is achieved within the proposed bridge crossing site. The floodplain has an average velocity x depth produce of less than $1 \text{ m}^2/\text{s}$, as shown in Figure L2-F in Volume 2 – Appendix L.

16.4.7 Existing Case flood duration of inundation

Results for the 1% AEP event highlights flood water currently inundating areas upstream of the Cremascos Road for periods greater than 5 hours. The profile also highlights ponding areas created by farming activities and existing road infrastructure, as shown in Figure L2-H in Volume 2 – Appendix L.

16.5 Developed Case modelling results—Macintyre Brook at Cremascos Road

16.5.1 Drainage structures

The hydraulic design of the major drainage structures was undertaken using the TUFLOW model (1d and 2d approach). Within the Macintyre Brook at Cremascos Road hydraulic model, the Project includes one waterway bridge and one 350 m long catch drain adjacent the rail embankment draining towards the Cremascos Road bridge crossing. A cost-benefit analysis determined that it was more feasible to construct and maintain a 350 m long catch drain at the toe of the rail embankment batter that diverted flows south-west to the bridge opening. Constructing several large culverts beneath over 5 m of fill would carry significant long-term operational risks for The Project. No culverts are included within the Macintyre Brook at Cremascos Road hydraulic model.

The proposed bridge is summarised in Table 16.15 and presented in Figure L1-F in Volume 2 – Appendix L. The 1% AEP flood level at the proposed bridge structure is presented in Appendix B.

Table 16.15 Macintyre Brook at Cremascos Road—Proposed bridge location and details

Chainage (km)	Structure ID	Approximate span (m)	Deck width (m)	Soffit level (mAHD)	Deck superstructure type	Deck depth (mm)
52.58	310-BR03	184	3.97	276.02	Type D1	2,000

Bridges were modelled as an opening in the rail embankment. This approach was adopted due to the high soffit level of the bridge (approximately 7 m above the 1% AEP event peak water surface level); therefore, interaction between the bridge deck and flood waters is unlikely. The optimisation of bridge lengths was balanced between minimising the changes to the hydraulic regime, primarily afflux and velocities, and the cost of replacing bridge spans by large earth embankments.

Blockage of hydraulic structures was assessed in accordance with ARR 2019. The predominant vegetation in the catchment is considered heavily forested; however, given the appreciable size of the bridge spans, and the relative size of upstream debris, a blockage factor was not applied to bridges. Significant amounts of debris accumulating against the piers of the bridge are considered low.

16.5.2 Hydraulic design criteria outcomes

The hydraulic model was run for the Developed Case for the range of events (20%, 10%, 5%, 2%, 1% AEP events and 1 in 2,000 AEP, 1 in 10,000 AEP and PMF events) with the outcomes presented in the following sections.

16.5.2.1 Rail immunity and structures results

Appendix B summarises the peak 1% AEP water levels on the upstream side of the Project alignment.

Results indicate that peak water levels remain below the proposed Project rail formation level. There is 7.3 m of freeboard to the bridge soffit level and the rail formation, noting that the formation levels were generally dictated by other engineering criteria and constraints.

The results of flood modelling therefore indicate that a 1% AEP event flood immunity to the proposed rail formation is achieved for the Project alignment across the Macintyre Brook at Cremascos Road floodplain.

16.5.3 Flood Impact Objectives outcomes—Macintyre Brook at Cremascos Road

The impact of the Developed Case was assessed through inclusion of the proposed rail design and comparison of model results against the Existing Case results.

Changes to flooding behaviour that were assessed, and are reported in the following sections, include potential changes in peak water levels, time of inundation, velocity, hazard and peak flows:

- Changes in 20% AEP and 1% AEP peak water levels are presented in Figures L4-A and L4-B in Volume 2 – Appendix L
- Changes in 20% AEP and 1% AEP velocities are presented in Figures L4-C and L4-D in Volume 2 – Appendix L
- Changes in 20% AEP and 1% AEP hazard are presented in Figures L4-E and L4-F in Volume 2 – Appendix L
- Changes in 20% AEP and 1% AEP time of inundation are presented in Figures L4-G and L4-H in Volume 2 – Appendix L
- Changes in 20% AEP and 1% AEP velocities and percentage change in velocities with cut-offs applied are presented in Figures L5-A to L5-F in Volume 2 – Appendix L
- Percentage changes in 20% AEP and 1% AEP in hazard are presented in Figures L6-A and L6-B in Volume 2 – Appendix L
- Percentage changes in 20% AEP and 1% AEP in time of inundation and with cut-offs applied are presented in Figures L7-A to L7-D in Volume 2 – Appendix L
- Refer to the [Online Digital Portal](#) for other modelled AEP results and higher resolution mapping.

One-on-one consultation with landowners who are expected to experience changes in flooding behaviour on their property was conducted by ARTC, supported by FFJV. Refer to EIS Chapter 14: Flooding and Geomorphology and EIS Appendix E: Consultation Report for further details.

The Project design outcomes relative to the FIOs (Table 4.2) are presented in the following sections.

Flood sensitive receptors referred to in this section are shown in Figures P1 (1 to 59) Volume 2 – Appendix P.

16.5.3.1 Flood impacts on flood sensitive receptors

Based on the available aerial imagery, no buildings or critical infrastructure are located within the area impacted by afflux and change in flood hazard in the Macintyre Brook at Cremascos Road floodplain for modelled events up to the 1 in 2,000 AEP event.

16.5.3.2 Flood impacts on State-controlled roads

The extent of the hydraulic model developed for Macintyre Brook at Cremascos Road is shown in Figure L1-D in Volume 2 – Appendix L. Within the extent, the only State-controlled road, influenced by the Project and flooding, is the Cunningham Highway.

Developed Case flooding from the Macintyre Brook at Cremascos Road catchment and comparison against the FIOs has demonstrated that no adverse impacts on the Cunningham Highway are expected due to the presence of the Project, for all modelled events up to and including the 1% AEP event.

16.5.3.3 Flood impacts on local public roads

Within the extent of the hydraulic model, two local public roads are influenced by flooding. Table 16.16 identifies the location of potential FIO exceedances and the details for each exceedance, including afflux, velocity, time of inundation and hazard. Justification and proposed mitigation measures are also provided in Table 16.16.

The following provide a summary of the key exceedances:

- The unnamed road (Location 2 in Table 16.16) shows the greatest impacts with a maximum afflux of approximately 0.032 m, and for an extent of 2.5 m for the 1% AEP design flood event.
- Cremascos Road (Location 5 in Table 16.16) has a 34 per cent change in time of inundation (0.5 hours) for an extent of 2.5 m for the 20% AEP design event.

The extent of the model, and location of the public local roads with exceedances is shown in Figure 16.2.

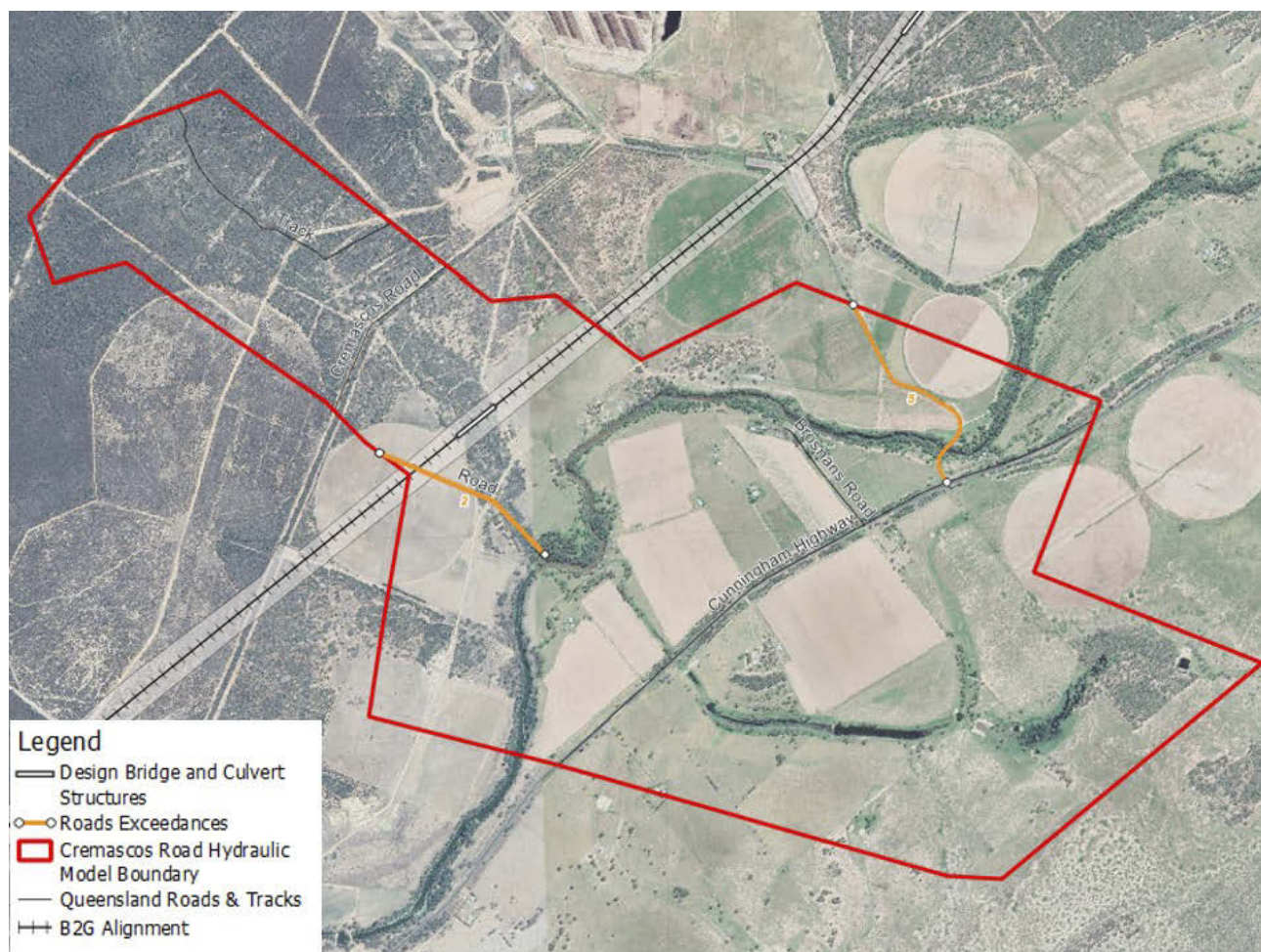


Figure 16.2 Macintyre Brook at Cremascos Road—Impacted local public roads

Table 16.16 Macintyre Brook at Cremascos Road - Summary of impacts on local public roads

Road Object ID	Road Segment	Road Name	Surface Type	Flood Impact Objective Target					AEP	Maximum FID Exceedance					Length (m)				Exceedance Justification	Proposed Mitigation Measures ⁷
				Change in Peak Water Level (mm)	Change in Time of Inundation (%)	Change in Velocity (%)	Change in Velocity (m/s)	Change in Hazard (%)		Change in Peak Water Level (mm)	Change in Time of Inundation (% Change)	Change in Velocity (%)	Change in Velocity (m/s)	Change in Hazard (%)	Afflux	Time on Inundation	Velocity	Hazard		
2	16015103990000432573	Road 0	Unknown	10	10	10	0.5	10	20%	12	14	-	-	-	145.00	2.50	-	-	This road is indicated to cross an existing waterway where the wide upstream waterway funnels through a narrow and deep section. As a result, this section is highly sensitive to the change in upstream flood dynamics. The diversion of runoff near CH52.9km towards the bridge opening leads to an increase in flood volume and results in an increase in afflux and time of inundation. The exceedances remains due to the current design being at a reference design stage with optimisation of the cross drainage (whether this be a diversion or culverts) yet to be completed in this location. Further iteration of cross drainage needs to be undertaken at detailed design stage.	To mitigate this exceedance, a review of cross drainage/diversion at approximate Chainage 52.9km will be undertaken in detailed design to identify the need for the diversion of overland flow. In addition, confirmation of the road location will be undertaken given aerial imagery does not indicate a road is present in this location.
									10%	-	23	-	-	-	-	2.50	-	-		
									5%	-	-	-	-	-	-	-	-	-		
									2%	11	-	-	-	-	2.50	-	-	-		
									1%	32	13	-	-	-	2.50	5.00	-	-		
									0.05%	-	-	-	-	-	-	-	-	-		
5	1601510399000087993	Cremascos Road	Sealed	10	10	20	1	10	20%	-	34	-	-	-	-	2.50	-	-	Rail design constraints alter the existing flow regime.	Stakeholder Consultation (Landowner/ Government).
									10%	-	-	-	-	-	-	-	-	-		
									5%	-	-	-	-	-	-	-	-	-		
									2%	-	-	-	-	-	-	-	-	-		
									1%	-	-	-	-	-	-	-	-	-		
									0.05%	-	-	-	-	-	-	-	-	-		

- Table Notes
1. Time of Submergence has been measured across the full length of the road segment and does not correspond to the exact location of exceedance within that segment
 2. Change in water level (afflux) is calculated based on the maximum water surface level difference between the existing and design case results, and does not consider the underlying topography. Depth values presented are based on the respective underlying topographies and in some cases do not align with afflux calculations. This can be attributed to interpolation calculations of results performed within the hydraulic model.
 3. The above table should be read in conjunction with change maps provided as part of Volume II and the Digital Platform.
 4. The symbol "-" denotes that no change above Flood Impact Objectives has been determined and therefore no result has been presented.
 5. Changes recorded above may only occur during a small fraction of model simulation time.
 6. Three decimal places do not infer absolute accuracy in modelling results, but it is necessary to identify small changes.
 7. Design optimisations that could be implemented (but not limited to) include adjustment of culvert spacing, realignment of culverts to existing flowpaths when more detailed survey is available, modifying the grade of culverts, road height and alignment adjustments, removal of bridge abutments from waterway extents etc

Depth (m)		Time of Inundation in hours (related to % exceedance)		Time of Submergence (hours) ¹		Velocity in m/s (related to % exceedance)		Velocity in m/s (related to actual change exceedance)		Hazard in m/s ² (related to % exceedance)	
Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case
3.793	3.805	0.51	0.58	9.95	9.96	-	-	-	-	-	-
-	-	1.06	1.31	6.47	6.47	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
0.030	0.033	-	-	-	-	-	-	-	-	-	-
-	0.032	1.52	1.72	4.71	4.71	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
-	-	1.53	2.06	5.36	5.36	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-

16.5.3.4 Flood impacts on existing rail infrastructure

No third-party rail infrastructure is located within the Macintyre Brook at Cremascos Road catchment hydraulic model boundary.

16.5.3.5 Flood impacts on private land outside the permanent footprint

In accordance with the FIOs, Table 16.17 presents the modelled changes and potential impacts on third-party property outside of the Project's permanent footprint. A summary of key impacts, exceedance justification and mitigation measures are provided in the below points, with additional detail provided in Table 16.17:

- One property was identified as having a potential impact exceeding the FIO criteria
- No increase in change in water level (afflux) are predicted on third-party property
- Maximum change in velocity for all properties is approximately 0.09 m/s
- Maximum change in time of inundation is 26 per cent
- No change in hazard is expected
- Exceedances may be a result of the maturity of current cross-drainage (i.e. bridge) design
- In order to mitigate potential exceedances, further cross-drainage optimisation investigations, and ongoing model strengthening may occur in detailed design.

Table 16.17 Macintyre Brook at Cremascos Road – Summary of flood impacts on private land outside the permanent footprint

ID	ID (Lotplan ID/ ¹ FSR ID/ Culvert ID)	Sensitive Receptor Type	Catchment Type	Landuse ¹¹	Lot Area (m ²)	Flood Impact Objective Target						AEP	Maximum FIO Exceedance						Area (ha) ¹			Exceedance Justification	Proposed Mitigation Measures ¹⁸	
						Change in Peak Water Level (mm)	Change in Time of Inundation (%)	Change in Time of Inundation (Hours)	Change in Velocity (%)	Change in Velocity (m/s)	Change in Hazard (%)		Change in Peak Water Level (mm)	Change in Time of Inundation (%) ²	Change in Time of Inundation (Hours)	Change in Velocity (%) ²	Change in Velocity (m/s)	Change in Hazard (%) ²	Afflux	Time on Inundation	Velocity			Hazard
1	1MH171	Lot Type Parcel	Regional	ANIMALS-SPECIAL	600,718	200	10	1	10	0.5	20	20%	-	-	-	-	0.09	-	-	-	0.01	-	- Maturity of current bridge design is limited by available information (e.g. detailed survey)	- Further cross-drainage optimization investigations are required during detailed design.
												10%	-	26	-	-	-	-	-	-	0.01	-		
												5%	-	-	-	-	-	-	-	-	-	-		
												2%	-	-	-	-	-	-	-	-	-	0.003		
												1%	-	23	-	-	0.01	-	-	0.01	0.003			
												0.05%	-	-	-	-	-	-	-	-	-			

Table Notes

1. Floodplains are dynamic in nature and therefore model results vary from cell to cell. The "Area" is therefore defined as the maximum area over which change above the Flood Impact Objective occurs and does not relate directly to the maximum FIO exceedance recorded.
2. Change in water level (afflux) is calculated based on the maximum water surface level difference between the existing and design case results, and does not consider the underlying topography. Depth values presented are based on the respective underlying topographies and in some cases do not align with afflux calculations. This can be attributed to interpolation calculations of results performed within the hydraulic model.
3. Percentage increases should be read in conjunction with existing and design case results. The FIOs require that change be presented in percentage change for time of inundation, velocity and hazard and can sometimes produce large increases when existing case results are significantly small. Some locations were identified to experience significant percentage increases however the "absolute" difference between Existing and Design results are relatively small.
4. The Flood Impact Objective exceedance has been identified due to a change in flood extent and a cell which was dry in the existing case and now flood-affected in the developed case.
5. The above table should be read in conjunction with change maps provided as part of Volume II and the Digital Platform.
6. The symbol "-" denotes that no change above Flood Impact Objectives has been determined and therefore no result has been presented.
7. Changes recorded above may only occur during a small fraction of model simulation time.
8. Three decimal places do not infer absolute accuracy in modelling results, but it is necessary to identify small changes.
9. Flood impact objectives are assessed based on landuse applied to the entire lot and exceedances identified do not necessarily indicate an impact to a dwelling.
10. Design optimisations that could be implemented (but not limited to) include adjustment of culvert spacing, realignment of culverts to existing flowpaths when more detailed survey is available, modifying the grade of culverts, road height and alignment adjustments, removal of bridge abutments from waterway extents etc
11. Landuse data, dated 2018, has been used for this assessment. Due to the changing nature of this dataset, this will be updated to the latest available information in Detailed Design.

Depth (m) ²		Time of Inundation in hours (related to % exceedance)		Time of Inundation in hours (related to actual change exceedance)		Velocity in m/s (related to % exceedance)		Velocity in m/s (related to actual change exceedance)		Hazard in m ² /s (related to % exceedance)	
Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case
-	-	-	-	-	-	-	-	0.414	0.504	-	-
-	-	1.25	1.58	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	0.499	0.507	-	-
-	-	1.04	1.28	-	-	-	-	0.495	0.503	-	-
-	-	-	-	-	-	-	-	-	-	-	-

16.5.3.6 Flow distribution

A key landowner concern is the potential change in flow distributions. To determine the change in flow distribution maximum velocity-depth grids were used to produce change in unit flow maps between existing and design conditions. Unit flow is the measure of discharge (velocity-depth product) and is often used for assessing peak flow along a cross-section, or to determine the effective flow corridors within a floodplain.

The presented flow distribution maps were filtered with a velocity-depth product cut-off of 0.15 m²/s. The cut-off is in line with FIO requirements, and was used to assist in identifying main channels (high flow zones) from shallow lower flood zones (i.e. outer extents of the flood surface).

The following points summarise the potential change in flow distribution for the Macintyre Brook at Cremascos Road catchment and should be considered in conjunction with flow distribution figures supplied in Figure Q12-1 to Figure Q12-8 of Volume 2 – Appendix Q:

- Negligible change in flow distribution patterns for all events up to and including 1 in 2,000 AEP event
- Within the higher flow zones (>0.15 m³/s/m), minimal changes are observed up to and including the 1 in 2,000 AEP event
- The majority of changes occurs within the Macintyre Brook, with a 2 to 5 per cent increase
- Some positive and negative change is observed directly adjacent the Cremascos Road bridge crossing in the 1 in 10,000 AEP event with this propagating upstream and downstream in the PMF event.

Ongoing mitigation measures to reduce changes in flow distribution are likely to occur during future design stages of this Project, and may include further refinement of the bridge abutments and additional survey where necessary.

16.5.4 Extreme events

16.5.4.1 Extreme event immunity

The risk of overtopping of the Project alignment was assessed for a range of extreme events (1 in 2,000 AEP, 1 in 10,000 AEP and PMF) with Table 16.18 presenting the depth of water above the rail formation level and over the top of rail at each structure. It is noted that the function of the floodplain culverts is to balance flood levels on the upstream and downstream sides of the alignment.

As such, overtopping of the rail is not predicted to result in significant excessive flows or velocities as would occur in a dam embankment overtopping scenario. Further analysis of the overtopping risk to FSRs can be found in Section 21.

The rail formation and Bridge 310-BR03 at Ch 52.6 km do not overtop in extreme local flood events.

Table 16.18 Macintyre Brook at Cremascos Road: Extreme events—Estimated depth of water above formation and top of rail levels

Chainage (km)	Structure ID	Depth of water above formation level (m)			Depth of water above top of rail (m)		
		1 in 2,000 AEP	1 in 10,000 AEP	PMF	1 in 2,000 AEP	1 in 10,000 AEP	PMF
52.6	310-BR03	-	-	-	-	-	-

16.5.4.2 Impact during extreme events

Flood immunity of FSRs is discussed in Section 16.5.3.1, and maps demonstrating the impacts during extreme events are shown in the [Online Digital Portal](#).

In accordance with the FIOs, FSRs are considered impacted by extreme flood events if the change in water level is greater than 250 mm in the 1 in 2,000 AEP event, subject to the receptor classification (i.e. at existing habitable dwellings and/or commercial and industrial buildings/premises). In addition, a stricter afflux objective of 10 mm in the 1 in 500 AEP event applies to sites with existing sensitive infrastructure, uses involving vulnerable people and/or any critical road network that was designed to be immune to flooding in any extreme event.

Review of the 1 in 2,000 AEP event results for the Cremascos Road catchment against the FIOs show that flood level changes at all FSRs were limited to 250 mm, and no identified existing sensitive infrastructure or uses involving vulnerable people are expected to experience a more than 10 mm increase in flood level. A separate assessment of the 1 in 500 AEP event is not required as they are conservatively represented by the 1 in 2,000 AEP event results. The flood impacts in extreme events due to the Project are therefore compliant against the stipulated FIO limits for extreme events.

Table 16.19 details the extreme event results, although it should be noted that FIO targets do not apply to the 1 in 10,000 AEP and PMF events.

Table 16.19 Macintyre Brook at Cremascos Road catchment: Extreme events—Impacts at flood sensitive receptors

FSR ID	FSR Type	1 in 2,000 AEP event		1 in 10,000 AEP event		PMF event	
		Change in peak water level (mm)	Existing Case flood depth (m)	Change in peak water level (mm)	Existing Case flood depth (m)	Change in peak water level (mm)	Existing Case flood depth (m)
MCB_ID_105	House	-	-	-	-	+1	0.77
MCB_ID_106	House	-	-	-	-	+1	0.23
MCB_ID_107	Shed	-	-	-	-	+1	0.56
MCB_ID_108	House	-	-	-	-	+2	1.39
MCB_ID_109	House	-	-	-	-	+2	1.13
MCB_ID_110	Shed	-	-	-	-	+1	1.15
MCB_ID_111	Shed	-	-	-	-	+1	1.34
MCB_ID_114	Shed	-	-	-	-	+1	0.66
MCB_ID_115	House	-	-	-	-	+1	0.64
MCB_ID_116	Shed	-	-	-10	0.33	-14	1.64
MCB_ID_117	House	-	-	-14	0.37	-76	1.78
MCB_ID_118	Shed	-	-	-	-	-82	1.46
MCB_ID_119	Shed	-	-	-	-	-2	1.10
MCB_ID_120	Shed	-	-	-	-	-10	0.79
MCB_ID_121	Shed	-	-	-15	0.24	-105	1.78
MCB_ID_122	Shed	-	-	-	-	-3	1.32
MCB_ID_123	House	-	-	-	-	-6	0.88
MCB_ID_124	House	-	-	-	-	-6	0.74
MCB_ID_125	House	-	-	-	-	-7	0.59
MCB_ID_126	Shed	-	-	-	-	-7	0.81
MCB_ID_127	Shed	-	-	-	-	-7	0.59
MCB_ID_128	Shed	-	-	-	-	-6	0.91
MCB_ID_129	House	-	-	-	-	-4	0.48
MCB_ID_130	Shed	-	-	-	-	-5	0.80
MCB_ID_131	Shed	-	-	-	-	-9	1.03
MCB_ID_132	House	-	-	-	-	-9	0.89

FSR ID	FSR Type	1 in 2,000 AEP event		1 in 10,000 AEP event		PMF event	
		Change in peak water level (mm)	Existing Case flood depth (m)	Change in peak water level (mm)	Existing Case flood depth (m)	Change in peak water level (mm)	Existing Case flood depth (m)
MCB_ID_133	Shed	-	-	-	-	-9	0.61
MCB_ID_134	Shed	-	-	-	-	-9	0.80
MCB_ID_135	House	-	-	-	-	-9	0.42
MCB_ID_136	Shed	-	-	-	-	-9	0.31
MCB_ID_137	House	-	-	-	-	-8	0.21
MCB_ID_138	Shed	-	-	-	-	-8	0.30
MCB_ID_139	Shed	-	-	-	-	-9	0.31
MCB_ID_140	Shed	-	-	-	-	-9	0.39
MCB_ID_141	House	-	-	-	-	-6	0.67
MCB_ID_142	Shed	-	-	-	-	-5	0.65
MCB_ID_143	Shed	-	-	-	-	-5	0.59
MCB_ID_144	Shed	-	-	-	-	-4	0.20
MCB_ID_145	House	-	-	-	-	-3	0.12
MCB_ID_146	Shed	-	-	-	-	-4	0.11
MCB_ID_148	House	-	-	-	-	-8	0.61
MCB_ID_149	House	-	-	-	-	-9	0.78
MCB_ID_563	Shed	-	-	-	-	+1	0.59
MCB_ID_564	House	-	-	-	-	-10	0.96

16.5.5 Sensitivity analysis—Macintyre Brook at Cremascos Road

The sensitivity of the model to blockages at hydraulic structures and an increase in rainfall intensity to reflect climate change scenario was assessed.

16.5.5.1 Climate change

The potential impacts of climate change in the Macintyre Brook at Cremascos Road floodplain were assessed for the 1% AEP design event to determine the sensitivity of the Project to the potential long-term changes in climate. The assessment was undertaken in accordance with ARR 2019.

The Representative Concentration Pathways 8.5 (2090 horizon) climate change scenario was adopted for the Project, with an associated increase in rainfall intensity of 23.9 per cent across the catchment area.

The climate change sensitivity assessment results for FSRs indicated that flood FSR are outside the 1% AEP flood extent under the existing and climate change scenario; therefore, no flood impacts are observed under this climate change scenario.

Table 16.20 presents the structure performance with Representative Concentration Pathways 8.5 climate change conditions. For the 1% AEP event, the change in peak water levels for the Representative Concentration Pathways 8.5 climate change scenario is presented in Figure L8-A in Volume 2 – Appendix L.

Climate change results are expected to increase peak water levels upstream of the Project alignment by up to 0.3 m at proposed bridge 310-BR03 for the 1% AEP event. The Project alignment is expected to retain 1% AEP flood immunity to formation level under the climate change scenario.

Table 16.20 Macintyre Brook at Cremascos Road: 1% AEP event with Representative Concentration Pathways 8.5 conditions—Structure performance

Structure ID	Structure type	1% AEP peak water level (mAHD)	1% AEP +CC peak water level (mAHD)	Difference in peak water level (m)	Freeboard to rail formation with CC (m)
310-BR03	Bridge	270.9	271.1	+0.3	7.0

For the 1% AEP event, the change in peak water levels for the Representative Concentration Pathways 8.5 climate change scenario is presented in Figure L8-A in Volume 2 – Appendix L.

16.5.5.2 Blockage

Blockage of hydraulic structures was assessed in accordance with ARR 2019. Despite the catchment being vegetated and heavily forested, ARR guidelines determined that the likelihood of significant amounts of debris accumulating against the piers of the bridge is low; therefore, a zero-blockage factor was applied at the Cremascos Road bridge. Additionally, there are no regional culverts in the Cremascos Road floodplain, hence no sensitivity scenarios were conducted.

During detailed design the blockage factors will be reviewed in line with the final design and local catchment conditions. This may result in a varied and/or lower blockage factors being applied along the Project alignment. Should impacts be determined due to a change in blockage factor, additional mitigation measures will be considered in detailed design.

16.6 Additional modelling items to be considered in detailed design

The following points summarise additional items to be included in the flood models for detailed design as per agreement with the Expert Flood Panel:

- Bridge loss coefficients and blockage factors to be reviewed and updated if necessary
- Sub-catchment delineation and hydrology to reviewed, and where appropriate updated
- A review of adopted roughness delineation will be completed, and where appropriate updated
- The application of SA polygon inflow boundaries are to be reviewed, and where appropriate updated
- A review of the latest TUFLOW version will be undertaken to determine whether updates of the TUFLOW version from 2020-10-AD-iSP-w64 are beneficial to modelling
- Undertake additional model strengthening by improving the representation of cross-drainage infrastructure, survey data, and bridge structures within the hydraulic model
- In order to meet FIOs, additional analysis of cross-drainage infrastructure may be required, along with potential stakeholder consultation.

Further details of mitigation measures can be found in Section 22. A review of proposed mitigation measures will be completed during the detailed design stage of the Project after hydraulic models have been updated to capture the above changes.

16.7 Planned and future roads

Consultation with the DTMR has not identified any planned or future road upgrades within the Macintyre Brook—Cremascos Road catchment. ARTC will continue to consult and engage with DTMR to determine any future or planned road projects that may need to be considered as part of the detailed design hydraulic modelling.

Consultation with GRC has not identified any planned or future road upgrades within the Macintyre Brook—Cremascos Road catchment. ARTC will continue to consult and engage with GRC to determine any future or planned road projects that may need to be considered as part of the detailed design hydraulic modelling.

17 Macintyre River

The hydrologic and hydraulic modelling of the Macintyre River system study area was undertaken as part of the NS2B section of the Inland Rail project. This work was originally documented in *North Star to NSW/QLD Border: Hydrology and Flooding Technical Report (FFJV, 2020)*. Following completion of this Technical Report an independent review of the flood modelling was undertaken on behalf of GRC.

Following the NS2B EIS public exhibition period, the NSW DPE requested a PIR be prepared for the Project. In preparing the PIR, the hydrologic and hydraulic modelling was revised to address the comments raised in the GRC independent review as well as the received NS2B EIS submission comments. The updated assessment is documented in *the North Star to NSW/QLD Border: Preferred Infrastructure Report – Hydrology and Flooding (FFJV, 2021)*. The information presented in this report is based on the updated hydrologic and hydraulic modelling from the PIR assessment.

The location of the Project alignment in relation to Macintyre River is shown in Figure M1-A in Volume 2 – Appendix M.

This report provides a summary of the flood assessment work undertaken and key outcomes that are relevant to the Project. The section considered relevant to the Project is from the NSW/Queensland Border north. Only infrastructure and FSRs north of the NSW/Queensland Border are reported in this catchment section.

17.1 Data collection and review

Available background information including existing hydrologic and hydraulic models, survey, streamflow data, available calibration information and anecdotal flood data was gathered and reviewed.

The Macintyre River catchment is located within the Moree Plains LGA, Gwydir LGA and Goondiwindi LGA. FFJV was provided with data from each of these Councils for review.

Table 17.1 provides a summary of the data and the relevant section in which it is discussed.

Table 17.1 Macintyre River—Data summary

Data	Reference
Existing flood studies	Section 17.1.1
Topography data	Section 17.1.2
Planning information	Section 17.1.3
Dam details	Section 17.1.4
Aerial imagery	Section 17.1.5
Existing infrastructure	Section 17.1.6
Stream gauge data	Section 17.1.7
Rainfall data	Section 17.1.8

17.1.1 Previous studies

The Macintyre River catchment is located within the Moree Plains LGA, Gwydir LGA and Goondiwindi LGA. FFJV was provided with modelling and historical data from these Councils for review. In addition, there are several previous studies of the Macintyre River catchment undertaken in earlier stages of Melbourne to Brisbane Inland Rail assessment, and other documents identified as potentially relevant to NS2B/B2G, these include:

- *North Star to NSW/QLD Border – Hydrologic and Hydraulic Modelling Illabo to Stockinbingal and North Star to Yelarbon* (SMEC, 2016a)

- *Toomelah Flood Risk Assessment* (Water Technology, 2016)
- *Melbourne-Brisbane Inland Rail – 2017 Phase 2 Preparatory Alignment Assessment Report North Star to Yelarbon*, (WSP, 2017)
- *Floodplain Management Plan for the Borders River Valley Floodplain 2020*, NSW DPE, 2020
- *North Star to NSW/QLD Border EIS – Hydrology and Flooding Technical Report* (FFJV, 2020)
- *North Star to NSW/QLD Border: Preferred Infrastructure Report – Hydrology and Flooding* (FFJV, 2021)
- Dam Details – Pindari Dam, Coolmunda Dam, Glenlyon Dam.

Key studies were reviewed in detail and a proposed approach for modelling the Macintyre River catchment was developed for the hydrologic and hydraulic modelling based on the information and data available.

17.1.1.1 North Star to NSW/QLD Border—Hydrologic and Hydraulic Modelling— Illabo to Stockinbingal and North Star to Yelarbon, SMEC, 2016a

This study included development of a RORB (runoff and streamflow routing program) hydrologic model and TUFLOW hydraulic model to assess impacts of the Project alignment from North Star to Yelarbon. The modelling works undertaken for this assessment were subsequently updated in December 2016 and May 2017 to assess changes to the Project alignment.

17.1.1.2 Toomelah Flood Risk Assessment, Water Technology, 2016

A flood assessment was undertaken for the Toomelah Community in NSW. The township is located on the Macintyre River approximately 900 m upstream of the confluence of the Macintyre and Dumaresq Rivers. The TUFLOW hydraulic model was developed covering the township and surrounding areas. No hydrologic modelling was undertaken. The results were used to inform development of a Flood Emergency Management Plan.

Key findings from the assessment are as follows:

- Flood depths were typically less than 1 m throughout the township in the 1976 flood event
- The peak level of the PMF at the existing Toomelah Community Hall was estimated to be 230.52 mAHD
- A large breakout of flow occurs upstream of Toomelah community from the Macintyre River and flows in a south-westerly direction around the town (the anabranch)
- A significant increase in flows upstream of the Toomelah community results in only a proportionally small increase in peak water levels.

17.1.1.3 Melbourne-Brisbane Inland Rail—2017 Phase 2 Preparatory Alignment Assessment Report North Star to Yelarbon, WSP, 2017

The Phase 2 Preparatory Alignment Report documents the assessment of changes to the Project alignment, with updates to the models documented in *North Star to NSW/QLD Border, Hydrologic and Hydraulic Modelling—Illabo to Stockinbingal and North Star to Yelarbon* (SMEC, 2016a).

17.1.1.4 Floodplain Management Plan for the Borders River Valley Floodplain 2020, NSW DPE, 2020

The Borders River Valley Floodplain management plan provides a framework for coordinating and assessing development works on a whole of valley basis.

As part of the BRVFMP, hydrologic and hydraulic models (URBS, RAFTS and TUFLOW) were established for the assessment of development impacts on flood characteristics within the floodplain. The hydrology uses previously established models from the *Border Rivers Floodplain Hydraulic Analysis* (Lawson and Treloar, 1998). The URBS models were originally developed by the BoM for the Weir River and Macintyre Brook. The hydrologic models were not modified for the Floodplain Management Plan, 2018. Details of the Lawson and Treloar, 1998 models are provided in Appendix 6 of the Floodplain Management Plan.

The NSW DPE TUFLOW hydraulic model uses current conditions including existing and approved development in floodplain, with small (1996 flood event) and large (1976 flood event) historical rainfall events to assess flood conditions and development impacts. Under the plan, development in the floodplain requires assessment using the NSW DPE hydraulic model to determine if the development meets nominated criteria in terms of changes to flood characteristics (i.e. changes in peak flood levels, changes in flowpaths, flow rates and velocities).

A TUFLOW GPU hydraulic model was developed by NSW DPE for the BRVFMP. The TUFLOW model covers an area of approximately 1.1 million ha extending from approximately 50 km upstream of Boggabilla to 40 km downstream of Mungindi. The main watercourses within the model are the Macintyre River, Weir River, Boomi River and Barwon River.

The topography in the TUFLOW model is defined using a high-resolution DEM. The DEM was created from a variety of LiDAR datasets including Macintyre 2013 and Gwydir 2013 datasets and supplemented to the north with Queensland LiDAR datasets. LiDAR was available for the majority of the model area. Where data was not available, SRTM1-second (~30 m) resolution elevation data was used.

The TUFLOW model grid size is 30 m. Topography modifiers were incorporated into the model to ensure that topographic features such as roads, rail and levee banks are correctly represented. There are no drainage structures included in the TUFLOW model (culverts/bridges).

The NSW DPE hydraulic model was calibrated to the 1996 historical event and verified with the 1976 historical event (noting that the topographic conditions were difficult to replicate for the 1976 conditions). For the 1976 event topographic features (roads, rail, farm levees, farm channels, etc.) known not to be in place in 1976 were removed from the 1976 calibration hydraulic model.

The NSW DPE is the custodian of the models and provided the models to ARTC for review and use.

17.1.1.5 North Star to NSW/QLD Border EIS—Hydrology and Flooding Technical Report (FFJV, 2020)

For the NS2B EIS assessment, NSW DPE provided the following existing information developed for the *Border Rivers Valley Floodplain Management Plan* (NSW DPE, 2020):

- Existing 1976 and 1996 URBS hydrologic models for the Macintyre River, Dumaresq River and Macintyre Brook catchments
- Existing 1976 and 1996 RAFTS hydrologic models for Ottleys Creek catchment
- Existing 1976 and 1996 TUFLOW hydraulic models for the Border Rivers Valley floodplain.

For the NS2B EIS, hydrologic and hydraulic modelling was undertaken using the provided information with the following amendments:

- Creation of a TUFLOW hydraulic sub-model based on the provided BRVFMP hydraulic model, which covers an area of approximately 1.1 million ha extending from approximately 50 km upstream of Boggabilla to 40 km downstream of Mungindi. The TUFLOW hydraulic sub-model has the same upstream boundary and extends to 18 km downstream of Goondiwindi.

- Inclusion of an additional historical, more recent, calibration event being the January 2011 event.
- Sourcing and inclusion of current LiDAR survey data across the floodplain to obtain details of current ground levels and levee heights/locations to support assessment of project. This information was included in the hydraulic sub-model based on community feedback that the BRVFMP levee layer was not representative of current topographic conditions.
- Inclusion of Goondiwindi town levee heights based on information provided by GRC to replace the unlimited heights used in the BRVFMP model.

The performance of the hydraulic sub-model was validated against a range of information collected for the three historical flood events (1976, 1996 and 2011) including:

- Stream gauge level and flow hydrographs
- Flood markers
- Flood event photography from landholders
- Aerial flood photography
- Landowner and community observations of the flood events.

This base modelling was used to develop the Project's revised reference design and assess the impacts on peak water levels, flood flow distribution, velocities and duration of inundation. The impacts of the refined design were documented in the NS2B EIS Chapter 13: Surface Water and Hydrology and associated EIS Appendix H: Hydrology and Flooding Technical Report.

The impact assessment was carried out primarily against the 1% AEP event with the levees based on the 2019 LiDAR data. In addition, modelling of the 1% AEP event with the BRVFMP levees and the 1976 flows with levees based on the 2019 LiDAR data was undertaken and documented in the EIS.

17.1.1.6 North Star to NSW/QLD Border: Preferred Infrastructure Report—Hydrology and Flooding (FFJV, 2021)

The hydrologic and hydraulic modelling undertaken for the NS2B EIS is based on the existing BRVFMP hydrologic and hydraulic modelling. An Independent Peer Review was undertaken for GRC which identified that there were a number of potential limitations with the modelling including that it was not reflective of current best practice including requirements of the recently updated ARR 2019.

The methodology followed for the updated hydrologic and hydraulic modelling to address PIR scope and the issues raised in the GRC Independent Peer Review included:

- Compliance to ARR 2019 in relation to hydrologic and hydraulic modelling guidelines for major infrastructure
- Extension of the existing URBS hydrologic models to form one linked hydrologic model that includes:
 - The four major upstream catchments to within the hydraulic model boundary—Macintyre River, Macintyre Brook, Dumaresq River and Ottleys Creek
 - The wider floodplain area down to Goondiwindi
 - The southern tributaries catchments—Mobbindry Creek, Back Creek, Strayleaves Creek and Forest Creek
 - The Brigalow Creek catchment.
- Review and update of available rainfall and streamflow data used to represent the three historical calibration events (1976, 1996 and 2011)
- Refinement of major upstream catchments URBS models to use common modelling parameters for each catchment (apart from loss rates and dam details relevant to date of historical flood event)

- Refinement of southern tributaries URBS modelling
- Update of the TUFLOW hydraulic model used for the NS2B EIS to include the following:
 - Revised flows from the updated URBS hydrologic model with no factoring of flows for historical events
 - Refinement of upstream inflow locations into the hydraulic model
 - Review and update of existing drainage structures under the non-operational Camurra-Boggabilla rail line, Bruxner Way and North Star Road using available information.
- Joint calibration of the hydrologic model and hydraulic sub-model including:
 - Validation against the available discoverable recorded stream gauge, recorded flood markers and anecdotal data for the 1976, 1996 and 2011 historical flood events
 - Comparison against the BRVFMP calibration outcomes.
- 1% AEP design event modelling and reconciliation to FFA of key stream gauges in the upper catchments and at Boggabilla
- Modelling of the Existing Case and Developed Case (including the Project's revised reference design) for the 1976 flows scenario for the following cases, being:
 - Verified 2019 levees and validated 1976 flows
 - BRVFMP levees and validated 1976 flows
 - BRVFMP levees and factored flows
 - Verified 2019 levees and 1% AEP flows
 - BRVFMP levees and 1% AEP flows.
- Determination of critical event for crossing locations (i.e. either 1976 flow scenario or 1% AEP flows)
- Assessment of the performance of the revised reference design against the Floodplain Management Objectives.

The PIR hydrologic and hydraulic modelling was adopted for the basis of impact assessment for the Project. Full details of the updates are provided in the *Preferred infrastructure Report* (FFJV, 2021) and detailed in the following hydrologic and hydraulic modelling sections.

17.1.2 Survey

The Macintyre River Study area includes many existing roads, levees, the non-operational rail line across the floodplain and road crossings over waterways. Road and rail embankments, levees and other key features were represented in the supplied NSW DPE model. The model was supplemented with the following additional data.

The NSW DPE model utilises a 10 m x 10 m gridded DEM derived from a variety of LiDAR survey datasets including Macintyre 2013 and Gwydir 2013 datasets. Where LiDAR was not available the dataset was supplemented with the Shuttle Radar Topography Mission 1-second (approximately 30 m) resolution elevation data. The extents of the data sources for the NSW DPE model are shown in Figure M1-B in Volume 2 – Appendix M. The majority of the sub-model area is covered by LiDAR data, and mostly covered by LiDAR collected for the Project as shown in Figure M1-B in Volume 2 – Appendix M.

Two sets of LiDAR data were collected for the Project revised reference design, to supplement the originally supplied NSW DPE data. The first was collected between September 2014 and January 2015. The second was collected in November 2019 to provide details of current floodplain conditions and floodplain features.

Where the Project LiDAR merges with the NSW DPE LiDAR differences in the levels are typically within 100 mm with some isolated areas up to 300 mm (with the Project dataset being lower). These areas of difference are outside of the main flow paths and do not appear to have an impact on peak water levels; therefore, no adjustment to the NSW DPE LiDAR elevations was undertaken.

Ground survey at five sites was completed to validate the 2014/15 ARTC LiDAR data and provide additional information for validation of floodplain waterways bed elevations.

The survey results showed the 2014/15 LiDAR Ground triangular Integrated Network to be consistently higher than the ground survey verification sections by 3 mm to 146 mm which is in line with what would be expected for LiDAR data of this nature as explained below:

- LiDAR survey data often measures the top of any vegetation such as grass, bushes or trees where it cannot directly measure the ground and therefore is quite often higher in level than ground survey
- The ARTC 2015 LiDAR was specified with the following metadata:
 - Vertical = 0.15 m (68 per cent confidence level or 1 sigma).
- The ARTC 2019 LiDAR was specified with the following metadata:
 - Vertical = 0.15 m (95 per cent confidence level or 2 sigma).

With a maximum mean difference of approximately 150 mm, it is considered that the ARTC 2019 LiDAR was appropriate for the purposes of this assessment.

17.1.3 Planning information

The NSW DPE supplied the approved levees layer for inclusion in the model. This includes all approved levees (constructed and not yet constructed). The levees are defined by the following approvals:

- Limited height levees—where the approval includes a maximum height to which the levee can be constructed
- Unlimited height levees—the approval does not specify a maximum height the levee can be constructed up to; therefore, the levee banks are considered higher than the flood levels for all flood events.

17.1.4 Reservoirs

Three notable reservoirs are located upstream of Boggabilla and included in the hydrologic models – Pindari Dam on the Severn River in the Macintyre River catchment upstream of Ashford, Glenlyon Dam on Pike Creek in the upper Dumaresq River catchment, and Coolmunda Dam on Macintyre Brook upstream of Inglewood. All three dams were completed between 1969 to 1972, with Pindari Dam undergoing a major upgrade in 1994 that significantly increased its capacity. Reservoir details were obtained from previous studies and Queensland and NSW operators (Sunwater and Water NSW). Further details are provided in Section 17.2.5.4.

17.1.5 Aerial imagery

Aerial imagery of the Macintyre River study area captured in 2015 and 2019 was provided by ARTC and was used to identify and confirm topographic and vegetative characteristics of the study area. Imagery has also been sourced outside of the study area from QGIS in an open-source format.

17.1.6 Existing drainage structure data

Drainage structure geometry information of the Study area was obtained from the following sources:

- Previous studies—GRC, Inland Rail
- Site inspection
- QR
- DTMR
- Field and validation survey.

17.1.7 Stream gauge data

Stream gauges record the water level at the gauge site. Ratings tables are then used to convert the recorded water levels into an equivalent flow. Ratings are typically based on flow gauging or measurements physically recorded during flood events. These are most commonly available for small to moderate flows as it is often difficult and/or dangerous to obtain physical measurements during major floods. The rating must then be extrapolated to higher flows, which may be done by a variety of means ranging from a basic best-fit curve through the available gauging data to using a calibrated hydraulic model. The reliability of a rating to predict high flows is dependent on the reliability of this projection. Discontinuities in the channel shape with elevation can significantly affect the accuracy of the projection, particularly if simplistic projection methods are employed.

Stream gauges may also be affected by backwater effects, where the water level at the gauge is increased due to conditions downstream of the gauge rather than the stream flow passing the gauge. Gauges located upstream of a major tributary are particularly susceptible to this. A gauge with an unreliable or inaccurate rating may still provide useful information on the timing of flow through the system.

Review of the stream gauge ratings identified that a number of the gauge ratings had changed since the BRVFP models were developed. Gauge ratings may change for a number of reasons, including:

- Change in gauge location or control, in which case historical events should use rating appropriate for gauge location/control at time of flood
- Additional or more reliable data and/or improved extrapolation methods, in which case historical events should be re-rated using the improved rating.

Stream gauges used in the calibration of the hydrologic model and/or hydraulic model against historical flood events are detailed in Table 17.2 and are also shown in the calibration figures within Appendix C.

Table 17.2 Macintyre River—Summary of key stream gauges used in the model calibration

Catchment	Gauge name	Gauge no.	Area ¹	Description
Macintyre Brook	Macintyre Brook @ Barongarook	DRDMW 416410A	114 km ²	Small catchment upstream of Coolmunda Dam. Relatively reliable rating. Site ceased in 2002.
	Bracker Creek @ Terraine	DRDMW 416404C	685 km ²	Small catchment upstream of Coolmunda Dam. Relatively reliable rating. Site ceased in 2002.
	Canning Creek @ Woodspring	DRDMW 415404A	1,245 km ²	Rating appears reasonable but based on limited flow gauging. Site ceased in 1980.
	Macintyre Brook @ Coolmunda T/W	DRDMW 416416A	1,760 km ²	Immediately downstream of Coolmunda Dam. Unknown rating quality. Site ceased in 2002.
	Macintyre Brook @ Inglewood CBM	Unknown	~1800 km ²	Upstream of the confluence of Macintyre Brook and Canning Creek. No information about this gauge is currently available.
	Macintyre Brook @ Inglewood	DRDMW 416402B/C	3,430 km ²	Downstream of the confluence of Macintyre Brook and Canning Creek. Site changed from 'B' to 'C' in 1981, keeping the same location but a modification of the control weir and rating.
	Macintyre Brook @ Ben Dor Weir	DRDMW 416406A	4,000 km ²	Operational between 1954 and 1988. Highest gauged flow is 41.9 m ³ /s. Significant uncertainty in the high flow rating and inconsistent with Inglewood.
	Macintyre Brook @ Booba Sands	DRDMW 416415A	4,092 km ²	Relatively reliable rating but site has only been operational since 1987

Catchment	Gauge name	Gauge no.	Area ¹	Description
Dumaresq River	Pike Creek @ Glenlyon T/W	DRDMW 416309B	1,320 km ²	Immediately downstream of Glenlyon Dam. Reasonable rating quality.
	Dumaresq River @ Farnbro	DRDMW 416310A	1,310 km ²	Small tributary in the upper Dumaresq River catchment. No high flow stream gauging. High flow rating appears reasonable but of unconfirmed quality
	Mole River @ Donaldson	WaterNSW 416032	~1,290 km ²	Small tributary in the upper Dumaresq River catchment. No high flow stream gauging. High flow rating appears reasonable but of unconfirmed quality
	Dumaresq River @ Riverton	DRDMW 416301 (Possibly)	~5,090 km ²	Department of Resources stream gauge operated only from 1920 to 1954. Source of data from BRVFMP calibration is unknown. Rating is not from the site but rather is based on the downstream gauge at Roseneath. Rating is therefore unreliable but useful for order-of-magnitude and timing.
	Dumaresq River @ Roseneath	WaterNSW 416011	~5,560 km ²	Mid- Dumaresq River gauge between the upper catchments and Beardy River. Site has been operational since 1937.
	Beardy River @ Haystack	WaterNSW 416008	~280 km ²	Small tributary in the southern Dumaresq catchment. Site has been operational since 1934.
	Dumaresq River @ Bonshaw	WaterNSW 416007	~7240 km ²	Lower Dumaresq River weir at Bonshaw Weir downstream of the Beardy River confluence.
	Dumaresq River @ Texas	Unknown	~7,810 km ²	Dumaresq River gauge believed to be located at the town of Texas upstream of the Oaky Creek confluence. Location and details of the gauge are unknown.
	Oaky Creek @ Texas	DRDMW 416312A	422 km ²	Small tributary in the lower Dumaresq River catchment. Note: This gauge is referred to as the Oaky Ck gauge in the calibration to avoid confusion with the Dumaresq River gauge at Texas.
	Dumaresq River @ Cunningham	DRDMW 416308A	8,755 km ²	Located in the lower Dumaresq River upstream of Beebo. Appear to be significant issues with the high flow rating. Site ceased in 1988.
	Brush Creek @ Beebo	DRDMW 416308A	335 km ²	Minor creek catchment in the lower Dumaresq River catchment.
	Dumaresq River @ Mauro	Unknown	9,090 km ²	Location and details of the gauge are unknown. Believed to be a short distance downstream of the Brush Creek confluence based on URBS model setup and catchment area.
	Dumaresq River @ Glenarbon	WaterNSW 416040	~10,004 km ²	Located in the lower Dumaresq River downstream of Beebo. Reliable rating for low flows but appear to be significant issues with the high flow rating.
Macintyre River	Severn River @ Fladbury	WaterNSW 416022	~1,110 km ²	Located in the upper Severn River. Rating based only on low flow gaugings and appears to have issues with the high-flow rating.
	Severn River @ Strathbogie	WaterNSW 416039	~1,740 km ²	Located in the upper Severn River between Fladbury and Pindari Dam. Rating based only on relatively low flow gaugings and appears to have issues with the high-flow rating.
	Severn River @ Pindari T/W	Unknown	~2,100 km ²	Site details are unknown. Low flow rating is similar to Ducca Marri gauge.
	Severn River @ Ducca Marri	WaterNSW 416067	~2,100 km ²	Located just downstream of Pindari Dam. Site has only been operational since 2002. Relatively reliable rating.

Catchment	Gauge name	Gauge no.	Area ¹	Description
	Frazers Creek @ Westholme	WaterNSW 416021	~810 km ²	Small tributary in the middle of the Macintyre River model catchment. Joins Severn River upstream of Ashford. High-flow rating potentially unreliable and/or backwater affected.
	Severn River @ Ashford	WaterNSW 416006	~3,285 km ²	Long record gauge on the Severn River downstream of Pindari Dam and Frazers Creek. Rating is well gauged but has changed between the NSW DPE and current calibration.
	Macintyre River @ Inverell	WaterNSW 416016	~745 km ²	Located in the upper Macintyre River catchment. Gauge is theoretically well rated but high-flow rating appears to be potentially unreliable and/or backwater affected.
	Macintyre River @ Tintot	WaterNSW 416068	~1,740 km ²	Located between Inverell and Wallangra. Site commenced in 2002. Rating based on limited flow gauging but to relatively high level.
	Macintyre River @ Wallangra	WaterNSW 416010	~2,130 km ²	Long record gauge upstream of the confluence with the Severn River. Rating is well gauged but has changed between the BRVFMP and current calibration.
	Macintyre River @ Ridgeland	WaterNSW 416031	6,170 km ²	Located downstream of the confluence of Macintyre and Severn Rivers. Site reportedly commenced in 1967, however stream gauge data is only available after 1997.
	Macintyre River @ Holdfast	WaterNSW 416012	~6,890 km ²	Located upstream of the Dumaresq River confluence. Well rated up to bank-full but appear to be issues with the rating above floodplain level. Minor changes to the rating between the BRVFMP and current calibration.
Ottleys Creek	Ottleys Creek @ Coolatai	WaterNSW 416312A	~385 km ²	Located in the upper Ottleys Creek catchment. Site reportedly commenced in 1967, however stream gauge data is only available after 1978.
Lower Macintyre	Macintyre River @ Boggabilla	WaterNSW 416002	~22,500 km ²	Operational since 1894 but the gauge has changed location several times. Significant breakout from the Macintyre River occurs upstream of the gauge. Rating has been reviewed and updated as part of the current study.
	Macintyre River @ Goondiwindi	DRDMW 416201A	23,090 km ²	Rating is based on significant flow gauging data but appears to become ineffectual above bank-full due to significant floodplain and bypass flows.

Table note:

¹ Catchment areas for DoR gauges are based on data published online. Areas indicated with ~ do not have published values and areas are estimated from the URBS hydrologic model layout.

17.1.8 Rainfall data

Rainfall data was sourced from BoM as well as WaterNSW and DoR. Detailed pluviograph data (rainfall distribution with a timestep less than 1 hour) is typically available at a relatively limited number of gauges across the catchment. This data is supplemented by daily (24 hour) rainfall data.

The number of rainfall gauges used for each historical calibration event are summarised in Table 17.3, noting that some of these gauges are located outside the catchment boundaries. The gauges are also not located uniformly across the catchment, and some areas of the catchment have significantly higher detail in terms of both spatial and temporal distribution of the rainfall. Rainfall gauge locations available for the three calibration events are shown in Figure C2 to C4 in Appendix C, together with the total rainfall depth of each event.

The gauge pluviograph data for each event was reviewed for quality to identify suspect or missing data and also for consistency with nearby gauges. A number of cases were identified where several days of rainfall were accumulated in a single record or where a gauge had the similar depths to surrounding gauges but offset by a day, most likely the result of errors in the transposition of manually recorded data.

Table 17.3 Macintyre River catchment—Number of rainfall gauges used in the model calibration

Event	Detailed pluviograph gauges			BoM daily gauges
	BoM	DoR (Qld)	WaterNSW	
February 1976	15	-	-	202
January 1996	18	4	2	165
January 2011	16	6	23	162

17.1.9 Anecdotal flood data

Anecdotal flood data for the historical flood events was collected from many sources including:

- Previous studies
- NSW DPE
- Landowners and stakeholders including GRC, Gwydir Shire Council and Moree Plains Regional Council.

Anecdotal data includes information obtained from a wide range of sources and as such it is of varying levels of accuracy and reliability. The anecdotal data was used to assess of the performance of the hydraulic model to replicate historical flood conditions.

17.1.10 Site inspection

A site inspection was undertaken on 9 to 10 April 2018. During the site inspection, all major waterway crossings were visited and inspected with photographs taken and details recorded of the crossing, existing drainage structures and surrounding catchment. An assessment of the relative roughness and blockage potential was undertaken during the site inspection.

Additional inspections were undertaken associated with stakeholder meetings. Footage of the late 2021 flood event was also collected by helicopter.

17.2 Hydrologic model development—Macintyre River

17.2.1 Model setup

The Border Rivers catchments are presented in Figure C1 in Appendix C. Initial hydrologic modelling was conducted using three separate URBS hydrologic models of the major Macintyre River tributaries upstream of Boggabilla, being:

- Upper Macintyre River catchment ending at Holdfast
- Dumaresq River catchment ending at Mauro
- Macintyre Brook catchment ending at Booba Sands.

The BRVFMP models were calibrated initially to the major February 1976 flood event (which remains the flood of record in the Macintyre River at Boggabilla and many other areas of the catchment) and then to a number of smaller events that occurred in 1996, the most significant of which occurred in late January. During this calibration process, different reach length factors (modifiers used to adjust the storage attenuation in different parts of the model) were used for each of the calibration events. These differences were not reconciled, effectively creating a different set of models for each flood event rather than a single set of models capable of reproducing flows from different events. Although it is acknowledged that catchment characteristics can vary between events, this situation is not ideal, particularly when conducting Design Event simulations that require the model to be used for events smaller or larger than the calibration event. Due to project and time constraints, the initial hydrologic NS2B modelling validated the BRVFMP models using data obtained for the January 2011 flood but did not attempt to recalibrate or reconcile all the models.

A recalibration of the combined hydrologic models has subsequently been undertaken for the purpose of creating a set of consistent models capable of reproducing all three calibration events using a fixed set of parameters, thereby greatly improving confidence in the model calibration as well as the models' ability to produce flows for non-calibration events. The recalibration has included:

- Review and update of previous model data including rainfall and gauge ratings
- Inclusion of additional rainfall and stream gauge data where available
- Increased emphasis on calibration to the January 2011 flood, which provides a significant amount of more recent and reliable data not available in the previous BRVFMP calibration.

In addition, new URBS hydrologic models were developed for Ottleys Creek and for the lower Macintyre River floodplain covering the extent of the TUFLOW hydraulic model. The lower floodplain model provides a single junction taking inflows from the upstream tributary models (Macintyre Brook, Dumaresq River, Macintyre River and Ottleys Creek) and producing tributary and local sub-area inflows for the hydraulic modelling.

Local hydrologic models were developed for Strayleaves Creek, Forest Creek, Back Creek and Mobbindry Creek and their inflows included into the TUFLOW hydraulic model. Figure M1-C in Volume 2 – Appendix M presents an overview of the hydrology model setup for the border river catchments.

17.2.2 Event selection for calibration

Calibration of the hydrologic models involves simulation of recorded historic rainfall events. Three rainfall/flood events were used for the hydrologic model recalibration. Each of the events have a unique rainfall distribution:

- **February 1976.** This is one of the largest and most widespread rainfall events recorded in the Macintyre River catchment and resulted in a significant flood event in much of the catchment (flood of record at Boggabilla and Goondiwindi). Most of the rainfall occurred within a 24-hour period on the 10 February, with almost the entire catchment receiving between 80 and 200 mm on that day. Much of the catchment also received rainfall on the preceding day, wetting the catchment and initiating runoff, and there was also some follow-up rainfall.
- **January 1996.** Several different floods were recorded in the Macintyre River in 1996. The main flood at Boggabilla, subsequently referred to as the 1996 flood event, was actually the combination of several relatively short-duration storms that fell across the catchment over successive days from 21 to 27 January. Rainfall distributions were highly variable, even between relatively closely spaced gauges, with rainfall more strongly concentrated in different parts of the catchment on different days. The rainfall depths shown in Figure C3 in Appendix C are the cumulative depth of all the storm events.

- **January 2011.** The January 2011 actually consisted of two major rainfall events approximately two days apart, and most of the recorded stream gauge hydrographs display two distinct peaks. The concentration of the rainfall varied across the catchment, however in most areas the second burst was the more significant. The rainfall was most heavily concentrated across the eastern side of the catchment.

Importantly, the January 2011 flood event provided a significant opportunity to improve the calibration of the hydrologic modelling. Firstly, there has been a significant increase in the number of rainfall gauges recording continuous pluviograph data, providing improved rainfall detail not present in the earlier events, and there are also a number of additional stream gauges (unfortunately excluding the Macintyre Brook catchment where the number of gauges has decreased since 1996). Secondly, the spatial and temporal distribution of the rainfall has several features that are advantageous for model calibration:

- Much of the rainfall occurred in the upper parts of the catchment, producing a large upstream flow that can then be routed downstream through subsequent gauges, greatly increasing confidence that the models are reliably representing the routing speed through the model.
- Assumed rainfall losses can have a significant influence on the rainfall distribution, and hence the generated hydrograph. The two strong distinct bursts and runoff peaks allow the effects of the initial catchment losses, which predominantly affect only the first peak, to be separated from the effects of the continuing losses on the second peak.

17.2.3 Model calibration

17.2.3.1 Macintyre River to Holdfast

The Macintyre River URBS hydrologic model covers an area of 6,890 km² to Holdfast. The catchment was subdivided into 50 subareas, these are shown as centroids in Figure C5-A in Appendix C. Pindari Dam is located on the Severn River upstream of Ashford. The dam was constructed in 1969 and was raised in 1994, significantly increasing the capacity. The dam was not included in the 1976 calibration model. Due to the magnitude of the flood event and the low capacity of the pre-upgrade dam, this approach is considered reasonable and has not been changed.

The BRVFMP calibration appears to have been conducted primarily using the downstream gauge at Holdfast and the major gauges at Wallangra on the Macintyre River and Ashford on the Severn River, with some supplementary information from Pindari Dam. A significant factor in the model recalibration is that the high-flow ratings for Holdfast, Wallangra and Ashford have all changed since the BRVFMP calibration was conducted. The recalibration has also examined a number of additional gauges, including Fladbury, Strathbogie and Westholme in the Severn River catchment and Inverell and Tintot in the Macintyre River catchment.

February 1976 flood

Calibration hydrographs for the Macintyre River stream gauges in the February 1976 flood are shown in Figure C5-A in Appendix C. The effect of the changes to the Wallangra and Ashford ratings are most pronounced on the February 1976 calibration, as the Holdfast gauge failed prior to the peak and these gauges represent the primary focus points of the calibration. The recalibrated model produces similar quality match of the recorded hydrographs at these locations but with a reduction in the magnitude of the flows. Downstream at Holdfast, the recalibrated model provides a better match of the rising limb of the hydrograph until the gauge failed. The calculated and rated flows begin to deviate once the water level reaches the level of the floodplain, suggesting an issue with the high-flow rating (see further discussion on the 2011 flood event below). The modelled peak flow of 2,860 m³/s compares well with the reported peak flow of 250,000 megalitres per day (ML/d) (2,893 m³/s) at Yetman No. 3 gauge upstream of Holdfast (NSW Department of Land and Water Conservation, 1996). Data for this site is not available from WaterNSW webservice.

The BRVFMP calibration matched the magnitude and timing of the peak flow at the Pindari TW gauge downstream of Pindari Dam but did not match the overall shape of the hydrograph particularly well. Details of the Pindari TW gauge and the origin of its rating are not known. The WaterNSW gauge on the Severn River at Ducca Marri is located just downstream Pindari Gauge, but only commenced operation in 2002 (i.e. the gauge was not present in the 1976 flood). The Ducca Marri low flow rating is similar to the Pindari TW rating from the BRVFMP model but deviates at high flows. The recalibrated model reasonably matches flows calculated using the Ducca Marri rating.

Westholme, on Frazers Creek upstream of Ashford, was not considered by the BRVFMP calibration. The recalibrated model provides a good fit of the rising and falling limbs hydrograph but underestimates the peak flow. This could potentially be attributed to a poor high flow rating, noting that the deviation between the rated and calculated flows occurs at around the level where the main channel breaks out onto the floodplain, which is significantly higher than the largest gauged flow measurement at the site. Alternatively, the gauge could potentially be affected by backwater given its proximity to the Severn River confluence at Ashford. The BRVFMP model does not match the gauged hydrograph well.

Similarly, issues with the high flow rating and/or backwater effects potentially affect rated flows at Inverell in the upper Macintyre River catchment. The BRVFMP calibration matches the peak flow relatively well (noting that this is partly coincidental as the Inverell gauge was not considered) but significantly overpredicts the flow on the rising and falling limbs of the hydrograph. The recalibration process found that it could match the full shape of the hydrograph using realistic routing and loss parameters. The recalibration has chosen to match the timing, and the rising and falling limbs of the hydrograph, as these are the most reliable aspects of the rated hydrograph and higher peak flows are inconsistent with the (revised rating) flows downstream at Wallangra.

January 1996 flood

The January 1996 flood consisted of several short-duration storm events across several days. Several of the gauge hydrographs show multiple peaks. Calibration hydrographs for the Macintyre River gauges in the January 1996 flood are shown in Figure C5-B in Appendix C. The BRVFMP calibration was based on relatively limited stream gauge information.

Flows from the upper Severn River were calibrated against the inflow into Pindari Dam. It is understood that this inflow was reverse-routed from changes to the reservoir level. The recalibration has included information from stream gauges at Fladbury and Strathbogie in the upper catchment. The BRVFMP calibration did not consider these gauges and, although it provides a reasonable match of the rated peak, it does not match the shape and timing of the hydrograph particularly well. The recalibrated model matches the shape and timing of the hydrographs well but overestimates the rated peaks, which may be an issue with the high-flow ratings. Neither calibration matches the calculated Pindari inflow particularly well – the recalibrated model appears to have too high a peak while the BRVFMP model has too much volume at the tail. The Pindari inflows do not appear particularly consistent with the upstream gauge at Strathbogie, and the accuracy of these calculated inflows is unknown.

Levels in Pindari Dam were low prior to the January 1996 flood and the dam did not release during the flood. Flows from the upper Severn River catchment therefore have no influence on the lower catchment. The flow recorded at Ashford is almost entirely generated in Frazers Creek (the Westholme gauge does not have a record for this event). The BRVFMP calibration at Ashford is a reasonable match of the gauged hydrograph shape although the timing appears out by approximately 3 hours. The recalibration produces lower flows due to the newer Ashford rating and provides a good fit of the timing and shape of the hydrograph.

Based on the model data provided, the BRVFMP calibration did not have and/or consider a rated flow at the Wallangra gauge. Consequently, the calibration of flows from the upper Macintyre River catchment was based solely on the combined Macintyre and Severn flow at Holdfast. The recalibration has included the Wallangra gauge record as well as the upstream gauge at Inverell. The BRVFMP calibration significantly overpredicts the amount of flow at Inverell and, although it provides a reasonable match of the flow magnitude at Wallangra (based on the old rating), the peak and trough pattern of the hydrograph is notably nowhere near as pronounced.

The BRVFMP calibration achieves a relatively good match of the rated hydrograph at Holdfast (based on the old rating). The revised calibration achieves a good match of the first peak and the receding limb of the hydrograph (using the current rating) but does not replicate the second ‘hump’ of the hydrograph occurring on 24 January. This second peak could not be matched without worsening the calibration of the upstream gauges at Ashford and Wallangra (the latter of which was not considered in the BRVFMP calibration).

Considering the good match of the upstream ratings, a likely explanation is that the ‘missing’ flow was caused by localised rainfall poorly represented by the surrounding gauges. As shown in Figure 17.1, there is very limited rainfall gauge information in the region between the major tributary gauges at Ashford and Wallangra and the downstream gauge at Holdfast. Rainfall gauge 54161, located just north of Ashford, recorded zero rainfall between 9:00 am on 23 January to 9:00 am on 24 January, which would correspond to the second peak. The gauge at Wallangra recorded 56 mm in this same period, while the gauge further west at Coolatai recorded over 80 mm. Due to triangulation of the gauges, the zero rainfall at Gauge 54161 affects much of the eastern side of the catchment between Ashford and Holdfast, potentially representing a significant underestimation of the rainfall. This conclusion is corroborated by the issue also affecting the Dumaresq River calibration, which uses the same rainfall data but otherwise was calibrated independently.

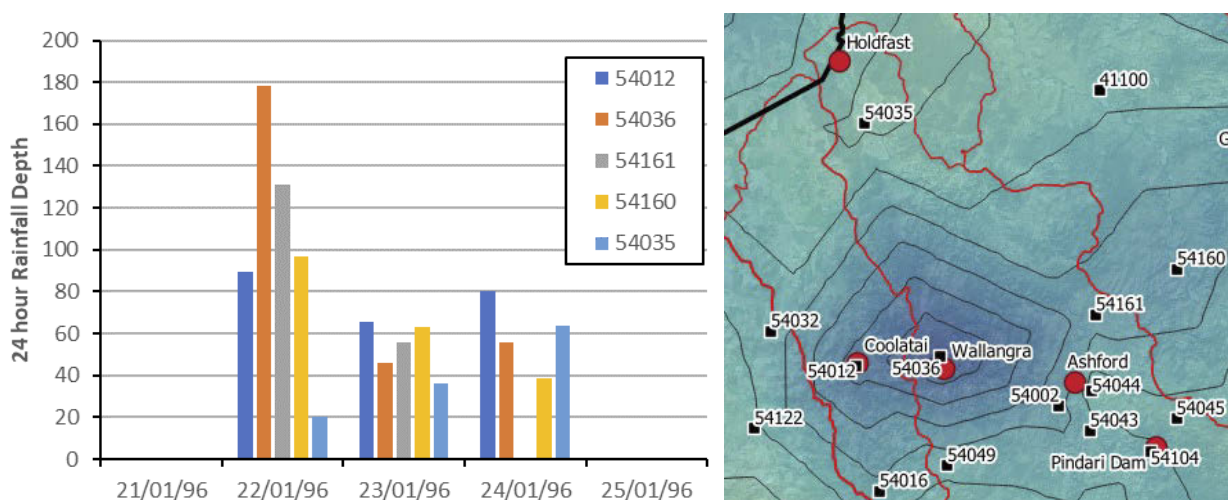


Figure 17.1 Rainfall gauge between Ashford/Wallangra and Holdfast for 1996 event

January 2011 flood

Like the January 1996 flood, the 2011 flood was generated by the combination of several short duration bursts. Significantly more continuous pluviographs were available for this event. A large number of these gauges are located in the upper Macintyre River catchment upstream of Inverell. Examination of the rainfall records shows that there was significant spatial variability (i.e. the rainfall was in the form of concentrated localised bursts), making it difficult to match the exact rainfall patterns, particularly across the upper Severn River catchment where there are no continuous rainfall records. Nevertheless, the January 2011 flood is an excellent event for calibrating the stream routing of the model. Much of the runoff was generated as short bursts in the upper catchment. Additional stream gauge locations not available for previous events, including Tintot and Ridgeland, allow these bursts to be tracked downstream, providing a concise record of stream travel times.

Overall, the recalibrated model provides a very good match of the gauged hydrographs throughout the catchment, as shown in Figure C5-C in Appendix C. The model has difficulty in matching some of the burst peaks generated in the upper catchments (e.g. upstream of Inverell). As discussed above, the flow was generated by short, localised rainfall bursts and the spatial detail of the available rainfall data, the model catchment resolution and timestep will all tend to smooth/average out the peaks. There are also some apparent issues with the gauged flows, some of which can be attributed to uncertainty in the high-flow gauge ratings (notably the smaller catchment gauges of Fladbury, Strathbogie and Westholme), and some of which cannot be readily reconciled (e.g. the peak Pindari Headwater gauge is higher than the downstream gauge at Ashford). Parameters were selected to provide good match of the rising and falling limbs of the flood (the most reliable record) and a balanced overall fit of the available gauge data.

Summary

The BRVFMP calibration achieved a reasonable match of Ashford and Wallangra gauges for the 1976 flood event (noting that the Holdfast gauge failed during the event) and of the Ashford and Holdfast gauges for the 1996 flood event (Wallangra was not considered and not particularly well matched) but did so using different model routing parameters. Significantly for this catchment, the ratings for these three primary calibration gauges were updated since the BRVFMP calibration was conducted.

Calibration of the Macintyre River model has been affected by changes to gauge ratings as well as the inclusion of additional upstream stream gauge information to improve the accuracy of the model representation of the upper catchments. The revised calibration is considered of equivalent or better accuracy than the previous modelling at each of the stream gauges throughout the catchment. This calibration was achieved using a single set of model routing parameters for all three events, confirming that the adopted routing parameters provide a good representation of the routing characteristics of the catchment.

17.2.3.2 Dumaresq River to Maura/Glenarbon Weir

The original BRVFMP Dumaresq River model covered an area of 9,090 km² down to Mauro, the largest of the tributaries contributing to the Macintyre River upstream of Boggabilla. The catchment was subdivided into 79 sub-areas and these are shown as centroids in Figure C6-A in Appendix C.

Glenlyon Dam is located on Pike Creek, one of the tributaries in the Upper Dumaresq catchment, with a contributing catchment area of 1,290 km². Stream gauges are located on many of the major tributaries; however, there is relatively limited reliable gauge information in the lower reaches of the river. Mauro is not a registered DoR or WaterNSW gauge and the exact location of the gauging station is unknown (the model setup implies that it is just downstream of the confluence with Brush Creek, upstream of Glenarbon Weir). The Dumaresq URBS model was extended down to the DoR gauge at Glenarbon Weir, which provides an additional calibration point for the 2011 flood.

February 1976 flood

Calibration hydrographs for the Dumaresq River gauges in the February 1976 flood are shown in Figure C6-A in Appendix C. No gauge information was available at Mauro, and it appears that the primary focus of the BRVFMP calibration was the gauges further upstream at Texas and Bonshaw Weir, noting that the former gauge is not an official WaterNSW gauge and details of the site and rating have not been provided. The BRVFMP calibration achieved a reasonable match of the rated peak flows at these gauges, although the peak flow at Bonshaw and the receding limb of the hydrograph at Texas are not particularly well represented.

The calibration becomes increasingly inaccurate further upstream, noting that there appears to be a significant issue with the Riverton rating used in the BRVFMP calibration, and the constant flow at Glenlyon TW gauge is not considered realistic indicating an issue with the gauge (Glenlyon is not a regulated outflow). The BRVFMP calibration at Farnbro matches the timing of the peak but otherwise is a poor fit, suggesting that the calibration at this gauge was either not considered important or had to be sacrificed in order to improve the calibration downstream. Good matches of the downstream minor tributaries at Beebo and Oak Creek were achieved.

The recalibration has improved the match of all the upstream gauges and achieved a generally good fit at Bonshaw and Texas gauge hydrographs. The recalibration has also sourced and included additional data at the now-closed Cunningham Weir gauge, upstream of Mauro. Although the high-flow rating (>1,000 m³/s) at this site appears highly suspect, the start and end of the hydrograph are well matched, giving increased confidence in the channel routing that occurs downstream of Texas.

January 1996 flood

As previously discussed, the January 1996 flood was generated by a series of short-duration storm events scattered across the catchment over several days. Although there are a large number of stream gauges in the catchment, rainfall gauge data in the Dumaresq catchment is relatively sparse (Figure C3 in Appendix C) limiting the accuracy of the rainfall distribution. Calibration hydrographs for the Dumaresq River gauges in the January 1996 flood are shown in Figure C6-B in Appendix C.

The original BRVFMP calibration appears to have focused almost exclusively on matching the stream gauge at Mauro, where a good match is achieved. At Texas, the model matches the general magnitude of the flood event but not the rise and fall of two peaks evident in the gauge record before it appears to have failed. Neither Mauro nor Texas are government-controlled gauges and details of the gauge ratings have not been provided. The BRVFMP model does not appear to have included or examined the WaterNSW gauge at either Bonshaw Weir or Roseneath. Further upstream, the BRVFMP calibration is generally poor. At the Haystack, Donaldson and Farnbro gauges the rated flows are all significantly over- or underestimated, and at Riverton (acknowledging that there appear to be significant issues with the rating) the timing of the calculated hydrograph appears to be out by nearly 24 hours.

The recalibration has significantly improved the match at virtually all the upstream gauges. A good match is achieved at the WaterNSW Dumaresq River gauges at Roseneath (the gauge does not appear to have properly recorded the first peak, the general magnitude of which can be confirmed from the upstream gauges) and at Bonshaw.

The recalibration appears to produce a worse match at the downstream gauge of Mauro; however, the calibration at Mauro could not be improved without correspondingly worsening the match at Bonshaw and Texas, as well as the other calibration events. Differences between the rated and calculated flows can be attributed to:

- The overestimate of the flow at the peak on late 23 January/early 24 January may be due to inaccuracy of the high-flow rating (details of the Mauro rating are unknown), and both Cunningham Weir upstream and Glenarvon Weir downstream exhibit this trait, possibly due to significant floodplain flow, and/or poor rainfall data.
- The subsequent underestimate of flow on late 24 January/early 25 January corresponds to the same 'missing' flow observed between Ashford and Holdfast in the Macintyre River catchment, which is immediately adjacent to the local Dumaresq catchment between Bonshaw and Mauro, and may be related to the same rainfall issue discussed in Section 17.2.3.1.
- The overestimate of flow on the receding hydrograph on 26 January is most likely again related to poor distribution of rainfall, this time having too much rainfall. This is evident in the calculated hydrographs at both the Oaky Creek and Beebo gauges.

January 2011 flood

The January 2011 flood was caused by significant rainfall concentrated most heavily across the eastern side of the catchment, although the heaviest rainfall actually fell outside the catchment as shown in Figure C4 in Appendix C. The event consisted of several separate storm bursts, with moderate rainfall on 5 January and 6 January and then heavier rainfall from 9 January to 11 January, resulting in two distinct peaks with a small initial peak followed by the main flood.

The calibration hydrographs for the Dumaresq River gauges in the January 2011 flood in Figure C6-C in Appendix C show a relatively good match at all the assessed stream gauges despite the relatively sparse rainfall data available in the Dumaresq catchment. The downstream extent of the model was extended to the WaterNSW gauge located at Glenarvon Weir, which provides an additional reference point. Although, as with Cunningham Weir, there appears to be a significant issue with the high-flow rating at the gauge, the model achieves a good match of the lower flows, including the smaller flood peak that occurred prior to the larger flood event, and timing of the major peak is relatively well matched.

Summary

The Dumaresq River catchment is unusual from a calibration perspective in that there is a large amount of stream gauge data (up to 10 gauges were used in the calibration process) but comparatively limited detail in the rainfall gauge data, particularly for the earlier calibration events.

The BRVFMP calibration achieved a reasonable match of mainstream gauges at Bonshaw and Texas for the 1976 event but had no calibration data further downstream. The 1996 event calibration parameters produced a good match of the gauged hydrograph at the downstream gauge of Mauro but had a poor match at many of the contributing tributary gauges, acknowledging the limitations on the available rainfall data but nevertheless suggesting issues with the routing characteristics of the major tributaries and raising concerns that the model could be used to represent other events with any confidence.

The recalibration has derived a single set of model routing parameters that produce a good and consistent match of the rated hydrographs at almost all the gauges for all three calibration flood events. Calibration of the mainstream routing downstream of Bonshaw and Texas is problematic due to the lack of reliable stream gauge data for the events. The Cunningham and Glenarbon Weir gauges, although not having reliable high-flow ratings, provide increased confidence in the model routing. The recalibrated model match of the 1996 hydrograph at Mauro does not appear as good as the original BRVFMP model; however, there is a plausible explanation for the differences. Considering the improvement of the calibration at the gauges throughout the rest of the catchment, the recalibration of the model is considered significantly more robust.

17.2.3.3 Macintyre Brook to Booba Sands

The Macintyre Brook URBS model covers an area of 3,983 km² to Booba Sands, subdivided into 43 sub-areas, and shown as centroids in Figure C7-A in Appendix C. Coolmunda Dam is located upstream of Inglewood. The catchment has limited rainfall gauge information, with few continuous gauges (none in 1976) and no gauges of any kind across much of the northern catchment upstream of Barongarook. Several of the stream gauges historically located in the catchment appear to have ceased operation in the early 2000s.

February 1976 flood

Calibration of the 1976 flood event is challenging due to the limited amount of reliable rainfall and stream gauge data. Calibration hydrographs for the Macintyre Brook gauges in the February 1976 flood are shown in Figure C7-A in Appendix C. The BRVFMP calibration matches the general shape of the hydrographs at the DoR stream gauges at Terraine and Barongarook upstream of Coolmunda Dam, although the peak flow at the latter is significantly underestimated. The source of the recorded inflows into Coolmunda Dam is unknown, and although the start of the hydrograph can be matched, the receding limb after the flood peak, do not appear consistent with the upstream gauges or the rainfall. The Coolmunda Headwater gauge appears to mirror the inflow. The BRVFMP calibration matches the timing of the peak inflow but not the magnitude, and also underestimates the peak flow at the Coolmunda TW gauge (only peak flow is available).

At Woodspring on Canning Creek, the major tributary downstream of Coolmunda Dam, the BRVFMP calibration matches the rated peak flow but does not reproduce the shape of the recorded hydrograph particularly well. The shape of the hydrograph is also poorly matched at the Inglewood CBM and Inglewood gauges. The rated peak flow is matched at this latter gauge; however, there is significant uncertainty regarding the accuracy of this value (see discussion below). There are no further calibration points downstream for the 1976 event.

The recalibration of the Macintyre Brook model has produced a good match of the hydrographs at the tributary gauges of Terrain, Barangarook and Woodspring, as well as a good representation of the flow through Coolmunda Dam based on matching the Coolmunda inflow and headwater peak hydrograph, the TW rated peak and the hydrograph at Inglewood CBM.

The DoR stream gauge at Inglewood, located downstream of the confluence of Macintyre Brook and Canning Creek, would ideally provide a key calibration point for the catchment as the downstream gauge at Booba Sands was not established until 1987. Critically, the Inglewood gauge changed from the now-closed 416402B to the current 416402C in 1981. Based on information from the DoR Water Monitoring Information Portal, the gauge location appears to have remained relatively unchanged but there has been a change in the control structure. This has resulted in (or at least contributed to) a significant change to the flow rating at the gauge. Figure 17.2 shows the ratings for the 'B' and current 'C' gauges together with the recorded flow gauging data used to derive the ratings.

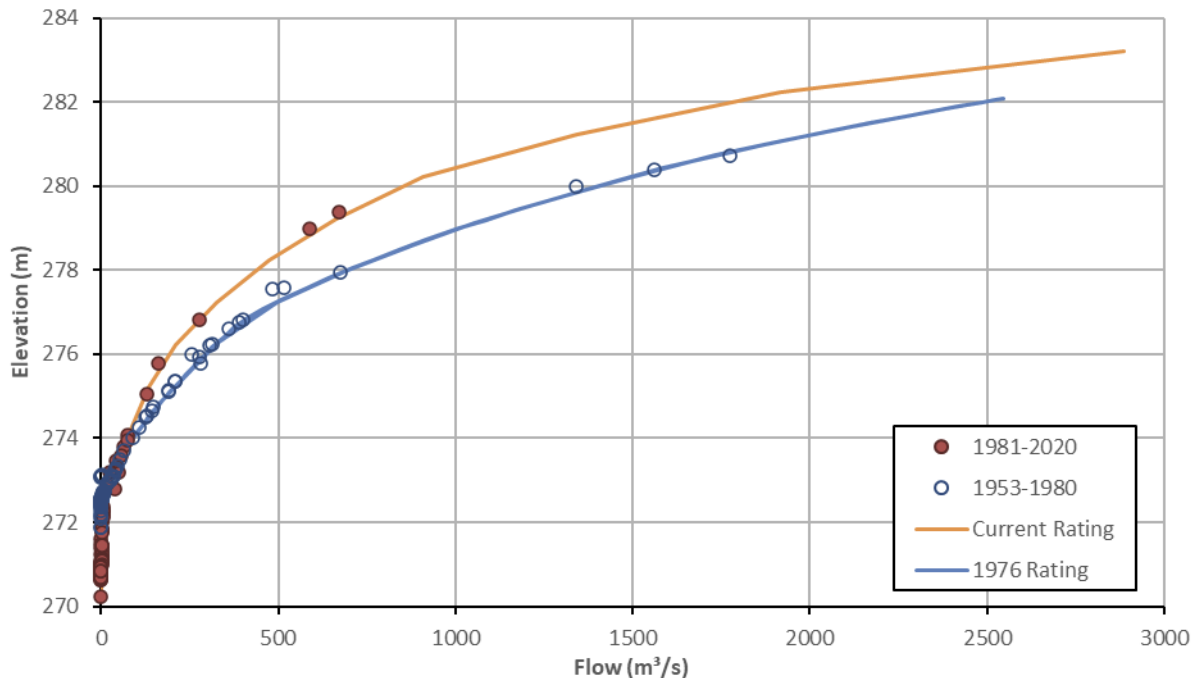


Figure 17.2 1976 flood hydrograph at Inglewood calculated using 416402B and current (416402C) ratings

The original BRVFMP calibration appears to have taken levels recorded at the 'B' gauge present at the time of the 1976 flood but then, after adjusting for the change in gauge datum, applied the rating for the current 'C' gauge to calculate flow. This method does not seem appropriate, particularly for low flows, due to the changes introduced by the modified control weir. Use of the more recent rating for high flows is debatable as it is unknown how much (if any) the changes to the control weir structure would affect the high-flow rating. The 'B' gauge is theoretically well rated being based on high recorded flows but these gaugings were all obtained in 1956. The methods used to obtain the more recent flow measurements and derive the flow rating for the 'C' gauge would likely be more reliable.

The revised calibration produces a good match of the general shape and timing of the hydrograph, with the only real uncertainty being the exact magnitude of the flow. The recalibration process found that it was impossible to achieve consistency between the upstream gauges and the peak flow estimate using either rating (i.e. matching the higher 'B' rating requires overestimating the upstream flows, while matching the lower 'C' rating requires underestimating the upstream flows). The adopted hydrograph matches the low flows using the 'B' rating, consistent with the control structure that was present during the 1976 flood. The peak of the hydrograph, which lies between the 'B' and 'C' rated values is considered a reasonable compromise given the uncertainty in the rating and is consistent with the calibration at the upstream gauges.

A review of two additional stream gauge locations (416413A – Macintyre Brook at 1.6 km and 416406A – Macintyre Brook at Ben Dor Weir) was completed to further inform the calibration process. The findings of this review identified that gauge location 416413A did not record in 1976 so no data was available for calibration. At stream gauge 416406A, the timing of the hydrograph corresponds reasonably well with the model flow and better than the previous DPIE calibrations, but recorded flows are nearly double that of the Inglewood gauge. Due to this inaccuracy, the Inglewood gauge was selected based on confidence and consistency with other gauges. It should be noted that the highest recorded flow at Ben Dor Weir is 41.9 m³/s (the peak 1976 flow at gauge 416406A was estimated to be approximately 4,000 m³/s based on the gauge rating curve). Based on the inaccuracy of the high flows of Ben Dor Weir, this gauge has not been used in the calibration of the hydrologic model.

January 1996 flood

The January 1996 flood was caused by a number of short, concentrated storms over a period of several days. Due to the limited number of rainfall gauges in the catchment the accuracy of the rainfall distribution across the catchment is likely to be limited. A number of the stream gauges also provide only limited information (the gauges recorded only a few levels and/or failed part-way through the flood). Calibration hydrographs for the Macintyre Brook gauges in the January 1996 flood are shown in Figure C7-B in Appendix C.

The BRVFMP model calibration appears to match the general timing of the hydrographs at the gauges in the Macintyre Brook catchment upstream of Inglewood relatively well, although the detail and distribution of the individual flood bursts is not particularly good. This can likely be attributed to limitations in the rainfall distribution in the BRVFMP model. The main downstream gauges of Inglewood and Booba Sands generally reflect the shape and peak of the recorded hydrographs, although the later peak occurring on the 26 January is significantly overestimated. It is unknown whether this is due to poor rainfall distribution for this burst or whether the calibration of the model and loss parameters focused too heavily on the major peak.

The BRVFMP calibration of the major tributary of Canning Creek at Woodspring appears to be poor. Acknowledging that the Woodspring gauge does not provide the full hydrograph, a very large reach length factor has been deliberately applied to this reach, suggesting that the calibration at this gauge has been sacrificed to improve the downstream calibration at Inglewood and Booba Sands.

The revised calibration conducted a review of available rainfall data that has potentially sourced a few additional rainfall pluviographs but has also employed more advanced techniques for distributing the rainfall across the catchment. This appears to have allowed a better calibration of the tributary stream gauges, although there are still evident discrepancies (for example the third distinct peak occurring on 24 January is present in the calculated flow at Barangaroo, but completely missing at Terraine). Overall, a good calibration is achieved at each of the stream gauges considering the limited detail of the input data.

January 2011 flood

The January 2011 flood was produced by two distinct rainfall events resulting in two distinct flood peaks. In the Macintyre Brook catchment these were of roughly equal magnitude (whereas in the Macintyre and Dumaresq River catchments the second event was typically significantly larger). The rainfall was more heavily concentrated over the eastern catchment upstream of Coolmunda Dam. Many of the stream gauges in the Macintyre Brook catchment were removed from service between the 1996 and 2011 floods, meaning that there is less calibration information available for this event. Several of these stream gauges also provided continuous rainfall data for the 1996 event that was therefore not available for the 2011 calibration.

The calibration hydrographs for the Macintyre Brook gauges in the January 2011 flood in Figure C7-C in Appendix C demonstrate a reasonable match of the rated flows at the major gauges at Inglewood and Booba Sands. The Coolmunda Headwater gauge rated flow is also relatively well matched for the first peak. The absence of the second peak from the rated hydrograph is surprising but not critical. Coolmunda Dam is a regulated dam with the gated outflow theoretically controlled as a function of the reservoir level. The first major peak and second minor peak appear to have been operated in a manner consistent with the published headwater rating (noting that maximum outflow of nearly 600 m³/s corresponds to a reservoir level increase of only around 160 mm above the zero flow level); however, review of the headwater level record indicates that the reservoir was drawn down and maintained at a level approximately 100 mm below full supply level throughout the second burst. Therefore, although there was almost certainly release from the dam during this period, with the outflow approximately equal to the inflow, this is below the zero-flow level of the headwater gauge rating.

Summary

Calibration of the Macintyre Brook catchment is based on the least reliable data of the three major tributaries, with relatively little detailed rainfall data and issues identified at a number of the stream gauges. The original BRVFMP calibration of the February 1976 flood event achieved a good match of the rated peak flow at Inglewood (although the actual value of this rated peak is questionable) but did not match the overall shape of the hydrograph particularly well. It is noted that while the rating at Ben Dor Weir is considered highly inaccurate for high flows the shape of the hydrograph and timing of the peak for the 1976 flood event matches well with the recorded hydrograph. Based on the inaccuracy of the high flows of Ben Dor Weir, this gauge has not been used in the calibration of the hydrologic model.

A reasonable match of the hydrographs at the downstream gauges at Inglewood and Booba Sands was achieved for the 1996 event; however, the match at some of the upstream gauges was not as good.

The recalibration has achieved a good representation of the shape of the hydrograph at the primary calibration gauge at Inglewood for the February 1976 flood (although there is some ambiguity over the magnitude due to uncertainty in the gauge rating for this event) as well as a good match of the upstream stream gauges. Good matches were also obtained for the almost all of the available stream gauges for both the 1996 and 2011 flood events. The model is therefore considered to be well calibrated throughout the catchment.

17.2.3.4 Ottleys Creek

The original Ottleys Creek hydrologic model provided by NSW DPE was developed in an old version of the XP-RAFTS software package rather than the URBS software used for the major tributaries. Calibration details for this model are unknown. For the current investigation this was replaced by a new URBS model. The model was developed from available 5 m LiDAR survey supplemented by 1 second (approximately 30 m) SRTM using in-house GIS manipulation software. The Ottleys Creek URBS model layout is shown in Figure C8 in Appendix C. The model covers an area of 1,210 km² to the transfer boundary where the model interfaces with the lower Macintyre River floodplain model. The stream gauge at Coolatai, with a catchment area of 385 km² is the sole calibration point within the catchment.

February 1976 flood

The Coolatai stream gauge was not operational during the February 1976 flood, hence there is no valid calibration data for the Ottleys Creek catchment for this event. The current modelling has adopted model routing parameters derived from the other calibration events combined with rainfall losses consistent with values derived for the surrounding catchments (Macintyre River and lower Macintyre Floodplain). Calculated hydrographs for Ottleys Creek in the February 1976 flood are shown in Figure C10-A in Appendix C.

The outflow hydrograph produced by the BRVFMP RAFTS model is shown for comparison. The peak outflow predicted by the BRVFMP model is significantly lower than the current URBS model and only slightly higher than the 1996 event. Although there is no data to confirm one way or the other, this is somewhat surprising because, as shown in Figures C2 and C3 in Appendix C, the rainfall depth across the Ottleys Creek catchment was significantly greater in 1976 and concentrated into a single event rather than a series of bursts.

January 1996 flood

The January 1996 flood in the Ottleys Creek catchment comprised two short storms on sequential days. The rainfall was most strongly concentrated over the central part of the catchment around Coolatai and Wallangra. Calibration hydrographs for Ottleys Creek in the January 1996 flood in Figure C10-B in Appendix C show that a good match is achieved at the Coolatai stream gauge. There is no calibration data for the lower catchment, so the same routing and loss parameters were adopted. The outflow hydrograph produced by the BRVFMP RAFTS model is shown for comparison, with the hydrograph having a delayed and reduced peak. Calibration information for the BRVFMP model at Coolatai has not been provided.

January 2011 flood

The rainfall that generated the January 2011 flood was concentrated primarily over the eastern side of the Macintyre River catchment. Comparatively little rain fell within the Ottleys Creek catchment, and only a minor flood peak was observed at Coolatai. The calibration hydrographs for Ottleys Creek in the January 2011 flood in Figure C10-C in Appendix C match the Coolatai stream gauge record.

Summary

A new URBS hydrologic model of the Ottleys Creek catchment was developed to replace the old XP-RAFTS model used in the BRVFMP. The model appears to be well calibrated against the stream gauge at Coolatai, although there is no gauge data to allow calibration of the lower catchment. The new model appears to produce shorter, higher peaks at the downstream boundary than the previous BRVFMP model. Details of the BRVFMP model calibration have not been provided for comment.

17.2.3.5 Lower Macintyre floodplain

The original BRVFMP hydraulic modelling used inputs from the four major tributaries (Macintyre River, Dumaresq River, Macintyre Brook and Ottleys Creek) but did not include hydrologic (rainfall) inputs from the lower Macintyre floodplain. A new URBS hydrologic model has therefore been developed for the current investigation covering the area downstream of the major tributary catchments. Outputs from the major tributary models are used as upstream inflows for the lower Macintyre floodplain model.

The model was developed using in-house GIS manipulation software from available 2 m and 5 m LiDAR survey sets covering the majority of the catchment area, supplemented by 1 second (approximately 30 m) SRTM data in areas where LiDAR was not available. The Lower Macintyre floodplain URBS model layout is shown in Figure C9 in Appendix C. The model can be divided into several separate regions:

- The main river channel system including the Macintyre River channel from Holdfast to downstream of Goondiwindi and the lower reaches of the Dumaresq River downstream of Glenarbon Weir and Macintyre Brook downstream of Booba Sands, as well as additional local tributaries flowing into the Macintyre River (subareas 1 to 24).
- The southern creek catchments that share the southern Macintyre River floodplain, consisting of Whalan Creek and its tributaries, including Mobbindry Creek, Back Creek and Forrest Creek (subareas 25 to 95). A more detailed discretisation of the catchment was used to allow generation of rainfall data for the local creek flood event flows at the Project alignment.
- The southern floodplain breakouts, including Morella and Boobera watercourses (subareas 96 to 114)
- The Brigalow Creek catchment north of Boggabilla (subareas 123 to 129).

The primary purpose of the Lower Macintyre Floodplain model is to provide inflows for the TUFLOW hydraulic model. This includes:

- Translation of the Macintyre Brook and Dumaresq River inflows between the current downstream extent of the URBS models and the TUFLOW model inflow location (these URBS models currently terminate upstream of the TUFLOW model boundary)
- Generation of inflow hydrographs for Brigalow Creek and a number of minor external catchments around the periphery of the TUFLOW model boundary
- Generation of local catchment runoff for the areas within the TUFLOW model boundary.

Although the lower Macintyre Floodplain URBS model is largely overlapped by the TUFLOW hydraulic model, it was set up to function as a standalone model capable of performing (hydrologic) flood routing of the river system down to Goondiwindi. Modelling of the lower Macintyre River is complicated by the floodplain flow characteristics. In larger flood events, the main Macintyre River system overflows into the adjacent catchments. Hydrologic modelling is generally based on a simple upstream to downstream routing procedure. URBS can, however, represent breakout flows in the form of a loss as a linear (or step-linear) function of the channel flow, which can then be input back into the system at another location. Using this technique to represent the floodplain breakout patterns requires complex continuous flow characteristics to be approximated by discrete linear relationships. The relationships between breakout flow and main channel flow were determined from the calibrated TUFLOW hydraulic model. Several breakout locations were identified and included in the hydrologic model:

- Across the southern floodplain into Whalan Creek upstream of Boggabilla
- Across the southern floodplain into Morella Watercourse upstream of Boggabilla
- Northwards into the Brigalow Creek system downstream of Boggabilla.

The locations of these breakouts are shown in Figure C9 in Appendix C. There is likely to be additional breakout into the Morella Watercourse between Boggabilla and Goondiwindi. This breakout flow has not currently been included in the model, and all floodplain and/or breakout flow is included in the reported Goondiwindi flow.

17.2.3.6 Lower Macintyre River stream gauges

Stream gauges are located at Boggabilla and Goondiwindi. The calibration results at the Boggabilla gauge presented below were assessed using updated ratings developed as part of the study. Results of the calibrated TUFLOW model were combined with flow-gauging measurements as part of a more rigorous assessment that accounts for hysteresis (an offset or lag between change in flow and change in level at the gauge, usually caused by downstream storage effects) and provides a clear distinction between the flow at the gauge and the total upstream catchment flow. Separate ratings were developed relating the stream gauge level recorded at the Boggabilla to the following:

- Flow at the Boggabilla gauge site, located downstream of Boggabilla that includes the main river flows and breakout to Brigalow Creek system (which occurs at and downstream of the gauge site) but exclusive of upstream breakout to Whalan Creek and Morella Watercourse
- Correlated total flow from the upstream catchment (measured approximately 7 km east of Boggabilla where Bruxner Highway crosses the floodplain).

Calibration results presented below use the rating derived for the flow at the Boggabilla gauge site, noting that there is a delay of several hours between the total flow upstream and the gauge level at Boggabilla.

Review of the Goondiwindi gauge rating and record indicate that there is a significant issue with the high flow rating. During larger flood events the river level rises to and plateaus at approximately 10.5 m gauge datum (~1,400 m³/s rated flow). This can be attributed to the significant floodplain flow and breakouts that occur upstream of and around the gauge site. Large changes in flow above this level cause very little change in river level. The derived gauge rating, based on extrapolation of entirely in-stream flow gaugings, does not accurately register the total river and floodplain flow above this level.

February 1976 flood

Calibration hydrographs for the lower Macintyre floodplain model in the February 1976 flood are shown in Figure C10-A in Appendix C. Only the peak level is available at the Boggabilla gauge for this event. The modelled peak shows good agreement with the rated flow. At Goondiwindi, although the gauged flow plateaus at approximately 1,400 m³/s as discussed above, the calculated hydrograph shows a very good match of the start and end of the hydrograph. This indicates that the combined models are representing the timing of the flood hydrograph very well.

January 1996 flood

The multiple short storm events that formed the January 1996 flood merge to form a single broad peak in the lower Macintyre River. The calibration hydrographs for the lower Macintyre floodplain model in the February 1976 flood are shown in Figure C10-B in Appendix C. A very good match of the rated flow is obtained at the Boggabilla stream gauge. The underestimate of the flow on back end of the flood peak on the 26 January can be directly related to the 'missing' flow in the Macintyre River and Dumaresq River catchment models, discussed in Sections 17.2.3.1 and 17.2.3.2. The stream gauge at Goondiwindi also shows a good match of the timing of the hydrograph.

January 2011 flood

The January 2011 flood produced a minor peak in the lower Macintyre River on the 9 January to 10 January, followed by a larger second peak on the 14 January. The calibration hydrographs for the lower Macintyre floodplain model in the January 2011 flood in Figure C10-C in Appendix C show a good match of the shape and timing of the recorded hydrographs at Boggabilla and Goondiwindi.

Summary

A new URBS hydrologic model of the Macintyre River floodplain was developed to cover the area from the upstream tributary models to downstream of Goondiwindi. The model includes Whalan Creek and its tributaries to the south of the Macintyre River and parts of Brigalow Creek catchment to the north. The primary purpose of the model is to provide both major inflow and local subarea runoff for input into the TUFLOW hydraulic model.

The complex flow characteristics of the Macintyre River floodplain test the limits of what would typically be expected from a hydrologic routing model. The lower floodplain URBS model is almost entirely overlapped by the TUFLOW hydraulic model; therefore, although desirable, it is not critical that a perfect calibration of the URBS hydrologic model is achieved. Nevertheless, the calibration results suggest that the model produces a very good representation of the river routing and that the breakout flow relationships derived from the TUFLOW modelling and implanted in the URBS model provide an acceptable approximation of the behaviour of the floodplain. The model shows a very good calibration at the Boggabilla stream gauge and a good representation of the in-channel flows at Goondiwindi, noting that additional work is required to represent the floodplain flow/breakout between Boggabilla and Goondiwindi and/or to improve the Goondiwindi rating so that it includes floodplain flows.

17.2.3.7 Conclusions

Operation of the revised Macintyre River hydraulic model is heavily dependent on the input from three URBS hydrologic models of the upstream Macintyre River, Dumaresq River and Macintyre Brook tributaries originally developed for the BRVFMP. These models were initially calibrated against the major February 1976 flood event, and later recalibrated to against a series of floods that occurred in January and May of 1996; however, these later calibrations used different channel routing parameters without revisiting the original 1976 calibration, resulting in two (or more) dissimilar models.

Recalibration of the hydrologic models was undertaken primarily for the purpose of developing a single set of models capable of replicating these (and other) events. The recalibration has taken advantage of additional stream gauge and rainfall information, improved rainfall distribution methodology, as well the significant additional information obtained during the more recent major flood event in January 2011. The recalibration has achieved several significant outcomes:

- The models were successfully calibrated at multiple stream gauges within the catchment, demonstrating that they are internally consistent and that they successfully represent the runoff contributions from all parts of the catchment in terms of both magnitude and timing
- Multiple events of different magnitude and duration were simulated using a single set of parameters, demonstrating that the routing characteristics are representative of the real catchment and are consistent over a wide range of flows. Adopted calibration parameters are summarised in Table 17.4.
- Inclusion of additional rainfall, stream gauge and event information not available for the previous calibration has provided additional calibration data at key points in the model, particularly the downstream reaches. The models are believed to provide an improved representation of the magnitude and timing of inflows into the lower Macintyre floodplain.
- Overall, these improvements indicate that the model is reliably representing the physical stream routing characteristics present in the catchment, providing improved confidence the models can be used to reliably represent events other than the specific calibration floods.

New models have also been developed for the Ottleys Creek catchment, and the lower Macintyre River floodplain area covering the area from the upstream catchments to downstream of Goondiwindi. Although the primary purpose of the lower floodplain model is to provide inputs for the hydraulic modelling, the model has nevertheless attempted to replicate the floodplain flow characteristics which see larger flood events overflow into the Whalan Creek, Morella Watercourse and Brigalow Creek systems.

The lower floodplain model was successfully calibrated to the stream gauges at Boggabilla and Goondiwindi. This successful calibration indicates that although the methods used to model the breakout patterns are relatively simplistic, they nevertheless provide a reasonable representation of the river flow properties. Importantly, this good calibration also confirms that the upstream catchment models are well representing the magnitude and timing of the inflows used by the lower catchment model.

Table 17.4 Macintyre River—Hydrologic model calibration parameters

Parameters	Macintyre River	Dumaresq River	Macintyre Brook	Ottleys Creek	Macintyre River Floodplain
Alpha	0.45	0.45	0.45	0.45	0.45
n	0.85 ¹	0.85	0.85 ²	0.85	1.0
Beta	1.2	1.2	1.2	1.2	1.2
m	0.8	0.8	0.8	0.8	0.8
x	0.3	0.3	0.3	0.3	0.1
Reach Length Factors	0.6 to 1.6	1.0 to 3.0	0.8 to 2.0	1.0	0.3 to 3.0
Initial loss (mm)					
1976	20 to 60	30 to 70	60 to 80	30	60
1996	50 to 100	40 to 80	45 to 60	30	30
2011	25 to 50	20 to 40	30	50	30
Continuing loss (mm/hr)					
1976	2.4 to 3.3	2.5 to 4.5	2.8 to 4.0	3.5	2.0
1996	1.2	0.5 to 3.0	0.5 to 3.0	3.5	3.0
2011	1.6 to 3.5	0.8 to 2.7	1.0 to 1.4	2.1	2.1

Parameters	Macintyre River	Dumaresq River	Macintyre Brook	Ottleys Creek	Macintyre River Floodplain
Baseflow					
Baseflow Constant (BC)	0.05 to 0.1 ³	0.1	0.05 to 0.1 ³	0.1	n/a ⁴
Baseflow Exponent (BM)	1	1	1	1	
Daily recession (BR)	0.5	0.5	0.5	0.5	

Table notes:

- 1 n=1.0 for lower reach between Ridgeland and Holdfast
- 2 n=1.0 for lower reach between Inglewood and Booba Sands
- 3 Baseflow values were determined by calibration to historical flood recession. A baseflow constant of 0.1 was found to be appropriate for most gauge locations. When routed through a reservoir, baseflow from the upstream catchment is combined with and included with the surface runoff and is subsequently routed downstream. For gauges having a significant proportion of the contributing flow coming from a reservoir (e.g. Inglewood, Ashford), a lower baseflow constant was found to be necessary to account for baseflow already included with the routed flow from the reservoir.
- 4 Baseflow is included in the inflows from the contributing upstream models. Due to the importance of breakout flows upstream of Boggabilla, dependent on the total flow inclusive of baseflow, these baseflows were deliberately combined with the quick flow so that the model routed the total flow.

17.2.3.8 Limitations

The assessment was conducted using best available information and practices. The URBS modelling is dependent upon data provided by outside sources.

- Modelling of the three major tributary catchments (Macintyre River, Dumaresq River and Macintyre Brook) was conducted using URBS models provided by NSW DPE. The model information includes subarea connectivity, catchment areas and reach lengths in text format. Subarea centroid locations were provided however other GIS information (e.g. subarea boundaries and reach alignments) was not provided. The general configuration of the model was assumed to be correct. Errors were identified with a number of the subarea centroid locations. These were repositioned using the best estimate of the correct location.
- The calibration has used stream gauge and rainfall data sourced from third parties (predominantly BoM, DoR (Qld) and WaterNSW). Data was cross-checked and verified where possible, and a number of apparent translation errors in the data records were identified and corrected; however, the data still potentially contains errors outside FFJV's control.
- The source of stream gauge data for a number of sites from the original BRVFMP calibration are unknown
- Flows at stream gauges were calculated using level-flow ratings. These ratings have been subject to change historically and will likely continue to change as additional information is recorded at the gauges. Flow calculations have used the best understanding of the appropriate stream gauge rating.

17.2.4 Model validation

The three historical events were simulated as calibration events to test the calibration parameters. No further validation events were undertaken for the Macintyre River catchment.

17.2.5 Design model parameters

Hydrologic modelling to determine design flows for the Macintyre River system was undertaken using a DEA based on the Ensemble Event methodology described in ARR 2019.

The URBS hydrologic models for the Macintyre River catchment were used to estimate the design hydrographs for the 20%, 10%, 5%, 2%, 1%, 1 in 2,000, 1 in 10,000 AEP and PMF events. A climate change scenario was also considered for the 1% AEP event based on Representative Concentration Pathways 8.5 scenario for 2090.

Details of the adopted hydrologic model parameters are presented in the following sections.

Hydrologic information to assist estimation of design event flows was sourced from the ARR 2019 Data Hub as summarised in Table 17.5.

Table 17.5 Macintyre River—Summary of URBS model design event inputs

Input parameter	Remarks
Design rainfall	Intensity-Frequency-Durations for each sub-catchment were downloaded from the BoM's website to account for variation in rainfall across the catchment.
Extreme event rainfall	Probable Maximum Precipitation depths for durations up to 6 hours (for use in modelling the PMF event) were obtained using the method presented in <i>The Estimation of Probable Maximum Precipitation in Australia: Generalised Short-Duration Method</i> (BoM, 2003). Probable Maximum Precipitation depths of durations of 24 hours and greater were obtained using the method presented in the 'Guidebook to the Estimation of Probable Maximum Precipitation: Generalised Tropical Storm Method' or Revised Generalised Tropical Storm Method (BoM, 2005). The rainfall depths for the 1 in 10,000 AEP event were estimated using the interpolation method presented in ARR 2019 Book 8 Section 3.5.
Losses	Refer Table 17.7
Areal reduction factor	Areal reduction factors for AEPs up to 1 in 2,000 were calculated as a function of AEP, duration and the catchment area to the area of interest were calculated using the methodology outlined in ARR 2019 Book 2 Section 4.3.1.
Ensemble temporal patterns	Central Slopes region.
Pre-burst depths	Median pre-burst depths were downloaded from the ARR 2019 Data Hub for each sub-catchment. Pre-burst depths vary by design storm event AEP and duration. Pre-burst depths were applied to the model by reducing the initial losses for each storm event.

17.2.5.1 Rainfall data for events up to the 1 in 2,000 AEP events

Spatial distribution

Rainfall IFD tables are used to describe the probability of occurrence of a rainfall depth or intensity within a specified duration. Intensity-Frequency-Duration relationships for the study were compiled using:

- Frequent to Rare probability (1EY to 1 in 2,000 AEP) data sourced from the BoM online Design Rainfall Data System (2016)
- Intermediate values were interpolated using the methods described in ARR 2019.

Individual IFD relationships were obtained at each of the subarea centroids. The design event methodology assumes that the rainfall across the catchment has the same frequency and duration for each modelled event but varies spatially according to the IFD relationship at each location.

The BoM rainfall IFD relationships are for specific points in the catchment and are not representative of the areal average rainfall intensity across the entire catchment. Areal reduction factors are applied and discussed below.

Areal reduction factors

Areal reduction factors are used to provide a correction between the catchment rainfall depth and the mean of the point rainfall depths across a catchment (for the same AEP and duration combination). ARR 2019 provides the methodology for calculating ARF as a complex function of catchment area, event duration and AEP. The ARF affects the average depth of rainfall across the catchment and does not account for variability in the spatial or temporal distributions over the catchment. The PMP calculation methodologies include depth as a function of catchment area and no ARF is required.

Because the ARF changes with contributing catchment area, the design event modelling has assessed the design flow conditions at several locations along the Project alignment applying catchment-average rainfall depths appropriate to each location. For the Macintyre River assessment this has included the major tributary flows from Macintyre River upstream of Boggabilla (combined Macintyre River, Macintyre Brook, Dumaresq River and Ottleys Creek), as well as floodplain creek crossings of Back Creek, Forest Creek, Mobbindry Creek and Stray Leaves Creek. The characteristic design floods at each location were identified and used as an ensemble for the hydraulic assessment to identify the worst-case design conditions along the length of the alignment.

The standard Design event methodology also assumes a constant AEP and ARF across the catchment. That is, for an event of specified AEP, the total rainfall depth at every point in the catchment has the same AEP (although the actual depth may vary in accordance with the statistical IFD relationship which again may vary across the catchment, as discussed in the Spatial distribution section above).

Real storm events over large catchments are usually characterised by concentrations of rainfall in different parts of the catchment and bursts that occur at different times in different areas. This potentially has significant impact on the relative magnitude and timing of the contributing flows from each of the tributaries.

Table 17.6 Macintyre River—Areal reduction factor coefficients from ARR 2019 Data Hub

a	b	c	d	e	f	g	h	i
0.159	0.283	0.25	0.308	7.3e-07	1.0	0.039	0.0	0.0

Temporal patterns

A design event is the simulation of a synthetic storm event with rainfall intensity of a particular duration and frequency (typically defined as an Annual Exceedance Probability or AEP) and a defined temporal pattern, catchment losses and other catchment properties. Key characteristics of the ensemble approach include:

- Events covering a range of durations are assessed for each AEP to determine the duration that produces the highest (worst-case) flow
- An ensemble of 10 TPs are simulated for each duration and AEP. These are assessed to produce a characteristic value for each duration and AEP (see discussion below).
- Multiple event durations are assessed for each AEP. The highest characteristic value and the event duration that produces it are taken as the design flow and the critical duration, respectively.

There is some ambiguity as to how the characteristic value should be determined from the 10 TPs. The average and median flood peaks tend to produce similar but non-identical values. The median value is theoretically less sensitive to the influence of outliers (noting that these may be high or low, so the average is not necessarily more or less conservative). The limitation of both these options is that they do not have an associated temporal pattern, which is necessary for dynamic hydraulic modelling. For the purposes of this investigation:

- The median (50 per cent probability) value was adopted for reconciliation with the FFA. Since 10 patterns are simulated, the median value actually lies halfway between the 5th and 6th highest values.
- The Rank 6 hydrograph was adopted for simulation in the hydraulic model. This produces a peak flow that is conservative, typically slightly higher but within the uncertainty of the modelling and reconciliation process. Each adopted Rank 6 hydrograph was reviewed to confirm that it had a relatively 'typical' shape as well as the peak. (The hydrograph shape is a function of both the TP and the catchment. Most hydrographs tend to have a similar shape characteristic of the catchment unless the causal pattern is highly unusual.)

The ensemble modelling was conducted using the standard set of temporal distribution patterns obtained from the online ARR Data Hub. This provides an ensemble of 10 patterns for each standard burst durations for a range of different catchment sizes. The Macintyre River catchment lies within the Central Slopes region.

Each design event has a single TP that is applied simultaneously across the whole catchment. The standard design event methodology also assumes a constant AEP and ARF across the catchment. That is, for an event of specified AEP, the total rainfall depth at every point in the catchment has the same AEP (although the actual depth may vary in accordance with the statistical IFD relationship which again may vary across the catchment, as discussed in Section 17.2.5.1).

Real storm events over large catchments are usually characterised by concentrations of rainfall in different parts of the catchment and bursts that occur at different times in different areas. This potentially has significant impact on the relative magnitude and timing of the contributing flows from each of the tributaries.

17.2.5.2 Design rainfall for 1 in 10,000-year AEP and PMF events

Rainfall depth estimation

In order to capture the spatial variability of rainfall intensity across the large catchment, rainfall depths were determined for each individual sub-catchment (315 sub-catchments across five models) by combining IFD information obtained from the BoM website, factored using the appropriate ARF, and interpolated to the PMP for extreme events in accordance with standard ARR recommendations. Different ARF and PMP area were applied for each focal location being investigated.

17.2.5.3 Design rainfall losses

Rainfall losses were modelled using a standard Initial Loss/Continuing Loss model. Although continuing losses are typically assumed to be relatively constant for a catchment (although they reduce during very long duration events as the catchment substrata becomes saturated), initial losses are dependent on catchment conditions at the start of the event. Rainfall depths from the BoM IFD tables and the ARR Data Hub TPs are both derived for storm bursts (a burst is not necessarily a complete storm and may be preceded and followed by additional rainfall).

In reality, the variability of catchment and rainfall conditions means that a flood of a particular AEP may be the product of a rainfall event of rarer AEP (higher intensity) with higher catchment losses, or of more frequent AEP (lower intensity) with lower losses. A fundamental aspect of the design event methodology is the requirement to use 'probability-neutral' losses (and other catchment parameters such as dam levels, discussed in Section 17.2.5.4) that produce a direct relationship between rainfall AEP and the AEP of the resulting flood.

The initial losses adopted for design event modelling must therefore consider antecedent rainfall prior to the modelled storm burst and be probability neutral. The ARR Data Hub provides regionalised values for initial storm loss and continuing loss, as well as pre-burst depths (rainfall that occurred prior to the design storm burst, nominally in the prior seven days) as a function of rainfall burst AEP and duration. These theoretically allow the probability-neutral burst initial depth to be calculated as the pre-burst depth subtracted from the storm initial depth. The ARR Data Hub losses for each catchment are listed in Table 17.7. Significant differences between different catchments are noted, particularly in the continuing loss rate, which appears to unrealistically distort the contributions from the tributaries (for example, for minor events there is almost no contribution from the Dumaresq River catchment if the regionalised losses are applied).

The ARR Data Hub also reports that a review by the NSW Office of Environment and Heritage identified a bias in the standard ARR 2016 design event method when using the default ARR data hub losses and pre-burst depths. Probability-Neutral Burst Initial Loss as a function of AEP and duration are now available from the ARR Data Hub for use in NSW regions. This method is difficult to implement for the current study because a portion of the Macintyre River catchment is located outside NSW where probability-neutral burst losses are not available. A consistent methodology is required to enable entirety of the catchment to be modelled.

These issues are not considered to be significant as both the regionalised storm losses and NSW reconciled losses are intended for use only where more reliable and/or site-specific information is unavailable. Loss values were based on experience with the model calibration and reconciliation to at-site stream gauge FFA's discussed in Section 17.2.7.2. Flood-frequency analysis were used in preference to the ARR Data Hub values. The reconciliation process is discussed further in Section 17.2.7. The reconciled losses adopted for the design event modelling are summarised in Table 17.7. (It should be noted that for calibration events, the applied catchment losses also represent a tool for correcting for errors/uncertainties in the rainfall data; therefore, although the median losses over multiple events would be expected to be similar to the typical catchment losses, the losses used for any particular event do not necessarily match the actual losses).

Table 17.7 Macintyre River—ARR regionalised and adopted at-site flood frequency analysis reconciled losses

Catchment	ARR Data Hub losses		Reconciled losses	
	Initial (mm)	Continuing (mm/hr)	Initial (mm)	Continuing (mm/hr)
Macintyre River to Holdfast	35	2.4	25	2.4
Dumaresq River to Glenarbron	29	4.6	29	3.0
Macintyre Brook to Booba Sands	27	1.2	30	2.5
Ottleys Creek	57	0.1	30	3.5
Lower Macintyre River floodplain	47	0.3	30	3.0

17.2.5.4 Reservoirs

Three notable reservoirs are located upstream of Boggabilla and included in the hydrologic models—Pindari Dam on the Severn River in the Macintyre River catchment upstream of Ashford; Glenlyon Dam on Pike Creek in the upper Dumaresq River catchment; and Coolmunda Dam on Macintyre Brook upstream of Inglewood. All three dams were completed between 1969 to 1972, with Pindari Dam undergoing a major upgrade in 1994 that significantly increased its capacity. Reservoir details are summarised in Table 17.8.

The storage-discharge relationships shown in Figure 17.3 are from the BRVFMP models and have been adopted for the design event assessment. It may be noted that Coolmunda Dam has a gate-controlled discharge, and the discharge relationship may vary depending on gate operation decisions. The relationship is understood to have changed between the 1996 and 2011 calibration events. Figure 17.4 compares the current URBS rating against a correlation between the reported reservoir storage and rated tailwater gauge flows for the 2022 water year, including the largest recent flood in November 2021, showing a reasonable agreement. The adopted rating is therefore considered appropriate for design purposes based on best currently available information.

The storage upstream of the reservoirs provides a retarding and attenuating effect on the flood hydrograph. This includes any initial empty capacity below the full supply level present at the start of the event that must be filled before the dam begins to spill, represented in the hydrologic model as an initial VBF, and the storage routing through the reservoir represented as a storage-discharge relationship in the model.

As with rainfall losses, for the design event approach, it is necessary to adopt probability-neutral VBF values. Analysis of the historical rainfall and reservoir volume data identified that there was no correlation between reservoir volume and rainfall magnitude, meaning that large rainfall events are equally likely to occur when the reservoir is empty as it is full. The reconciliation discussed in Section 17.2.7 identified that using a median VBF achieved good consistency between the design event and FFA.

Table 17.8 Macintyre River—Reservoir storage details

Reservoir name	Catchment	Stream	Completion date	Storage volume (ML)	Median VBF (ML)
Pindari Dam	Macintyre River	Severn River	1969	38,000	1,000
			1994	312,000	110,000
Glenlyon Dam	Dumaresq River	Pike Creek	1972	253,600	148,000
Coolmunda Dam	Macintyre Brook	Macintyre Brook	1968	69,000	22,000

Table note:

ML = Megalitres

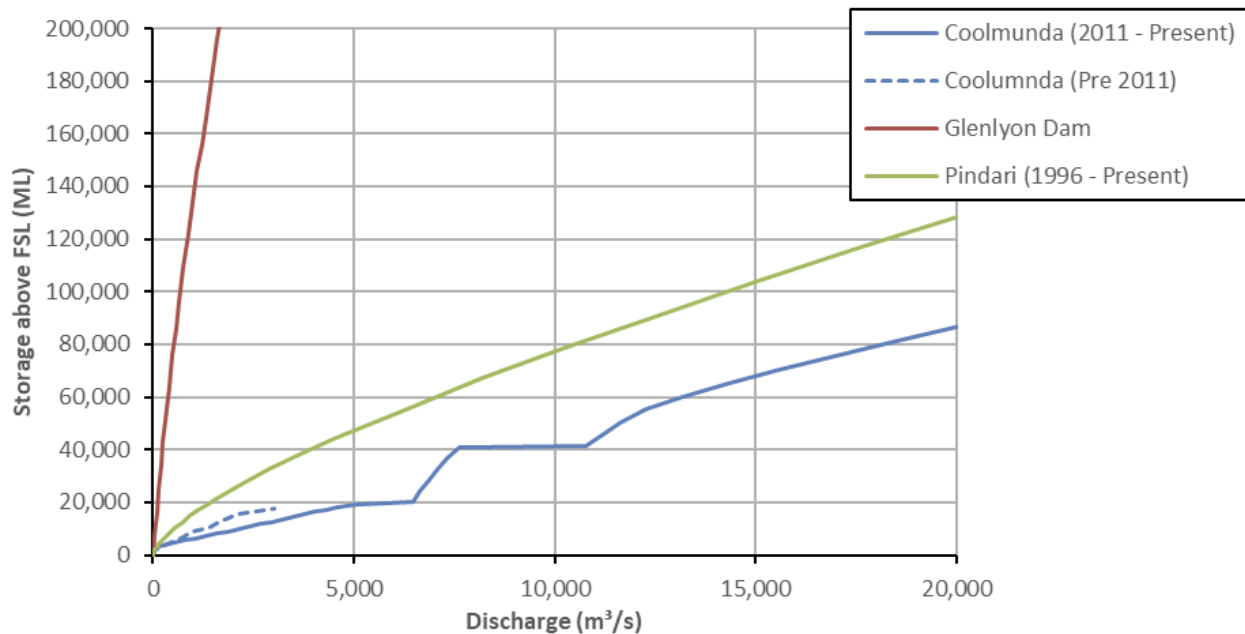


Figure 17.3 Macintyre River—Storage-discharge relationships for major reservoirs

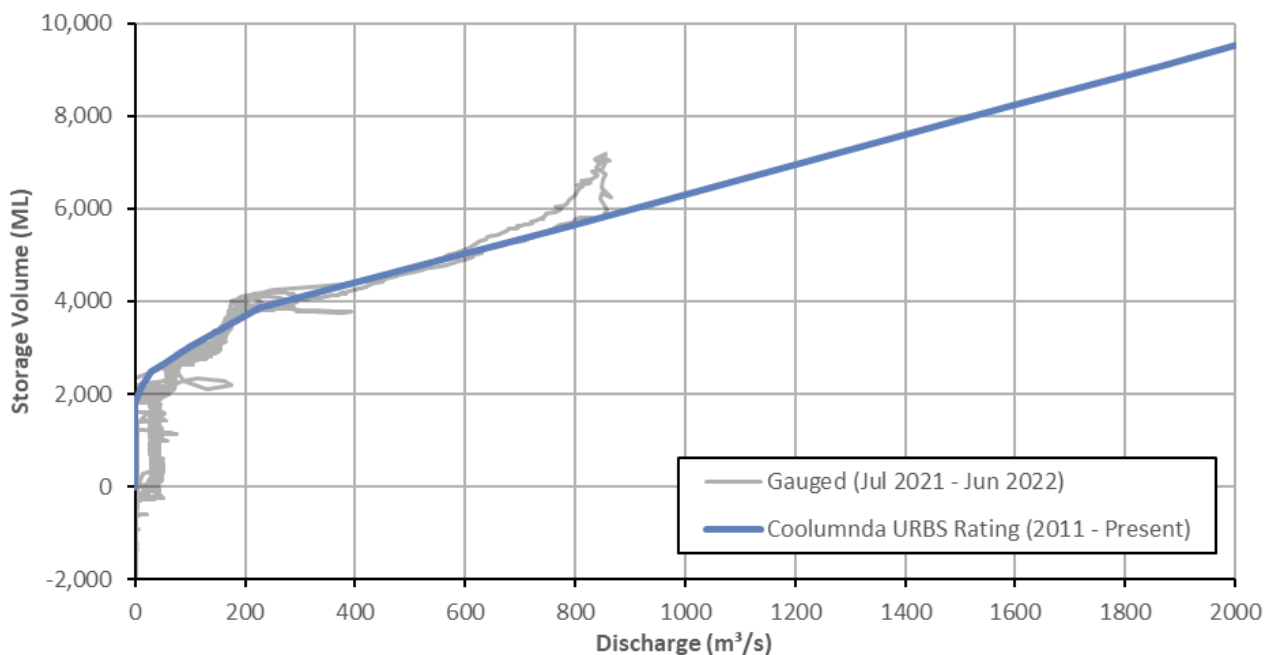


Figure 17.4 Comparison of Coolmunda storage volume against tailwater gauge flow

17.2.6 Baseflow

Baseflow for the design events was implemented using URBS variable baseflow function, adopting the model parameters derived from the calibration. These show in general a very good match of the timing and shape of hydrograph regression indicating that the model is replicating the baseflow generation well. The parameters were found to be consistent at multiple locations and across the different calibration events.

The adopted URBS baseflow parameters (Baseflow recession factor (BR)=0.5, Baseflow constant (BC)=0.1, Baseflow exponent (BM)=1) values correspond to a baseflow volume factor of 0.167, which correspond relatively well with the ARR Data Hub values reported in Table 17.9, with the exception of the Macintyre Brook. It is not clear why these values are so much higher. The ARR Data Hub values appear to be highly variable within the Macintyre Brook catchment. For locations including Macintyre Brook east of Inglewood and areas upstream of Coolmunda Dam the Baseflow Volume Factor and Peak Factor are 0.103 and 0.027, respectively, yet for Canning Creek these jump significantly to 0.37 and 0.17. There is no obvious reason for such a significant difference (or how reliable the Data Hub regionalised values are) when a good recession fit of gauges on Macintyre Brook, Canning Creek and the Coolmunda inflow tributaries can be achieved using similar model parameters. The calibrated model parameters have therefore been adopted.

Table 17.9 Macintyre River—ARR Data Hub baseflow values

Location	Baseflow volume factor	Baseflow peak factor
Macintyre River at Holdfast	0.167	0.035
Dumaresq River at Glenarbon	0.109	0.037
Macintyre Brook at Booba Sands	0.421	0.103
Macintyre River at Boggabilla	0.144	0.043

In a basic catchment, URBS only generates baseflow empirically at an export location based on specified parameters at that location. The baseflow is not routed through the system; however, the baseflow implementation is complicated by the presence of reservoirs because (if baseflow is specified upstream of the reservoir) the reservoir routing is performed on the total flow (quickflow plus baseflow). This is because the baseflow can be an important component of the volume and reservoir routing, particularly in long or multi-peak events (e.g. 1996 and 2011 events).

This also means, however, that the generated reservoir outflow is a function of both the quickflow and baseflow. It is not a simple procedure to then remove what could be considered a 'baseflow' component, nor does it seem particularly necessary (in real life the flow exiting a reservoir will be routed downstream in exactly the same manner, regardless of where it comes from), provided that it is recognised that the generated stream flow includes the baseflow from the area upstream of the catchment. Because a large proportion of the catchment to the downstream gauges such as Holdfast and Booba Sands is reservoir routed, the BC is lower than for upstream gauges on streams without reservoirs as the baseflow generation at the lower gauges only needs to account for the additional baseflow from the unregulated part of the catchment.

This feature has been deliberately manipulated for the Lower Macintyre River model. A major feature of the river flow is the breakout from the main river channel into the Whalan, Morella and Brigalow Creek systems. The breakout flow is a function of the total flow of the river, irrespective of whether this originated from quickflow or baseflow. The loss functions used to represent the bypasses is calculated from the routed flow only. Dummy storages have therefore been used to convert the combined quickflow and baseflow from the upstream tributaries into routed flow, so that overflow will correctly occur when the river flow reaches the requisite level. It is acknowledged that additional baseflow from the lower catchments will not be generated; however, this is considered to be a very small proportion of the catchment to the area of interest at the rail line.

17.2.7 Hydrologic model validation

17.2.7.1 Regional flood frequency analysis

At-site FFA involves the statistical analysis of flood data recorded at in the catchment (typically a stream gauge) to identify and fit a probability distribution function, which can then be used to perform risk-based design and flood risk assessment. An FFA was conducted at 10 of the major stream gauges throughout the catchment to provide input to design flow estimates and to help determine probability-neutral catchment parameters for the design event modelling. Period of record and catchment area for each of the gauges are listed in Table 17.10.

Table 17.10 Macintyre River—Flood frequency analysis gauge data summary

Catchment	Gauge name	Stream	Catchment area (km ²)	Period of record
Macintyre River	Ashford	Severn River	3,285	1970 to 2021
	Wallangra	Macintyre River	2,130	1972 to 2021
	Holdfast	Macintyre River	6,800	1972 to 2021
Dumaresq River	Roseneath	Dumaresq River	5,560	1972 to 2021
	Bonshaw	Dumaresq River	7,240	1972 to 2021
	Cunningham/Glenarbon ¹	Dumaresq River	~9,000	1954 to 1988, 1997 to 2021
Macintyre Brook	Inglewood	Macintyre Brook	3,430	1954 to 2021
	Booba Sands	Macintyre Brook	4,092	1984 to 2021
Ottleys Creek	Coolatai	Ottleys Creek	385	1979 to 2021
Lower Macintyre	Boggabilla	Macintyre River	22,500	1886 to 2021 ²

Table notes:

1 Flood records from the gauges at Cunningham and Glenarbon Weir were combined

2 Only significant flood peaks available prior to 1982

Stream gauge data was obtained from the Continuous Water Monitoring Network webservice provided by WaterNSW and the Water Monitoring Information Portal operated by DRDMW, formerly the Department of Resources. A brief review of the gauge history was undertaken for each gauge and, where considered appropriate, historical peak flows were re-rated using current rating information.

The analysis was conducted using an annual series of peak flows, extracted using a 'Water Year', commencing in September. The FLIKE software package was used to identify low outliers using the multiple Grubbs Beck test and to fit both the LPIII and Generalised Extreme Variable (GEV) probability distributions. The LPIII and GEV are two distributions commonly used for flood frequency fitting in Australia. ARR does not recommend one distribution over the other, and current practice is usually to adopt the distribution that best fits the data. Where multiple gauges in a catchment examined, the same distribution would be adopted for all gauges.

For the Macintyre River gauges, the LPIII distribution generally appears to provide a better fit for the majority of gauges, which is common for catchments in Queensland and NSW; however it should be noted that the appropriateness of the distribution and validity of the FFA at a number of gauges is affected by non-homogeneity of the gauge record. The reconciliation process has focused on matching the design event flows to the historical frequency distribution for frequent events, where both probability distributions tend to give similar results.

17.2.7.2 Reconciliation to at-site flood frequency analysis

Design event modelling and at-site FFA provide two independent methods for estimating design flows. Each method has specific strengths and weaknesses:

- Flood frequency analysis is based on site-specific information; however, it is dependent on (among other things) the reliability of the rating used to derive the flow, the statistical representativeness of the historical data, and the validity of the probability distribution, all of which tend to decrease with increasing event magnitude.
- Design event modelling uses rainfall frequency distributions based on amalgamation and filtering of multiple rainfall gauges that generally have access to longer historical record and are (theoretically) less susceptible to outliers that may affect individual sites; however, it requires reliable model calibration and the estimation of appropriate probability-neutral catchment parameters.

Preferred practice is to use the frequent to infrequent range of the FFA distribution, which is the most reliable region of the FFA, to help derive probability-neutral catchment properties. Extrapolation to rare events is then conducted using the rainfall-based design event method, minimising the uncertainty in the projection of the at-site gauge data relationship. Reconciliation to produce probability-neutral catchment parameters included:

- Probability-neutral rainfall losses were derived based on reconciliation at typically two to three of the major gauges in each of the catchment models. Reconciliation was achieved using consistent losses for all gauges in each catchment model; however, different loss rates were derived for each catchment. The losses used for the reconciliation are summarised in Table 17.7.
- A constant storm initial loss value was used for each catchment, with the burst initial losses required for the design event burst calculated using the median pre-burst depth relationships obtained from the ARR Data Hub typically producing a good reconciliation (see discussion below regarding non-homogeneous record influence on FFA and reconciliation).
- Good reconciliation was achieved using the median VBF for the reservoirs allowing gauges unaffected or only partially affected by the reservoir to be reconciled using the same losses as those affected by the reservoir.

17.2.7.3 Non-homogeneous record influence on flood frequency analysis and reconciliation

A fundamental principle of FFA is that the flood peak frequency distribution fits a standard probability distribution. The majority of the empirical evidence used to support the use of the LPIII and GEV distributions is from consistent record from natural catchments. The reduction in flood peak resulting from pre-flood reservoir storage availability is likely to be most pronounced for minor rainfall events. The reservoirs will therefore potentially alter the shape of the historical flood probability distribution, which would affect the ability of the theoretical distribution to fit the data and the reliability of the projection to rarer events.

An example of the effect of the dams on both the FFA and DEA is provided in Figure 17.5, which compares the FFA and DEA predictions with the historical flood record at Inglewood, located just downstream of Coolmunda Dam. (Note that the data in Figure 17.5 considers only the period post-1970 when Coolmunda Dam was constructed). The equivalent chart for Inglewood, shown in Figure 17.9, incorporates data back to 1954. The FFA confidence limits were omitted for clarity. Figure 17.5 demonstrates that:

- The GEV distribution in particular does not provide a particularly good fit of the historical flood probability distribution, with the influence of the frequent events causing a significant deviation of the projection to rare events
- The initial storage volume has a significant effect on the frequent events. A similar effect could be achieved by increasing the initial loss (the reservoir effectively captures the initial runoff from the upstream catchment). The key difference is that the reservoir VBF affects only the area upstream of the reservoir. Use of a median historical VBF was found to allow reconciliation of different gauges within the same catchment using the same rainfall losses.

Additionally, for several of the examined stream gauges the reservoirs have not been present for the full period of the gauge record; in particular, Ashford and Holdfast (upgrade of Pindari Dam), Inglewood (construction of Glenlyon Dam) and Boggabilla (construction of Coolmunda and Glenlyon Dams then upgrade of Pindari Dam). To address these issues and provide design flows that represent the current condition of the catchment and reservoirs, the following approach was adopted:

- The design event methodology was conducted using models that represent both the current and pre-reservoir condition of the catchment. The reconciliation design flows were calculated as the weighted average of the pre- and post-reservoir flows based on the period of the historical gauge record for which those conditions were present.
- Catchment parameters (losses and reservoir VBF) were modified to reconcile the design event flows to the historical frequency distribution for frequent events. Projection to rare events removes dependence on the fit and appropriateness of the theoretical probability distribution.
- Flows for design were calculated using models representing the current catchment condition with the reconciled catchment parameters. The current design flows may therefore be different to the reconciled flows due to the influence of the current reservoirs.

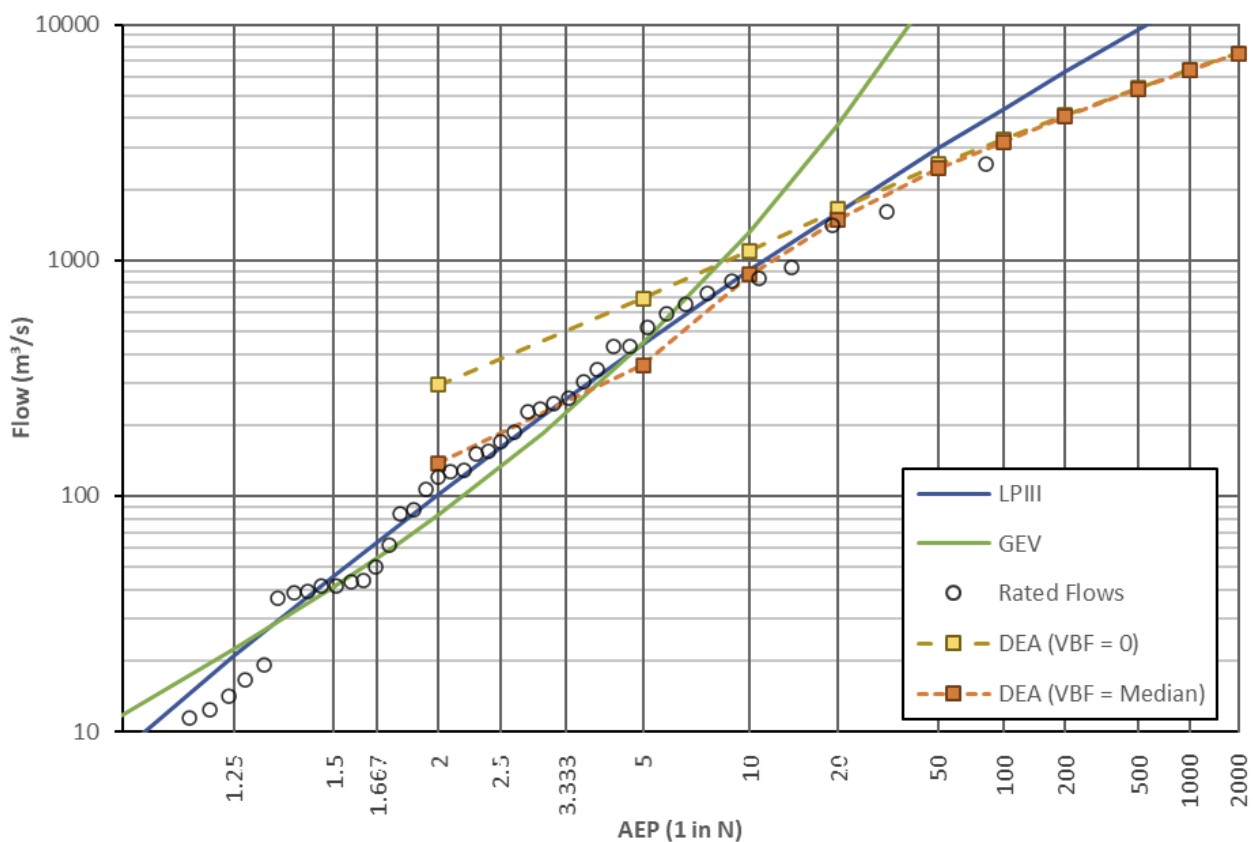


Figure 17.5 Flood frequency analysis fit and effect of Coolmunda VBF on the DEA flows at Inglewood

17.2.7.4 Macintyre River to Holdfast

Flood frequency analysis and reconciliation was conducted at three gauges—the Severn River at Ashford and the Macintyre River at Wallangra and Holdfast. All three gauges have roughly the same period of record (48 to 50 years). Pindari Dam, which controls 64 per cent of the catchment to Ashford and 30 per cent of the catchment to Holdfast, underwent a major upgrade in 1994, roughly halfway through the Ashford and Holdfast gauge records.

Figure 17.6 demonstrates the significant effect of the Pindari Dam upgrade on the design flows at Ashford. (Note that Pindari Dam had comparatively little capacity prior to the 1994 upgrade with a median historical VBF of ~1,000 ML compared to the current 110,000 ML. The influence of the pre-upgrade dam was neglected, and the dam omitted from both the 1976 model calibration and pre-upgrade DEA modelling). The historical flood frequency distribution shows a very good agreement with the weighted average of the pre- and post-upgrade DEA flows. Use of the median historical VBF for Pindari Dam for the DEA approach allows the Ashford gauge to be reconciled using the same catchment losses as the Wallangra (no influence) and Holdfast (partial influence) gauges. The FFA and reconciled design event flows for these gauges are shown in Figure 17.7. A very good reconciliation to all three gauges was achieved using an initial loss of 25 mm and continuing loss of 2.4 mm/hr.

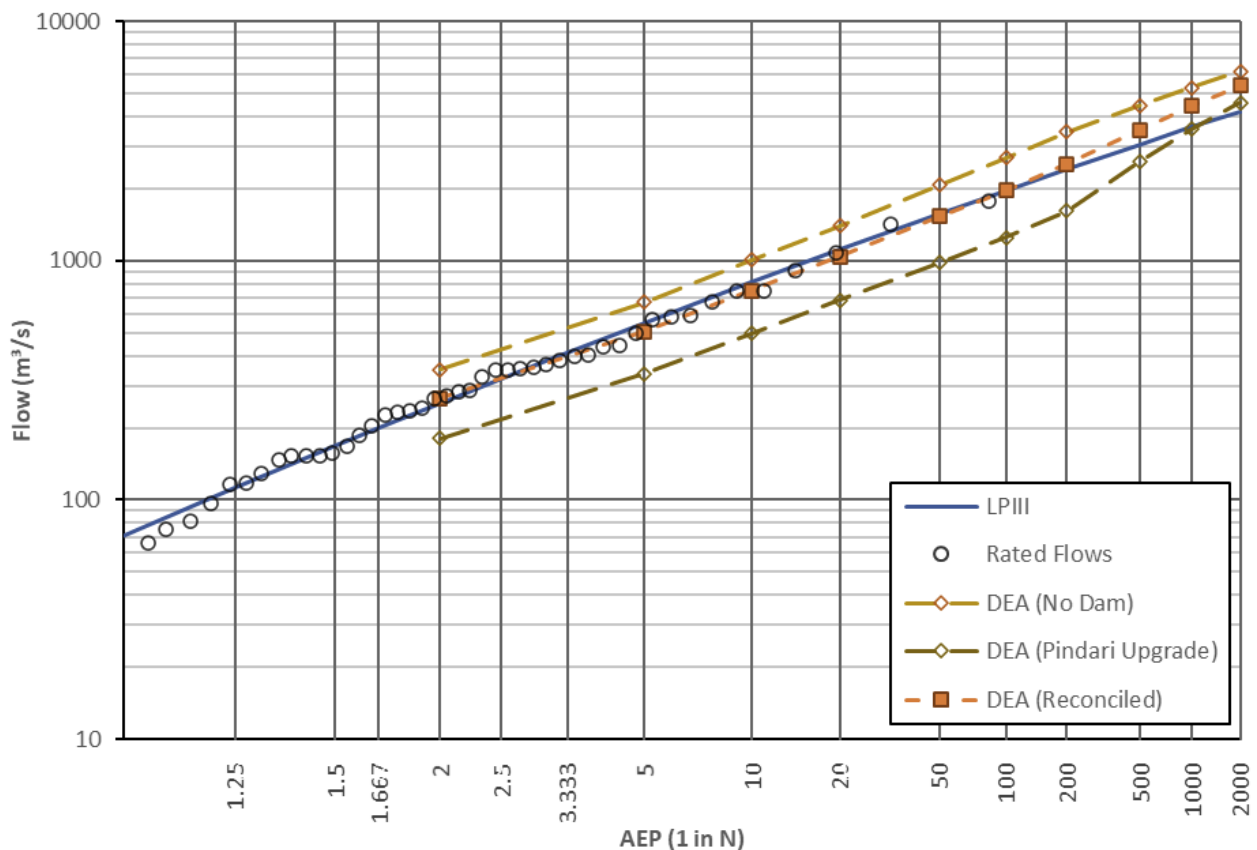


Figure 17.6 Effect of Pindari Dam on the DEA flows at Ashford

MACINTYRE RIVER TO HOLDFAST - RECONCILIATION

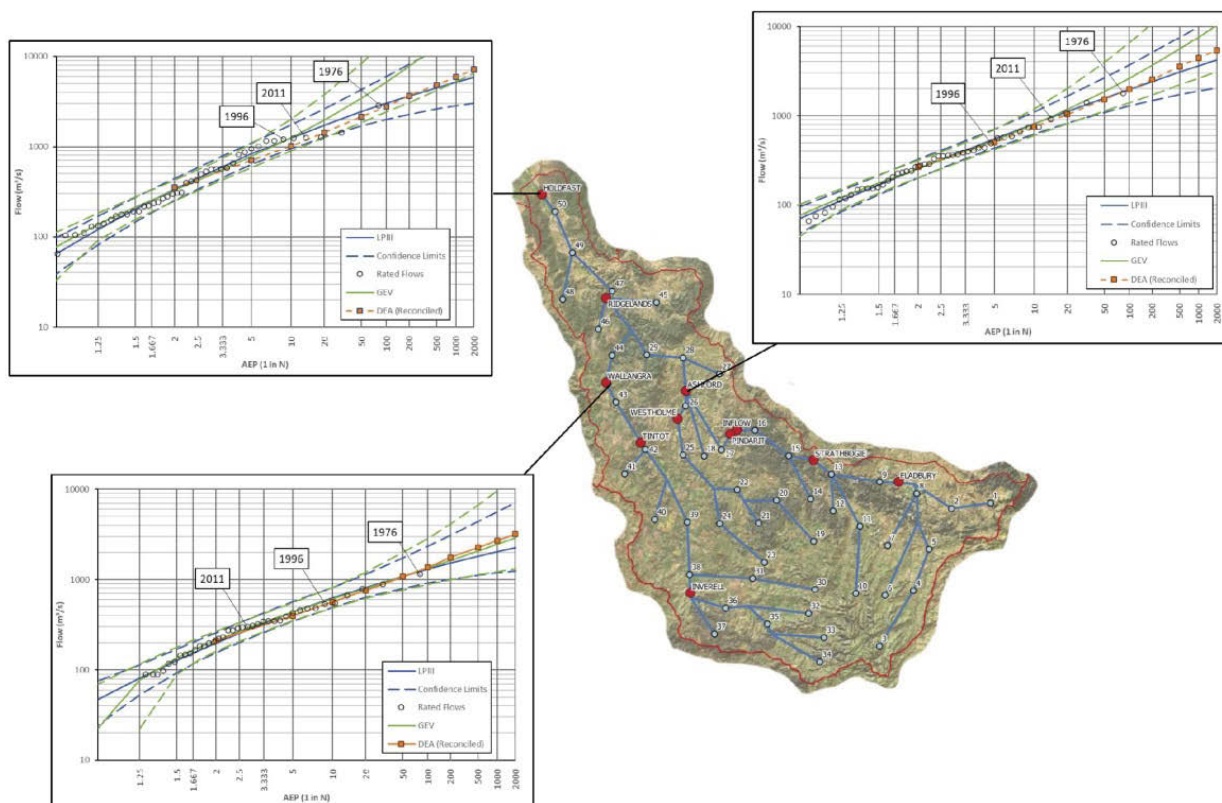


Figure 17.7 Macintyre River to Holdfast—Reconciliation

17.2.7.5 Dumaresq River to Glenarvon

Flood frequency analysis and reconciliation was conducted for the Dumaresq River gauges at Roseneath and Bonshaw. A combined historical record, representative of the downstream end of the model, was compiled from the gauges at Cunningham Weir (closed in 1988) and Glenarvon Weir (opened in 1997). This was produced only to examine the reconciliation to frequent events, as the high-flow ratings at both these gauges are considered highly unreliable (the FFA was conducted by censoring high flows greater than 1,000 m³/s). Glenlyon Dam is located upstream of all these gauges, although it controls a relatively small proportion of the catchment compared to Pindari and Coolmunda Dams, decreasing from 23 per cent at Bonshaw to 14 per cent at Glenarvon.

Flood frequency analysis and reconciled design event flows for the Dumaresq River catchment are shown in Figure 17.8. A good reconciliation is achieved at Roseneath and Bonshaw, using an initial loss of 29 mm, continuing loss of 3 mm/hr and median VBF in Glenlyon Dam; however, if these values are used at Cunningham/Glenarvon Dams then the design event flows are lower than the FFA for frequent events (< 1 in 5 AEP). This is possibly the uniform spatial rainfall distribution across the larger catchment exaggerating the effect of the losses, which is discussed in greater detail in the Lower Macintyre River section below. There are also potentially issues with the reliability of the now-closed Cunningham Weir rating—there is a significant difference in the median annual flow at the Cunningham gauge (500 m³/s) and the Glenarvon gauge (190 m³/s).

The reconciled DEA 1-in-2 AEP flow actually shows good agreement with the probability-plot of the rated Glenarvon flows (noting that with less than 20 years data this is not a reliable FFA). Given these uncertainties, the reconciliation is considered reasonable based on the good agreement at the two other gauges.

DUMARESQ RIVER TO MAURO – RECONCILIATION

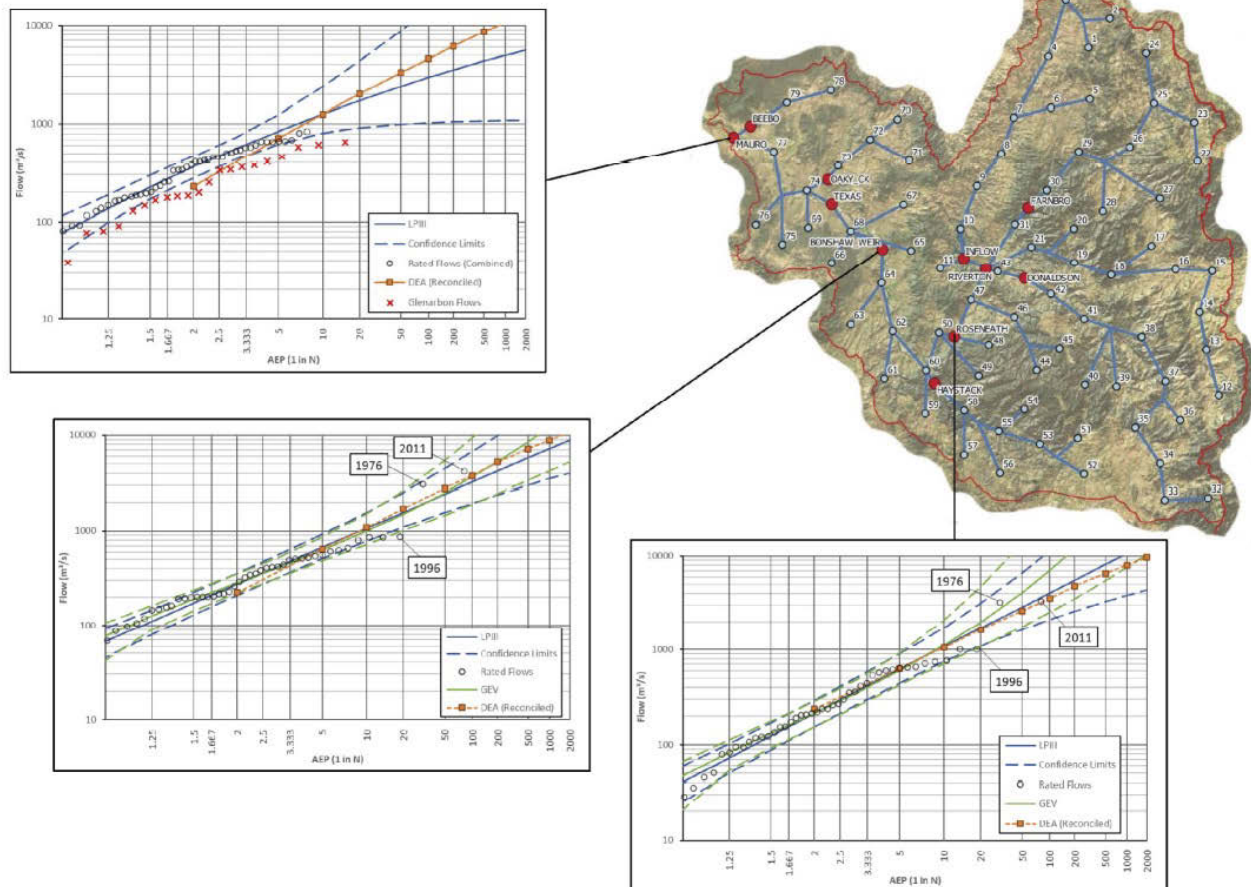


Figure 17.8 Dumaesq River to Glenarvon –Reconciliation

17.2.7.6 Macintyre Brook to Booba Sands

Flood frequency analysis and reconciliation was conducted for the Macintyre Brook gauges at Inglewood and Booba Sands, shown in Figure 17.9. As identified above, Inglewood is located downstream of Coolmunda Dam, which has a significant influence on the gauge record and design flows. The Inglewood gauge control structure was modified in 1981. The high-flow rating from the older gauge is based on flow measurements recorded in 1956 and quite different from the current gauge rating, despite being located at the same site (the control structure would be expected to be fully submerged and have only minor influence on high flows). This results in some uncertainty in the rated flow of the highest floods (which are all from the older gauge) and the effect that they have on the shape of the FFA probability curve. Nevertheless, the DEA flows can be reconciled relatively successfully to the historical flood probability distribution.

The Booba Sands gauge has only been operational since 1984, which is the shortest period on record of the examined gauges and does not include the largest floods registered at Inglewood (1956, 1976 and 1971). The shorter record decreases the reliability of the FFA; nevertheless, the DEA reconciles relatively well to the gauged data probability distribution. Reconciliation of both gauges was achieved using an initial loss of 30 mm, continuing loss of 2.5 mm/hr and median VBF of 22,000 ML for Coolmunda Dam.

MACINTYRE BROOK TO BOOBA SANDS – RECONCILIATION

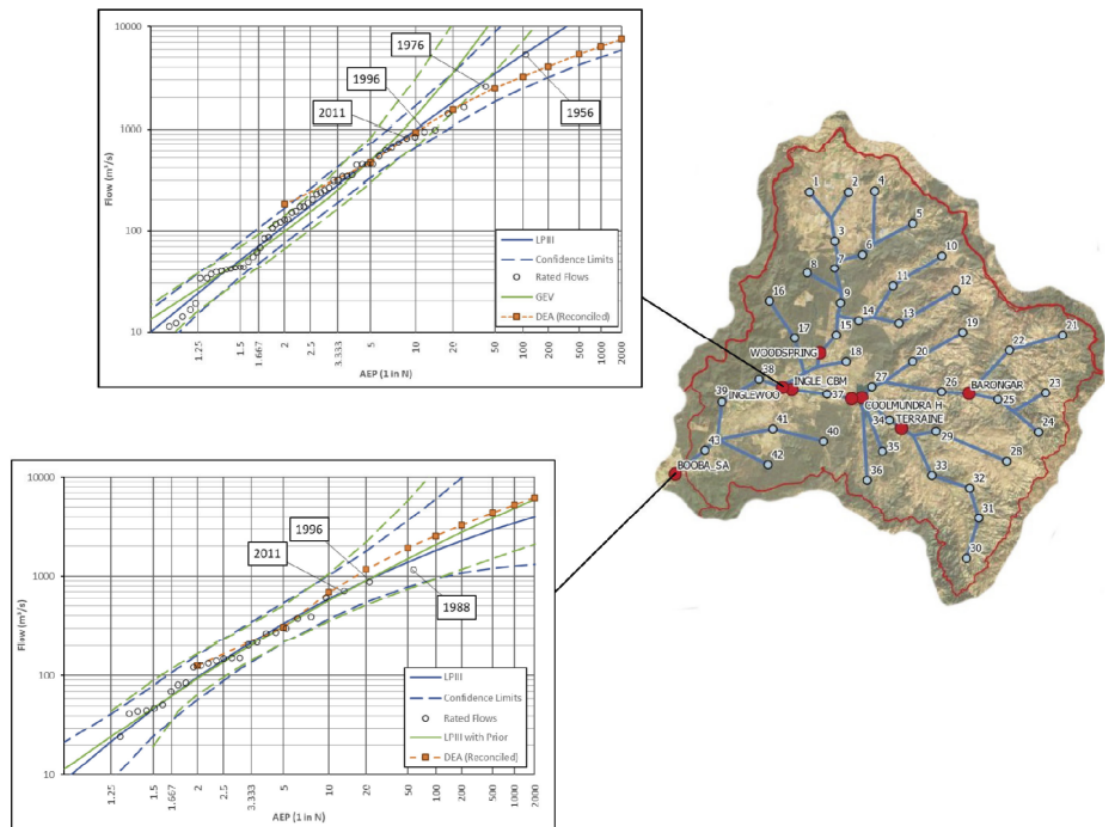


Figure 17.9 Macintyre Brook to Booba Sands –Reconciliation

17.2.7.7 Ottleys Creek

Coolatai is currently the only stream gauge in the Ottleys Creek catchment. The gauge has been operational since 1979 (43 years of data; however, review of the data from the WaterNSW webservice indicates that the gauge rating has been quite variable in this time. Photographs of the gauge site available on the WaterNSW webservice suggest that this may be due to scour damage circumventing the control weir, as shown in Figure 17.10. These issues reduce the reliability of the gauged flows and, consequently, the FFA results. Nevertheless, a reasonable reconciliation to frequent events can be achieved using an initial loss of 30 mm and a continuing loss of 3.5 mm/hr, as shown in Figure 17.11.

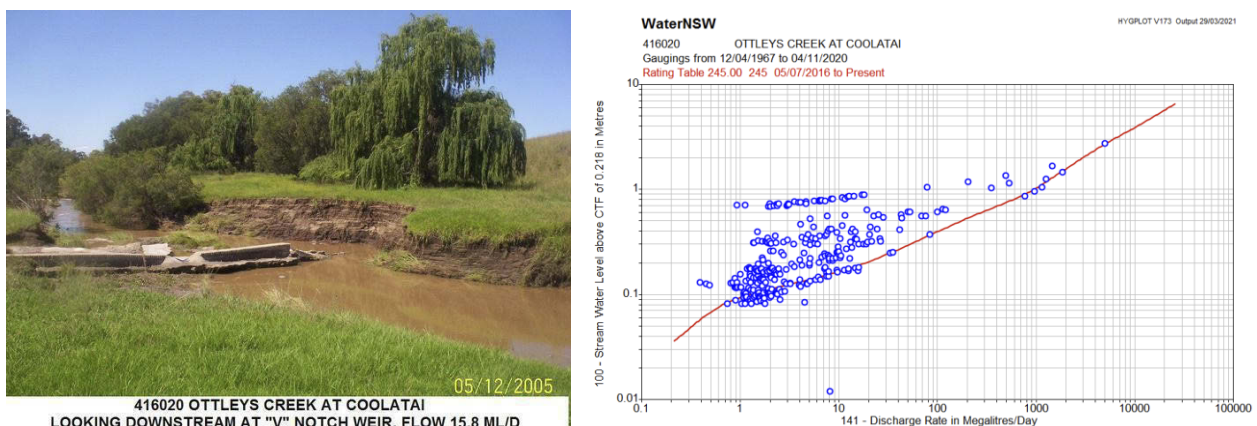


Figure 17.10 Control weir and comparison of flow gauging and current gauge rating at Coolatai

OTTLEYS CREEK AND LOWER MACINTYRE TO GOONDIWINDI – RECONCILIATION

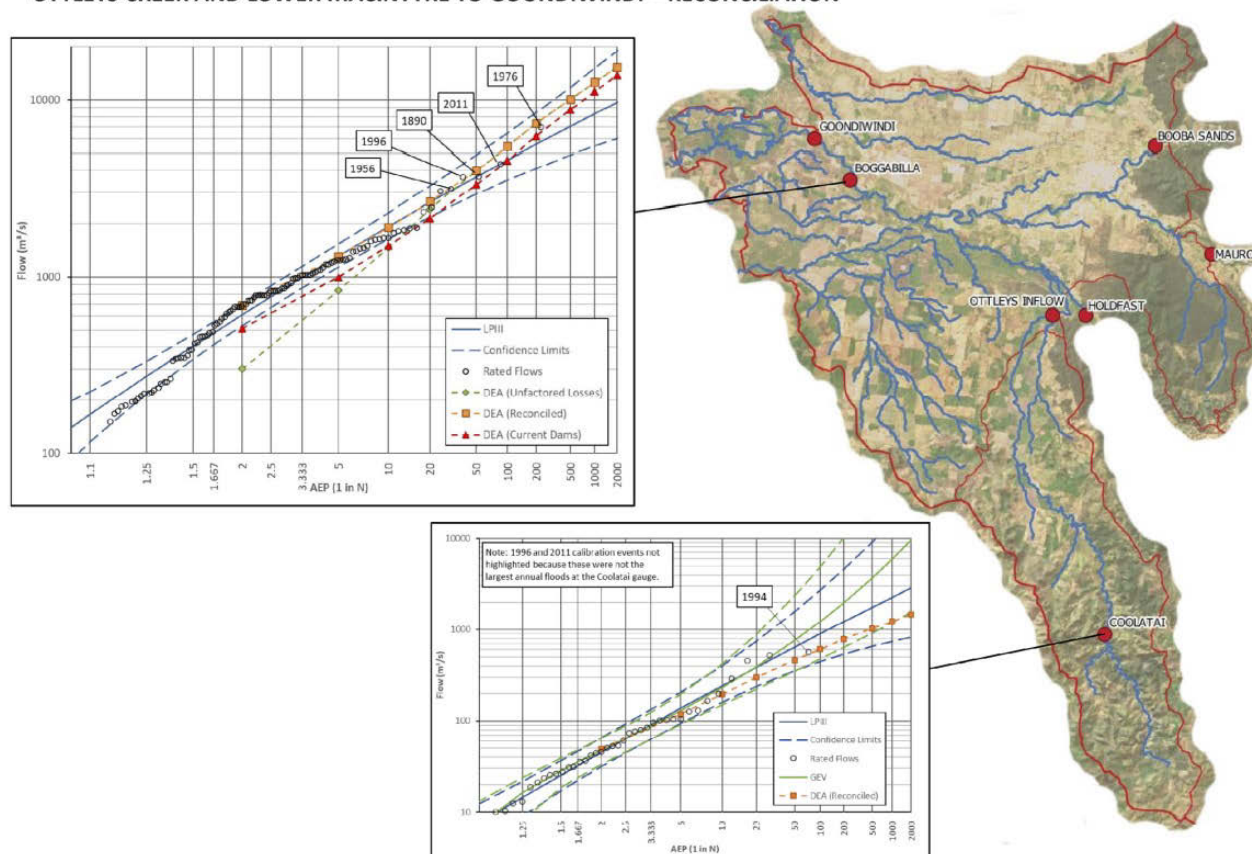


Figure 17.11 Ottleys Creek to Goondiwindi—Reconciliation

17.2.7.8 Lower Macintyre River

Flood frequency analysis and DEA reconciliation was conducted at the Boggabilla stream gauge, shown in Figure 17.11. The Boggabilla gauge record extends back to 1886, although continuous gauge data is only available since 1982. The gauge site has changed location several times; however, review of historical stream gauging information suggests that this has had little impact on the flow rating. Nevertheless, the site is complicated by significant flow breakout into the Whalan Creek and Morella Watercourse systems upstream of the gauge site, and there is significant uncertainty as to how much (if any) of this breakout has been included in the official historical flow ratings at the gauge. Rated flows used in the current FFA were calculated using the rating for total catchment flow. This is considered to give the most reliable estimate of total flows generated by the catchment, and consistency with the current hydrologic and hydraulic models.

The historical flood record is also non-homogeneous, being affected by the construction of Coolmunda Dam, Glenlyon Dam, the original Pindari Dams between 1968 and 1971, and the later significant upgrade of Pindari Dam in 1994. The reconciled DEA flows have therefore been calculated using a weighted average of three stages of catchment condition—no dams, with Coolmunda and Glenlyon Dams only (neglecting minor impact of the original Pindari Dam), and current dam conditions.

Reconciliation of the DEA flows was initially conducted using the catchment parameters derived for each of the independent catchments; however, these produced design flows for the frequent events that were significantly lower than the FFA, as shown in Figure C11 in Appendix C. Similarly, the DEA total catchment flows for the 50% (1 in 2) AEP event were lower than the reconciled DEA flows for the individual Macintyre River and Dumaresq River tributaries at Holdfast or Glenarvon (calculated using the ARF to those locations).

An issue potentially contributing to the flow underestimate for frequent events is the use of a uniform AEP rainfall distribution. The ARF is intended to maintain AEP neutrality of the average rainfall depth across a large catchment (compared to point rainfall data); however, in reality, rainfall would usually tend to have localised concentrations over parts of the catchment. When the average rainfall depth is of similar magnitude to the losses, the rainfall excess of concentrated bursts will be greater than the rainfall excess of a uniform pattern, noting that this applies to both concentrations in space and within the time interval of a temporal pattern. Figure 17.12 illustrates this principle, assuming two constant areas/intervals. The larger catchment area both reduces the average intensity and increases the probability that the rainfall would not be uniform. The analysis discussed demonstrates that Macintyre River flood events at Boggabilla are frequently the result of flows from only part of the catchment.

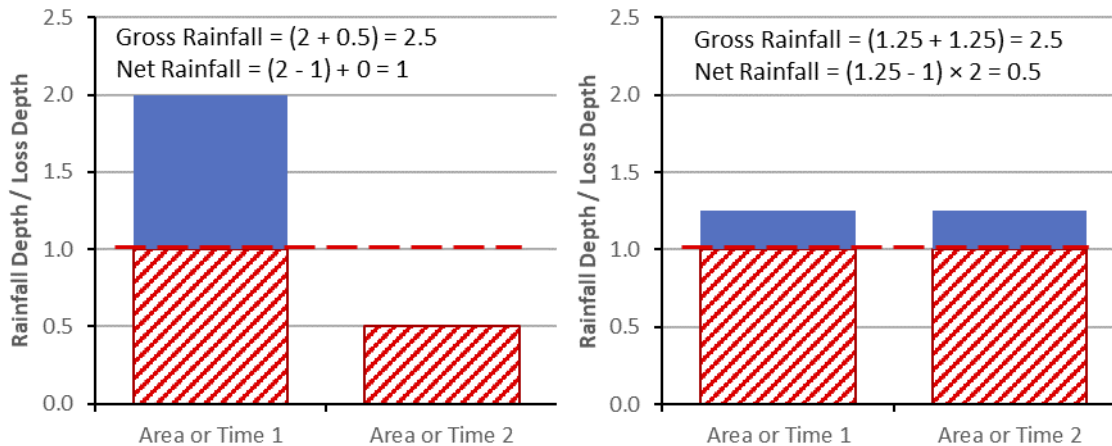


Figure 17.12 Macintyre River catchment—Effect of losses on concentrated versus uniform rainfall

In order to maintain probability-neutral loss values and reconcile the DEA flows to the historical record, the loss values were decreased for events more frequent than the 1 in 20 AEP event. The significant increase in the 1 in 2 AEP design flow (130 per cent increase for an 18 per cent reduction in losses) highlights just how sensitive the frequent-event DEA results are to the adopted losses. The reconciled and current catchment condition DEA flows are summarised in Figure C11 in Appendix C and Table 17.11.

Table 17.11 Reconciled and current catchment design flows upstream of Boggabilla

AEP (1 in N)	Loss multiplier	DEA flow (m³/s)	
		Reconciled	Current
2	0.82	691	511
5	0.85	1,309	995
10	0.90	1,906	1,506
20	0.95	2,648	2,145
50	1.0	4,002	3,322
100	1.0	5,445	4,565
200	1.0	7,317	6,208
500	1.0	10,091	8,785
1,000	1.0	12,547	11,147
2,000	1.0	15,261	13,767

17.2.7.9 Limitations of the design event approach

The DEA modelling was conducted using the procedures recommended in ARR (2019). One of the most significant limitations of the standard DEA methodology is the adoption of uniform catchment properties. Rainfall depth with constant AEP is applied across the catchment, reduced uniformly with the same ARF. An ensemble of TPs is assessed to help minimise the sensitivity to the distribution; however, this is still assumed to be distributed coincidentally across the entire catchment. As discussed in previous sections of this report, a fundamental aspect of the DEA modelling is the requirement to adopt probability-neutral catchment parameters. This includes the losses (on which the effect of the uniform rainfall distribution is discussed in Section 17.2.5.2) and also the reservoirs, for which a median VBF was adopted for all events.

Figure 17.13 to Figure 17.15 show the relationship between the peak Macintyre River flow upstream of Boggabilla and the peak flow in the tributaries at Holdfast, Bonshaw and Inglewood. The blue circles represent all the modelled design events (1-in-2 to 1-in-2,000 AEP, 12-hour to 120-hour durations). These demonstrate a distinct relationship between the tributary and combined flows, with the duration, TP and other variables having relatively little influence. The yellow circles show the gauged flows of historical flood events since 1972 (based on availability of gauge data). These demonstrate that there is significant variation in the possible distribution of tributary inflows for historical events—significant flood events ($>1,000 \text{ m}^3/\text{s}$) can occur with negligible inflow from one or more of the tributaries, or conversely peak flow from a tributary may even exceed the peak flow downstream. Looking at the largest flood events, the 1976 flood event is relatively evenly distributed across the catchment; however, the 2011 flood was strongly concentrated in the Dumaresq River while the recent 2021 flood had little inflow from Macintyre Brook. This assessment considers only the peak-flow relationship, and there may also be differences in the relative timing of the peaks.

Although the DEA flow estimates theoretically return the median (or at least some characteristic) relationship between the contribution of the different tributaries for a given AEP, they do not capture the spatial or temporal variability of real flood events.

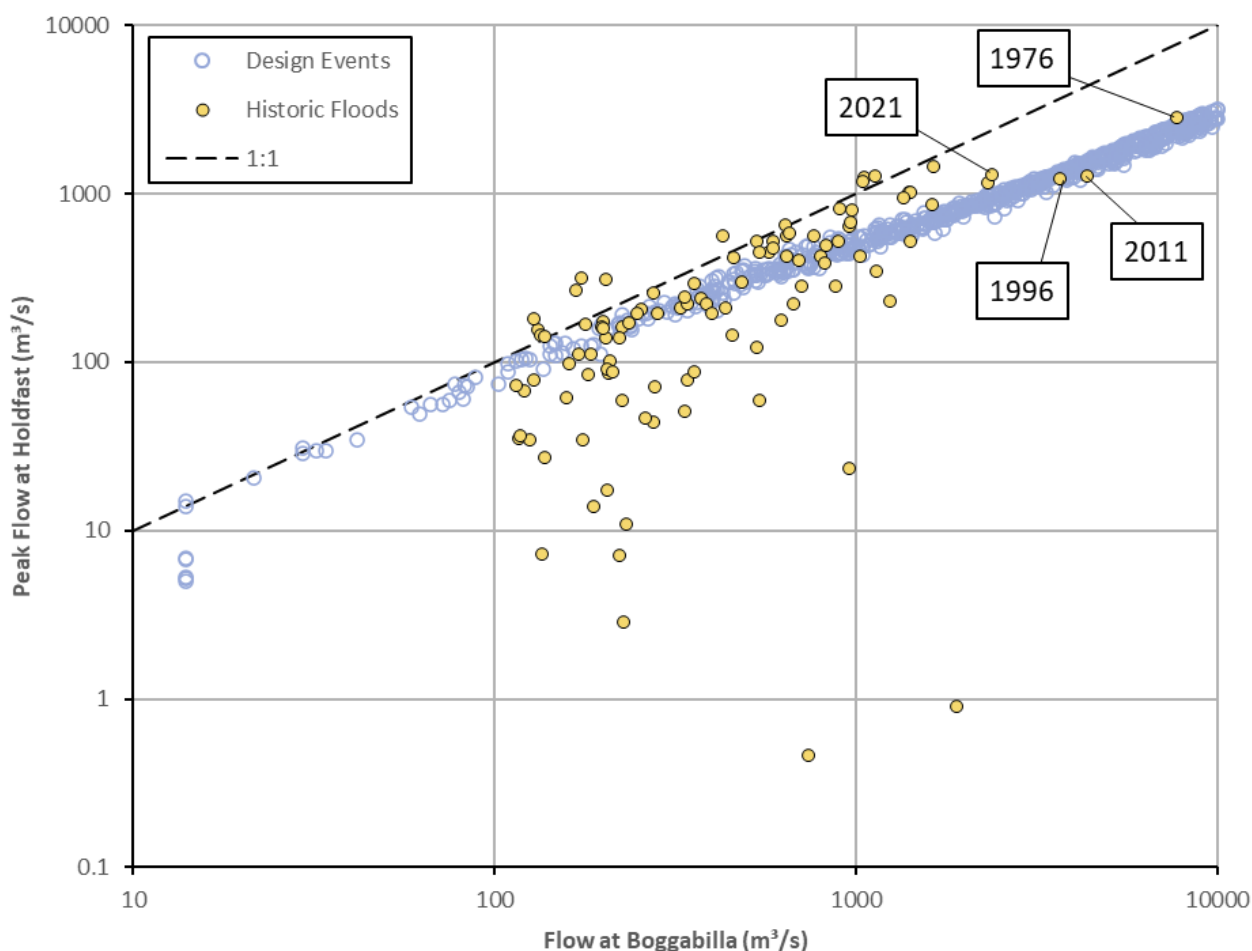


Figure 17.13 Relationship between total flow upstream of Boggabilla and Macintyre River flow at Holdfast

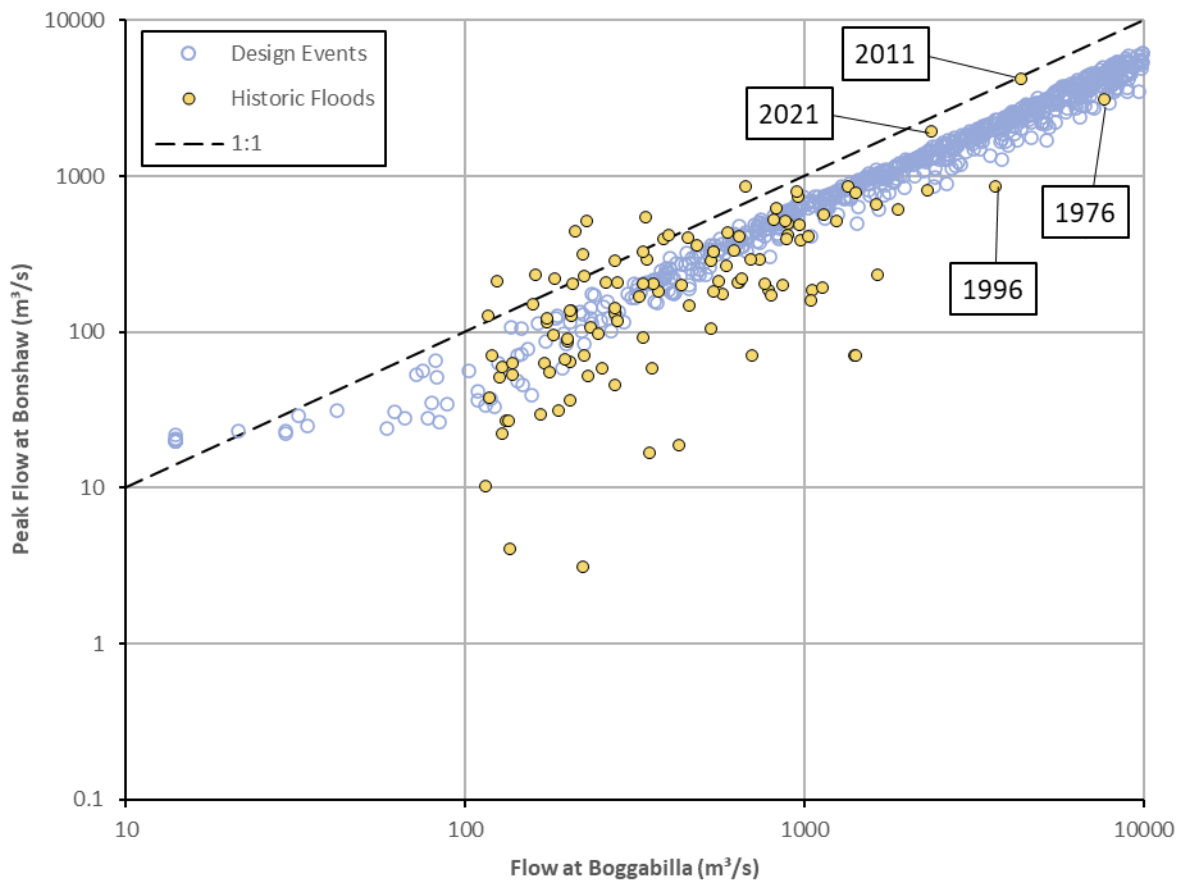


Figure 17.14 Relationship between total flow upstream of Boggabilla and Dumaresq River flow at Bonshaw

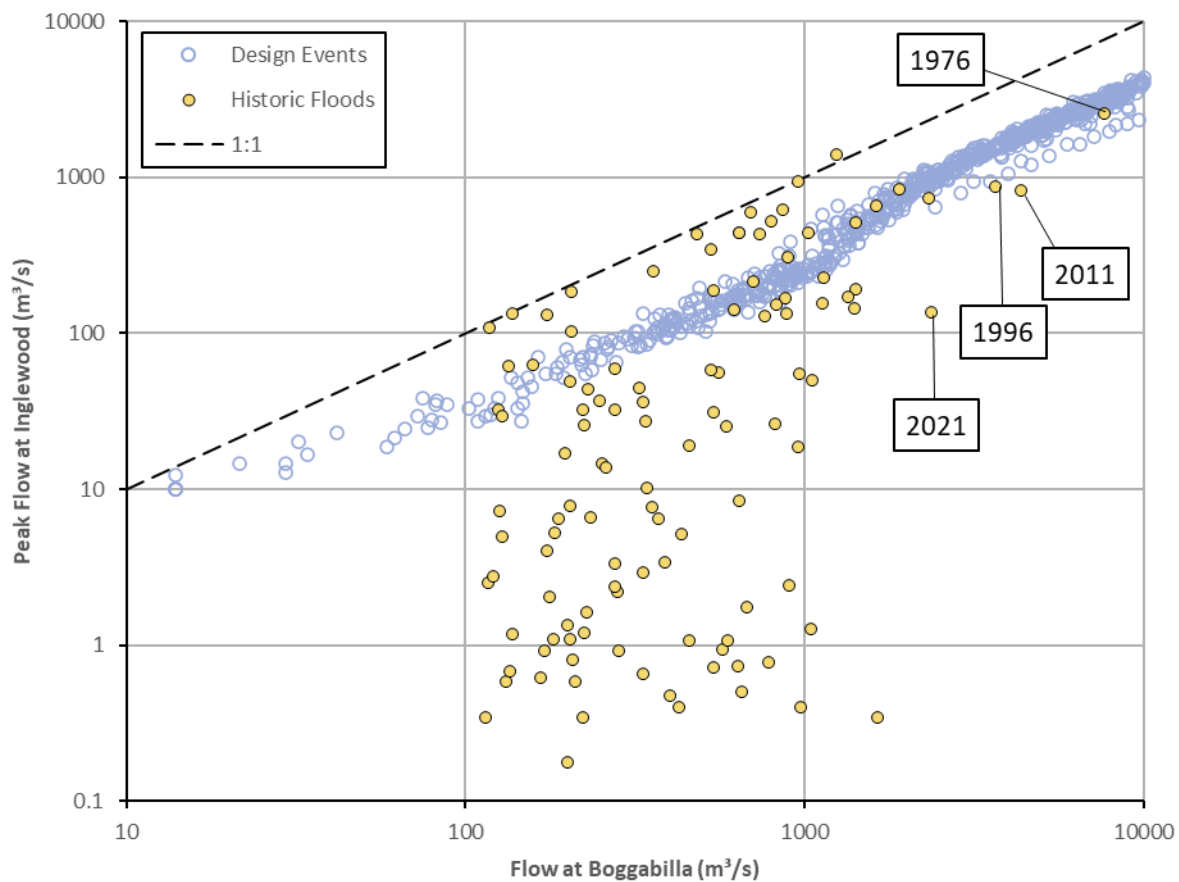


Figure 17.15 Relationship between total flow upstream of Boggabilla and Macintyre Brook flow at Inglewood

17.3 Hydraulic model development—Macintyre River

The following sections present the development of hydraulic model for the NS2B PIR that was used as the basis for the Project impact assessment.

17.3.1 Model setup

The TUFLOW hydraulic sub-model used in this report (developed from the BRVFMP hydraulic model) was adopted for the NS2B PIR. The hydraulic model boundary extends from upstream of the junction of the rivers to approximately 18 km downstream of Goondiwindi. The key updates to the hydraulic model are:

- Hydraulic model inflows—Figure M1-D in Volume 2—Appendix M presents the extents and key elements of the hydraulic model. Key changes adopted include:
 - Revised inflow locations for main upstream catchments consistent with the updated hydrologic model
 - Revised inflows from southern tributaries consistent with the updated hydrologic model
 - Inclusion of inflows from the lower floodplain area and Brigalow Creek from the updated hydrologic model.
- Advanced modelling techniques—in 2020 an updated version of the TUFLOW modelling software was released with a number of new applications that were able to be applied to the hydraulic modelling. These are:
 - SGS applied across the hydraulic models
 - QT analysis applied along the Project's permanent footprint.

Table 17.12 provides a summary of the hydraulic model setup.

Table 17.12 Macintyre River—TUFLOW hydraulic model setup overview

Parameter	Information
Completion date	August 2022
AEP's assessed	20%, 10%, 5%, 2%, 1%, 1 in 2,000, 1 in 10,000 AEP and PMF events
Hydraulic model build	TUFLOW HPC GPU – version 2020-10-AD-iSP-w64
Hydrologic modelling approach	URBS: Unified River Basin Simulator runoff-routing models which converts rainfall excess into flow
Hydraulic modelling approach	TUFLOW: 2-Dimensional numerical model
Grid size	30 m
SGS size	5 m
QT parameters	2 nd Order 3.75 m, 10 m upstream and downstream of the Project corridor
DEM (year flown)	2015, 2019
Roughness	Floodplain 1 – 0.08 Floodplain 2 (vegetated) – 0.12 River channel – 0.03
Eddy viscosity	Wu et. Al. 2005
Model calibration	1976, 1996, 2011
U/S model boundary	Flow verse time/SA
D/S model boundary	Normal depth
Internal boundaries	SA
Hydraulic mode timestep	Adaptive

Parameter	Information
Modelled scenarios	Existing Case, Developed Case
Sensitivity analysis	Climate change and blockage (0% and 50% blockage on culverts)

17.3.2 Model topography and grid size

The topography within the TUFLOW hydraulic model was based on 5 m DEM data collected during 2015 and 2019 which was supplied by ARTC. A 30 m grid size was selected to allow the features of the channels and floodplain to be represented with sufficient accuracy while maintaining efficient model run times. The 30 m x 30 m grid size is considered adequate to represent floodplain topography and is consistent with the approach typically adopted in similar flood studies. This was overlain with SGS at 5 m across the entire model.

Where necessary, the 2D model topography was modified so that the existing drainage flows, existing road/rail crowns and levee banks were represented accurately in the model.

For the Developed Case, the topography was modified to include the Project rail embankment, drainage structures and associated infrastructure.

Quadtree was applied north of the NSW/QLD boundary at 3.75 m grid spacing. The QT extent covers 10 m upstream and downstream of the Project corridor.

17.3.3 Existing hydraulic structures

A review of the existing drainage structures on the floodplain was undertaken using available information. The available information was limited to some initial survey of a number of cross-drainage structures along North Star Road, Bruxner Way, the existing non-operational rail line in NSW and the existing operational rail in Queensland. A number of these structures are located at the major creek crossings associated with the southern tributaries of the lower Macintyre River floodplain being Mobbindry Creek, Back Creek and Forest Creek. The following existing structure information was available from the initial survey capture:

- Mobbindry Creek:
 - A three-span bridge on North Star Road and five span bridge on the existing non-operational rail line. Invert/deck levels as well as opening/pier geometry information were not captured.
- Back Creek:
 - A four-span bridge along the existing non-operational rail line. Invert/deck levels and opening/pier geometry information were not captured
 - A set of five circular culverts under North Star Road. Invert levels and pipe diameter information were captured.
- Forest Creek:
 - A set of 33 box culverts under North Star Road. Invert levels and box geometry information were captured.

The bridges listed above were modelled as 2D 'Layered Flow Constriction Shapes' (2d_lfcsh) features in TUFLOW, which adopts form and blockage factors in order to represent flow conveyance and restriction through the structures. The culverts were represented as 1D 'Domain Network' (1d_nwk) features in TUFLOW, within which the culvert geometry and invert levels were incorporated along with conventional form losses.

These structures were included in the Existing Case hydraulic model. For the Developed Case, the structures on North Star Road were included and the existing structures under the non-operational rail line are removed as they are replaced by the revised reference design with associated new structures.

Several other cross-drainage structures were identified from available imagery covering the model area. For structures where only satellite imagery was available, these structures were represented as 'Z Shape Layers' (2d_zsh), which lower the digital terrain to provide appropriate flow conveyance. Some locations of known cross-drainage structures did not require additional representation in the model as they were already represented by Z Shape Layers of watercourses, such as along Whalan Creek. Any remaining existing structures not included in the hydraulic model are small with limited conveyance and located under roadways that would be substantially inundated during the flood events under consideration.

17.3.4 Roughness

The hydraulic roughness generally reflects the types of development and ground cover that exists within the hydraulic model extent. The distribution of roughness categories adopted for this assessment was based on the NSW DPE BRVFMP TUFLOW model, aerial imagery and confirmed during the site inspection. Specific roughness values applied to the model are detailed in Table 17.13.

Table 17.13 Macintyre River—Adopted roughness values

Category	Manning's 'n' value
Floodplain (Floodplain 1)	0.08
Vegetated floodplain (Floodplain 2)	0.12
Waterway (Floodplain 3)	0.03

17.3.5 Boundary conditions

17.3.5.1 External and internal boundary conditions

The inflow boundary and internal source boundaries were based on results of the hydrologic modelling. A normal depth boundary was applied for the downstream outflow model boundary. It was confirmed that it was a sufficient distance away from the area of interest so as not to impact on modelling results.

Delineated sub-catchments in the hydrologic model were used as internal inflow boundaries within the hydraulic model extent. The modelled flows in the hydrologic model were applied as SA polygons within each sub-catchment. Where sub-catchments were within unlimited height levees, the flow was applied to the low point outside of the levee. Figure M1-D in Volume 2 – Appendix M shows the hydraulic model boundaries.

17.3.6 Calibration

Hydrologic models are based on simplistic empirical runoff routing equations using coefficients determined primarily by calibration to a specific point of interest. By contrast, hydraulic models are more physically based, providing a (relatively) realistic representation of the catchment geometry and solving equations of motion within the model domain. Some differences between the hydrologic and hydraulic routing must realistically be expected; nevertheless, the hydraulic model should closely replicate the flow characteristics (attenuation, timing, etc.) that in the hydrologic model were validated by calibration to historical flood events.

The hydraulic model must also produce flood levels consistent with the flows. This can be confirmed by comparison with flood levels recorded during historical flood events, although the reliability is dependent on the accuracy of the modelled flows, which are in turn dependent on the accuracy of the recorded rainfall. Further validation across a wide range of flows can be achieved by comparison of the modelled level-flow relationships at the stream gauge sites with the gauge ratings, which allows the level-flow relationship to be confirmed without necessarily having to exactly match a specific flow.

The TUFLOW hydraulic model was validated using historical events. The primary objectives of the calibration process were:

- To confirm hydraulic model roughness factors required to match level-flow relationships at the stream gauges, particularly those where the ratings are well defined by in-streamflow measurements
- To confirm that the flood routing through the TUFLOW hydraulic model reasonably matches the hydrologic model (TUFLOW physically represents storage and other catchment characteristics that are represented in hydrology software by empirical coefficients) and that the adopted roughness parameters do not adversely affect the timing or attenuation of the flood routing.

The historical events were selected to represent a range of magnitudes and duration. The hydraulic sub-model was run using the updated inflows from the hydrologic model as detailed in Section 17.2 for each of the three historical calibration events (1976, 1996 and 2011). An iterative joint calibration process was undertaken between the hydrologic and hydraulic models to achieve the best overall match to the available historical flood data across the three historical flood events.

A number of the hydraulic model parameters were adjusted as part of the calibration process including:

- Roughness values
- Definition of waterway channel invert levels to improve in-channel conveyance
- Use of SGS to improve representation of conveyance
- Inclusion of flows from lower floodplain area, the southern tributaries and Brigalow Creek
- Variation of inflows from hydrologic model based on varying hydrologic model calibration outcomes.

The following datasets were used to assess the calibration of the hydraulic model:

- Data from Boggabilla (416002) and Goondiwindi (416201A) stream gauges, including:
 - Recorded peak water levels and estimated flows
 - Match in shape and timing of recorded flood hydrographs
 - Comparison against the level-flow relationships at the stream gauges.
- Recorded flood markers across the floodplain for each event – these markers come from a range of sources and were collated from previous studies and/or survey of property
- Comparison against the BRVFMP calibration outcomes
- Anecdotal flood information, including flood markers, for the historical flood events collected from many sources including:
 - Previous studies
 - NSW DPE
 - Landowners and stakeholders including GRC, Gwydir Shire Council and Moree Plains Regional Council.

Anecdotal data includes information obtained from a wide range of sources and, as such, it is of varying levels of accuracy and reliability. The anecdotal data was used to assess the performance of the hydraulic model to replicate historical flood conditions.

The following sections detail the outcomes of the hydraulic model calibration for each of the historical flood events. These details cover the entire floodplain (Queensland and NSW) as the majority of calibration data is in NSW.

17.3.6.1 1976 Historical flood markers

There were 38 recorded flood markers provided by NSW DPE or extracted from the *Goondiwindi Environs Flooding Investigation* (Cardno Lawson Treloar, 2007) within the floodplain area for the 1976 flood event. A further flood marker was surveyed by ARTC on a landholder's property in 2020 (ID 76-39).

A comparison of the peak water levels from the hydraulic model against the recorded flood markers is presented in Figure M2-A in Volume 2 – Appendix M and summarised in Table 17.14.

Table 17.14 Macintyre River—1976 event flood marker comparison

Location	Source	Recorded level at flood marker (mAHD)	Modelled level (mAHD)	Difference (m)
76-01	Border Rivers Valley Floodplain Management Plan	214.60	214.40	-0.20
76-02		213.80	213.11	-0.69
76-03		210.50 ^{*1}	210.16	-0.34
76-04		224.72	225.26	0.54
76-05		218.10	218.35	0.25
76-06		224.93	225.25	0.32
76-07		220.12	220.19	0.07
76-08		223.62	223.74	0.12
76-09		224.96	225.01	0.05
76-10		217.90	217.89	-0.01
76-11		224.72	224.71	-0.01
76-12		223.68	223.54	-0.14
76-13		223.31	223.07	-0.24
76-14		222.39	222.28	-0.12
76-15		222.32	222.13	-0.19
76-16		224.96	224.72	-0.24
76-17		219.60	219.10	-0.50
76-18		224.63	224.31	-0.32
76-19		226.51 ¹	225.91	-0.60
76-20		227.33 ¹	226.71	-0.62
76-21		224.26	223.70	-0.56
76-22		226.10 ¹	225.38	-0.72
76-23		226.92 ¹	226.09	-0.83
76-24		217.78	216.94	-0.84
76-25		218.85 ¹	218.37	-0.49
76-26		208.87 ¹	207.77	-1.10
76-27		207.42 ¹	Dry	-
76-28		217.78	217.03	-0.75
76-29		217.05	217.01	-0.04
76-30		212.60 ¹	212.32	-0.28
76-31		213.00 ¹	212.29	-0.71
76-32		212.40	211.94	-0.46
76-33		215.88	216.04	0.16
76-34		213.00	212.04	-0.96

Location	Source	Recorded level at flood marker (mAHD)	Modelled level (mAHD)	Difference (m)
76-35		210.56 ¹	210.94	0.38
76-36		219.88	220.16	0.28
76-37	Goondiwindi Environs Flooding Investigation	218.06	218.34	0.28
76-38		209.06 ¹	Dry	-
76-39	Field survey on property	234.15	235.28	0.13

Table note:

¹ Outliers—inconsistent with adjacent flood markers or near hydraulic model boundary

Given the size and the recognition of the 1976 event among the community, and its use in development control/planning, this event is a key calibration event. The calibration achieved is reasonable, as shown in Table 17.15, with 62 per cent of flood markers in the ± 0.3 m range. This considers the age of the event, the changes to the floodplain since this event, uncertainties regarding the quality of the flood marker data, and changes to stream gauges since the event occurred.

Table 17.15 Macintyre River—Summary of calibration against 1976 recorded flood markers

Model accuracy range (m)	1976 event	
	No of flood markers	% in range
Flooded but predicted to be dry	0	0%
<0.3	8	30%
-0.3 to -0.2	3	11%
-0.2 to -0.1	3	11%
-0.1 to 0	3	11%
0 to 0.1	2	7%
0.1 to 0.2	3	11%
0.2 to 0.3	3	11%
>0.3	2	7%
Outliers removed ¹	12	-
Total number of markers	39	-

Table note:

¹ Outliers—inconsistent with adjacent flood markers or near hydraulic model boundary

17.3.6.2 1996 historical flood marker comparison

There were eight recorded flood markers provided by NSW DPE or extracted from the *Goondiwindi Environs Flooding Investigation* (Cardno Lawson Treloar, 2007) for the 1996 flood event within the floodplain area. The location of the recorded flood markers is shown in Figure M2-B in Volume 2 – Appendix M. The recorded and predicted flood levels are presented in Table 17.16.

Table 17.16 Macintyre River—1996 flood marker comparison

Location	Source	Recorded level at flood marker (mAHD)	Modelled level (mAHD)	Difference (m)
96-01	NSW DPE—marks derived from high water marks on signposts, etc.	220.95	220.85	-0.10
96-02		219.23	219.23	0.00
96-03		218.73	218.64	-0.09
96-04		218.13	218.50	0.37

Location	Source	Recorded level at flood marker (mAHD)	Modelled level (mAHD)	Difference (m)
96-05	Goondiwindi Environs Flooding Investigation	215.73	215.65	-0.09
96-06		221.71	221.92	0.21
96-07	Border Rivers Valley Floodplain Management Plan	221.10 ¹	222.08	0.98
96-08	Goondiwindi Environs Flooding Investigation	215.04	215.17	0.13

Table note:

1 Outliers—inconsistent with adjacent flood markers or near hydraulic model boundary

The revised hydraulic model flood levels generally compare well to the 1996 recorded flood heights, with six of the eight points within 0.3 m of the recorded heights. Table 17.17 presents a summary of the accuracy of the model. For the 1996 event, 85 per cent of the flood markers lie within the ± 0.3 m range although this event has a reduced number of flood markers to compare against.

The recorded level 96-07 is 221.1 mAHD and located approximately 3 km upstream of the Boggabilla stream gauge which recorded a peak flood height of 221.03 mAHD; hence, the hydraulic model was unable to match this level (predicted level 222.08 mAHD). It is likely there is an error in this recorded flood level.

Table 17.17 Macintyre River—Summary of calibration against 1996 recorded flood markers

Model accuracy range (m)	1996 event	
	No of flood markers	% in range
Flooded but predicted to be dry	0	0%
<0.3	0	0%
-0.3 to -0.2	0	0%
-0.2 to -0.1	0	0%
-0.1 to 0	4	57%
0 to 0.1	0	0%
0.1 to 0.2	1	14%
0.2 to 0.3	1	14%
>0.3	1	14%
Outliers removed ¹	1	-
Total number of markers	8	-

Table note:

1 Outliers—inconsistent with adjacent flood markers or near hydraulic model boundary.

17.3.6.3 2011 historical flood marker comparison

For the 2011 historical flood event there were 52 historical flood markers available for comparison. These are summarised in Table 17.18 and presented in Figure M2-C in Volume 2 – Appendix M.

Table 17.18 Macintyre River—2011 flood marker comparison

Name	Source	Recorded level at flood marker (mAHD)	Modelled level (mAHD)	Difference (m)
11-01	NSW DPE—marks derived from high water marks on signposts, etc.	219.15	219.60	0.46
11-02		220.98	221.21	0.23
11-03		224.79 ¹	225.52	0.74
11-04		219.23	219.46	0.22

Name	Source	Recorded level at flood marker (mAHD)	Modelled level (mAHD)	Difference (m)
11-05		221.85	222.12	0.27
11-06		218.28 ¹	219.20	0.93
11-07		219.60	219.61	0.00
11-08		219.57	219.86	0.28
11-09		221.94	222.12	0.18
11-10		217.87	218.11	0.23
11-11		220.32 ¹	219.91	-0.41
11-12		220.41	220.30	-0.12
11-13		220.44	220.45	0.01
11-14		220.79	220.50	-0.30
11-15		220.85	220.84	-0.01
11-16		221.05	221.19	0.14
11-17		222.22	221.90	-0.31
11-18		221.12	221.19	0.07
11-19		223.96	224.00	0.04
11-20		220.79	220.50	-0.29
11-21		224.02	223.99	-0.02
11-22		224.06	223.95	-0.11
11-23		224.06	223.92	-0.14
11-24		224.06	223.96	-0.10
11-25		224.06	223.89	-0.17
11-26	NSW DPE—marks derived from high water marks on signposts, etc.	220.73	220.38	-0.36
11-27		228.27	228.14	-0.14
11-28		225.73	Dry	-
11-29		216.96	216.99	0.03
11-30		216.80	216.85	0.05
11-31		216.50	216.15	-0.34
11-32		216.34	216.07	-0.28
11-33		215.19	215.09	-0.11
11-34		214.89 ¹	214.49	-0.40
11-35		214.83 ¹	214.52	-0.31
11-36		217.53	217.30	-0.23
11-37		217.96	217.78	-0.18
11-38		218.55	218.33	-0.23
11-39		218.44	218.61	0.17
11-40		213.52 ¹	212.71	-0.82
11-41		213.73 ¹	212.71	-1.03
11-42		215.87	215.49	-0.38
11-43		218.49	218.40	-0.09
11-44		218.69	219.01	0.31
11-45		219.21	219.48	0.27

Name	Source	Recorded level at flood marker (mAHD)	Modelled level (mAHD)	Difference (m)
11-46		219.35	219.57	0.22
11-47		219.56	219.70	0.15
11-48		219.98	220.09	0.12
11-49		218.54	219.22	0.68
11-50		218.46	218.36	-0.10
11-51		217.13	217.67	0.55
11-52		216.18 ¹	212.71	-3.47

Table note:

1 Outliers—inconsistent with adjacent flood markers or near downstream model boundary

With 2011 being the most recent event on the floodplain, and most representative of current floodplain conditions, it was deemed important that this event be considered the primary calibration event for the modelling. As can be seen from Table 17.19, a good match is achieved with 79 per cent of flood markers within the ± 0.3 m range.

Table 17.19 Macintyre River—Summary of calibration against 2011 recorded flood markers

Model accuracy range (m)	2011 event	
	No of flood markers	% in range
Flooded but predicted to be dry	1	2%
<0.3	4	9%
-0.3 to -0.2	5	11%
-0.2 to -0.1	7	16%
-0.1 to 0	5	11%
0 to 0.1	6	14%
0.1 to 0.2	5	11%
0.2 to 0.3	7	16%
>0.3	4	9%
Outliers removed ¹	8	-
Total number of markers	52	-

Table note:

1 Outliers—inconsistent with adjacent flood markers or near hydraulic model boundary

17.3.6.4 Boggabilla stream gauge

The recorded and predicted flood levels and flows at the Boggabilla stream gauge are presented in Table 17.20. As discussed in Section 17.2, the current rating curve (used for 1996 and 2011 events) includes floodplain flows that break out into Whalan Creek and Morella Watercourse (i.e. represents flows across the wider floodplain upstream of Boggabilla). For comparison purposes, flows were extracted at two locations—being upstream (U/S) of Boggabilla to give the floodplain wide flow and downstream (D/S) of Boggabilla to give the flows at Boggabilla township—as presented in Table 17.20.

Table 17.20 Comparison of results at the Boggabilla stream gauge

Event	Recorded stream gauge data			TUFLOW results		
	Level (mAHD)	Flow u/s of Boggabilla	Flow d/s of Boggabilla	Level (mAHD)	Flow u/s of Boggabilla	Flow d/s of Boggabilla
1976	221.27	n/a ¹	3,700 m ³ /s (319,600 ML/d)	221.22 (-0.05 m)	n/a ¹	3,680 m ³ /s (317,952 ML/d)
1996	221.03	3,486 m ³ /s (301,200 ML/d)	2,485 m ³ /s ¹ (214,700 ML/d)	220.98 (-0.05 m)	3,470 m ³ /s (299,808 ML/d)	2,791 m ³ /s (241,142 ML/d)
2011	221.12	3,803 m ³ /s (328,600 ML/d)	n/a	221.11 (-0.01 m)	4,493 m ³ /s (388,195 ML/d)	3,197 m ³ /s (276,221 ML/d)

Table notes:

ML/d = Megalitres per day

1 1976 event rating curve only considered flows at Boggabilla and not the full floodplain

2 From flow measurement data

Figure 17.16 presents a comparison of the modelled flows and level information at Boggabilla for each of the three historical events against the rated data from the stream gauge (i.e. measured during events). The rising limbs of the modelled events are the lower lines, with the receding hydrograph limbs resulting in the high peak levels. As recordings are typically taken on the falling limb of flood events, the model results are a good match to the rated data.

For all three historical events, the hydraulic model results give a good representation of conditions at the Boggabilla stream gauge, with close matches in peak water levels, good matches in peak flows and hydrograph shapes and timing as demonstrated in Figure 17.17 to Figure 17.22. It should be noted that only a peak water level was recorded at Boggabilla in 1976 as shown in Figure 17.18. The flow hydrograph presented in therefore is an estimation only and not based on recorded data.

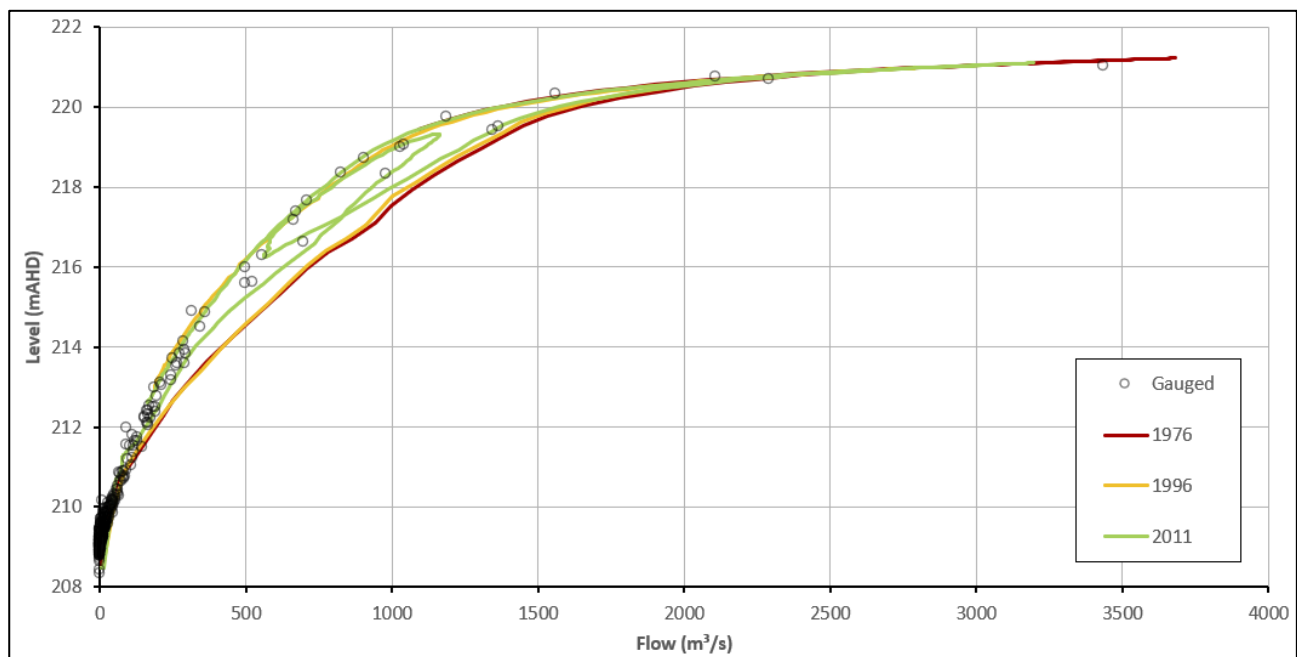


Figure 17.16 Boggabilla stream gauge—Model results compared to rated data

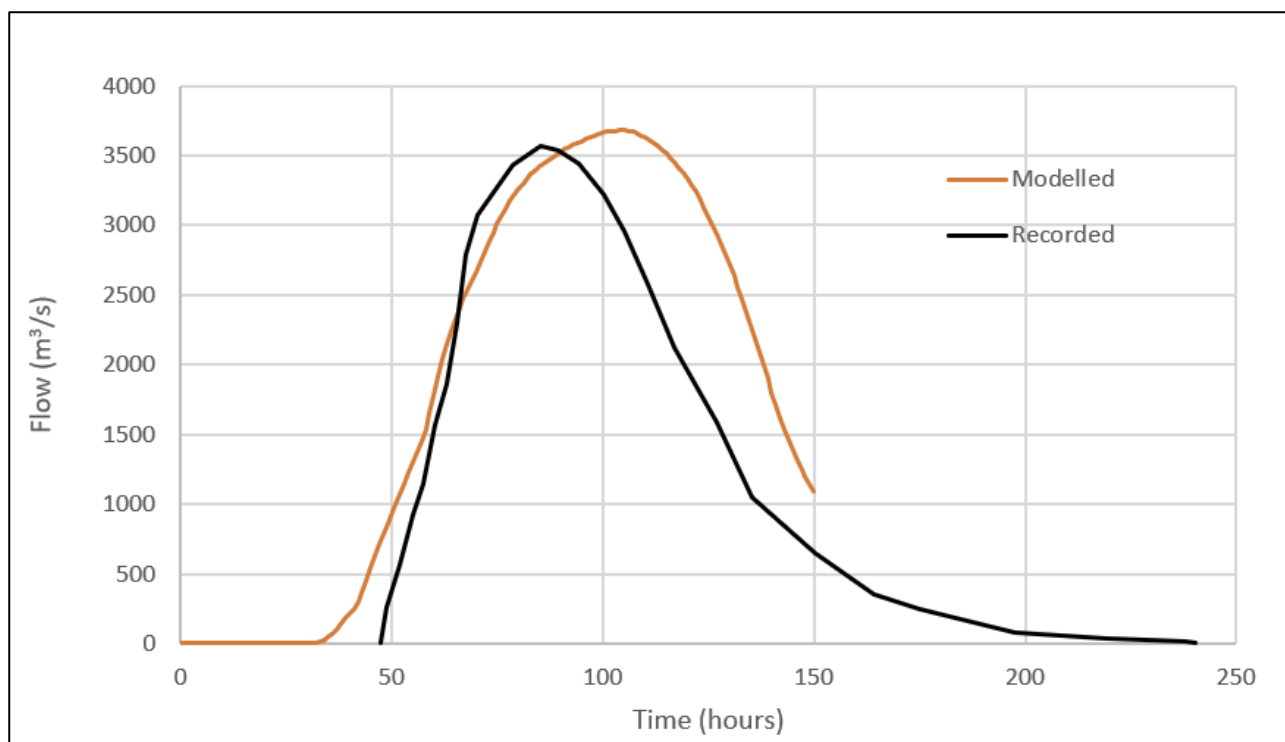


Figure 17.17 Boggabilla gauge—1976 flows recorded and modelled

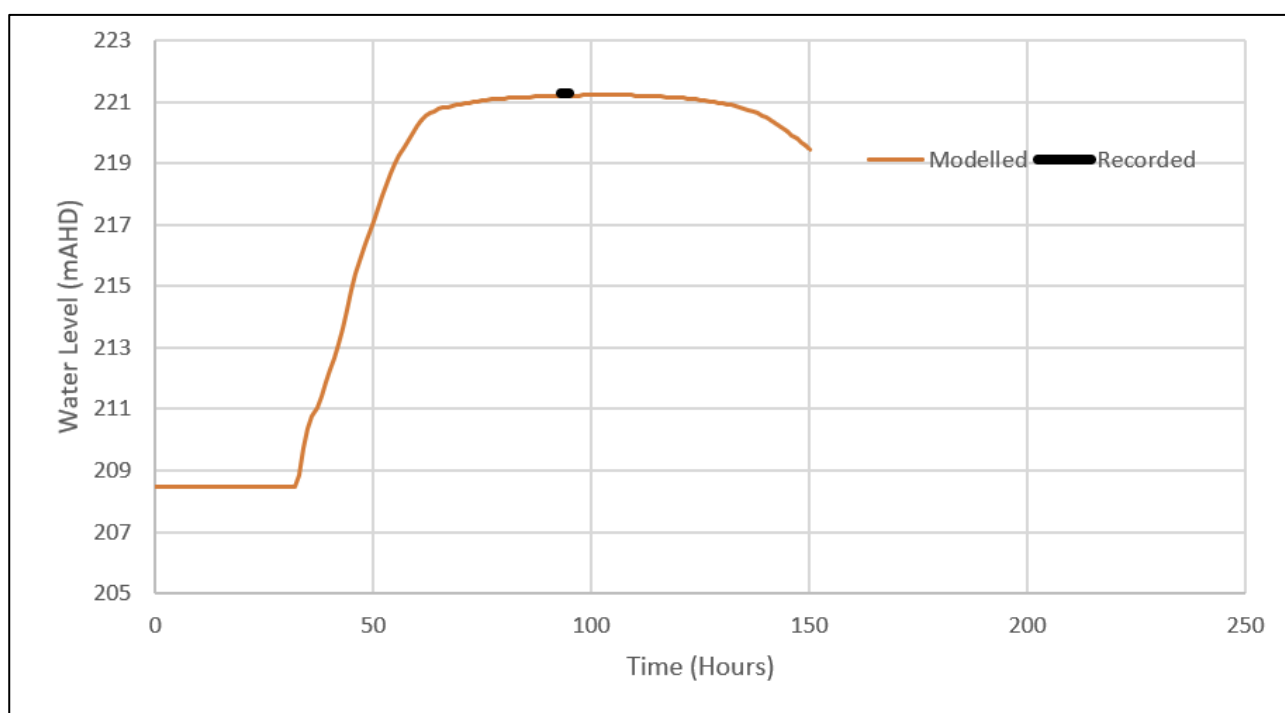


Figure 17.18 Boggabilla gauge—1976 levels recorded and modelled (time series not available for 1976 recorded level)

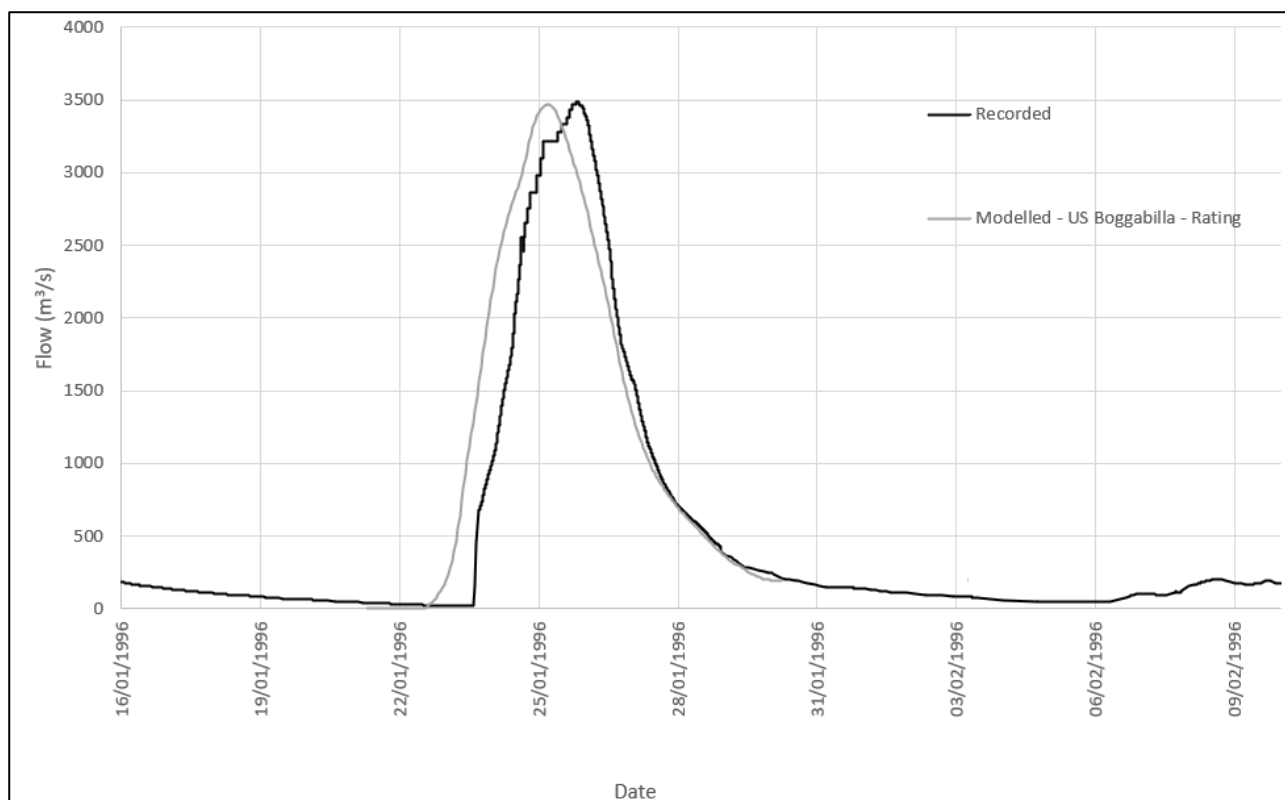


Figure 17.19 Boggabilla gauge—1996 flows recorded and modelled

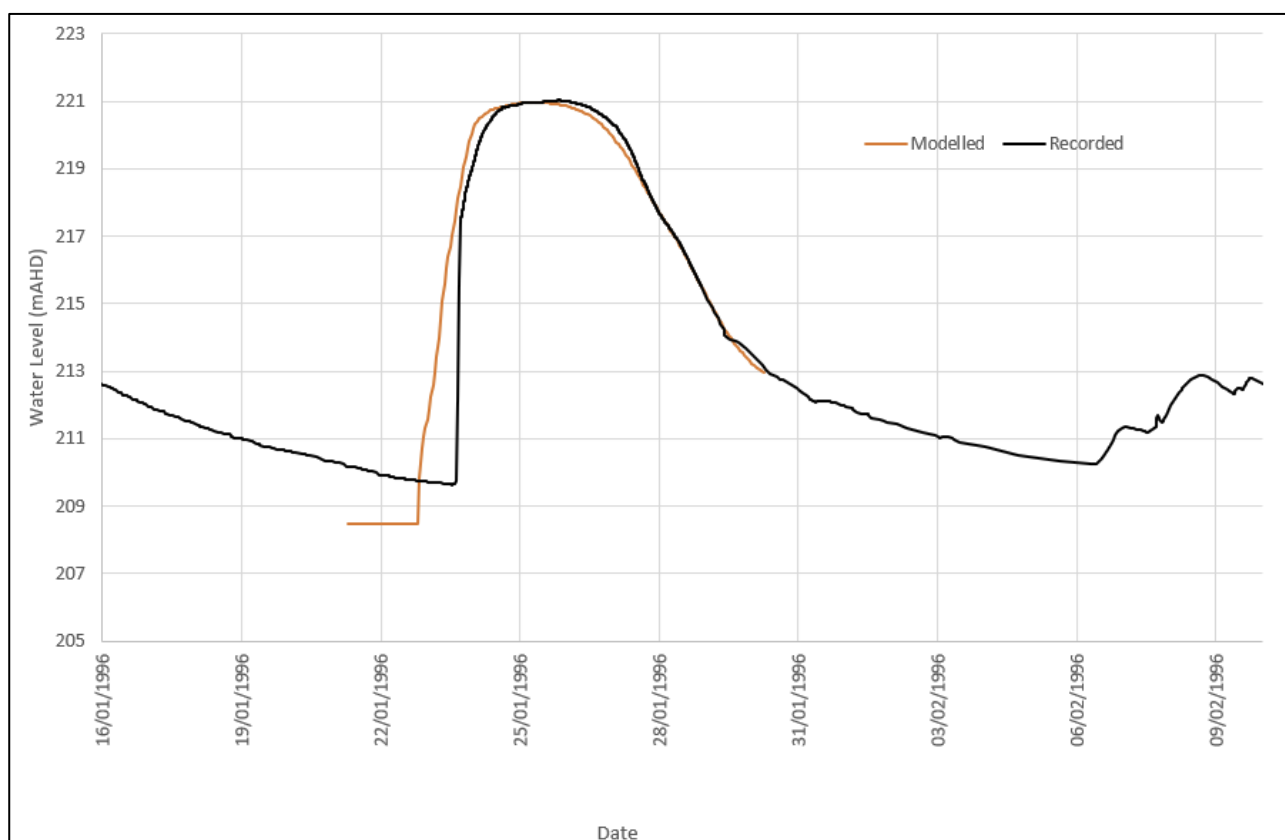


Figure 17.20 Boggabilla gauge—1996 levels recorded and modelled

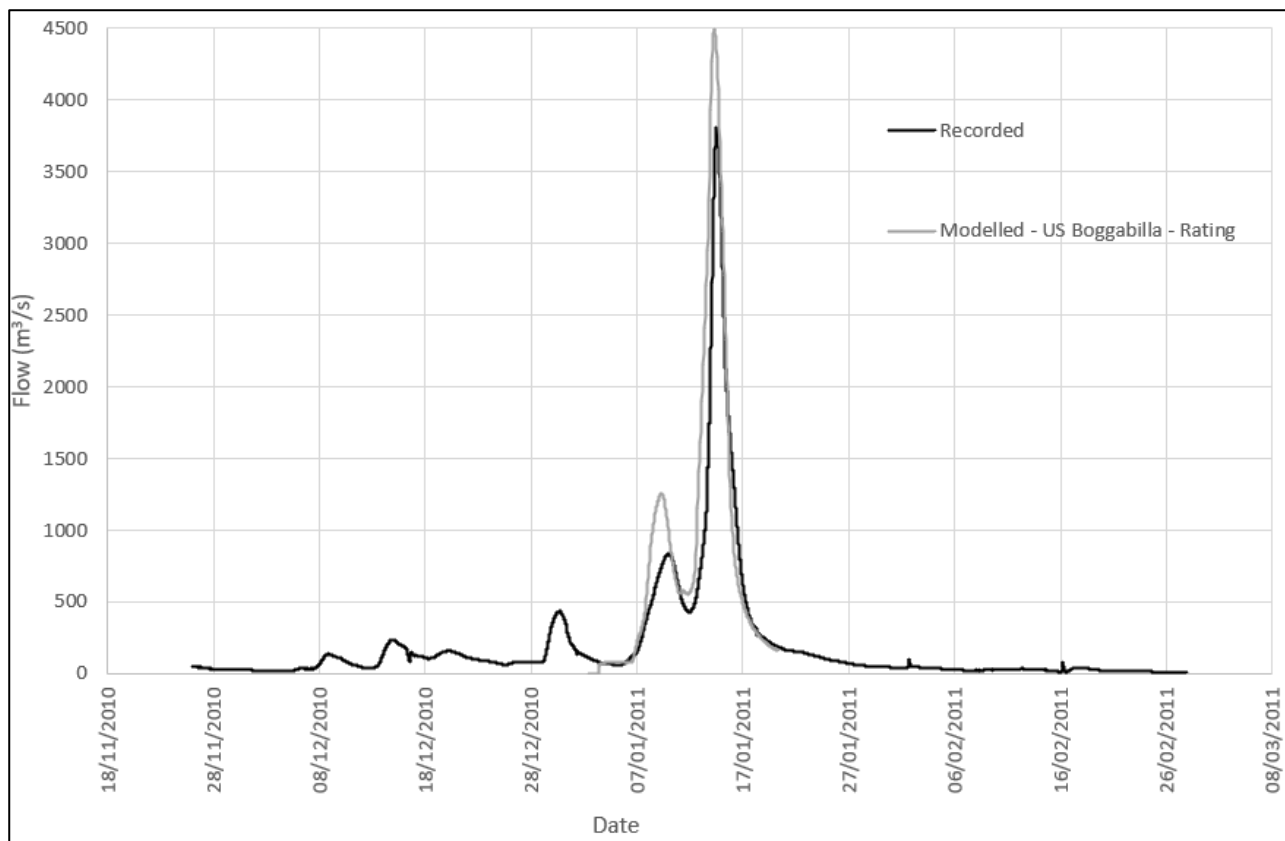


Figure 17.21 Boggabilla gauge—2011 flows recorded and modelled

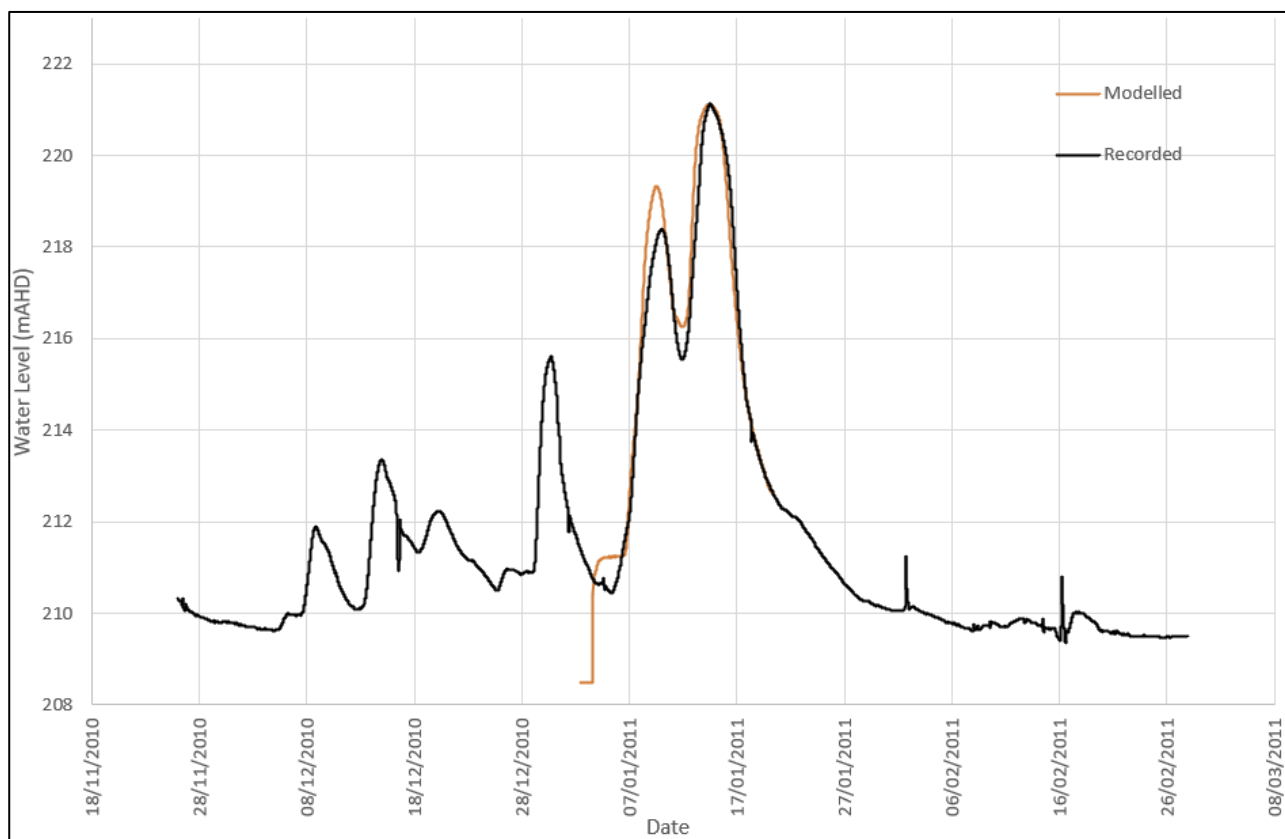


Figure 17.22 Boggabilla gauge—2011 levels recorded and modelled

17.3.6.5 Goondiwindi stream gauge

The recorded and predicted flood levels and flows at the Goondiwindi stream gauge are presented in Table 17.21.

Table 17.21 Comparison of results at the Goondiwindi stream gauge

Event	Recorded level (mAHD)	TUFLOW modelled level (mAHD)	Rated gauge flow (m ³ /s)	Rated gauge flow (ML/d)	TUFLOW modelled flow (m ³ /s)	TUFLOW modelled flow (ML/d)
1976	218.08	218.37 (+0.29)	1,560	134,784	2,128	183,859
1996	218.19	218.34 (+0.15)	1,767	152,669	1,987	171,677
2011	218.195	218.36 (+0.16)	1,767	152,669	2,128	183,859

Figure 17.23 presents a comparison of the modelled flows and level information at Goondiwindi for each of the three historical events against the rated data from the stream gauge (i.e. measured during events). The rising limbs of the modelled events are the lower lines with the receding hydrograph limbs resulting in the high peak levels. As recordings are typically taken on the falling limb of flood events, the model results are a good match to the rated data.

For all three historical events, the hydraulic model results give a good representation of conditions at the Boggabilla stream gauge. With good matches in peak water levels, peak flows, hydrograph shapes and timing as demonstrated in Figure 17.24 to Figure 17.29.

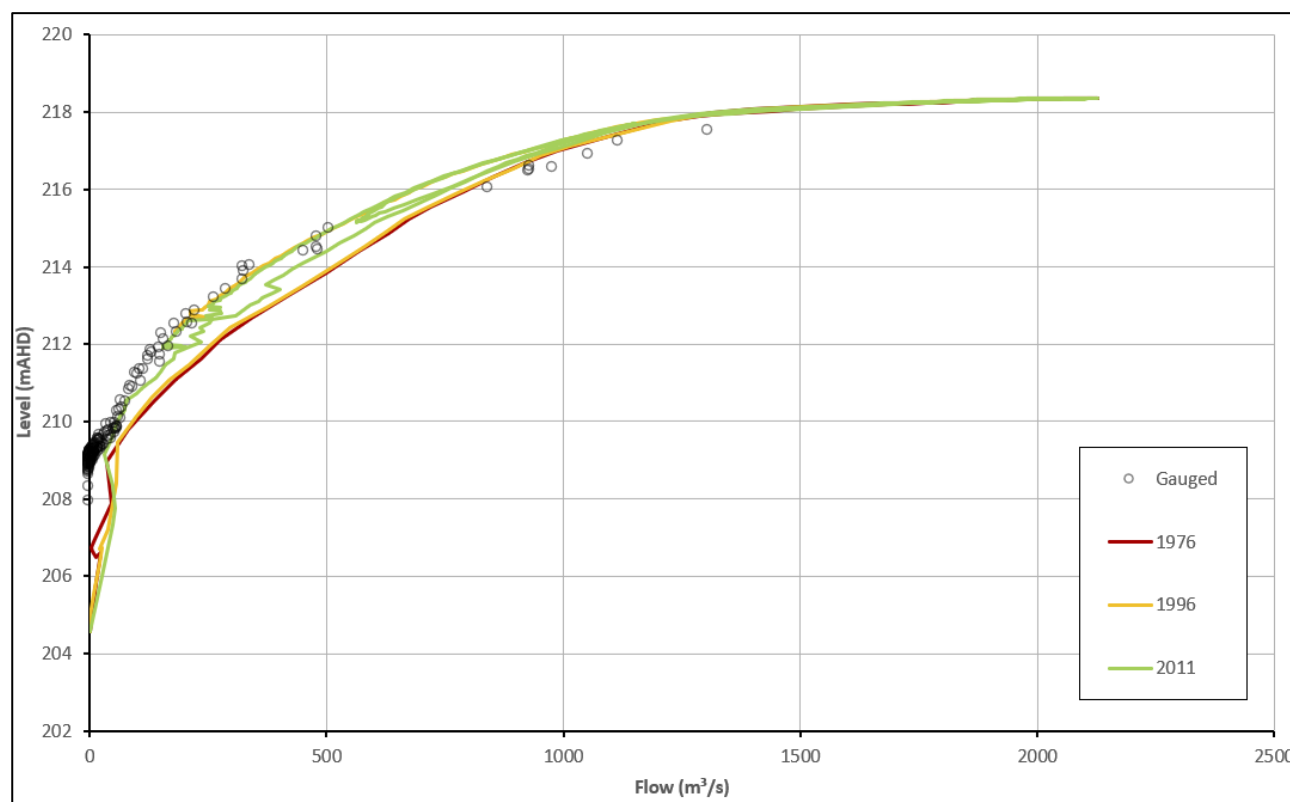


Figure 17.23 Goondiwindi stream gauge—Model results compared to rated data

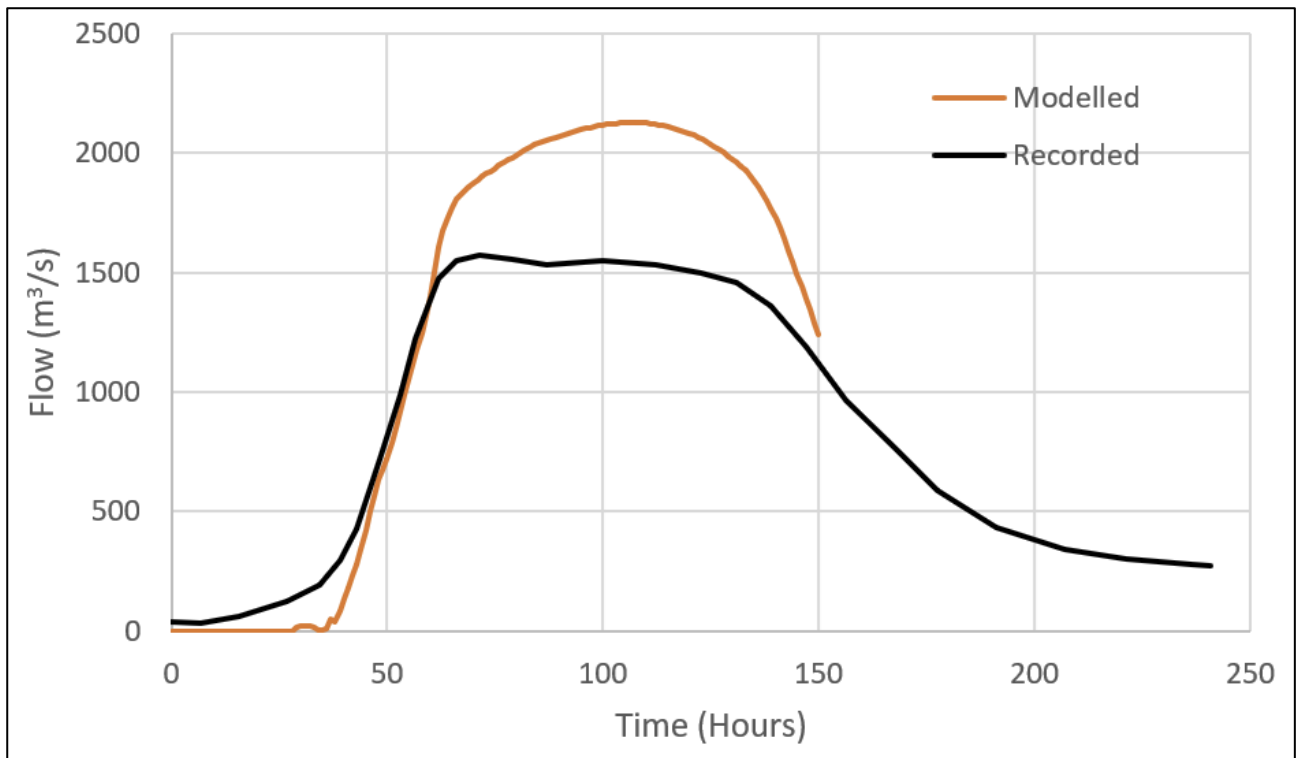


Figure 17.24 Goondiwindi gauge—1976 flows recorded and modelled

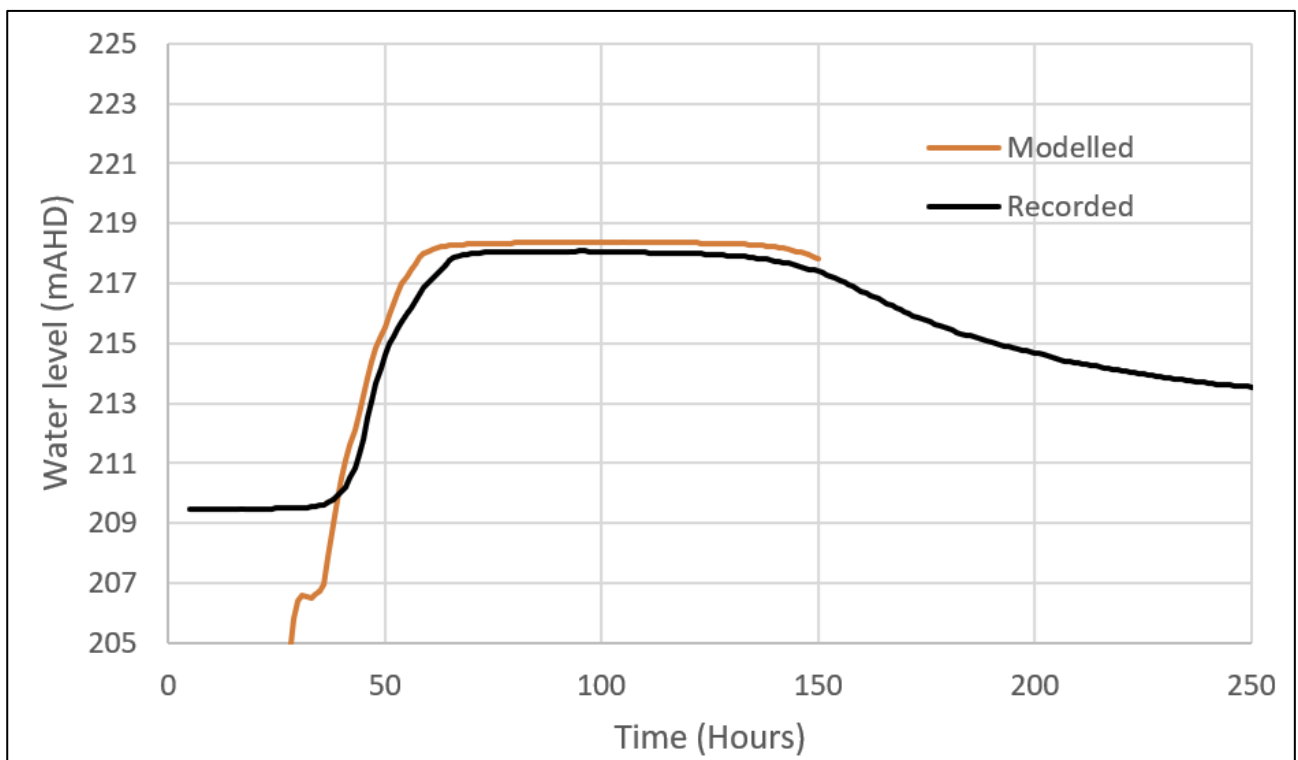


Figure 17.25 Goondiwindi gauge—1976 levels recorded and modelled

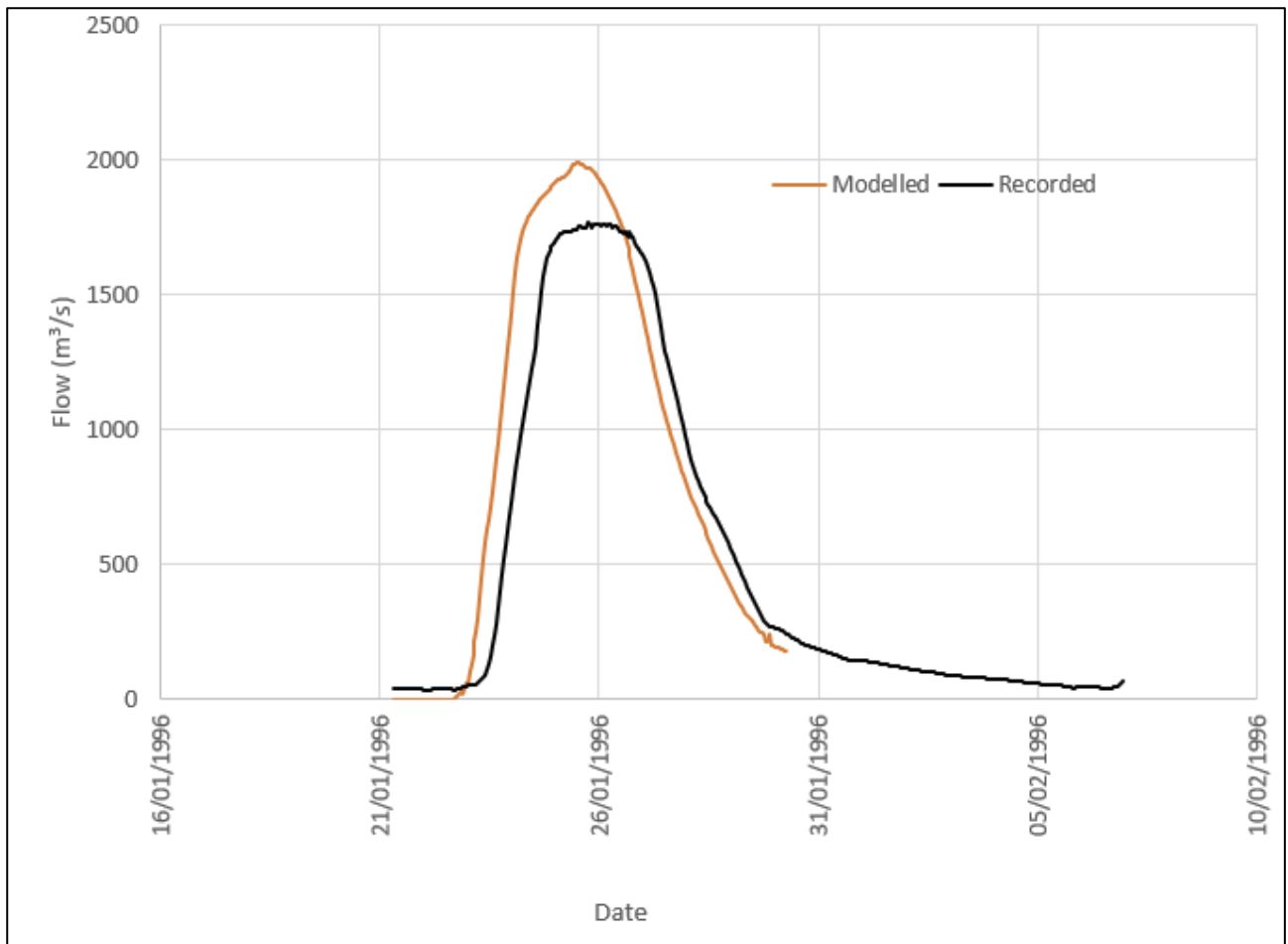


Figure 17.26 Goondiwindi gauge—1996 flows recorded and modelled

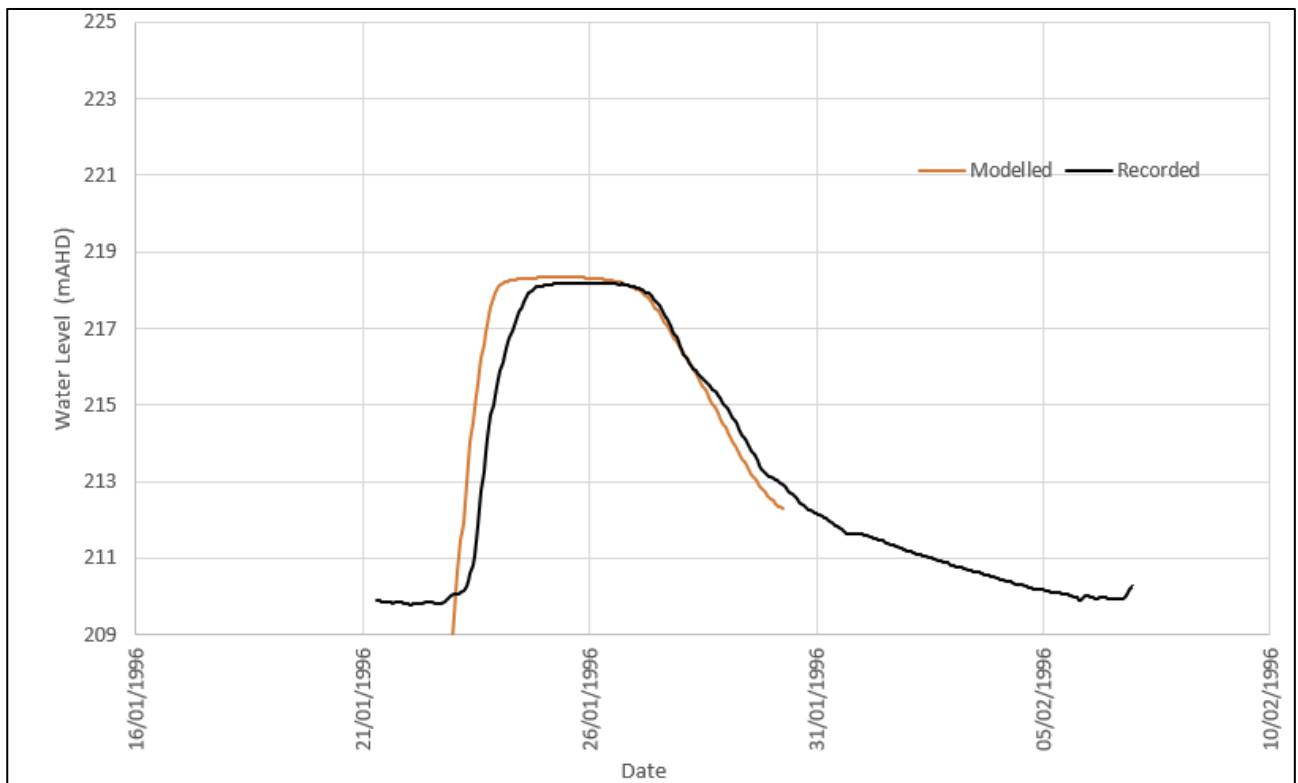


Figure 17.27 Goondiwindi gauge—1996 levels recorded and modelled

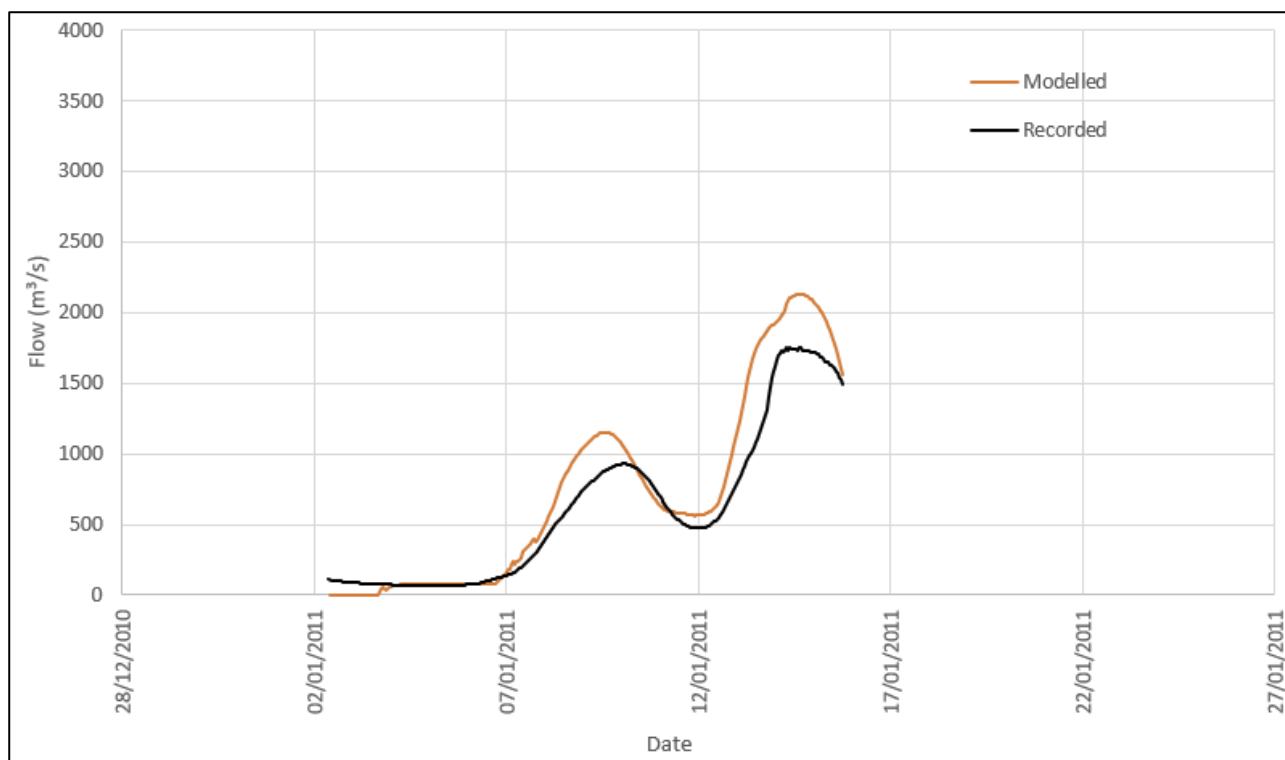


Figure 17.28 Goondiwindi gauge—2011 flows recorded and modelled

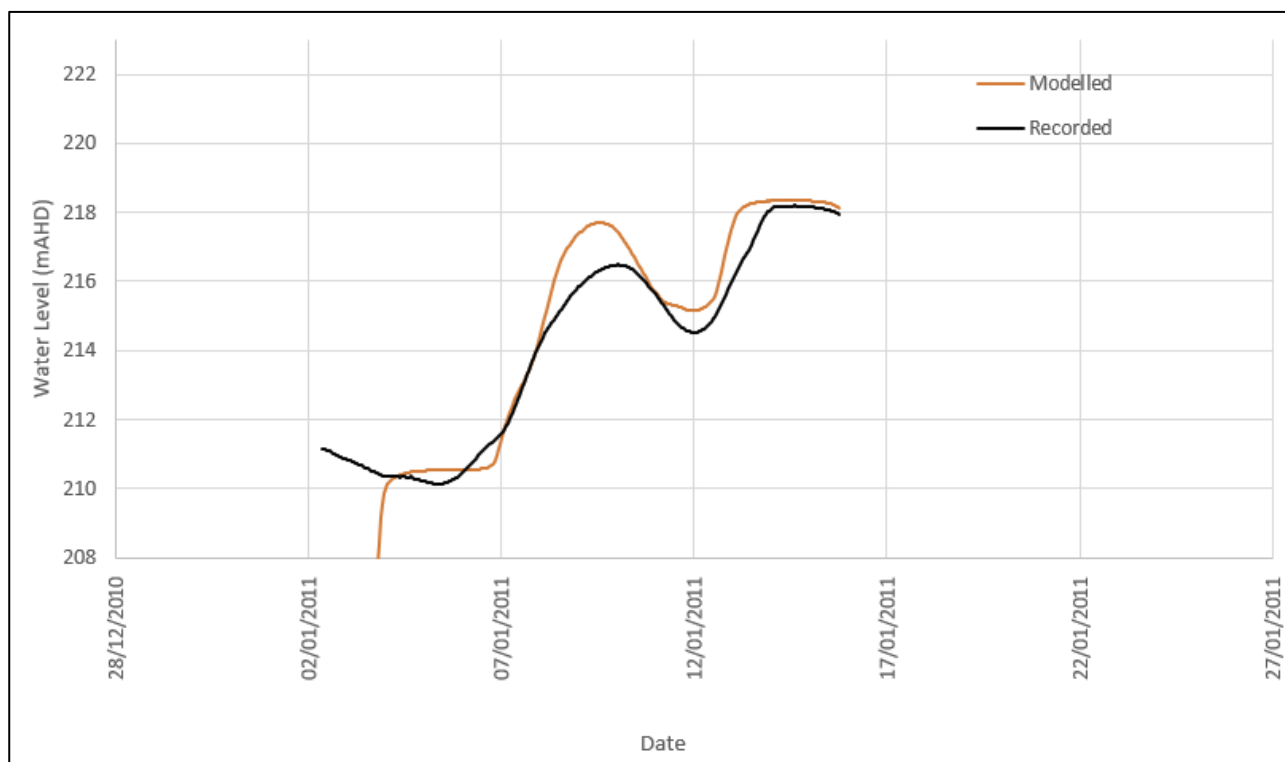


Figure 17.29 Goondiwindi gauge—2011 levels recorded and modelled

17.3.7 Validation

No validation was undertaken for the Macintyre River hydraulic model.

17.4 Existing Case modelling results—Macintyre River

17.4.1 Hydrologic model peak flow assessment

A critical duration assessment was undertaken to determine which storm duration(s) produced peak flood flows at the stream flow gauges locations and where the major waterways are intersected by the Project alignment. To assess the critical storm duration the following methodology was adopted:

- The models were modelled for a range of AEP events: 20% AEP, 10%, 5%, 2%, 1%, 1 in 2,000, 1 in 10,000 AEP and PMF:
 - Each AEP was modelled for a range of durations (12h to 120h)
 - Each duration was modelled for each of the 10 associated TPs.
- A critical duration assessment was undertaken at the locations mentioned above to determine which duration produced the highest median flow of the 10 TPs for each event.

Table 17.22 presents the estimated peak flow applied to the hydraulic model at outlets of several sub-catchments (reference maps).

Table 17.22 Macintyre River—1% AEP event peak flow at key locations as applied in the hydraulic model

Sub-catchment name	1% AEP event peak flow (m ³ /s)	Critical storm (hrs) duration and temporal pattern
Macintyre River (upstream of Boggabilla)	4,647	24 hour, A=20,000 km ² Pattern 2 (ID #3816)
Mobbindry Creek	213	24 hour, A=100 km ² Pattern 9 (ID #3753)
Back Creek	110	12 hour, A=100 km ² Pattern 8 (ID #3572)
Forest Creek	155	12 hour, A=100 km ² Pattern 9 (ID #3573)
Stray Leaves Creek	114	48 hour, A=100 km ² Pattern 1 (ID #3925)

The critical duration/design event was selected using the standard ARR 2019 ensemble methodology. It is acknowledged that the design events were selected based solely on the hydrologic modelling/routing results. The adopted approach was considered to be appropriate for the level of detail as both the hydrologic and hydraulics models were calibrated to a range of historical flood events and a relatively good match of the hydrologic and hydraulic routing was achieved. In addition, the overall effect of the floodplain attenuation relative to the magnitude of the combined inflows from the hydrologic models is comparatively minor.

The critical duration of the Macintyre River (to Boggabilla) is noted to be relatively short (24 hours) for a catchment of this size. This can be attributed to the configuration of the catchment, which sees three compact tributary sub-catchments of relatively similar size/length come together upstream of Boggabilla. As shown in Table 17.23, the Macintyre River to Holdfast and Dumaresq River to Glenarbon, which combined contribute over 71 per cent of the catchment area to the Project alignment, have virtually identical average and maximum stream lengths to the tributary outlet. Since these have the same critical duration of 24 hours, and are a similar distance upstream of Boggabilla, as are the other major tributaries of Macintyre Brook (24 hours critical duration) and Ottleys Creek (18 hours critical duration), which contribute an additional 23 per cent of the catchment area (i.e. <6 per cent of the catchment is downstream of these independent inflows) it is logical that the critical event at Boggabilla is dominated by the combination of these inflows and has a similar critical duration. The joint calibration of the hydrologic and hydraulic models and the derivation of the critical duration is proposed to be reviewed in future stages.

Table 17.23 Macintyre River— Tributary characteristics

Location	Catchment area (km ²)	Average stream length to outlet (km)	Maximum stream length to outlet (km)
Macintyre River to Holdfast	6,892	138.7	246.8
Dumaresq River to Glenarbon	9,093	141.4	251.4
Macintyre Brook to Booba Sands	3,983	79.1	127.5
Ottleys Creek	1,211	62.7	119.2
Macintyre River u/s of Boggabilla	22,425	177.6	312.9

17.4.2 Existing Case flood maps

Existing condition flood maps are presented in Volume 2 – Appendix M and via an [Online Digital Portal](#). Appendix M contains the existing 20% and 1% AEP flood maps, while the [Online Digital Portal](#) contains the remainder of the modelled events. The location of each map set is summarised below:

- 20% AEP peak levels: Figure M3-A
- 1% AEP peak levels: Figure M3-B
- 20% AEP peak velocity: Figure M3-C
- 1% AEP peak velocity: Figure M3-D
- 20% AEP peak hazard: Figure M3-E
- 1% AEP peak hazard: Figure M3-F
- 20% AEP peak duration of inundation: Figure M3-G
- 1% AEP peak duration of inundation: Figure M3-H
- 20% AEP, 10% AEP, 5% AEP, 2% AEP, 1% AEP, 1 in 2,000 year AEP, 1 in 10,000 year AEP and PMF Existing Case results: [Online Digital Portal](#)
- For higher resolution mapping refer to the [Online Digital Portal](#).

17.4.3 Flood inundation extent and flood levels

Figure M3-B in Volume 2 – Appendix M shows the 1% AEP indicative flood extent and peak water levels within the Macintyre River floodplain for the Existing Case.

Widespread inundation is predicted under the 1% AEP event on the Macintyre River floodplain, with depths of approximately 10 to 12 m in the Macintyre River, 6 m in Whalan Creek and up to 2 m on the floodplain area.

The inundation from the NSW/QLD Border to the north in the 1% AEP event can be summarised as:

- Downstream of the Kippenbung Creek and Macintyre River junction—the ground elevations are higher, and the 1% AEP flooding extends approximately 2 km from the border. This area is predicted to be inundated during extreme floods.
- Upstream of the Kippenbung Creek and Macintyre River junction—inundation extends north to the existing QR railway.

17.4.4 Flood immunity of existing infrastructure

Kildonan Road is predicted to have immunity up to the 2% AEP flood event within the vicinity of the Project alignment. In the 1% AEP event overtopping of up to 280 mm is predicted. The overtopping depths for all flood events are presented in Table 17.24.

Table 17.24 Macintyre River: Existing Case—Estimated overtopping depths of key infrastructure

Infrastructure	Location	Overtopping depth (m)							
		PMF	1 in 10,000 AEP	1 in 2,000 AEP	1% AEP	2% AEP	5% AEP	10% AEP	20% AEP
Kildonan Road	1 km downstream of the Project alignment	4.20	2.48	0.70	0.28	Dry	Dry	Dry	Dry
Kildonan Road	1 km upstream of the Project alignment	3.76	2.04	1.6	Dry	Dry	Dry	Dry	Dry

17.4.5 Existing Case velocities

Velocities of approximately 0.5 m/s are predicted across the floodplain area under the 1% AEP event with higher velocities within the creeks and river channels. Flow remains mainly in the creek and river channels up to the 10% AEP event and breakouts occur downstream of the Toomelah township between a 10% and 5% AEP event. Velocities on the northern side of the Macintyre River are predicted to be approximately 0.23 m/s in the 1% AEP event. Velocity in the Macintyre River is predicted to be up to 4.5 m/s in the 1% AEP event.

17.4.6 Existing Case hazard

In the 1% AEP event hazard across the floodplain on average is 0.48 m²/s, to the north of the Macintyre River an average velocity-depth product of 0.68 m²/s is predicted. In the Macintyre River velocity-depth product up to 26.38 m²/s is predicted in the 1% AEP event.

17.4.7 Existing Case time of inundation

In the 1% AEP event time of inundation is predicted to be approximately 47 hours across the main floodplain with some locations exceeding 100 hours. To the north of the river the duration of inundation is predicted to be approximately 44 hours.

17.5 Developed Case modelling results—Macintyre River

17.5.1 Drainage structures

The hydraulic design of the major drainage structures was undertaken using the TUFLOW model (1d and 2d approach). In the Macintyre River floodplain north of the NSW/QLD Border, the Project includes the following floodplain (or regional structures):

- Three waterway bridges
- Four RCP locations (a total of 50 cells).

Local drainage structures are presented in Appendix B. These structures were sized through the local drainage design and depending on their interaction with the Macintyre River floodplain were incorporated in the hydraulic model. North of the NSW/QLD Border. The locations of the structures are presented in Figure M1-C in Volume 2 – Appendix M.

Bridges were represented within the TUFLOW model through use of Ifcsh. Each bridge within the model has had a flow constriction coefficient applied to represent obstruction of waterway area due to the piers. Bridge form losses are presented in Table 17.25.

Table 17.25 Macintyre River—Proposed bridge locations and details

Chainage (km)	Structure ID	Bridge name	Soffit level (mAHD)	Deck depth (m)	Bridge length (m)	Form loss (Pier loss)
30.35	270-BR11	270-BR11	229.8	1.2	1,750	0.09
31.52	270-BR12	270-BR12	229.8	1.2	140	0.20
32.30	270-BR13	270-BR13	227.7	1.2	546	0.23

An assessment on form loss was undertaken for the larger bridge structures and the corresponding value applied to Layer 1. For other bridges a form loss value of 0.2 was applied to Layer 1 (beneath the bridge deck) of the Ifcsh to represent the waterway opening area. This value is considered conservative, although it is noted that changing form loss would not have a significant impact in this floodplain where the floodwaters are slow moving. No additional blockage was applied to the waterway area. The bridge deck (Layer 2) was modelled as 100 per cent blocked, and above the bridge deck (Layer 3), 50 per cent blocked. These parameters will be revisited in detailed design.

All drainage structures are presented in the cross-drainage register in Appendix B. It is noted that the above structure details reflect how the structures are represented in the hydraulic sub-model and minor variations may occur between the modelled structures and the Project's revised reference design structures (i.e. culvert lengths).

17.5.2 Hydraulic design criteria outcomes

The hydraulic model was run for the Developed Case with the drainage structures and embankment areas included. Modelling of a range of events was undertaken (20%, 10%, 5%, 2%, 1% AEP events and 1 in 2,000 AEP, 1 in 10,000 AEP and PMF events). The Project revised reference design outcomes relative to the hydraulic design criteria are presented in the following sections.

17.5.2.1 Rail immunity and structures results

Appendix B presents hydraulic model results at each structure for the 1% AEP event.

The formation level of the Project alignment is driven by several factors including achieving flood immunity and meeting geometric requirements (e.g. allowing for grade separations). Therefore, the freeboard achieved varies along the Project alignment with the 1% AEP event flood immunity achieved with at least a minimum freeboard of 300 mm, noting that the formation levels were generally dictated by other engineering criteria and constraints.

The results of flood modelling indicate that a 1% AEP event flood immunity to the proposed rail formation is achieved for the Project alignment across the Macintyre River floodplain.

17.5.3 Flood Impact Objectives outcomes—Macintyre River

The impact of the Developed Case was assessed through inclusion of the proposed rail design and comparison of model results against the Existing Case results.

Changes to flooding behaviour that were assessed, and are reported in the following sections, include potential changes in peak water levels, duration of inundation, velocities, hazard and peak flows within the floodplain.

- Changes in 20% AEP and 1% AEP peak water levels are presented in Figures M5-A and M5-B in Volume 2 – Appendix M
- Changes in 20% AEP and 1% AEP velocities are presented in Figures M5-C and M5-D in Volume 2 – Appendix M
- Changes in 20% AEP and 1% AEP hazard are presented in Figures M5-E and M5-F in Volume 2 – Appendix M

- Changes in 20% AEP and 1% AEP duration of inundation are presented in Figures M5-G and M5-H in Volume 2 – Appendix M
- Changes in 20% AEP and 1% AEP velocities and percentage change in velocities with cut-offs applied are presented in Figures M6-A to M6-F in Volume 2 – Appendix M
- Percentage changes in 20% AEP and 1% AEP in hazard are presented in Figures M7-A and M7-B in Volume 2 – Appendix M
- Percentage changes in 20% AEP and 1% AEP in duration of inundation and with cut-offs applied are presented in Figures M8-A and M8-D in Volume 2 – Appendix M.
- Refer to the [Online Digital Portal](#) for other modelled AEP results and higher resolution mapping.

One-on-one consultation with landowners who are expected to experience changes in flooding behaviour on their property was conducted by ARTC supported by FFJV. Refer to EIS Chapter 14: Flooding and Geomorphology and EIS Appendix E: Consultation Report for further details.

The Project design outcomes relative to the FIOs (Table 4.2) are presented in the following sections.

Flood sensitive receptors referred to in this section are shown in Figures P1 (1 to 59) in Volume 2 – Appendix P.

17.5.3.1 Flood impacts on flood sensitive receptors

Based on the available aerial imagery, no buildings or critical infrastructure are located within the area predicted to be impacted by changes in peak water levels on the Macintyre River floodplain north of the NSW/Queensland Border for events up to the 1% AEP.

Under the 1 in 2,000 AEP event there are three habitable or commercial buildings with predicted afflux above 250 mm, and one newly wet FSR. Table 17.26 provides a summary of the FSRs where exceedances are predicted above the FIO target for all events up to and including the 1 in 2,000 AEP event.

Table 17.26 Macintyre River—Flood Impact Objective exceedances

FSR ID	FSR type	FIO target (up to 1% AEP)		FIO target (1 in 2,000 AEP)	Afflux for each AEP (mm)				
				20%	10%	5%	2%	1%	1 in 2,000
MCR_ID_41	Shed	100 mm	250 mm	-	-	-	-	-	+373
MCR_ID_46	Houses	10 mm	250 mm	-	-	-	-	-	+502 ¹
MCR_ID_44	Stock yards	100 mm	250 mm	-	-	-	-	-	+415
MCR_ID_98	House	10 mm	250 mm	-	-	-	-	-	+33 ¹

Table note:

1 Newly wet FSR

As indicated in Table 17.26, MCR_ID_46 represents three houses within close proximity of one another. At this location, in the existing case under the 1% AEP event, the three houses are dry and not impacted by flood waters. Furthermore, in the existing case 1 in 2,000 AEP event, although the three houses are dry, floodwaters approach the vicinity of the houses. As indicated in Table 17.26, in the developed case 1 in 2,000 AEP event, all three houses experience increases in water levels above the 250 mm FIO target.

The increase in water levels remain due to limited drainage infrastructure currently proposed in the Project's revised reference design to convey the 1 in 2,000-year AEP event. This results in ponding of water on the upstream side of the Project alignment (causing an increase in water levels at the houses). Currently, proposed cross-drainage is sized to the 1% AEP event, and as this portion of the alignment is not impacted under the 1% AEP event no drainage structures have been proposed at this location. A review of cross-drainage design will be undertaken at this location to potentially increase the conveyance under the Project alignment as well as analysis associated with a more frequent event between the 1% AEP and 1 in 2,000-year AEP event will be undertaken in detailed design.

There are no FSRs subject to a change of hazard (velocity-depth product) greater than the FIO target (10 per cent) for all events up to and including the 1% AEP event. At this stage, the change in duration of inundation has not been assessed for the FSRs as details of floor levels need to be confirmed through survey.

Mitigation of impacts on FSRs is individual to each landowner and will require ongoing consultation with those impacted to agree on appropriate mitigation specific to each site. ARTC will continue through the next stages of design to engage with impacted landowners to negotiate site-specific mitigation measures.

17.5.3.2 Flood impacts on State-controlled roads

There are no FIO exceedances predicted on State-controlled roads for events up to and including the 1% AEP event.

17.5.3.3 Flood impacts on local public roads

There are no FIO exceedances predicted on the local public roads for events up to and including the 1% AEP event.

17.5.3.4 Flood impacts on existing rail infrastructure

There are no FIO exceedances predicted on the existing rail infrastructure for events up to and including the 1% AEP event.

17.5.3.5 Flood impacts on private land outside the permanent footprint

In accordance with the FIOs, Table 17.27 presents the modelled changes and potential impacts on third-party property outside of the Project's permanent footprint, with appropriate justification and mitigation also identified. A summary of key impacts is provided in the below points:

- Twenty-three separate properties with individual land-use categories were identified as having a potential impact exceeding the Project FIO criteria
- The majority of all potential exceedances occurred in Cotton and Vacant Rural Land land-use types. Several properties identified as Vacant – Large House site are identified as impacted and this may be due to the stringent criteria placed on this lots. Further investigations into the current land use are required in detailed design.
- Maximum change in water level (afflux) for all properties is 2.55 m (Lot Plan 484SP119198). This location is subject to an increase in inundation extent, which results in a depression being impacted by flood waters, hence resulting in such a large increase. It should be noted that this increase does not translate to a uniform flood level increases across the lot. Overall, the area of impact is <1 per cent (0.09 ha) of the total lot area and therefore can be considered localised.
- Maximum change in velocity for all properties is approximately 1.1 m/s on Lot Plan 14SP127017, occurring in the 2% AEP event. This exceedance impacts an area of 12.1 ha, which is <50 per cent of the overall lot area and occurs due to a change in inundation extent. Based on the area of impact, this exceedance can be considered localised.
- Maximum change in the duration of inundation is 4.5 hours and a 334 per cent increase, occurring on Lot Plan 1RP49163. These exceedances impact an area of up to 2.60 ha (which equates to approximately 1 per cent of the total lot area).
- The maximum percentage change in hazard occurs on Lot Plan 37MH878, increasing by 8,000 per cent from 0.002 m²/s to 0.15 m²/s, across an area of 1.27 ha (<2 per cent of the lot area). The increase to 0.15 m²/s in the Developed Case is still considered low risk and localised.

Table 17.27 Macintyre River – Summary of flood impacts on private land outside the permanent footprint

[illegible]

Table Notes:

1. Floodplains are dynamic in nature and therefore model results vary from cell to cell. The "Area" is therefore defined as the maximum area over which change above the Flood Impact Objective occurs and does not relate directly to the maximum F100 exceedance recorded.
2. Change in water level (afflux) is calculated based on the maximum water surface level difference between the existing and design case results, and does not consider the underlying topography. Depth values presented are based on the respective underlying topographies and in some cases do not align with afflux calculations. This can be attributed to interpolation calculations of results performed within the hydraulic model.
3. Percentage increases should be read in conjunction with existing and design case results. The F100s are presented in percentage change for time of inundation, velocity and hazard and can sometimes produce large increases when existing case results are significantly small. Some locations were identified to experience significant percentage increases however the "absolute" difference between Existing and Design results are relatively small.
4. The Flood Impact Objective exceedance has been identified due to a change in flood volume and a cell which was dry in the existing case and now flood-affected in the developed case.
5. The above table should be read in conjunction with change maps provided as part of Volume II and the Digital Platform.
6. The symbol "0" denotes that no change above Flood Impact Objectives has been determined and therefore no result has been presented.
7. Changes recorded above may only occur during a small fraction of model simulation time.
8. Three decimal places do not infer absolute accuracy in modelling results, but it is necessary to identify small changes.
9. Flood Impact Objectives are assessed based on landslide applied to the entire lot and exceedances identified do not necessarily indicate an impact to a dwelling.
10. Design assumptions that could be implemented (but limited to) include adjustment of culvert spacing, realignment of culverts to existing flowpaths when more detailed survey is available, modifying the grade of culverts, road height and alignment adjustments, removal of bridge abutments from waterway extents etc.
11. Landslide data, dated 2018, has been used for this assessment. Due to the changing nature of this dataset, this will be updated to the latest available information in Detailed Design.

17.5.3.6 Flow distribution

A key landowner concern is potential change to flow distribution. To understand the magnitude of these flows, flows were extracted from the hydraulic model at key locations.

For the Border Rivers catchment, the BRVFMP uses a series of discharge calculations locations to review changes in flow distribution associated with development on the floodplain areas. The location closest to the Project alignment is the long section (known as Section 1) shown in Figure 17.30 and extracted from Appendix 6 of the BRVFMP.

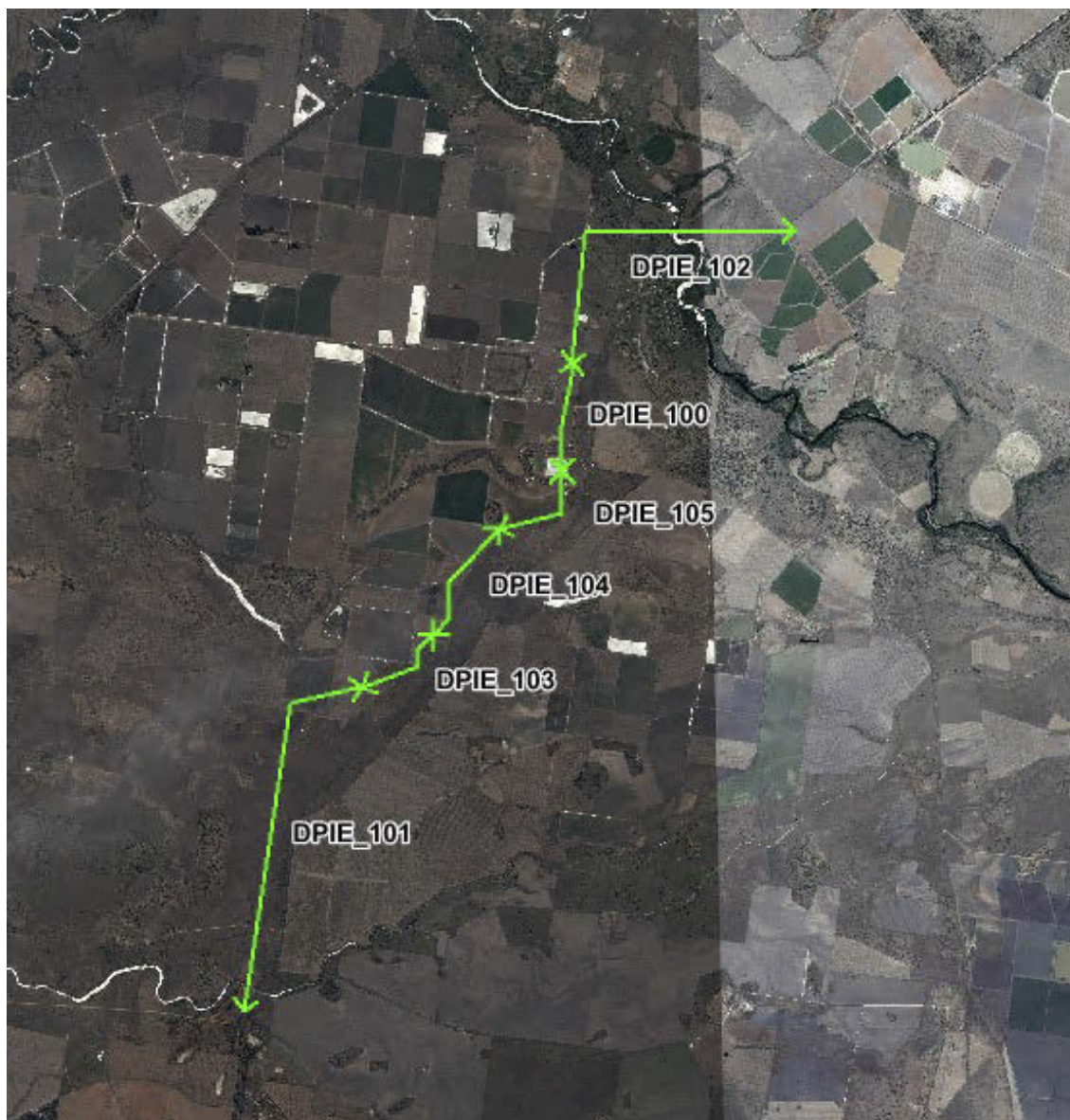


Figure 17.30 Boggabilla Gauge—Section 1 from BRVFMP

The details of Section 1 were obtained digitally from NSW DPE and used to extract flows within each section for the 1976 flows scenario. The BRVFMP requires that there is no more than a 5 per cent change in flow distribution between the Existing and Developed Cases (noting the Developed Case includes the Project alignment and associated works).

Table 17.28 presents the flows for each segment of Section 1 for both the Existing and Developed Cases as well as the percentage change. As can be seen the percentage changes values are all less than the 5 per cent limit, with the majority around 0 to 2.5 per cent. It should be noted that this assessment was undertaken on peak flows and that consideration of flood volume is likely to result in closer volumes between the Existing and Developed Cases.

Table 17.28 Peak flow variation assessment along Section 1

Segment of Section 1	BRVFMP levees and validated 1976 flows	
	Existing Case flows (m ³ /s)	Developed Case flows (m ³ /s) and % Change
DPIE_100	402	406 (1%)
DPIE_101	2,228	2,193 (-2%)
DPIE_102	3,362	3,366 (0%)
DPIE_103	193	195 (1%)
DPIE_104	380	386 (1%)
DPIE_105	121	124 (2.5%)

In addition to the above, flow distribution mapping was produced for all events up to the PMF. To determine the change in flow distribution, the maximum velocity x depth product results were used to produce change in unit flow maps between existing and design conditions. Unit flow is the measure of discharge (velocity x depth product) and is often used for assessing peak flow along a cross-section, or to determine the effective flow corridors within a floodplain.

The presented flow distribution maps were filtered with a velocity x depth product cut-off of 0.15 m²/s. The cut-off is in line with the Project FIO requirements and was used to assist in identifying main channels (high flow zones) from shallow lower flood zones (i.e. outer extents of the flood surface).

Flow distribution mapping can be found in Volume 2 – Appendix Q Figures Q13-1 to Q13-8.

Ongoing mitigation measures to reduce changes in flow distribution are likely to occur during future design stages of this Project, and may include further refinement of the bridge abutments, culvert skews, and additional survey where necessary.

17.5.4 Extreme events

17.5.4.1 Extreme event immunity

The risk of overtopping of the Project alignment was assessed for a range of extreme events (1 in 2,000 AEP, 1 in 10,000 AEP and PMF) with Table 17.29 presenting the depth of water above the formation level and over the top of rail at each structure. It is noted that the function of the floodplain culverts is to balance flood levels on the upstream and downstream sides of the Project alignment. As such, overtopping of the rail is not predicted to result in significant excessive flows or velocities as would occur in a dam embankment overtopping scenario.

Table 17.29 Macintyre River: Extreme events—Estimated depth of water above formation level and/or over top of rail level

Chainage (km)	Structure ID	Depth of water above formation level (m)			Depth of water above top of rail (m)		
		1 in 2,000 AEP	1 in 10,000 AEP	PMF	1 in 2,000 AEP	1 in 10,000 AEP	PMF
31.26	C31.26	-	-	-	-	-	-
31.32	C31.32	-	-	-	-	-	-
31.52	310-BR12	-	-	1.23	-	-	0.53
31.87	C31.87	-	0.44	2.01	-	-	1.31
32.00	C32.0	-	0.41	1.90	-	-	1.20
32.55	310-BR13	-	0.84	2.17	-	0.14	1.47

In addition to the above detail, an overtopping failure assessment for the 1 in 2,000 AEP, 1 in 10,000 AEP and PMF events was completed in Section 21.

17.5.4.2 Impacts during extreme events

Flood immunity of FSRs is discussed in Section 17.5.3.1, and maps demonstrating the impacts during extreme events are shown on the [Online Digital Portal](#).

In accordance with the FIOs, FSRs are considered impacted by extreme flood events if the change in water level is greater than 250 mm in the 1 in 2,000 AEP event, subject to the receptor classification (i.e. at existing habitable dwellings and/or commercial and industrial buildings/premises). In addition, a stricter afflux objective of 10 mm in the 1 in 500 AEP event applies to sites with existing sensitive infrastructure, uses involving vulnerable people, and/or any critical road network that was designed to be immune to flooding in any extreme event.

Review of the 1 in 2,000 AEP event results for the Macintyre River floodplain against the FIOs show that flood level changes above the 250 mm limit were observed at three habitable or commercial buildings, and one newly wet FSR. No identified existing sensitive infrastructure or uses involving vulnerable people are expected to experience a more than 10 mm increase in flood level. A separate assessment of the 1 in 500 AEP event is not required as they are conservatively represented by the 1 in 2,000 AEP event results.

Table 17.30 details extreme event results, although it should be noted that FIO targets do not apply to the 1 in 10,000 AEP and PMF events.

Table 17.30 Macintyre River –Summary of extreme event impacts at flood sensitive receptors

FSR ID	FSR Type	1 in 2,000 AEP event		1 in 10,000 AEP event		PMF event	
		Change in peak water level (mm)	Existing Case flood depth (m)	Change in peak water level (mm)	Existing Case flood depth (m)	Change in peak water level (mm)	Existing Case flood depth (m)
MCR_ID_31	Centre Pivot Irrigation	+55	1.25	+85	2.14	+96	3.87
MCR_ID_32	Centre Pivot Irrigation	+52	1.34	+84	2.25	+89	4.01
MCR_ID_36	River Pump Station	+58	3.28	+85	4.15	+102	5.83
MCR_ID_40	Stockyards	-69	2.15	-83	3.19	-34	5.00
MCR_ID_41	Sheds	+373	1.64	+511	2.66	+358	4.44
MCR_ID_42	Rainbow Reserve	+58	0.50	+96	1.47	+92	3.28
MCR_ID_43	Additional Development Constructed	+152	0.17	+850	0.87	+254	2.69
MCR_ID_44	Stockyards	+415	0.19	+1116	1.42	+280	3.36
MCR_ID_46	House	+502	-	+111	0.81	+4	2.57
MCR_ID_49	Grazing Property	+119	1.91	+224	2.99	+205	4.86
MCR_ID_50	Quarry Access	+66	1.10	+142	2.33	+141	4.43
MCR_ID_98	House	+33 ¹	-	+211	0.10	+71	0.75
MCR_ID_104	House	+75	1.79	+88	2.74	+81	4.50
MCR_ID_105	House	+80	1.71	+93	2.73	+86	4.56
MCR_ID_106	Shed	+79	2.38	+91	3.37	+84	5.18
MCR_ID_108	Sheds Commercial	+74	0.47	+92	1.45	+89	3.26
MCR_ID_110	House	Dry	0.65	-421	1.35	-119	2.59
MCR_ID_111	House	+243	1.06	+367	1.85	+221	3.20
MCR_ID_112	House	Dry	0.50	-362	1.24	-36	2.52

FSR ID	FSR Type	1 in 2,000 AEP event		1 in 10,000 AEP event		PMF event	
		Change in peak water level (mm)	Existing Case flood depth (m)	Change in peak water level (mm)	Existing Case flood depth (m)	Change in peak water level (mm)	Existing Case flood depth (m)
MCR_ID_115	House	+71	0.80	+91	1.75	+89	3.55
MCR_ID_118	Sheds	+67	1.85	+89	2.72	+97	4.42
MCR_ID_119	House	Dry	0.68	-369	1.42	-46	2.70
MCR_ID_120	Shed	+176	1.64	+243	2.56	+135	4.18
MCR_ID_123	House	+109	0.39	+158	1.61	+114	3.64
MCR_ID_124	Commercial	-	-	+517		+135	0.46
MCR_ID_125	House	-	-	-98	0.30	+57	1.61
MCR_ID_126	House	+14	2.87	+42	4.03	+96	6.04
MCR_ID_127	Water Tanks	+14	1.72	+41	2.88	+96	4.90
MCR_ID_128	House	+5	1.39	+28	2.59	+82	4.83
MCR_ID_129	House	+5	1.39	+28	2.58	+82	4.82
MCR_ID_130	House	+5	1.13	+27	2.32	+81	4.55
MCR_ID_131	Shed	+6	1.26	+28	2.45	+83	4.67
MCR_ID_132	House	-	2.56	+6	3.65	+41	5.86
MCR_ID_133	Shed	-	2.24	+5	3.34	+40	5.54
MCR_ID_134	Shed	-	2.47	+7	3.55	+43	5.75

Table note:

1 FSR was wet in the Existing Case and is now dry in the Developed Case

17.5.5 Sensitivity analysis—Macintyre River

The sensitivity of the model to various parameters was assessed using the following scenarios:

- An increase in rainfall intensity (i.e. to reflect climate change scenario)
- An increase and decrease in blockage of culverts from 25 to 50 per cent and 0 per cent.

17.5.5.1 Climate change

The potential impacts of climate change in the Macintyre River floodplain were assessed for the 1% AEP design event to determine the sensitivity of the Project to the potential long-term changes in climate. The assessment was undertaken in accordance with ARR 2019.

The Representative Concentration Pathways 8.5 (2090 horizon) climate change scenario was adopted for the Project with an associated increase in rainfall intensity of 23 per cent across the catchment area.

The predicted flow resulting from a 23 per cent increase in rainfall is 3,500 m³/s in the Macintyre River at Boggabilla (where the flows are controlled by the topography) for the 1% AEP event (compared to 3,215 m³/s in existing climate conditions). In the upper sections of the hydraulic model in the Dumaresq and Macintyre Rivers, the flows are predicted to increase by approximately 25 per cent as a result of the increase in rainfall in the 1% AEP event with climate change.

The 1% AEP change in peak water levels for the Representative Concentration Pathways 8.5 climate change scenario is presented in Figure M9-C in Volume 2 – Appendix M.

The model predicts that with an increase in rainfall intensity across the catchment the flood levels increase by up to 200 mm in the vicinity of the Project alignment. The Project alignment is not predicted to be overtopped as a result of the increase in rainfall intensity with water levels predicted to be below formation level. Change in peak water levels experienced at sensitive receptors due to climate change are presented in Table 17.31.

Table 17.31 Macintyre River: 1% AEP event—Climate change assessment

FSR ID	FSR type	1% AEP climate change event	
		Change in peak water level (mm)	Existing Case flood depth (m)
MCR_ID_31	Centre pivot irrigation	+1	0.59
MCR_ID_32	Centre pivot irrigation	-1	0.63
MCR_ID_36	River pump station	+1	2.58
MCR_ID_40	Stockyards	-42	1.19
MCR_ID_41	Sheds	+186	0.69
MCR_ID_104	House	+2	1.06
MCR_ID_105	House	+2	0.92
MCR_ID_106	Shed	+2	1.60
MCR_ID_108	Shed	+1	0.03
MCR_ID_110	House	-1	0.01
MCR_ID_111	House	+6	0.22
MCR_ID_115	House	+1	0.11
MCR_ID_118	Sheds	+2	1.17
MCR_ID_120	Sheds	+3	0.68
MCR_ID_126	House	+1	1.81

Table note:

1 FSR was wet in the Existing Case and is now dry in the Developed Case

17.5.5.2 Blockage

A blockage factor of 25 per cent was incorporated into the culvert design as per ARR 2019 guidelines. In addition, two blockage sensitivity scenarios were tested: 0 and 50 per cent blockage of all culverts.

The results are presented in Figure M9-A and Figure M9-B in Volume 2 – Appendix M for the 0 and 50 per cent blockage, respectively.

The model predicts that in both the 0 and 50 per cent blocked cases, the predicted changes in peak water levels meet the guiding design criteria for the 1% AEP event. Table 17.32 provides a summary of 1% AEP peak flood levels at FSRs for the blockage scenarios. Water surface levels at FSRs to the north of the Macintyre River are not sensitive to the blockage of the culverts due to the small number of culverts in proportion to bridge waterway area and the proximity of the FSRs to the culverts in this section of the Project alignment.

During detailed design the blockage factors will be reviewed in line with the final design and local catchment conditions. This may result in a varied and/or lower blockage factors being applied along the Project alignment. Should impacts be determined due to a change in blockage factor, additional mitigation measures will be considered in detailed design.

Table 17.32 Macintyre River—Summary of 0% and 50% blockage impacts at flood sensitive receptors

FSR ID	FSR type	Existing Case flood depth (m)	0% blockage Change in peak water level (mm)	50% blockage Change in peak water level (mm)
MCR_ID_31	Centre pivot irrigation	0.18	+1	+1
MCR_ID_32	Centre pivot irrigation	0.17	+1	+1
MCR_ID_36	River pump station	2.11	+1	+1
MCR_ID_40	Stockyards	0.45	-1	-1

FSR ID	FSR type	Existing Case flood depth (m)	0% blockage Change in peak water level (mm)	50% blockage Change in peak water level (mm)
MCR_ID_41	Sheds	0.02	+8	+8
MCR_ID_104	House	0.60	+2	+2
MCR_ID_105	House	0.40	+2	+2
MCR_ID_106	Shed	1.10	+2	+2
MCR_ID_118	Sheds	0.72	+2	+2
MCR_ID_130	House	0.01	+1	+1

17.6 Additional modelling items to be considered in detailed design

Detailed design provides the opportunity to incorporate further detail in the modelling and using the improved detail to assess the elements of the Project's revised reference design further.

Items to be incorporated into the detailed design for this section of the Project are:

- Blockage factors will be reviewed in line with the final design and local catchment conditions
- Significant new developments, with a development approval, that are likely to affect the local hydrology and floodplain behaviour, and that are likely to be constructed prior to the construction of Inland Rail, will be included in the flood modelling. Developments for inclusion will be discussed and agreed with the applicable approval authority.
- During detailed design specific consideration will be given to fencing and local access roads on a case-by-case basis
- Inclusion in the hydraulic modelling of missing existing structures from survey information if available
- Consideration of form losses on the bridge decks for extreme events
- Drainage diversions
- Proposed noise barriers.

Further details of mitigation measures can be found in Section 22. A review of proposed mitigation measures will be completed during the detailed design stage of the Project after hydraulic models have been updated to capture the above changes.

17.7 Planned and future roads

Consultation with DTMR has not identified any planned or future road upgrades within the Macintyre River catchment. ARTC will continue to consult and engage with DTMR to determine any future or planned road projects that may need to be considered as part of the detailed design hydraulic modelling.

Consultation with GRC has not identified any planned or future road upgrades within the Macintyre River catchment. ARTC will continue to consult and engage with GRC to determine any future or planned road projects that may need to be considered as part of the detailed design hydraulic modelling.

18 Local catchment assessment

Drainage for the minor catchments study area (areas less than 10 km²) are classified as 'local' drainage.

Cross-drainage structures in the form of bridges or culverts are provided where the Project alignment intercepts existing flowpaths. Proposed local cross-drainage structures located within the Project extent are identified in Appendix B.

Longitudinal drainage is proposed along the Project alignment to collect runoff from the track and from overland flows draining towards the rail. The longitudinal drainage was designed to convey these catchment flows to the proposed cross-drainage structures or other suitable discharge points.

Details of the hydrologic and hydraulic assessments for the local catchment drainage are provided in the sections below. Descriptions of the catchment classifications for the Project can be found in Section 3.3.

18.1 Local catchment hydrology

18.1.1 Local cross-drainage

The 20%, 10%, 5%, 2%, 1%, 1 in 2,000, 1 in 10,000 AEP and PMF event catchment flows for the minor catchments were generated in accordance with ARR 2019 using 12D software (Time Area Hydrology model with Horton Losses (ILSAX) method within the 12D Drainage Network Editor). Flows were extracted based on the peak flow critical storm duration.

The Project alignment was divided into eight equal sections and a 12D model was created for each of these sections. For each 12D model, an IFD was extracted from the ARR Data Hub at the centre point of the Project alignment section located within each 12D model. The IFD extraction locations for each of the 12D models are as follows:

- Latitude = -28.598, Longitude = 150.661
- Latitude = -28.439, Longitude = 150.960
- Latitude = -28.235, Longitude = 151.156
- Latitude = -28.042, Longitude = 151.203
- Latitude = -27.898, Longitude = 151.293
- Latitude = -27.719, Longitude = 151.572
- Latitude = -27.560, Longitude = 151.776
- Latitude = -27.492, Longitude = 151.834.

Ten TPs were run for each storm duration and the median TPs from each duration were compared to determine the peak runoff for each catchment.

The losses adopted within ILSAX for the local catchment flows were taken from the calibrated regional hydrologic models along the Project alignment.

As no calibration data was available to compare against the local catchment flows, the 1% AEP flows generated from ILSAX was compared against the traditional Rational Method. The flows generated using ILSAX compare closely with the flows generated from the traditional Rational Method and are within a tolerance of -8 to 15 per cent.

The Rational Method is no longer compliant with ARR 2019; however, it is still considered to give a reasonable approximation of local catchment flows; therefore, the parameters and resultant ILSAX flows were adopted for the Project's revised reference design. A review of the hydrology will be completed in detailed design through establishment of a more detailed hydrologic model.

18.1.2 Longitudinal drainage

The longitudinal drainage catchments fall within the minor catchment category, with catchment areas generally significantly less than the upper limit of 10 km². The catchment runoff was calculated using ILSAX within the 12D software package as per the methodology outlined in Section 18.1.1.

18.2 Local catchment hydraulic design

18.2.1 Local cross-drainage

Cross-drainage structures are provided where the Project alignment intercepts existing flow paths. The type of structures adopted depends on a range of factors, including the natural topography, rail formation levels, design flows and soil type.

The cross-drainage design was undertaken in accordance with the Project hydraulic design criteria set out in Table 4.1. Cross-drainage structures outside the regional floodplains were sized based on the flows generated from the local drainage catchments. Cross-drainage structures that have well-defined local catchment boundaries and are located within or near the regional floodplains were assessed for both the local catchment flows and regional floodplain conditions to determine the governing design conditions.

Once preliminary sizing of local cross-drainage structures had been performed within 12D, a 2D modelling approach was adopted to simulate the flood regime around the proposed rail crossings. The platform used for hydraulic modelling is the TUFLOW HPC software package. Sixteen TUFLOW models were developed, with the parameters detailed in Table 18.1, to simulate flood regimes of local drainage catchments.

Table 18.1 Local drainage hydraulic model summary

Parameter	Information
Completion date	September 2022
AEPs assessed	20%, 10%, 5%, 2%, 1%, 1 in 2,000, 1 in 10,000 and PMF
Hydraulic model build	TUFLOW HPC GPU – version 2020-10-AD-iSP-w64
Model extent	Refer to Figure 3.3
Grid size	5 m
SGS size	1 m
QT Parameters	2.5 m, 2 nd Order
DEM (year flown)	ARTC dataset (2015), Queensland Government SRTM 25 m hydrologic-enforced shuttle radar
Roughness	Spatially varying roughness values compliant with industry norms.
Eddy viscosity	Wu (default)
Model calibration	N/A
D/S model boundary	Height-Discharge (HQ) Boundary with normal slope approximated based upon topography dataset.
Hydraulic model timestep	Adaptive Timestep
Modelled scenarios	Existing Case, Developed Case

The local hydraulic model extents are presented in Figure 3.3.

18.2.2 Longitudinal drainage hydraulic design

Longitudinal drains are contained within the Project footprint and convey flows to proposed cross-drainage structures, or outlet to existing creek lines.

The following longitudinal drainage types are proposed for The Project:

- Embankment drain—longitudinal drain adjacent to the rail formation where the rail is in embankment conditions
- Cut drain—longitudinal drain adjacent to the rail formation where the rail is in cut conditions
- Catch drain and catch bank—cut-off drainage at the top of cuttings
- Batter drain—drainage along cut-and-fill batters
- Batter chute—drainage down cut-and-fill batters
- Daylight drain—drainage channels to convey flows from cross-drainage to the downstream ground level.

The type of flow within the longitudinal drainage is one-dimensional and the hydraulic design was undertaken based on Manning's Formula, using Excel calculation spreadsheets. Trapezoidal drains are the preferred shape and were adopted for the longitudinal drainage. The preferred drain material type is grass lining. The drains are rock lined in steep areas and grass lined with velocity stop (as necessary) in flat areas. A minimum grade of 0.2 per cent was adopted for lined drains and 0.5 per cent for unlined drains.

18.2.3 Diversion drains

Existing drainage paths were maintained where possible; however, diversions are required where a rail cutting, or embankment intersects an existing drainage path. In these locations, the existing drainage path will be diverted away from the rail and connected back to existing flow path by a diversion drain.

Two diversion drains are proposed as part of the Project:

- One trapezoidal diversion drain is provided from Ch 121.92 km to Ch 120.77 km to divert runoff originating from the west away from the Project where the proposed rail it's in cut. The rail in cut intersects a mapped waterway at Ch 121.43 km, and this diversion drain connects this waterway to another one at Ch 120.77 km. The diverted flow returns to the original flow path 750 m downstream of the Project alignment.
- Another trapezoidal diversion drain is proposed where the Project rail embankment is on top of an existing flow path. This affected flow path is a mapped waterway that is 150 m long and runs from Ch191.96 km to Ch 192.11 km.

Some further minor diversions may be required, and these will be confirmed in detailed design once field survey data becomes available.

18.3 Fish passage

Designs for fish passageways in Queensland are to be undertaken in accordance with *Accepted development requirements for operational work that is constructing or raising waterway barrier works* (Department of Agriculture and Fisheries, 2018a). Where compliance with these requirements is not possible, the works become assessable development and require a Development Application.

Fish passage requirements are dictated by the hierarchy of waterways and the risk of impact determined by the Queensland Government. The level of risk is based on stream order, stream slope, flow regime, number of fish species and fish swimming ability.

The waterways intersected by the Project alignment were classified using the *Queensland waterways for waterway barrier works* (WWBW) data layer (Department of Agriculture and Fisheries, 2018b), which is used to define the legislative fish passage requirements for the cross-drainage structures within Queensland.

The waterway classifications from the WWBW mapping layer and the culverts requiring fish passageway designs are identified in Appendix B.

Table 18.2 outlines the fish passageway requirements from the Department of Agriculture and Fisheries legislation for culverts based on the waterway classification from the WWBW data later.

Table 18.2 Fish passage requirements for culvert crossings

Waterway zoning colour	Risk of impact	Work type for culvert crossing	Main design requirements
Green	Low	1.3 Construction of new or replacement of existing culvert crossing on a low-impact (green) waterway	■ Culvert installed at no steeper gradient than the waterway bed gradient ■ New culvert cells must be aligned parallel (within 10°) to the direction of water flow to minimise turbulence ■ The minimum (combined) culvert aperture width must be 1.2 m or span 100% of the main channel width ■ All new or replacement culvert cells must be installed at or below bed level.
Amber	Moderate	1.2 – Construction of new or replacement of existing culvert crossing on a moderate-impact (amber) waterway	■ Culvert installed at no steeper gradient than the waterway bed gradient ■ New culvert cells must be aligned parallel (within 10°) to the direction of water flow to minimise turbulence ■ The crossing must have a minimum (combined) culvert aperture width of 2.4 m or span 100% of the main channel width ■ All new or replacement culvert cells must be installed at or below bed level.
Red	High	1.1 – Construction of new or replacement of existing culvert crossing on a high impact (red) waterway	■ Culvert installed at no steeper gradient than the waterway bed gradient ■ Outermost culvert cells must incorporate roughening elements such as baffles on the bankside sidewalls ■ The width of a culvert cell aperture or the width of a culvert array must span a minimum of 100% of the low flow channel width ■ New culvert cells must be aligned parallel (within 10°) to the direction water flow to minimise turbulence ■ All culverts in the crossing are set at a minimum of 300 mm below bed level unless installed on bedrock, where the natural bed surface is maintained through the culvert.
Purple	Major	Development Approval	Culvert works requires development approval.

Fish passageway culverts were upsized by 300 mm height for box culverts and 300 mm diameter for pipes from the calculated hydraulic sizes in order to submerge the culverts by 300 mm at the upstream and downstream ends to comply with the fish passage requirements. Further calculations and assessment will be undertaken at detailed design to confirm fish passageway designs for all WWBW waterways meet velocity requirements for fish passage.

18.4 Stormwater quality management

An assessment of stormwater quality was completed as part of this revised draft EIS. Further information on the methodology adopted, criteria applied, and outcomes are presented in EIS Chapter 13: Surface Water and EIS Appendix S: Surface Water Technical Report.

18.5 Existing Case modelling results – Local catchments

Existing conditions flood maps for the 1% AEP and 20% AEP events are presented in Volume 2 – Appendix N and Appendix O, respectively, as well as via an [Online Digital Portal](#). The [Online Digital Portal](#) contains the remainder of the modelled events. The location of each map set is summarised below:

- 20% AEP peak levels: Figure O1-1 to O1-48
- 1% AEP peak levels: Figure N1-1 to N1-48
- 20% AEP peak velocity: Figure O2-1 to O2-48
- 1% AEP peak velocity: Figure N2-1 to N2-48
- 20% AEP peak hazard: Figure O3-1 to O3-48
- 1% AEP peak hazard: Figure N3-1 to N3-48
- 20% AEP peak time of inundation: Figure O4-1 to O4-48
- 1% AEP peak time of inundation: Figure N4-1 to N4-48
- 20% AEP, 10% AEP, 5% AEP, 2% AEP, 1% AEP, 1 in 2,000 AEP, 1 in 10,000 AEP and PMF Existing Case results: [Online Digital Portal](#)
- For higher resolution mapping refer to the [Online Digital Portal](#).

18.5.1 Existing Case flood inundation extent and flood levels

The peak modelled flood depth in the 1% AEP event is approximately 5 m, which is generally confined to main drainage lines such as Back Creek. This depth reduces to an average modelled flood depth of around 0.5 m in other areas. Due to the extent and nature of the local drainage catchments, there is significant variability in the flood characteristics experienced along the Project alignment.

Figures N1-1 to N1-48 in Volume 2 – Appendix N shows the 1% AEP event indicative flood extent and peak water levels within the local drainage catchments for the Existing Case.

18.5.2 Existing Case flood velocities

The peak modelled velocity across the local drainage catchments in the 1% AEP event generally varies from 2.5 m/s to 3.5 m/s in discrete locations. A peak velocity of 7 m/s is observed at a narrow, steep section of flow path, near the confluence with another waterway, and is situated away from any sensitive receptors. In the broader local drainage lines, velocities experienced are generally between 0.5 m/s and 1 m/s.

Figures N2-1 to N2-48 in Volume 2 – Appendix N shows the 1% AEP indicative peak velocities within the local drainage catchments for the Existing Case.

18.5.3 Existing Case flood hazard

For the 1% AEP event, a velocity-depth product (or 'hazard') of greater than 6 m²/s is achieved within existing creek lines, or at the confluence of main drainage lines and creeks. Excluding the channelised high flow zones, sheet flow in the local drainage lines have an average velocity-depth product of less than 0.5 m²/s.

Figures N3-1 to N3-48 in Volume 2 – Appendix N shows the 1% AEP indicative peak hazard within the local drainage catchments for the Existing Case.

18.5.4 Existing Case time of inundation

Results for the 1% AEP event highlight time of inundation within the local drainage catchments greater than 8 hours. Time of inundation of this order is generally confined to areas of ponding around or behind existing infrastructure. Excluding these areas, sheetflow in local drainage lines results in an average time of inundation of approximately 2 hours to 5 hours.

Figures N4-1 to N4-48 in Volume 2 – Appendix N shows the 1% AEP indicative peak time of inundation within the local drainage catchments for the Existing Case.

18.6 Developed Case modelling results—Local catchments

For each of the local catchment crossings, the impact of the Project on the existing flood regime was quantified and compared against FIOs, as detailed in Table 4.2. These objectives were used to guide the Project's revised reference design. Acceptable impacts will ultimately be determined on a case-by-case basis with interaction with stakeholders/landowners through the community engagement process, using these objectives as guidance; this takes into account FSRs and land use.

The land use across the Project footprint was classified as agricultural cropping/grazing/pastureland or forest areas based on aerial imagery. Sensitive agricultural land may be identified during further consultation with landowners, which will need to be considered in detailed design. The hydraulic impacts in the local catchments are considered 'localised' in comparison to regional flood impacts, due to the shorter time of flood inundation, shallower depths and smaller flood extents.

18.6.1 Drainage structures

The hydraulic design of the minor drainage structures was undertaken using 12D as well as the TUFLOW model (1D and 2D approach) as stated above.

In the local models, the Project includes the following minor drainage structures:

- 74 RCP locations (a total of 527 cells)
- 101 RCBC locations (a total of 891 cells)
- One waterway bridge (310-BR42)
- Five rail-over-road bridges that also convey flood flows in large events (310-BR28, 310-BR29, 310-BR30, 310-BR40 and 310-BR51).

Local drainage structures are presented in Appendix B. These structures were sized through the local drainage design and, depending on their interaction with the regional floodplains, were incorporated in the regional hydraulic models. The 1% AEP event flood level at each drainage structure is presented in Appendix B, including the anticipated change in upstream peak water levels and change in duration of inundation at each proposed cross-drainage structure likely to be caused by the Project.

A minimum culvert size of 900 mm diameter was adopted to reduce potential for blockage and maintenance.

Design bridge details are summarised in Table 18.3 and locations are presented in Figures N1-1 to N1-48 in Volume 2 – Appendix N. The 1% AEP flood level at the bridge structure is presented in Appendix B.

Table 18.3 Local catchment drainage—Proposed bridge locations and details

Chainage (km)	Structure ID	Bridge name	Span (approx.) (m)	Deck width (m)	Soffit level (mAHD)	Deck super structure type	Deck depth (mm)
60.51	310-BR51	Bybera Road Rail Bridge	75	3.97	308.59	Type D1	2,000
115.49	310-BR28	Millmerran Inglewood Road Rail Bridge #2	75	3.97	456.88	Type D1	2,000 + 2,450

Chainage (km)	Structure ID	Bridge name	Span (approx.) (m)	Deck width (m)	Soffit level (mAHD)	Deck super structure type	Deck depth (mm)
162.49	310-BR29	Yarranlea Road Rail Bridge	69	3.97	433.16	Type D1	2,000
172.19	310-BR30	Oakey Pittsworth Road Rail Bridge	69	3.97	513.94	Type D1	2,000
173.68	310-BR40	Lochaber Road Rail Bridge	75	3.97	528.95	Type D1	2,000 + 2,450
184.83	310-BR42	Biddeston Southbrook Road Rail Bridge	144	3.97	530.43	Type D1	2,000 + 2,450

Bridges were modelled as an opening in the rail embankment. The optimisation of bridge lengths was balanced between minimising the changes to the hydraulic regime, primarily afflux and velocities, and the cost of replacing bridge spans by large earth embankments.

Blockage of hydraulic structures was assessed in accordance with ARR 2019. Despite the catchment being vegetated and heavily forested, ARR guidelines determined that the likelihood of significant amounts of debris accumulating against the piers of the bridge is low; therefore, a zero-blockage factor was applied at all of the above bridges.

18.6.2 Hydraulic design criteria outcomes

The hydraulic model was run for the Developed Case for the range of events (20%, 10%, 5%, 2%, 1% and 1 in 2,000, 1 in 10,000 AEP and PMF events) with the outcomes presented in the following sections.

18.6.2.1 Rail immunity and structures results

Appendix B summarises the peak 1% AEP water levels on the upstream side of the Project alignment at all proposed cross-drainage structures.

The flood modelling results indicate that a 1% AEP event flood immunity to the proposed rail formation is achieved for the Project alignment across the local catchments, and that 1% AEP peak water levels remain below the proposed rail formation level. Freeboard between the 1% AEP peak water levels and the Project rail formation level varies across the alignment, with a minimum of 0.33 m to a maximum of 20 m, noting that the formation levels were generally dictated by other engineering criteria and constraints.

18.6.3 Flood Impact Objectives outcomes—Local catchments

The impact of the Developed Case was assessed through inclusion of the proposed rail design and comparison of model results against the Existing Case results.

Changes to flooding behaviour that were assessed and reported in the following sections, include potential changes in peak water levels, time of inundation, velocities and hazard within the local drainage catchments.

- Changes in peak water levels for the 1% and 20% AEP events assessed are presented in Figures N9-1 to N9-48 in Volume 2 – Appendix N, and Figures O9-1 to O9-48 in Volume 2 – Appendix O, respectively
- Changes in velocities are presented in Figures N10-1 to N10-48 and N13-1 to N15-48 for the 1% AEP event as well as O10-1 to O10-48 and O13-1 to O15-48 for the 20% AEP event in Volume 2 – Appendix N and Appendix O

- Changes in time of inundation are presented in Figures N12-1 to N12-48 and N17-1 to N18-48 for the 1% AEP event as well as O12-1 to O12-48 and O17-1 to O18-48 for the 20% AEP event in Volume 2 – Appendix N and Appendix O
- Changes in hazard are presented in Figures N11-1 to N11-48 and N16-1 to N16-48 for the 1% AEP event as well as O11-1 to O11-48 and O16-1 to O16-48 for the 20% AEP event in Volume 2 – Appendix N and Appendix O
- Refer to the [Online Digital Portal](#) for other modelled AEP results and higher resolution mapping.

One-on-one consultation with landowners who are expected to experience changes in flooding behaviour on the property was conducted by ARTC, supported by FFJV. Refer to EIS Chapter 14: Flooding and Geomorphology and EIS Appendix E: Consultation Report for further details.

The Project design outcomes relative to the FIOs (Table 4.2) are presented in the following sections.

Flood sensitive receptors referred to in this section are shown in Figures P1 (1 to 59) in Volume 2 – Appendix P.

18.6.3.1 Flood impacts on flood sensitive receptors

Based on the available aerial imagery, no buildings or critical infrastructure are located within the area predicted to be impacted by changes in peak water levels within local drainage catchments, for events up to the 1% AEP. Under the 1 in 2,000 AEP event there are two habitable or commercial buildings that are impacted by a change in flood extent and were identified as 'dry' in the Existing Case scenario and 'now wet' in the Developed Case.

Table 18.4 provides a summary of the FSRs where exceedances are predicted.

Table 18.4 Local catchment drainage—Estimated impacts to peak water levels at flood sensitive receptors

FSR ID	Description	FIO target (up to the 1% AEP event) (mm)	FIO target (1 in 2,000 AEP event) (mm)	Afflux (mm)					
				20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 2,000 year
CON_ID_392	Shed	100	250	-	-	-	-	-	+9 ¹
GC6_ID_3	House	10	250	-	-	-	-	-	+2 ¹

Table note:

1 Newly wet FSRs are considered an impact by the FIOs

There are no FSRs subject to a change of hazard (velocity-depth product) greater than the FIO target (10 per cent) for all modelled events up to and including the 1% AEP event.

The change in duration of inundation has not been assessed for the FSRs, as details of floor levels need to be confirmed through survey during detailed design.

Mitigation of impacts on FSRs is individual to each landowner and will require ongoing consultation with those impacted to agree on appropriate mitigation specific to each site. ARTC will continue through the next stages of design to engage with impacted landowners to negotiate site-specific mitigation measures.

18.6.3.2 Flood impacts on State-controlled roads

Within the extent of the local catchment hydraulic models, a total of five State-controlled roads are expected to be influenced by the Project, including the Cunningham Highway, Yelarbon-Keetah Road, Millmerran-Inglewood Road, Oakey Pittsworth Road, Gore Highway and Toowoomba Cecil Plains Road. In total, 21 individual segments of the below roads may be subject to impact, with the majority being associated with Millmerran-Inglewood Road.

Table 18.5 presents potential impacts on State-controlled roads along with justification and mitigation measures. A summary of key impacts (afflux, velocity, time of inundation and hazard), FIO exceedance justification and proposed mitigation measures are provided in the table. The proposed mitigation measures are to be further investigated in the following stage of the Project.

The below points summarise the key impacts on State-controlled roads:

- Cunningham Highway may be impacted at three locations, with a maximum afflux of 172 mm over a distance of 20 m. This increase in peak water levels (or afflux) is likely to be experienced in the vicinity of Yelarbon. An increase in time of inundation from 0.01 hours in the Existing Case to 0.09 hours in the Developed Case is predicted within the vicinity of Whetstone. The FIOs require this change to be expressed as a percentage, which translates into an increase of 875 per cent; however, as demonstrated above, the Developed Case time of inundation remains below 0.09 hours. Some minor changes in velocity and hazard are also predicted on varying segments of the Cunningham Highway.
- Yelarbon-Keetah Road may be impacted at one location with a maximum afflux of 162 mm over a distance of 118 m, as well as a maximum change in time of inundation of 713 per cent, from 0.01 hours in the Existing Case to 0.06 hours in the Developed Case (note the commentary regarding percentage increases above).
- Millmerran-Inglewood Road is expected to be impacted at 13 separate locations along the Project alignment, with a maximum afflux of 261 mm over a distance of 438 m (Road Object ID #21). A maximum increase of 800 per cent in time of inundation is predicted from 0.06 hours to 0.57 hours (Road Object ID #50). Undertaking a review of time of submergence of Millmerran-Inglewood Road in this location shows that the impacted segment actually experiences a decrease in overall time of submergence from the Existing Case (1.32 hours) to the Developed Case (0.94 hours). Due to the segment already being inundated in another location for longer than the predicted exceedance, it is not anticipated that the exceedance would impact the trafficability of this segment of road.
- Oakey-Pittsworth Road may be impacted at one location, with a maximum afflux of 79 mm over a distance of 115 m in the 1% AEP event. This segment of road also experiences changes in time of inundation, velocity and hazard.
- Toowoomba-Cecil Plains Road is predicted to be impacted at one location with maximum afflux of 91 mm over a distance of 65 m, and a maximum change in time of inundation of 200 per cent in the 2% AEP event.

Table 18.5 Local catchments - Summary of impacts on State-controlled roads

[illegible]

Road Object ID	Model ID	Road Segment	Road Name	Surface Type	Flood Impact Objective Target					AEP	Maximum FIO Exceedance					Length (m)				Exceedance Justification	Proposed Mitigation Measures ⁵	
					Change in Peak Water Level (mm)	Change in Time of Inundation (%)	Change in Velocity (%)	Change in Velocity (m/s)	Change in Hazard (%)		Change in Peak Water Level (mm)	Change in Time of Inundation (% Change) ²	Change in Velocity (%) ³	Change in Velocity (m/s)	Change in Hazard (%) ³	Afflux	Time on Inundation	Velocity	Hazard			
33	Model 12	16015103990000258645	Oakley - Pittsworth Road	Sealed	10	10	20	1	10		20%	45	34	-	0.56	-	65.00	32.50	10.00	-	> Maturity of current cross-drainage design is limited by available information (e.g. detailed survey) > Rail design constraints alter the existing flow regime. Government).	> Further cross-drainage optimization investigations are required during detailed design. > Stakeholder Consultation (Landowner/Government).
											10%	50	29	-	0.60	101	80.00	32.50	17.50	10.00		
											5%	59	27	58	0.62	93	90.00	32.50	17.50	17.50		
											2%	73	24	55	0.63	83	100.00	32.50	22.50	27.50		
											1%	79	23	53	0.63	72	115.00	30.00	25.00	27.50		
											0.05%	-	-	-	-	-	-	-	-	-		
											20%	-	16	-	-	-	-	5.00	-	-		
22	Model 13	16015103990000145813	Gore Highway	Sealed	10	10	20	1	10		10%	-	-	-	-	-	-	-	-	-	> Maturity of current cross-drainage design is limited by available information (e.g. detailed survey) > Rail design constraints alter the existing flow regime. Government).	> Further cross-drainage optimization investigations are required during detailed design. > Stakeholder Consultation (Landowner/Government).
											5%	-	-	-	-	-	-	-	-	-		
											2%	-	-	-	-	-	-	-	-	-		
											1%	-	-	-	-	-	-	-	-	-		
											0.05%	-	-	-	-	-	-	-	-	-		
											20%	-	-	-	-	-	-	-	-	-		
											10%	-	-	-	-	-	-	-	-	-		
10	Model 14	16015103990000366753	Toowoomba - Cecil Plains Road	Sealed	10	10	20	1	10		20%	50	149	-	-	-	47.50	7.50	-	Between chainage 195.0 km and 195.8 km, the proposed rail embankment is constructed in "cut". Due to this, the existing upstream overland flow path is required to be diverted to an adjacent culvert crossing (C195.98) in order to provide adequate drainage across the alignment. This diverted concentrated runoff discharging through Culvert C195.98 increases the amount of runoff flowing north and reaching Toowoomba - Cecil Plains Road which otherwise would flow along a longer reach where more floodplain storage is available. In addition, the proposed rail embankment impedes the existing overland flow regime (chainage 196.2 km to 196.8 km) diverting flow to the culvert structure C196.46. The resulting runoff flows downstream and is intercepted by the existing Toowoomba - Cecil Plains Road. The above alterations to the flood dynamics cause the increase in water level (Afflux) and time of inundation experienced at Toowoomba-Cecil Plains Road. The current maturity of both the proposed cross drainage, flow diversion and embankment design is the reason these exceedances still remain. Complete optimisation of the design has not yet been completed to date, only to a reference design level, and therefore further optimisation of cross drainage structures and placement needs to be undertaken in the detailed stage. In addition, the representation of existing cross drainage structures under Toowoomba-Cecil Plains Road is limited due to limited as-built information or detailed survey.	Due to the diversion of overland flow, it is expected that some impact may remain to Toowoomba-Cecil Plains Road post mitigation. The hydraulic model will be updated with detailed survey to capture existing culverts under Toowoomba-Cecil Plains Road which may improve conveyance across the road itself. Further optimisation of proposed cross drainage placement and flow diversion will also be undertaken in the detailed design stage. Ongoing consultation with the appropriate road manager will be undertaken in the detailed design stage to discuss any residual exceedances and agree on further appropriate mitigation measures.	
											10%	61	54	-	-	-	55.00	7.50	-			-
											5%	76	92	-	-	-	62.50	22.50	-			-
											2%	91	200	-	-	-	65.00	30.00	-			-
											1%	71	101	-	-	-	70.00	30.00	-			-
											0.05%	-	-	-	-	-	-	-	-			-
											20%	-	-	-	-	-	-	-	-			-

Table Notes

1. Time of Submergence has been measured across the full length of the road segment and does not correspond to the exact location of exceedance within that segment

2. Change in water level (afflux) is calculated based on the maximum water surface level difference between the existing and design case results, and does not consider the underlying topography. Depth values presented are based on the respective underlying topographies and in some cases do not align with afflux calculations. This can be attributed to interpolation calculations of results performed within the hydraulic model.

3. Percentage increases should be read in conjunction with existing and design case results. The FIOs require that change be presented in percentage change for time of inundation, velocity and hazard and can sometimes produce large increases when existing case results are significantly small. Some locations were identified to experience significant percentage increases however the "absolute" difference between Existing and Design results are relatively small.

4. The above table should be read in conjunction with change maps provided as part of Volume II and the Digital Platform.

5. The symbol "-" denotes that no change above Flood Impact Objectives has been determined and therefore no result has been presented.

6. Changes recorded above may only occur during a small fraction of model simulation time.

7. Three decimal places do not infer absolute accuracy in modelling results, but it is necessary to identify small changes.

8. Design optimisations that could be implemented (but not limited to) include adjustment of culvert spacing, realignment of culverts to existing flowpaths when more detailed survey is available, modifying the grade of culverts, road height and alignment adjustments, removal of bridge abutments from waterway extents etc

Depth (m) ²		Time of Inundation in hours (related to % exceedance)		Time of Submergence (hours) ²		Velocity in m/s (related to % exceedance)		Velocity in m/s (related to actual change exceedance)		Hazard in m/s (related to % exceedance)	
Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case
0.022	0.033	0.42	0.56	2.11	1.25	-	-	0.660	1.222	-	-
-	0.050	0.49	0.64	2.23	1.38	-	-	0.747	1.347	0.09	0.18
-	0.059	0.52	0.67	2.29	1.43	1.016	1.604	0.802	1.426	0.11	0.21
0.107	0.006	0.58	0.72	2.38	1.52	1.098	1.701	0.885	1.514	0.09	0.16
0.009	0.012	0.60	0.74	2.43	1.55	1.148	1.753	0.935	1.563	0.11	0.18
-	-	-	-	-	-	-	-	-	-	-	-
-	-	0.22	0.26	2.57	2.56	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
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-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
-	0.050	0.67	1.67	2.63	2.60	-	-	-	-	-	-
-	0.061	1.39	2.15	2.88	2.95	-	-	-	-	-	-
-	0.076	0.87	1.67	2.82	2.92	-	-	-	-	-	-
-	0.091	0.46	1.37	3.11	3.33	-	-	-	-	-	-
0.070	0.104	0.76	1.52	3.14	3.25	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-

18.6.3.3 Flood impacts on local public roads

A total of 34 roads, streets or tracks (63 total segments) were identified to be potentially impacted by the Project, within the domain of the local catchment hydraulic models. Table 18.6 presents a summary of these impacts, including a summary of key impacts (afflux, velocity, time of inundation and hazard), FIO exceedance justification and proposed mitigation measures.

The below points summarise the key impacts on public roads:

- The maximum change in peak water level (afflux) is 2.16 m, over Draper Road. The impacted road segment is within the Project boundary where further mitigation and optimisation of drainage design and road/rail interface will occur. For all other road segments, the maximum anticipated afflux is 452 mm at McEwan Lane.
- The maximum percentage change in time of inundation is an increase from 0.05 hours in the Existing Case to 1.21 hours, resulting in a 2,323 per cent increase, in the Developed Case for the 1% AEP event over Longhurst Road. As significant percentage increase results even though the actual change is relatively small of 1.16 hours; therefore, the percentage increase could be considered somewhat academic. A total of six road segments were identified where the estimated percentage change in time of inundation is greater than 1,000 per cent, as outlined in Table 18.6.
- The maximum change in time of inundation is 5.19 hours (over Yelarbon Kurumbul Road). Concentrated runoff directly discharges towards the impacted road section in the revised reference design, which is just outside the Project corridor. The proposed rail culverts at this location are to be further optimised in the detailed design stage.
- The maximum percentage change in velocity is estimated at 109 per cent over Yelarbon Kurumbul Road, increasing from 0.51 m/s in the Existing Case to 1.07 m/s in the Developed Case. Within the revised reference design, concentrated runoff directly discharges towards this impacted road section, which is just outside the Project corridor. The proposed rail culverts at this location are to be further optimised in the detailed design stage.
- The maximum change in hazard is over Yelarbon Kurumbul Road, with an increase expected from 0 m²/s in the Existing Case to 0.20 m²/s in the Developed Case, which results in a large percentage increase across an estimated length of 72.5 m. Within the revised reference design, concentrated runoff directly discharges towards the impacted road section which is just outside the Project corridor. The proposed rail culverts at this location are to be further optimised in the detailed design stage.

Table 18.6 Local catchments - Summary of impacts on local public roads

Road Object ID	Model ID	Road Segment	Road Name	Surface Type	Flood Impact Objective Target					AEP	Maximum FIO Exceedance					Length (m)				Exceedance Justification	Proposed Mitigation Measures ⁸	Depth (m) ⁷		Time of Inundation in hours (related to % exceedance)		Time of Submergence (hours) ⁷		Velocity in m/s (related to % exceedance)		Velocity in m/s (related to actual change in exceedance)		Hazard in m/s ² (related to % exceedance)																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																		
					Change in Peak Water Level (mm)	Change in Time of Inundation (%)	Change in Velocity (%)	Change in Velocity (m/s)	Change in Hazard (%)		Change in Peak Water Level (mm)	Change in Time of Inundation (% Change)	Change in Velocity (%) ¹	Change in Velocity (m/s)	Change in Hazard (%) ¹	Afflux	Time on Inundation	Velocity	Hazard			Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																	
184	Model 2	1623655352000000000	Yelarbon Kurumbul Road	Unsealed	10	10	10	0.5	10	20%	53	923	-	0.28	-	90.00	32.50	42.50	In the existing case, flow veers over the existing rail embankment and is distributed across a large area directly upstream of Yelarbon Kurumbul Road. In the design case, the proposed rail embankment causes upstream ponding and concentrates runoff through culverts C22.43 and C23.05. This concentration of flow through structures increases the water level (afflux), time of inundation, and velocity on Yelarbon Kurumbul Road. This road runs parallel to the embankment and is directly downstream (within 20m) of the alignment. The exceedances remain due to the current design being at a reference design stage and culvert placement and spacing being grouped (rather than spaced across the embankment), resulting in concentrated flow. Further iteration of cross drainage and placement needs to be undertaken at detailed design stage. It is anticipated that a reduction in FIO exceedance can occur through mitigation, although some areas of exceedance may remain due to the presence of the project. In addition, the maturity of the existing cross-drainage is limited by available information along Yelarbon Kurumbul Road (e.g. detailed survey). A review of the time of submergence across the length of this road segment demonstrates that for all modelled events a reduction in time of submergence occurs from the existing case to the design case, improving accessibility along this road segment.	Model improvements to be undertaken in detailed design to include detailed survey of existing infrastructure. Further optimisation of design cross drainage will be undertaken in the detailed design phase to mitigate these exceedances, including alteration to culvert location and spread, culvert grade and invert levels and increase in culvert cell numbers. Appropriate consultation with the road manager will be undertaken to discuss residual exceedances and agree on further mitigation measures should exceedances not be designed out in the detailed design phase.	0.055	0.108	0.21	2.15	5.28	2.91	-	-	0.404	0.684																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																				
										10%	56	1067	-	0.30	-	90.00	56.00	57.50			0.062	0.118	0.18	2.10	5.69	3.29	-	-	0.345	0.641																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																				
										5%	60	717	-	0.31	-	90.00	77.50	67.50			0.067	0.128	0.26	2.10	6.02	3.49	-	-	0.468	0.781																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																				
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										1%	66	380	65	0.32	-	85.00	92.50	75.00			0.078	0.143	0.58	2.81	6.30	3.79	0.505	0.833	0.444	0.765																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																				
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										20%	168	17	-	-	408	75.00	2.50	-			17.50	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	0.03	0.16																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																										
10%	129	22	-	-	316	55.00	7.50	-	20.00	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	0.04	0.18																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																						
189	Model 2	1623655352000000043	Yelarbon Kurumbul Road	Sealed	10	10	20	1	10	20%	139	66	-	-	28.00	25.00	40.00	1.39	1.25	6.52	> Maturity of current cross-drainage design is limited by available information (e.g. detailed survey). > Rail design constraints alter the existing flow regime.	> Further cross-drainage optimization investigations are required during detailed design. > Stakeholder Consultation (Landowner/ Government).	0.123	4.01	4.90	6.48	5.96	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-</

Road Object ID	Model ID	Road Segment	Road Name	Surface Type	Flood Impact Objective Target					AEP	Maximum FIO Exceedance					Length (m)				Exceedance Justification	Proposed Mitigation Measures ⁸		
					Change in Peak Water Level (mm)	Change in Time of Inundation (%)	Change in Velocity (%)	Change in Velocity (m/s)	Change in Hazard (%)		Change in Peak Water Level (mm)	Change in Time of Inundation (% Change) ³	Change in Velocity (%)	Change in Velocity (m/s)	Change in Hazard (%) ⁴	Afflux	Time on Inundation	Velocity	Hazard				
667	Model 11	16015103990000564254	Yamaries Road	Sealed	10	10	20	1	10		20%	-	-	24	-	-	-	5.00	-	-	-	> Rail design constraints alter the existing flow regime.	> Further cross-drainage optimization investigations are required during detailed design. > Stakeholder Consultation (Landowner/ Government).
											10%	-	-	42	-	-	12.50	20.00	-	-			
											5%	-	-	37	-	-	42.50	22.50	-	-			
											2%	-	-	130	-	-	60.00	17.50	-	-			
											1%	-	-	-	-	-	-	27.50	-	-			
											0.05%	-	-	-	-	-	-	-	-	-			
0	Model 11	16015103990000246633	Murlaggan Road	Unsealed	10	10	10	0.5	10		20%	-	-	11	-	-	-	2.50	-	-	-	> Rail design constraints alter the existing flow regime.	> Further cross-drainage optimization investigations are required during detailed design. > Stakeholder Consultation (Landowner/ Government).
											10%	-	-	39	-	-	10.00	5.00	-	-			
											5%	-	-	12	-	-	15.00	10.00	-	-			
											2%	-	-	18	-	-	35.00	12.50	2.50	2.50			
											1%	-	-	18	-	-	52.50	12.50	-	12.50			
											0.05%	-	-	-	-	-	-	-	-	-			
119	Model 11	16015103990000218054	Mann Silo Road	Sealed	10	10	20	1	10		20%	-	-	-	-	-	-	-	-	-	-	> Rail design constraints alter the existing flow regime.	> Further cross-drainage optimization investigations are required during detailed design. > Stakeholder Consultation (Landowner/ Government).
											10%	-	-	-	-	-	-	-	-	-			
											5%	-	-	13	-	-	-	2.50	-	-			
											2%	-	-	12	-	-	-	2.50	-	-			
											1%	-	-	15	-	-	-	2.50	-	-			
											0.05%	-	-	-	-	-	-	-	-	-			
182	Model 11	16015103990000047430	Brookstead Scrubby Road	Sealed	10	10	20	1	10		20%	-	-	-	-	-	-	-	-	-	-	> Maturity of current cross-drainage design is limited by available information (e.g. detailed survey). > Rail design constraints alter the existing flow regime.	> Further cross-drainage optimization investigations are required during detailed design. > Stakeholder Consultation (Landowner/ Government).
											10%	-	-	-	-	-	-	-	-	-			
											5%	-	-	13	-	-	-	2.50	-	-			
											2%	-	-	12	-	-	-	2.50	-	-			
											1%	-	-	15	-	-	-	2.50	-	-			
											0.05%	-	-	-	-	-	-	-	-	-			
0	Model 11	1601510399000318767	Rocher Road	Unsealed	10	10	10	0.5	10		20%	-	-	-	-	-	-	-	-	-	-	> Rail design constraints alter the existing flow regime.	> Further cross-drainage optimization investigations are required during detailed design. > Stakeholder Consultation (Landowner/ Government).
											10%	-	-	-	-	-	-	-	-	-			
											5%	-	-	-	-	-	-	-	-	-			
											2%	-	-	-	-	-	-	-	-	-			
											1%	-	-	11	-	-	-	2.50	-	-			
											0.05%	-	-	-	-	-	-	-	-	-			
13	Model 12	1601510399000207475	Lochaber Road	Sealed	10	10	20	1	10		20%	-	-	46	-	-	5.00	7.50	-	-	> Maturity of current cross-drainage design is limited by available information (e.g. detailed survey). > Rail design constraints alter the existing flow regime.	> Further cross-drainage optimization investigations are required during detailed design. > Stakeholder Consultation (Landowner/ Government).	
											10%	-	-	84	-	-	7.50	12.50	-	-			
											5%	-	-	209	-	-	87.50	12.50	-	-			
											2%	-	-	161	-	-	157.50	15.00	-	-			
											1%	-	-	193	-	-	170.00	17.50	-	-			
											0.05%	-	-	-	-	-	-	-	-	-			
42	Model 12	1601510399000226639	McEwan Lane	Sealed	10	10	20	1	10		20%	-	-	426	-	-	897	152.50	82.50	-	30.00	Exceedances result on McEwan Lane due to the maturity of both the proposed road and rail cross drainage design. Upstream of McEwan Lane, the proposed rail embankment impedes the existing overland flow regime diverting flow to the culvert structures and the bridge opening. The resulting flow propagates downstream and is intercepted by the proposed McEwan Lane upgrade. Exceedances recorded are based on the centreline and levels of the existing McEwan Lane. 35.00 The exceedances remain on this road due to the current level of design performed, to a reference design stage, and full optimisation of the road/rail/cross drainage interface not being completed. In addition, the representation of road cross drainage within the hydraulic model is also limited at this stage of design. Further iteration of cross drainage and placement needs to be undertaken at detailed design stage, to optimise culvert placement and diversion of flow through the structures. As this road is proposed to be upgraded in the location of exceedance, it is anticipated this exceedance can be mitigated in the detailed design stage of the project. 32.50	To mitigate these exceedances, a review of the McEwan Lane upgrade with being undertaken and modifications to the hydraulic made to ensure adequate cross drainage under the road is proposed to prevent impacts to the road. In addition, further optimisation of the rail cross drainage design will be undertaken including the addition of cross-drainage under Lochaber Road, alteration to culvert location and spread, culvert grade and invert levels, increase in culvert cell numbers and scour protection. A review of the location of the bridge abutment will also be undertaken in detailed design to potentially reduce FIO exceedances. Stakeholder Consultation with the appropriate road Authority will be undertaken should residual exceedances remain on McEwan Lane.
											10%	-	-	452	-	-	496	157.50	87.50	-	35.00		
											5%	-	-	411	-	-	319	162.50	102.50	-	35.00		
											2%	-	-	380	-	-	203	162.50	115.00	-	32.50		
											1%	-	-	358	-	-	140	162.50	112.50	-	32.50		
											0.05%	-	-	-	-	-	-	-	-	-			
71	Model 12	1601510399000365665	Tip Road	Unsealed	10	10	10	0.5	10		20%	-	-	-	-	-	-	-	-	-	-	> Rail design constraints alter the existing flow regime.	> Further cross-drainage optimization investigations are required during detailed design. > Stakeholder Consultation (Landowner/ Government).
											10%	-	-	-	-	-	-	-	-	-			
											5%	-	-	-	-	-	0.00	-	-	-	2.50		
											2%	-	-	-	-	-	-	-	-	-			
											1%	-	-	-	-	-	-	-	-	-			
											0.05%	-	-	-	-	-	-	-	-	-			
27	Model 13	1601510399000205571	Linhope Road	Sealed	10	10	20	1	10		20%	-	-	-	-	-	-	-	-	-	-	> Maturity of current cross-drainage design is limited by available information (e.g. detailed survey). > Rail design constraints alter the existing flow regime.	> Further cross-drainage optimization investigations are required during detailed design. > Stakeholder Consultation (Landowner/ Government).
											10%	-	-	-	-	-	-	-	-	-			
											5%	-	-	12	-	-	-	2.50	-	-			
											2%	-	-	-	-	-	-	-	-	-			
											1%	-	-	-	-	-	-	-	-	-			
											0.05%	-	-	-	-	-	-	-	-	-			
30	Model 13	1601510399000205580	Linhope Valley Road	Sealed	10	10	20	1	10		20%	-	-	-	-	-	-	-	-	-	-	> Maturity of current cross-drainage design is limited by available information (e.g. detailed survey). > Rail design constraints alter the existing flow regime.	> Further cross-drainage optimization investigations are required during detailed design. > Stakeholder Consultation (Landowner/ Government).
											10%	-	-	-	-	-	-	-	-	-			
											5%	-	-	21	-	-	-	2.50	-	-			
											2%	-	-	-	-	-	-	-	-	-			
											1%	-	-	-	-	-	-	-	-	-			
											0.05%	-	-	-	-	-	-	-	-	-			
3	Model 16	1601510399000105107	Draper Road	Unsealed	10	10	10	0.5	10		20%	-	-	1667	-	-	-	47.50	-	-	-	> Rail design constraints alter the existing flow regime. > Localised depression causing ponding	> Further cross-drainage optimization investigations are required during detailed design. > Stakeholder Consultation (Landowner/ Government).
											10%	-	-	1886	-	-	-	52.50	-	-			
											5%	-	-	2027	-	-	-	52.50	-	-			
											2%	-	-	2160	-	-	57.50	57.50	-	-			
											1%	-	-	2160	-	-	-	57.50	-	-			
											0.05%	-	-	-	-	-	-	-	-	-			

18.6.3.4 Flood impacts on existing rail infrastructure

The impacted sections of the existing rail is either incorporated into the Project, with a new 1% AEP flood immunity, or proposed to be decommissioned once the proposed Project is operational. Table 18.7 presents potential impacts (afflux, velocity, time of inundation and hazard) on existing rail infrastructure along with FIO exceedance justification and mitigation measures. The Toowoomba to Thallan and Millmerran Branch lines have all been identified as being impacted by the Project.

Table 18.7 Local catchments - Summary of impacts on existing rail infrastructure

Route Name	Section Name	Status	Flood Impact Objective Target					AEP	Maximum FIO Exceedance					Length (m)				Exceedance Justification	Proposed Mitigation Measures ⁷
			Change in Peak Water Level (mm)	Change in Time of Inundation (%)	Change in Velocity (%)	Change in Velocity (m/s)	Change in Hazard (%)		Change in Peak Water Level (mm)	Change in Time of Inundation (% Change)	Change in Velocity (%)	Change in Velocity (m/s)	Change in Hazard (%)	Afflux	Time on Inundation	Velocity	Hazard		
4911	Toowoomba to Thallon	Operational	10	10	10	0.5	10	20%	-	-	-	-	-	-	-	-	-	> Rail design constraints alter the existing flow regime.	> This portion of rail is to be removed by the Project
								10%	26	-	-	-	-	7.5	-	-	-		
								5%	41	-	-	-	-	7.5	-	-	-		
								2%	51	-	-	-	-	7.5	-	-	-		
								1%	62	-	-	-	-	7.5	-	-	-		
								0.05%	-	-	-	-	-	-	-	-	-		
								20%	-	-	-	-	-	-	-	-	-		
7505	Millmerran Branch	Operational	10	10	10	0.5	10	10%	-	-	-	-	-	-	-	-	-	> Rail design constraints alter the existing flow regime.	> This portion of rail is to be removed by the Project
								5%	-	-	-	-	-	-	-	-	-		
								2%	35	-	-	-	-	5.0	-	-	-		
								1%	39	-	-	-	-	7.5	-	-	-		
								0.05%	-	-	-	-	-	-	-	-	-		
								20%	-	-	-	-	-	-	-	-	-		
								10%	-	-	-	-	-	-	-	-	-		

- Table Notes
1. Time of Submergence has been measured across the full length of the road segment and does not correspond to the exact location of exceedance within that segment
 2. Change in water level (afflux) is calculated based on the maximum water surface level difference between the existing and design case results, and does not consider the underlying topography. Depth values presented are based on the respective underlying topographies and in some cases do not align with afflux calculations. This can be attributed to interpolation calculations of results performed within the hydraulic model.
 3. The above table should be read in conjunction with change maps provided as part of Volume II and the Digital Platform.
 4. The symbol "-" denotes that no change above Flood Impact Objectives has been determined and therefore no result has been presented.
 5. Changes recorded above may only occur during a small fraction of model simulation time.
 6. Three decimal places do not infer absolute accuracy in modelling results, but it is necessary to identify small changes.
 7. Design optimisations that could be implemented (but not limited to) include adjustment of culvert spacing, realignment of culverts to existing flowpaths when more detailed survey is available, modifying the grade of culverts, road height and alignment adjustments, removal of bridge abutments from waterway extents etc

Depth (m) ²		Time of Inundation in hours (related to % exceedance)		Time of Submergence (hours) ³		Velocity in m/s (related to % exceedance)		Velocity in m/s (related to actual change exceedance)		Hazard in m ³ /s (related to % exceedance)	
Existing Case	Design Case	Existing	Design	Existing	Design	Existing	Design	Existing	Design	Existing	Design
-	-	-	-	-	-	-	-	-	-	-	-
0.096	0.003	-	-	-	-	-	-	-	-	-	-
0.073	0.004	-	-	-	-	-	-	-	-	-	-
0.077	0.004	-	-	-	-	-	-	-	-	-	-
0.062	0.005	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
-	0.015	-	-	-	-	-	-	-	-	-	-
-	0.020	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-

18.6.3.5 Flood impacts on private land outside the permanent footprint

In accordance with the FIOs, Table 18.8 presents the modelled changes and potential impacts on private land outside of the Project's permanent footprint. The table includes a summary of key impacts, FIO exceedance justification and proposed mitigation measures.

The below points summarise the key impacts on private land:

- 181 separate properties with individual land use categories (excluding watercourse) have been flagged as having a potential impact exceeding the FIO targets.
- The majority of all potential exceedances occurred in the Grains and Cattle Breeding land-use types.
- The maximum change in peak water level (afflux) is estimated at 2.07 m (Lot Plans 86MH675 and 79FTY1967). This large increase in afflux can be attributed to a lack of 'As-Built' information for inclusion of culverts under the Cunningham Highway; as such, no existing culverts are within the model leading to a large increase in water level in the Developed Case on this lot. During detailed design, the model will be updated, and 'as-built' information further sort, to reduce and mitigate this exceedance. The inclusion of existing culverts under the Cunningham Highway in this location is anticipated to result in a significant reduction of afflux on this lot. A total of 9 properties were identified where the maximum change in water surface level was greater than 1 m. A total of 16 properties were identified where the change in peak water level is between 0.5 m and 1.0 m. Generally, these impacts are localised on individual lots and will be considered further in detailed design.. It should be noted that a localised increase does not translate to a uniform flood level increases across the lot.
- The maximum change in velocity is expected to be around 3.1 m/s (Lot Plan 79FTY1967); however, the increase in velocity is due to a newly flood-affected cell being impacted, which can be attributed to a change in flood extent. Maximum existing flow velocity at a nearby modelling cell to the above location was estimated to be 2.28 m/s. A total of three properties were identified where the maximum change in velocity is expected to be greater than 2 m/s. A total of 39 properties were identified where the change in velocity is estimated between 1 m/s and 2 m/s, and a total of 40 properties were identified where the change in velocity is expected to be between 0.5 m/s and 1 m/s. The maximum percentage change in velocity is 253 per cent, increasing from 0.53 m/s in the Existing Case to 1.85 m/s in the Developed Case (Lot Plan 3385A341727). A total of six properties were identified where the maximum percentage change in velocity is greater than 100 per cent.
- The maximum change in time of inundation is estimated at 6.51 hours (Lot Pla: 413SP119197). A total of 15 properties were identified where the maximum change in time of inundation may be greater than 4 hours. The maximum percentage change in time of inundation is approximately 540 per cent (Lot Plan 8MH160). As noted previously, these large percentage increases should be interpreted within the context of the actual change and bearing in mind the cellular nature of hydraulic models.
- The maximum change in hazard occurs on Lot Plan 9MH365, increasing from 0 m²/s to 0.45 m²/s and is confined to a farm dam. This impact affects <1 per cent of the total lot area (impacting approximately 40 m²), and therefore can be considered localised. A total of 17 properties were identified where the maximum percentage change in hazard is greater than 1,000 per cent.

The dominant FIO exceedance was identified to be an increase in time of inundation across local drainage catchments. An increase in time of inundation generally means flood waters will be present longer than the current scenario, although given the short duration of local catchment flooding, increases experienced are unlikely to have material impacts to land use such as cropping.

Table 18.8 Local catchments – Summary of flood impacts on private land outside the permanent footprint

ID	ID (Lotplan DAFSR E) Culvert ID	Sensitive Receptor Type	Model ID	Catchment Type	Landuse	Lot Area (m2)	Flood Impact Objective Target						AEP	Maximum FIO Exceedance						Area (ha)²			Exceedance Justification	Proposed Mitigation Measures¹¹	Depth (m)¹²		Time of inundation in hours (related to % exceedance)		Time of inundation in hours (related to actual change exceedance)		Velocity in m/s (related to % exceedance)		Velocity in m/s (related to actual change exceedance)		Hazard in m²/s (related to % exceedance)																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																	
							Change in Peak Water Level (mm)	Change in Time of Inundation (%)	Change in Time of Inundation (Hours)	Change in Velocity (%)	Change in Velocity (m/s)	Change in Hazard (%)		Change in Peak Water Level (mm)	Change in Time of Inundation (%)	Change in Time of Inundation (Hours)	Change in Velocity (%)	Change in Velocity (m/s)	Change in Hazard (%)	Afflux	Time on inundation	Velocity			Hazard	Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																															
1	481SP119198	Lot Type Parcel	Model 2	Local	VACANT - LARGE HOUSESITE	45,385	10	5	1	10	0.5	10	20%	36	0.41	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-

ID	ID (Loplin 64FSR E) Culvert ID	Sensitive Receptor Type	Model ID	Catchment Type	Landuse	Lot Area (m2)	Flood Impact Objective Target						AEP	Maximum FIO Exceedance						Area (ha) ¹				Exceedance Justification	Proposed Mitigation Measures ¹⁶																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																												
							Change in Peak Water Level (mm)	Change in Time of Inundation (h)	Change in Time of Inundation (Hours)	Change in Velocity (%)	Change in Velocity (m/s)	Change in Hazard (%)		Change in Peak Water Level (mm)	Change in Time of Inundation (% Change) ¹⁷	Change in Time of Inundation (hours)	Change in Velocity (m/s) ¹⁸	Change in Velocity (m/s)	Change in Hazard (m/s) ¹⁹	Afflux	Time on inundation	Velocity	Hazard																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																														
43	19K3781	Lot Type Parcel	Model 2	Local	DWELLING - LARGE HOUSESITE	569	10	5	1	10	0.5	10	20%	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-

ID	ID (Lorplan ID/FSR ID/ Culvert ID)	Sensitive Receptor Type	Model ID	Catchment Type	Landuse	Lot Area (m2)	Flood Impact Objective Target						AEP	Maximum FIO Exceedance						Area (ha) ²			Exceedance Justification	Proposed Mitigation Measures ¹⁰																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																										
							Change in Peak Water Level (mm)	Change in Time of Inundation (hrs)	Change in Time of Inundation (Hours)	Change in Velocity (%)	Change in Velocity (m/s)	Change in Hazard (%)		Change in Peak Water Level (mm)	Change in Time of Inundation (% Change) ¹	Change in Time of Inundation (hours)	Change in Velocity (m/s) ²	Change in Velocity (m/s)	Change in Hazard (m/s) ²	Afflux	Time on Inundation	Velocity			Hazard																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																									
60	56P984730	Lot Type Parcel	Model 3	Local	GRASS	5,068,990	200	10	1	10	0.5	20	20%	1082	96	1.03	46	1.00	664	2.21	25.26	0.87	0.70	> Afflux exceedances can be attributed to a localised depression becoming inundated in the developed case scenario > Maturity of current cross drainage design is limited by available information (e.g. detailed survey) > Rail design constraints alter the existing flow regime. > The Flood Impact Objective exceedance has been identified due to a change in flood extent and a cell which was dry in the existing case and now flood-affected in the developed case > Percentage increases should be read in conjunction with existing and design case results. The FIOs require that change be presented in percentage change for time of inundation, velocity and hazard and can sometimes produce large increases when existing case results are significantly small. Some locations were identified to experience significant percentage increases however the 'absolute' difference between Existing and Design results are relatively small.	> Further cross-drainage optimisation investigations are required during detailed design > Stakeholder Consultation (Landowner/ Government).																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																									
							10%	1113	92	1.08	47	1.01	3126	1.14	21.58	0.83	0.61																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																	
							5%	957	49	1.09	51	1.05	4416	1.74	23.76	0.98	0.60																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																	
							2%	729	78	0.89	60	1.10	640	1.23	33.14	0.76	0.60																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																	
							1%	585	72	0.72	59	0.88	169	0.28	32.67	0.61	0.61																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																	
							0.05%	-	-	-	-	-	-	-	-	-	-	-	-	-	-																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																													
61	86M4675	Lot Type Parcel	Model 3	Local	CATTLE BREEDING & FATTENING	7,365,388	200	10	1	10	0.5	20	20%	985	75	1.75	25	1.05	384	1.18	20.68	1.07	0.73	> Afflux exceedances can be attributed to a lack of 'As-Built' information of existing infrastructure under the Currenham Highway > Maturity of current cross drainage design is limited by available information (e.g. detailed survey) > Rail design constraints alter the existing flow regime. > Identified property has multiple land use designations, or potentially incorrect land use designations assigned, resulting in a conservative FIO assessment. > The Flood Impact Objective exceedance has been identified due to a change in flood extent and a cell which was dry in the existing case and now flood-affected in the developed case > Percentage increases should be read in conjunction with existing and design case results. The FIOs require that change be presented in percentage change for time of inundation, velocity and hazard and can sometimes produce large increases when existing case results are significantly small. Some locations were identified to experience significant percentage increases however the 'absolute' difference between Existing and Design results are relatively small.	> Ongoing model strengthening and inclusion of 'As-Built' information will help reduce these exceedances > Further cross-drainage optimisation investigations are required during detailed design > Stakeholder Consultation (Landowner/ Government).																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																									
							10%	2070	82	1.83	64	1.33	14520	1.06	22.47	2.04	1.97																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																	
							5%	-	81	1.85	29	0.61	76	-	18.71	2.07	2.11																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																	
							2%	-	72	1.84	29	0.67	83	-	11.95	2.79	2.04																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																	
							1%	-	71	1.92	31	0.71	105	-	9.81	3.02	2.38																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																	
							0.05%	-	-	-	-	-	-	-	-	-	-	-	-	-	-																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																													
62	79FTY1967	Lot Type Parcel	Model 3	Local	0	7,365,388	20	10	1	10	0.5	10	20%	985	75	1.75	25	1.05	384	27.58	21.95	1.07	0.60	> Afflux exceedances can be attributed to a lack of 'As-Built' information of existing infrastructure under the Currenham Highway > Maturity of current cross drainage design is limited by available information (e.g. detailed survey) > Rail design constraints alter the existing flow regime. > Identified property has multiple land use designations, or potentially incorrect land use designations assigned, resulting in a conservative FIO assessment. > The Flood Impact Objective exceedance has been identified due to a change in flood extent and a cell which was dry in the existing case and now flood-affected in the developed case > Percentage increases should be read in conjunction with existing and design case results. The FIOs require that change be presented in percentage change for time of inundation, velocity and hazard and can sometimes produce large increases when existing case results are significantly small. Some locations were identified to experience significant percentage increases however the 'absolute' difference between Existing and Design results are relatively small.	> Ongoing model strengthening and inclusion of 'As-Built' information will help reduce these exceedances > Further cross-drainage optimisation investigations are required during detailed design > Stakeholder Consultation (Landowner/ Government).																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																									
							10%	2070	82	1.83	64	1.33	14520	27.29	23.99	2.04	2.11																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																	
							5%	344	81	1.85	29	0.61	76	26.76	28.17	2.07	2.68																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																	
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63	66M43454	Lot Type Parcel	Model 3	Local	ANIMALS-SPECIAL	634,157	200	10	1	10	0.5	20	20%	-	19	0.78	-	0.68	-	0.05	0.04	-	-	0.05	> Rail design constraints alter the existing flow regime. > Further cross-drainage optimisation investigations are required during detailed design > Stakeholder Consultation (Landowner/ Government).	> Further cross-drainage optimisation investigations are required during detailed design > Stakeholder Consultation (Landowner/ Government).																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																								
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64	72SPY3218	Lot Type Parcel	Model 3	Local	ANIMALS-SPECIAL	608,670	200	10	1	10	0.5	20	20%	-	59	0.87	-	0.55	-	0.25	0.16	-	-	0.05	> Rail design constraints alter the existing flow regime. > Further cross-drainage optimisation investigations are required during detailed design > Stakeholder Consultation (Landowner/ Government).	> Further cross-drainage optimisation investigations are required during detailed design > Stakeholder Consultation (Landowner/ Government).																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																								
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65	13M461	Lot Type Parcel	Model 3	Local	CATTLE BREEDING & FATTENING	2,514,887	200	10	1	10	0.5	20	20%	-	32	0.43	37	0.18	78	-	0.04	0.08	0.05	-	-	0.05	> Rail design constraints alter the existing flow regime. > Further cross-drainage optimisation investigations are required during detailed design > Stakeholder Consultation (Landowner/ Government).	> Further cross-drainage optimisation investigations are required during detailed design > Stakeholder Consultation (Landowner/ Government).																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																						
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66	70M43454	Lot Type Parcel	Model 3	Local	ANIMALS-SPECIAL	587,588	200	10	1	10	0.5	20	20%	-	0.07	8	0.03	0.03	0.00	-	-	0.03	0.00	-	-	0.05	> Rail design constraints alter the existing flow regime. > Further cross-drainage optimisation investigations are required during detailed design > Stakeholder Consultation (Landowner/ Government).	> Further cross-drainage optimisation investigations are required during detailed design > Stakeholder Consultation (Landowner/ Government).																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																						
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67	0	Watercourse	Model 3	Local	0	131,813	20	10	1	10	0.5	10	20%	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-

ID	ID (Loplin 84FSR E) Culvert ID	Sensitive Receptor Type	Model ID	Catchment Type	Landuse	Lot Area (m2)	Flood Impact Objective Target						AEP	Maximum FIO Exceedance						Area (ha) ¹			Exceedance Justification	Proposed Mitigation Measures ¹⁶																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																			
							Change in Peak Water Level (mm)	Change in Time of Inundation (hrs)	Change in Time of Inundation (hours)	Change in Velocity (%)	Change in Velocity (m/s)	Change in Hazard (%)		Change in Peak Water Level (mm)	Change in Time of Inundation (%) Change ¹	Change in Time of Inundation (hours)	Change in Velocity (m/s) ²	Change in Velocity (m/s)	Change in Hazard (%) ³	Afflux	Time on inundation	Velocity			Hazard																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																		
139	15P166689	Lot Type Parcel	Model 9	Local	GRAINS	1,198,121	200	10	1	10	0.5	20	20%	69	0.91	44	0.70	28	1.00	0.61	0.01	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-

[illegible]

18.6.3.6 Flow distribution

A key landowner concern is the potential change in flow distribution. To estimate the change in flow distribution, the maximum velocity-depth product modelling results were used to identify a change in unit flow between Existing and Developed Case conditions. Unit flow is a measure of discharge that is commonly used for assessing peak flow along cross-sections, or to identify the efficient flow corridors within a flow path.

The estimated flow distribution was filtered with a velocity-depth product cut-off of $0.15 \text{ m}^2/\text{s}$. The cut-off is in line with FIO requirements and was used to differentiate main channels (high flow zones) from shallow sheet flows (the assumed threshold for shallow sheet flow is $0.15 \text{ m}^2/\text{s}$).

The runoff interactions with various hydraulic controls govern the local flow distribution, including the rail embankment, cross-drainage, road levels, farm dams, channels, and floodplain storage. The following sections summarise the potential flow distribution impacts associated with each local hydraulic model.

General observations with respect to the impact of the Project on the flow distribution within local catchments include:

- The raised rail embankment restricts runoff and potentially causes ponding in some locations along the Project alignment. This results in an increased flow distribution impact upstream of the proposed rail embankment at these locations.
- At a majority of culvert crossing locations, the improved conveyance of the proposed culverts to cater for rare events reduces flow distribution impact at culverts upstream during more frequent events.
- At some locations along the Project alignment, the concentrated runoff at the outlets potentially increases flow distribution impact at culverts downstream.

Within local catchments, overland flow is considered a source of water supply for farm dams. Due to the reliance of landowners on this flow, an assessment of potential flow distribution impacts on farm dams has been provided in Table 18.9.

Chainage 33.30 km (NS2B) to 37.10 km (NS2B)

Small to medium flow distribution impacts (less than ± 10 per cent) were observed in all modelled events up to 1% AEP event. Moderate flow distribution impacts (greater than ± 20 per cent) were observed in the vicinity of culvert inlets/outlets and along the main flow paths.

The current modelling and assessment indicate no flow distribution impacts at the downstream model boundary.

Chainage 4.80 km to 33.60 km

Small-to-moderate flow distribution impacts (less than ± 10 per cent) were observed in all modelled events up to 1% AEP event. Localised larger flow distribution impacts (greater than ± 20 per cent) were observed in the vicinity of culvert inlets/outlets and along the main flow paths.

The current modelling and assessment indicate that there are no flow distribution impacts at the downstream model boundary.

Chainage 33.0 km to 55.30 km

Small-to-medium flow distribution impacts (less than ± 15 per cent) were observed in all modelled events up to 1% AEP event. Localised larger flow distribution impacts (greater than ± 20 per cent) were observed in the vicinity of culvert inlets/outlets and along the main flow paths.

The current modelling and assessment indicate that there are no flow distribution impacts at the downstream model boundary.

Chainage 56.1 km to 67.607 km

Small-to-medium flow distribution impacts (less than ± 10 per cent) were observed in all modelled events up to 1% AEP event. Larger localised flow distribution impacts (greater than ± 20 per cent) were observed in the vicinity of culvert inlets/outlets and along the main flow paths.

The current modelling and assessment indicate that there are no flow distribution impacts at the downstream model boundary.

Chainage 70.18 km to 86.1 km

Small-to-medium flow distribution impacts (less than ± 10 per cent) were observed in all modelled events up to 1% AEP event. Localised larger flow distribution impacts (greater than ± 20 per cent) were observed in the vicinity of culvert inlets/outlets and along the main flow paths.

The current modelling and assessment indicate that there are flow distribution impacts at the model downstream boundary. A review of the hydraulic model downstream boundary will be undertaken at detailed design to fully capture any flow distribution changes.

Chainage 86.5 km to 93.7 km

Moderate flow distribution impacts (less than ± 20 per cent) were observed in all modelled events up to the 1% AEP event. Similarly, moderate flow distribution impacts (less than ± 20 per cent) were observed in the vicinity of culvert inlets/outlets and along the main flow paths.

The current modelling and assessment indicates that there are flow distribution impacts at the model downstream boundary. A review of the hydraulic model downstream boundary will be undertaken at detailed design to fully capture any flow distribution changes.

Chainage 94.3 km to 103.8 km

Small to medium flow distribution impacts (less than ± 10 per cent) were observed in all modelled events up to 1% AEP event. Localised larger flow distribution impacts (greater than ± 20 per cent) were observed in the vicinity of culvert inlets/outlets and along the main flow paths.

The current modelling and assessment indicate that there are flow distribution impacts at the model downstream boundary. A review of the hydraulic model downstream boundary will be undertaken at detailed design to fully capture any flow distribution changes.

Chainage 104.30 km to 116.20 km

Moderate flow distribution impacts (less than ± 20 per cent) were observed in all modelled events up to 1% AEP event. Larger localised flow distribution impacts (greater than ± 20 per cent) were observed in the vicinity of culvert inlets/outlets and along the main flow paths.

The current modelling and assessment indicate that there are flow distribution impacts at the model downstream boundary. A review of the hydraulic model downstream boundary will be undertaken at detailed design to fully capture any flow distribution changes.

Chainage 115.10 km to 127.00 km

Small to medium flow distribution impacts (less than ± 10 per cent) were observed in all modelled events up to the 1% AEP event. Larger localised flow distribution impacts (greater than ± 20 per cent) were observed in the vicinity of culvert inlets/outlets and along the main flow paths.

The current modelling and assessment indicates that there are no flow distribution impacts at the downstream model boundary.

Chainage 128.20 km to 138.20 km

Small to medium flow distribution impacts (less than ± 10 per cent) were observed in all modelled events up to 1% AEP events. Larger localised flow distribution impacts (greater than ± 20 per cent) were observed in the vicinity of culvert inlets/outlets and along the main flow paths.

The current modelling and assessment indicates that there are no flow distribution impacts at the downstream model boundary.

Chainage 152.10 km to 165.80 km

Small flow distribution impacts less than ± 5 per cent) were observed in all modelled events up to 1% AEP event. Medium localised flow distribution impacts (greater than ± 10 per cent) were observed in the vicinity of culvert inlets/outlets and along the main flow paths.

The current modelling and assessment indicates that there are flow distribution impacts at the model downstream boundary. A review of the hydraulic model downstream boundary will be undertaken at detailed design to fully capture any flow distribution changes.

Chainage 166.70 km to 174.50 km

Small to medium flow distribution impacts (less than ± 10 per cent) were observed in all modelled events up to 1% AEP event. Larger localised flow distribution impacts (greater than ± 20 per cent) were observed in the vicinity of culvert inlets/outlets and along the main flow paths.

The current modelling and assessment indicate that there are flow distribution impacts at the model downstream boundary. A review of the hydraulic model downstream boundary will be undertaken at detailed design to fully capture any flow distribution changes.

Chainage 176.30 km to 188.00 km

Small to medium flow distribution impacts (less than ± 10 per cent) were observed in all modelled events up to 1% AEP event. Larger localised flow distribution impacts (greater than ± 20 per cent) were observed in the vicinity of culvert inlets/outlets and along the main flow paths.

The current modelling and assessment indicates that there are flow distribution impacts at the model downstream boundary. A review of the hydraulic model downstream boundary will be undertaken at detailed design to fully capture any flow distribution changes.

Chainage 186.25 km to 196.37 km

Small to medium flow distribution impacts (less than ± 10 per cent) were observed in all modelled events up to 1% AEP event. Larger localised flow distribution impacts (greater than ± 20 per cent) were observed in the vicinity of culvert inlets/outlets and along the main flow paths.

The current modelling and assessment indicates that there are no flow distribution impacts at the downstream model boundary.

Chainage 198.95 km to 202.25 km

Medium flow distribution impacts (less than ± 15 per cent) were observed in all modelled events up to 1% AEP event. Larger localised flow distribution impacts (greater than ± 20 per cent) were observed in the vicinity of culvert inlets/outlets and along the main flow paths.

The current modelling and assessment indicates that there are no flow distribution impacts at the downstream model boundary.

Chainage 205.40 km to 208.20 km

Small flow distribution impacts (less than ± 5 per cent) were observed in all modelled events up to the 1% AEP event. Larger localised flow distribution impacts (greater than ± 20 per cent) were observed in the vicinity of culvert inlets/outlets and along the main flow paths.

The current modelling and assessment indicates that there are no flow distribution impacts at the downstream model boundary.

Ongoing mitigation measures to reduce changes in flow distribution will occur during the detailed design stage and may include further refinement of the drainage control measures and additional surveys where necessary. Further details on proposed mitigation measures and proposed strategies to manage impacts can be found in Section 22. In addition, refined hydrologic and hydraulic models are proposed in subsequent stages to provide further details of the flow distribution.

Table 18.9 Local catchments—Impact on farm dams

Approximate Chainage (km)	Lot Plan	Commentary	Potential mitigation
36.60 (NS2B)	8SP146067	Flow distribution reduction <5% in the 1%, 2% and 5% AEP events	Distribution of culverts along the Project alignment are required to represent existing flow regimes.
36.20 (NS2B)	8SP146067	Flow distribution reduction >20% in the 1%, 2% and 5% AEP events	Distribution of culverts along the Project alignment are required to represent existing flow regimes.
34.50 (NS2B)	37MH878	This farm dam is located within the project corridor and will likely be required to be removed.	Ongoing consultation with the landowner regarding the impact to the farm dam.
5.90	11K5781	Minor impacts – Yelarbon – Kurumbul Road earthworks limit runoff reaching the farm dam.	Road design and associated earthworks are to be reviewed
6.40	28MH210	A minor reduction (<5%) in flow distribution during 20% AEP. No change to flow distribution for other modelled events up to the 1%AEP event.	The design culverts need to be distributed along the rail to depict the existing condition.
6.40	24MH143	Flow distribution reduction during 20% and 10% AEP events are >20% and >15%, respectively. A minor reduction (<5%) to the flow distribution during other modelled events up to the 1% AEP event.	The design culverts need to be distributed along the rail to depict the existing condition.
8.20	104MH143	Flow distribution reduction >20% during 5% and 2% AEP events. No impacts during other modelled events up to the 1% AEP event.	The design culverts need to be distributed along the rail to depict the existing condition. The balancing of culvert conveyance and the mitigation of impacts need to be assessed.
8.20	413SP119197	Flow distribution reduction >20% during all modelled events up to the 1% AEP event.	Distribution of culverts along the Project alignment are required to represent existing flow regimes.
8.50	54SP169191	Flow distribution reduction >20% during 5% and 2% AEP events. No impacts during other modelled events up to the 1% AEP event.	Distribution of culverts along the Project alignment are required to represent existing flow regimes.
12.50	71CP850191	Flow distribution reduction during 2% and 1% AEP events are <10% and 15%, respectively.	Distribution of culverts along the Project alignment are required to represent existing flow regimes.
13.00	70CP850191	Flow distribution reduction during 2% and 1% AEP events are <10% and 15%, respectively.	Distribution of culverts along the Project alignment are required to represent existing flow regimes.
18.50	41MH141	Flow distribution reduction <15% during 5%, 2%, and 1% AEP events.	Distribution of culverts along the Project alignment are required to represent existing flow regimes.
22.60	35MH150	Flow distribution reduction >20% during 1% AEP, no impacts during all the other modelled events up to the 1% AEP event.	Distribution of culverts along the Project alignment are required to represent existing flow regimes.
23.70	38MH150	Flow distribution reduction >20% during all modelled events up to the 1% AEP event.	Distribution of culverts along the Project alignment are required to represent existing flow regimes.
30.40	55MH233	Flow distribution reduction <10% during 1% AEP event, no impacts during all the other modelled events up to the 1% AEP event.	Distribution of culverts along the Project alignment are required to represent existing flow regimes.
30.40	78MH399	Flow distribution reduction during 2% and 1% AEP events are <5% and <15%, respectively.	Distribution of culverts along the Project alignment are required to represent existing flow regimes.
31.40	7MH160	Flow distribution reduction during 1% AEP event is <10%, for all other modelled events up to the 2% AEP is <5%.	Distribution of culverts along the Project alignment are required to represent existing flow regimes.
53.00	1MH171	Flow distribution reduction during 20%, 10% and 5% AEP events are <10%. A minor reduction (<5%) to the flow distribution during all the other modelled events up to 1% AEP event.	Distribution of culverts along the Project alignment are required to represent existing flow regimes.
63.90	78MA3480	Minor flow reduction (<5%) in all events up to the 1% AEP event.	Distribution of culverts along the Project alignment are required to represent existing flow regimes.
64.00	85MH62	Minor flow reduction (<5%) in the 10%, 5%, 2% and 1% AEP events, flow reduction <10% in the 20% AEP event.	Distribution of culverts along the Project alignment are required to represent existing flow regimes.
65.80	121MH94	Minor flow reduction (<5%) in all modelled events up to the 1% AEP event.	Distribution of culverts along the Project alignment are required to represent existing flow regimes.
66.60	87MH16	Minor flow reduction (<5%) in all modelled events up to the 1% AEP event.	Distribution of culverts along the Project alignment are required to represent existing flow regimes.
73.80	4MH75	Minor flow distribution reduction (<5%) in all modelled events up to the 1% AEP event.	Distribution of culverts along the Project alignment are required to represent existing flow regimes.
74.50	102MH643	Minor flow distribution reduction (<5%) in all modelled events up to the 1% AEP event.	Distribution of culverts along the Project alignment are required to represent existing flow regimes.
76.60	102MH643	Flow distribution reduction <10% in the 1%, 2% and 5% AEP events. Flow distribution reduction <5% in the 10% and 20% AEP events. This farm dam is located within the Project corridor and will likely be required to be removed.	Distribution of culverts along the Project alignment are required to represent existing flow regimes. Ongoing consultation with the landowner regarding the impact to the farm dam.
77.40	102MH643	Flow distribution reduction <10% in the 1% and 2% AEP events. Flow distribution reduction >20% in the 5% AEP event.	Distribution of culverts along the Project alignment are required to represent existing flow regimes.
79.50	2SP214649	This farm dam is located within the Project corridor and will likely be required to be removed.	Ongoing consultation with the landowner regarding the impact to the farm dam.
81.20	2SP214649	Flow reduction >20% in all modelled events up to the 1% AEP event. The farm dam needs to be removed, which is within the Project corridor.	Ongoing consultation with the landowner regarding the impact to the farm dam.
84.00	2SP214649	Flow distribution reduction of <20% and <15% in the 1% and 2% AEP events, respectively. No impacts in all the other modelled events up to the 2% AEP event.	Distribution of culverts along the Project alignment are required to represent existing flow regimes.
84.60	2SP214649	Minor flow distribution reduction (<5%) in all modelled events up to the 1% AEP event.	Distribution of culverts along the Project alignment are required to represent existing flow regimes.
101.00	147MA3431	Flow reduction <10% in the 1% AEP event. Flow reduction <5% in all modelled events up to the 1% AEP event.	Distribution of culverts along the Project alignment are required to represent existing flow regimes.
101.80	168MH367	No impacts during 20%, 10% and 5% AEP events. Flow reduction <5% in the 2% and 1% AEP events.	Distribution of culverts along the Project alignment are required to represent existing flow regimes.

Approximate Chainage (km)	Lot Plan	Commentary	Potential mitigation
101.90	168MH367	No impacts during 20%, 10% and 5% AEP events. Flow reduction <5% in the 2% and 1% AEP events.	Distribution of culverts along the Project alignment are required to represent existing flow regimes. Earthworks, including a table drain block are to be provided.
106.80	2RP145435	Flow distribution reduction <5% in the 2% and 1% AEP events.	Distribution of culverts along the Project alignment are required to represent existing flow regimes.
106.80	12MH368	Minor flow distribution reduction <5% in the 1% AEP event.	Distribution of culverts along the Project alignment are required to represent existing flow regimes.
108.00	9MH365	Flow distribution reduction >20% in the 10%, 5%, 2% and 1% AEP events. Flow distribution reduction <20% in the 20% AEP event.	Distribution of culverts along the Project alignment are required to represent existing flow regimes.
108.10	9MH365	Flow distribution reduction >20% in the 10%, 5%, 2% and 1% AEP events. Flow distribution reduction <20% in the 20% AEP event.	Distribution of culverts along the Project alignment are required to represent existing flow regimes.
108.20	9MH365	Flow distribution reduction >20% in the 10%, 5%, 2% and 1% AEP events. Flow distribution reduction <20% in the 20% AEP event.	Distribution of culverts along the Project alignment are required to represent existing flow regimes.
108.50	20MH363	Flow distribution reduction <5% in the 5% AEP event. No impact during all the other modelled events up to the 1% AEP event.	Distribution of culverts along the Project alignment are required to represent existing flow regimes.
108.80	9MH365	Flow distribution reduction <5% in the 20% AEP event. No impacts during other modelled events.	Distribution of culverts along the Project alignment are required to represent existing flow regimes.
110.20	21MH363	Flow distribution reduction >20% in the 20%, 10% and 5% AEP events. Flow distribution reduction <5% in the 2% AEP event.	Distribution of culverts along the Project alignment are required to represent existing flow regimes.
110.60	21MH363	Flow distribution reduction >20% in the 20% AEP event. Flow distribution reduction <10% in 10% AEP and 5% AEP events.	Distribution of culverts along the Project alignment are required to represent existing flow regimes.
110.70	6MH364	Flow distribution reduction <20% in the 5% AEP event. Flow distribution reduction <15% in the 20% AEP and 2% AEP events.	Distribution of culverts along the Project alignment are required to represent existing flow regimes.
111.80	6MH364	Flow distribution reduction <5% in the 20% AEP event.	Distribution of culverts along the Project alignment are required to represent existing flow regimes.
112.20	1RP125660	Flow distribution reduction <10% in the 1%, 2% and 5% AEP events. Flow distribution reduction <5% in the 10% and 20% AEP events.	Distribution of culverts along the Project alignment are required to represent existing flow regimes.
112.90	5SP194159	Flow distribution reduction <5% in the 1% AEP event.	Distribution of culverts along the Project alignment are required to represent existing flow regimes.
113.20	5SP194159	Flow distribution reduction <5% in the 1%, 2% and 5% AEP events.	Distribution of culverts along the Project alignment are required to represent existing flow regimes.
113.80	1SP204014	Flow distribution reduction <5% in the 20% AEP event. No impacts on all the other modelled events up to the 1% AEP event.	Distribution of culverts along the Project alignment are required to represent existing flow regimes.
117.40	99DY182	This farm dam is located within the project corridor and will likely be required to be removed.	Ongoing consultation with the land owner regarding the impact to the farm dam. Cross-drainage to be reviewed and optimised.
118.40	1SP166689	Flow distribution reduction <5% in all modelled events up to the 1% AEP event.	Distribution of culverts along the Project alignment are required to represent existing flow regimes.
119.00	2RP202866	Flow distribution reduction <5% in all modelled events up to the 1% AEP, except 10% AEP event.	Distribution of culverts along the Project alignment are required to represent existing flow regimes.
119.40	NA	Flow distribution reduction <5% in the 20% AEP event.	Distribution of culverts along the Project alignment are required to represent existing flow regimes.
119.40	NA	Flow distribution reduction <5% in the 20% AEP event.	Distribution of culverts along the Project alignment are required to represent existing flow regimes.
126.00	2DY1006	Flow distribution reduction <10% in the 2% and 1% AEP events.	Distribution of culverts along the Project alignment are required to represent existing flow regimes.
126.80	3RP233820	This farm dam is located within the Project corridor and will likely be required to be removed.	Ongoing consultation with the landowner regarding the impact to the farm dam.
129.00	8SP128192	Flow distribution reduction >20% in the 5% and 1% AEP events.	Distribution of culverts along the Project alignment are required to represent existing flow regimes.
160.50	2RP53657	Flow distribution reduction >20% during all modelled events up to the 1% AEP event. This farm dam is located within the project corridor and will likely be required to be removed.	Longitudinal and cross-drainages are to be reviewed and optimised. Ongoing consultation with the landowner regarding the impact to the farm dam.
161.60	11SP254402	Flow distribution reduction <20% in all modelled events up to the 1% AEP event.	Improve upstream culvert conveyance. Distribution of culverts along the Project alignment are required to represent existing flow regimes.
162.80	1RP7478	This farm dam is located within the project corridor and will likely be required to be removed.	Ongoing consultation with the landowner regarding the impact to the farm dam.
164.10	1RP7482	Flow distribution reduction <5% in all modelled events up to the 1% AEP event, except 20% AEP event (<10%).	Improve upstream culvert conveyance. Distribution of culverts along the Project alignment are required to represent existing flow regimes.
170.40	6RP7446	Minor flow reductions (<5%) in the 10% and 20% AEP events. No impacts during all the other modelled events up to the 1% AEP event.	Distribution of culverts along the Project alignment are required to represent existing flow regimes.
173.80	12RP212366	Flow reduction >20% in all modelled events up to the 1% AEP event. This farm dam is located within the project corridor and will likely be required to be removed.	Ongoing consultation with the landowner regarding the impact to the farm dam.

Approximate Chainage (km)	Lot Plan	Commentary	Potential mitigation
176.80	2RP142680	Flow reduction <5% in the 1% AEP event and no impacts during all the other modelled events up to the 1% AEP event.	Distribution of culverts along the Project alignment are required to represent existing flow regimes.
181.80	2RP110779	Flow reduction <10% in the 1% and 2% AEP events. Flow reduction <5% in all the other modelled events up to the 2% AEP event.	Distribution of culverts along the Project alignment are required to represent existing flow regimes.
181.80	1RP114861	Flow reduction <5% in the 1% and 2% AEP events.	Distribution of culverts along the Project alignment are required to represent existing flow regimes.
183.40	1495A34822	Flow reduction <5% in the 1% and 2% AEP events.	Distribution of culverts along the Project alignment are required to represent existing flow regimes.

18.7 Additional modelling items to be considered in detailed design

The following points summarise additional items to be included in the flood models for detailed design:

- Where information is available, additional existing QR cross-drainage infrastructure is to be included within the local drainage models
- Floor level survey (reduced level (RL), habitable floor level, etc.) of FSRs to better inform FIO exceedances, impacts and mitigation measures
- A review of hydrological inputs to further strengthen hydrologic estimates
- Flow lines (2d_po) to be added to models to provide further clarity on flow distribution within local models
- Where appropriate, additional infrastructure applicable to the Project, such as noise walls, will be added to the models
- For local flood events in catchments that contain large areas of irrigated land, sensitivity testing will be undertaken at future design stages to conservatively test the effects of fully saturated soil (i.e. zero rainfall losses), and to subsequently evaluate how the local drainage design performs in such a scenario.

Further details of mitigation measures can be found in Section 22. A review of proposed mitigation measures will be completed during the detailed design stage of the Project after hydraulic models have been updated to capture the above changes.

18.8 Planned and future roads

Consultation with DTMR identified future upgrades to several roads within the local catchment hydraulic model boundaries. These included:

- Gore Highway Safety Works—Brookstead Silos to Haremar Road
- Oakey-Pittsworth Road Future Proofing.

The details and construction timing of these road upgrades is not yet known and therefore these have not currently been considered in the hydraulic modelling completed for the revised draft EIS. As these projects progress further into their design stage, and potential construction timing is identified, ARTC will continue discussions with DTMR to better understand, and where appropriate, include these designs within the Project's detailed design hydraulic modelling.

Consultation with TRC has identified the following rehabilitation works being conducted within the vicinity of the Project:

- Hazelwood Road south of Southbrook
- Heckendorf Road approximately 1 km west of the Project alignment
- Nothdurft Road south of Yarranlea
- Vines Street at Pittsworth.

A review of the above has identified that Hazelwood Road and Heckendorf Road rehabilitation works are outside the local catchment hydraulic modelling domain, and therefore have not been considered. Nothdurft Road and Vines Street are both located within the hydraulic modelling domains; however, due to the minor nature of the works, these have not been considered within the current modelling. Consultation with GRC has not identified any planned or future road upgrades within the local catchment hydraulic modelling boundaries.

ARTC will continue to consult and engage with DTMR, TRC and GRC to determine any future or planned road projects that may need to be considered as part of detailed design hydraulic modelling.

19 Emergency access route assessment

Evacuation management of regional communities during disasters is a key measure for local councils in maintaining the safety of their communities. It is paramount that safe access and egress be maintained, where possible, to efficiently move residents throughout LGAs to places of safety during natural disasters.

The Project crosses two LGAs, GRC and TRC, with both having established Local Disaster Management Plans in place, administered by the Local Disaster Management Group (LDMG). The introduction of the Project within these LGAs has the potential to impact on planned access and egress routes during flood events. As such, consultation with each LDMG has taken place to determine current and planned evacuation routes during flood events, and an impact assessment performed on these routes utilising the feedback provided by councils.

19.1 Toowoomba Regional Council

Preliminary feedback from the TRC LDMG identified up to 32 potential evacuation centres (within the planning stage) across the region, with the selection of use during an emergency dependant on a range of factors including the type and scale of disaster. Due to variability of emergencies and evacuation routes selected, TRC LDMG identified key localities within the vicinity of the Project, and nominated evacuation mechanisms and potential evacuation centres for these localities. Feedback included:

- Millmerran, Pampas and surrounding localities—three potential evacuation centres exist based in the Millmerran township. Due to riverine flooding cutting the Gore Highway at Centenary Bridge (Condamine River) and surrounding lower-level roads, localities to the east of this would need to be directed to potential evacuation centres back towards Toowoomba.
- Brookstead—one potential evacuation centre located in this vicinity
- Pittsworth—two potential evacuation centres located in the township
- Southbrook—no evacuation centres present in the township, with evacuation to Pittsworth or Toowoomba being required
- Wellcamp—no evacuation centres present in the area, with evacuation to Toowoomba being required
- Gowrie Junction—one evacuation centre present within the area
- Gowrie Mountain—impact to Warrego Highway might have impact on any evacuations from the western localities into Toowoomba.

Based on the above feedback, a review of the State-controlled roads within and around these areas was completed and a flood impact assessment conducted to determine the effects of the Project on these routes.

19.1.1 Millmerran and Pampas

State-controlled roads within the vicinity of Millmerran and Pampas include the Gore Highway, Pampas-Horrane Road, Millmerran-Leyburn Road, Millmerran-Cecil Plains Road and Millmerran-Inglewood Road. A review of Existing Case flood modelling results shows the following immunity of these State-controlled roads:

- Gore Highway—generally a low immunity with portions being impacted by the 20% AEP event across the Condamine floodplain
- Pampas-Horrane Road—impacted by the 10% AEP event within the vicinity of the Project
- Millmerran-Leyburn Road—generally a low immunity with portions being impacted by the 20% AEP event
- Millmerran-Cecil Plains Road—generally a low immunity with portions being impacted by the 20% AEP event. This is confined to areas approximately 9 km downstream from Millmerran
- Millmerran-Inglewood Road—generally a low immunity with portions being impacted by the 20% AEP event.

An impact assessment was completed against the FIOs established in consultation with the Expert Flood Panel, and FIO exceedances were identified for the above roads identified in Table 19.1. Localities of FIO exceedances can be seen in Figure 7.54, Figure 8.3 and Figure 10.2. Note that FIO exceedances experienced within Local Drainage models have not been displayed on figures.

Table 19.1 Millmerran and Pampas emergency evacuation route impact assessment

Catchment	Road Object ID	Road Segment	Road Name	Surface Type	Flood Impact Objective Target					AEP	Maximum FIO Exceedance					Length (m)				Exceedance Justification	Proposed Mitigation Measures ⁴	Depth (m) ²	Time of Inundation in hours (related to % exceedance)		Time of Submergence (hours) ¹		Velocity in m/s (related to % exceedance)		Velocity in m/s (related to actual change exceedance)		Hazard in m ² s (related to % exceedance)			
					Change in Peak Water Level (mm)	Change in Time of Inundation (%)	Change in Velocity (%)	Change in Velocity (m/s)	Change in Hazard (%)		Change in Peak Water Level (mm)	Change in Time of Inundation (% Chance) ²	Change in Velocity (%)	Change in Velocity (m/s)	Change in Hazard (%)	Afflux	Time on Inundation	Velocity	Hazard				Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case		
Back Creek	3	16015103990000200000	Millmerran - Ingleswood Road	Sealed	10	10	20	1.0	10	20%	38	137	26	0.67	84	82.50	87.50	10.00	7.5	This road segment experiences increases in water level, time of inundation, velocity and hazard due to the proposed rail and road embankments constricting the existing flow paths and reducing the available storage. Bridge 310-BR37 abutment encroaches into the existing flow path causing a reduction in the waterway area and directing the runoff towards the bridge opening. In addition the introduction of the road upgrade along Millmerran-Ingleswood Road also reduces conveyance within the existing flowpath. This combination of these two changes results in an increase in water level (afflux), time of inundation, velocity and hazard on this road.	It is anticipated through optimisation of the bridge length and location of abutments as well as improvement in road drainage representation within the hydraulic model that a reduction in exceedances can be obtained. To further mitigate, a review of cross drainage culverts C126.77 and C126.81 will be undertaken to optimise conveyance through the embankment and prevent upstream ponding. In addition, the hydraulic model improvements will be undertaken in the detailed design stage to better represent road drainage around the Schwarten Road/Millmerran-Ingleswood Road interface.	0.045	0.020	4.45	10.53	13.21	17.83	1.06	1.34	0.82	1.49	0.09	0.16	
										10%	55	186	45	0.34	45	107.50	95.00	30.00	17.50	0.014		0.001	1.33	3.80	13.77	25.39	1.26	1.83	0.98	1.32	0.20	0.29		
										5%	172	309	40	0.53	152	190.00	127.50	32.50	25.00	0.026		0.002	0.42	1.71	19.04	22.27	1.05	1.47	0.89	1.41	0.07	0.16		
										2%	191	490	32	0.26	94	237.50	167.50	37.50	47.50	0.063		0.057	0.26	1.53	13.20	19.41	1.26	1.67	0.94	1.20	0.11	0.22		
										1%	189	388	34	-	105	225.00	152.50	32.50	47.50	0.057		0.050	0.40	1.93	12.92	19.08	1.15	1.53	-	-	0.10	0.20		
										0.05%	-	-	-	-	-	-	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-	-	-
										20%	-	-	-	-	-	-	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-	-	-
Back Creek	520	16015103990000100000	Gore Highway	Sealed	10	10	20	1.0	10	10%	23	25	-	-	-	5.00	2.50	-	-	- Rail design constraints alter the existing flow regime.	- Stakeholder Consultation (Landowner/ Government) - Ongoing model strengthening	-	0.023	0.23	0.29	6.79	6.80	-	-	-	-	-	-	
										5%	-	-	-	-	-	-	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-	-	
										2%	-	-	-	-	-	-	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-	-	
										1%	-	-	-	-	-	-	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-	-	
										0.05%	-	-	-	-	-	-	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-	-	
										20%	-	-	-	-	-	-	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-	-	
										10%	22	-	-	-	-	2.50	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-	-	-
Back Creek	1	16015103990000200000	Millmerran - Ingleswood Road	Sealed	10	10	20	1.0	10	5%	21	-	-	-	-	5.00	-	-	-	- Stakeholder Consultation (Landowner/ Government)	- Stakeholder Consultation (Landowner/ Government) - Ongoing model strengthening	0.003	0.002	-	-	-	-	-	-	-	-	-	-	
										2%	39	-	-	-	-	7.50	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-	-	
										1%	36	-	-	-	-	7.50	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-	-	
										0.05%	-	-	-	-	-	-	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-	-	
										20%	-	-	-	-	-	-	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-	-	
										10%	21	-	-	-	-	2.50	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-	-	
										5%	39	-	-	-	-	7.50	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-	-	
Back Creek	8	16015103990000200000	Millmerran - Ingleswood Road	Sealed	10	10	20	1.0	10	5%	35	-	-	-	-	5.00	-	-	-	- Stakeholder Consultation (Landowner/ Government)	- Stakeholder Consultation (Landowner/ Government)	-	0.035	-	-	-	-	-	-	-	-	-	-	
										2%	55	-	-	-	-	12.50	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-	-	
										1%	62	-	-	-	-	12.50	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-	-	
										0.05%	-	-	-	-	-	-	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-	-	
										20%	-	-	-	-	-	-	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-	-	
										10%	-	-	-	-	-	-	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-	-	
										5%	35	-	-	-	-	5.00	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-	-	
Back Creek	111	16015103990000000000	Charles Street	Sealed	10	10	20	1.0	10	5%	19	-	-	-	-	2.50	-	-	-	- Stakeholder Consultation (Landowner/ Government)	- Stakeholder Consultation (Landowner/ Government)	-	1.51	1.80	12.47	12.45	-	-	-	-	-	-	-	-
										2%	-	-	-	-	-	-	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-	-	
										1%	-	-	-	-	-	-	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-	-	
										0.05%	-	-	-	-	-	-	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-	-	
										20%	-	-	-	-	-	-	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-	-	
										10%	-	-	-	-	-	-	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-	-	
										5%	19	-	-	-	-	2.50	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-	-	
Back Creek	232	16015103990000100000	Gore Highway	Sealed	10	10	20	1.0	10	5%	15	-	-	-	-	2.50	-	-	-	- Stakeholder Consultation (Landowner/ Government)	- Stakeholder Consultation (Landowner/ Government)	-	3.32	3.82	20.42	20.42	-	-	-	-	-	-	-	-
										2%	34	-	-	-	-	10.00	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-	-	
										1%	-	-	-	-	-	-	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-	-	
										0.05%	-	-	-	-	-	-	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-	-	
										20%	-	-	-	-	-	-	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-	-	
										10%	-	-	-	-	-	-	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-	-	
										5%	15	-	-	-	-	2.50	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-	-	
Back Creek	233	16015103990000100000	Gore Highway	Sealed	10	10	20	1.0	10	5%	70	-	-	-	-	5.00	-	-	-	- Stakeholder Consultation (Landowner/ Government)	- Stakeholder Consultation (Landowner/ Government)	-	0.25	0.43	17.75	17.75	-	-	-	-	-	-	-	-
										2%	26	167	-	-	-	2.50	2.50	-	-	-		-	-	-	-	-	-	-	-	-	-	-	-	
										1%	-	-	-	-	-	-	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-	-	
										0.05%	-	-	-	-	-	-	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-	-	
										20%	-	-	-	-	-	-	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-	-	
										10%	-	-	-	-	-	-	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-	-	
										5%	26	167	-	-	-	2.50	2.50	-	-	-		-	-	-	-	-	-	-	-	-	-	-	-	
Bringally Creek	26	16015103990000232847	Millmerran - Ingleswood Road	Sealed	10	10	20	1.0	10	20%	-	18	-	-	-	-	2.50	-	-	This road section is subjected to exceedances due to the rail embankment impeding the existing overland flow regime increasing the water level upstream of the embankment which impacts Millmerran-Ingleswood Road. In the existing case, water sheet flows over Millmerran Ingleswood Road towards Bringally Creek. In the design case, flow is directed by the rail embankment towards new cross drainage structures (Ch199.7km – Ch100.0km) preventing this sheet flow. The increase in water levels and time of inundation at this location, are likely due to the efficiency of the proposed cross drainage under the rail embankment.	To mitigate the exceedances experienced on Millmerran-Ingleswood Road, a review of cross drainage design at Ch199.7km to Ch100.00km along with update of hydraulic models with more detailed survey will be undertaken in detailed design phase. Potential modifications to cross drainage design could include alteration to culvert location and spread and an increase in culvert coil numbers. Ongoing consultation with the Department of Transport and Main Roads will be undertaken during the detailed design stage to discuss any residual exceedances and agree on further mitigation measures should residual exceedances remain.	-	1.86	2.20	6.40	6.40	-	-	-	-	-	-	-	-
										10%	-	-	-	-	-	-	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-		
										5%	-	-	-	-	-	-	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-		
										2%	18	-	-	-	-	49	27.50	-	-	27.50		-	-	-	-	-	-	-	-	-	-	-	-	
										1%	95	68	-	-	25	187.50	15.00	-	65.00	-		-	-	-	-	-	-	-	-	-	-	-	-	
										0.05%	-	-	-	-	-	-	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-	-	
										20%	-	-	-	-	-	-	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-	-	
Bringally Creek	25	16015103990000232869	Millmerran - Ingleswood Road	Sealed	10	10	20	1.0	10	10%	-	-	-	-	-	-	-	-	-	- Maturity of current cross-drainage design is limited by available information (e.g. detailed survey)	-Further cross-drainage optimization investigations are required during detailed design.	-	-	-	-	-	-	-	-	-	-	-	-	
										5%	-	-	-	-	-	-	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-		
										2%	27	-	-	-	-	10.00	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-	-	
										1%	-	-	-	-	-	-	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-		
										0.05%	-	-	-	-	-	-	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-	-	
										20%	-	-	-	-	-	-	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-	-	
										10%	-	-	-	-	-	-	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-	-	
Bringally Creek	30	16015103990000232854	Millmerran - Ingleswood Road	Sealed	10	10	20	1.0	10	5%	-	-	-	-	-	-	-	-	-	- Maturity of current cross-drainage design is limited by available information (e.g. detailed survey)	-Further cross-drainage optimization investigations are required during detailed design.	-	-	-	-	-	-	-	-	-	-	-	-	
										2%	18	-	-	-	-	7.50	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-	-	
										1%	-	-	-	-	-	-	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-		
										0.05%	-	-	-	-	-	-	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-		
										20%	-	-	-	-	-	-	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-	-	
										10%	-	-	-	-	-	-	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-	-	
										5%	18	-	-	-	-	7.50	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-	-	
Bringally Creek	28	16015103990000232868	Millmerran - Ingleswood Road	Sealed	10	10	20	1.0	10	10%	-	-	-	-	-	-	-	-	-	- Maturity of current cross-drainage design is limited by available information (e.g. detailed survey)	-Further cross-drainage optimization investigations are required during detailed design.	-	-	-	-	-	-	-	-	-	-	-	-	
										5%	-	-	-	-	-	-	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-		
										2%	-	-	-	-	-	-	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-		
										1%	-	-	-	-	-	-	-	-	-	-		-	-	-	-	-	-	-	-	-	-	-		

Table 19.1 Millmerran and Pampas emergency evacuation route impact assessment

Catchment	Road Object ID	Road Segment	Road Name	Surface Type	Flood Impact Objective Target					AEP	Maximum FIO Exceedance					Length (m)				Exceedance Justification	Proposed Mitigation Measures ⁴
					Change in Peak Water Level (mm)	Change in Time of Inundation (%)	Change in Velocity (%)	Change in Velocity (m/s)	Change in Hazard (%)		Change in Peak Water Level (mm)	Change in Time of Inundation (% Chance) ²	Change in Velocity (% ³)	Change in Velocity (m/s)	Change in Hazard (% ³)	Afflux	Time on Inundation	Velocity	Hazard		
Condamine River	350	16015103990000270389	Pampas - Horrane Road	Sealed	10	10	10	0.5	10		20%	-	-	-	-	-	-	-	-	Lack of available data for model validation and ongoing model strengthening.	Ongoing model strengthening.
											10%	-	-	-	-	-	-	-	-		
											5%	-	38	-	-	-	60.00	-	-		
											2%	-	16	-	-	-	10.00	-	-		
											1%	-	-	-	-	-	-	-	-		
											0.05%	-	-	-	-	-	-	-	-		
Local Model 9	50	16015103990000232850	Millmerran - Inglewood Road	Sealed	10	10	20	0.5	10		20%	-	-	-	-	-	-	-	-	> Maturity of current cross-drainage design is limited by available information (e.g. detailed survey) > Rail design constraints after the existing flow regime. > Percentage increases should be read in conjunction with existing and design case results. The FIOs require that change be presented in percentage change for time of inundation, velocity and hazard and can sometimes produce large increases when existing case results are significantly small. Some locations were identified to experience significant percentage increases however the "absolute" difference between Existing and Design results are relatively small.	> Further cross-drainage optimization investigations are required during detailed design. > Stakeholder Consultation (Landowner/ Government).
											10%	49	-	-	-	-	22.50	-	-		
											5%	69	77	-	-	-	70.00	5.00	-		
											2%	75	799	-	-	-	97.50	15.00	-		
											1%	91	256	-	-	-	102.50	15.00	-		
											0.05%	-	-	-	-	-	-	-	-		
Local Model 9	52	16015103990000232864	Millmerran - Inglewood Road	Sealed	10	10	20	0.5	10		20%	30	10	-	-	-	25.00	2.50	-	> Maturity of current cross-drainage design is limited by available information (e.g. detailed survey) > Rail design constraints after the existing flow regime.	> Further cross-drainage optimization investigations are required during detailed design. > Stakeholder Consultation (Landowner/ Government).
											10%	30	56	-	-	-	60.00	10.00	-		
											5%	43	69	-	-	11	95.00	35.00	-		
											2%	116	260	-	-	11	177.50	62.50	-		
											1%	35	42	-	-	11	90.00	90.00	-		
											0.05%	-	-	-	-	-	-	-	-		
Local Model 9	9	16015103990000232860	Millmerran - Inglewood Road	Sealed	10	10	20	0.5	10		20%	-	-	-	-	-	-	-	-	> Maturity of current cross-drainage design is limited by available information (e.g. detailed survey) > Rail design constraints after the existing flow regime.	> Further cross-drainage optimization investigations are required during detailed design. > Stakeholder Consultation (Landowner/ Government).
											10%	-	20	-	-	-	5.00	-	-		
											5%	-	-	-	-	-	-	-	-		
											2%	58	20	-	-	-	5.00	7.50	-		
											1%	-	-	-	-	-	-	-	-		
											0.05%	-	-	-	-	-	-	-	-		
Local Model 9	54	16015103990000232849	Millmerran - Inglewood Road	Sealed	10	10	20	0.5	10		20%	-	-	-	-	-	-	-	-	> Maturity of current cross-drainage design is limited by available information (e.g. detailed survey) > Rail design constraints after the existing flow regime.	> Further cross-drainage optimization investigations are required during detailed design. > Stakeholder Consultation (Landowner/ Government).
											10%	-	-	-	-	-	-	-	-		
											5%	-	-	-	-	-	-	-	-		
											2%	15	-	-	-	-	2.50	-	-		
											1%	-	-	-	-	-	-	-	-		
											0.05%	-	-	-	-	-	-	-	-		
Local Model 7	21	16015103990000232869	Millmerran - Inglewood Road	Sealed	10	10	20	0.5	10		20%	79	-	-	-	16	152.50	-	-	> Maturity of current cross-drainage design is limited by available information (e.g. detailed survey) > Rail design constraints after the existing flow regime.	> Further cross-drainage optimization investigations are required during detailed design. > Stakeholder Consultation (Landowner/ Government).
											10%	115	49	-	-	60	240.00	65.00	-		
											5%	155	80	-	-	68	285.00	95.00	-		
											2%	214	187	-	-	108	387.50	112.50	-		
											1%	261	136	-	-	204	437.50	125.00	-		
											0.05%	-	-	-	-	-	-	-	-		
Local Model 7	25	16015103990000232854	Millmerran - Inglewood Road	Sealed	10	10	20	0.5	10		20%	-	-	-	-	-	-	-	-	> Maturity of current cross-drainage design is limited by available information (e.g. detailed survey) > Rail design constraints after the existing flow regime.	> Further cross-drainage optimization investigations are required during detailed design. > Stakeholder Consultation (Landowner/ Government).
											10%	46	120	-	-	-	7.50	5.00	-		
											5%	46	-	-	-	-	7.50	-	-		
											2%	16	-	-	-	-	7.50	-	-		
											1%	46	-	-	-	-	55.00	-	-		
											0.05%	-	-	-	-	-	-	-	-		
Local Model 7	19	16015103990000232853	Millmerran - Inglewood Road	Sealed	10	10	20	0.5	10		20%	-	-	-	-	-	-	-	-	> Maturity of current cross-drainage design is limited by available information (e.g. detailed survey) > Rail design constraints after the existing flow regime.	> Further cross-drainage optimization investigations are required during detailed design. > Stakeholder Consultation (Landowner/ Government).
											10%	-	-	-	-	-	-	-	-		
											5%	18	-	-	-	-	22.50	-	-		
											2%	32	-	-	-	-	37.50	-	-		
											1%	41	-	-	-	-	45.00	-	-		
											0.05%	-	-	-	-	-	-	-	-		
Local Model 8	11	16015103990000232851	Millmerran - Inglewood Road	Sealed	10	10	20	1.0	10		20%	62	-	-	-	-	5.00	-	-	> Maturity of current cross-drainage design is limited by available information (e.g. detailed survey) > Rail design constraints after the existing flow regime.	> Further cross-drainage optimization investigations are required during detailed design. > Stakeholder Consultation (Landowner/ Government).
											10%	-	-	-	-	-	-	-	-		
											5%	-	13	-	-	-	2.50	-	-		
											2%	-	-	-	-	-	-	-	-		
											1%	-	-	-	-	-	-	-	-		
											0.05%	-	-	-	-	-	-	-	-		
Local Model 8	2	16015103990000232858	Millmerran - Inglewood Road	Sealed	10	10	20	1.0	10		20%	-	-	-	-	-	-	-	-	> Maturity of current cross-drainage design is limited by available information (e.g. detailed survey) > Rail design constraints after the existing flow regime.	> Further cross-drainage optimization investigations are required during detailed design. > Stakeholder Consultation (Landowner/ Government).
											10%	-	-	-	-	-	-	-	-		
											5%	-	-	-	-	-	-	-	-		
											2%	10	-	-	-	-	2.50	-	-		
											1%	18	-	-	-	-	42.50	-	-		
											0.05%	-	-	-	-	-	-	-	-		

Table Notes

1. Time of Submergence has been measured across the full length of the road segment and does not correspond to the exact location of exceedance within that segment
- 2

The FIO assessment has identified potential impacts to the Gore Highway, Millmerran-Inglewood Road, Millmerran-Leyburn Road and Pampas-Horrane Road. Access to Millmerran from the south will generally be via Millmerran-Inglewood Road which is subject to exceedances in both regional and local flooding. Millmerran-Inglewood Road experiences flooding within the vicinity of Bringalily Creek and Back Creek catchments and generally has low-immunity in the Existing Case scenario. The majority of exceedances along Millmerran-Inglewood Road are due to an increase in duration of inundation.

Access to Millmerran from the north-east would generally be via the Gore Highway. The Gore Highway is impacted by flooding from both Back Creek and Condamine River in the Existing Case scenario and has low existing immunity across the Condamine floodplain. Some exceedances of the FIOs are experienced on the Gore Highway due to the presence of the Project and are generally related to afflux and change in duration of inundation. In instances where the Gore Highway is inaccessible due to flooding, evacuation of residents from Pampas should be considered closely to mitigate isolation due to flooding from both the Main Branch and North Branch of the Condamine River.

Although some impact to State-controlled roads due to the Project occurs, within the vicinity of the Millmerran and Pampas, existing road infrastructure currently has low immunity and is flooded during the Existing Case. It is not anticipated that the FIO exceedances identified will affect the trafficability of the road network within this region as generally the road network, as a whole, experiences access issues due to other low-lying areas (away from the Project alignment) preventing access.

19.1.2 Brookstead

State-controlled roads within the vicinity of Brookstead include the Gore Highway and Brookstead-Norwin Road. A review of Existing Case flood modelling results shows the following immunity of these State-controlled roads:

- Gore Highway—generally a low immunity, with portions being impacted by the 20% AEP event across the Condamine floodplain
- Brookstead-Norwin Road—generally a low immunity with portions being impacted by the 20% AEP event to the west of Brookstead.

An impact assessment was completed against the FIOs established in consultation with the Expert Flood Panel, and potential FIO exceedances were identified for the above roads identified in Table 19.2. Localities of FIO exceedances can be seen in Figure 7.54.

Table 19.2 Brookstead emergency evacuation route impact assessment

Catchment	Road Object ID	Road Segment	Road Name	Surface Type	Flood Impact Objective Target					AEP	Maximum FIO Exceedance					Length (m)				Exceedance Justification	Proposed Mitigation Measures ⁷
					Change in Peak Water Level (mm)	Change in Time of Inundation (%)	Change in Velocity (%)	Change in Velocity (m/s)	Change in Hazard (%)		Change in Peak Water Level (mm)	Change in Time of Inundation (%)	Change in Velocity (%)	Change in Velocity (m/s)	Change in Hazard (%)	Afflux	Time on Inundation	Velocity	Hazard		
Condamine River	331	16015103990000145822	Gore Highway	Sealed	10	10	20	1	10	20%	78	17	-	-	-	20.00	20.00	-	-	Exceedances occur along Gore Hwy due to the proposed rail embankment impeding floodwater and diverting flow to the bridge and culvert structures. The Gore Highway is located immediately downstream and runs parallel to the proposed rail embankment. The exceedances experienced on the highway are concentrated around the outlets of culverts C150.08 to C151.30 and adjacent bridge 310-BR27. Due to the presence of the embankment, flow is redirected to cross drainage structures resulting in more concentrated flowpaths than the existing case. A review of the time of submergence results in both the existing and design case for this road segment demonstrates that the localised increase in water level and time of inundation along this road due to the presence of the Project do not detrimentally impact the operational capacity of the road. The exceedances remain due to the current design being at a reference design stage and culvert placement and spacing being grouped (rather than spaced across the embankment), resulting in concentrated flow. Further iteration of cross drainage and placement needs to be undertaken at detailed design stage. Significant optimisation of the design around this location has been undertaken to date and therefore it is anticipated that there will be some residual exceedance on this road after design optimisation in the detailed design phase.	Further rail design optimisation investigations will be undertaken in detailed design, but it is anticipated residual exceedances will remain. In this case, discussions with the appropriate road authority and relevant stakeholders will be undertaken to discuss the impacts and their overall impact on the operational capacity of the road link.
										10%	177	216	-	-	-	70.00	30.00	-	-		
										5%	148	11	-	-	-	60.00	20.00	-	-		
										2%	148	12	-	-	16	140.00	20.00	-	140.00		
										1%	89	31	-	-	39	260.00	70.00	-	280.00		
										0.05%	-	-	-	-	-	-	-	-	-		

Table Notes

1. Time of Submergence has been measured across the full length of the road segment and does not correspond to the exact location of exceedance within that segment

2. Change in water level (afflux) is calculated based on the maximum water surface level difference between the existing and design case results, and does not consider the underlying topography. Depth values presented are based on the respective underlying topographies and in some cases do not align with afflux calculations. This can be attributed to interpolation calculations of results performed within the hydraulic model.

3. The above table should be read in conjunction with change maps provided as part of Volume II and the Digital Platform.

4. The symbol "-" denotes that no change above Flood Impact Objectives has been determined and therefore no result has been presented.

5. Changes recorded above may only occur during a small fraction of model simulation time.

6. Three decimal places do not infer absolute accuracy in modelling results, but it is necessary to identify small changes.

7. Design optimisations that could be implemented (but not limited to) include adjustment of culvert spacing, realignment of culverts to existing flowpaths when more detailed survey is available, modifying the grade of culverts, road height and alignment 7adjustments, removal of bridge abutments from waterway extents etc

Depth (m) ²		Time of Inundation in hours (related to % exceedance)		Time of Submergence (hours) ¹		Velocity in m/s (related to % exceedance)		Velocity in m/s (related to actual change exceedance)		Hazard in m ² /s (related to % exceedance)	
Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case
0.001	0.001	32.50	38.03	139.04	139.42	-	-	-	-	-	-
0.013	0.012	2.53	7.99	144.03	144.32	-	-	-	-	-	-
0.021	0.021	22.93	25.44	146.93	147.23	-	-	-	-	-	-
0.002	0.002	24.34	27.31	126.02	103.61	-	-	-	-	0.16	0.19
0.030	0.030	22.82	29.90	131.44	112.14	-	-	-	-	0.13	0.19
-	-	-	-	-	-	-	-	-	-	-	-

Generally, access to Brookstead from the north (via Brookstead-Norwin Road) and east (via the Gore Highway) is not impacted due to the presence of the Project. Potential impact occurs to the Gore Highway to the south of Brookstead due to flooding from the North Branch of the Condamine River.

19.1.3 Southbrook

State-controlled roads within the vicinity of Southbrook include the Gore Highway and Oakey-Pittsworth Road. A review of Existing Case flood modelling results shows the following immunity of these State-controlled roads:

- Gore Highway—generally a low immunity, with portions being impacted by the 20% AEP event between Pittsworth and Southbrook as well as to the north of Southbrook
- Oakey-Pittsworth Road—generally a low immunity with portions being impacted by the 20% AEP event near the intersection of Oakey-Pittsworth Road and Gore Highway.

An impact assessment was completed against the FIOs established in consultation with the Expert Flood Panel and potential FIO exceedances were identified for the above roads identified in Table 19.3. Note that FIO exceedances experienced within Local Drainage models have not been displayed on figures.

Table 19.3 Southbrook emergency evacuation route impact assessment

Catchment	Road Object ID	Road Segment	Road Name	Surface Type	Flood Impact Objective Target					AEP	Maximum FIO Exceedance					Length (m)				Exceedance Justification	Proposed Mitigation Measures ⁷	
					Change in Peak Water Level (mm)	Change in Time of Inundation (%)	Change in Velocity (%)	Change in Velocity (m/s)	Change in Hazard (%)		Change in Peak Water Level (mm)	Change in Time of Inundation (%)	Change in Velocity (%)	Change in Velocity (m/s)	Change in Hazard (%)	Afflux	Time on Inundation	Velocity	Hazard			
Local Model 12	33	16015103990000258645	Oakley - Pittsworth Road	Sealed	10	10	20	1	10		20%	45	34	-	0.56	-	65.00	32.50	10.00	-	> Maturity of current cross-drainage design is limited by available information (e.g. detailed survey). > Road design constraints alter the existing flow regime.	> Further cross-drainage optimization investigations are required during detailed design. > Stakeholder Consultation (Landowner/ Government).
											10%	50	29	-	0.60	101	80.00	32.50	17.50	10.00		
											5%	59	27	58	0.62	93	90.00	32.50	17.50	17.50		
											2%	73	24	55	0.63	83	100.00	32.50	22.50	27.50		
											1%	79	23	53	0.63	72	115.00	30.00	25.00	27.50		
											0.05%	-	-	-	-	-	-	-	-	-		
											20%	-	16	-	-	-	-	5.00	-	-		
Local Model 13	22	16015103990000145813	Gore Highway	Sealed	10	10	20	1	10		10%	-	-	-	-	-	-	-	-	-	> Maturity of current cross-drainage design is limited by available information (e.g. detailed survey). > Rail design constraints alter the existing flow regime.	> Further cross-drainage optimization investigations are required during detailed design. > Stakeholder Consultation (Landowner/ Government).
											5%	-	-	-	-	-	-	-	-	-		
											2%	-	-	-	-	-	-	-	-	-		
											1%	-	-	-	-	-	-	-	-	-		
											0.05%	-	-	-	-	-	-	-	-	-		
											20%	-	-	-	-	-	-	-	-	-		
											10%	-	-	-	-	-	-	-	-	-		

- Table Notes
- Time of Submergence has been measured across the full length of the road segment and does not correspond to the exact location of exceedance within that segment.
 - Change in water level (afflux) is calculated based on the maximum water surface level difference between the existing and design case results, and does not consider the underlying topography. Depth values presented are based on the respective underlying topographies and in some cases do not align with afflux calculations. This can be attributed to interpolation calculations of results performed within the hydraulic model.
 - The above table should be read in conjunction with change maps provided as part of Volume II and the Digital Platform.
 - The symbol "-" denotes that no change above Flood Impact Objectives has been determined and therefore no result has been presented.
 - Changes recorded above may only occur during a small fraction of model simulation time.
 - Three decimal places do not infer absolute accuracy in modelling results, but it is necessary to identify small changes.
 - Design optimisations that could be implemented (but not limited to) include adjustment of culvert spacing, realignment of culverts to existing flowpaths when more detailed survey is available, modifying the grade of culverts, road height and alignment adjustments, removal of bridge abutments from waterway extents etc

Depth (m) ²		Time of Inundation in hours (related to % exceedance)		Time of Submergence (hours) ¹		Velocity in m/s (related to % exceedance)		Velocity in m/s (related to actual change exceedance)		Hazard in m ³ /s (related to % exceedance)	
Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case
0.022	0.033	0.42	0.56	2.11	1.25	-	-	0.66	1.22	-	-
-	0.050	0.49	0.64	2.23	1.38	-	-	0.75	1.35	0.09	0.18
-	0.059	0.52	0.67	2.29	1.43	1.02	1.60	0.80	1.43	0.11	0.21
0.107	0.006	0.58	0.72	2.38	1.52	1.10	1.70	0.88	1.51	0.09	0.16
0.009	0.012	0.60	0.74	2.41	1.55	1.15	1.75	0.93	1.56	0.11	0.18
-	-	-	-	-	-	-	-	-	-	-	-
-	-	0.22	0.26	2.57	2.56	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-

Access along the Gore Highway from Pittsworth to Southbrook is not impacted due to the Project. To the north of Southbrook, some afflux (up to 11 mm) and an increase in duration of inundation (15.5 per cent increase) may be experienced on the highway due to the presence of the Project. Although this technically exceeds the FIO target, the impacted length (2.5 m and 5 m, respectively) and the magnitude of the impacts are considered negligible. Further consultation with road managers regarding exceedance in this area will be undertaken during detailed design.

Oakey-Pittsworth Road is impacted due to the presence of the Project alignment adjacent to the intersection with the Gore Highway. Afflux is experienced on this segment of road up to maximum of 79 mm in the 1% AEP event across a length of approximately 115 m. A maximum velocity increase of 0.63 m/s is experienced in this same location. This segment of road is proposed to be upgraded as part of the Project and likely the cause of change from the Existing Case. Further optimisation of this design will be undertaken as part of detailed design to minimise impacts on the existing trafficability of the road.

19.1.4 Wellcamp

State-controlled roads within the vicinity of Wellcamp include Toowoomba-Cecil Plains Road. The Existing Case flood modelling results indicate that Toowoomba-Cecil Plains roads has very low flood immunity, with sections overtopping with small flood depths in the 20% AEP event. An impact assessment was completed against the FIOs established in consultation with the Expert Flood Panel and potential FIO exceedances were identified for the above roads identified in Table 19.4. Localities of FIO exceedances can be seen in Figure 6.5. Note that FIO exceedances experienced within Local Drainage models have not been displayed on figures.

Table 19.4 Wellcamp emergency evacuation route impact assessment

Catchment	Road Object ID	Road Segment	Road Name	Surface Type	Flood Impact Objective Target					AEP	Maximum FIO Exceedance					Length (m)				Exceedance Justification	Proposed Mitigation Measures ⁷
					Change in Peak Water Level (mm)	Change in Time of Inundation (%)	Change in Velocity (%)	Change in Velocity (m/s)	Change in Hazard (%)		Change in Peak Water Level (mm)	Change in Time of Inundation (%)	Change in Velocity (%)	Change in Velocity (m/s)	Change in Hazard (%)	Afflux	Time on Inundation	Velocity	Hazard		
Westbrook Creek	134	16015103990000366753	Toowoomba - Cecil Plains Road	Sealed	10	10	20	1	10	20%	16	51	-	0.32	13	57.50	10.00	2.50	10.00	Exceedances experienced on this road segment are due to the representation of existing and design culvert structures and their interface with road upgrades within the hydraulic model. The main area of exceedance along this road segment occur directly under Bridge 310-BR20 where an upgrade to Toowoomba-Cecil Plains Road is proposed. In the existing case a culvert is present under the road draining north towards Westbrook Creek. This flowpath is impeded due to the presence of the proposed road upgrade which is causing fringe impacts on Toowoomba Cecil Plains Road. The Existing Case model results indicate that Toowoomba-Cecil Plains Road has a very low existing flood immunity, with sections overtopping with small flood depths in the 20% AEP event. The impact of the Project alignment has minimal impact on the overall flood immunity. It is noted that in the 1% AEP event, the Project alignment does result in other small road segments being overtopped; however, it should be noted that the road is already submerged in other parts of the link with the time of inundation changing from 11.65 hours in the Existing Case to 11.67 hours in the Developed Case. Exceedances remain on this road segment due to the maturity of the hydraulic model and the representation of existing and proposed road cross drainage infrastructure within the model.	In detailed design, model improvements are to be undertaken to better represent existing culvert structures and their interface with proposed road upgrades. Detailed survey will be obtained to better represent existing structures under Toowoomba-Cecil Plains Road. A review of the bridge abutment location adjacent the flowpath will also be undertaken to mitigate these fringe impacts on the road.
										10%	65	25	45	-	14	5.00	17.50	15.00	2.50		
										5%	-	-	32	-	-	-	-	15.00	-		
										2%	102	-	30	-	-	27.50	-	17.50	-		
										1%	83	373	22	-	-	52.50	27.50	7.50	-		
										0.05%	-	-	-	-	-	-	-	-	-		
										20%	38	-	-	-	-	15.00	-	-	-		
Westbrook Creek	202	16015103990000366754	Toowoomba - Cecil Plains Road	Sealed	10	10	20	1	10	10%	-	-	-	-	-	-	-	-	-	- Further cross-drainage optimization investigations are required during detailed design. - Further road and or rail design optimization investigations are required during detailed design.	
										5%	-	-	-	-	-	-	-	-	-		
										2%	-	-	-	-	-	-	-	-	-		
										1%	-	-	-	-	-	-	-	-	-		
										0.05%	-	-	-	-	-	-	-	-	-		
										20%	38	-	-	-	-	15.00	-	-	-		
Local Model 14	10	16015103990000366753	Toowoomba - Cecil Plains Road	Sealed	10	10	20	1	10	20%	50	149	-	-	-	47.50	7.50	-	-	Between chainage 195.0 km and 195.8 km, the proposed rail embankment is constructed in "cut". Due to this, the existing upstream overland flow path is required to be diverted to an adjacent culvert crossing (C195.98) in order to provide adequate drainage across the alignment. This diverted concentrated runoff discharging through Culvert C195.98 increases the amount of runoff flowing north and reaching Toowoomba - Cecil Plains Road which otherwise would flow along a longer reach where more floodplain storage is available. In addition, the proposed rail embankment impedes the existing overland flow regime (chainage 196.2 km to 196.8 km) diverting flow to the culvert structure C196.48. The resulting runoff flows downstream and is intercepted by the exiting Toowoomba - Cecil Plains Road. The above alterations to the flood dynamics cause the increase in water level (Afflux) and time of inundation experienced at Toowoomba-Cecil Plains Road. The current maturity of both the proposed cross drainage, flow diversion and embankment design is the reason these exceedances still remain. Complete optimisation of the design has not yet been completed to date, only to a reference design level, and therefore further optimisation of cross drainage structures and placement needs to be undertaken in the detailed stage. In addition, the representation of existing cross drainage structures under Toowoomba-Cecil Plains Road is limited due to limited as-built information or detailed survey.	Due to the diversion of overland flow, it is expected that some impact may remain to Toowoomba-Cecil Plains Road post mitigation. The hydraulic model will be updated with detailed survey to capture existing culverts under Toowoomba-Cecil Plains Road which may improve conveyance across the road itself. Further optimisation of proposed cross drainage placement and flow diversion will also be undertaken in the detailed design stage. Ongoing consultation with the appropriate road manager will be undertaken in the detailed design stage to discuss any residual exceedances and agree on further appropriate mitigation measures.
										10%	61	54	-	-	-	55.00	7.50	-	-		
										5%	76	92	-	-	-	62.50	22.50	-	-		
										2%	91	200	-	-	-	65.00	30.00	-	-		
										1%	71	101	-	-	-	70.00	30.00	-	-		
										0.05%	-	-	-	-	-	-	-	-	-		
										20%	38	-	-	-	-	15.00	-	-	-		

- Table Notes
- Time of Submergence has been measured across the full length of the road segment and does not correspond to the exact location of exceedance within that segment
 - Change in water level (afflux) is calculated based on the maximum water surface level difference between the existing and design case results, and does not consider the underlying topography. Depth values presented are based on the respective underlying topographies and in some cases do not align with afflux calculations. This can be attributed to interpolation calculations of results performed within the hydraulic model.
 - The above table should be read in conjunction with change maps provided as part of Volume II and the Digital Platform.
 - The symbol "-" denotes that no change above Flood Impact Objectives has been determined and therefore no result has been presented.
 - Changes recorded above may only occur during a small fraction of model simulation time.
 - Three decimal places do not infer absolute accuracy in modelling results, but it is necessary to identify small changes.
 - Design optimisations that could be implemented (but not limited to) include adjustment of culvert spacing, realignment of culverts to existing flowpaths when more detailed survey is available, modifying the grade of culverts, road height and alignment adjustments, removal of bridge abutments from waterway extents etc

Depth (m) ²		Time of Inundation in hours (related to % exceedance)		Time of Submergence (hours) ¹		Velocity in m/s (related to % exceedance)		Velocity in m/s (related to actual change exceedance)		Hazard in m ² /s (related to % exceedance)	
Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case
0.792	0.801	0.39	0.59	16.92	16.97	-	-	0.89	1.21	0.46	0.52
-	0.065	0.55	0.69	16.90	16.93	1.04	1.51	-	-	0.66	0.74
-	-	-	-	-	-	1.94	2.58	-	-	-	-
0.008	0.023	-	-	-	-	2.15	2.79	-	-	-	-
0.018	0.031	0.21	1.01	11.65	11.67	2.31	2.82	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
0.421	0.424	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
-	0.050	0.67	1.67	2.63	2.60	-	-	-	-	-	-
-	0.061	1.39	2.15	2.88	2.95	-	-	-	-	-	-
-	0.076	0.87	1.67	2.82	2.92	-	-	-	-	-	-
-	0.091	0.46	1.37	3.11	3.33	-	-	-	-	-	-
0.070	0.104	0.76	1.52	3.14	3.25	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-

The FIO impact assessment of the Project's permanent footprint has demonstrated that some exceedance of the FIOs occurs along Toowoomba-Cecil Plains Road in both regional and local flooding. Afflux, change in duration of inundation, change in velocity, and change in hazard impacts are all recorded within the same stretch of road. These exceedances are generally confined to the regional flood event, with afflux and change in duration of inundation exceedances also occurring in the local flood event. Impact on Toowoomba-Cecil Plains Road may further constrain access from the west heading towards Toowoomba, although this section of road, as stated above, currently has low flood immunity in the Existing Case scenario.

19.1.5 Gowrie Mountain

State-controlled roads within the vicinity of Gowrie include the Warrego Highway. Based on Existing Case flood modelling results, the Warrego Highway exhibits 1% AEP immunity (on the western bound lane) within the vicinity of the Project footprint, with overtopping in the 20% AEP event on the eastbound lane. Due to the variability in immunity, a review of the modelling files and aerial imagery indicates that the eastbound lane was constructed in 2017, with LiDAR utilised in the TUFLOW model dated 2015. The discrepancy in data timelines has led to the eastbound lane not being captured within the current flood modelling, hence demonstrating a lower immunity. It is likely that the immunity of the eastbound lane is higher than that modelled and potentially has similar immunity to that of the westbound lane. This will be further revised in the detailed design stage of the Project.

An impact assessment was completed against the FIOs, established in consultation with the Expert Flood Panel, and potential FIO exceedances were identified for the above roads identified in Table 19.5. Localities of FIO exceedances can be seen in Figure 5.23.

FIO exceedances occur on the eastbound lane within the vicinity of the Project alignment and, as stated above, this is likely due to the misrepresentation of the eastbound lane within the utilised LiDAR. Two other locations demonstrate exceedances along the Warrego Highway, with both these occurring on the westbound lane. A review of these exceedances shows up to a maximum of 51 mm of afflux occurring within the proximity of the Project alignment and an 11 per cent increase in duration of inundation approximately 1 km to the east of the Project alignment crossing. A review of the extent of impact shows approximately 6 m of road segment is impacted by increased afflux and 2 m of road segment impacted by the increase of time of inundation.

19.1.6 Gowrie Junction

No impacts to State-controlled roads within the vicinity of Gowrie Junction were identified due to the Project.

19.2 Goondiwindi Regional Council

Preliminary feedback from the GRC LDMG identified up to three potential evacuation centres/assembly points across the region, with the selection of use during an emergency dependant on a range of factors, including the type and scale of disaster. Feedback provided by GRC LDMG included:

- Evacuation centres/Assembly point:
 - Goondiwindi Evacuation Centre—located at the Goondiwindi Showgrounds, 323 Boundary Road, Goondiwindi, advertised to the public as the primary evacuation centre site
 - Inglewood Assembly Point—located at the Inglewood Landfill Facility, off Inglewood-Texas Road, Inglewood. This is a new site to that used in Nov-Dec flood event 2021
 - Texas Evacuation Centre—located at the Texas Sport Complex, off Middleton Street, Texas. Advertised to the public as the primary evacuation centre site
 - Due to the terrain and nature of the GRC area, most flooding is caused more by riverine flooding than flash flooding and, as a result, most of our centres can be completely isolated by road for periods during floods. Although the State-controlled road network is very important for access at all times, part of the Kildonan Road and the Yelarbon Kurumbul Road provide an important access route to Goondiwindi at times during floods.

Based on the above feedback, a review of the State-controlled roads (and additional local roads identified) within and around these areas was completed and an impact assessment conducted to determine the potential effects of the Project on these routes.

19.2.1 Goondiwindi and Yelarbon

Based on feedback from the GRC LDMG, the Goondiwindi Evacuation Centre can service not only Goondiwindi but also the Yelarbon area. Key State-controlled roads within this region include the Cunningham Highway, Inglewood-Texas Road, Yelarbon-Keetah Road and Texas-Yelarbon Road. As indicated by the GRC LDMG, two local roads also form a key part of the evacuation routes during flood events including Kildonan Road and Yelarbon-Kurumbul Road. A review of Existing Case results shows the following existing flood immunity of these roads:

- Cunningham Highway—generally a low immunity with portions being impacted by the 20% AEP event
- Inglewood-Texas Road—generally a low immunity with portions being impacted by the 20% AEP event
- Yelarbon-Keetah Road—generally a low immunity with portions being impacted by the 20% AEP event
- Texas-Yelarbon Road—generally a low immunity with portions being impacted by the 20% AEP event
- Yelarbon-Kurumbul Road—generally a low immunity with portions being impacted by the 20% AEP event
- Kildonan Road—generally a low immunity with portions being impacted by the 20% AEP event.

An impact assessment was completed against the FIOs established in consultation with the Expert Flood Panel and potential FIO exceedances were identified for the above roads identified in Table 19.6. Localities of FIO exceedances can be seen in Figure 14.13. Note that FIO exceedances experienced within Local Drainage models have not been displayed on figures.

Table 19.6 Goondiwindi and Yelarbon emergency evacuation route impact assessment

Catchment	Road Object ID	Road Segment	Road Name	Surface Type	Flood Impact Objective Target					AEP	Maximum FIO Exceedance					Length (m)				Exceedance Justification	Proposed Mitigation Measures ⁵
					Change in Peak Water Level (mm)	Change in Time of Inundation (%)	Change in Velocity (%)	Change in Velocity (m/s)	Change in Hazard (%)		Change in Peak Water Level (mm)	Change in Time of Inundation (% Change) ¹	Change in Velocity (%) ²	Change in Velocity (m/s)	Change in Hazard (%) ³	Afflux	Time on Inundation	Velocity	Hazard		
Macintyre Brook	70	16015103990000356048	Talum Street	Sealed	10	10	20	1	10	20%	451	-	-	-	-	45.00	-	-	-	This segment of Taloom Street is located at the existing Cunningham Highway/Wondall Street/Talum St intersection which interfaces with the existing rail level crossing. As part of the Project works, significant diversion of this road occurs to remove the level crossing and create an over pass of the rail alignment. As such this segment of Taloom Street becomes redundant and will not be used under developed case conditions. Although an increase in afflux and time of inundation is experienced on this segment, a review of the time of submergence demonstrates a slight improvement. Due to this segment becoming redundant under developed case conditions, the impact is deemed not significant. Exceedances still remain on this segment due to the maturity of the road, rail and cross drainage design being at a reference design level with further optimisation to be completed. Optimisation of the road design as well as rail and cross drainage design will be completed in the detailed design stage to finalise road height, rail and road cross drainage placement and sizing and how they interact with the rail design.	Further optimisation of the road/rail interface will be undertaken at this location in the detailed design phase. Therefore as part of the ongoing design optimisation some change in exceedance may occur from that presented in the revised draft EIS. Further consultation with the State Road Authority will be undertaken to discuss this impact and agree if further mitigation is required.
										10%	471	-	-	-	-	37.50	-	-	-		
										5%	473	-	-	-	-	37.50	-	-	-		
										2%	487	-	-	-	-	37.50	-	-	-		
										1%	274	918	-	-	-	15.00	30.00	-	-		
										0.05%	-	-	-	-	-	-	-	-	-		
Macintyre Brook	72	16015103990000356050	Talum Street	Sealed	10	10	20	1	10	20%	152	90	-	-	-	37.50	7.50	-	-	This portion of Taloom Street is proposed to be upgraded as part of the Project with the design varying in height by approximately +/- 100mm from the existing case scenario. The impacts on this road segment are due to the introduction of the new road upgrades within the area having a localised increase in water level from the existing case due to the redistribution of flood waters. Although there may be some localised increases in afflux and time of inundation, a review of the time of submergence results for all modelled events from the 20% to 1% AEP events, shows that with the proposed design, a significant reduction, and improvement, on the operational capacity of this segment can be achieved through the proposed upgrades. Exceedances still remain on this segment due to the maturity of the road, rail and cross drainage design being at a reference design level with further optimisation to be completed. Optimisation of the road design as well as rail and cross drainage design will be completed in the detailed design stage to finalise road height, rail and road cross drainage placement and sizing and how they interact with the rail design.	It is expected that reduction in exceedances on this segment of road can be achieved in the detailed design phase through further optimisation of road cross drainage structures (with particular focus on optimising culvert location, culvert spread, culvert invert levels and grade). Ongoing consultation with the State Road authority will be undertaken to discuss the road design, associated flood impacts and any site specific mitigations required to minimise the impact the operational capacity of the road.
										10%	173	56	-	-	-	45.00	7.50	-	-		
										5%	167	45	-	-	-	45.00	7.50	-	-		
										2%	117	178	-	-	-	52.50	15.00	-	-		
										1%	-	50	-	-	-	-	30.00	-	-		
										0.05%	-	-	-	-	-	-	-	-	-		
Macintyre Brook	366	16015103990000396093	Wondall Street	Sealed	10	10	20	1	10	20%	52	11	-	-	-	15.00	7.50	-	-	This portion of Wondall Street is proposed to be upgraded as part of the Project with the design varying in height by approximately +/- 100mm from the existing case scenario. The impacts on this road segment are due to the introduction of the new road upgrades within the area having a localised increase in water level from the existing case due to the redistribution of flood waters. The increase in road height, plus overtopping, results in a greater increase in water level on this segment. Exceedances still remain on this segment due to the maturity of the road, rail and cross drainage design being at a reference design level with further optimisation to be completed. Optimisation of the road design as well as rail and cross drainage design will be completed in the detailed design stage to finalise road height, rail and road cross drainage placement and sizing and how they interact with the rail design.	It is expected that reduction in exceedances on this segment of road can be achieved in the detailed design phase through further optimisation of road cross drainage structures (with particular focus on optimising culvert location, culvert spread, culvert invert levels and grade). Ongoing consultation with the State Road authority will be undertaken to discuss the road design, associated flood impacts and any site specific mitigations required to minimise the impact the operational capacity of the road.
										10%	173	-	-	-	-	7.50	-	-	-		
										5%	167	-	-	-	-	7.50	-	-	-		
										2%	88	178	-	-	-	7.50	22.50	-	-		
										1%	-	42	-	-	-	-	22.50	-	-		
										0.05%	-	-	-	-	-	-	-	-	-		
Macintyre Brook	369	16015103990000396090	Wondall Street	Sealed	10	10	20	1	10	20%	213	1457	-	-	-	52.50	7.50	-	-	This segment of Wondall Street is located at the existing Cunningham Highway/Wondall Street/Talum St intersection which interfaces with the existing rail level crossing. As part of the Project works, significant diversion of this road occurs to remove the level crossing and create an over pass of the rail alignment. As such this segment of Wondall Street becomes redundant/realigned under developed case conditions. Due to this segment becoming redundant/realigned under developed case conditions, the impact is deemed not significant. Exceedances still remain on this segment due to the maturity of the road, rail and cross drainage design being at a reference design level with further optimisation to be completed. Optimisation of the road design as well as rail and cross drainage design will be completed in the detailed design stage to finalise road height, rail and road cross drainage placement and sizing and how they interact with the rail design.	Further optimisation of the road/rail interface will be undertaken at this location in the detailed design phase. Therefore as part of the ongoing design optimisation some change in exceedance may occur from that presented in the revised draft EIS. Further consultation with the State Road Authority will be undertaken to discuss this impact and agree if further mitigation is required.
										10%	233	2032	-	-	-	52.50	22.50	-	-		
										5%	235	1194	-	-	-	52.50	22.50	-	-		
										2%	249	816	-	-	-	52.50	30.00	-	-		
										1%	259	239	-	-	-	7.50	37.50	-	-		
										0.05%	-	-	-	-	-	-	-	-	-		
Macintyre Brook	370	16015103990000396091	Wondall Street	Sealed	10	10	20	1	10	20%	343	2009	-	-	-	67.50	127.50	-	-	This portion of Wondall St (ID 370) is partially proposed to be upgraded as part of the Project. Exceedances recorded are based on the centreline of the existing road, although the new centreline of the upgrade is slightly shifted east by approximately 10m. This offset of centrelines results in the current centreline running along the toe of the embankment of the proposed upgrade, likely within table drains. This area experiences a localised increase in water level and time of inundation due to the presence of the embankment and hence the recorded exceedances. It is not anticipated that the exceedances recorded will impact the new upgraded road and therefore are not deemed significant. Exceedances still remain on this segment due to the maturity of the road, rail and cross drainage design being at a reference design level with further optimisation to be completed. Optimisation of the road design as well as rail and cross drainage design will be completed in the detailed design stage to finalise road height, rail and road cross drainage placement and sizing and how they interact with the rail design.	Further optimisation of road cross drainage structures (with particular focus on optimising culvert location, culvert spread, culvert invert levels and grade) will be undertaken in Detailed Design to mitigate the exceedances experienced along the existing segment centreline. Further consultation with the State Road Authority will be undertaken to discuss this impact and agree if further mitigation is required.
										10%	366	1969	-	-	-	75.00	187.50	-	-		
										5%	376	1076	-	-	-	232.50	210.00	-	-		
										2%	-	473	-	-	-	-	157.50	-	-		
										1%	-	213	-	-	-	-	187.50	-	-		
										0.05%	-	-	-	-	-	-	-	-	-		
Macintyre Brook	466	16015103990000090934	Cunningham Highway	Sealed	10	10	20	1	10	20%	97	-	-	-	-	30.00	-	-	-	In this location the Cunningham Highway (ID 466) has been raised to sit in with the proposed Yelarbon Levee raising the increase in peak water levels and time of inundation on this segment are as a result of flow redistribution caused by the raising and the extension of the Yelarbon levee in the proposed design. The levee provides increased flood immunity to a significant portion of Yelarbon with floodwaters being diverted south west. The raising and extension of the levee at Yelarbon was required to minimise the impact of the Project on the township. As described in Section 14.5.2 of the Technical Report, several configurations of the levee were investigated for the 1% AEP event to minimise impacts on the township as well as maintaining the trafficability of the Cunningham Highway. These exceedances remain due to the design maturity of the Yelarbon Levee. Optimisation of the levee raise needs to occur in the detailed design phase to find the appropriate balance between protecting the Yelarbon township and impacting land/inspectors outside the levee area.	Further work on the Yelarbon Levee and its influence on flow behaviour and impacts on lots within the vicinity of Yelarbon will continue to be investigated at Detailed Design to determine the optimal solution. This will be undertaken in consultation with Goodwin Regional Council and the State Road Authority to ensure the wider protection of the township is achieved and to minimise the impact to access on this road. Ongoing consultation will be undertaken with the State Road Authority, to discuss this exceedance and agree any site specific mitigation measures to further reduce impact. Due to the locality of this road to the levee upgrade, it is anticipated some exceedance will remain after mitigation in the detailed design phase. In addition, optimisation of road cross drainage structures (with particular focus on optimising culvert location, culvert spread, culvert invert levels and grade) will be undertaken in Detailed Design to further reduce impacts.
										10%	26	22	-	-	-	75.00	7.50	-	-		
										5%	64	99	-	-	-	150.00	67.50	-	-		
										2%	150	91	-	-	107	315.00	187.50	-	112.50		
										1%	280	136	-	-	88	517.50	232.50	-	375.00		
										0.05%	-	-	-	-	-	-	-	-	-		
Macintyre Brook	521	16015103990000396092	Wondall Street	Sealed	10	10	20	1	10	20%	339	-	-	-	-	30.00	-	-	-	This portion of Wondall St (ID 521) is partially upgraded and partially redundant as part of the proposed road upgrade works. In the developed case the redundant portion of this section of Wondall Street (adjacent to existing Railway Pole intersection) is lowered and a flow path is created. The reported afflux exceedances occur in this location due to the introduction of the drainage channel and culverts that were dry in the existing case becoming newly flood-affected in the developed case. The increase in time of inundation is due to a decrease in road height in the design case at the intersection of Wondall Street and Yelarbon Kurumbul Road. This increase in time of inundation increases the time of submergence of the segment by 16 hours in the 1% AEP event. It is not anticipated that this increase is significant and detrimental to access around Yelarbon due to both Cunningham Highway to the north west and Yelarbon Kurumbul Road to the south west having greater time of inundation in the Existing Case than the predicted value of this segment in the design case. Exceedances remain on this road segment due to the change in alignment of the road in the design case and the road design only being at a reference design level. Further optimisation on the road height and cross drainage design is to be undertaken in the detailed design stage.	Further optimisation of the road design and road cross drainage structures (with particular focus on optimising culvert location, culvert spread, culvert invert levels and grade) will be undertaken in Detailed Design to mitigate the exceedances experienced along this segment. Some reduction in exceedance is expected. Further consultation with the State Road Authority will be undertaken to discuss this impact and agree if further mitigation is required.
										10%	401	-	-	-	-	45.00	-	-	-		
										5%	406	-	-	-	-	45.00	-	-	-		
										2%	461	-	-	-	-	30.00	-	-	-		
										1%	636	546	-	-	36517	30.00	45.00	-	7.50		
										0.05%	-	-	-	-	-	-	-	-	-		
Macintyre Brook	633	16015103990000090893	Cunningham Highway	Sealed	10	10	20	1	10	20%	26	14	-	-	-	7.50	7.50	-	-	This section of the Cunningham Highway (ID 633) is partially upgraded and realigned with a portion of the existing highway becoming redundant as a part of the proposed road upgrade works. The main area of exceedances is clustered around the realignment works to the north west of Yelarbon which experiences an increase in time of inundation. This can be attributed to the change in flood dynamics in this area through the introduction of the new road embankments. A review of the time of submergence for this segment of road, demonstrates that the exceedances identified have little impact on the operational capacity of the overall road segment with change between the existing and design time of submergence results being negligible. Therefore it is not deemed that the increases in time of inundation at individual cells significantly impact the road. Exceedances remain on this road segment of the Cunningham Highway due to the road design only being at a reference design level. Further optimisation on the road height and cross drainage design is to be undertaken in the detailed design stage.	Through optimisation of road cross drainage structures (with particular focus on optimising culvert location, culvert spread, culvert invert levels and grade) it is anticipated some minor reductions in exceedance could occur in this area. Further consultation with the State Road Authority will be undertaken to discuss this impact and agree if further mitigation is required.
										10%	19	29	-	-	-	7.50	22.50	-	-		
										5%	-	78	-	-	-	-	37.50	-	-		
										2%	-	96	-	-	-	-	217.50	-	-		
										1%	-	87	-	-	-	-	75.00	-	-		
										0.05%	-	-	-	-	-	-	-	-	-		

[illegible]

Catchment	Road Object ID	Road Segment	Road Name	Surface Type	Flood Impact Objective Target					AEP	Maximum FIO Exceedance					Length (m)				Exceedance Justification	Proposed Mitigation Measures ³			
					Change in Peak Water Level (mm)	Change in Time of Inundation (%)	Change in Velocity (%)	Change in Velocity (m/s)	Change in Hazard (%)		Change in Peak Water Level (mm)	Change in Time of Inundation (% Change) ²	Change in Velocity (%) ³	Change in Velocity (m/s)	Change in Hazard (%) ³	Afflux	Time on Inundation	Velocity	Hazard					
Macintyre Brook	498	1601510399000018965	Kildonan Road	Sealed	10	10	20	1	10	20%	-	-	-	-	-	-	-	-	-	-	This section of Kildonan Rd (ID 498) is approximately 5km away from the alignment. The reported afflux exceedance is isolated and localised and occurs at a location where a cell is dry in the existing case and flood-affected in the developed case. No other exceedances are reported on this road and therefore this exceedance can be considered a modelling artefact and not representative of an actual exceedance. It is deemed that this impact will not be detrimental to the operational capacity of this road. Exceedances on this segment of Kildonan Road still exist as optimisation of the design to mitigate this exceedance has not been undertaken due to its minor nature of the exceedance and the distance from the alignment.	Model updates will be made in line with those recommended by the Expert Panel. No further mitigation of this exceedance is proposed. Further consultation with the State Road Authority will be undertaken to discuss this impact and agree if further mitigation is required.		
										10%	-	-	-	-	-	-	-	-	-	-			-	-
										5%	-	-	-	-	-	-	-	-	-	-			-	-
										2%	-	-	-	-	-	-	-	-	-	-			-	-
										1%	199	-	-	-	30.00	-	-	-	-	-			-	
										0.05%	-	-	-	-	-	-	-	-	-	-			-	
Macintyre Brook	631	16015103990000564761	Yelarbon - Keetah Road	Sealed	10	10	20	1	10	20%	-	-	-	-	-	-	-	-	-	-	This section of Yelarbon - Keetah Rd (ID 631) is approximately 3km away the alignment. The reported afflux exceedance is isolated and localised and occurs at a location where a cell is dry in the existing case and flood-affected in the developed case. No other exceedances are reported on this road and therefore this exceedance can be considered a modelling artefact and not representative of an actual exceedance. It is deemed that this impact will not be detrimental to the operational capacity of this road. Exceedances on this segment of Yelarbon-Keetah Road still exist as optimisation of the design to mitigate this exceedance has not been undertaken due to its minor nature of the exceedance and the distance from the alignment.	Model updates will be made in line with those recommended by the Expert Panel. No further mitigation of this exceedance is proposed. Further consultation with the State Road Authority will be undertaken to discuss this impact and agree if further mitigation is required.		
										10%	-	-	-	-	-	-	-	-	-	-			-	-
										5%	-	-	-	-	-	-	-	-	-	-			-	-
										2%	-	-	-	-	-	-	-	-	-	-			-	-
										1%	306	-	-	-	7.50	-	-	-	-	-			-	-
										0.05%	-	-	-	-	-	-	-	-	-	-			-	-
Macintyre Brook	634	16015103990000564756	Yelarbon - Keetah Road	Sealed	10	10	20	1	10	20%	-	-	-	-	-	-	-	-	-	-	The increase in peak water levels on this section of Yelarbon Keetah Rd (ID 634), occurs as a result of flow redistribution caused by the raising and the extension of the Yelarbon levee in the proposed design. The levee provides increased flood immunity to a significant portion of Yelarbon with floodwaters being diverted south west. The raising and extension of the levee at Yelarbon was required to minimise the impact of the Project on the township. As described in Section 14.5.2 of the Technical Report, several configurations of the levee were investigated for the 1% AEP event to minimise impacts on the township as well as maintaining the trafficability of the Cunningham Highway. On this section of the road, the increases in water level range between 10mm to 20mm over a distance of approximately 900m under the 1% AEP event. Under events smaller than the 1% AEP event, no afflux exceedances occur. In the existing case 1% AEP event there is widespread inundation that affects Yelarbon-Keetah Road leading into Yelarbon. This section of the highway is already inundated by around 0.8m of floodwater and would not be trafficable and therefore it is not deemed this exceedance to impact the long term operational capacity of the road. This exceedance remains due to limited design maturity of the road design at this location and its interface with the existing road level. Further optimisation of this design along with optimisation of the road cross drainage is required to be undertaken in detailed design to finalise the interface between the existing road and Yelarbon Levee raise.	Further work on the Yelarbon Levee and its influence on flow behaviour and impacts on lots within the vicinity of Yelarbon will continue to be investigated at Detailed Design to determine the optimal solution. This will be undertaken in consultation with Goonswind Regional Council and the State Road Authority to ensure the wider protection of the township is achieved and to minimise the impact to access on this road. Ongoing consultation will be undertaken with the State Road Authority, to discuss this exceedance and agree any site specific mitigation measures to further reduce impact. Furthermore, optimisation of road cross drainage structures (with particular focus on culvert location, culvert spread, culvert invert levels and grade) will be undertaken in Detailed Design.		
										10%	-	-	-	-	-	-	-	-	-	-			-	-
										5%	-	-	-	-	-	-	-	-	-	-			-	-
										2%	-	-	-	-	-	-	-	-	-	-			-	-
										1%	22	-	-	-	13	990.00	-	-	45.00	-			-	-
										0.05%	-	-	-	-	-	-	-	-	-	-			-	-
Macintyre Brook	630	16015103990000564760	Yelarbon - Keetah Road	Sealed	10	10	20	1	10	20%	-	-	-	-	-	-	-	-	-	-	This section of Yelarbon Keetah Rd (ID 630) is approximately 5km downstream of the alignment. The percentage change in time of inundation exceedance is an isolated modelling artefact that occurs on the fringe of the flood extent. A very localised increase in time of inundation occurs along this segment due to minor changes in flood dynamics between the existing and design cases. A review of the time of submergence existing and design case results demonstrates that minimal change in time of submergence occurs on this segment and therefore it is not deemed that this exceedance will have a detrimental impact on the operational capacity of the road. Exceedances on this segment of Yelarbon-Keetah Road still exist as optimisation of the design to mitigate this exceedance has not been undertaken due to its minor nature of the exceedance and the distance from the alignment.	Model updates will be made in line with those recommended by the Expert Panel. No further mitigation of this exceedance is proposed. Further consultation with the State Road Authority will be undertaken to discuss this impact and agree if further mitigation is required.		
										10%	-	-	-	-	-	-	-	-	-	-			-	-
										5%	-	-	-	-	-	-	-	-	-	-			-	-
										2%	-	-	-	-	-	-	-	-	-	-			-	-
										1%	36	-	-	-	22.50	-	-	-	-	-			-	-
										0.05%	-	-	-	-	-	-	-	-	-	-			-	-
Local Model 2	184	16236553520000000167	Yelarbon Kurumbul Road	Unsealed	10	10	10	0.5	10	20%	53	923	-	0.280	-	90.00	32.50	42.50	-	-	In the existing case, flow weirs over the existing rail embankment and is distributed across a large area directly upstream of Yelarbon Kurumbul Road. In the design case, the proposed rail embankment causes upstream ponding and concentrates runoff through culverts C22.43 and C23.05. This concentration of flow through structures increases the water level (afflux), time of inundation, and velocity on Yelarbon Kurumbul Road. This road runs parallel to the embankment and is directly downstream (within 20m) of the alignment. The exceedances remain due to the current design being at a reference design stage and culvert placement and spacing being grouped (rather than spaced across the embankment), resulting in concentrated flow. Further iteration of cross drainage and placement needs to be undertaken at detailed design stage. It is anticipated that a reduction in FIO exceedance can occur through mitigation, although some areas of exceedance may remain due to the presence of the project. In addition, the maturity of the existing cross-drainage is limited by available information along Yelarbon Kurumbul Road (e.g. detailed survey). A review of the time of submergence across the length of this road segment demonstrates that for all modelled events a reduction in time of submergence occurs from the existing case to the design case, improving accessibility along this road segment.	Model improvements to be undertaken in detailed design to include detailed survey of existing infrastructure. Further optimisation of design cross drainage will be undertaken in the detailed design phase to mitigate these exceedances, including alteration to culvert location and spread, culvert grade and invert levels and increase in culvert cell numbers. Appropriate consultation with the road manager will be undertaken to discuss residual exceedances and agree on further mitigation measures should exceedances not be designed out in the detailed design phase.		
										10%	56	1067	-	0.296	-	90.00	55.00	57.50	-	-			-	
										5%	60	717	-	0.314	-	90.00	77.50	67.50	-	-			-	
										2%	64	826	-	0.327	-	87.50	90.00	72.50	-	-			-	
										1%	66	380	64.89	0.321	-	85.00	92.50	75.00	-	-			-	
										0.05%	-	-	-	-	-	-	-	-	-	-			-	
Local Model 2	189	16236553520000000143	Yelarbon Kurumbul Road	Sealed	10	10	20	1	10	20%	168	17	-	-	408	75.00	2.50	-	17.50	-	> Maturity of current cross-drainage design is limited by available information (e.g. detailed survey) > Rail design constraints alter the existing flow regime.	> Further cross-drainage optimization investigations are required during detailed design. > Stakeholder Consultation (Landowner/ Government).		
										10%	123	22	-	-	316	55.00	7.50	-	20.00	-			-	-
										5%	139	66	-	-	247	52.50	40.00	-	25.00	-			-	-
										2%	153	933	-	-	201	267.50	77.50	-	27.50	-			-	-
										1%	123	22	-	-	315	55.00	7.50	-	20.00	-			-	-
										0.05%	-	-	-	-	-	-	-	-	-	-			-	-
Local Model 2	215	16015103990000564745	Yelarbon Kurumbul Road	Sealed	10	10	20	1	10	20%	99	15	-	-	17.50	2.50	-	-	-	-	> Maturity of current cross-drainage design is limited by available information (e.g. detailed survey) > Rail design constraints alter the existing flow regime.	> Further cross-drainage optimization investigations are required during detailed design. > Stakeholder Consultation (Landowner/ Government).		
										10%	29	-	-	-	17.50	-	-	-	-	-			-	-
										5%	18	17	-	-	15.00	5.00	-	-	-	-			-	-
										2%	13	-	-	-	7.50	-	-	-	-	-			-	-
										1%	-	-	-	-	-	-	-	-	-	-			-	-
										0.05%	-	-	-	-	-	-	-	-	-	-			-	-
Local Model 2	216	16015103990000564746	Yelarbon Kurumbul Road	Sealed	10	10	20	1	10	20%	143	343	-	-	82.50	17.50	-	-	-	-	In the existing case, flow weirs over the existing rail embankment (between chainage 23.4 km and 24.6 km) and is distributed across a large area directly upstream of Yelarbon Kurumbul Road. In the design case, the proposed rail embankment causes upstream ponding and concentrates runoff through culverts C23.53 and C24.41. This road runs parallel to the embankment and is directly downstream (within 20m) of the alignment. The exceedances remain due to the current design being at a reference design stage and culvert placement and spacing being grouped (rather than spaced across the embankment), resulting in concentrated flow. Further iteration of cross drainage and placement needs to be undertaken at detailed design stage. It is anticipated that a reduction in FIO exceedance can occur through mitigation, although some areas of exceedance may remain due to the presence of the project. In addition, the maturity of the existing cross-drainage is limited by available information along Yelarbon Kurumbul Road (e.g. detailed survey).	Model improvements to be undertaken in detailed design to include detailed survey of existing infrastructure. Further optimisation of design cross drainage will be undertaken in the detailed design phase to mitigate these exceedances, including alteration to culvert location and spread, culvert grade and invert levels and increase in culvert cell numbers. Appropriate consultation with the road manager will be undertaken to discuss residual exceedances and agree on further mitigation measures should exceedances not be designed out in the detailed design phase.		
										10%	150	475	-	-	77.50	47.50	-	-	-	-			-	-
										5%	157	352	-	-	80.00	50.00	-	-	-	-			-	-
										2%	164	350	-	-	77.50	52.50	-	-	-	-			-	-
										1%	162	339	-	-	82.50	52.50	-	-	-	-			-	-
										0.05%	-	-	-	-	-	-	-	-	-	-			-	-
Local Model 2	218	16015103990000564748	Yelarbon Kurumbul Road	Unsealed	10	10	10	0.5	10	20%	200	668	-	0.953	9695	175.00	87.50	77.50	52.50	-	In the existing case, flow weirs over the existing rail embankment and is distributed across a large area directly upstream of Yelarbon Kurumbul Road. In the design case, the proposed rail embankment causes upstream ponding and concentrates runoff through culvert C13.00. This road runs parallel to the embankment and is directly downstream (within 20m) of the alignment. The exceedances remain due to the current design being at a reference design stage and culvert placement and spacing being grouped (rather than spaced across the embankment), resulting in concentrated flow. Further iteration of cross drainage and placement needs to be undertaken at detailed design stage. It is anticipated that a reduction in FIO exceedance can occur through mitigation, although some areas of exceedance may remain due to the presence of the project. In addition, the maturity of the existing cross-drainage is limited by available information along Yelarbon Kurumbul Road (e.g. detailed survey).	Model improvements to be undertaken in detailed design (e.g. detailed survey). Further optimisation of cross drainage will be undertaken in the detailed design phase to mitigate these exceedances, including alteration to culvert location and spread, culvert grade and invert levels, increase in culvert cell numbers, scour protection, and earthworks. Appropriate consultation with the road manager will be undertaken to discuss residual exceedances and agree on further mitigation measures should exceedances not be designed out in the detailed design phase.		
										10%	209	414	-	1.009	7227	167.50	92.50	92.50	55.00	-			-	-
										5%	222	719	-	1.073	25977	162.50	117.50	105.00	72.50	-			-	-
										2%	238	1035	86.02	1.100	18943	160.00	122.50	130.00	87.50	-			-	-
										1%	247	768	109.38	1.152	17930	160.00	130.00	142.50	95.00	-			-	-
										0.05%	-	-	-	-	-	-	-	-	-	-			-	-
Local Model 2	222	16015103990000564752	Yelarbon Kurumbul Road	Unsealed	10	10	10	0.5	10	20%	79	29	-	-	167.50	7.50	-	-	-	-	> Further cross-drainage optimization investigations are required during detailed design. > Stakeholder Consultation (Landowner/ Government).			
										10%	81	127	-	0.297	117.50	22.50	10.00	-	-	-		-	-	
										5%	85	162	-	0.312	112.50	20.00	22.50	-	-	-		-	-	
										2%	92	242	-	0.327	112.50	25.00	42.50	-	-	-		-	-	
										1%	98	262	-	0.340	115.00	37.50	45.00	-	-	-		-	-	
										0.05%	-	-	-	-	-	-	-	-	-	-		-	-	
Local Model 2	223	16015103990000564753	Yelarbon Kurumbul Road	Unsealed	10	10	10	0.5	10	20%	-	16	0.518	-	-	2.50	7.50	-	-	> Maturity of current cross-drainage design is limited by available information (e.g. detailed survey) > Rail design constraints alter the existing flow regime.	> Further cross-drainage optimization investigations are required during detailed design. > Stakeholder Consultation (Landowner/ Government).			
										10%	-	13	0.510	-	-	2.50	10.00	-	-			-	-	-
										5%	-	27	30.08	0.510	36	7.50	27.50	10.00	-			-	-	-
										2%	-	35	23.36	0.484	39	15.00	27.50	15.00	-			-	-	-
										1%	-	41	28.30	0.477	64	22.50	27.50	17.50	-			-	-	-
										0.05%	-	-	-	-	-	-	-	-	-			-	-	-

Table Notes

1. Time of Submergence has been measured across the full length of the road segment and does not correspond to the exact location of exceedance within that segment.
2. Offage in water level (offlag) is calculated based on the maximum water surface level difference between the existing and design case results, and does not consider the underlying topography. Depth values presented are based on the respective underlying topographies and in some cases do not align with afflux calculations. This can be attributed to interpolation calculations of results performed within the hydraulic model.
3. Percentage increases should be read in conjunction with existing and design case results. The FIOs require that change be presented in percentage change for time of inundation, severity and hazard and can sometimes produce large increases when existing case results are significantly small. Some locations were identified to experience significant percentage increases however the "absolute" difference between Existing and Design results are relatively small.
4. The Flood Impact Objective exceedance has been identified due to a change in flood extent and a cell which was dry in the existing case and now flood-affected in the developed case.
5. The above table should be read in conjunction with change maps provided as part of Volume II and the Digital Platform.
6. The symbol "-" denotes that no change above Flood Impact Objectives has been determined and therefore no result has been presented.
7. Changes recorded above may only occur during a small fraction of model simulation time.
8. Three decimal places do not infer absolute accuracy in modelling results, but it is necessary to identify small changes.
9. Design optimisations that could be implemented (but not limited to) include adjustment of culvert spacing, realignment of culverts to existing flowpaths when more detailed survey is available, modifying the grade of culverts, road height and alignment adjustments, removal of bridge abutments from waterway extents etc.

The FIO assessment results show that due to the low immunity of roads around the Goondiwindi and Yelarbon area, some impacts result due to the introduction of the Project. Generally, most FIO exceedances identified related to regional flooding are afflux and change in duration of inundation, with some increase in hazard identified on the Cunningham Highway. Local flooding mainly impacts Yelarbon-Kurumbul Road which can be attributed to the maturity of the cross-drainage design and lack of existing survey data in this area. Further work will be undertaken in detailed design to reduce these exceedances through use of additional survey information and consultation with QR to identify further existing infrastructure under the rail line.

19.2.2 Inglewood

State-controlled roads within the vicinity of the Inglewood Assembly Point include the Cunningham Highway, Inglewood-Texas Road and Millmerran-Inglewood Road. A review of Existing Case results shows the following existing flood immunity of these State-controlled roads:

- Cunningham Highway—generally a low immunity, with portions being impacted by the 20% AEP event
- Millmerran-Inglewood Road—generally a low immunity, with portions being impacted by the 20% AEP event
- Inglewood-Texas Road—immunity up to and including the 5% AEP.

An impact assessment was completed against the FIOs established in consultation with the Expert Flood Panel and potential FIO exceedances were identified for the above roads identified in Table 19.7. Localities of FIO exceedances can be seen in Figure 13.7. Note that FIO exceedances experienced within Local Drainage models have not been displayed on figures.

Table 19.7 Inglewood emergency evacuation route impact assessment

Catchment	Road Object ID	Road Segment	Road Name	Surface Type	Flood Impact Objective Target					AEP	Maximum FIO Exceedance					Length (m)			Exceedance Justification	Proposed Mitigation Measures ⁸		Depth (m) ²		Time of Inundation in hours (related to % exceedance)		Time of Submergence (hours) ⁷		Velocity in m/s (related to % exceedance)		Velocity in m/s (related to actual change exceedance)		Hazard in m ² /s (related to % exceedance)																																																																																																																																																																																																																																																																																																																																																																																																																																																	
					Change in Peak Water Level (mm)	Change in Time of Inundation (%)	Change in Velocity (%)	Change in Velocity (m/s)	Change in Hazard (%)		Change in Peak Water Level (mm)	Change in Time of Inundation (% Change) ³	Change in Velocity (%) ³	Change in Velocity (m/s)	Change in Hazard (%) ³	Afflux	Time on Inundation	Velocity				Hazard	Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case	Existing Case	Design Case																																																																																																																																																																																																																																																																																																																																																																																																																																															
Local Model 5	73	16015103990000232840	Millmeran - Inglewood Road	Sealed	10	10	20	1	10	20%	-	-	-	-	-	-	-	-	-	> Rail design constraints alter the existing flow regime.	> Further cross-drainage optimization investigations are required during detailed design. > Stakeholder Consultation (Landowner/ Government).		-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-

The FIO impact assessment identified potential impacts to Millmerran-Inglewood Road and the Cunningham Highway. Generally, FIO exceedances recorded are related to afflux and range from 10 mm to 60 mm. It is anticipated through cross-drainage optimisation, these impacts can be further reduced in detailed design.

19.3 Ongoing consultation

As identified in the above tables, some potential impacts were identified to current potential emergency evacuation routes due to the presence of the Project. Due to variable nature of flooding, emergency access routes are modified for each event, based on severity and location, to safely evacuate residents of each LGA.

The impact assessment has identified mitigation measures for each exceedance to reduce the impact of the Project on surrounding property and infrastructure. During the detailed design stage, these mitigation measures will be implemented and refined within the hydraulic modelling to reduce FIO exceedances where possible. The following workflow will be utilised to address the implementation of mitigation and any residual FIO exceedances:

- Mitigation modelling and FIO exceedance assessment
- Identification of impacted roads where FIO exceedances can't be mitigated
- Assessment of the impacted road as a "link" to determine if the impact is negligible due to flooding elsewhere and the road already inaccessible. This will be assessed against the existing case flooding (i.e. without The Project).
- Classification of impacts into High/Medium/Low impacts based on the road hierarchy (i.e. State-controlled/local/track) and the above "link" assessment
- Consultation with both TRC and GRC LDMG to discuss residual impacts and identify alternative local evacuation procedures, where necessary.

20 Temporary footprint assessment

This section details the assessments completed for temporary Project facilities, including Whetstone MDC, non-resident workforce accommodations, Turallin facility, laydown and other construction infrastructure.

20.1 Site-selection criteria

The selection of temporary Project facility locations for the Project is driven by several factors, including flooding, site access, proximity to the permanent infrastructure and safety. The Project proposes to construct multiple facilities including:

- Whetstone MDC
- Non-resident workforce accommodation at Yelarbon and Inglewood
- A facility at Turallin
- Approximately 77 laydown areas.

Generally, site selection has been undertaken based on the current flood immunity of the site and chosen to avoid areas impacted by the 1% AEP regional floodplain extents wherever possible. It is important to note, due to the constrained nature of the Project footprint, not all sites were able to meet this criteria.

A FRA has been completed for each of the non-resident workforce accommodation locations, Whetstone MDC, the Turallin site facility and the laydown areas. The outcomes of these assessments are detailed in Sections 20.2 to 20.5.

20.2 Whetstone Material Distribution Centre

As part of the overall track pre-construction, early works and construction works delivery strategy for the Project, ARTC will establish a temporary MDC in Whetstone, Queensland (the Whetstone site).

A detailed description of the Whetstone site, including proposed staging, is presented in EIS Appendix AE: Whetstone Material Distribution Centre Supporting Technical Information. A flood risk assessment was undertaken for the MDC, as presented in the following sections.

20.2.1 Site location

The Whetstone site is located approximately 18 km south-west of Inglewood and 59 km east of Goondiwindi in the GRC LGA, as shown on Figure 20.1. The land on which the MDC is proposed is described as Part of Lot Plan 2MH784, 4MH287, 74MH313, 76MH313 and 352SP116434.

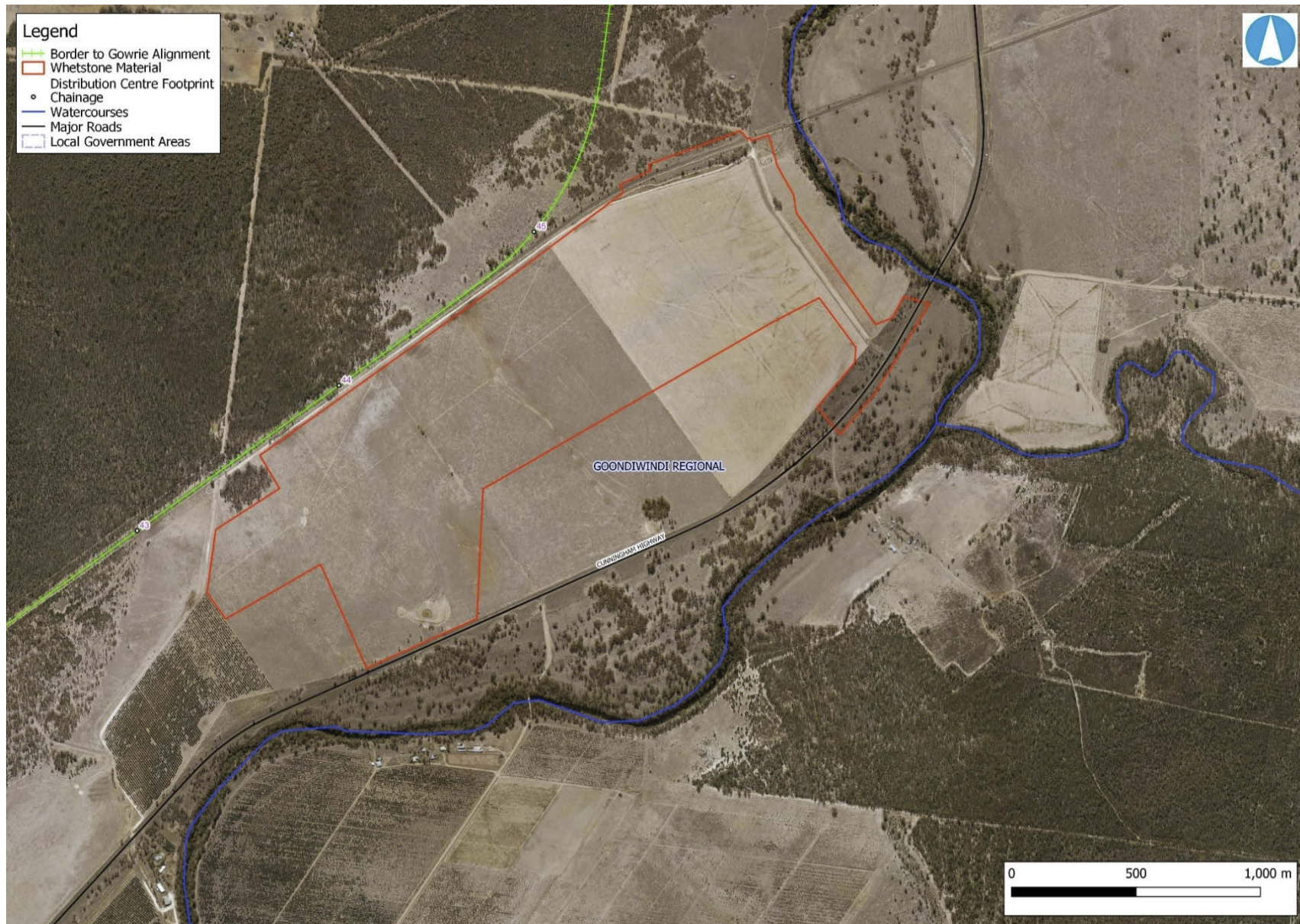


Figure 20.1 Whetstone MDC site –Location Plan

20.2.2 Results

The Macintyre Brook—Yelarbon to Inglewood regional flood model, as detailed in Section 14, was used to assess the existing flooding behaviour within and around the Whetstone site for the 10%, 5% and 1% AEP events.

The simulated flood depth, velocity and velocity-depth product results within and around the Whetstone site are discussed in the subsections below.

Flood maps for the 10%, 5% and 1% AEP event are provided below.

20.2.2.1 Flow depth

Figure 20.2, Figure 20.3 and Figure 20.4 presents the predicted peak overland flood depth for the 10% AEP, 5% AEP and 1% AEP events, respectively.

The flood depth results show:

- This MDC is situated directly adjacent to the Macintyre Brook main channel and is susceptible to breakout flow from the watercourse. The flood depths at the MDC site vary significantly. The site generally starts to become inundated in the 10% AEP event on the north-eastern boundary. In the 5% AEP, the site is fully inundated with flood depths generally ranging from 0.2 m to 0.8 m (with isolated pockets up to 2.6 m on the southern boundary), while the 1% AEP depths generally reach up to 2 m across the site.
- Localised areas of the site are predicted to experience between 2.5–3.7m of inundation in the 1% AEP event, and are isolated to the southern boundary of the MDC.
- The north-eastern corner of the site experiences the shallowest depth with flood depths expected to be approximately 0.3 m in a 5% AEP and 0.8 m to 1.2 m in a 1% AEP event.
- The proposed MDC access roads along with the connecting roads, Whetstone Access Road and Cunningham Highway, are inundated over 1 m, and as such would not be accessible under a 5% AEP and 1% AEP event.
- According to the AIDR (2017), under events of this magnitude the site would be inaccessible and is considered unsafe for both vehicles and people, with some potential for structural damage and building failure, specifically within localised areas of the site that are subject to the deeper flow.

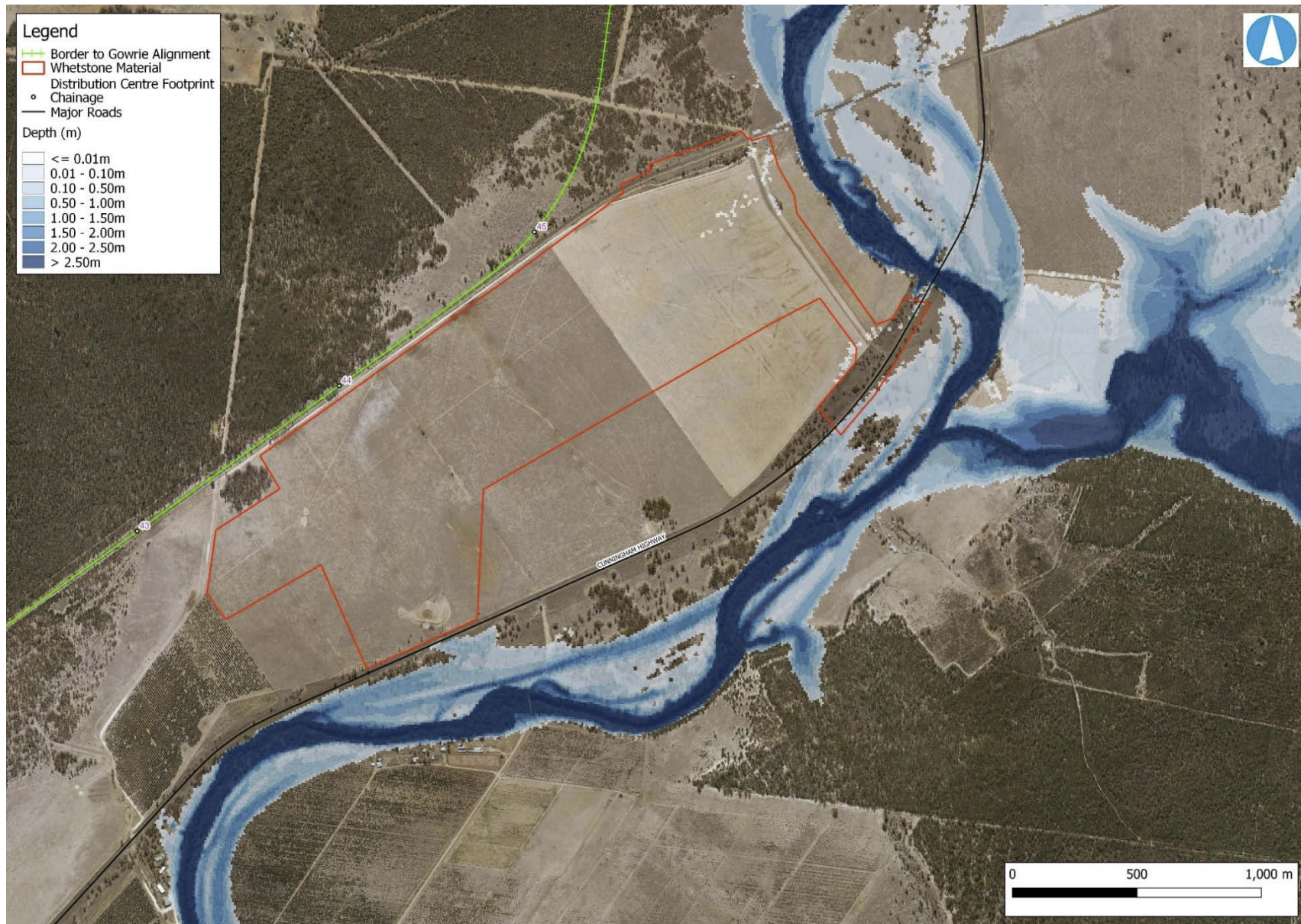


Figure 20.2 Whetstone MDC site—10% AEP peak flood depth

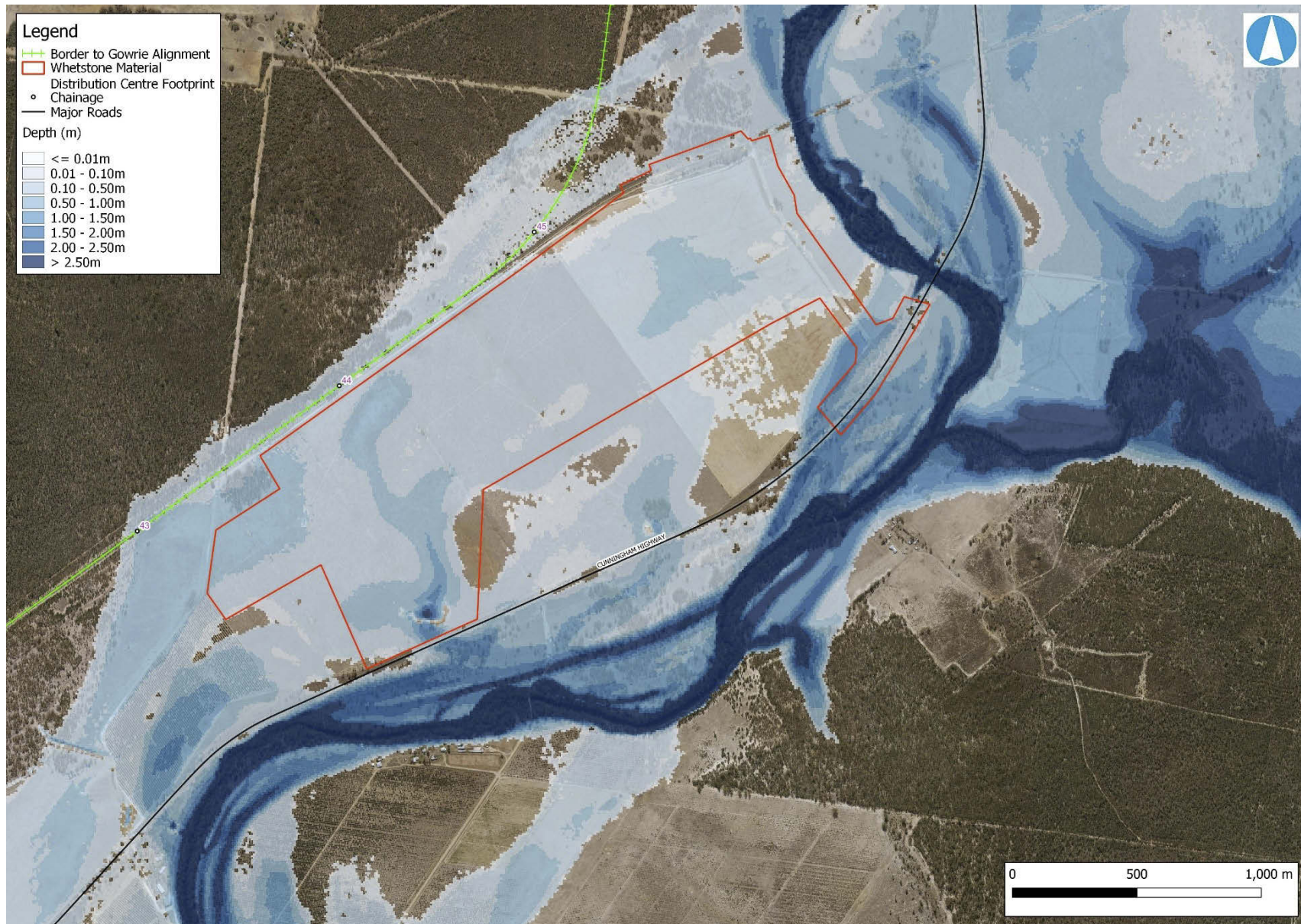


Figure 20.3 Whetstone MDC site—5% AEP peak flood depth

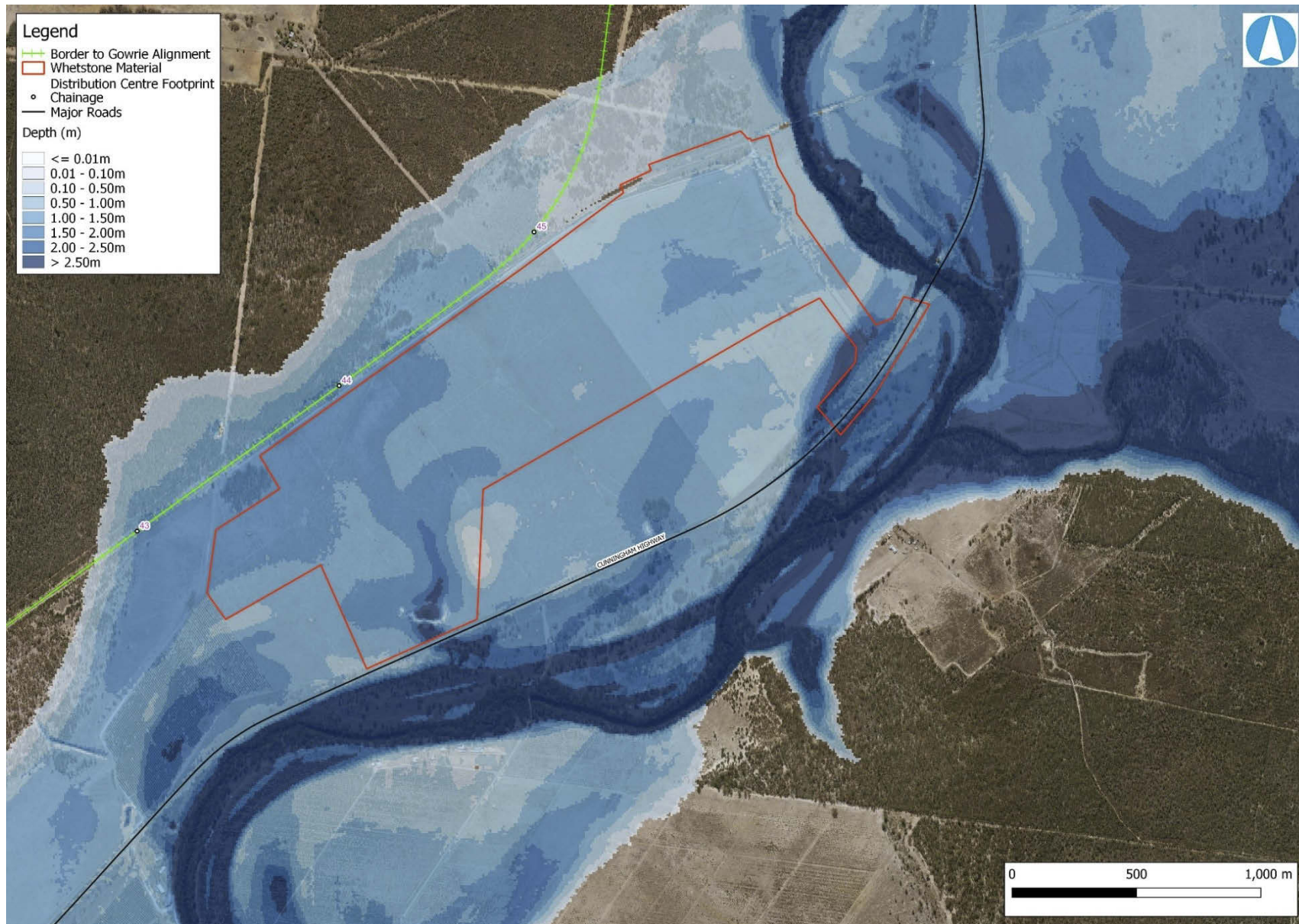


Figure 20.4 Whetstone MDC site—1% AEP peak flood depth

20.2.2.2 Flow velocities

Figure 20.5, Figure 20.6 and Figure 20.7 presents the predicted peak overland flow velocity for the 10% AEP, 5% AEP and 1% AEP events, respectively.

The flow velocity results show:

- In general, the MDC site is subject to slower moving breakout flow from the Macintyre Brook, with velocities generally less than 0.5 m/s in the 10% AEP and 5% AEP events, with an average peak velocity of approximately 0.56 m/s in the 1% AEP event. These typically lower velocities are expected as the floodplain function at the MDC site location is mainly flood storage.
- Localised areas of the site are subject to velocities up to 1 m/s in the 1% AEP event, these areas are mainly on the southern edge of the MDC, located directly adjacent to the main Macintyre Brook Channel and the Cunningham Highway.

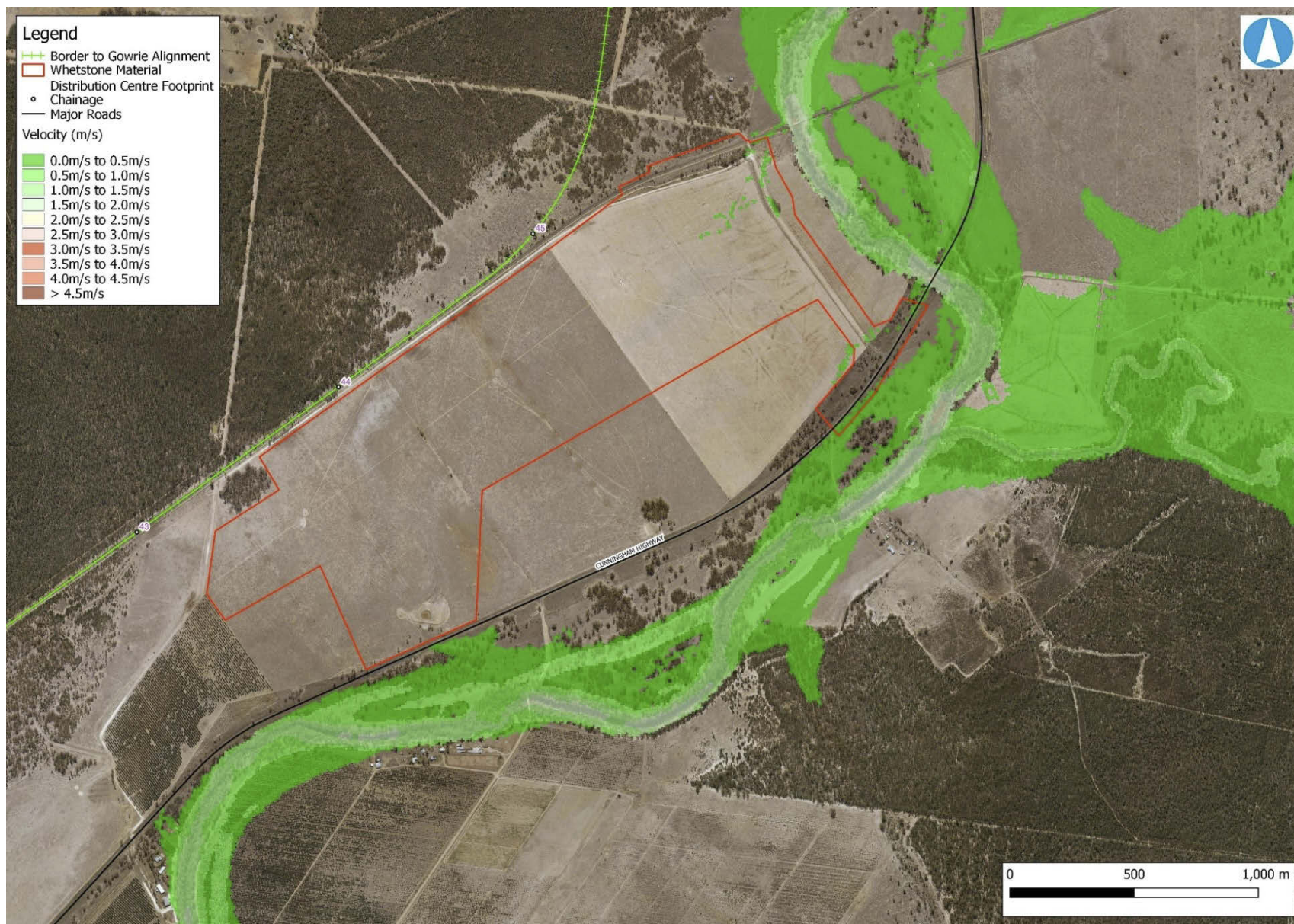


Figure 20.5 Whetstone MDC site—10% AEP peak flood flow velocities

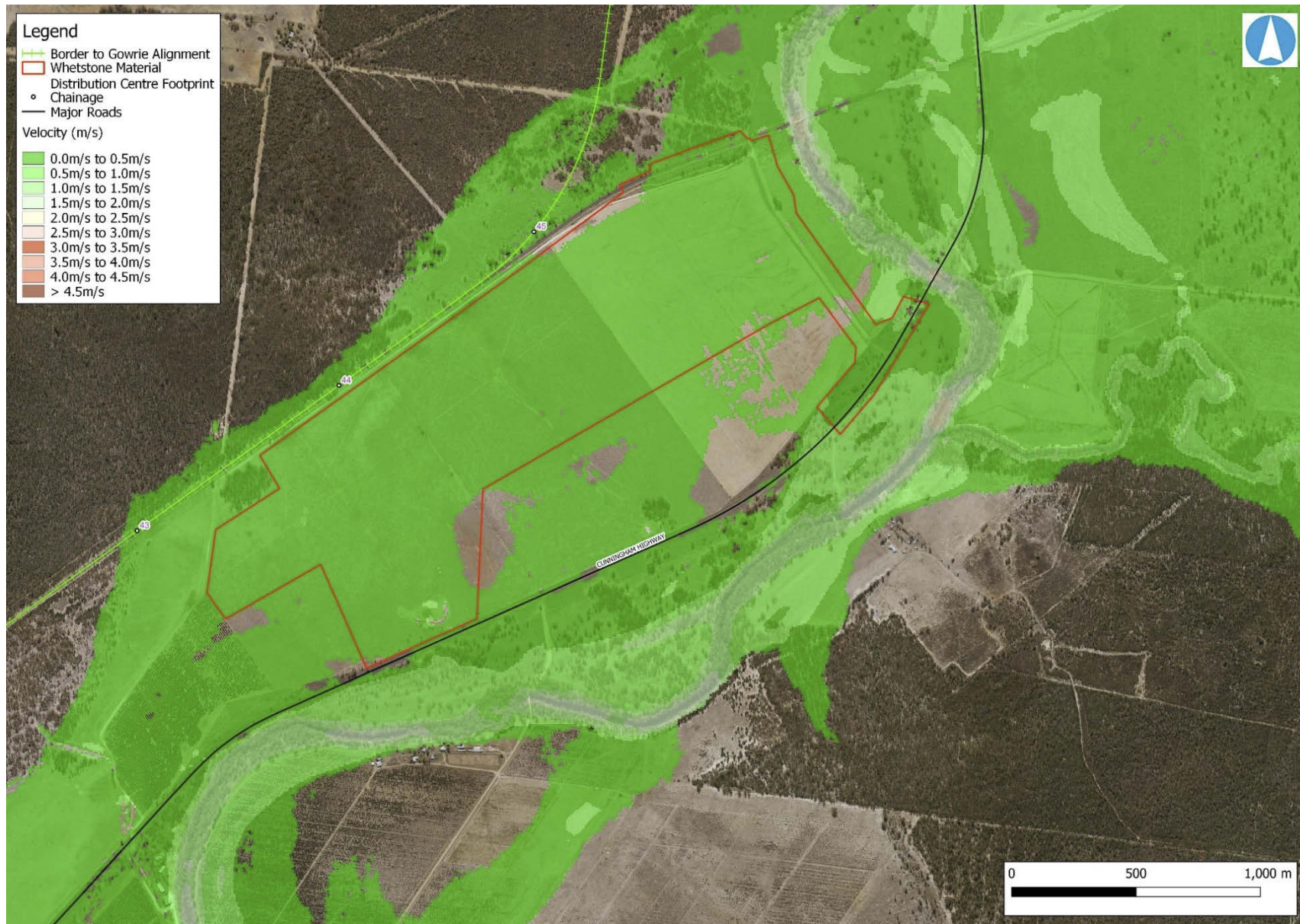


Figure 20.6 Whetstone MDC site—5% AEP peak flood flow velocities

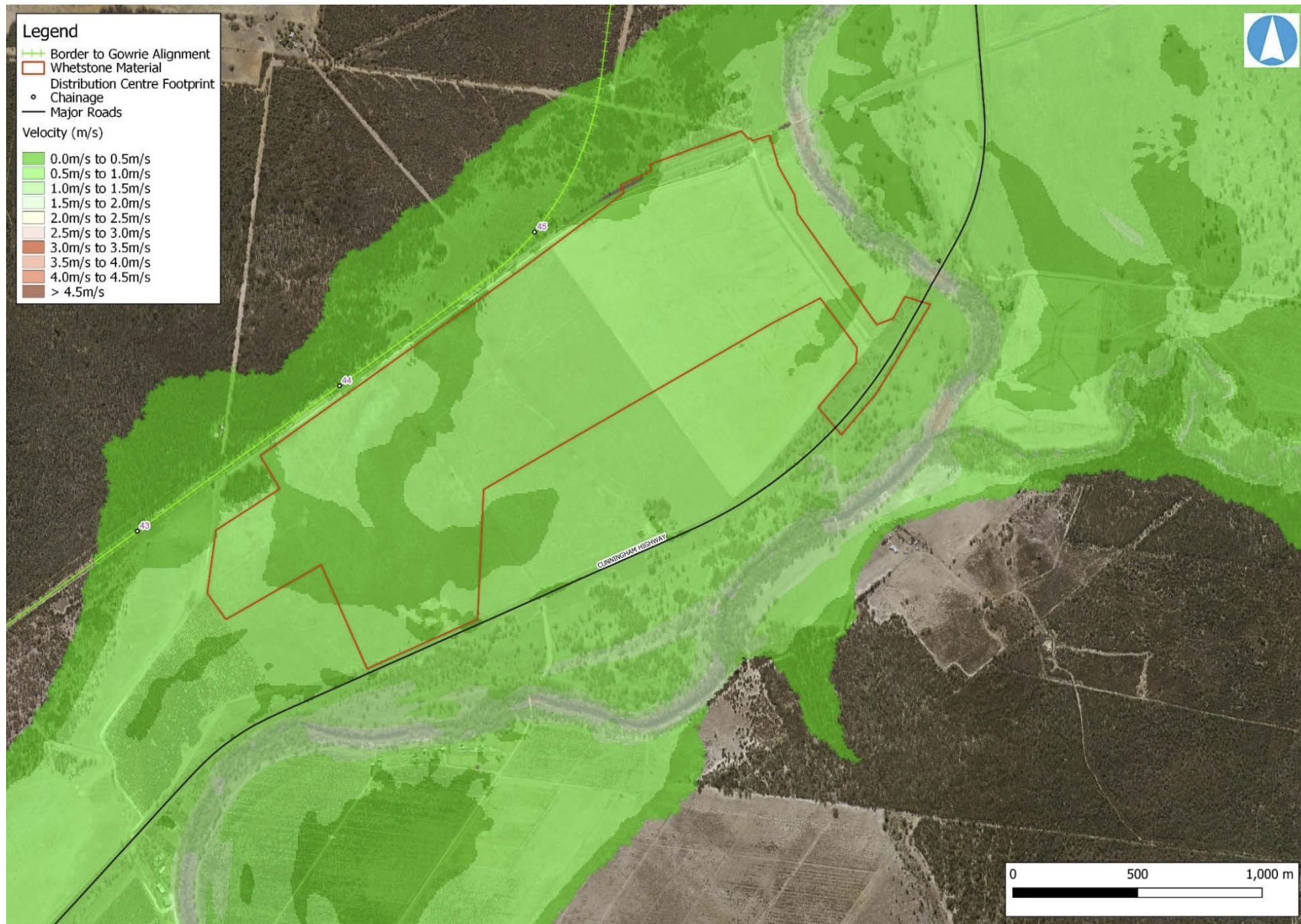


Figure 20.7 Whetstone MDC site—1% AEP peak flood flow velocities

20.2.2.3 Flood hazard

Figure 20.8, Figure 20.9 and Figure 20.10 present the predicted peak overland flow velocity × depth product for the 10% AEP, 5% AEP and 1% AEP events, respectively. The velocity × depth product represents the combined dynamics of flood depth and velocity, and is applied to quantify and classify flood hazard in a risk-management setting, as defined by the AIDR (2017).

The velocity × depth product results show:

- In the 10% AEP and 5% AEP events, the velocity x depth product values across the site are generally below 0.15 m²/s, with small pockets reaching values of 0.6 m²/s in the 5% AEP event.
- For the 1% AEP event, the velocity x depth product values across the site range between 0.02 m²/s—2.67 m²/s, with the average peak velocity x depth product across the MDC site equal to 0.75 m²/s.
- As most of the site is subject to inundation over 1 m (although the velocities are considered low) under a 5% AEP and 1% AEP event, the MDC site would be considered unsafe for people and vehicles (AIDR, 2017). Areas of the MDC site where deeper inundation is predicted would also be vulnerable to structural damage with less robust building types vulnerable to failure (AIDR, 2017).
- The proposed MDC access roads along with the connecting roads, Whetstone Access Road and Cunningham Highway, are predicted to be inundated over 1 m and a velocity x depth product > 0.6 m²/s in the 5% AEP and 1% AEP events. These roads would be considered unsafe for people and vehicles (AIDR, 2017) and therefore would not be accessible under the 5% AEP and 1% AEP events.

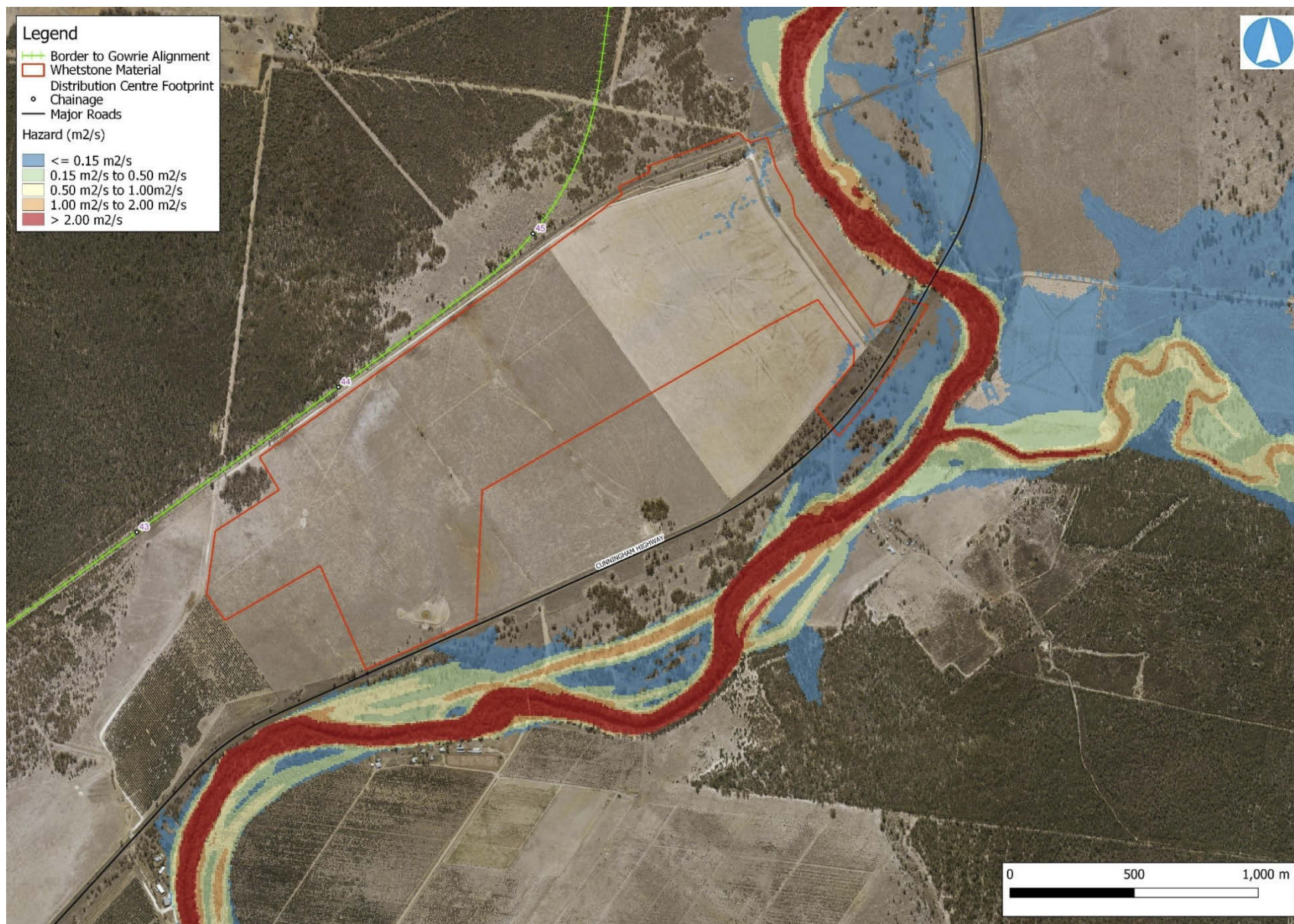


Figure 20.8 Whetstone MDC site—10% AEP peak flow velocity × depth product

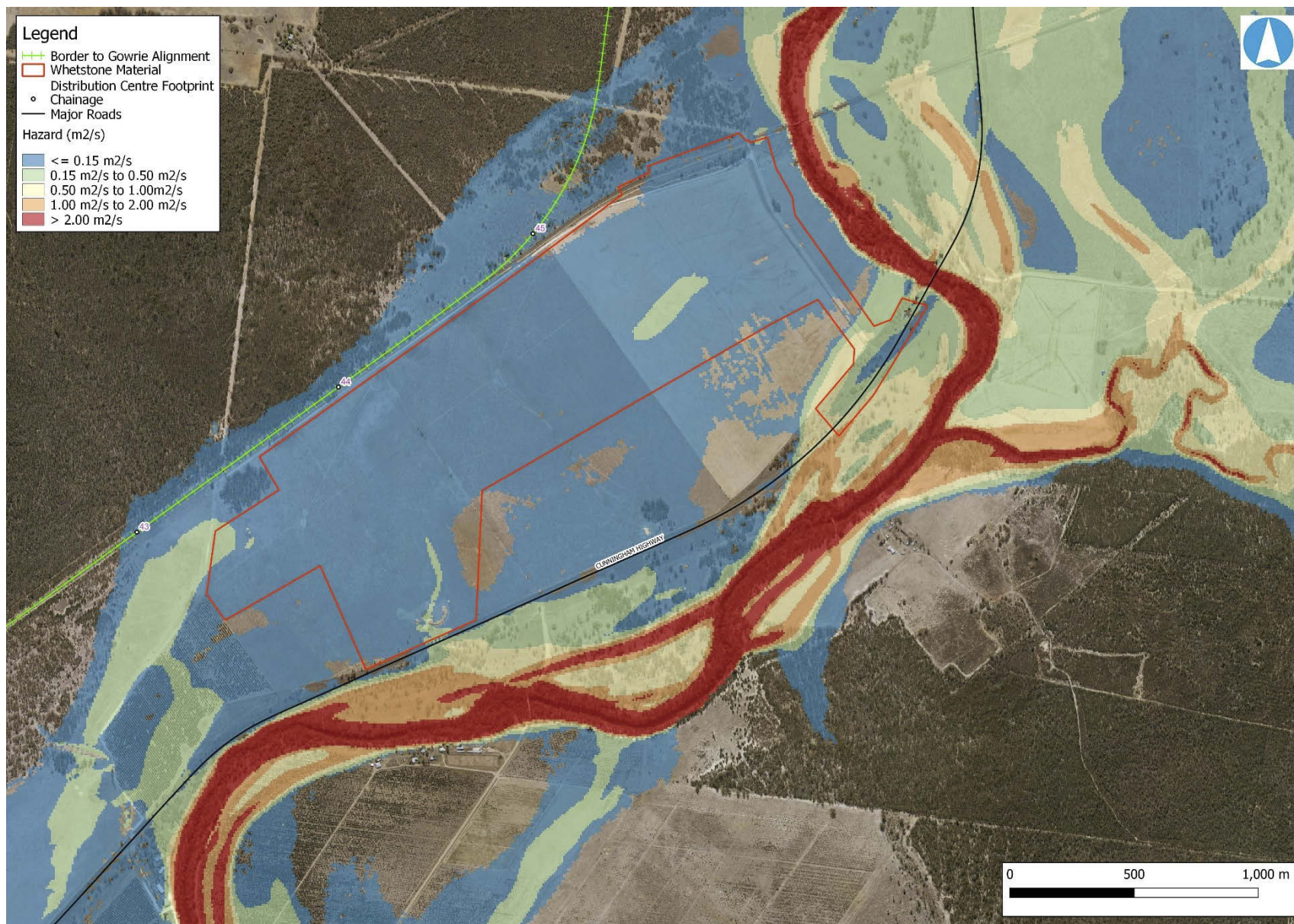


Figure 20.9 Whetstone MDC site—5% AEP peak flow velocity × depth product

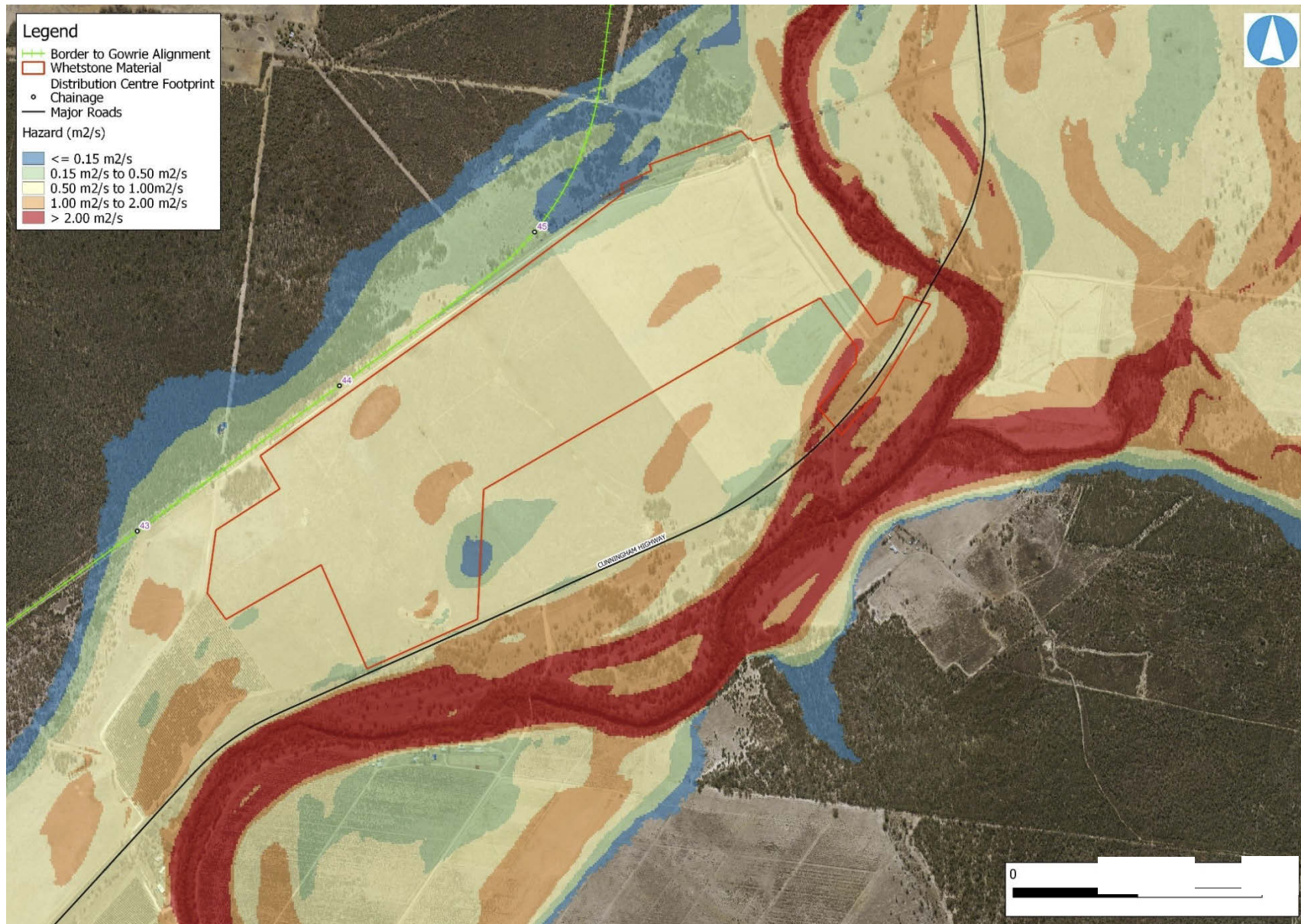


Figure 20.10 Whetstone MDC site—1% AEP peak flow velocity × depth product

20.2.3 Conclusion and mitigation measures

The main outcomes of the Whetstone MDC flood risk assessment are summarised as follows:

- The MDC site at Whetstone is located in a high-risk flood zone on the Macintyre Brook floodplain and the north-eastern parts of the site starts to become partially inundated in the 10% AEP event
- The Whetstone MDC becomes fully inundated in the 5% AEP event, with surrounding access roads also being inundated in this event
- In a 5% AEP and 1% AEP event, the MDC site at Whetstone would be subject to flood depths over 1 m in locations and as per AIDR (2017) guidelines would be considered hazardous for vehicles and people, with potential for structural damage and/or failure to less robust building types
- The proposed MDC access roads along with the connecting roads, Whetstone Access Road and Cunningham Highway are predicted to be inundated with over 1 m of depth and a velocity x depth product $> 0.6 \text{ m}^2/\text{s}$. These roads would be considered unsafe for people and vehicles and inaccessible during a 5% AEP and 1% AEP event.

The additional mitigation measures below are proposed for future use of the Whetstone MDC in consideration of the current flood risk assessment outcomes:

- Further optimisation of the Project's cross-drainage culverts to mitigate afflux at Whetstone MDC
- The site be evacuated during the 5% AEP event and greater due to access to and from the site not being available
- Testing of mitigation options to alleviate and reduce flood levels within the site should be conducted during the detailed design stage. Any changes resulting from the construction of the Whetstone MDC are anticipated to be limited to localised impacts contained primarily within the property footprint. The design is intended to allow the land to handle flooding events without adversely affecting adjacent stakeholders, ensuring a balance between the site's flood immunity and the broader community's.
- An onsite stormwater management plan is recommended to effectively manage the flow within the site and discharge to the receiving environment under developed conditions
- A Flood Risk Management Plan will be developed for the site to ensure risks to people/workers and the environment are managed through the construction and operation of the facility
- Citing of any office/workshop buildings should consider the outcomes of this flood risk assessment and be located in areas away from hazardous areas. Offices are proposed to be located in the northeast of the site falling within the lower flood depths areas.
- Buildings and other critical infrastructure, such as electrical and chemical equipment, are proposed to be constructed on elevated foundations or pilings to lift them above the expected flood levels. The piles will utilise reinforced concrete or steel to support elevated platforms and working spaces above the peak flood depth.

20.3 Non-resident workforce accommodations

The non-resident workforce accommodations have been proposed to primarily service the accommodation requirements of workforce of the Project. The initial assessment of workforce demand and safe commutable distances has identified a potential need for non-resident workforce accommodation in the vicinity of Yelarbon, Inglewood and Millmerran townships.

The proposed non-resident workforce accommodation locations included in the assessment have been identified near the Yelarbon and Inglewood. A third non-resident workforce accommodation location will be identified near the Millmerran area by the Contractor during detailed design. Each accommodation will be required to hold a minimum of 300 staff during the peak.

The locations of the two non-resident workforce accommodation facilities have been included in the Project footprint. The locations of these are summarised below:

- Cunningham Highway, Yelarbon—Lot 30 MH721
- Millmerran-Inglewood Road, Inglewood—Lot 5 MH75.

Further information regarding the non-resident workforce accommodation facilities is provided in Sections 20.3.1 and 20.3.2.

20.3.1 Yelarbon workforce accommodation

20.3.1.1 Site location

The Yelarbon non-resident workforce accommodation (the Yelarbon site) will be located approximately 2.5 km northwest of Yelarbon, with access into the accommodation off the Cunningham Highway. The Yelarbon site has been identified as having a total footprint area of approximately 200,000 m² for assessment as part of this revised draft EIS.

The Yelarbon site is located in the headwater region of Brigalow Creek, and its flood characteristics are driven by breakout flood flows from the Macintyre Brook north of Yelarbon. The Yelarbon site is located within the bounds of the existing Macintyre Brook (Yelarbon to Inglewood) flood model; therefore, an assessment was completed based on the revised draft EIS modelling, as reported in Section 14.

A locality plan of the proposed Yelarbon site is shown in Figure 20.11.

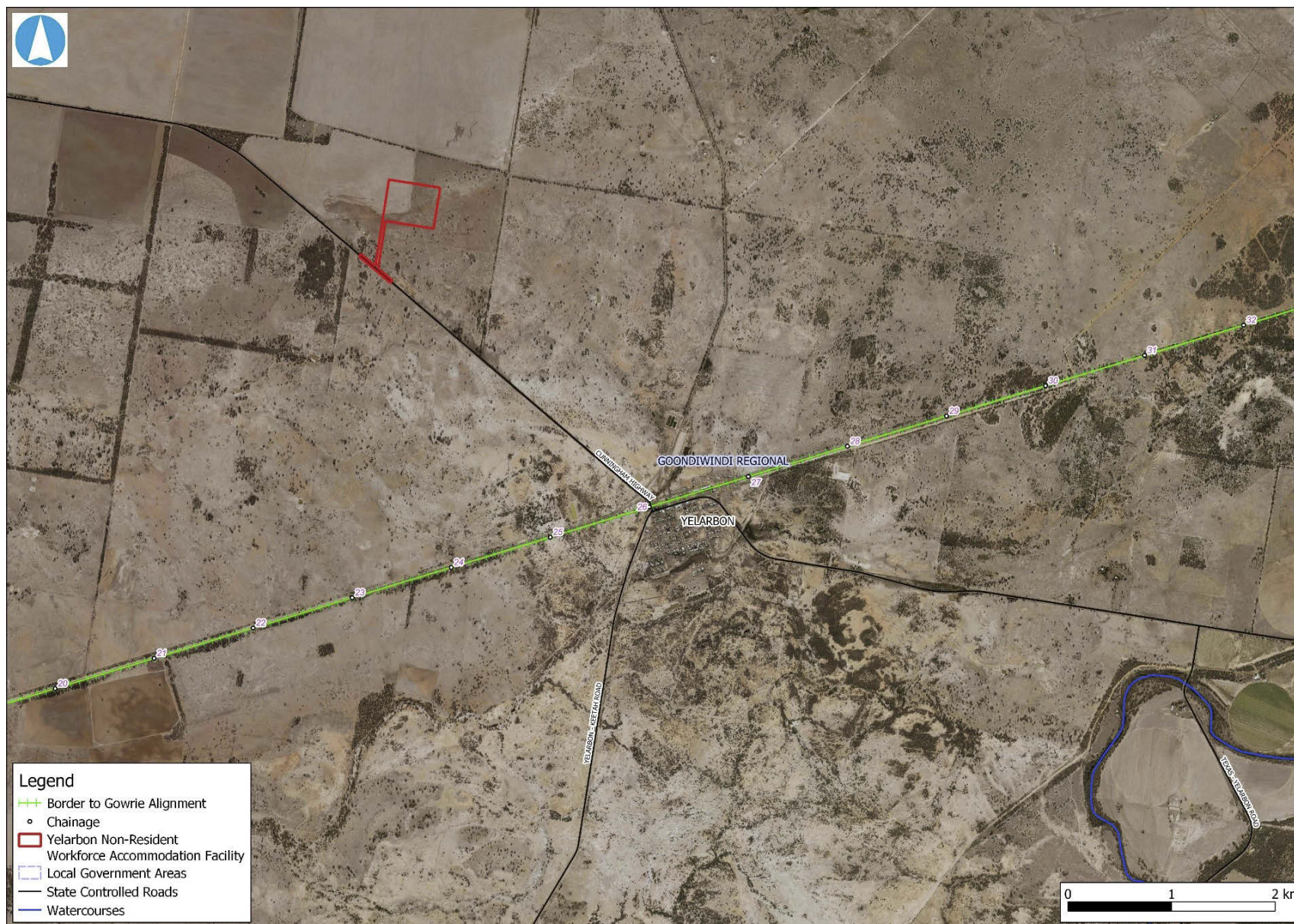


Figure 20.11 Yelarbon site—Locality plan

20.3.1.2 Results

The Macintyre Brook—Yelarbon to Inglewood regional flood model, as detailed in Section 14, was used to assess the existing flooding behaviour within and around the Yelarbon site for the 1% AEP event.

The simulated flood depth, velocity and velocity-depth product results within and around the Yelarbon site are discussed in the subsections below.

Flood maps for the 1% AEP event are provided below.

Flow depth

Figure 20.12 presents the predicted peak overland flood depth for the 1% AEP event.

The flood depth results show:

- The main Yelarbon site is flood free in the 1% AEP event
- The proposed access to Cunningham Highway is potentially affected by flood depths of up to 0.67 m.

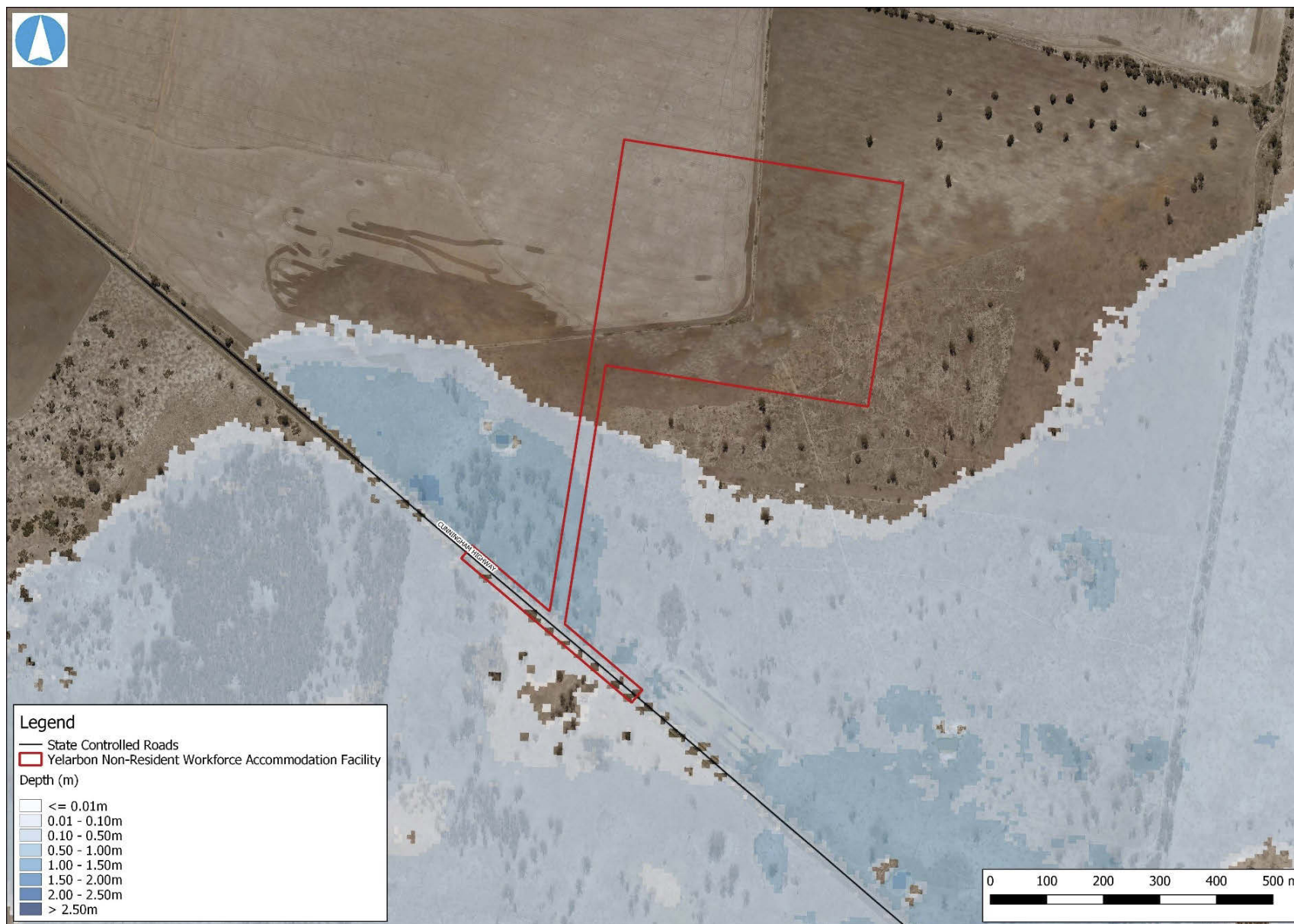


Figure 20.12 Yelarbon site—1% AEP peak flood depth

Flow velocities

Figure 20.13 presents the predicted peak overland flow velocity for the 1% AEP event.

The flow velocity results show:

- The main Yelarbon site is flood free in the 1% AEP event
- The proposed access to Cunningham Highway is affected by very low flow velocities due to ponding behind the highway. The peak velocities affecting the access is <0.3 m/s.

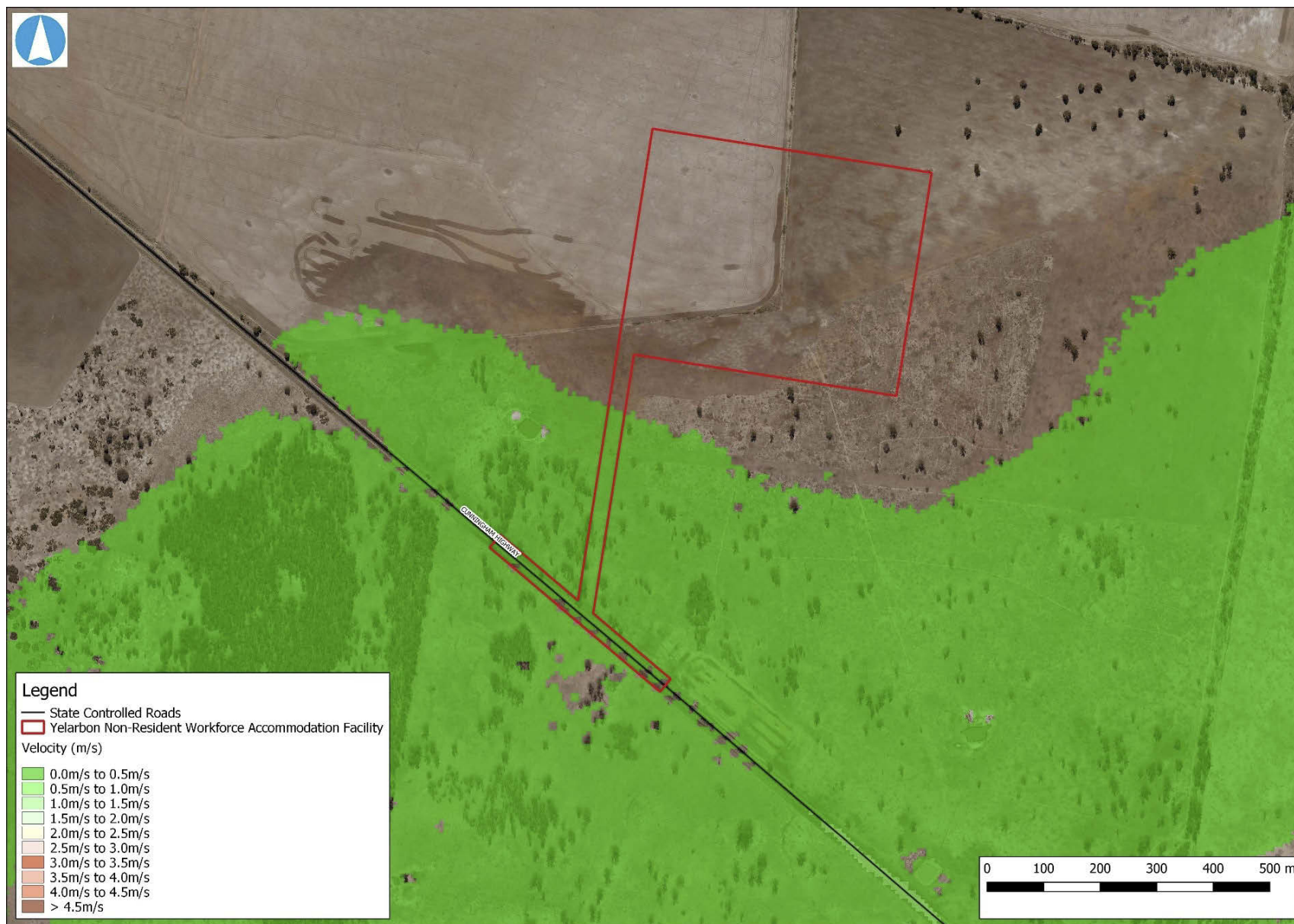


Figure 20.13 Yelarbon site –1% AEP peak flood flow velocities

Flood hazard

Figure 20.14 presents the predicted peak overland flow velocity $\times$ depth product for the 1% AEP event. The velocity $\times$ depth product represents the combined dynamics of flood depth and velocity and is applied to quantify and classify flood hazard in a risk management setting as defined by the AIDR (2017).

The velocity $\times$ depth product results show:

- The proposed access to Cunningham Highway is affected by velocity $\times$ depth product values $< 0.15 \text{ m}^2/\text{s}$. These areas are considered generally safe for all patrons as per the AIDR (2017) guideline.
- On the Cunningham Highway south of the Yelarbon site the velocity $\times$ depth values increase to $0.2 \text{ m}^2/\text{s}$.

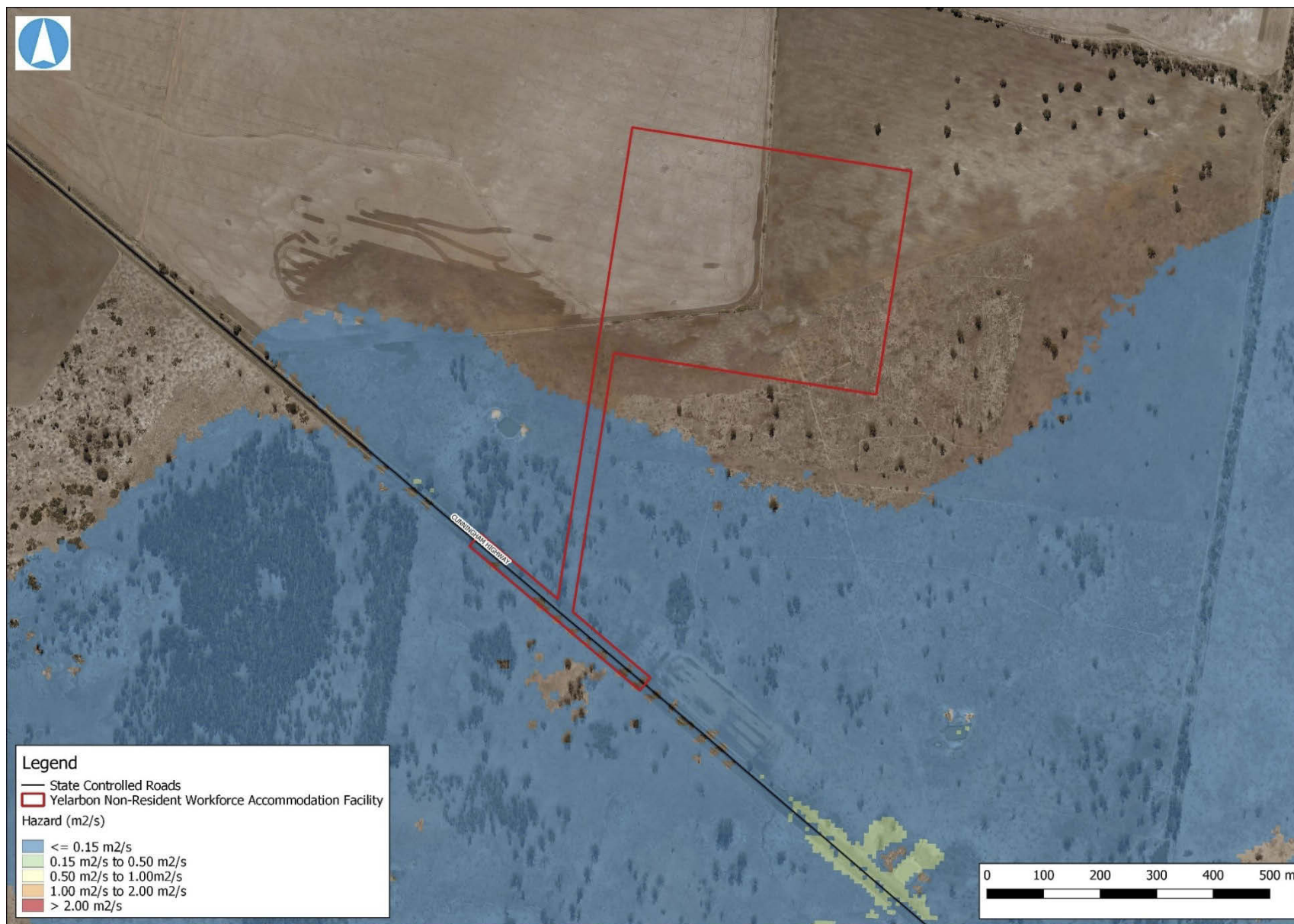


Figure 20.14 Yelarbon site -1% AEP peak flow velocity × depth product

20.3.1.3 Conclusions and mitigation measures

The main outcomes of the Yelarbon site flood risk assessment are summarised as follows:

- The Yelarbon site is not affected by flooding in the 1% AEP event under existing conditions; however, the proposed access to Cunningham Highway is potentially affected by flooding.

The following mitigation measures are proposed for future use of the Yelarbon site in consideration of the current flood risk assessment outcomes:

- During the detailed design stage, the location and nature of the access road needs to be re-considered to ensure that safe access and egress to and from the Yelarbon site is achievable, and that any road works don't exacerbate flood risk elsewhere
- An onsite storm water management plan is recommended to effectively manage the overland flow within the Yelarbon site and discharge to the receiving environment under developed conditions
- A Flood Risk Management Plan will be developed for the site to ensure risks to people/workers and the environment are managed through the construction and operation of the facility.

20.3.2 Inglewood workforce accommodation

20.3.2.1 Site location

The Inglewood non-resident workforce accommodation site (the Inglewood site) is to be located approximately 12 km northeast of Inglewood, with access to the accommodation from Millmerran–Inglewood Road. The Inglewood site has been identified as having a total footprint area of approximately 200,000 m² for assessment as part of this revised draft EIS.

The Inglewood site area straddles the ridgeline splitting the Pariagara Creek and Canning Creek catchments. Sandy Creek—a tributary of Pariagara Creek—flows southwest of the site. The Inglewood site area is only subject to overland flow and not affected by creek/river flooding. A flood model was developed to assess flood risk associated with the site.

A locality plan of the proposed Inglewood site is shown in Figure 20.15.

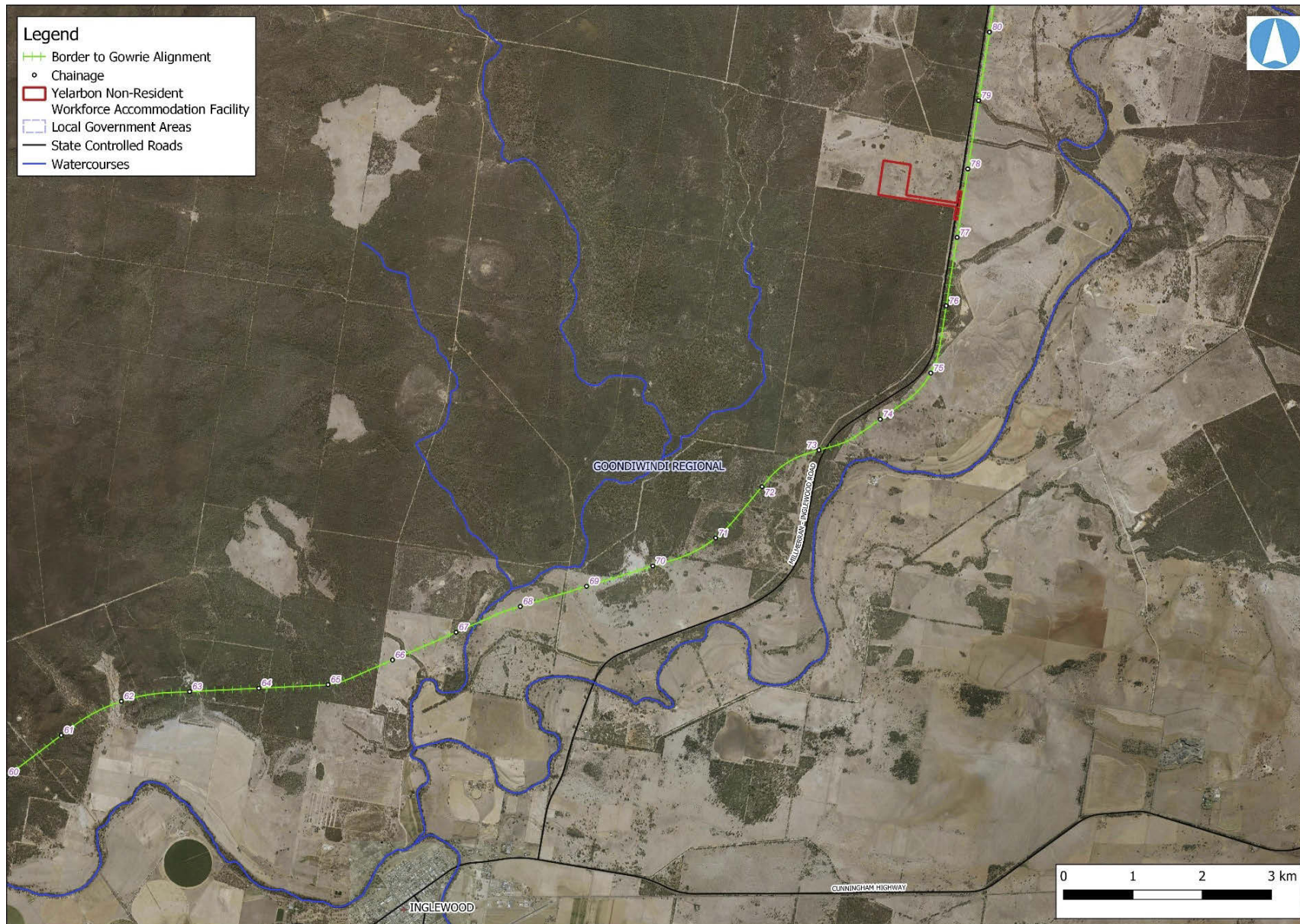


Figure 20.15 Inglewood site –Locality plan

20.3.2.2 Hydrology

A Rain on Grid (RoG) modelling approach was selected based on the Inglewood site characteristics. This is a commonly adopted approach for overland flow assessment. The RoG application applies rainfall directly to the hydraulic model cells and eliminates the need for a separate hydrology model.

The RoG modelling was undertaken in accordance with the ensemble method described in ARR 2019 guidelines, where each AEP event is assessed for a range of storm durations across 10 TPs.

The adopted 1% AEP IFD depths are summarised in Table 20.1. These were extracted from the BoM design rainfall database at the Inglewood site centroid.

Table 20.1 Adopted 1% AEP IFD depths (mm)

Duration	Depth (mm)
30 min	60
1 hour	75.8
1.5 hour	84.7
3 hour	100
4.5 hour	110
6 hour	118
9 hour	131

The adopted 1% AEP design losses are summarised in Table 20.2. Design losses were applied uniformly on the RoG model and were informed by a review of the adopted catchment losses on adjacent Project catchments:

- A conservative storm initial loss (IL) of 0 mm was adopted for the present assessment, noting that no additional pre-burst loss was applied. The resulting 1% AEP burst initial loss is similar to that applied in the Canning Creek model (Section 9).
- A continuing loss (CL) of 0.43 mm/hr was adopted based on the Canning Creek model, which is more conservative than that adopted for the adjacent Pariagara Creek catchment (1 mm/hr) (Section 13).

Table 20.2 Adopted 1% AEP design losses and comparison with adjacent Project catchment losses

Description	Storm initial loss (mm)	Continuing loss (mm/hr)
Adopted	0 ¹	0.43
Canning Creek	6.4	0.43
Pariagara Creek	15	1

Table note:

1 Pre-burst was not applied on zero initial storm loss

20.3.2.3 Hydraulics

A 2-dimensional RoG hydraulic model was developed using the TUFLOW software (version 2020-10-AB) to simulate the overland flow characteristics within and around the Inglewood site area.

The hydraulic model schematisation is shown in Figure 20.16.

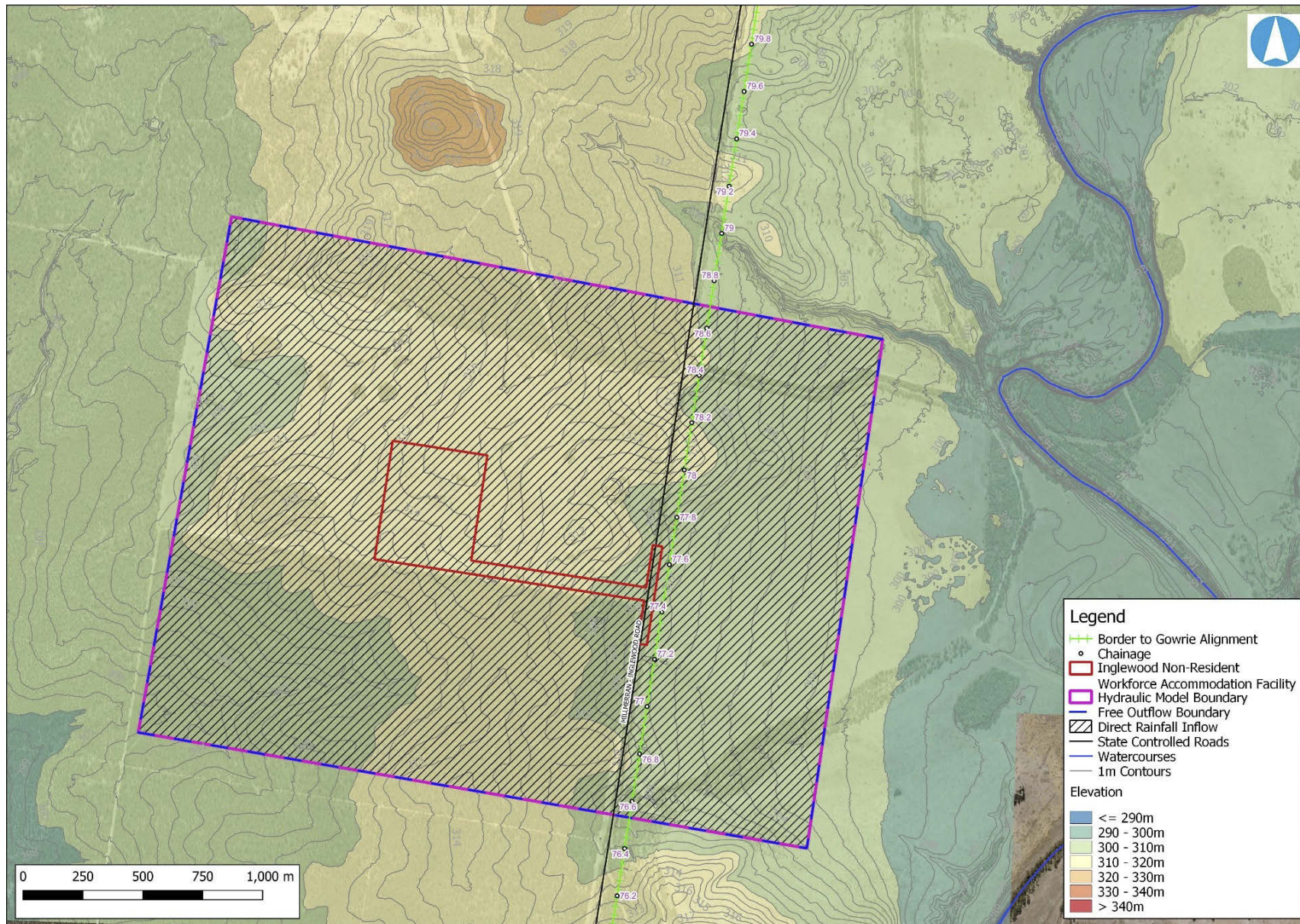


Figure 20.16 Ingelwood site –Hydraulic model schematisation

The model was implemented as follows:

- The model terrain was defined using Airborne Laser Scanning (ALS) data provided by ARTC. The Inglewood site area is located on a hilltop surrounded by relatively steep terrain with an approximate elevation range of 307 to 316 mAHD sloping towards south and east.
- The model domain was defined by a 5 m × 5 m rectilinear grid supplemented by a 1 m SGS resolution to characterise small-scale topographic variations. This configuration was selected to achieve an optimal balance between modelling resolution and computational efficiency.
- Direct rainfall was applied over the entire hydraulic model domain. A free overfall boundary was applied along the perimeter of the hydraulic model to ensure free outflow of water. The outflow boundary was delineated sufficiently far from the site area to ensure no impact on the hydraulics within and immediately around the Inglewood site area.
- Surface roughness values in the hydraulic model were assigned based on land use types determined from Google Earth aerial imagery. Table 20.3 summarises the adopted Manning's 'n' values informed by those adopted in the draft EIS submission. The land-use classifications are shown in Figure 20.17.



Figure 20.17 Inglewood site—Land-use classification

The hydraulic model was used to simulate the 1% AEP overland flow within and around the Inglewood site. A total of 11 storm durations between 10 minutes and 9 hours were assessed in line with the ARR 2019 ensemble method.

Table 20.3 Manning's 'n' values

Description	Manning's n
Pasture	0.055
Roads	0.022
Buildings	0.10
Open water	0.025
Waterway	0.035
High vegetation	0.65

20.3.2.1 Results

The RoG model was used to assess the existing overland flow characteristics within and around the Inglewood site for the 1% AEP event. The Inglewood site area is not affected by flooding from other sources (i.e. creek or river flooding).

The simulated flood depth, velocity and velocity-depth product results within and around the Inglewood site are discussed in the subsections below.

Flow depth

Figure 20.18 presents the predicted peak overland flood depth for the 1% AEP event.

The flood depth results show:

- The Inglewood site area is split into a western local catchment and an eastern local catchment at a high point around the middle of the site. The western site catchment drains south while the eastern site catchment drains east, as indicated by the flow arrows in Figure 20.18.
- The southern drainage path is briefly intercepted by a local access track before continuing south. The local access track may experience overtopping depths up to 250–300 mm in the 1% AEP overland flow event.
- The eastern drainage path flows along the local access track and is intercepted at the Millmerran-Inglewood Road intersection before turning south. Millmerran-Inglewood Road may experience very shallow overtopping (i.e. less than 50 mm overtopping depth) in the 1% AEP overland flow event.
- The Inglewood site area is characterised by generally shallow inundation below 200 mm
- Subject to a mild flow velocity, a flow depth below 300 mm is considered generally safe for all patrons according to the AIDR (2017) classification
- Figure 20.18 shows one area next to the proposed Inglewood site access onto Millmerran-Inglewood Road with a ponding depth of 350 mm.
- A flow depth between 300 mm and 500 mm is considered generally safe despite slightly elevated risk to small vehicles (AIDR, 2017).

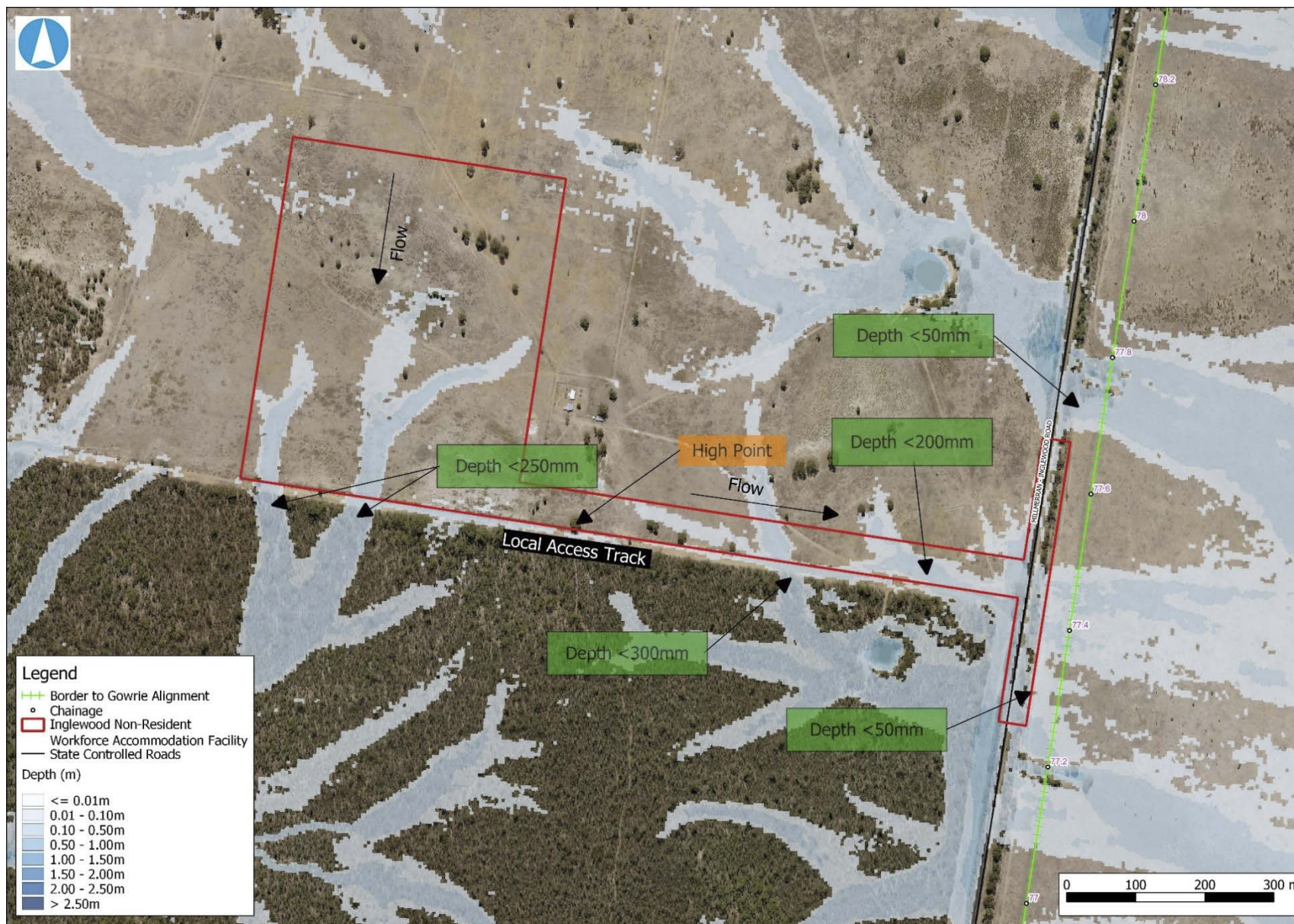


Figure 20.18 Inglewood site -1% AEP peak flood depth

Flow velocities

Figure 20.19 presents the predicted peak overland flow velocity for the 1% AEP event.

The flow velocity results show:

- The proposed Inglewood site area is characterised by slow moving overland flow generally up to 0.6 m/s
- The AIDR (2017) guideline considers a velocity below 2.0 m/s generally safe for all patrons subject to shallow flood depths across the site
- The location of maximum ponding at the Inglewood site entrance is associated with nearly stagnant water (i.e. velocity < 0.1 m/s). The flood risk in this area is considered minimal except for a minor increase in hazard to parked small vehicles.

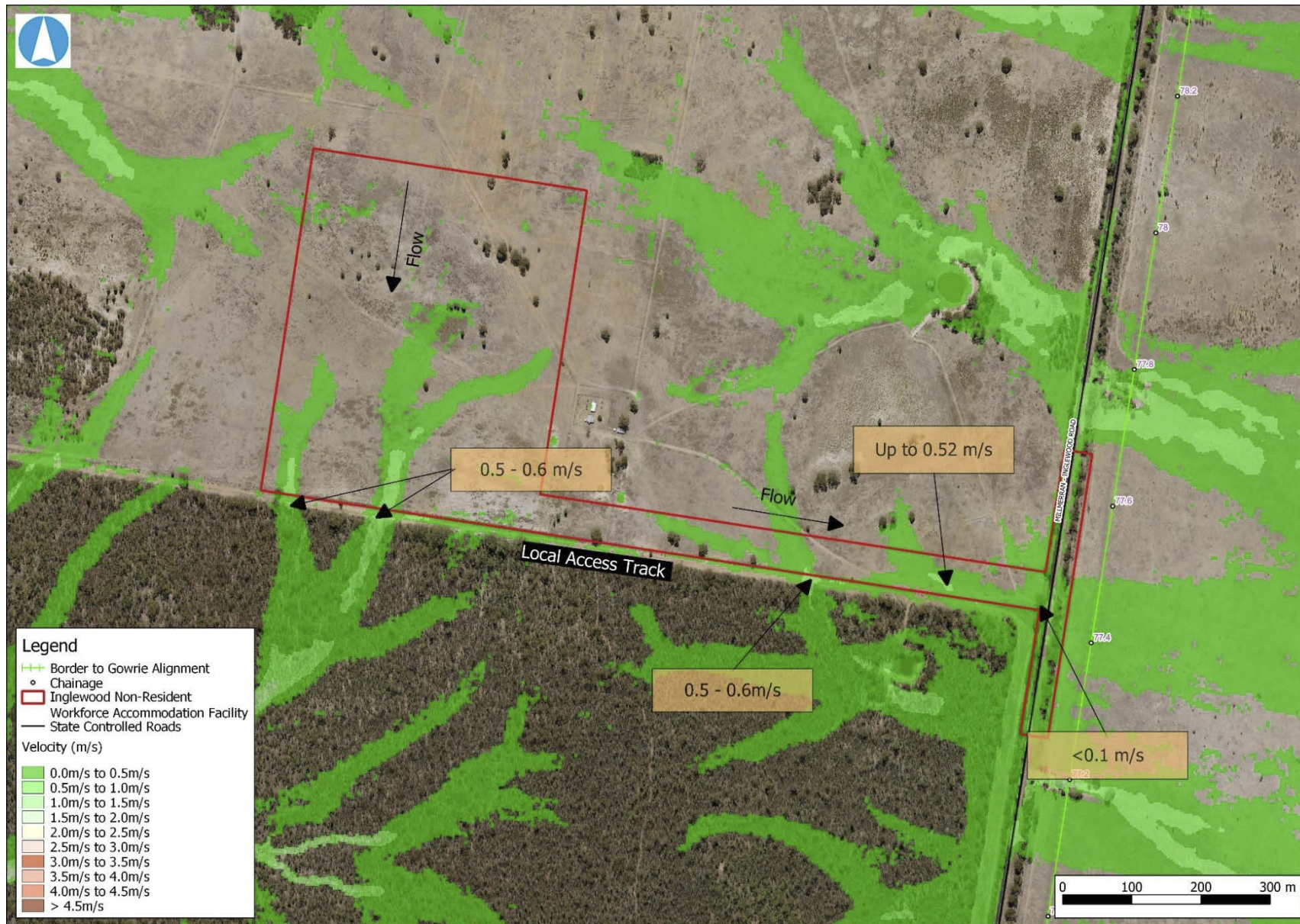


Figure 20.19 Inglewood site -1% AEP peak flood flow velocities

Flood hazard

Figure 20.20 presents the predicted peak overland flow velocity $\times$ depth product for the 1% AEP event. The velocity $\times$ depth product represents the combined dynamics of flood depth and velocity, and is applied to quantify and classify flood hazard in a risk management setting (AIDR, 2017).

The velocity $\times$ depth product results show:

- The velocity $\times$ depth product values remain below $0.3 \text{ m}^2/\text{s}$ everywhere on the Inglewood site, which is considered generally safe for all patrons as per the AIDR (2017) guideline
- The location of maximum ponding (350 mm maximum depth) is associated with velocity $\times$ depth values below $0.1 \text{ m}^2/\text{s}$, which is considered safe to all patrons except for a minor increase in hazard to parked small vehicles.

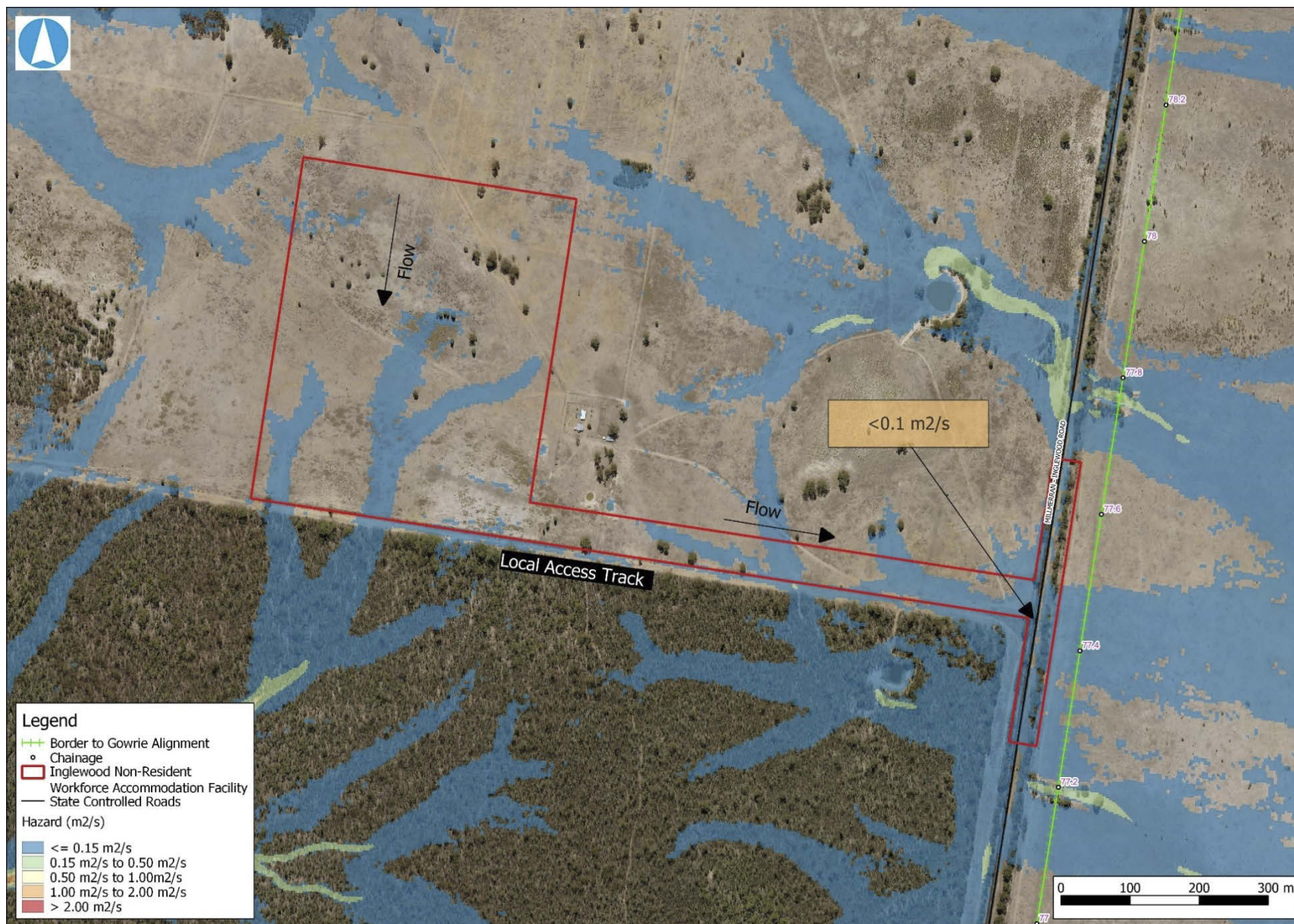


Figure 20.20 Inglewood site—1% AEP peak flow velocity $\times$ depth product

20.3.2.2 Conclusions and mitigation measures

The main outcomes of the Inglewood site flood risk assessment are summarised as follows:

- The Inglewood site is subject to overland flow in the 1% AEP event under existing conditions. It is not exposed to flooding risk from any other source including creek or river flooding
- The overland flow conditions within the Inglewood site area are assessed as generally benign, with no obvious vulnerability or constraints identified for future patrons or proposed site use
- Small areas with ponded depth up to 500 mm were identified within the Inglewood site, which could pose minor risk to small vehicles. These are not considered major constraints as they are isolated along the site perimeter and pose minimal risk to other site patrons or structures.

The mitigation measures are proposed for future use of the Inglewood site in consideration of the current flood risk assessment outcomes:

- Future development of the site should maintain the existing flow regimes within and around the site
- An onsite storm water management plan is recommended to effectively manage the overland flow within the site and discharge to the receiving environment under developed conditions
- A Flood Risk Management Plan will be developed for the site to ensure risks to people/workers and the environment are managed through the construction and operation of the facility
- Any proposed floor levels should provide appropriate freeboards above the peak flood levels determined in the current flood risk assessment
- Planning of small vehicle parking areas should avoid areas with greater than 300 mm flood depth identified in the current flood risk assessment.

Cross-drainage may be required under the Millmerran-Inglewood Road connection to accommodate any future increase in runoff.

20.4 Turallin facility

A site facility is proposed at Turallin near Millmerran (the Turallin facility), which may have a number of potential uses:

- Native plants nursery and traditional land management training
- Training and material testing facility
- General construction laydown and storage.

Further information regarding the Turallin facility is provided in EIS Chapter 5: Project Description.

20.4.1 Site location

The Turallin facility site is proposed to be located approximately 8 km northwest of Millmerran with access from Turallin Road. Clearing works to construct the Turallin facility will require an approximate total footprint area of 200,000 m².

The site area is bound between the Pine Creek main channel and one of its western tributaries. Pine Creek is a tributary of Condamine River. The Turallin site area is located approximately 10 km upstream of the Condamine River confluence and not affected by regional flooding.

The flooding risks within and around the Turallin facility are governed by breakout characteristics from the Pine Creek main channel and one of its western tributaries. The Pine Creek main channel directly crosses the proposed eastern access for the site to Turallin Road, with overbank flow meandering through a portion of the eastern site area. The western site area also overlaps with a breakout flow path from one of Pine Creeks tributaries, which becomes engaged in moderate-to-rare flood events.

The Turallin facility area is not affected by overland flow based on available information on the Flood Risk Information Portal (FRIP) provided by TRC.

A locality plan of the proposed site is shown in Figure 20.21.

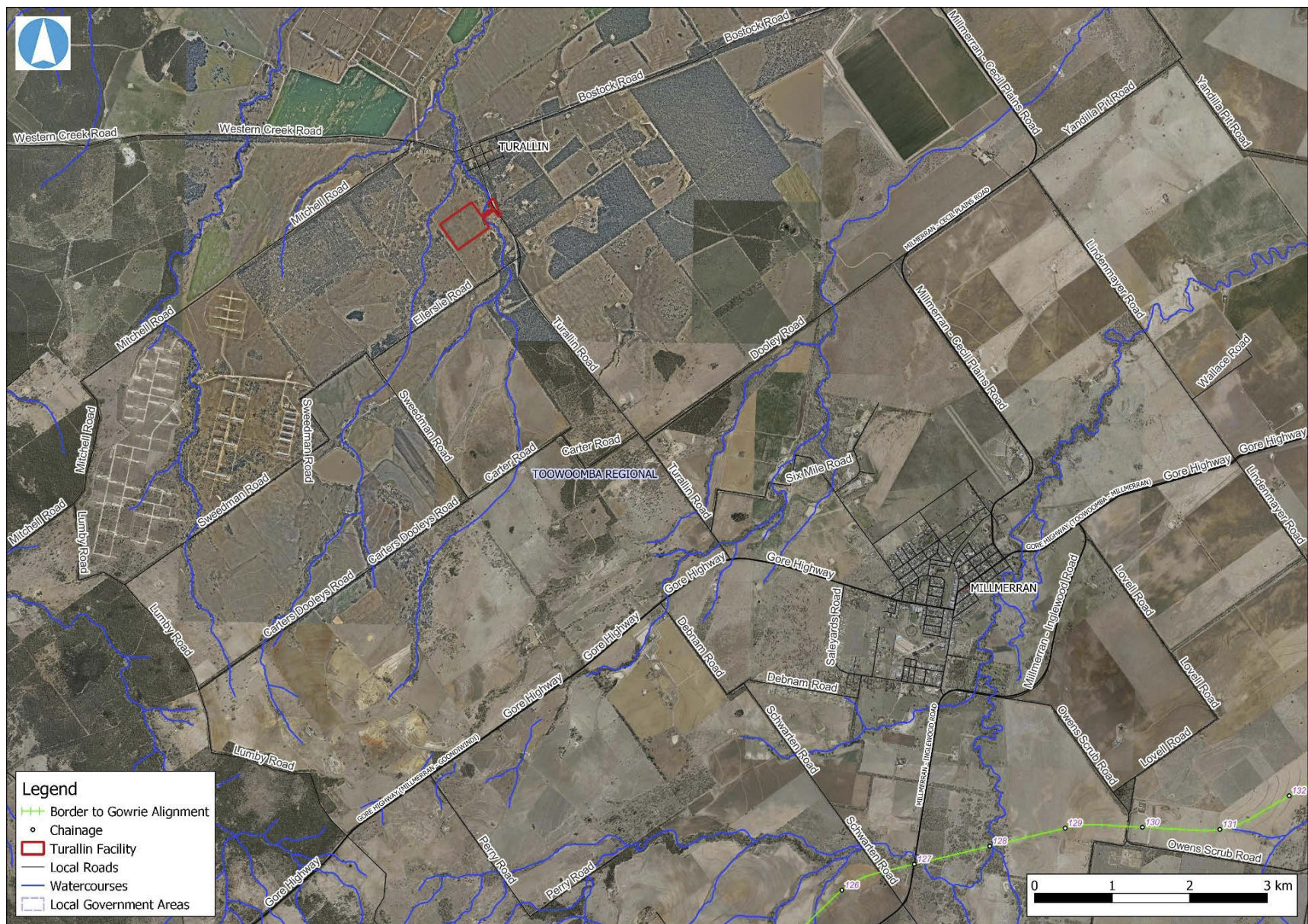


Figure 20.21 Turallin facility—Locality plan

The indicative flood risk area within and around the Turallin facility as extracted from TRC Flood Risk Information Portal is shown in Figure 20.22.

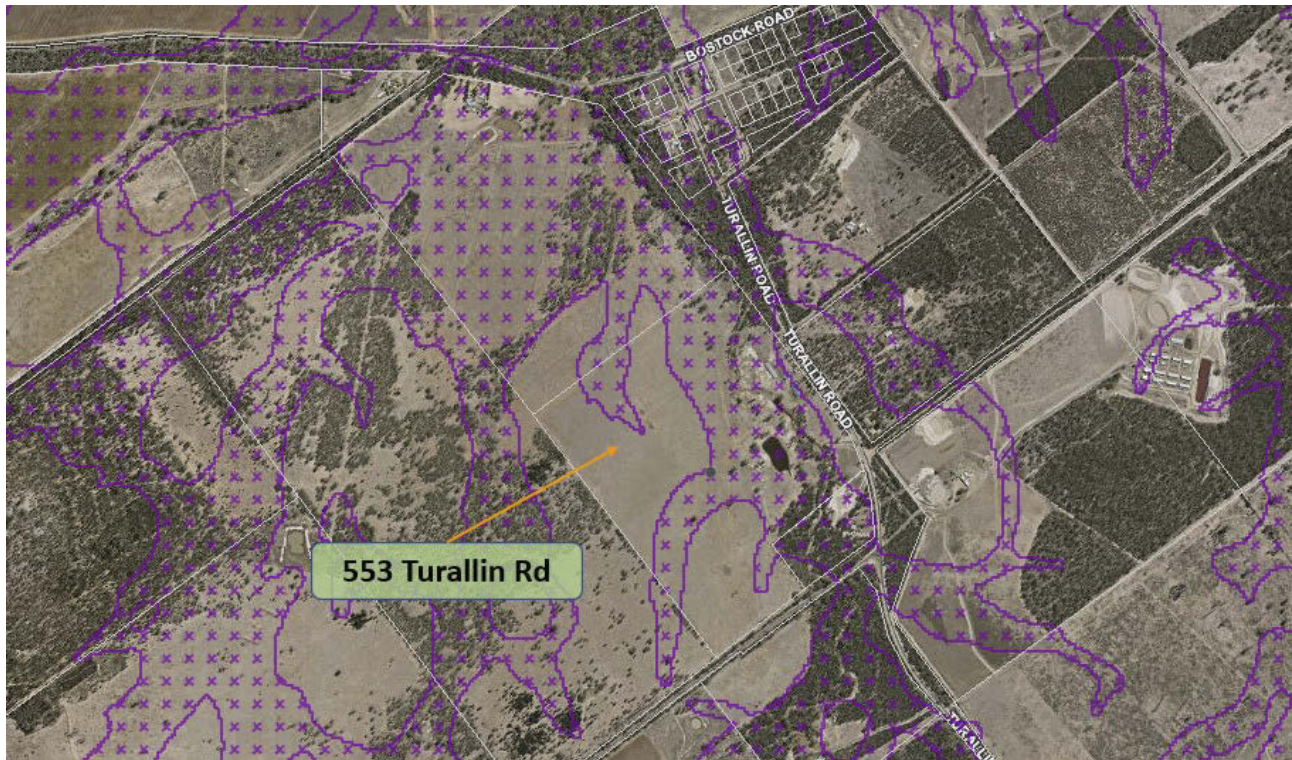


Figure 20.22 Flood risk area within and around the proposed site (TRC Flood Risk Information Portal)

20.4.2 Hydrology

A rainfall-runoff model was developed using the XP-RAFTS software (version 2018.1.3) for the Pine Creek main channel and its western tributaries. The hydrology model covers a total area of 20 km², with approximately half of which drain directly through the proposed Turallin site.

The 1% AEP event only was considered in this flood risk assessment.

The hydrologic model setup comprised the following steps:

- Catchment delineation for Pine Creek and its tributaries was undertaken based on ARTC supplied ALS data. The catchment delineation terminated at a confluence approximately 1 km downstream of Main Street, beyond which Pine Creek continues as a unified single channel. The adopted catchment plan is shown in Figure 20.23.

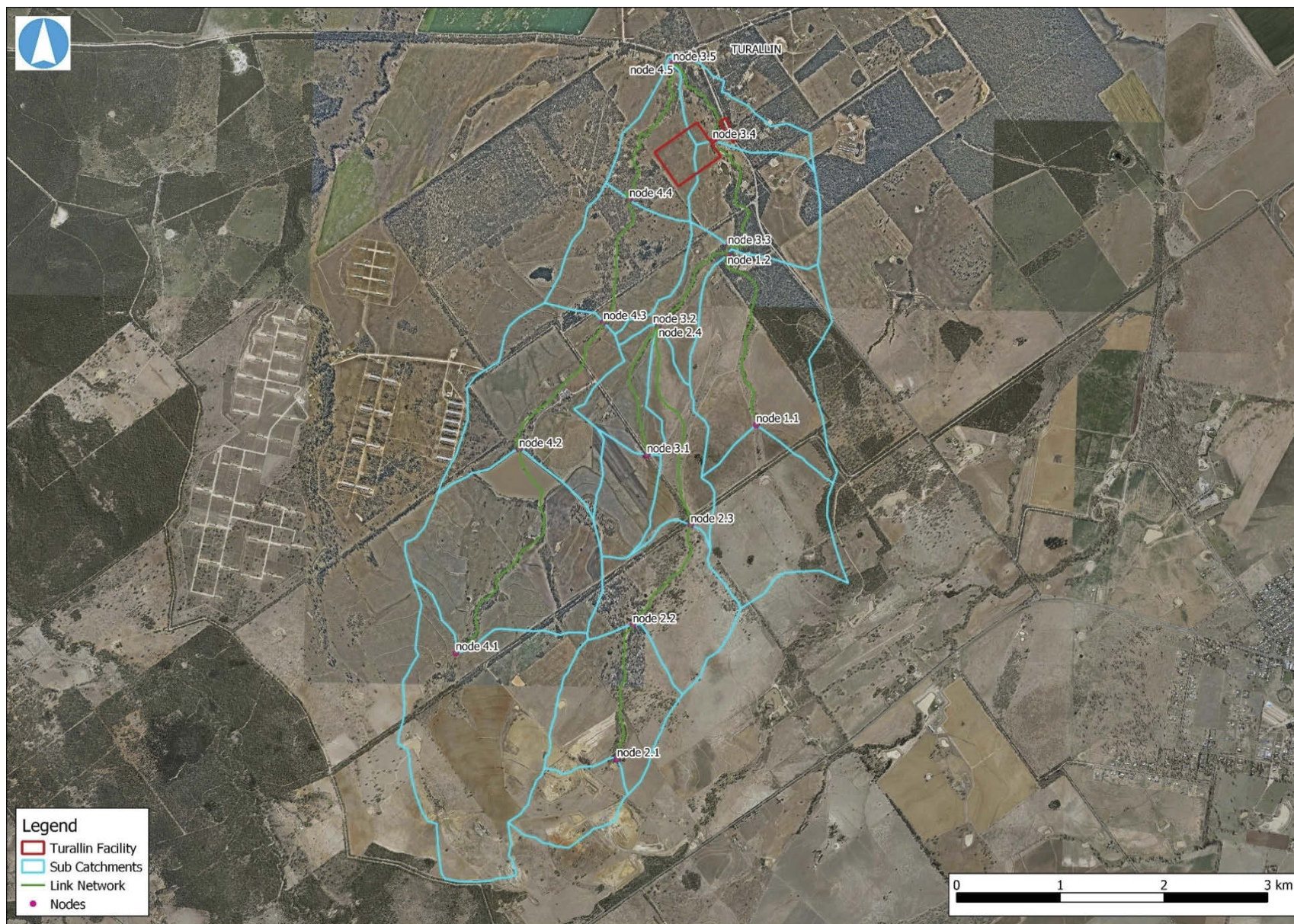


Figure 20.23 Turallin facility—Catchment plan

The topographic characteristics (area, imperviousness and roughness) of each sub-catchment were computed based on the catchment delineation and inspection of Google Earth aerial imagery. The roughness values were broadly classified into a 'low', 'moderate' or 'high' category, noting that these are fundamentally associated with the catchment storage. Uncertainties in assignment of roughness (i.e. catchment storage) were accounted for during the model calibration as discussed further below.

- The length, equal-area-slope and lag time were computed for every link connecting a pair of sub-catchment outlets. The lag times were calculated from kinematic wave celerity based on stream velocities estimated as a function of the average catchment slope. Uncertainties in lag times were accounted for during the model calibration as discussed further below.
- The storm initial loss (IL = 0 mm) and continuing loss (CL = 0.4 mm/hr) parameters were adopted from the calibrated regional Condamine River model as part of the draft EIS. These values were deemed conservative and were applied uniformly within the hydrology model.
- The 1% AEP design IFDs at the catchment centroid were extracted from the BoM design rainfall database. These were applied in conjunction with point TPs extracted from the ARR datahub. The adopted 1% AEP IFDs are summarised in Table 20.4.

Table 20.4 Adopted 1% AEP IFD depths

Duration	Depth (mm)
30 min	62.6
1 hour	80.5
1.5 hour	90.3
3 hour	106
4.5 hour	114
6 hour	121
9 hour	131

- The 1% AEP ARF was computed based on the total catchment size reporting to the Turallin facility location (10 km²).

The hydrologic assessment was undertaken in accordance with the ensemble method described in ARR 2019 guidelines. All standard storm durations between 30 minutes and 180 minutes were assessed, with 10 TPs considered for each storm duration, where:

- Within each TP ensemble: the critical TP was selected as that which produced the Rank 6 peak flow at the Turallin site facility
- Within the storm duration ensemble: the critical storm duration was selected as that which produced the maximum peak flow at the Turallin site among the critical TPs.

The 90-minute storm was assessed to be critical at the Turallin site.

Initial trials indicated that the hydrology model produced much lower peak flows compared to typical regional estimates. Noting that the Pine Creek catchment is ungauged, the hydrologic model was calibrated to match the QRT peak flow. The calibration resulted in:

- An adjusted global storage factor of 0.3 (i.e. 70 per cent reduction in global catchment storage)
- A model peak flow within 1 per cent of the QRT estimate at the site area (model: 84 m³/s; QRT: 83.1 m³/s).

The calibrated XP-RAFTS model was considered fit-for-purpose and was used to generate design flow inputs for the hydraulic assessment.

20.4.3 Hydraulics

A 2D hydraulic model was developed using the TUFLOW software (version 2020-10-AB) to simulate the flooding characteristics within and around the site area. The hydraulic model schematisation is shown in Figure 20.24.

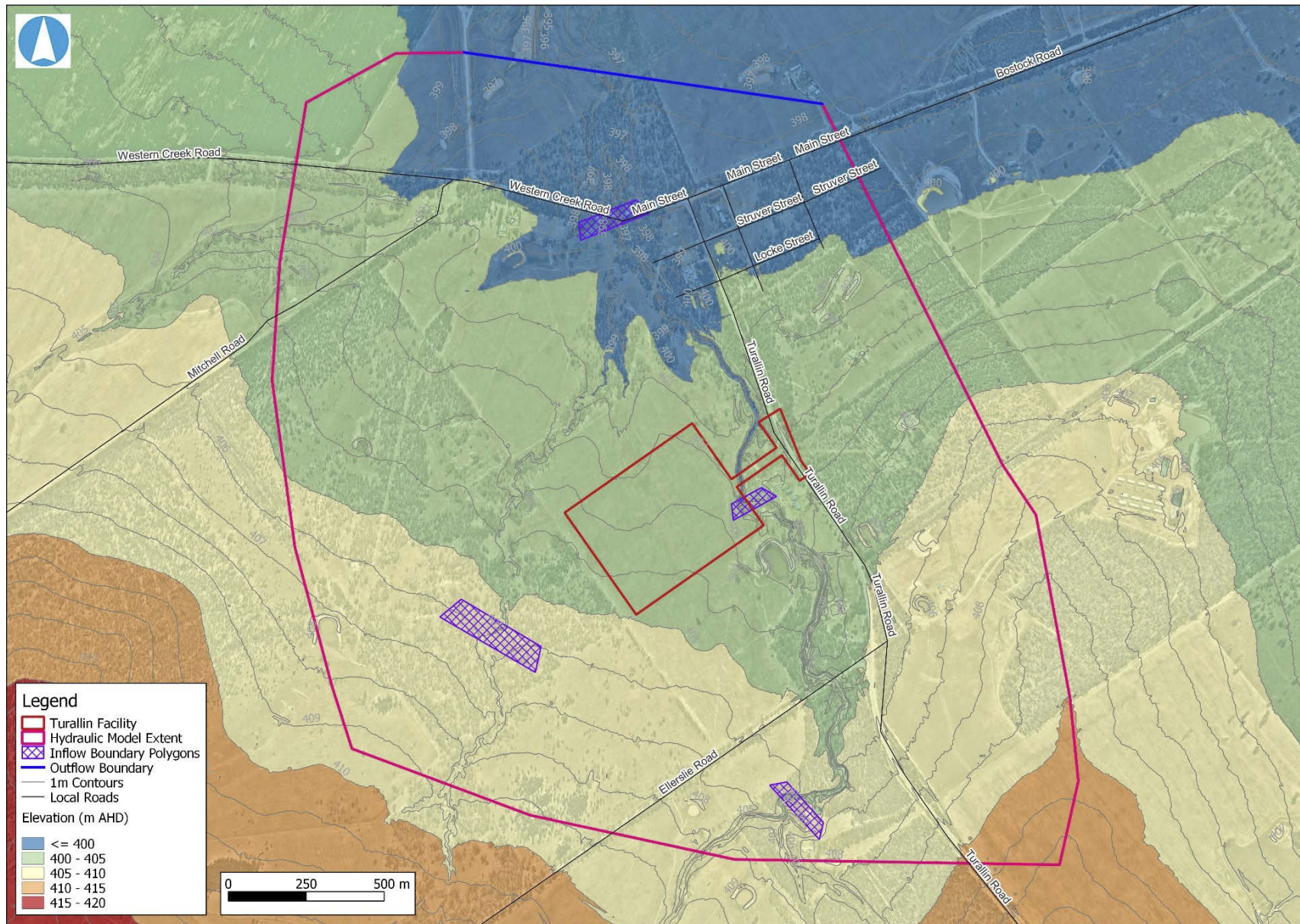


Figure 20.24 Turallin site –Hydraulic model schematisation

The model was implemented as follows:

- The model terrain was defined using ALS data provided by ARTC. The Turallin site is surrounded by generally flat country with an approximate elevation range of 401–405 mAHD sloping in a northerly direction.
- The model domain was defined by a 5 m x 5 m rectilinear grid supplemented by a 1 m SGS resolution to characterise small-scale topographic variations. This configuration was selected to achieve an optimal balance between modelling resolution and computational efficiency.
- Inflow boundary placements within the hydraulic model were consistent with the catchment plan shown in Figure 20.24. Total and local inflows from the hydrologic model were applied depending on their locations relative to the hydraulic model boundary.
- A constant TW level of 397 mAHD was applied at the downstream model boundary, located at the Pine Creek confluence, approximately 500 m downstream (north) of the Main Street crossing.
- The TW level was estimated by superimposing the flood risk area published on Flood Risk Information Portal with the model topography. Pilot model runs indicate that the combined Pine Creek waterway and overbank corridor was expected to be narrower than the published Flood Risk Information Portal flood risk area; as such the selected TW level is considered conservative.
- As the Turallin site is located a further 1 km upstream (south) of the Main Street crossing, the flooding characteristics at the site are not expected to be sensitive-to-moderate fluctuations in TW level.
- Surface roughness values in the hydraulic model were assigned based on land-use types determined from Google Earth aerial imagery. Table 20.5 summarises the adopted Manning's 'n' values informed by those adopted in the draft EIS. The land use classification is shown in Figure 20.24.
- Breaklines were enforced for key local roads surrounding the Turallin facility.
- The Pine Creek crossings at Ellerslie Road and Main Street were maintained at existing ground levels since structural details were not available. As both existing crossings have less than a 1% AEP flood immunity, the flood risk assessment outcomes are not expected to be sensitive to model representation of structural details at these locations.

Subsequent assessment of the 1% AEP critical storm (90 min) in the hydraulic model produced a peak flow within 5 per cent of the hydrologic model estimate at the site area (TUFLOW: 88 m³/s; XP-RAFTS: 84 m³/s), which was considered a very good joint validation. The hydraulic model was therefore considered fit-for-purpose and required no further adjustment.

Table 20.5 Adopted Manning's 'n' values

Description	Manning's n
Pasture	0.055
Roads	0.022
Buildings	0.10
Open water	0.025
Waterway	0.035
Cropping	0.05
Low vegetation	0.045
Medium vegetation	0.055
High vegetation	0.065
Riparian	0.04

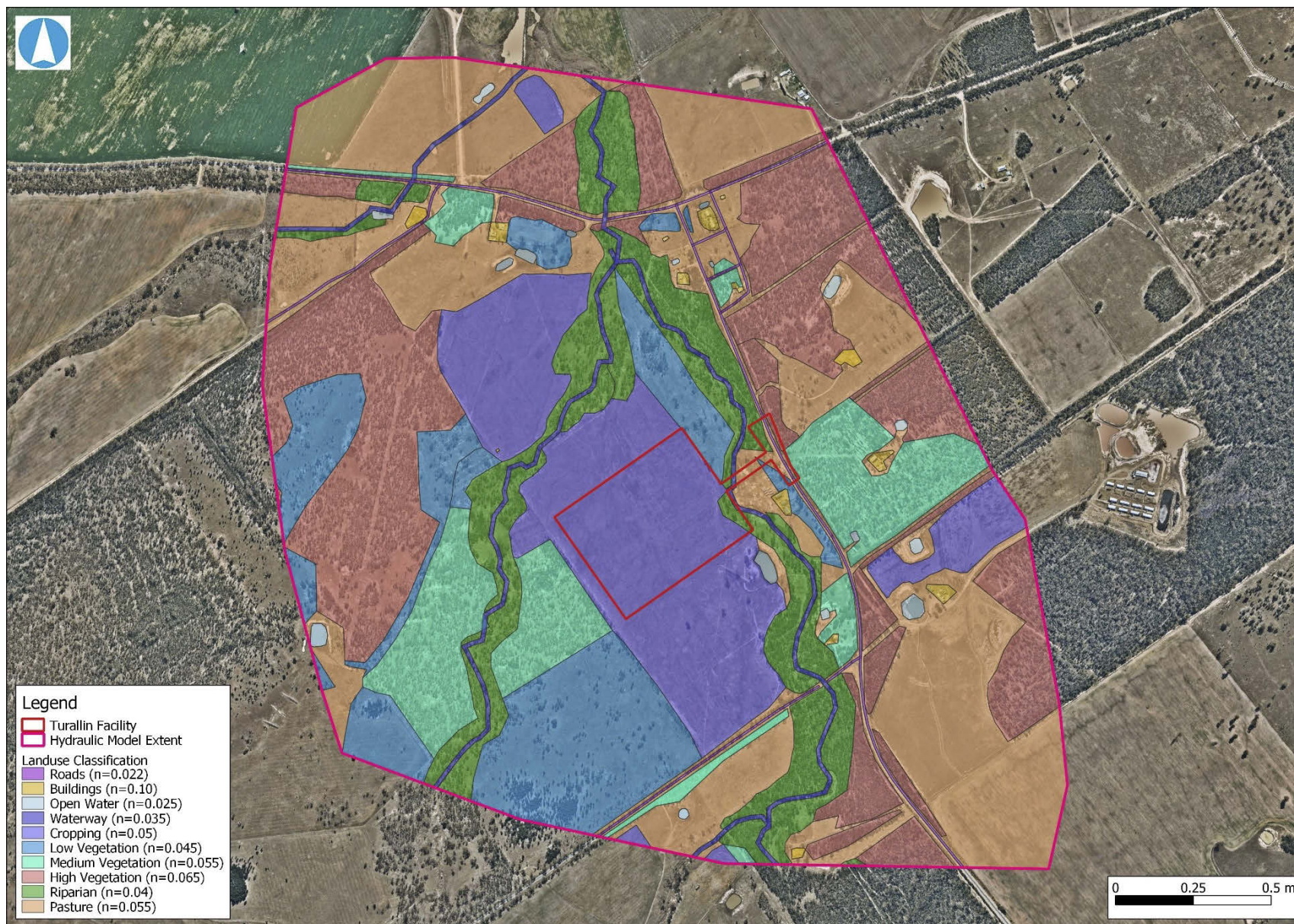


Figure 20.25 Turallin site—Land-use classification

20.4.4 Results

The hydraulic model was used to assess the existing flooding characteristics within and around the Turallin facility for the 1% AEP event. The site area is not affected by overland flow or regional flooding.

The simulated flow depth, flow velocity and velocity-depth product results within and around the Turallin site are discussed in the subsections below.

20.4.4.1 Flood depth

Figure 20.26 presents the predicted peak flood depth within and around the Turallin site area for the 1% AEP event.

The flood depth results show:

- The central portion of the site area is flood-free in the 1% AEP event
- The western portion of the site is subject to shallow inundation below 250 mm by a breakout flow path from a tributary of Pine Creek
- Subject to a mild flow velocity, a flow depth below 300 mm is considered generally safe for all patrons according to the AIDR (2017) classification
- The eastern portion of the site, including the proposed connection to Turallin Road, are traversed by the Pine Creek main channel and therefore subject to deep inundation over 2 m. These areas are considered unsafe for vehicles and people with some potential for structural damage according to AIDR (2017).

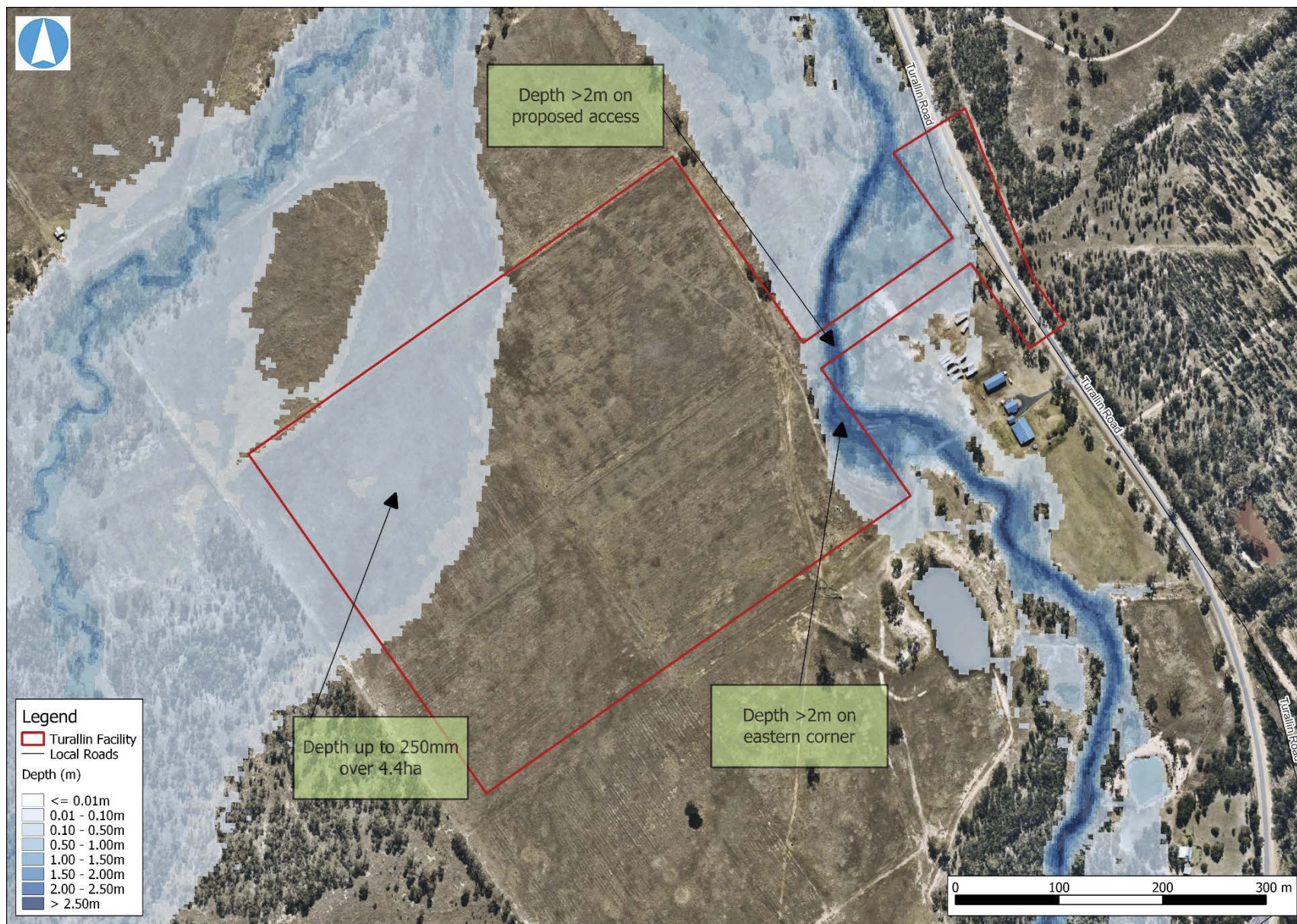


Figure 20.26 Turallin facility—1% AEP peak flood depth

20.4.4.2 Flow velocities

Figure 20.27 presents the predicted peak flood velocities within and around the Turallin facility area for the 1% AEP event.

The flood velocity results show:

- The central portion of the site area is flood-free in the 1% AEP event
- The western portion of the site is subject to generally mild velocities less than 0.5 m/s
- The eastern portion of the site is affected by peak velocities up to 2 m/s on the Pine Creek main channel. Due to the model approximation of the channel cross section (based on available LiDAR), actual channel velocities are likely to exceed those predicted in the current model.
- The eastern corner of the site is inundated by slow-moving breakout flow on the western overbank of the Pine Creek main channel, with a typical velocity less than 0.5 m/s.

The AIDR (2017) guideline considers a velocity below 2.0 m/s generally safe for all patrons subject to shallow flood depths across the site.

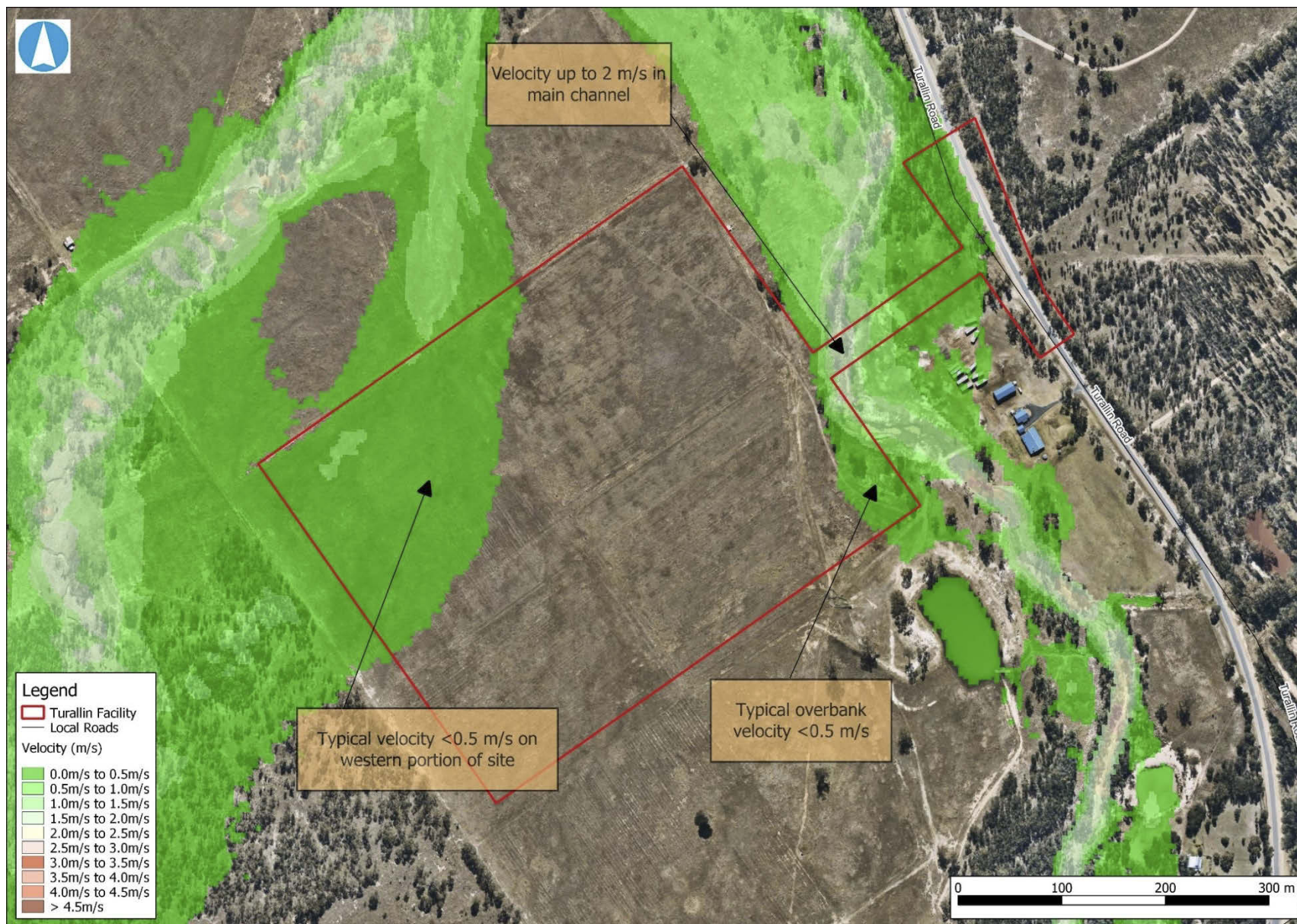


Figure 20.27 Turallin facility—1% AEP peak flood flow velocities

20.4.4.3 Flood hazard

Figure 20.28 presents the predicted peak velocity × depth product results for the 1% AEP event. The velocity × depth product represents the combined dynamics of flow depth and velocity, and is applied to quantify and classify flood hazard in a risk management setting (AIDR, 2017).

The velocity × depth product results show:

- The central portion of the Turallin facility area is flood-free in the 1% AEP event
- The western portion of the site is affected by mild velocity × depth product values below 0.15 m²/s associated with inundation by shallow, slow-moving flood water. This area is considered generally safe for all patrons as per the AIDR (2017) guideline.
- The eastern portion of the site traversed by the Pine Creek main channel is affected by significant velocity × depth product values above 2 m²/s. These areas are considered hazardous to all people and vehicles and some building types (AIDR, 2017).
- The eastern corner of the site on the western overbank area of Pine Creek experiences low velocity × depth product values typically below 0.15 m²/s. As a portion of the area is also prone to flood depth greater than 500 mm, the flood risk may be increased for small vehicles, children, or the elderly.

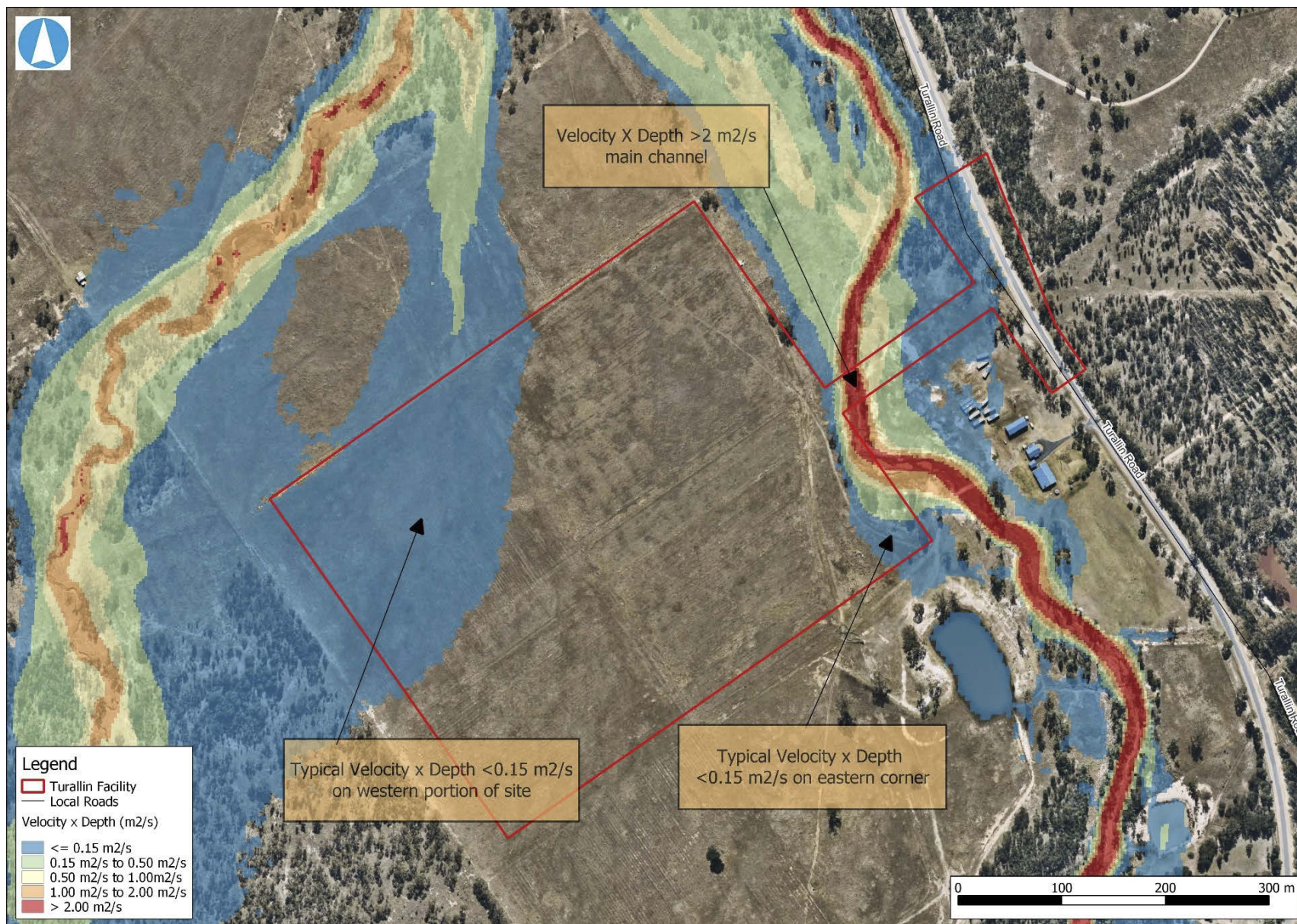


Figure 20.28 Turallin facility—1% AEP peak flood velocity × flood depth

20.4.5 Conclusion and mitigation measures

The main outcomes of the Turallin facility flood risk assessment are summarised as follows:

- The site is subject to Pine Creek flooding in the 1% AEP event under existing conditions. It is not exposed to flooding risk from overland flow or regional flooding sources.
- The central portion of the site is flood-free in the 1% AEP event and therefore imposes no constraints on future site activities.
- The eastern site area traversed by the Pine Creek main channel corridor is considered highly hazardous, which should be avoided by all non-essential development activities.
- The eastern corner of the site is subject to relatively deep inundation and may pose increased flood risk to small vehicles, children, or the elderly.

In order to inform the next stage of design and to take into consideration the current flood risk conditions, the following mitigation measures are proposed to minimise impact to the flooding regime of the Turallin facility:

- Future connection to Turallin Road should ensure adequate provisions of conveyance and erosion protection on the Pine Creek main channel and avoid substantial development in its immediate overbank areas to minimise flood risk to site patrons and environmental impact to the watercourse.
- A future connection to Ellerslie Road (to the south of the site) may be considered as an alternative to the current proposed access to Turallin Road. The alternative provides an opportunity to minimise development impact on the watercourse. It is noted, however, that the benefit of the alternative access might be offset by Ellerslie Road having a lesser flood immunity than Turallin Road.
- Future developments on the western portion of the site should minimise impact on the Pine Creek tributary. Despite the generally low existing flood risk, development in this area may require significant earthworks or bunding along a substantial length of the site perimeter. The site boundary may be reconfigured so that developments are restricted to flood-free land if practical.
- Any future development needs to consider impacts on nearby FSRs including the residential property present on the eastern bank of Pine Creek, adjacent the site access and Turallin Road connection.
- Any proposed floor levels in flood-prone areas should provide appropriate freeboards above the peak flood levels determined in the current flood risk assessment.
- An on-site storm water management plan is recommended to effectively manage the overland flow within the site and discharge into its receiving environment under developed conditions.
- A Flood Risk Management Plan will be developed for the site to ensure risks to people/workers and the environment are managed through the construction and operation of the facility.

20.5 Laydown areas and other temporary Project facilities

20.5.1 Laydown areas

A total of 77 construction laydown areas (excluding the Turallin facility) are proposed along the Project alignment as described in EIS Chapter 5: Project Description. A review of the Existing Case flood modelling results identifies which locations are at risk of being impacted by flooding as shown in Table 20.6.

Table 20.6 Laydown areas flooding risk

ID	Location	Chainage (km)	Impacted by 1% AEP regional flooding	Commentary
NS2B–LDN031.0	Kildonan Road	31.0	Yes	Impacted in the 5% AEP event
NS2B–LDN032.5	Kildonan Road	32.5	Yes	Impacted in the 5% AEP event
NS2B–LDN033.2	Kildonan Road	33.2	No	Not impacted up to and including the 1% AEP event
NS2B–LDN035.6	Eukabilla Road	35.6	No	Not impacted up to and including the 1% AEP event
B2G–LDN000.9	Georges Ln (extension)	0.4	No	Not impacted up to and including the 1% AEP event
B2G–LDN006.3	Yelarbon–Kurumbul Road	6.2	No	Impacted by local drainage flooding in the 20% AEP event
B2G–LDN016.0	Yelarbon–Kurumbul Road	16.0	No	Not impacted up to and including the 1% AEP event
B2G–LDN020.3	Yelarbon–Kurumbul Road	20.4	No	Impacted by local drainage flooding in the 5% AEP event
B2G–LDN025.9	Yelarbon (south)	25.8	Yes	Impacted by the 20% AEP event regional flooding
B2G–LDN025.9	Yelarbon (north)	25.9	Yes	Impacted by the 20% AEP event regional flooding
B2G–LDN030.0	Suttons Road	30.0	No	Not impacted up to and including the 1% AEP event
B2G–LDN037.6	Springborg Road	37.6	No	Not impacted up to and including the 1% AEP event
B2G–LDN049.8	McDougalls Crossing Road	49.70	No	Not impacted up to and including the 1% AEP event
B2G–LDN052.8	Cremascos Road	52.8	Yes	Minor impact in the 20% AEP event
B2G–LDN054.2	Cremascos Road	54.3	No	Impacted by local drainage flooding in the 20% AEP event
B2G–LDN055.4	Cremascos Road	55.4	Yes	Impacted by the 20% AEP event
B2G–LDN060.4	Bybera Road	60.5	No	Impacted by local drainage flooding in the 20% AEP event
B2G–LDN065.8	Lovells Crossing Road	65.8	No	Not impacted up to and including the 1% AEP event
B2G–LDN067.6	Thornton Road	67.6	Yes	Impacted in the 5% AEP event
B2G–LDN069.0	Thornton Road	68.8	Yes	Impacted in the 20% AEP event
B2G–LDN073.0	Millmerran–Inglewood Road	72.9	No	Not impacted up to and including the 1% AEP event
B2G–LDN074.0	Millmerran–Inglewood Road	74.2	No	Impacted by local drainage flooding in the 20% AEP event
B2G–LDN081.0	Wongavale Yugibar Road	81.1	No	Impacted by local drainage flooding in the 20% AEP event
B2G–LDN088.0	Millmerran–Inglewood Road	88.1	Yes	Impacted in the 20% AEP event

ID	Location	Chainage (km)	Impacted by 1% AEP regional flooding	Commentary
B2G-LDN091.8	Site track off Millmerran–Inglewood Road	91.9	No	Impacted by local drainage flooding in the 20% AEP event
B2G-LDN093.8	Millmerran–Inglewood Road	93.8	No	Not impacted up to and including the 1% AEP event
B2G-LDN098.0	Millmerran–Inglewood Road	97.9	Yes	Impacted in the 20% AEP event
B2G-LDN100.6	Millmerran–Inglewood Road	100.7	No	Not impacted up to and including the 1% AEP event
B2G-LDN104.5	Partons Road	104.6	Yes	Impacted in the 10% AEP event
B2G-LDN112.1	Millwood Road	112.2	No	Minor impacted by local drainage flooding in the 20% AEP event
B2G-LDN115.6	Millmerran–Inglewood Road/Heckendorf Road via LDN 116.00	115.7	No	Not impacted up to and including the 1% AEP event
B2G-LDN116.0	Heckendorf Road	116.2	No	Not impacted up to and including the 1% AEP event
B2G-LDN120.2	Blackwell Road	120.2	No	Minor impact by local drainage flooding in the 20% AEP event
B2G-LDN123.6	Scragg Road	123.7	No	Not impacted up to and including the 1% AEP event
B2G-LDN123.8	Scragg Road	123.8	No	Not impacted up to and including the 1% AEP event
B2G-LDN126.9	Schwartens Road	126.9	Yes	Impacted in the 20% AEP event
B2G-LDN128.2	Owens Scrub Road	128.4	Yes	Impacted in the 20% AEP event
B2G-LDN129.5	Foxwood Road	129.4	No	Not impacted up to and including the 1% AEP event
B2G-LDN130.0	Owens Scrub Road	130.0	No	Not impacted up to and including the 1% AEP event
B2G-LDN130.6	Owens Scrub Road	130.6	No	Not impacted up to and including the 1% AEP event
B2G-LDN130.4	Owens Scrub Road	130.7	No	Not impacted up to and including the 1% AEP event
B2G-LDN138.5	Hall Road	138.6	Yes	Impacted in the 20% AEP event
B2G-LDN139.0	Hall Road	139.2	Yes	Impacted in the 20% AEP event
B2G-LDN140.2	Millmerran Leyburn Road	141.6	Yes	Impacted in the 20% AEP event
B2G-LDN141.3	Millmerran Leyburn Road	142.6	Yes	Impacted in the 20% AEP event
B2G-LDN143.0	Gilgai Lane	144.4	Yes	Impacted in the 20% AEP event
B2G-LDN144.6	Gilgai Lane	146.0	Yes	Impacted in the 20% AEP event
B2G-LDN147.1	Fysh Road	148.4	No	Not impacted up to and including the 1% AEP event
B2G-LDN149.0	Gore Highway	150.1	Yes	Impacted in the 20% AEP event

ID	Location	Chainage (km)	Impacted by 1% AEP regional flooding	Commentary
B2G-LDN150.5	Dieckmann Road	152.1	No	Not impacted up to and including the 1% AEP event
B2G-LDN150.9	Ware Street	152.1	No	Minor impact by local drainage flooding in the 20% AEP event
B2G-LDN152.9	Madelaine Street	153.3	No	Minor impact by local drainage flooding in the 20% AEP event
B2G-LDN153.1	Gore Highway	154.6	No	Minor impact by local drainage flooding in the 20% AEP event
B2G-LDN161.0	Yarralea Road	162.3	No	Minor impact by local drainage flooding in the 20% AEP event
B2G-LDN163.3	Roche Road	164.7	No	Not impacted up to and including the 1% AEP event
B2G-LDN164.3	Murlaggan Road	165.6	No	Not impacted up to and including the 1% AEP event
B2G-LDN165.6	Kahler Road	166.9	No	Not impacted up to and including the 1% AEP event
B2G-LDN169.6	Gore Highway	171.0	No	Impacted by local drainage flooding in the 20% AEP event
B2G-LDN171.0	Oakey Pittsworth Road	172.3	No	Impacted by local drainage flooding in the 20% AEP event
B2G-LDN172.0	Lochaber Road	173.3	No	Minor impact by local drainage flooding in the 20% AEP event
B2G-LDN172.6	Lochaber Road	173.8	No	Impacted by local drainage flooding in the 20% AEP event
B2G-LDN173.5	Paint Mine Road	174.8	No	Not impacted up to and including the 1% AEP event
B2G-LDN175.5	Linthorpe Road	176.9	No	Impacted by local drainage flooding in the 20% AEP event
B2G-LDN179.0	Geitz Road via Linthorpe Valley Road	180.2	No	Not impacted up to and including the 1% AEP event
B2G-LDN183.0	Bushy Lane	184.3	No	Not impacted up to and including the 1% AEP event
B2G-LDN183.8	Biddeston Southbrook Road	185.1	No	Not impacted up to and including the 1% AEP event
B2G-LDN185.0	Gore Highway	186.5	No	Impacted by local drainage flooding in the 20% AEP event
B2G-LDN188.2	Athol School Road	189.5	No	Not impacted up to and including the 1% AEP event
B2G-LDN192.3	via Athol Road	193.6	No	Not impacted up to and including the 1% AEP event
B2G-LDN196.2	Toowoomba-Cecil Plains Road via existing side road	197.4	Yes	Impacted in the 20% AEP event
B2G-LDN197.0	Toowoomba-Cecil Plains Road	198.3	No	Not impacted up to and including the 1% AEP event
B2G-LDN198.1	Brimblecombe Road	199.4	Yes	Impacted in the 1% AEP event
B2G-LDN198.7	Brimblecombe Road	200.0	Yes	Impacted in the 20% AEP event
B2G-LDN203.0	Warrego Highway	204.3	Yes	Impacted in the 20% AEP event

ID	Location	Chainage (km)	Impacted by 1% AEP regional flooding	Commentary
B2G-LDN204.2	Chamberlain Road	205.5	No	Not impacted up to and including the 1% AEP event
B2G-LDN206.3	Leesons Road	207.6	No	Not impacted up to and including the 1% AEP event
B2G-LDN206.9	Draper Road	208.2	Yes	Impacted in the 20% AEP event

20.5.1.1 Conclusions

The assessment completed above identifies that 44 of the 77 laydown areas are impacted by flooding up to the 1% AEP event. It is recommended that during the detailed design stage, a high-level flood risk assessment be completed on these laydown areas that are impacted utilising Existing Case modelling results or freely available information from government sources.

20.5.2 Temporary wastewater treatment plants and chemical storage

During construction of the Project, areas will be required to be allocated for the storage of hazardous chemicals as well as the need for the installation of temporary Wastewater Treatment Plants (WWTPs) at nominated locations. Each of these installations pose a potential risk to the environment if impacted during flood events, through a potential accidental release of contaminants. In order to minimise environmental impact from The Project, it is necessary to ensure these installations are managed and controlled to prevent releases.

20.5.2.1 Temporary wastewater treatment plants

It is proposed that temporary WWTPs will be located at each of the non-resident workforce accommodation facilities. It is likely that packaged WWTPs will be procured, supplied and operated by the contractor during the construction works stage. Should this be the case, the contractor will be guided by the supplier on appropriate siting and control measures to manage the WWTP during a flood event. At a minimum, the following controls are proposed to manage WWTPs before and during a flood event:

- Siting of WWTPs will be outside overland flow paths, drainage lines and 1% AEP flood extents where possible
- The WWTP will be specified to include materials that will minimise infiltration /inflow into the sewerage system (i.e. Smart Sewers)
- Electrical components, controls and circuitry will be installed in water-resistant cabinets and located above the 1% AEP peak water level
- All tanks will be specified to have a freeboard of at least 500 mm and the top of the tanks will be located above the 1% AEP flood level
- The WWTPs will be specified to have a capacity to treat wet-weather flows
- Should a flood warning being issued by BoM or the LDMG, WWTP equipment will be anchored to prevent movement should they be impacted by flood waters
- All chemical storages will be located within a bund and will be located/sited above the 1% AEP flood level
- Should the WWTP be required to be operational during a flood event, the contractor will liaise with the supplier on the minimum operational requirements and a risk assessment completed to identify appropriate mitigation measures beyond those listed above
- Post flood events, where required, the Emergency Response Plan for the WWTP will need to specify the minimum equipment required (i.e. tankers, to service WWTPs and any temporary Project facilities until full operation of the WWTP is returned).

20.5.2.2 Other hazardous areas

Other hazardous areas such as chemical storages will be utilised intermittently across the Project during construction. It is not envisaged that significant quantities of chemicals or dangerous goods will be required for construction but, where storages are required, these will be designed in accordance with appropriate Australian Standards and Dangerous Goods Codes. The following controls are proposed for these storages before and during a flood event:

- Siting of chemical storages will be outside of overland flow paths, drainage lines and 1% AEP flood extents where possible
- Due to the expanse of The Project, some temporary storages may be located within the 1% AEP flood extents. Should this be the case, storage quantities will be minimised and supplemented from larger storages outside flood-prone areas.
- Should a flood warning be issued by BoM or the LDMG, chemical storages will be locked, and smaller quantities of chemicals removed if possible and timing allows. Chemical storages will be anchored to the ground to minimise movement during a flood event.

20.6 Pre-construction, early works and construction works flood immunity

A Draft Outline EMP has been developed for the Project to:

- Provide an environmental management framework to enable the identified environmental and social outcomes to be achieved for the life of the Project, through detailed design, pre-construction and early work, construction works, and commissioning and operations stages
- To establish the process for the preparation and implementation of the CEMP and the Operation EMP.

The Draft Outline EMP is presented in EIS Chapter 24: Draft Outline Environmental Management Plan, and flooding and geomorphology is discussed in Section 24.9.7 of the document.

In the pre-construction, early works and construction works stages, mitigation measures determined as part of Flood Risk Assessments undertaken for construction works will be implemented. Where reasonable and practical, construction tasks will be scheduled to minimise bulk earthworks activities within the 1% AEP extent during periods of elevated flood risk. Where works cannot be scheduled outside of this time period, activity-specific flood readiness and response planning will be required. This planning will be developed in consultation with the relevant local government and Queensland Fire and Emergency Services.

Temporary Project facility (laydowns, etc.) flood immunity mitigations will include scheduling and site layout of construction sites and facilities to consider existing flowpaths (both in the regional and local context), and where possible, avoid land prone to flooding in more frequent events as well as overland flow paths. Site-specific control measures will be required during the pre-construction, early works and construction works stage to minimise impacts on the existing flooding regime and any overland flowpaths due to construction activities.

The following control measures are proposed:

- Maintain existing flowpaths and avoid direct impact on existing drainage lines
- Where possible, avoid areas affected by high flood flows and by inundation by flood flows
- Siting of facilities on the fringes of regional floodplains, where applicable
- Avoidance of raised construction pads in areas impacted by flooding
- Undertake modelling of more frequent events (50% AEP and more frequent) to determine the impacts associated with the temporary works

- As required, undertake secondary approvals processes for temporary works/facilities
- The potential site will be surveyed prior to site establishment to understand the exact extent of potential flooding impact to facilities and storage areas.

The earthworks and temporary drainage will be designed to minimise flooding impacts.

21 Embankment overtopping failure risk assessment

A collapse of the rail embankment during rare-to-extreme flood events has the potential to increase the degree of flood risk experienced by occupants at FSR locations relative to that incurred during no-failure flood conditions.

Overtopping flow is a key component of most potential failure modes resulting from floods (United States Department of the Interior Bureau of Reclamation and United States Army Corps of Engineers (US DIBR), 2019a). There are two main types of overtopping:

- Overtopping from a continuous flow when the flood elevation exceeds the lowest section of embankment
- Over wash, where the flood surface elevation approaches but does not exceed the low point in the embankment profile, and wind-driven waves run-up and overtop the embankment. Typically, a significant upstream surface area (fetch) is required to allow wind to develop waves that could overtop an embankment.

Most embankments are not likely to withstand sustained overtopping by 300 mm to 600 mm without a high probability of failure (United States Department of the Interior Bureau of Reclamation and United States Army Corps of Engineers, 2019a). The depth and duration of overtopping, and the erodibility and strength of the embankment materials to withstand heightened seepage forces are the key parameters to determine the likelihood of failure as a result of overtopping.

Figure 21.1 illustrates the typical progression of embankment overtopping failure for cohesive and cohesionless materials.

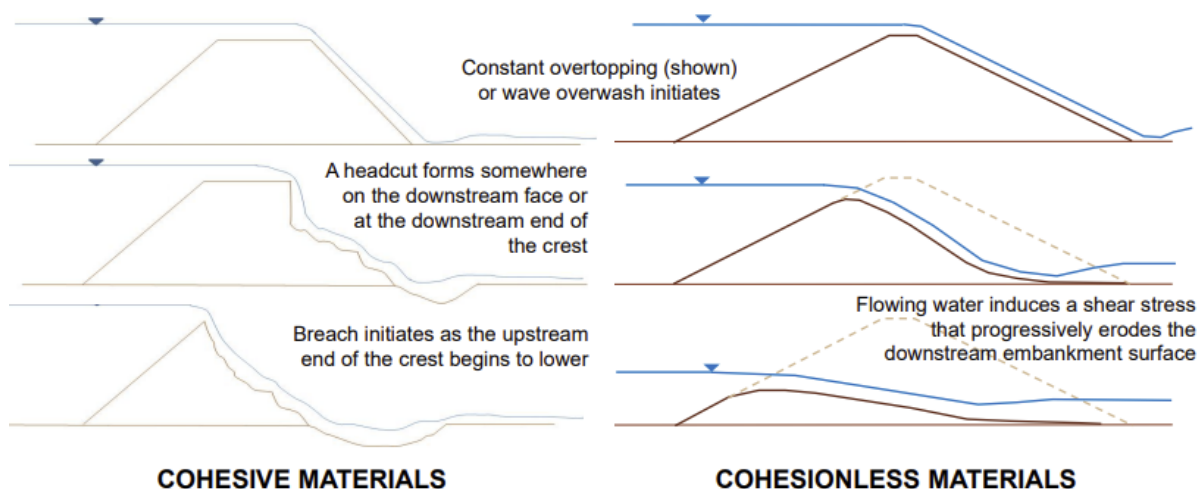


Figure 21.1 Overtopping failure progression for an embankment

Source: United States Department of the Interior Bureau of Reclamation and United States Army Corps of Engineers (2019b)

For this study, a risk assessment has been performed to identify key locations along the Project where an increased risk to life is present due to an overtopping failure of the embankment. The assessment was undertaken for the 1 in 2,000 AEP, 1 in 10,000 AEP and PMF design flood events using the methodology described in the following sections, and included a stability assessment of the embankment at key locations. A detailed Failure Impact Assessment (FIA) using hydraulic modelling has not been undertaken for the purposes of this revised draft EIS. It is proposed to undertake this FIA in detailed design, as referenced in Section 21.1.3.

21.1 Risk-to-life assessment

The potential risk to life due to overtopping flood failure of an embankment is related to:

- The location of FSRs, including places of permanent and temporary occupation (e.g. dwellings, commercial buildings, sheds etc.) relative to the potential breach location
- The number of occupants expected at each FSR
- The severity of flooding experienced under flood conditions prior to failure of the embankment
- The flood characteristics experienced at FSR locations resulting from embankment failure.

It is important to note that for an overtopping failure event, there may be extensive floodplain inundation prior to the failure occurring. In this instance, there could be occupants who have already been significantly affected by pre-failure flooding and consequently who could be considered to no longer remain in the flood zone. For example, people who have been subjected to significant flood depth and flood velocity, or people who are within an area covered by an emergency management plan that requires them to leave the area (Australian National Committee on Large Dam, 2012). This sentiment is also reflected in the *Queensland Guidelines for failure impact assessment of water dams* (Department of Natural Resources, Mines and Energy, 2018), where FSRs do not contribute to the number of persons at risk if they experience flood depths ≥ 300 mm over a building's plinth in the no-failure flood event.

21.1.1 Methodology

The methodology employed to assess the risk to life due to embankment overtopping included the following steps:

- Hydraulic modelling of 1 in 2,000 AEP, 1 in 10,000 AEP and PMF flood events for rail design conditions, based on the revised reference design
- Creation of a GIS database of places of occupation including assignment of default populations in accordance with Appendix 4 of Department of Natural Resources, Mines and Energy (2018)
- Establishment of criteria to identify sections of the levee alignment that are at risk of overtopping and the relative risk of failure at these locations based on depth of overtopping. Figure 21.2 provides a summary of the embankment overtopping risk categories adopted.
- For each AEP design flood event, a GIS analysis of model results against the overtopping criteria was undertaken to identify and map sections of the Project alignment that experience a low, low to medium, medium to high, and high risk of overtopping failure.
- Identification and mapping of receptors likely to be impacted by incremental flood inundation resulting from embankment failure. This included:
 - Identification of receptors located downstream of identified overtopping locations which experience flood depths of less than 300 mm in the no-failure scenario (i.e. FSRs that experienced depths > 300 mm in the no-failure scenario were not considered to be at an increased level of risk)
 - Consideration of FSRs located within a 50 m buffer of the no-failure flood extent where a failure may increase the existing (no failure) flood extent, resulting in additionally impacted FSRs.
- Additionally, slope stability analysis under flood events for critical sections was undertaken to assess the likelihood of a sudden embankment failure in worst-case scenario flood events (1 in 2,000 AEP, 1 in 10,000 AEP and PMF events). The assessment of slope stability is discussed in more detail in Section 21.1.1.1.

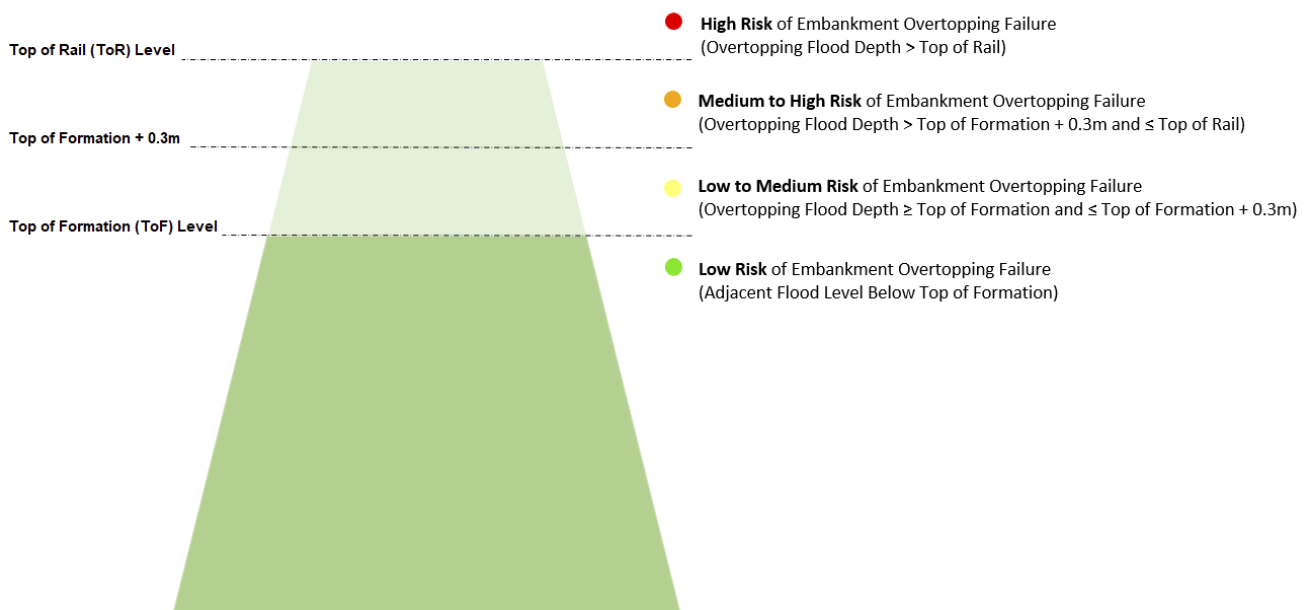


Figure 21.2 Embankment overtopping risk categories

21.1.1.1 Slope stability analysis

As modelling during the reference design stage was limited to performance under the 1% AEP event, further assessment of slope stability under flood conditions in excess of the 1% AEP event has been undertaken to assess the likelihood of a sudden embankment failure to occur in events greater than the 1% AEP event.

Design inputs

Slope stability analysis under worst-case scenario flood events, which included the 1 in 2,000 AEP, 1 in 10,000 AEP and PMF events for critical sections, has been undertaken based on the following inputs:

- The Project alignment is based on a maximum 1:80 grade design.
- Project extents considered for the purposes of the stability assessment comprise Ch 0 km to Ch 208.22 km of the Project alignment.
- Data outputs from the hydraulic modelling were organised per catchment and contained processed water level differentials across the two sides (upstream and downstream) of the embankment. The sections with the most significant difference in water levels on either side of the embankment are seen as the areas with potentially the greatest risk of slope instability due to destabilising forces leading to sudden failure.
- The embankment profile, including zoning, is assumed to be as proposed under the revised reference design and therefore comprises the following configuration:
 - Batter side slopes of 1V:3H throughout all flood zones
 - Low-permeability structural fill and capping. This will require an amendment to the standard material specification to provide structural fill with the same permeability requirements as that currently specified for capping where the water level is expected to reach the base of formation level:
 - Based on seepage modelling, water infiltration through the structural fill has been identified as a potential risk that may lead to increased saturation of the embankment fill.
 - Application of a sub-layered formation design to maintain the required bearing capacity under load in the event that these materials are inundated by flood water
 - Scour protection to be specified where critical velocities exceed 1 m/s

- The following material profile and as illustrated in Figure 21.3:
 - 200 mm capping layer
 - 600 mm sub-capping layer
 - 750 mm structural fill
 - Type A/B earth fill to base of embankment.

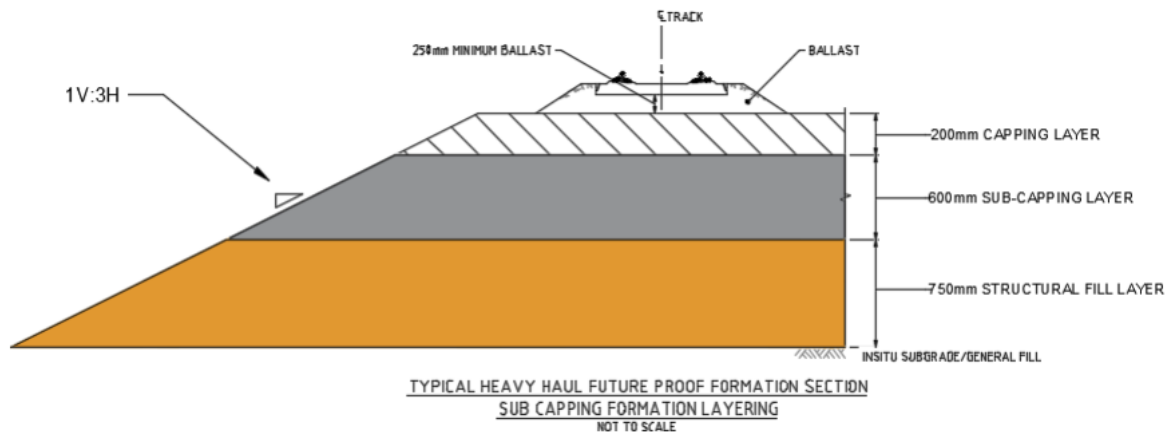


Figure 21.3 Typical flood zone embankment profile

- From the review of the flood conditions, the critical flood extents are identified to be located primarily on alluvial flood plains. The expected ground model, including subgrade conditions, is based on the following information:
 - Phase 1 and Phase 2 geotechnical investigation results:
 - Phase 1 and Phase 2 results, as obtained during the revised reference design investigations.
 - Phase 3 geotechnical investigation results:
 - Phase 3 geotechnical investigations have been undertaken to inform the Collaborative Framework Agreement phase within the Inland Rail Central and Northern packages extending from North Star to Gowrie.
 - The interpretation of the anticipated subgrade conditions is based on the following assessments:
 - Ch 0 km to Ch 137 km: A preliminary level geological assessment, using models developed in support of the EIS works
 - Ch 137 km to Ch 208 km: A detailed level of geological assessment, using models developed as part of the Civil Works Package Request and as documented in Geotechnical Interpretive Reports (4-0029-310-IGE-M2-RP-0002 and 4-0029-320-IGE-M2-RP-0002).
- Based on the available data, ground conditions for the embankment foundation along the flood plains are expected to be generally competent, typically comprising stiff to very stiff clay. There is, however, a risk that the surficial alluvium may be subject to softening where exposed to extended periods of saturation or inundation and treatment further foundation for these areas may be required depending on conditions during the construction period.
- Material parameters are assumed to be in accordance with the parameters developed for the revised reference design. These are as detailed in the *Geotechnical Interpretive Report* (FFJV, 2019) and *Geotechnical Design Criteria Report* (FFJV, 2019).
- Table 21.1 and Table 21.2 provide a summary of the design parameters. Comparison of the material design parameters adopted for the feasibility design, with later parameter refinements undertaken in the CWPR, indicate the feasibility design parameters to be generally similar or conservative.

Table 21.1 Geotechnical material parameters

Geological unit	Lithology	Description/weathering class	Unit weight (kN/m ³)	Mohr-Coulomb Design parameters		
				c _u (kPa)	c' (kPa)	φ'
Engineered Fill	Clayey Sand/ Sandy Clay	Type A/B	20	-	5	30
Engineered Fill	-	Capping	18	-	4	38
Engineered Fill	-	Sub-capping	18	-	6	36
Engineered Fill	-	Structural Fill	18	-	10	32
Qa (Alluvium)	Clays	Cohesive – Firm	17	4	1 to 5	20
	Clays/Sandy Clay	Cohesive – Stiff	18	8	5 to 10	20 to 24
		Cohesive – Very Stiff/Hard	19	12	10 to 15	20 to 24
	Sands/Gravelly Sands/Clayey Sands	Granular – Medium Dense	19	-	1	34
		Granular – Dense/Very Dense	20	-	1	38
Qs (Alluvium)	Clays/Sandy Clay	Cohesive – Stiff	18	8	5 to 10	20 to 25
		Cohesive – Very Stiff/Hard	19	13	10 to 15	20 to 25
	Sands/Gravelly Sands/Clayey Sands	Granular – Medium Dense	19	-	1	34
		Granular – Dense/Very Dense	20	-	1	40

Table 21.2 Geotechnical material parameters—Seepage

Unit	Saturated volumetric moisture content (%)	Saturated conductivity (m/s)	SWCC curve
Capping	0.3	1 x 10 ⁻⁷	Clayey Sand
Structural fill	0.3	1 x 10 ⁻⁷	Silty Sand
Type A/B fill	0.4	1 x 10 ⁻⁷	Silty Clay
Alluvial clay (CH)	0.4	1 x 10 ⁻⁷	Clay

Scenarios and critical sections

The results of the flood modelling undertaken as part of the Feasibility Design indicate that a 1% AEP event flood immunity to the proposed rail formation level is achieved for the Project alignment across the wider Project extents, including the Condamine River floodplain; however, the risks of overtopping of the Project alignment assessed for a range of extreme events indicate that at the PMF level, overtopping may potentially occur along select sections of the Project alignment.

If an extreme event occurs, one of the following four scenarios is anticipated to occur with respect to the rise in surface water levels:

- Rise in water levels in both the upstream and downstream of the embankment simultaneously (head difference between upstream and downstream is less than 0.5 m):
 - With overtopping
 - Without overtopping.
- A significantly greater rise in water levels on the upstream side of the embankment with a relatively lesser rise in water levels on the downstream side:
 - With overtopping
 - Without overtopping.

Of the four scenarios identified, only two scenarios (1a and 2b) are applicable based on the flood modelling results. These two scenarios occur at three distinct locations along the Project alignment, upstream of existing rural community hotspots, and are summarised in Table 21.3.

Table 21.3 Applicable scenarios

Chainage (km)	Applicable scenario
66.3	2b – flood head level difference without overtopping
147.1	1a – no flood head level and with overtopping
190.0	2b – flood head level difference without overtopping

Based on the flood data results, the greatest water level differentials occur in the northern section of the Project alignment between approximately Ch 189.9 km to Ch 196.5 km, and in the southern end of the alignment between approximately Ch 66.3 km to Ch 70.0 km. These areas are considered to represent the critical sections in terms of water-level differentials.

The Condamine River Valley (the largest extent subject to flooding) between Ch 137.0 km to Ch 151.0 km is considered likely to represent the critical conditions for infiltration due to having the longest inundation periods. Water level differentials through this section are, however, not significant with embankments through this zone that are typically less than 2.0 m in height. Overtopping is therefore expected to occur during events in excess of the 1 in 2,000 AEP event.

Infiltration assessments undertaken during the revised reference design indicate that infiltration of the embankment during a 10-day flood event for Type A/B fill is limited to the outer zone and toe of the embankment, except in the case that the flood level is above the structural fill of the formation. Where saturation of the embankment fill occurs, strength loss may be experienced within the material, which would lead to a reduced factor of safety in the stability of the rail embankment. It is anticipated that, in accordance with operating procedures during flood events, rail operations will not be undertaken during periods of overtopping, and collapse of the embankment due to flood waters and therefore no presence of live loads on the embankment is anticipated. This risk is further mitigated by the recommendation in the revised reference design to adopt the use of low-permeability materials in the structural fill layers, which would assist in limiting infiltration into the core of the embankment.

Noting the above, critical sections for the stability assessment have been chosen on the basis of the following:

- Sections with maximum water level differential, where collapse due to flood waters during the flood event are considered the most likely
- Sections with maximum infiltration, where collapse following receded flood waters and restart of rail operations may occur due to strength loss from the saturation of materials.

Table 21.4 summarises three chainage sections chosen using these principles.

Table 21.4 Flood sections adopted for geotechnical impact analysis

Applicable scenario	Chainage (km)	Flood level differential (m)	Ground surface level (mAHD)	Top of Formation (mAHD)	Embankment height (m)	Comment
2b	66.3	4.7	285.1	289.9	4.8	Maximum height embankment subject to maximum water level differential (southern section)
1a	147.1	1.5	381.3	382.4	1.1	Maximum likelihood of infiltration due to the inundation period experienced
2b	190.0	6.6	510.4	527.0	16.6	Maximum height embankment subject to maximum water level differential (northern section)

Slope-stability analysis methodology

Slope-stability modelling was undertaken using 'Slope/W' and 'Seep/W' in Geostudio for the applicable scenarios, shown in Table 21.4. The key assumptions, ground model and scenarios are as follows:

- No rail operations or live loads will apply during a flood event in excess of the 1% AEP (flood level above formation level)
- For the rapid drawdown scenario, pore pressure within the core of the embankment is assumed to rise due to saturation to maximum flood level. This is considered likely the worst-case scenario based on the results of the infiltration assessment undertaken during Feasibility Design, which indicates that saturation of the core zone is generally limited to the outer zones when overtopping does not occur.
- No cracking is present in the embankment fill material that may lead to increased rates of infiltration
- "Any potential risk to life as a result of the potential collapse of rail infrastructure by flood waters" is modelled in stability analysis as a factor of safety, which should not be less than unity; therefore, the minimum acceptable factor of safety is 1.0
- Scour protection is provided where required. No scour has been allowed for in the model
- Saturated parameters have been conservatively adopted for the model, regardless of whether the entire embankment is fully or partially inundated, as a worse-case scenario.

The embankment profile and ground model for the three selected sections have been developed in accordance with the design inputs listed. Table 21.5 summarises the model with respect to each of the critical sections.

Table 21.5 **Geotechnical model**

Chainage (km)	Maximum embankment batter	Embankment fill	Subsurface profile (m bgl)		
			Material description	From	To
66.3	1V:3H	Earth Fill	Alluvium (Qa) – St	0	0.5
			Alluvium (Qa) – VSt/H	0.5	1.0
			Alluvium (Qa) – D/VD	1.0	5.0+
147.1	1V:3H	Earth Fill	Alluvium (Qa) – H	0	3.0+
190.0	1V:3H	Earth Fill	Residual – D/VD	0	1.0
			MW Basalt	1.0	3.0+

21.1.2 Assessment results

21.1.2.1 Impact assessment results

The results of the risk assessment for the 1 in 2,000 AEP, 1 in 10,000 AEP and PMF flood events are illustrated in Figures R1-1 to R4-11 in Volume 2 – Appendix T2, respectively. These maps illustrate:

- The sections of Project alignment that would likely be overtopped by each AEP design flood event, and the relative risk of embankment failure at these locations.
- The location of FSRs that are already at risk of flooding in the no-failure scenario (where no-failure flood depths are > 300 mm), and consequently are not considered to be at increased level of flood risk, are indicated by black coloured receptors on Figures R2-1 to R4-11.
- The location of FSRs that are not at risk of flooding in the no-failure scenario (i.e. no-failure flood depths < 300 mm) but may be at risk of being impacted during an overtopping failure event, are indicated by the pink coloured receptors on Figures R2-1 to R4-11).

Based on the rarity of these flood events, the number of potentially impacted receptors located downstream of each of the identified embankment breach locations, and in consideration of the minor hydrostatic pressure differentials expected across the embankment during significant flood events, the potential risk to life due to overtopping failure of an embankment is considered to be low.

21.1.2.2 Slope-stability analysis

The results of the stability analysis are presented in Table 21.6 to Table 21.8 for the critical sections identified at Ch 66.3 km, Ch 147.1 km and Ch 190.0 km, respectively.

Table 21.6 Stability analysis results for the embankment at Ch 66.3 km (applicable scenario 2b)

Model type	Traffic condition	Required Factor of Safety	Achieved Factor of Safety	Embankment saturation level ¹	Satisfactory (Yes/No)
Rapid drawdown model	Without rail traffic	1.0	1.1	Full	Yes
Seepage model	Without rail traffic	1.0	1.3	Partial	Yes

Table note:

¹ Embankment saturation level refers to the maximum saturation observed in the seepage analysis.

Table 21.7 Stability analysis results for the embankment at Ch 147.1 km (applicable scenario 1a)

Model type	Traffic condition	Required Factor of Safety	Achieved Factor of Safety	Embankment saturation level ¹	Satisfactory (Yes/No)
Rapid drawdown model	Without rail traffic	1.0	1.5+	Full	Yes
Seepage model	Without rail traffic	1.0	1.5+	Partial	Yes

Table note:

¹ Embankment saturation level refers to the maximum saturation observed in the seepage analysis.

Table 21.8 Stability analysis results for the embankment at Ch 190.0 km (applicable scenario 2b)

Model type	Traffic condition	Required Factor of Safety	Achieved Factor of Safety	Embankment saturation level ¹	Satisfactory (Yes/No)
Rapid drawdown model	Without rail traffic	1.0	1.5+	Full	Yes
Seepage model	Without rail traffic	1.0	1.1	Partial	Yes

Table note:

¹ Embankment saturation level refers to the maximum saturation observed in the seepage analysis.

The results of the impact assessment to the geotechnical stability of the critical embankments upstream of communities that may be adversely impacted by extreme flooding events indicate the following:

- In the rapid drawdown-based stability assessments, where full saturation of the embankments is modelled to occur, the embankment slope stability meets and exceeds the target factor of safety of 1.0. Therefore, assuming proper construction practices occur or embankment material is not lost due to scour, no failure is anticipated to occur during the flood event.

- Seepage analysis indicated that only partial saturation of embankments is likely to occur, even where the scenario of relatively high head level differences between the upstream and downstream of embankments appears to occur. It should be noted that, in general, this level of saturation is unlikely to occur, and this assessment has been undertaken based on the conservative estimates of the following parameters:
 - Saturated material parameters: Literature studies show that soil, including both engineered fill and natural material, has the highest conductivity (permeability) when saturation. Although this is assumed in the model for conservative purposes, this is seldom the case prior to inundation due to flooding and the permeability of unsaturated material is much lesser.
 - Flood duration: The worst-case flood duration is anticipated to occur within the Condamine River floodplain and is forecasted to be in the order of 5 to 7 days at maximum based on the hydrographs developed as part of the flood modelling; however, this geotechnical assessment has conservatively modelled the flood duration to be 10 days across all three of the sections analysed. Even with such conservative estimates, it appears that the minimum required factor of safety under extreme flood events is achieved.
- Where overtopping occurs at the Condamine River floodplain section of the rail embankment (analysis Ch 147.1 km), the minimum required Factor of Safety of 1.0 is met for slope stability in both the fully saturated, rapid-drawdown as well as the seepage-based partially saturated scenarios.
- For sensitivity analysis, an additional scenario corresponding to a reduced flood level (following the maximum flood event), where the top of the upstream flood level is approximately at the subgrade level (i.e. below structural fill), was analysed with an 85 kPa rail traffic load (representing 30T axle load moving at 80 km/h). This analysis also confirms that, at the reduced upstream head level, the embankment global slope stability still achieves a minimum Factor of Safety of 1.0, even with the immediate application of a rail load.

21.1.3 Conclusions and mitigations

The following mitigations and conclusions have been made from the above assessment:

- A failure impact assessment be undertaken, where required, during detailed design at selected plausible/potential failure locations to assess risk to life, and to further optimise/enhance the design to manage any such risks.
- It should be noted that although the minimum requirements for embankment global geotechnical stability are met for the modelled scenarios, there are potential processes or actions that may cause embankments to undergo failure; this includes, but is not limited to, the following:
 - Embankment erosion, including the erosion of protective covering such as grassing, causes the migration of embankment fill due to the sheer velocity of floodwater:
 - Erosion at the embankment toe: Given that the upstream and downstream toe of embankments are subject to the effects of saturation the longest relative to the remainder of the embankment coupled with the flow of water (velocity), erosion may occur. Adequate provisions against toe protection are, therefore, necessary to mitigate this risk.
 - Poor construction practices during the construction of the rail embankments, lack of proper maintenance over the design life, or inadequate inspection regimes that are not in accordance with best industry practice guidelines and specifications can cause embankments to undergo failure during flooding events:
 - It is assumed that appropriate inspection regimes are incorporated into the pre-construction, early works and construction works, and commissioning and operations stages over the design life of the embankments, in line with industry best practice.

- If an extreme flooding event occurs, the embankment following the subsidence of the floodwater must be inspected prior to recommencement of rail operations and, where failed sections are identified, such sections are rectified adequately to allow operation. An inspection manual such as the *Track Drainage – Inspection and Maintenance* (RTS 432) (ARTC, 2006) or similar shall be used.

21.2 Design measures to mitigate overtopping failure

21.2.1 Formation, earthworks and cross-drainage design measures

Embankments located in flood areas are identified as potential stability risks, as described in Section 21.1.1.1. In addition to saturation of fill, flood zones may further impact on the performance of the embankment due to causing softening, extrusion and undercutting at the embankment toe; external erosion from scour; internal erosion from water ingress; and impact from debris or objects carried by flood water. To manage these risks, the following formation and earthworks design measures were adopted within the revised reference design:

- Two formation designs were considered, namely the standard ARTC formation and the formation subject to flood. The standard ARTC formation uses only the layers prescribed in the ARTC specification, and the formation subject to flood adds two additional sub capping layers to improve the performance of the formation when it is inundated with water (Figure 21.4).

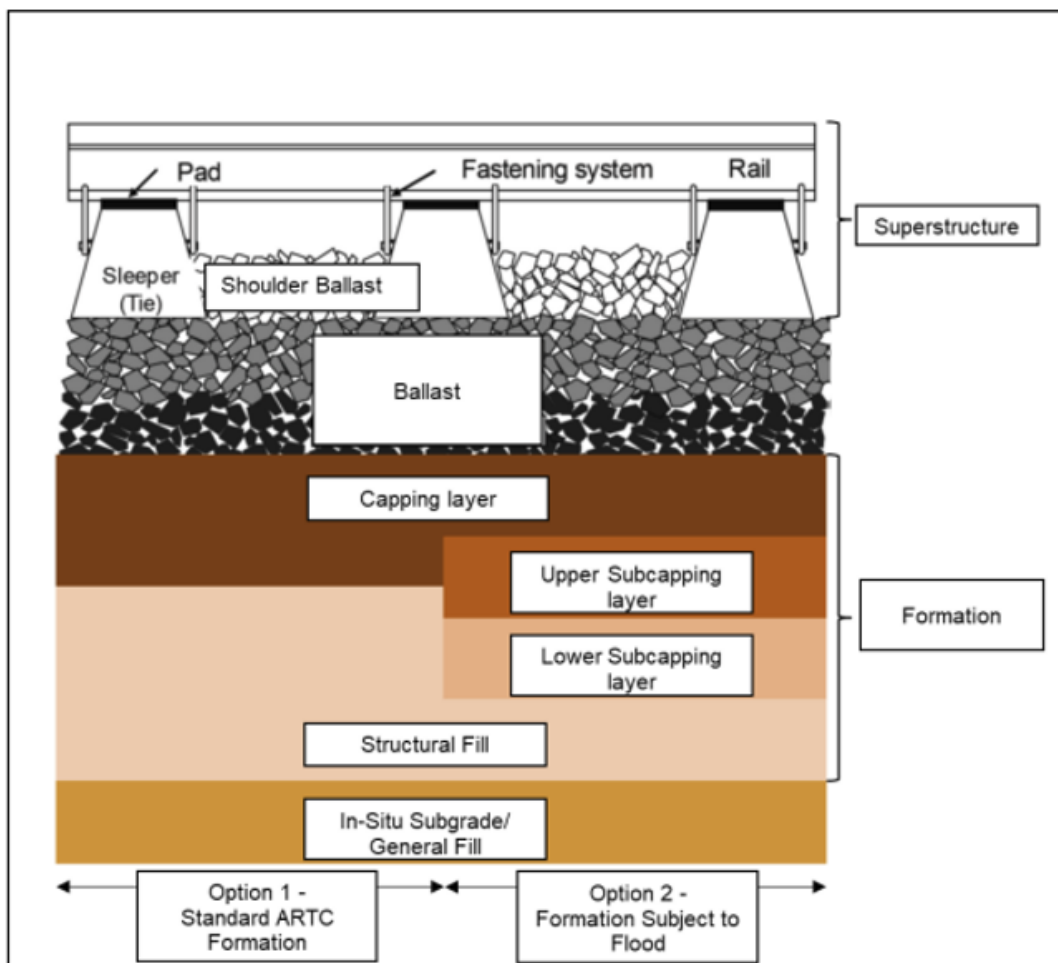


Figure 21.4 Rail formation design

- The revised reference design provides embankment geometry and foundation preparation requirements to meet the Basis of Design for overall stability and settlement. The Project's revised reference design considers embankment zoning, foundation treatment and drainage. All significant areas of mainline embankment over 10 m were assessed. Additionally, the stability of low embankments over soft soils was reviewed. Embankments underlain by similar subsurface profiles were identified and analysed using the material model, with surface water modelling corresponding to the 1% AEP flood event characteristics applied where one or more of these embankment sections are found to cross areas susceptible to flood. The ground models assumed by the revised reference design are detailed in the *Geotechnical Design Criteria Report* (FFJV, 2019a).
- Generally, embankment fill slopes are protected against erosion through the adoption of appropriate batter geometry and material specifications. Batter side slopes of 1V:3H were adopted for all flood zones with rapid drawdown (seepage) models having been undertaken as a stability check for low-height embankments identified within flood zones. Based on the results and the anticipated subgrade conditions, the proposed embankment geometry achieved the required minimum factor of safety.
- Low-permeability structural fill and capping were assumed to be provided by the Project design. This will require an amendment to the standard material specification to provide structural fill with the same permeability requirements as that currently specified for capping (ARTC, 2020), where the water level is expected to reach the base of formation level. Seepage modelling was conducted where water infiltration through the structural fill was identified as a potential risk that may lead to increased saturation of the embankment fill as described in Section 21.1.1.1.
- For sections of the embankment exposed to flooding and scour, scour protection is to be implemented where 1% AEP velocities exceed 1 m/s at structures, and on embankment faces where longitudinal flow may occur along the embankment as it moves towards a cross-drainage opening. For preliminary purposes only, it is assumed 1,000 mm thick of rock armour (rockfill shall be in accordance with ETC-08-03 (ARTC, 2020)) for slopes no steeper than 1V:2H at the locations outlined in Table 21.9. Note, however, that these details may be subject to change as the design is refined in future Project stages—alternate design considerations/measures would also be looked at beyond what is discussed in this section.
- In locations where longitudinal drainage is to be provided, drains are rock-lined in steep areas and grasses lined with velocity stop (as necessary) in flat areas.

Table 21.9 1,000 mm thick of rock armour required in the 1% AEP event

Ref	Length (m)	Flood model	Peak depth above toe (m)	Peak velocity at embankment (m/s)	Modelled slope profile	Rock protection
At grade	1,510	Bringalily Creek	1.9	2.6	1V:2H – 1 st	To 1% AEP
E38	2,800	Bringalily Creek	1.9	2.6	1V:2H – 1 st	To 1% AEP
At grade	260	Bringalily Creek	1.9	2.6	1V:2H – 1 st	To 1% AEP
E39	130	Bringalily Creek	1.9	2.6	1V:2H – 1 st	To 1% AEP
At grade	180	Bringalily Creek	1.9	2.6	1V:2H – 1 st	To 1% AEP
E40	880	Bringalily Creek	1.9	2.6	1V:2H – 1 st	To 1% AEP
E53	3,350	Back Creek	1.5	1.6	1V:2H – 1 st	To 1% AEP
E73	5,390	Westbrook Creek	2.6	1.0	1V:2H – 1 st	To 1% AEP
E76	3,030	Gowrie Creek	1.5	1.5	1V:3H – 1 st	To 1% AEP
At grade	160	Gowrie Creek	1.5	1.5	1V:3H – 1 st	To 1% AEP
E77	900	Gowrie Creek	1.5	1.5	1V:3H – 1 st	To 1% AEP

Cross-drainage infrastructure (e.g. bridges and culverts) has been included at major waterway crossings and key drainage paths to limit head differential and hydrostatic pressure across the embankment during significant flood events. This reduces the risk associated with extreme events, and any sudden surge of floodwater associated with embankment failure, as elevated tailwater levels help to mitigate against this scenario and significantly dampen the effects of any failure in the unlikely event it was ever to occur.

21.2.2 Recommendations for risk mitigation in future design stages

It is recommended that the following items be considered on a case-by-case basis during detailed design:

- Embankment failure mechanisms during overtopping events for selected AEP design flood events
- Design for overtopping events, including:
 - Crest level against design water surface profiles for selected AEP design flood events
 - Overtopping flow rate for selected AEP design flood events
 - Landward slope steepness, topsoil material and revetment properties
 - Maintenance regime employed
 - Armouring of sections of embankment (e.g. rock, geotextile, etc.) to minimise overtopping scour potential.
- Local solutions to reduce the effects of overtopping, including a 'low point' or spillway, to help control extreme situations in which a levee is overtopped by ensuring that the water overtops in the least vulnerable area, combined with sufficient drainage systems to get rid of the overtopped water
- Ensuring design provides sufficient cross-drainage capacity to limit any head differential and hydrostatic pressure across the embankment during extreme events
- Consideration of blockage risk and, in turn, at any critical locations, consideration of the risk this could pose to potentially high-risk receptors. Mechanisms to manage this could include blockage mitigation measures (e.g. fencing, debris screens, deflector walls, etc.) or redundant capacity, alternate structure types or configurations.

21.3 Flood emergency response measures

As discussed, the Project embankment is designed with 1% AEP flood immunity to formation level (a typical design standard for contemporary linear infrastructure more generally). Accordingly, the occurrence of extreme flood events, such as the 1 in 2,000 AEP, 1 in 10,000 AEP and PMF events, implies the potential for linear infrastructure embankments to fail under such extreme flood conditions is heightened. In order to minimise risk to life with respect to the Project's revised reference design, the following ARTC flood emergency management measures and procedures are recommended:

- Development and application of an *Inland Rail Border to Gowrie Emergency Management Plan and Inland Rail Border to Gowrie Flood Emergency Response Plan*, including coordination with the relevant local council and LDMG
- Addition of ARTC as an Advisory Member of the relevant LDMG
- Application of the *ARTC Emergency Management* (ARTC, 2024)), including coordination with the relevant local council and LDMG
- Ensuring an appropriate Inland Rail embankment inspection and maintenance program as per relevant ARTC Rail Track Engineering Practice Manuals for Civil Engineering.

It is noted that during significant flood events, it is likely that the BoM and LDMG would have issued a flood alert to the community, and the community would be preparing for a flood event, including potential for evacuation. The key BoM, LDMG and ARTC management actions and communications would include:

- ARTC would enact the *Inland Rail Border to Gowrie Emergency Management Plan and Inland Rail Border to Gowrie Flood Emergency Response Plan*, and other relevant ARTC emergency management procedures. This would include notifying the relevant LDMG of any potential or known embankment failures (or areas of concern) and associated potential downstream impacts. ARTC emergency management flood response protocols would also be followed to safeguard ARTC personnel.
- Warnings would be coordinated and issued to the local community (including receptors located downstream of the Project) in line with the *Inland Rail Border to Gowrie Emergency Management Plan and Inland Rail Border to Gowrie Flood Emergency Response Plan*, which would involve liaison between GRC, TRC, BoM, and relevant LDMGs. This would include any required evacuation orders and preferred evacuation routes.
- It is recognised that the BoM flood watches, flood warnings and severe weather warnings could be issued well in advance of flooding occurring (typically on larger, regional catchments), which may allow also precautionary action to be taken prior to a flood event on specific catchment crossings. Any such action would be done in line with the *Inland Rail Border to Gowrie Emergency Management Plan and Inland Rail Border to Gowrie Flood Emergency Response Plan*. Note that:
 - A BoM flood watch is likely to be issued when forecast rainfall information suggests that local and/or riverine flooding is possible across the flood watch area
 - A flood warning is likely to be issued when the BoM is more certain that flooding is expected, often when rainfall has started. Flood warnings are more targeted and are issued for specific catchments or even sub-catchments in some of the larger river basins. Flood warnings will generally include specific predictions of the severity of expected flooding.

These coordinated multi-agency flood response actions would help to reduce risk to life during extreme flood events.

21.4 Conclusion

ARTC recognises that each catchment will have its own specific characteristics (catchment response, flood behaviour, receptor presence, etc.) and therefore its own distinct risk profile. Accordingly, ARTC will ensure that, at the detailed design stage, mitigation strategies are implemented on a crossing-specific basis, and tailored to each catchment's characteristics and risk profile. For example, this may involve a risk mitigation strategy principally centred on the engineering of the design (e.g. for catchments that may have a short response time and flash-flood mechanism, or which are potentially isolated) but may also incorporate response-type measures as discussed above (i.e. where they are suitable to apply). ARTC will undertake a detailed assessment to consider each risk location specifically, and to optimise the design (and emergency planning measures) to suit each location.

22 Mitigation measures and strategies

This section provides details of mitigation measures and controls that have already been incorporated into the revised reference design to avoid and minimise pre-construction, early works and construction works, and commissioning and operational impacts of the Project; those that would be implemented in future Project phases to further avoid and reduce impacts; and the approach for the management of residual impacts after the application of all feasible and reasonable mitigations.

The Project's proposed approach to environmental management, including mitigation measures, performance criteria, responsibilities and monitoring, is detailed in EIS Chapter 24: Draft Outline Environmental Management Plan.

22.1 Mitigation—Revised reference design stage

Following additional work undertaken since notification of the draft EIS and consideration of the issues raised from the community, agencies and other stakeholders in their submissions to the draft EIS and follow-up consultation, the reference design was amended to its current design, the revised reference design as presented in EIS Appendix B1: Design Drawings, described in EIS Chapter 5: Project Description and, with respect to flooding infrastructure, the Cross-Drainage Register in Appendix B.

The current revised reference design for the Project has been developed with the objectives of minimising potential impacts on the surrounding environment and land use. The Project has been designed to achieve the hydraulic design criteria including 1% AEP flood immunity to rail formation level. At the same time, the design seeks to minimise impacts that do not meet the FIOs for the flooding and drainage regime.

Key mitigation strategies were incorporated in developing the revised reference design to minimise flood impacts, including:

- Flood models were developed in consultation with stakeholders and, where possible, models were calibrated and validated using stakeholder-supplied information. Some examples of this information include surveyed flood markers of historical flooding observed by long-term residents, and general observations of residents of timing and behaviour of historical floods. Recent flood events (i.e. December 2021) have also been used to validate the flood models. The models were strengthened further following a comprehensive review by the Expert Flood Panel. A detailed summary of the issues raised by the Expert Flood Panel and ARTC's responses are presented in the Issues Management Register in Appendix A.
- The Project uses the existing Southwestern Line and Millmerran Branch Line rail corridors as far as possible to avoid introducing a new linear infrastructure corridor across floodplains. This means that 68.0 km of the total 217.48 km Project length is located within existing rail corridor (brownfield development).
- In greenfield sections of the Project, the selection of the Project alignment considered flooding constraints to avoid and minimise impact to areas experiencing existing inundation. For example, the Project alignment predominantly avoids the 1% AEP regional floodplains of the Macintyre Brook and Gowrie Creek.
- The Project incorporates bridge and culvert structures to maintain existing flow paths and flood-flow distributions, such as across the Condamine River floodplain, where four significant bridges were incorporated into the Project's revised reference design with a combined opening width of greater than 6 km. The Project's cross-drainage provisions (bridges and culverts) are presented in the Cross-Drainage Register in Appendix B and include the following:
 - 1,071 reinforced concrete box culverts at 123 locations
 - 2,057 reinforced concrete pipe culverts at 214 locations

- A series of viaducts/bridge structures spanning 10 km watercourses and floodplains
- 1.6 km of rock scour protection at culvert outlets.
- Bridge and culvert structures were located and sized to minimise increases in peak water levels, flow distribution, velocities, duration of inundation and hazard in accordance with the FIOs
- Additional assessments and the refinement of the revised reference design to address key stakeholder concerns regarding changed flooding conditions leading to erosion, sedimentation, increased risks of flooding and environmental concerns. Design refinements include bridge and culvert designs (e.g. location, number of barrels and dimensions) and additional scour and erosion protection measures in areas identified to be at risk from increases in velocity.
- In response to concerns regarding the dispersive nature of soils particularly in floodplains, site-specific soil and geomorphology assessments were undertaken to identify erosion and landform risks, inform the scour and erosion protection strategy and allow updates to the flooding mitigations.
- An extreme event assessment was undertaken to identify whether there is any potential risk to life as a result of the potential collapse of rail infrastructure by flood waters, and suitable mitigation and management measures were identified
- The revised reference design includes the modification of the existing Yelarbon flood levee for the township of Yelarbon with the addition of the Project. At a minimum, the flood levee augmentation will ensure no worsening of existing flood risk to the township of Yelarbon.
- A climate change assessment was undertaken for the 1% AEP design event. This assessment is in accordance with the ARR 2019 Guidelines and was used to determine the sensitivity of the Project's revised reference design and FSRs to the potential increase in rainfall intensity.
- Potential blockage of hydraulic structures caused by floodplain debris, as highlighted by stakeholders, has been factored into the revised reference design by allowing an additional 25 per cent flow capacity in culverts, and by placing appropriately sized bridges over major debris transportation paths.
- Consultation with potentially impacted flood sensitive receptors and landowners was undertaken during the reference design development to validate flood model outcomes, and inform acceptable mitigation measures and design outcomes.

22.2 Overlap between geomorphology and flooding

As discussed in Section 1.6, there is a distinct overlap between hydrology and flooding, and geomorphology; as such, the below mitigation framework has been combined to provide one framework to manage both flooding and geomorphic impacts. The below framework identified in Section 22.3 should be read in conjunction with mitigation measures identified in EIS Appendix H: Geomorphology Report.

22.3 Proposed mitigation framework—Future Project stages

It is recognised that further refinement and confirmation of site-specific mitigation measures will be undertaken during detailed design, pre-construction, early works and construction works; and commissioning and operations stages with respect to FIOs developed in consultation with the Expert Flood Panel. It is noted that the purpose of the objectives is to identify areas where additional consideration is required and to determine whether the impact is acceptable, or mitigation is required. Mitigations will be determined on a case-by-case basis subject to consideration of environmental, economic, engineering and landowner constraints, and informed by ongoing and collaborative consultation with landowners. This approach will allow for a robust process to mitigation selection to ensure that a focus on reducing FIO exceedances is not considered in isolation from other Project aspects.

Additional detailed survey and local soils/vegetation information will be gathered to inform detailed design, along with ongoing consultation and discussion with landowners/stakeholders. These steps will aid in defining the preferred detailed design solution, notwithstanding that Project/design constraints will also play a role in this definition process to ensure defensible and robust design solutions are achieved, as is the norm for linear infrastructure projects of this nature. It is recognised that it may not be possible to eliminate FIO exceedances due to environmental or engineering constraints, or prohibitive costs. It also may not be necessary where further assessment determines that the exceedances are not material and do not warrant further action.

Updated modelling will be undertaken in detailed design, which will consider the effect of additional inputs, such as noise walls, at a local catchment scale, for all defined AEP events. Mitigation measures will be reviewed and updated as required based on updated modelling in the detailed design stage.

The Project's Flood and Geomorphic Impacts Mitigation Framework is presented in Figure **22.1** and discussed below.

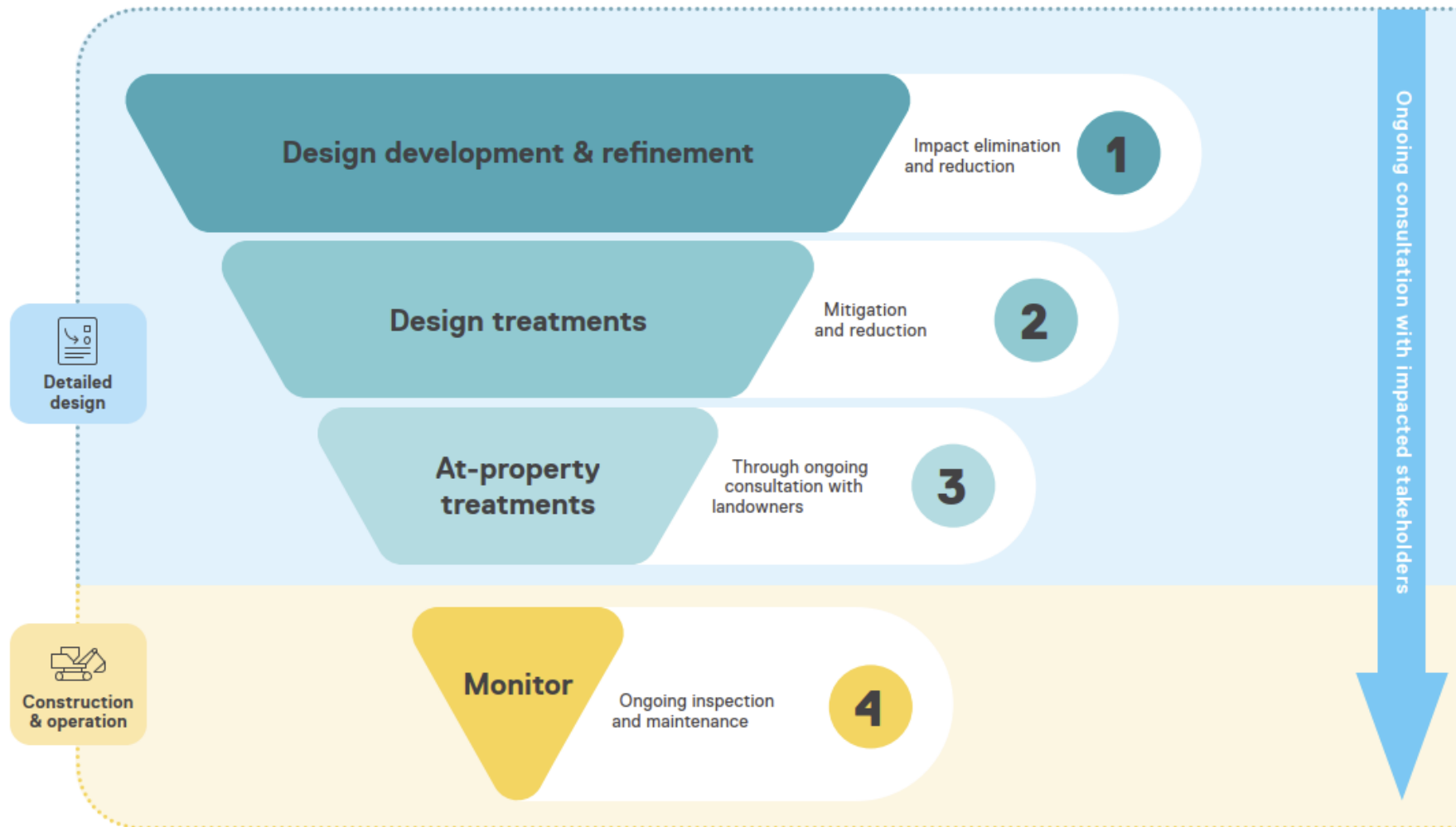


Figure 22.1 ARTC Flood and Geomorphic Impacts Mitigation Framework

Where residual FIO exceedances or geomorphic impacts not yet eliminated (or not yet reduced to an acceptable level during the revised reference design) were identified, ARTC will in the first instance investigate alternative design measures, directly related to the Project's revised reference design, to mitigate these impacts. The following mitigation process will be applied in the detailed design stage (Steps 1 to 3) and pre-construction, early works and construction works, and commissioning and operations stages (Step 4) and further expands on the four-step process identified in Figure 22.1:

- **Step 1:** Further design development and refinement (detailed design):
 - Refine the flood models further in line with Expert Flood Panel recommendations that relate to detailed design requirements
 - Further detailed consideration of (or investigation into) FIO exceedances and geomorphic impacts to understand their consequences on land, waterways and assets
 - Engage with impacted landowners on a case-by-case basis to understand how FIO exceedances and geomorphic impacts are likely to affect their assets and/or operations
 - Develop the Project hydraulic design further (e.g. through additional cross-drainage structures, relocation or reconfiguration of cross-drainage structures) to eliminate or reduce FIO exceedances and geomorphic impacts further, in consultation with directly impacted landowners.
- **Step 2:** Revised design treatments (Section 22.4.1) (detailed design), for example:
 - Introduce alternative cross-drainage solutions (e.g. additional cells, alternative types of structures, changes to pipe/box angles, changes to invert levels/grades, spreading of culvert cells, etc).
 - Scour and erosion protection measures
 - Modifications to bridge configurations to minimise encroachment into waterway channels and geomorphic impacts
 - Flow energy dissipaters, etc.
 - Consideration of the proposed design treatment's ability to reduce FIO exceedances and geomorphic impacts balanced with their social, environmental and economic impact.
- **Step 3:** At-property treatment (Section 22.4.2) (detailed design, pre-construction and early works), for example:
 - Consideration of the at-property treatment's ability to manage residual flooding and geomorphic impacts, balanced with their social, environmental and economic impact
 - Land agreements, such as deed of release, easements, positive covenant, freehold acquisition
 - Compensation for mitigation, such as raising or restumping a house.
- **Step 4:** Monitor or adjust and refine (Section 22.4.3) (Pre-construction, early works and construction works, and commissioning and operations):
 - Inspection and monitoring:
 - Establish baseline ground conditions through a detailed survey
 - Identify (if any) non-conformances
 - Corrective action if required
 - Investigate non-conformances
 - Identification and implementation of corrective/preventative actions
 - Develop reporting, close-out and ongoing maintenance/monitoring requirements
 - Ongoing inspection, maintenance and reporting.

It is important to note that a FIO exceedance would not necessarily cause an *impact*. As discussed in Section 4.2, the FIOs define the threshold as to the point at which a more detailed consideration of impacts is required when they are exceeded. Depending on land use, the underlying ground conditions, gradient etc., a site-specific assessment in consultation with the landowner will determine if the FIO exceedance is likely to lead to an impact.

Mitigation strategies were developed by ARTC to demonstrate how FIO exceedances will be managed in the commissioning and operations stages. Modelling was undertaken to demonstrate the viability and practicality of proposed mitigation strategies. This process is discussed in Section 22.4.

The consultation undertaken with landowners regarding the proposed flood impacts to their property, including preliminary mitigation that's been identified through this consultation process, is discussed in EIS Appendix E: Consultation Report. As demonstrated in Figure 22.1, ongoing consultation between ARTC and impacted landowners will occur throughout all four stages of the *Flood and Geomorphic Impacts Mitigation Framework* to ensure landowner sentiment is utilised to guide the mitigation process.

A management framework has been provided for the Project to enable the identified outcomes to be achieved for the life of the Project, through detailed design, pre-construction and early works, construction, commissioning and operations stages. This includes the process for the preparation and implementation of the CEMP and the Operation EMP.

The CEMP will provide a structured approach to the management of environmental issues during the delivery of the Project. The CEMP will be endorsed by the Environmental Monitor as being consistent with the Draft Outline EMP, Conditions of Approval, and all relevant laws, prior to the commencement of any relevant Project works. Implementation of flooding and geomorphology elements of the CEMP and compliance against conditions of approval will be monitored, audited and reported.

As part of the CEMP, a Flood Risk Management Plan will be developed to outline management and control measures, as well as emergency procedures, to be implemented during the for pre-construction, early works and construction works in order to mitigate flood impacts. As part of this plan, details around the requirements associated with Flood Risk Assessments will be included to manage the impacts of high-risk construction works. Pre-construction, early works and construction works will be managed by the following criteria:

- Minimise re-direction of flood water flow
- No adverse increases to flood levels at FSRs
- Maintain existing velocities where practical

Erosion and sediment control is managed in accordance with the Construction Erosion and Sediment Control Plan (ESCP).

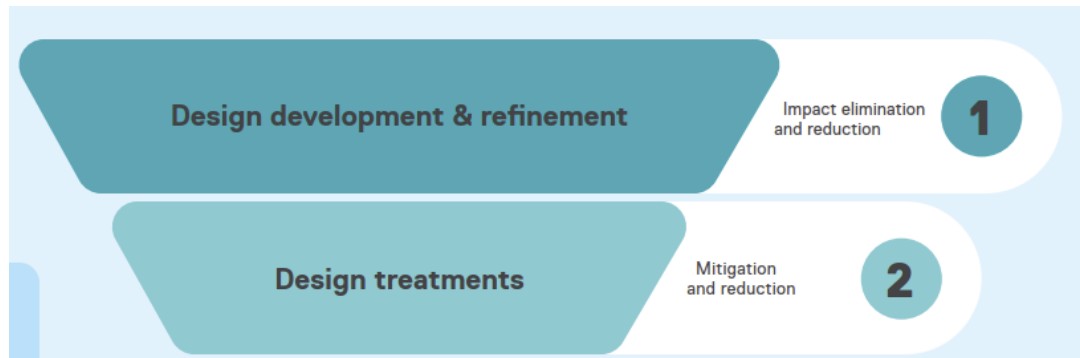
The effectiveness of flood mitigation controls will be monitored in accordance with the CEMP and associated Flood Risk Management Plan.

During the commissioning and operations stage of the Project, flood impacts will be managed via the Operation EMP, which will be supplemented by a Flood Emergency Management Plan.

22.4 Mitigation strategy for Flood Impact Objective exceedances

Specific proposed mitigation strategies to effectively manage FIO exceedances, including proposed monitoring programs to be implemented during construction and operation, are discussed below.

22.4.1 Design development, refinement and treatment



22.4.1.1 Potential detailed design mitigation measures

FIO exceedance mitigation measures are generally site-specific and can require a balance between mitigating one FIO exceedance without creating another. Although this may be the case, several mitigation measures can be explored with respect to each FIO exceedance category. During the detailed design stage, mitigation measures detailed below (but not limited to) will be explored to reduce FIO exceedances as far as reasonably practicable prior to consideration of site-specific solutions. Mitigation options will be informed by ongoing consultation with impacted landowners.

Table 22.1 details a hierarchy of potential mitigation measures for each FIO category. It should be noted that, in some cases, a combination of these measures may be required to mitigate FIO exceedances and not all exceedances will be mitigatable.

Figure 22.2 provides examples of different industry-standard engineering solutions available.

Table 22.1 Hierarchy of measures to reduce flood impacts



Hierarchy of control measures to reduce flood impacts

Mitigation Steps		Afflux	Velocity	Duration of inundation	Hazard	Distribution of Flow
General order of mitigation preference	Step 1 Design development and refinement	- Improved survey and LiDAR - Hydrologic/hydraulic model refinement - Bathymetric survey - Incorporation of EIS Conditions of approval - Confirmation of geotechnical conditions - Confirmation of vertical/horizontal alignment	- Improved survey and LiDAR - Hydrologic/hydraulic model refinement - Bathymetric survey - Incorporation of EIS Conditions of approval - Confirmation of geotechnical conditions - Confirmation of vertical/horizontal alignment	- Improved survey and LiDAR - Hydrologic/hydraulic model refinement - Bathymetric survey - Incorporation of EIS Conditions of approval - Confirmation of geotechnical conditions - Confirmation of vertical/horizontal alignment	- Improved survey and LiDAR - Hydrologic/hydraulic model refinement - Bathymetric survey - Incorporation of EIS Conditions of approval - Confirmation of geotechnical conditions - Confirmation of vertical/horizontal alignment	- Improved survey and LiDAR - Hydrologic/hydraulic model refinement - Bathymetric survey - Incorporation of EIS Conditions of approval - Confirmation of vertical/horizontal alignment
	Step 2 Design treatments	- Culvert location and spread - Culvert grade and invert levels - Culvert inlet/outlet realignment to existing flowpaths - A reduction or increase in culvert cell numbers - Realignment of abutments outside flow paths - Temporary water storage during a flood (flood mitigation dams and retarding and detention basins) with pumping systems - Revegetation of the catchment	- Scour protection (rip rap or gabions) - Culvert location and spread - Culvert grade and invert levels - Culvert inlet/outlet realignment to existing flowpaths - A reduction or increase in culvert cell numbers - Realignment of abutments outside flow paths - Lowering of culvert cell invert level to allow boulders and rocks to be deposited in cells - Flow baffles/energy dissipators	- Culvert location and spread - Culvert grade and invert levels - Culvert inlet/outlet realignment to existing flowpaths - A reduction or increase in culvert cell numbers - Realignment of abutments outside flow paths - Improved flood conveyance, including cutoff drains - Temporary water storage during a flood (flood mitigation dams and retarding and detention basins) with pumping systems	- Culvert location and spread - Culvert grade and invert levels - Culvert inlet/outlet realignment to existing flowpaths - A reduction or increase in culvert cell numbers - Realignment of abutments outside flow paths	- Culvert inlet/outlet realignment to existing flowpaths - Realignment of abutments outside flow paths - Formalising flowpaths for any diversions that may be required
	Step 3 At-property treatment	- Permanent levee and weir systems - Temporary barriers - Temporary water storage during a flood (flood mitigation dams and retarding and detention basins) with pumping systems - Revegetation of the catchment - Property specific solutions/compensation in consultation with impacted landowner	- Riparian vegetation/revegetation along the riverbanks and within the catchment - Property specific solutions/compensation in consultation with impacted landowner	- Permanent levee and weir systems - Temporary barriers - Temporary water storage during a flood (flood mitigation dams and retarding and detention basins) with pumping systems - Property specific solutions/compensation in consultation with impacted landowner	Property specific solutions/compensation in consultation with impacted landowner	Property specific solutions/compensation in consultation with impacted landowner
	Step 4 Monitor	- Compliance and monitoring in accordance with the finalised Construction Environmental Management Plan (CEMP), Operations Environmental Management Plan (Operations EMP) and other relevant project requirements. - Inspection and maintenance of stormwater infrastructure - Corrective action if required	- Compliance and monitoring in accordance with the finalised CEMP, Operations EMP and other relevant project requirements. - Inspection and maintenance of stormwater infrastructure - Corrective action if required	- Compliance and monitoring in accordance with the finalised CEMP, Operations EMP and other relevant project requirements. - Inspection and maintenance of stormwater infrastructure - Corrective action if required	- Compliance and monitoring in accordance with the finalised CEMP, Operations EMP and other relevant project requirements. - Improving availability of flood information i.e. searchability and sharing of information under appropriate arrangements and public availability of key project outputs - Development of Flood Emergency Management Plans - Evacuation route planning and improvement - Community flood awareness and preparedness - Development controls	- Compliance and monitoring in accordance with the finalised CEMP, Operations EMP and other relevant project requirements. - Improving availability of flood information i.e. searchability and sharing of information under appropriate arrangements and public availability of key project outputs - Development of Flood Emergency Management Plans - Evacuation route planning and improvement - Community flood awareness and preparedness - Development controls

Mitigation measures – Examples



Erosion and scour rock protection at culvert outlets (Catchments and Creeks Pty Ltd)



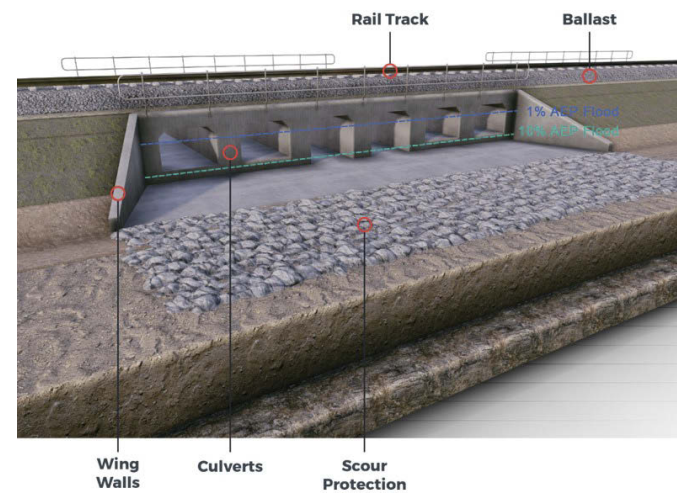
Spaced culvert cells to spread flow (ARR 2019)



Rock revetment mattress—Culvert outlet application (Global Synthetics, 2020)



Energy dissipater to reduce velocities (PACE Advanced Water Engineering)



Varying culvert invert levels to balance conveyance (ARTC, N2NS EIS)



Flexible concrete mattresses (IECS, 2023)



Concrete pour as part of the drainage construction ('apron') (ARTC, 2022)



Finished Croppa Creek Bridge with rehabilitated riverbank (ARTC, 2022)



Backfilling the culverts as part of the formation works south of Moree (ARTC, 2022)

Figure 22.2 Example mitigation measures

22.4.1.2 Scour and erosion protection downstream of culverts

The revised reference design maintains existing water flow paths, and cross-drainage structures are located to ensure efficient conveyance and spread of floodwaters. Mitigation measures were included in the revised reference design where increases in flow velocities above a defined threshold (Erosive threshold velocity values) are expected, with flood events up to the 1% AEP event considered. Mitigation is achieved by changing the drainage structure design or through providing appropriate scour protection or energy dissipation.

The following sections outline the operational erosion and scour protection strategy for the Project, and the assessment that was undertaken to inform the strategy. Further refinement and confirmation of exact mitigation measures will be undertaken during detailed design, when detailed survey and site-specific soils and vegetation information becomes available. In addition, consideration of the controls detailed within the Department of Primary Industries (1993) *Floodplain Management for Erosion Control and High Productivity on the Darling Downs* document, will be undertaken during the detailed design stage to further refine the revised reference design.

Scour protection provisions identified during the revised reference design stage

During the initial reference design stages, scour protection requirements for culverts were calculated based on the velocities predicted from the hydraulic modelling. The scour protection was designed in accordance with Austroads *Guide to Road Design Part 5B: Drainage* (2013). Scour protection was specified where the culvert outlet velocities for the 1% AEP event exceeded the recommended soil velocity thresholds.

The scour protection length and minimum rock size (d50) were determined from Figure 3.15 and Figure 3.17 in *Guide to Road Design Part 5B: Drainage* (Austroads, 2013). Resulting length of scour protection required were determined through the drainage assessment.

A conservative scour estimation was undertaken at the bridge locations based on available information and will be refined during detailed design.

The Expert Flood Panel identified additional assessment requirements in relation to managing changes to flood-flow velocities. The additional assessments are outlined below.

Flood Impact Objectives—Velocity

The FIOs developed in consultation with the Expert Flood Panel for the revised reference design include the following requirements relating to the management of potential changes to velocities:

- Maintain existing flood flow velocities where practical or minimise increases in velocities
- Identify changes to velocities and impacts on external properties³
- Establish ETV values for natural ground surfaces from a site-specific assessment by a suitably qualified specialist, and in consideration of engineering guidelines (for sealed surfaces, this same approach could be applied, or through reference to suitable guidelines/specifications)
- Assess changes in velocities against the performance objectives for sealed surfaces and natural ground surfaces.

Erosive threshold velocity values

Initial ETV values for soil types encountered by the Project were determined by SGM Environmental and documented in their report *Inland Rail Border to Gowrie (B2G) Erosion Threshold Velocities* (SGM, 2022), located in Appendix D.

³ Properties external to the Inland Rail Border to Gowrie Project permanent (or operational) footprint.

Table 8 in SGM (2022) presents estimated ETVs for bare soil and 50, 70 and 100 per cent vegetation cover scenarios by soil groups and erodibility classes. The ETVs along the Project for a 50 per cent vegetation cover scenario is estimated at between 0.9 and 1.2 m/s.

Supplementary testing using direct methods (flume testing) and subsequent ETV determination based on revised results is currently underway at a number of potentially high-risk sites, including on floodplains and overland flow paths. Relevant mitigations including those relating to erosion treatment options within and external the corridor will be reviewed and revised as necessary based on the updated analysis.

Velocity impact assessment

An impact assessment was undertaken against the FIOs using the Existing Case and Development Case hydraulic modelling results to define velocity exceedances on properties external to the Project's permanent footprint, based on the revised reference design. A representative and conservative average bare soil ETV value of 0.5 m/s was adopted for the purposes of this impact assessment.

Unsealed surfaces likely to be impacted by a velocity FIO exceedance (measured against a target of 0.5 m/s) were identified and reported in the 'Flood Impact Objective Outcomes' sections of each floodplain/local catchment section of this report.

Sealed surfaces (i.e. surfaced roads) likely to be impacted by a velocity FIO exceedance (measured against a target of 1.0 m/s) were identified and reported in the 'Flood Impact Objective Outcomes' sections of each floodplain/local catchment section of this report.

In addition to initial scour protection requirements identified during the revised reference design, areas immediately downstream of culvert outlets that may experience FIO velocity exceedances were also identified, and additional scour protection allowed for within the revised reference design.

22.4.1.3 Detailed design mitigation

During the detailed design stage of the Project, design modifications will occur that will be subject to re-runs of the existing models, to demonstrate continued compliance with design objectives of the Project, including for flood extent, time of inundation, afflux, velocity and hazard. Modelling will consider the effect of additional inputs such as noise walls, at a local catchment scale, for all defined AEP events. As a part of this assessment, mitigation measures can be embedded within the Project's revised reference design to further reduce FIO exceedances across the Project alignment.

Mitigation measures adopted within the detailed design will be selected on a case-by-case basis and be subject to an analysis of their ability to reduce FIO exceedances as well as their respective social, environmental and economic impacts. This approach will allow for a robust process to mitigation selection to ensure that a focus on reducing FIO exceedances is not considered in isolation from other Project aspects.

Dependant on the outcomes of the detailed design modifications, the following mitigation measures will be further explored in order to mitigate and reduce FIO exceedances:

- Culvert location and spread
- Culvert grade and invert levels
- Culvert inlet/outlet realignment to existing flowpaths
- A reduction or increase in culvert cell numbers, etc.

To demonstrate the effectiveness of mitigation measures, a representative sample of culvert sites that currently experience a range of FIO exceedances, including afflux, time of inundation, velocity and hazard, were selected and hydraulic simulations conducted at these locations with a varying degree of mitigation applied. The outcomes of this assessment are detailed below. It should be noted the proposed measures below are only for the purpose of demonstrating the effectiveness of mitigation measures and will not necessarily constitute the preferred detailed design for these locations. As detailed above, a robust assessment of each mitigation option will be undertaken to consider FIO exceedance reduction as well as social, environmental and economic impacts.

Site selection

Four sites (of multiple culvert locations), as listed in Table 22.2, were selected along the Project alignment to demonstrate the effectiveness of proposed mitigation measures.

Table 22.2 Mitigation modelling—Site selection

Site ID	Culvert ID	FIO exceedances
1	C62.94 and C63.15	Afflux, velocity, hazard, time of inundation
2	C77.20, C77.47 and C77.77	Afflux, velocity, hazard, duration of inundation
3	C84.38 and C85.35	Afflux, velocity, hazard, duration of inundation
4	C119.74, C119.88, C120.08, C120.25 and C120.31	Velocity, hazard, duration of inundation

Mitigation option effectiveness assessment

Site 1—C62.94 and C63.15

Site 1 is located between Ch 62.9 km and Ch 63.2 km, and experiences afflux, velocity, duration of inundation and hazard exceedances against the FIOs. The proposed culvert structures and configurations for this site are as follows:

- C62.94—3 x 2.4 m x 1.8 m RCBC
- C63.15—9 x 1.8 m RCP.

Figure 22.3 demonstrates current FIO exceedances for afflux, velocity, duration of inundation and hazard (pre-mitigation) for all events up to and including the 1% AEP event.

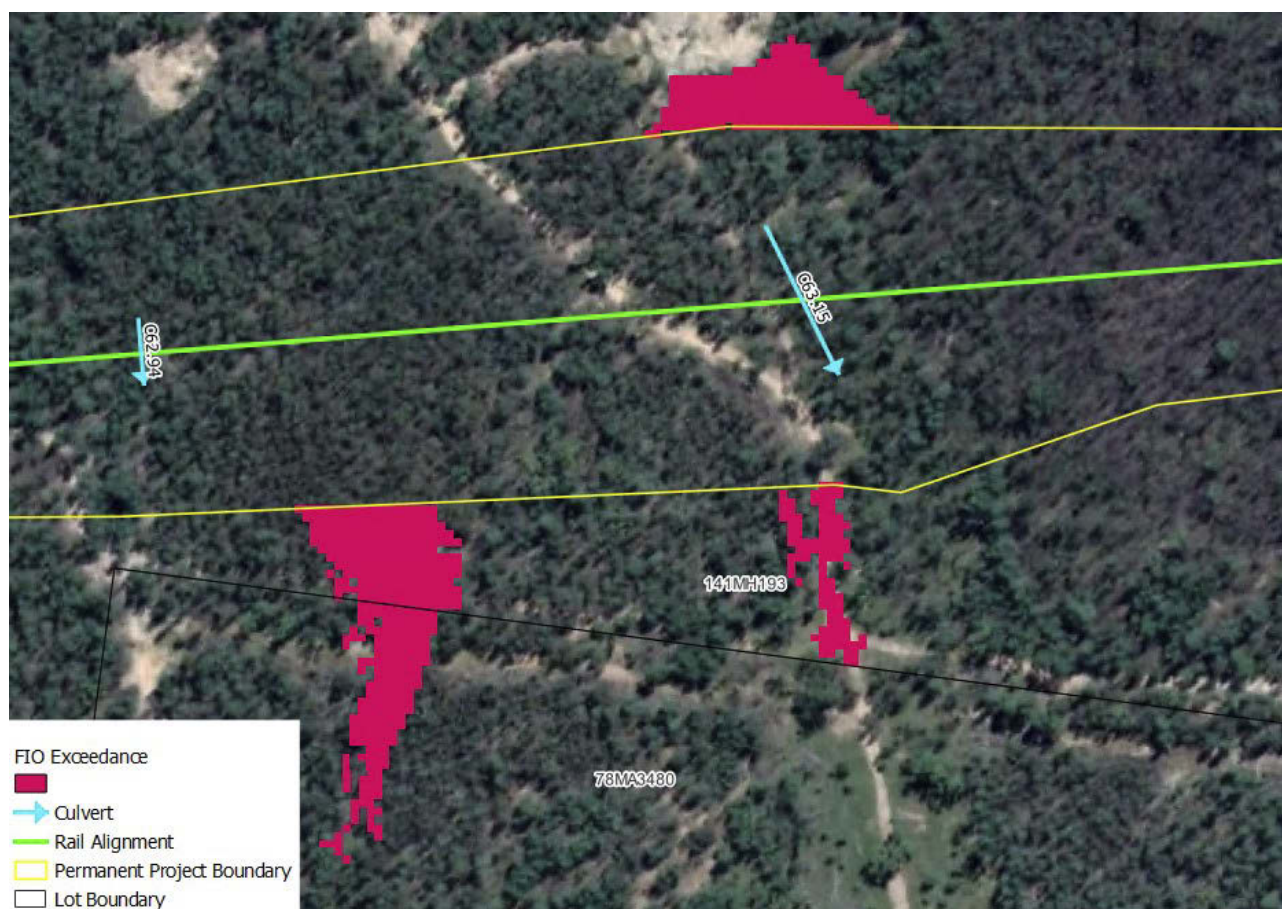


Figure 22.3 Site 1—Flood Impact Objective exceedances (pre-mitigation)

In order to mitigate these FIO exceedances, the following amendments to culvert structure configurations were adopted including:

- Increase in box culvert span
- Increase in culvert cell numbers and spread along the Project alignment
- Culvert inlet/outlet realignment to existing flowpaths.

Table 22.3 details culvert configurations pre- and post- mitigation, with Figure 22.4 demonstrating post-mitigation exceedance extents for afflux, velocity, duration of inundation and hazard for all events up to and including the 1% AEP event.

Table 22.3 Site 1 structure details—C62.94 and C63.15

Structure details	Pre-mitigation	Post-mitigation
C62.94	3 x 2.4 m x 1.8 m RCBC	C62.94a – 2 x 3.0 m x 1.8 m RCBC C62.94b – 2 x 3.0 m x 1.8 m RCBC
C63.15	9 x 1.8 m RCP	18 x 1.8 m RCP

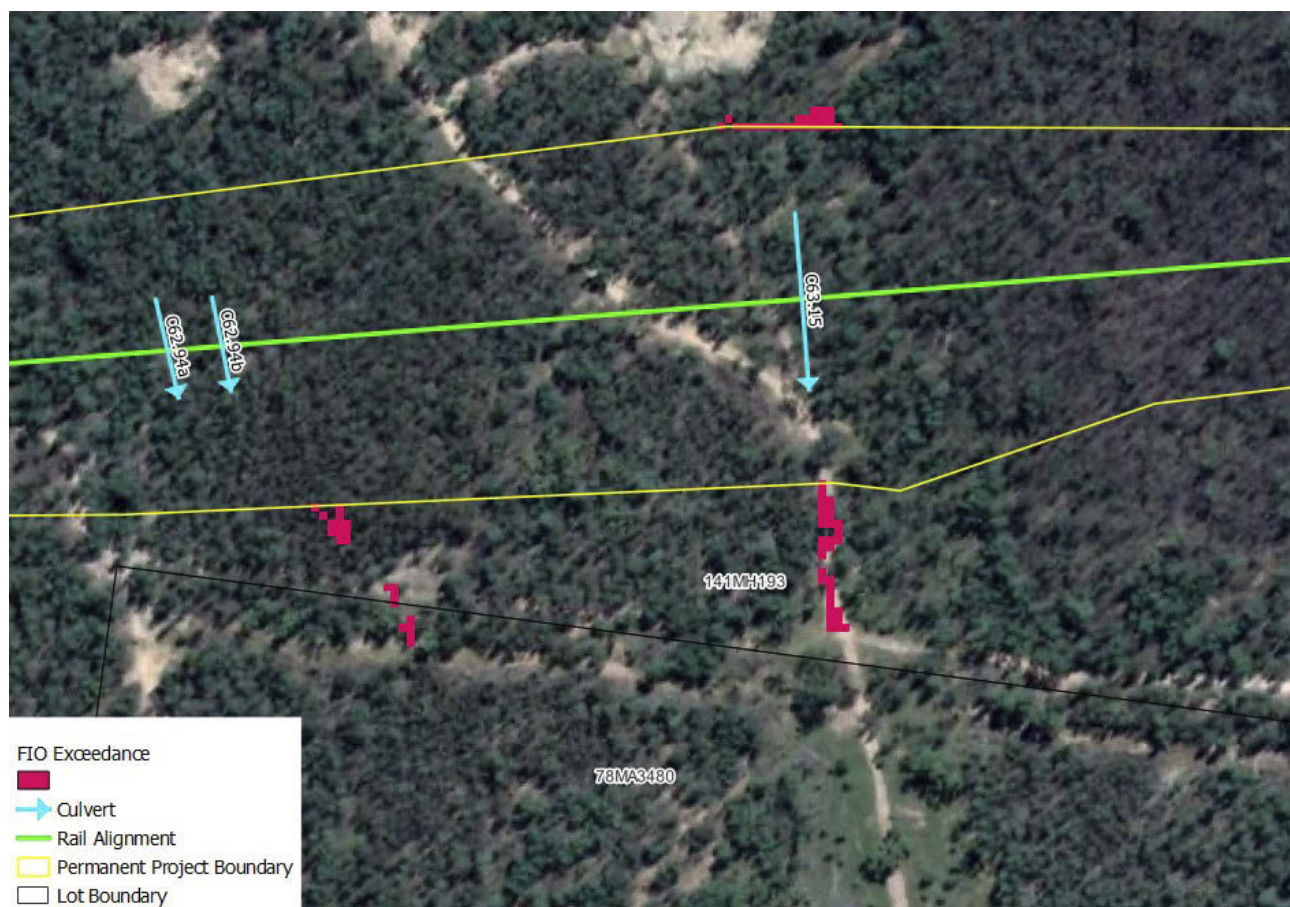


Figure 22.4 Site 1—Flood Impact Objective exceedance (post-mitigation)

To further quantify exceedances, Table 22.4, Table 22.5, Table 22.6 and Table 22.7 demonstrate the pre- and post- mitigation exceedances and impacted areas for afflux, change in duration of inundation, change in velocity and change in hazard.

Table 22.4 Site 1 pre- and post-mitigation—Afflux

Lot Plan number	Pre-mitigation		Post-mitigation	
	Maximum afflux (mm)	Impacted area (ha)	Maximum afflux (mm)	Impacted area (ha)
78MA3480	-	-	-	-
141MH193	344	0.183	39	0.015

Table 22.5 Site 1 pre- and post-mitigation—Change in time of inundation

Lot Plan number	Pre-mitigation		Post-mitigation	
	Maximum change in time of inundation (%)	Impacted area (ha)	Maximum change in time of inundation (%)	Impacted area (ha)
78MA3480	23.9	0.083	6	0.003
141MH193	21.1	0.106	10.1	0.023

Table 22.6 Site 1 pre- and post-mitigation—Change in velocity

Lot Plan number	Pre-mitigation		Post-mitigation	
	Maximum change in velocity (%)	Impacted area (ha)	Maximum change in velocity (%)	Impacted area (ha)
78MA3480	21.9	0.063	-	-
141MH193	22.3	0.058	-	-

Table 22.7 Site 1 pre- and post-mitigation—Change in hazard

Lot Plan number	Pre-mitigation		Post-mitigation	
	Maximum change in hazard (%)	Impacted area (ha)	Maximum change in hazard (%)	Impacted area (ha)
78MA3480	34.8	0.026	-	-
141MH193	17.3	0.016	-	-

Reviewing the above results, it can be seen through the application of mitigation measures, such as increasing culvert spans, increasing culvert cell numbers and spread and realignment of the inlet and outlet to existing flowpaths, FIO exceedances can be mitigated in this location.

Site 2—C77.20, C77.47 and C77.77

Site 2 is located between Ch 77.0 km and Ch 78.0 km and experiences afflux, velocity, duration of inundation and hazard exceedances against the FIOs. The proposed culvert structures and configurations for this site are as follows:

- C77.20—8 x 1.5 m RCP
- C77.47—5 x 0.9 m RCP
- C77.77—10 x 1.2 m x 1.2 m RCBC.

Figure 22.5 demonstrates current FIO exceedances for afflux, velocity, duration of inundation and hazard (pre-mitigation) for all events up to and including the 1% AEP event.



Figure 22.5 Site 2—Flood Impact Objective exceedances (pre-mitigation)

In order to mitigate FIO exceedances, the following amendments to culvert structure configurations were adopted, including:

- Increase in culvert barrel numbers and spread along the Project alignment
- Culvert inlet/outlet realignment to existing flowpaths
- Modification of culvert invert levels
- Inlet/outlet works to direct runoff through culvert structures.

Table 22.8 details culvert configurations pre- and post- mitigation, with Figure 22.6 demonstrating post-mitigation exceedance extents for afflux, velocity, duration of inundation and hazard for all events up to and including the 1% AEP event.

Table 22.8 Site 2 structure details—C77.20, C77.47 and C77.77

Structure details	Pre-mitigation	Post-mitigation
C77.20	8 x 1.5 m RCP	C77.20a – 5 x 1.5 m RCP C77.20b – 5 x 1.5 m RCP
C77.47	5 x 0.9 m RCP	C77.47a – 4 x 0.9 m RCP C77.47b – 4 x 0.9 m RCP C77.47c – 4 x 0.9 m RCP
C77.77	10 x 1.2 m x 1.2 m RCBC	C77.77a – 2 x 1.2 m x 1.2 m RCBC C77.77b – 2 x 1.2 m x 1.2 m RCBC C77.77c – 9 x 1.2 m x 1.2 m RCBC C77.77d – 7 x 1.2 m x 1.2 m RCBC



Figure 22.6 Site 2—Flood Impact Objective exceedances (post-mitigation)

To further quantify exceedances, Table 22.9, Table 22.10, Table 22.11 and Table 22.12 demonstrate the downstream pre- and post- mitigation exceedances and impacted areas for afflux, change in duration of inundation, change in velocity and change in hazard.

Table 22.9 Site 2 pre- and post-mitigation—Afflux

Lot Plan number	Pre-mitigation		Post-mitigation	
	Maximum afflux (mm)	Impacted area (ha)	Maximum afflux (mm)	Impacted area (ha)
102MH643	-	-	-	-
Millmerran-Inglewood Road Easement	372	0.322	254	0.133

Table 22.10 Site 2 pre- and post-mitigation—Change in time of inundation

Lot Plan number	Pre-mitigation		Post-mitigation	
	Maximum change in time of inundation (%)	Impacted area (ha)	Maximum change in time of inundation (%)	Impacted area (ha)
102MH643	48.8	0.354	73.2	0.214
Millmerran-Inglewood Road Easement	-	-	-	-

Table 22.11 Site 2 pre- and post-mitigation—Change in velocity

Lot Plan number	Pre-mitigation		Post-mitigation	
	Maximum change in velocity (%)	Impacted area (ha)	Maximum change in velocity (%)	Impacted area (ha)
102MH643	21.8	0.374	6.8	0.022
Millmerran-Inglewood Road Easement	72.3	0.016	65.7	0.013

Table 22.12 Site 2 pre- and post-mitigation—Change in hazard

Lot Plan number	Pre-mitigation		Post-mitigation	
	Maximum change in hazard (%)	Impacted area (ha)	Maximum change in hazard (%)	Impacted area (ha)
102MH643	177.4	0.048	177.4	0.034
Millmerran-Inglewood Road Easement	210.5	0.029	76.9	0.020

Similar to Site 1, reviewing the above results, it can be seen through the application of mitigation measures, such as increasing culvert cell numbers and spread, modification of culvert inverts, realignment of culvert inlets/outlets to existing flowpaths and performing inlet and outlet works to direct flow through culvert structures, FIO exceedances can be mitigated in this location.

Site 3—C84.38 and C85.35

Site 3 is located between Ch 84.20 km and Ch 85.40 km and experiences afflux, velocity, duration of inundation and hazard exceedances against the FIOs. The proposed culvert structures and configurations for this site are as follows:

- C84.38—30 x 2.4 m x 2.4 m RCBC
- C85.35—10 x 2.4 m x 0.9 m RCBC.

Figure 22.7 demonstrates current FIO exceedances for afflux, velocity, duration of inundation and hazard (pre-mitigation) for all events up to and including the 1% AEP event.

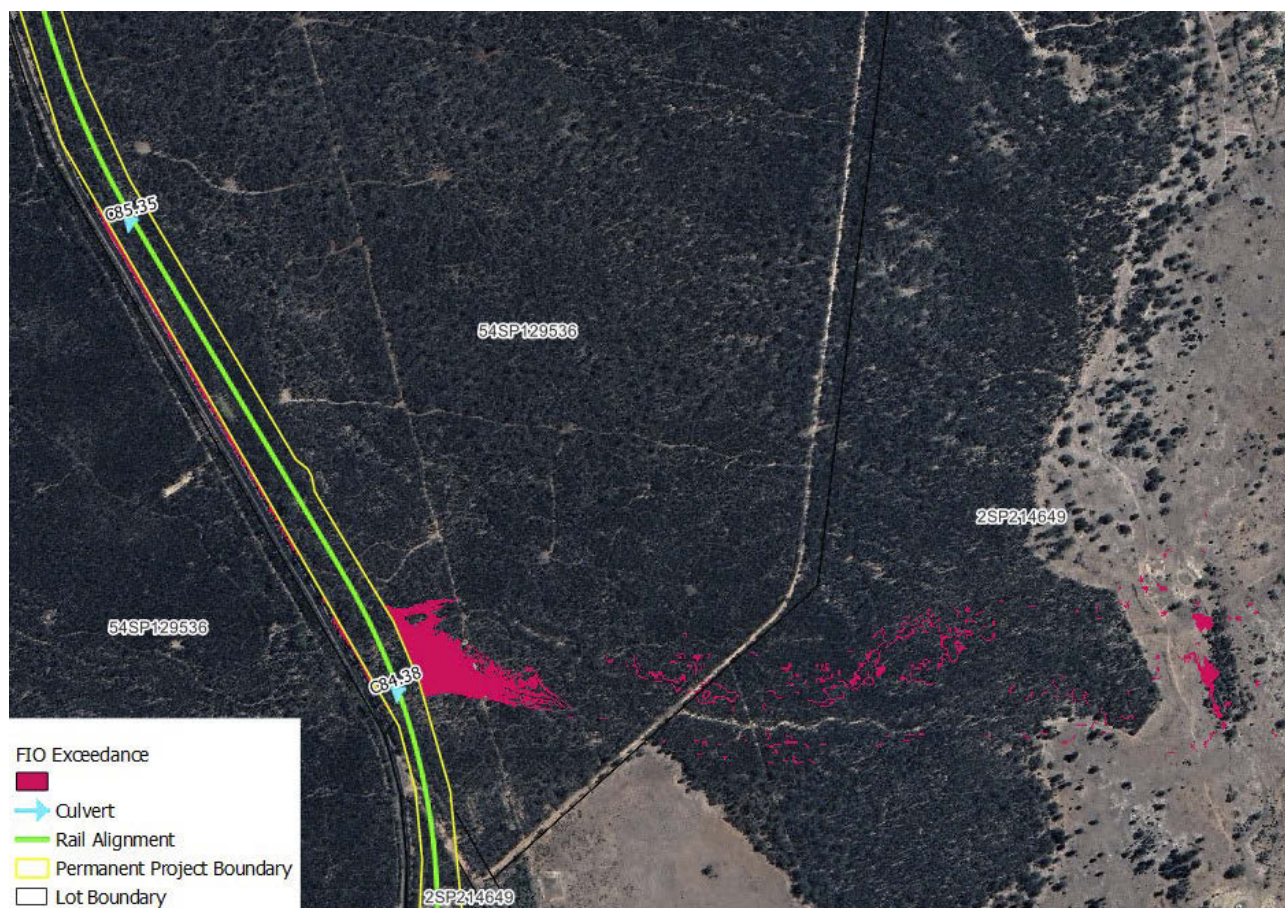


Figure 22.7 Site 3—Flood Impact Objective exceedances (pre-mitigation)

In order to mitigate FIO exceedances, the following amendments to culvert structure configurations were adopted, including:

- Increase in culvert barrel numbers and spread along the Project alignment
- Culvert inlet/outlet realignment to existing flowpaths
- Modification of culvert invert levels
- Inlet/outlet works to direct runoff to/from the dedicated culvert structure.

Table 22.13 details culvert configurations pre- and post- mitigation, with Figure 22.8 demonstrating post-mitigation exceedance extents for afflux, velocity, duration of inundation and hazard for all events up to and including the 1% AEP event. The reduced impacted area indicates that the proposed mitigation measures were effective in reducing FIO exceedances.

Table 22.13 Structure details—C84.38 and C85.35

Structure details	Pre-mitigation	Post-mitigation
C84.38	30 x 2.4 m x 2.4 m RCBC	C84.38a – 6 x 2.4 m x 2.4 m RCBC C84.38b – 6 x 2.4 m x 1.2 m RCBC C84.38c – 6 x 2.4 m x 1.2 m RCBC C84.38d – 6 x 2.4 m x 1.2 m RCBC C84.38e – 8 x 2.4 m x 1.2 m RCBC
C85.35	10 x 2.4 m x 0.9 m RCBC	C85.35a – 15 x 2.4 m x 0.9 m C85.35b – 5 x 2.4 m x 0.9 m

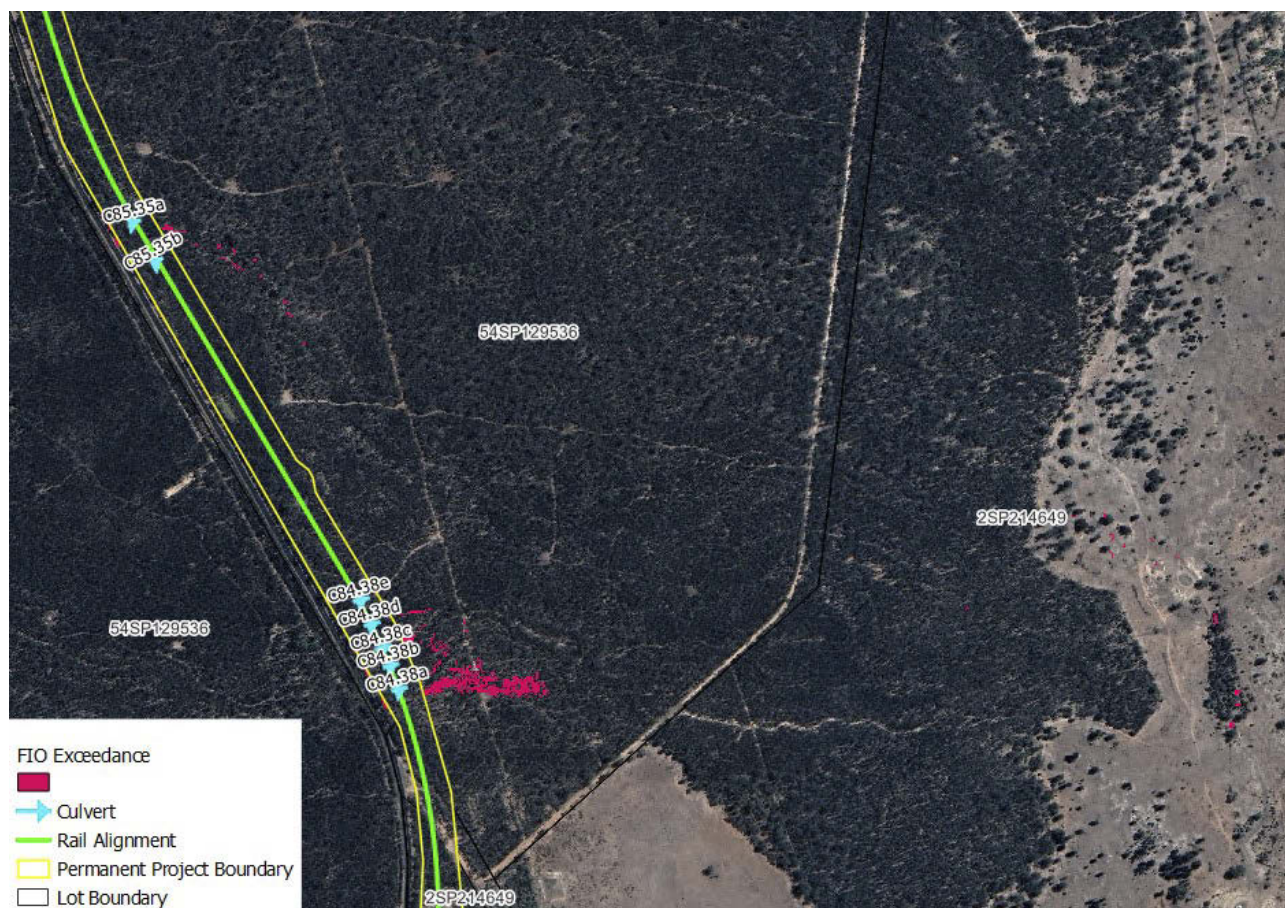


Figure 22.8 Site 3—Flood Impact Objective exceedances (post-mitigation)

To further quantify exceedances, Table 22.14, Table 22.15, Table 22.16 and Table 22.17 demonstrate the downstream pre- and post- mitigation exceedances and impacted areas for afflux, change in duration of inundation, change in velocity and change in hazard.

Table 22.14 Site 3 pre- and post-mitigation—Afflux

Lot Plan number	Pre-mitigation		Post-mitigation	
	Maximum afflux (mm)	Impacted area (ha)	Maximum afflux (mm)	Impacted area (ha)
54SP129536	-	-	-	-
2SP214649	-	-	-	-
Millmerran-Inglewood Road Easement	440	0.596	144	0.044

Table 22.15 Site 3 pre- and post-mitigation—Change in time of inundation

Lot Plan number	Pre-mitigation		Post-mitigation	
	Maximum change in time of inundation (%)	Impacted area (ha)	Maximum change in time of inundation (%)	Impacted area (ha)
54SP129536	127.7	0.587	88.4	0.514
2SP214649	71.1	0.052	70.6	0.008
Millmerran-Inglewood Road Easement	46.2	0.089	31.7	0.008

Table 22.16 Site 3 pre- and post-mitigation—Change in velocity

Lot Plan number	Pre-mitigation		Post-mitigation	
	Maximum change in velocity (%)	Impacted area (ha)	Maximum change in velocity (%)	Impacted area (ha)
54SP129536	33.4	1.234	36.5	0.133
2SP214649	46.7	0.353	22.8	0.034
Millmerran-Inglewood Road Easement	170.4	0.362	73.0	0.030

Table 22.17 Site 3 pre- and post-mitigation—Change in hazard

Lot Plan number	Pre-mitigation		Post-mitigation	
	Maximum change in hazard (%)	Impacted area (ha)	Maximum change in hazard (%)	Impacted area (ha)
54SP129536	169.6	0.845	79.4	0.039
2SP214649	-	-	-	-
Millmerran-Inglewood Road Easement	737	0.394	56	0.020

Reviewing the above results, it can be seen through the application of mitigation measures, such as increasing culvert cell numbers and spread, modification of culvert inverts, realignment of culvert inlets/outlets to existing flowpaths and performing inlet and outlet works to direct flow through culvert structures, FIO exceedances can be mitigated in this location.

Site 4—C120.25 to C120.31

Site 4 is located between Ch 119.60 km and Ch 120.40 km, and experiences velocity, duration of inundation and hazard exceedances against the FIOs. The proposed culvert structures and configurations for this site are as follows:

- C119.75—2 x 2.4 m x 2.4 m RCBC
- C119.88—8 x 1.5 m x 1.2 m RCBC
- C120.08—5 x 1.5 m x 0.9 m RCBC
- C120.25—26 x 2.4 m x 1.5 m RCBC
- C120.31—3 x 2.4 m x 1.5 m RCBC.

Figure 22.9 demonstrates current FIO exceedances for velocity, duration of inundation and hazard (pre-mitigation) for all events up to and including the 1% AEP event.



Figure 22.9 Site 4—Flood Impact Objective exceedances (pre-mitigation)

In order to mitigate FIO exceedances, the following amendments to culvert structure configurations were adopted, including:

- Increase in culvert barrel numbers and spread along the Project alignment
- Culvert inlet/outlet realignment to existing flowpaths
- Modification of culvert invert levels
- Inlet/outlet works to direct runoff to/from the dedicated culvert structure.

Table 22.18 details culvert configurations pre- and post- mitigation, with Figure 22.10 demonstrating post-mitigation exceedance extents for velocity, duration of inundation and hazard for all events up to and including the 1% AEP event. The reduced impacted area indicates that the proposed mitigation measures were effective in reducing FIO exceedances.

Table 22.18 Structure details—C119.75, C119.88, C120.08, C120.25 and C120.31

Structure details	Pre-mitigation	Post-mitigation
C119.75	2 x 2.4 m x 1.2 m RCBC	C119.75a – 2 x 2.4 m x 1.2 m RCBC C119.75b – 2 x 2.4 m x 1.2 m RCBC
C119.88	8 x 1.5 m x 1.2 m RCBC	C119.88a – 4 x 1.5 m x 1.2 m RCBC C119.88b – 6 x 1.5 m x 1.2 m RCBC C119.88c – 4 x 1.5 m x 1.2 m RCBC
C120.08	5 x 1.5 m x 0.9 m RCBC	C120.08 – 5 x 1.5 m x 0.9 m RCBC
C120.25	26 x 2.4 m x 1.5 m RCBC	C120.25a – 6 x 2.4 m x 1.5 m RCBC C120.25b – 11 x 2.4 m x 1.5 m RCBC C120.25c – 12 x 2.4 m x 1.5 m RCBC C120.25d – 10 x 2.4 m x 1.5 m RCBC

Structure details	Pre-mitigation	Post-mitigation
C120.31	3 x 2.4 m x 1.5 m RCBC	C120.31a – 3 x 2.4 m x 1.5 m RCBC C120.31b – 3 x 2.4 m x 1.5 m RCBC



Figure 22.10 Site 4—Flood Impact Objective exceedances (post-mitigation)

To further quantify exceedances, Table 22.19, Table 22.20 and Table 22.21 demonstrate the downstream pre- and post- mitigation exceedances and impacted areas for change in duration of inundation, change in velocity and change in hazard.

Table 22.19 Site 4 pre- and post-mitigation—Change in time of inundation

Lot Plan number	Pre-mitigation		Post-mitigation	
	Maximum change in time of inundation (%)	Impacted area (ha)	Maximum change in time of inundation (%)	Impacted area (ha)
94DY180	79	0.045	41.6	0.029
3SP166689	209.5	4.149	59.9	2.951

Table 22.20 Site 4 pre- and post-mitigation—Change in velocity

Lot Plan number	Pre-mitigation		Post-mitigation	
	Maximum change in velocity (%)	Impacted area (ha)	Maximum change in velocity (%)	Impacted area (ha)
94DY180	21.9	0.009	51.1	0.024
3SP166689	92.1	1.122	22.2	0.463

Table 22.21 Site 4 pre- and post-mitigation—Change in hazard

Lot Plan number	Pre-mitigation		Post-mitigation	
	Maximum change in hazard (%)	Impacted area (ha)	Maximum change in hazard (%)	Impacted area (ha)
94DY180	274.4	0.045	44.7	0.016
3SP166689	284	0.429	33.3	0.006

Reviewing the above results, it can be seen through the application of mitigation measures, such as increasing culvert cell numbers and spread, modification of culvert inverts, realignment of culvert inlets/outlets to existing flowpaths and performing inlet and outlet works to direct flow through culvert structures, FIO exceedances can be mitigated in this location.

22.4.1.4 Summary

Mitigation of FIO exceedances is an important step to reducing impacts on property, infrastructure and FSRs due to the introduction of the revised reference design. During the future stages of design, modifications will be made to the Project that will require reassessment to ensure compliance with the Project FIOs. As part of this assessment, mitigation measures will be explored and implemented into the Project's revised reference design, to reduce FIO exceedances. These will include (but not be limited to):

- Culvert location and spread
- Culvert grade and invert levels
- Culvert inlet/outlet realignment to existing flowpaths
- A reduction or increase in culvert cell numbers.

In order to demonstrate the effectiveness of these mitigation measures, several representative culvert locations were selected across the Project alignment that experience a varying range of exceedances, with simulations performed pre- and post- mitigation implementation to determine the reduction in exceedances. It was found that through the introduction of the above measures, significant reduction in FIO exceedances both in extent and magnitude resulted, demonstrating the effectiveness of the mitigation measures. During future stages of design, FIO exceedances that are determined to occur will be explored further and measures such as though demonstrated above, implemented to further mitigate these exceedances.

Mitigation of potential flood impacts will be resolved primarily through engineering solutions. Alternatively, property solutions or alternative arrangements would be negotiated between ARTC and the impacted landowner. Should, through the proposed mitigation, there be a predicted impact on third-party infrastructure, consultation will occur with the infrastructure owner. Should the mitigation trigger any other planning or legislative requirement, ARTC will consult with the relevant State agency or local government as required. Further details can be found below in Sections 22.4.2 and 22.4.3.

22.4.2 At-property treatments



22.4.2.1 Residual exceedances

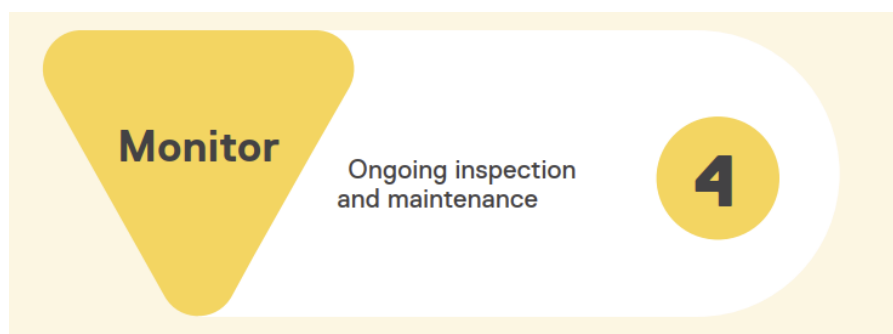
It is expected that through the implementation of the above mitigation measures, and refinement of the design in the detailed design stage, that some FIO exceedances detailed in this report will be reduced or removed. It needs to be acknowledged that not all FIO exceedances will be able to be mitigated through engineering design. Due to the presence of the Project, some constriction, expansion and redirection of flow will result due to embankment and cross-drainage locations resulting in remnant FIO exceedances.

All endeavours will be made during the detailed design stage to mitigate and reduce FIO exceedances by modification of the Project's revised reference design using measures detailed above. In the instance an FIO exceedance remains owing to certain Project/design constraints, post detailed design mitigation, engagement with impacted landowners will be undertaken to discuss the residual exceedances, the impact of the exceedance on their property and potential further site-specific mitigation measures. Site-specific mitigation measures will be individual to each landowner and agreed via a consultation process. Where material exceedance of FIOs is predicted, notwithstanding feasible and practicable engineering mitigations, ARTC will consult with affected landowners in good faith to determine potential impacts and mitigation. Where ARTC is not able to reach agreement with the affected landowner and further mitigation is not cost-effective or practicable, but where a material residual impact remains, ARTC will consider appropriate forms of alternative dispute resolution processes including the opportunity for mediation.

22.4.2.2 Dispute resolution of unresolved matters

The Project will continue to incorporate environmental management and design features to ensure that indirect flooding impacts are managed and mitigated to the extent practicable, based on a case-by-case, risk-based approach. ARTC will consult with relevant stakeholders in good faith, guided by advice from qualified and experienced personnel. In situations where impacted landowners and ARTC cannot reach agreement on sufficient mitigation or a mutually accepted outcome regarding flood impacts above the target FIO levels, ARTC will consider alternative dispute resolution processes, including mediation. The selection of the appropriate dispute resolution mechanism will depend on the specific circumstances and will be negotiated between landowners and ARTC.

22.4.3 Monitor



22.4.3.1 Baseline scour survey

Prior to any construction works, a baseline survey will first be undertaken at cross-drainage structures and any other relevant stormwater-related infrastructure, to confirm the occurrence of, or pre-existence of, scour. The future occurrence (or worsening) of scour associated with the pre-construction, early works and construction works, and commissioning and operations of the Project will be evident in comparison with the baseline survey. Equally, the pre-existence of scour in advance of any construction or operational works will also be evident in reviewing the baseline survey. This is critical to accurately determine whether the Project has potentially been a factor in causing or exacerbating scour, or not.

The baseline survey shall meet the following requirements:

- Topographic data is to be collated to represent the pre-existing/natural terrain profile in advance of the pre-construction, early works, and construction works and commissioning and operational works stages. The resolution of this data shall be sufficient to determine appreciable changes to the terrain profile that could be caused by scour effects. The resolution will be determined in consultation with ARTC and the survey service provider.
- Collation of aerial imagery to depict the pre-existing/natural terrain land usage and landform in advance of the pre-construction, early works and construction works; and commissioning and operations works. The resolution of this imagery shall be sufficient to determine appreciable changes to the land usage and landform that could be caused by scour effects. The resolution will be determined in consultation with the ARTC and the survey service provider.
- The width and lateral extent of the data listed in the previous two data points will include the permanent rail corridor and any temporary construction works zones and corridors. The minimum extent of this data upstream and downstream of the structures will be confirmed during the detailed design stage. This will typically extend into adjacent land, which may be private land, or land owned by other entities including ARTC. The land may also contain infrastructure operated by other asset owners and/or managers.
- Accordingly, the approximate width and lateral extent of the baseline topographic and aerial survey would vary, thus changing the overall width and lateral extent of the baseline topographic and aerial survey along the Project route. An exception to this is that where any pre-existing and/or continuous erosion exists, the width and lateral extent of the baseline survey shall encompass the full extent of this pre-existing and/or continuous erosion.
- The baseline survey data shall be collated, managed, and maintained by ARTC.

22.4.3.2 Pre-construction, early works and construction works mitigation and monitoring

During the pre-construction, early works and construction works stage of the Project, the Project's final design will be in varying stages of construction, involve temporary works, and be susceptible to flooding without the above mitigation measures implemented. As such during this stage of works, additional mitigation measures may be required to minimise the impact to property, infrastructure, and FSRs. The following mitigation measures may be implemented during the pre-construction, early works and construction works stage to mitigate scour and erosion at cross-drainage structures and other relevant stormwater-related infrastructure:

- Establishment of a baseline of the surrounding land and environment via detailed survey, as detailed in Section 22.4.2.
- Completion of FRA for sites located within the 1% AEP flood extent within the temporary footprint that are deemed to be high risk based on their proximity to FSRs, including:
 - Prior to construction, to determine any potential changes in flood risk as a result of staged construction activities that occur within the 1% AEP event floodplain.
 - The FRA will include frequent design storms (i.e. 2 Exceedances per Year (EY), 1 EY, 39% AEP (1 in 2 year ARI), 20% AEP and 10% AEP event) given the temporary and short-term nature of construction activities. Larger (rarer) flood events will be considered on a case-by-case basis, as deemed necessary and where the risk profile of the proposed construction works would warrant it. Where there are interfaces between other Inland Rail projects, flood risk assessments will be undertaken in consultation with other projects to ensure consideration of whole of Inland Rail impacts are considered rather than the Project in isolation.

- As detailed in Section 4.2, the designated FIOs are design and operationally focused, and not considered appropriate for the pre-construction, early works and construction works stage due to the temporary nature of work. Pre-construction, early works and construction works will be managed by the following criteria:
 - Minimise re-direction of flood water flow
 - No adverse increases to flood levels at FSRs
 - Maintain existing velocities where practical
 - Erosion and sediment control is managed in accordance with the Construction ESCP.
- The FRAs will be used to inform appropriate site controls and documented as part of a Flood Risk Management Plan.
- The FRA will be a dynamic document and will be updated as construction progresses to reflect the staged nature of construction activities.
- Maintaining conveyance through existing drainage lines, where possible, to minimise diversion of flows to adjacent land
- Implementation of Erosion and Sediment Control measures (as per the CEMP) to adequately manage scour and sedimentation on surrounding lands
- Monitoring for scour at cross-drainage structures (and any other relevant stormwater-related infrastructure) during the pre-construction, early works and construction works stage shall be undertaken on the following periodic basis:
 - Following rainfall events that have generated significant flow through either permanent or temporary cross-drainage structures (or at any other relevant stormwater-related infrastructure)
 - Frequency is to be determined by ARTC during the detailed design stage.
- The pre-construction, early works and construction works stage monitoring process may involve the collation of geo-referenced photographs and images, to be evaluated by a suitably qualified engineer (or scientist with sufficient qualifications and experience), under the direction of the Environmental Monitor⁴:
 - The engineer may request additional data to help in the evaluation process if they deem it necessary (e.g. additional localised topographic and/or bathymetric survey, aerial imagery, etc.).
- Alternatively, the monitoring process may make use of technology to assist, or it may be a combination of both physical inspection and technology. This will be confirmed during the detailed design stage, following further consideration and investigation by ARTC.
- Where clear evidence of scour is observed as per the monitoring data, this shall be contrasted against the baseline survey data to confirm whether it is (a) pre-existing scour with no observed change, (b) pre-existing scour that has been clearly exacerbated, or (c) newly generated scour.
- If the findings point to (b) or (c) above, the engineer shall document this in a report of their findings and recommend mitigation/remediation measures that shall be implemented onsite as soon as practicable. This may include, but is not limited to:
 - Terrain remediation and/or stabilisation, including vegetation and/or revegetation strategies, or the introduction of other natural forms of energy dissipation
 - Dumped rock (riprap) (i.e. flexible erosion protection measures)
 - Gabion baskets and reno-mattresses (i.e. semi-flexible erosion protection measures).

⁴ The responsibility of the Environmental Monitor is to monitor the compliance of the Project to the CEMP and its associated sub-plans. Further details of the role and responsibilities of the Environmental Monitor can be found in EIS Chapter 24: Draft Outline Environmental Management Plan (Section 24.2).

- Consultation with affected landowners, asset owners, or stakeholders may be warranted as part of the mitigation and remediation works, depending on the nature and extent of (i) the works and (ii) any scour being experienced. This will ensure that community concerns are understood, and impacts rectified in consultation with the landowner.
- This pre-construction, early works and construction works stage monitoring data, and all subsequent findings, recommendations, design of works, etc. shall be collated, managed, and maintained by ARTC.

Ongoing consultation will occur with surrounding landowners during this stage of works to ensure community concerns are understood and any impacts to adjacent property determined to be due to the Project, rectified in consultation with the landowner.

Construction activities in defined watercourses

In some instances, it will be necessary to undertake construction works in defined watercourses. During the detailed design stage, construction activities around watercourses will be reviewed to identify any opportunities to avoid the disturbance of natural flow paths and surrounding banks. As this will not always be achievable, the following mitigation measures will be implemented during the pre-construction, early works and construction works stage in order to minimise impacts on watercourses due to Project works:

- In-stream works will also be undertaken in accordance with the requirements of the *Riverine Protection Permit Exemption Requirements* (DRDMW, 2023)
- Disturbance areas will be minimised, and work zones delineated to ensure areas of disturbance are limited
- Appropriate erosion and sediment controls will be implemented and monitored in accordance with the ESCP
- Excavation will be undertaken in a manner to prevent erosion or slip, working faces shall be limited to safe height and slopes, with surfaces drained to avoid ponding and erosion
- Reinstatement, stabilisation and rehabilitation of disturbed areas will be undertaken progressively and in accordance with the site Rehabilitation and Landscaping Management Plan
- Stockpiling of material will be undertaken away from defined watercourses in accordance with the ESCP
- In the event of wet weather, the following additional controls would be implemented:
 - Contractors subscribed to BoM flood warning notifications
 - Protection of exposed bank areas by geofabrics or similar
 - Removal of any construction waste and equipment away from potential areas of inundation and watercourses.

22.4.3.3 Commissioning and operations monitoring

During the operations stage of the Project, ongoing monitoring will occur to ensure flood impacts, including scour and erosion, due to the Project are minimised. Inspections and monitoring will be undertaken in accordance with the outline pre-construction, early works and construction works, and commissioning and operations stage “Erosion Monitoring and Mitigation Plan” as detailed in EIS Chapter 24: Draft Outline Environmental Management Plan. The following monitoring programs may be implemented:

- Scheduled waterway and drainage system inspections (frequency is to be determined by ARTC during the detailed design stage) pre-commencement of operations
- It is expected that the commissioning and operations stage monitoring process will involve technology and physical inspection

- Owing to the scale and distance of the Inland Rail alignment, along with the number of watercourses it crosses, technology will be used to optimise the effort and generate increased efficiency, reliability, and accuracy in its results. Technological advancements being considered to assist in the process include:
 - Drones for survey and aerial imagery
 - Satellite topographic measurement which can be done on a near-continuous basis to extremely high levels of accuracy
 - Machine learning and artificial intelligence to evaluate and compare large datasets to identify discrepancies and/or differences that could be scour, erosion and/or sedimentation related
- Physical inspection may be implemented in any defined high-risk or sensitive areas, or where it might be deemed preferable and more suitable
- Physical inspection would also be implemented where the technology-based process identified any discrepancies and differences that could be scour and erosion-related. This will be done in order to validate the technology-based monitoring outcomes, and to gain an onsite appreciation of potential scour issues and suitable mitigation measures.
- Where clear evidence of scour is observed as per the monitoring data, this shall be contrasted against the baseline survey data to confirm whether it is (a) pre-existing scour with no observed change, (b) pre-existing scour that has been clearly exacerbated, or (c) newly generated scour
- If the findings point to (b) or (c) above, the engineer shall document this in a report of their findings and recommend mitigation and/or remediation measures to ensure the integrity of the drainage system is reinstated, and any impact outside the permanent footprint rectified as soon as practicable and in consultation with the relevant landowner/asset manager, as discussed in Section 22.4.3.4.
- This commissioning and operations stage monitoring data, and all subsequent findings, recommendations, design of works, etc, shall be collated, managed, and maintained by ARTC.

22.4.3.4 Commissioning and operations remediation and mitigation of scour

Where clear evidence of scour is observed, this shall be contrasted against the baseline survey data to confirm whether it is (a) pre-existing scour with no observed change, (b) pre-existing scour that has been clearly exacerbated, or (c) newly generated scour.

If the findings point to (b) or (c) above, the engineer shall document this in a report of their findings and recommend mitigation and/or remediation measures that shall be implemented as soon as practicable.

This may include, but is not limited to, the following, listed in an approximate hierarchy of preference (notwithstanding the fact that any specific scour situation will have its own unique characteristics that will typically necessitate a bespoke mitigation solution to be developed):

- Terrain remediation and stabilisation, including vegetation and/or revegetation strategies, or the introduction of other natural forms of energy dissipation
- Dumped rock (riprap) (i.e. flexible erosion protection measures)
- Gabion baskets and reno-mattresses (i.e. semi-flexible erosion protection measures)
- Other bespoke semi-flexible erosion protection measures (engineered/manufactured product-based solutions)
- Baffles at culvert outlet
- Other more heavy-duty reinforced concrete energy dissipation measures
- Non-preferable solutions, but which could be considered if required:
 - negotiated and agreed property-based solutions (e.g. easements, acquisition, etc.).

Consultation with affected landowners, asset owners, or stakeholders shall be undertaken as part of the mitigation and/or remediation works, if the works could in any way affect or impact the landowners, asset owners, or stakeholders, or if the scour impacts associated with the Inland Rail infrastructure are already impacting landowners, asset owners, or stakeholders.

Information pertaining to the mitigation of scour and erosion impacts (including design information, records of consultation etc.) shall be collated, managed, and maintained by ARTC.

Where defects are identified during inspections, adequate rectification works will be determined to ensure the integrity of the drainage system is reinstated and any impact outside the permanent footprint rectified in consultation with the relevant landowner/asset manager.

23 Limitations

FFJV has prepared this report in accordance with the usual diligence and thoroughness of the consulting profession with reference to current standards, procedures and practices.

This report should be read in full and no excerpts are to be taken as representative of the findings. No responsibility is accepted by FFJV for use of any part of this report in any other context.

This report was prepared for the exclusive use of the Project. FFJV accepts no liability or responsibility whatsoever for, any use of, or reliance upon, this report by any third party.

This report was prepared based on information available at the time of writing. The models detailed in this report are based on LiDAR survey taken generally in 2015 (or as detailed in each catchment section). Therefore, any development or topographical change occurring within the catchment after the surveys taken is not included in this investigation, unless directly specified.

There are a number of limitations that apply to the modelling to date, some of which include:

- Stakeholder engagement will continue during detailed design, pre-construction, early works and construction works, and commissioning and operations stages; as such, proposed impacts and structural solutions still need to be confirmed with relevant stakeholders. Modelling may need to be updated as a result of any ongoing stakeholder engagement.

ARR 2019 outlines several fundamental themes that are also particularly relevant to this investigation:

- All models are coarse simplifications of very complex processes. No model can therefore be perfect, and no model can represent all of the important processes accurately:
 - Model accuracy and reliability will always be limited by the accuracy of the terrain and other input data
 - Model accuracy and reliability will always be limited by the reliability/uncertainty of the inflow data
 - No model is 'correct' therefore the results require interpretation
 - A model developed for a specific purpose is probably unsuitable for another purpose without modification, adjustment, and recalibration; the responsibility must always remain with the modeller to determine whether the model is suitable for a given problem
 - Recognition that no two flood events behave in exactly the same manner
 - Design floods are a best estimate of an 'average' flood for their probability of occurrence.

The interpretation of results and other presentations in this report should be done with an appreciation of any limitations in their accuracy, as noted above. In particular, all model results need to be interpreted with a level of engineering judgment, and this is particularly relevant to the application of the FIOs. It is common for models to show localised impacts that are not realistic and that are referred to as model artefacts or model 'noise'. This type of impact should be disregarded and can include isolated impacts far removed from the project corridor or marginal exceedances around the flood fringe.

Unless otherwise stated, presentations in this report are based on peak values of water surface level, flow, depth and velocity. Therefore, using water levels as an example, the peak level does not occur everywhere at the same time and, therefore, the values presented are based on taking the maximum value that occurred at each computational point in the model during the entire flood event. Hence, a presentation of peak water levels does not represent an instantaneous point in time but rather an envelope of the maximum values that occurred at each computational point over the duration of the flood event.

24 Conclusions

The key objectives of the Hydrology and Flooding Technical Report are to provide information on the data investigation, hydrology and hydraulic calibration, impact assessment and mitigation and to provide comment on the performance on the Project's revised reference design. This report outlines the methodology followed, the outcomes of this investigation and the assessment of the Project's revised reference design.

There are several major waterways within the vicinity of the Project, with the key waterways being the Macintyre River, Macintyre Brook, the Condamine River and Gowrie Creek. Other major creek crossings include Pariagara Creek, Cattle Creek, Native Dog Creek, Bringalily Creek, Nicol Creek, Back Creek and Westbrook Creek. The Project alignment crosses 18 major waterways (stream order ≥ 3) and 81 minor waterways (stream order <3).

Hydrologic and hydraulic modelling was undertaken for each of these catchments, with the models calibrated to multiple historical events using stream gauges records and anecdotal data where available. Based on this performance, the hydrologic and hydraulic models were considered validated and appropriate to use to assess the potential impacts associated with the Project.

Design event hydrology was developed using the calibrated hydrologic models using ARR 2019 flood flow estimation techniques. The hydraulic models were run for a suite of design events from the 20% AEP event to the 1 in 10,000 AEP and PMF events. The flows and levels predicted by the hydrologic and hydraulic models were compared to the results of an FFA at stream gauges within each catchment as well as results from previous flood studies.

Modelling of the current state of development (Existing Case) was undertaken and details of the existing flood regime were determined for the modelled design events. The proposed works associated with the Project were incorporated into the hydraulic models to form the Developed Case. Assessment of the potential impacts upon the existing flood regime was undertaken and refinement of the Project revised reference design was undertaken to mitigate impacts.

Consultation with stakeholders, including landowners, was undertaken at key stages including validation of the performance of the modelling in replicating experienced historical flood events and presentation of the Project design outcomes and impacts on properties and infrastructure.

The Project design was guided and refined using hydraulic design criteria Table 4.1 and FIOs (Table 4.2). The resulting design outcomes relative to the hydraulic design criteria are detailed in Table 24.1.

Table 24.1 Project hydraulic design criteria outcomes

Performance criteria	Design outcomes
Flood immunity	Rail line – 1% AEP flood immunity with minimum of 300 mm freeboard to formation level has been achieved.
Hydraulic analysis and design	Hydrologic and hydraulic analysis and design has been undertaken using ARR 2019 and state/local government guidelines. The Project revised reference design includes significant rail drainage structures under the Project alignment to convey flood flows on floodplains and minimise impacts under the full range of design events, being: ■ 28 rail bridges ■ 123 rail RCBC banks ■ 214 rail RCP banks ■ Inclusion of road drainage structures under local roads adjacent to the Project alignment.

Performance criteria	Design outcomes
Scour protection of structures	Culvert scour protection has been designed in accordance with <i>Guide to Road Design Part 5B: Drainage</i> (Austroads, 2013). Scour protection was specified where the culvert outlet velocities for the 1% AEP event exceeded the allowable soil velocities shown in Table 3.1 of <i>Guide to Road Design Part 5B: Drainage</i> (Austroads, 2013). Where a velocity FIO exceedance was determined at a culvert outlet, additional scour protection was included within the design to mitigate the risk of erosion. Required lengths of scour protection were determined and are predicted to fit within the proposed permanent footprint, except at three locations. A conservative scour estimation has been undertaken at each bridge site based on available information and will be refined during detailed design.
Structural design	1 in 2,000 AEP event has been modelled with details used for bridge design purposes.
Extreme events	Overtopping of the Project alignment under extreme events occurs at limited locations being: ■ Gowrie Creek – Above formation level at drainage structure C207.15 in the 10,000 AEP event, and above top of rail level at structure C207.15 and C208.21 for PMF event ■ Westbrook and Dry Creeks – Above formation and rail levels at drainage structure 310-BR31 in the 2,000 AEP event, above formation and top of rail in several locations in the 10,000 AEP and PMF events ■ Condamine River – Above formation at several locations in 10,000 AEP and PMF events; above top of rail at several locations in a PMF event ■ Back Creek – No overtopping predicted at cross-drainage structures. Some overtopping is predicted at other locations in the catchment where cross-drainage structures aren't proposed ■ Nicol Creek – Above formation at cross-drainage location C104.94 in the 10,000 AEP event and above formation and top of rail at multiple locations in the PMF event ■ Bringalily Creek – Above formation and top of rail in several locations for the 10,000 AEP and PMF events ■ Native Dog Creek – Above formation at drainage structure C94.91 and C95.07 for the PMF events. Other areas along the Project alignment also overtop where cross-drainage structures are not proposed in the 10,000 AEP and PMF events ■ Cattle Creek – Above formation and top of rail at drainage structure C87.37 for the PMF events ■ Pariagara Creek – Above formation and top of rail at multiple locations for the PMF events ■ Macintyre Brook – Above formation and top of rail at multiple locations for the PMF events ■ Macintyre River – Above formation and top of rail at drainage structures C21.26, C31.32, 270-BR12, C31.87, C32.00, C32.55 and 270-BR13 for the 10,000 AEP and PMF event.
Flood flow distribution	Structures were located along the Project alignment to maintain existing flood conveyance and spread of floodwaters.
Sensitivity testing	The risk to the Project's revised reference design from climate change and blockage has been assessed in accordance with ARR 2019. Key outcomes are: ■ The Project design (to the Top of Formation) maintains 1% AEP flood immunity under 2090 climate change conditions ■ Based on ARR 2019, a blockage factor of 25 per cent has been applied to culverts and no blockage factor has been applied to bridges ■ Varying the level of blockage to culverts between 0% and 50% does not impact upon the Project's revised reference design

Flood Impact Objectives, as presented in Table 4.2, were established and used to guide the Project's revised reference design, including mitigation of impacts through refinement of the hydraulic design, including adjustment of the numbers, dimensions and location of major drainage structures. Table 24.2 summarises how the Project's revised reference design performs against each of the FIOs.

Table 24.2 Flood Impact Objectives and outcomes

Parameter	Objectives and outcomes					
Change in peak water levels	Existing habitable ² and/or commercial and industrial buildings/premises (e.g. dwellings, schools, hospitals, shops) and sensitive infrastructure	Yards or gardens of residential or commercial/ industrial properties/lots (excluding habitable dwellings/ buildings)	Existing non-habitable structures (e.g. agricultural sheds, pump-houses)	Existing local roads currently in use Existing rail lines	State-Controlled Roads	Agricultural and grazing land/forest areas and other non-agricultural land
	≤ 10 mm	≤ 50 mm	≤ 100 mm	10 to 100 mm	10 to 20 mm	≤ 200 mm with localised areas up to 400 mm
	Objective: Changes in peak water levels are to be assessed against the above proposed limits. Outcome: Generally, the Project's revised reference design meets the above limits with number of localised areas along the Project alignment where these limits are exceeded. These areas are generally on agricultural land. FSRs that are impacted by changes in peak water levels under the 1% AEP event that exceed the FIOs include: 21 dwellings (6 between Pampas and Yandilla, 14 at Yelarbon and Kurumbul, one at Pittsworth) 2 sheds at Pampas, 1 shed at Yarranlea and 10 sheds at Yelarbon and Kurumbul 4 commercial buildings (grain silos) at Yandilla and 1 Stockyard near Kurumbul A review of property and road exceedances demonstrates that some afflux occurs across multiple properties, existing roads and rail infrastructure across the Project alignment.					
Change in time of inundation	Objective: Identify changes to time of inundation by determining time of submergence in Existing and Developed Cases. Assess impacts against the following objectives for habitable floors: - Where existing flood inundation is less than 1 hour – up to 1 hour duration of inundation - Where existing flood inundation of 1 hour or more occurs – up to a 5% increase in duration of inundation For impacted roads/rail, the duration of inundation can increase by up to 10%, subject to the determination of the time of submergence and consideration of impacts on accessibility/egress during flood events in consultation with the relevant authority. Assess impacts against the following objectives for all other land uses: - Where existing flood inundation is less than 1 hour – up to 1 hour duration of inundation - Where existing flood inundation of 1 hour or more occurs – up to a 10% increase in duration of inundation The duration performance targets do not apply to newly flooded land, where compliant with afflux criteria.					
	Outcome: Time of inundation is the predominant exceedance across the Project alignment on property, roads and rail. The presence of the Project results in some minor changes of flow behaviour from the Existing Case scenario, resulting in an increase in time of inundation. In instances where the Project is removing the existing rail line, the upgrade of cross-drainage infrastructure results in increased conveyance across the alignment and increase in time of inundation in these areas.					
	Objective: Aim to minimise changes in natural flow patterns and minimise changes to flood-flow distribution across floodplain areas. This includes the objective of maintaining drainage paths that are conveying runoff from adjoining terrain, minor watercourses, and gullies, to avoid ponding of water and/or excessive duration of inundation. Identify any changes and justify acceptability of changes by assessing the risk with a focus on land use and FSR. The identification of changes to flow distribution is to include the consideration of a range of floods, from small (frequent) events (for conditions throughout the event) to large and extreme (infrequent) events. This exercise will be undertaken to identify (and where deemed necessary, mitigate) any increased risk to FSR associated with flow distribution changes.					
Flood flow distribution	Outcome: The Project has minimal impacts on flood flows and floodplain conveyance/storage with significant floodplain structures included to maintain the existing flood regime. Some change occurs directly adjacent cross-drainage structures and abutments due to the constriction of flow through these areas.					
Velocities	Objective: Maintain existing velocities where practical or minimise increases in velocities. Identify changes to velocities and impacts on external properties.					

Parameter	Objectives and outcomes
	The ETV for natural ground surfaces should be established from a site-specific assessment by a suitably qualified specialist, and in consideration of engineering guidelines. For sealed surfaces, this same approach could be applied, or through reference to suitable guidelines/specifications. Assess against the following performance objectives: ■ Sealed surfaces (or surfaces otherwise protected against erosion): – For existing velocities equal to or greater than 1 m/s (or the defined ETV), the increase in velocity is limited to 20% – For existing velocities (or velocities associated with new flowpaths) less than 1 m/s, the maximum design velocity is 1.2 m/s (or the defined ETV) ■ Natural ground surfaces including watercourses, agricultural land, unimproved grazing land and other unsealed or unprotected areas: – For existing velocities equal to or greater than 0.5 m/s (or the defined ETV), the increase in velocity is limited to 10% (or within an acceptable range as determined by geomorphological assessment) – For existing velocities (or velocities associated with new flowpaths) less than 0.5 m/s (or the defined ETV), the maximum design velocity is 0.5 m/s (or the defined ETV). Outcome: In general, changes in velocities occur directly adjacent to the Project alignment, around cross-drainage structures. Some exceedances appear to be large but are generally attributed to an increase in flood extent and newly flood-affected cells with adjacent cells having similar high velocities. Scour protection has been specified where the outlet velocities for the 1% AEP event exceed the allowable soil velocities for the particular soil type for each location, which was identified from published soil mapping and erosive threshold assessment completed as part of the revised draft EIS. Additional scour protection measures have been provided at culvert outlets where FIO exceedances have been identified.
Hazard	Objective: Assess against the following objectives: Roads/rail, urban and commercial areas, dwellings: ■ A 10% increase in velocity x depth product. Other land: ■ A 20% increase in velocity x depth product. (In both cases where the velocity x depth product is below 0.15 square metres per second (m²/s) in the Developed⁶ Case, no percentage change performance targets apply) Outcome: In general, changes in hazard occur directly adjacent to the Project alignment (in association with increased velocity), around cross-drainage structures. Some exceedances appear to be large percentage changes and should be read in conjunction with Existing and Design Case results. The FIOs require that change be presented in percentage change and can sometimes produce large increases when Existing Case results are significantly small. Some locations were identified to experience significant percentage increases; however, the “absolute” difference between Existing and Design results is relatively small.
Extreme event risk management	Objective: Consider risks posed to neighbouring properties and emergency access/egress for events larger than the 1% AEP event to minimise unexpected or unacceptable impacts. At sites with existing sensitive infrastructure, uses involving vulnerable people and/or any critical road network that was designed to be immune to flooding in any extreme event, the objective for increase in peak water level under the 0.2% AEP (1 in 500) is 10 mm. At existing habitable dwellings and/or commercial and industrial buildings/premises under the 0.05% (1 in 2,000) AEP event, a maximum increase in peak water level of 250 mm applies. Outcome: A review of impacts under the 1 in 2,000 AEP, 1 in 10,000 AEP and PMF events has been undertaken with the existing flood depths and increase in peak water levels at FSRs identified on each floodplain. Considering the flood depths that occur, particularly under the PMF event, indicates that the changes in peak water levels would be unlikely to exacerbate flood conditions during extreme events. An overtopping failure assessment has been completed as part of the revised draft EIS and a Population at Risk assessment completed.
Sensitivity testing	Objective: Consider risks posed by climate change and blockage in accordance with ARR 2019. Undertake assessment of impacts associated with Project alignment for both scenarios

Parameter	Objectives and outcomes
	Outcomes: Climate change—climate change has been assessed in accordance with ARR 2019 requirements with the representative concentration pathway 8.5 (2090 horizon) scenario adopted. The impacts resulting from changes in peak water levels under the 1% AEP event with climate change are generally similar to those seen under the 1% AEP event, with some additional impacts on FSRs. Blockage—Blockage of drainage structures has been assessed in accordance with ARR 2019 requirements. The blockage assessment resulted in no blockage factor being applied to bridges and a blockage factor of 25% being applied to culverts. Two blockage sensitivity scenarios were tested with both 0% and 50% blockage of all culverts assessed. The resulting changes in peak water levels associated with the Project alignment are localised but impact on some FSRs.

A comprehensive consultation exercise was undertaken to provide the community with detailed information and certainty around the flood modelling and the Project design. In future stages, ARTC will:

- Continue to work with landowners concerned with hydrology and flooding throughout the detailed design, pre-construction, early works and construction, and commissioning and operations stages of the Project.
- Continue to work with directly impacted landowners affected by the Project alignment throughout the detailed design, pre-construction, early works and construction, and commissioning and operations stages of the Project.
- Continue to work with local councils and State government departments throughout the detailed design, pre-construction, early works and construction, and commissioning and operations stages of the Project.

In order to mitigate impacts due to the Project, a Flood and Geomorphic Impacts Mitigation Framework has been developed that will be implemented throughout the detailed design, pre-construction, early works and construction, and commissioning and operations stages. The four- step process includes:

1. Design development and refinement
2. Design treatments
3. At-property treatments
4. Monitor.

Ongoing consultation with impacted stakeholders will be undertaken throughout this four-step process as detailed above.

25 References

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APPENDIX T1

Hydrology and Flooding Technical Report—Volume 1

Appendix A Independent International Panel of Experts for Flood Studies—Issues management register

BORDER TO GOWRIE REVISED DRAFT ENVIRONMENTAL IMPACT STATEMENT

[illegible]

Item No.	Model Name	Issue	Impact on Design	Level of Importance	Action Required	Comment	Status	Action Required	Comment	Impact	Action Required	RFM Response
S2G.B.8	Brangly Creek	Minor differences between URBES output hydrographs for TUFLOW model and input hydrographs for TUFLOW model.	likely to be negligible - note only for further design.	Low	To be addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Minor differences between URBES output hydrographs and input hydrographs for TUFLOW model to be addressed in Detailed Design.	To be addressed in Detailed Design. Refer to section 3.8 of Appendix 11 - Flooding and Hydrology Technical Report.	No change noted to URBES model and differences remain. As a minor issue, can be left to Detailed Design.	Open Action required during detailed design.	To be achieved during detailed design.
S2G.B.9	Brangly Creek	Location of riffles applied to the hydraulic model are inconsistent. The location of riffles applied to the hydraulic model are inconsistent. The location of riffles applied to the hydraulic model are inconsistent.	Inconsistent application of the riffle location will impact on the relevant timing of the hydrographs from the respective sub-catchments and total inflows.	High	Updated response has been provided in updated Technical Notes, with updated text highlighted.	Detailed Design to adopt the revised riffle boundaries. Further assessment and justification of impacts in Detailed Design and subject to acceptance of revised FIO is an acceptable outcome.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Detailed Design to adopt the revised riffle boundaries. Further assessment and justification of impacts in Detailed Design and subject to acceptance of revised FIO is an acceptable outcome.	To be addressed in Detailed Design. Refer to section 3.8 of Appendix 11 - Flooding and Hydrology Technical Report.	No changes to sub-catchment riffles as part of the updates, leaving revisions to Detailed Design phase.	Open Action required during detailed design.	To be achieved during detailed design.
S2G.B.10	Brangly Creek	No known applied to bridge "opening". Bridge loss coefficients not calibrated or verified against previous methods.	Larger/reducer bridge structures could be required to minimise impacts to satisfy verified against previous methods.	Low	To be addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Detailed Design to confirm reasonableness of bridge losses using alternate approaches.	To be addressed in Detailed Design. Refer to section 3.8 of Appendix 11 - Flooding and Hydrology Technical Report.	No changes to bridge representation as part of the updates, leaving revisions to Detailed Design phase.	Open Action required during detailed design.	To be achieved during detailed design.
S2G.B.11	Brangly Creek	Level of detail in Technical Report.	Without additional detail, there are uncertainties in relation to the level of detail of the modelling and its appropriateness.	Very High	Agreed with Flood Panel this item would be closed.	The Panel agrees that the response is acceptable.	Closed	N/A	N/A	Additional detail has been provided. Further detail in terms of items such as bridge losses etc to be provided at Detailed Design phase. Details regarding calibration to Macdonald Brook FFA no longer required per the M20 B.9.	Open Action required during detailed design.	To be achieved during detailed design.
S2G.B.12	Brangly Creek	Calculation for flood level impacts and changes to time of submergence.	If an increase is observed to be excessive then the design will need to be modified.	Very High	Comment added to updated Technical Note to provide updated impact mapping and discussion around impacts in revised Draft ES. Update text has been highlighted in technical notes.	Updated impact mapping will be presented in the revised Draft ES, including an updated summary and discussion regarding PC exceedances and proposed mitigation.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Updated impact mapping to be presented in the revised Draft ES, including an updated summary and discussion regarding PC exceedances and proposed mitigation.	Refer to section 3.8 of Appendix 11 - Flooding and Hydrology Technical Report. Impact assessment has been updated to capture new flood impact objectives targets. Updated mapping has been provided in Appendix 11 of Appendix 11 - Flooding and Hydrology Technical Report - Volume 1 and in the Digital Platform.	Updated information has been provided in relation to the exceedances and their quantum for each FIO. Justification of the exceedances and consideration of the potential for mitigation is required.	Open Action required during detailed design.	Item 3.12 of Appendix 11 - Flooding and Hydrology Technical Report - Volume 1 has been updated with additional justification and mitigation against high risk state controlled roads as identified by the Flood Advisory Panel as well as those identified in the Flood Advisory Report dated 25 September 2023
S2G.B.13	Brangly Creek	Intensity of flood risk.	May need to adjust design if the intensity of the flood is reduced.	Medium	Responded to in Technical Note.	The additional information provided is considered sufficient to justify minimal impacts on the road would have been preferable to get the results for a range of events and not just the 1% but based on the results for the 1% event probably acceptable for the lower events.	Closed	N/A	N/A	Updated information has been provided in relation to the exceedances and their quantum for each FIO. Justification of the exceedances and consideration of the potential for mitigation is required.	Open Action required during detailed design.	Item 3.12 of Appendix 11 - Flooding and Hydrology Technical Report - Volume 1 has been updated with additional justification and mitigation against high risk state controlled roads as identified by the Flood Advisory Panel as well as those identified in the Flood Advisory Report dated 25 September 2023
S2G.B.14	Byerra Road	The substation delineation is not accurate in the south-western portion of the catchment.	Possibly a small impact on peak flow rates resulting in the south-western portion of the catchment.	Low	To be addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Substation delineation to be reviewed in Detailed Design. Refer to section 3.8 of Appendix 11 - Flooding and Hydrology Technical Report.	No Panel accepts the commitment to address the issue in Detailed Design.	Open Action required during detailed design.	To be achieved during detailed design.	
S2G.B.15	Byerra Road	The substation delineation is not accurate in the south-western portion of the catchment.	Possibly a small impact on peak flow rates resulting in the south-western portion of the catchment.	Low	To be addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Substation delineation to be reviewed in Detailed Design. Refer to section 3.8 of Appendix 11 - Flooding and Hydrology Technical Report.	No Panel accepts the commitment to address the issue in Detailed Design.	Closed. No further action.		
S2G.B.16	Byerra Road	The substation delineation is not accurate in the south-western portion of the catchment.	Possibly a small impact on peak flow rates resulting in the south-western portion of the catchment.	Low	To be addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Substation delineation to be reviewed in Detailed Design. Refer to section 3.8 of Appendix 11 - Flooding and Hydrology Technical Report.	No Panel accepts the commitment to address the issue in Detailed Design.	Closed. No further action.		
S2G.B.17	Byerra Road	The substation delineation is not accurate in the south-western portion of the catchment.	Possibly a small impact on peak flow rates resulting in the south-western portion of the catchment.	High	Responded to in Technical Note.	Substation assessment undertaken for substation boundary location as requested with very little change to modelled impacts. Updated substation boundary to be adopted in revised Draft ES. Sensitivity assessment also undertaken for regional 20 AEP flood event with very little change to modelled impacts.	Closed	N/A	N/A	Substation boundary of the hydraulic model has been updated for the revised draft ES. Impact assessment has been updated to capture new flood impact objectives targets. Updated mapping has been provided in Appendix 11 of Appendix 11 - Flooding and Hydrology Technical Report - Volume 2.	Open Action required during detailed design.	Item 3.12 of Appendix 11 - Flooding and Hydrology Technical Report - Volume 1 has been updated with additional justification and mitigation against high risk state controlled roads as identified by the Flood Advisory Panel as well as those identified in the Flood Advisory Report dated 25 September 2023
S2G.B.18	Byerra Road	The substation delineation is not accurate in the south-western portion of the catchment.	Possibly a small impact on peak flow rates resulting in the south-western portion of the catchment.	Medium	To be addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Substation delineation to be reviewed in Detailed Design. Refer to section 3.8 of Appendix 11 - Flooding and Hydrology Technical Report.	No Panel accepts the commitment to address the issue in Detailed Design.	Closed. No further action.		
S2G.B.19	Byerra Road	The substation delineation is not accurate in the south-western portion of the catchment.	Possibly a small impact on peak flow rates resulting in the south-western portion of the catchment.	Low	To be addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Substation delineation to be reviewed in Detailed Design. Refer to section 3.8 of Appendix 11 - Flooding and Hydrology Technical Report.	No Panel accepts the commitment to address the issue in Detailed Design.	Closed. No further action.		
S2G.B.20	Byerra Road	The substation delineation is not accurate in the south-western portion of the catchment.	Possibly a small impact on peak flow rates resulting in the south-western portion of the catchment.	Low	To be addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Substation delineation to be reviewed in Detailed Design. Refer to section 3.8 of Appendix 11 - Flooding and Hydrology Technical Report.	No Panel accepts the commitment to address the issue in Detailed Design.	Closed. No further action.		
S2G.B.21	Byerra Road	The substation delineation is not accurate in the south-western portion of the catchment.	Possibly a small impact on peak flow rates resulting in the south-western portion of the catchment.	Low	To be addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Substation delineation to be reviewed in Detailed Design. Refer to section 3.8 of Appendix 11 - Flooding and Hydrology Technical Report.	No Panel accepts the commitment to address the issue in Detailed Design.	Open Action required during detailed design.	To be achieved during detailed design.	
S2G.B.22	Byerra Road	The substation delineation is not accurate in the south-western portion of the catchment.	Possibly a small impact on peak flow rates resulting in the south-western portion of the catchment.	Low	To be addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Substation delineation to be reviewed in Detailed Design. Refer to section 3.8 of Appendix 11 - Flooding and Hydrology Technical Report.	No Panel accepts the commitment to address the issue in Detailed Design.	Open Action required during detailed design.	To be achieved during detailed design.	
S2G.B.23	Byerra Road	The substation delineation is not accurate in the south-western portion of the catchment.	Possibly a small impact on peak flow rates resulting in the south-western portion of the catchment.	Low	To be addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Substation delineation to be reviewed in Detailed Design. Refer to section 3.8 of Appendix 11 - Flooding and Hydrology Technical Report.	No Panel accepts the commitment to address the issue in Detailed Design.	Open Action required during detailed design.	To be achieved during detailed design.	
S2G.B.24	Byerra Road	The substation delineation is not accurate in the south-western portion of the catchment.	Possibly a small impact on peak flow rates resulting in the south-western portion of the catchment.	Low	To be addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Substation delineation to be reviewed in Detailed Design. Refer to section 3.8 of Appendix 11 - Flooding and Hydrology Technical Report.	No Panel accepts the commitment to address the issue in Detailed Design.	Open Action required during detailed design.	To be achieved during detailed design.	
S2G.B.25	Byerra Road	The substation delineation is not accurate in the south-western portion of the catchment.	Possibly a small impact on peak flow rates resulting in the south-western portion of the catchment.	Low	To be addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Substation delineation to be reviewed in Detailed Design. Refer to section 3.8 of Appendix 11 - Flooding and Hydrology Technical Report.	No Panel accepts the commitment to address the issue in Detailed Design.	Open Action required during detailed design.	To be achieved during detailed design.	
S2G.B.26	Byerra Road	The substation delineation is not accurate in the south-western portion of the catchment.	Possibly a small impact on peak flow rates resulting in the south-western portion of the catchment.	Low	To be addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Substation delineation to be reviewed in Detailed Design. Refer to section 3.8 of Appendix 11 - Flooding and Hydrology Technical Report.	No Panel accepts the commitment to address the issue in Detailed Design.	Open Action required during detailed design.	To be achieved during detailed design.	
S2G.B.27	Byerra Road	The substation delineation is not accurate in the south-western portion of the catchment.	Possibly a small impact on peak flow rates resulting in the south-western portion of the catchment.	Low	To be addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Substation delineation to be reviewed in Detailed Design. Refer to section 3.8 of Appendix 11 - Flooding and Hydrology Technical Report.	No Panel accepts the commitment to address the issue in Detailed Design.	Open Action required during detailed design.	To be achieved during detailed design.	
S2G.B.28	Byerra Road	The substation delineation is not accurate in the south-western portion of the catchment.	Possibly a small impact on peak flow rates resulting in the south-western portion of the catchment.	Low	To be addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Substation delineation to be reviewed in Detailed Design. Refer to section 3.8 of Appendix 11 - Flooding and Hydrology Technical Report.	No Panel accepts the commitment to address the issue in Detailed Design.	Open Action required during detailed design.	To be achieved during detailed design.	
S2G.B.29	Byerra Road	The substation delineation is not accurate in the south-western portion of the catchment.	Possibly a small impact on peak flow rates resulting in the south-western portion of the catchment.	Low	To be addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Substation delineation to be reviewed in Detailed Design. Refer to section 3.8 of Appendix 11 - Flooding and Hydrology Technical Report.	No Panel accepts the commitment to address the issue in Detailed Design.	Open Action required during detailed design.	To be achieved during detailed design.	
S2G.B.30	Byerra Road	The substation delineation is not accurate in the south-western portion of the catchment.	Possibly a small impact on peak flow rates resulting in the south-western portion of the catchment.	Low	To be addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Substation delineation to be reviewed in Detailed Design. Refer to section 3.8 of Appendix 11 - Flooding and Hydrology Technical Report.	No Panel accepts the commitment to address the issue in Detailed Design.	Open Action required during detailed design.	To be achieved during detailed design.	
S2G.B.31	Byerra Road	The substation delineation is not accurate in the south-western portion of the catchment.	Possibly a small impact on peak flow rates resulting in the south-western portion of the catchment.	Low	To be addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Substation delineation to be reviewed in Detailed Design. Refer to section 3.8 of Appendix 11 - Flooding and Hydrology Technical Report.	No Panel accepts the commitment to address the issue in Detailed Design.	Open Action required during detailed design.	To be achieved during detailed design.	
S2G.B.32	Byerra Road	The substation delineation is not accurate in the south-western portion of the catchment.	Possibly a small impact on peak flow rates resulting in the south-western portion of the catchment.	Low	To be addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Substation delineation to be reviewed in Detailed Design. Refer to section 3.8 of Appendix 11 - Flooding and Hydrology Technical Report.	No Panel accepts the commitment to address the issue in Detailed Design.	Open Action required during detailed design.	To be achieved during detailed design.	
S2G.B.33	Byerra Road	The substation delineation is not accurate in the south-western portion of the catchment.	Possibly a small impact on peak flow rates resulting in the south-western portion of the catchment.	Low	To be addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Substation delineation to be reviewed in Detailed Design. Refer to section 3.8 of Appendix 11 - Flooding and Hydrology Technical Report.	No Panel accepts the commitment to address the issue in Detailed Design.	Open Action required during detailed design.	To be achieved during detailed design.	
S2G.B.34	Byerra Road	The substation delineation is not accurate in the south-western portion of the catchment.	Possibly a small impact on peak flow rates resulting in the south-western portion of the catchment.	Low	To be addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Substation delineation to be reviewed in Detailed Design. Refer to section 3.8 of Appendix 11 - Flooding and Hydrology Technical Report.	No Panel accepts the commitment to address the issue in Detailed Design.	Open Action required during detailed design.	To be achieved during detailed design.	
S2G.B.35	Byerra Road	The substation delineation is not accurate in the south-western portion of the catchment.	Possibly a small impact on peak flow rates resulting in the south-western portion of the catchment.	Low	To be addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Substation delineation to be reviewed in Detailed Design. Refer to section 3.8 of Appendix 11 - Flooding and Hydrology Technical Report.	No Panel accepts the commitment to address the issue in Detailed Design.	Open Action required during detailed design.	To be achieved during detailed design.	
S2G.B.36	Byerra Road	The substation delineation is not accurate in the south-western portion of the catchment.	Possibly a small impact on peak flow rates resulting in the south-western portion of the catchment.	Low	To be addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Substation delineation to be reviewed in Detailed Design. Refer to section 3.8 of Appendix 11 - Flooding and Hydrology Technical Report.	No Panel accepts the commitment to address the issue in Detailed Design.	Open Action required during detailed design.	To be achieved during detailed design.	
S2G.B.37	Byerra Road	The substation delineation is not accurate in the south-western portion of the catchment.	Possibly a small impact on peak flow rates resulting in the south-western portion of the catchment.	Low	To be addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Substation delineation to be reviewed in Detailed Design. Refer to section 3.8 of Appendix 11 - Flooding and Hydrology Technical Report.	No Panel accepts the commitment to address the issue in Detailed Design.	Open Action required during detailed design.	To be achieved during detailed design.	
S2G.B.38	Byerra Road	The substation delineation is not accurate in the south-western portion of the catchment.	Possibly a small impact on peak flow rates resulting in the south-western portion of the catchment.	Low	To be addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Substation delineation to be reviewed in Detailed Design. Refer to section 3.8 of Appendix 11 - Flooding and Hydrology Technical Report.	No Panel accepts the commitment to address the issue in Detailed Design.	Open Action required during detailed design.	To be achieved during detailed design.	
S2G.B.39	Byerra Road	The substation delineation is not accurate in the south-western portion of the catchment.	Possibly a small impact on peak flow rates resulting in the south-western portion of the catchment.	Low	To be addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Substation delineation to be reviewed in Detailed Design. Refer to section 3.8 of Appendix 11 - Flooding and Hydrology Technical Report.	No Panel accepts the commitment to address the issue in Detailed Design.	Open Action required during detailed design.	To be achieved during detailed design.	
S2G.B.40	Byerra Road	The substation delineation is not accurate in the south-western portion of the catchment.	Possibly a small impact on peak flow rates resulting in the south-western portion of the catchment.	Low	To be addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Substation delineation to be reviewed in Detailed Design. Refer to section 3.8 of Appendix 11 - Flooding and Hydrology Technical Report.	No Panel accepts the commitment to address the issue in Detailed Design.	Open Action required during detailed design.	To be achieved during detailed design.	
S2G.B.41	Byerra Road	The substation delineation is not accurate in the south-western portion of the catchment.	Possibly a small impact on peak flow rates resulting in the south-western portion of the catchment.	Low	To be addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Substation delineation to be reviewed in Detailed Design. Refer to section 3.8 of Appendix 11 - Flooding and Hydrology Technical Report.	No Panel accepts the commitment to address the issue in Detailed Design.	Open Action required during detailed design.	To be achieved during detailed design.	
S2G.B.42	Byerra Road	The substation delineation is not accurate in the south-western portion of the catchment.	Possibly a small impact on peak flow rates resulting in the south-western portion of the catchment.	Low	To be addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Substation delineation to be reviewed in Detailed Design. Refer to section 3.8 of Appendix 11 - Flooding and Hydrology Technical Report.	No Panel accepts the commitment to address the issue in Detailed Design.	Open Action required during detailed design.	To be achieved during detailed design.	
S2G.B.43	Byerra Road	The substation delineation is not accurate in the south-western portion of the catchment.	Possibly a small impact on peak flow rates resulting in the south-western portion of the catchment.	Low	To be addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Substation delineation to be reviewed in Detailed Design. Refer to section 3.8 of Appendix 11 - Flooding and Hydrology Technical Report.	No Panel accepts the commitment to address the issue in Detailed Design.	Open Action required during detailed design.	To be achieved during detailed design.	
S2G.B.44	Byerra Road	The substation delineation is not accurate in the south-western portion of the catchment.	Possibly a small impact on peak flow rates resulting in the south-western portion of the catchment.	Low	To be addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Substation delineation to be reviewed in Detailed Design. Refer to section 3.8 of Appendix 11 - Flooding and Hydrology Technical Report.	No Panel accepts the commitment to address the issue in Detailed Design.	Open Action required during detailed design.	To be achieved during detailed design.	
S2G.B.45	Byerra Road	The substation delineation is not accurate in the south-western portion of the catchment.	Possibly a small impact on peak flow rates resulting in the south-western portion of the catchment.	Low	To be addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Substation delineation to be reviewed in Detailed Design. Refer to section 3.8 of Appendix 11 - Flooding and Hydrology Technical Report.	No Panel accepts the commitment to address the issue in Detailed Design.	Open Action required during detailed design.	To be achieved during detailed design.	
S2G.B.46	Byerra Road	The substation delineation is not accurate in the south-western portion of the catchment.	Possibly a small impact on peak flow rates resulting in the south-western portion of the catchment.	Low	To be addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Substation delineation to be reviewed in Detailed Design. Refer to section 3.8 of Appendix 11 - Flooding and Hydrology Technical Report.	No Panel accepts the commitment to address the issue in Detailed Design.	Open Action required during detailed design.	To be achieved during detailed design.	
S2G.B.47	Byerra Road	The substation delineation is not accurate in the south-western portion of the catchment.	Possibly a small impact on peak flow rates resulting in the south-western portion of the catchment.	Low	To be addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Substation delineation to be reviewed in Detailed Design. Refer to section 3.8 of Appendix 11 - Flooding and Hydrology Technical Report.	No Panel accepts the commitment to address the issue in Detailed Design.	Open Action required during detailed design.	To be achieved during detailed design.	
S2G.B.48	Byerra Road	The substation delineation is not accurate in the south-western portion of the catchment.	Possibly a small impact on peak flow rates resulting in the south-western portion of the catchment.	Low	To be addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Substation delineation to be reviewed in Detailed Design. Refer to section 3.8 of Appendix 11 - Flooding and Hydrology Technical Report.	No Panel accepts the commitment to address the issue in Detailed Design.	Open Action required during detailed design.	To be achieved during detailed design.	
S2G.B.49	Byerra Road	The substation delineation is not accurate in the south-western portion of the catchment.	Possibly a small impact on peak flow rates resulting in the south-western portion of the catchment.	Low	To be addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Substation delineation to be reviewed in Detailed Design. Refer to section 3.8 of Appendix 11 - Flooding and Hydrology Technical Report.	No Panel accepts the commitment to address the issue in Detailed Design.	Open Action required during detailed design.	To be achieved during detailed design.	
S2G.B.50	Byerra Road	The substation delineation is not accurate in the south-western portion of the catchment.	Possibly a small impact on peak flow rates resulting in the south-western portion of the catchment.	Low	To be addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Substation delineation to be reviewed in Detailed Design. Refer to section 3.8 of Appendix 11 - Flooding and Hydrology Technical Report.	No Panel accepts the commitment to address the issue in Detailed Design.	Open Action required during detailed design.	To be achieved during detailed design.	
S2G.B.51	Byerra Road	The substation delineation is not accurate in the south-western portion of the catchment.	Possibly a small impact on peak flow rates resulting in the south-western portion of the catchment.	Low	To be addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Substation delineation to be reviewed in Detailed Design. Refer to section 3.8 of Appendix 11 - Flooding and Hydrology Technical Report.	No Panel accepts the commitment to address the issue in Detailed Design.	Open Action required during detailed design.	To be achieved during detailed design.	
S2G.B.52	Byerra Road	The substation delineation is not accurate in the south-western portion of the catchment.	Possibly a small impact on peak flow rates resulting in the south-western portion of the catchment.	Low	To be addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Substation delineation to be reviewed in Detailed Design. Refer to section 3.8 of Appendix 11 - Flooding and Hydrology Technical Report.	No Panel accepts the commitment to address the issue in Detailed Design.	Open Action required during detailed design.	To be achieved during detailed design.	
S2G.B.53	Byerra Road	The substation delineation is not accurate in the south-western portion of the catchment.	Possibly a small impact on peak flow rates resulting in the south-western portion of the catchment.	Low	To be addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Substation delineation to be reviewed in Detailed Design. Refer to section 3.8 of Appendix 11 - Flooding and Hydrology Technical Report.	No Panel accepts the commitment to address the issue in Detailed Design.	Open Action required during detailed design.	To be achieved during detailed design.	
S2G.B.54	Byerra Road	The substation delineation is not accurate in the south-western portion of the catchment.	Possibly a small impact on peak flow rates resulting in the south-western portion of the catchment.	Low	To be addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Substation delineation to be reviewed in Detailed Design. Refer to section 3.8 of Appendix 11 - Flooding and Hydrology Technical Report.	No Panel accepts the commitment to address the issue in Detailed Design.	Open Action required during detailed design.	To be achieved during detailed design.	
S2G.B.55	Byerra Road	The substation delineation is not accurate in the south-western portion of the catchment.	Possibly a small impact on peak flow rates resulting in the south-western portion of the catchment.	Low	To be addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Substation delineation to be reviewed in Detailed Design. Refer to section 3.8 of Appendix 11 - Flooding and Hydrology Technical Report.	No Panel accepts the commitment to address the issue in Detailed Design.	Open Action required during detailed design.	To be achieved during detailed design.	
S2G.B.56	Byerra Road	The substation delineation is not accurate in the south-western portion of the catchment.	Possibly a small impact on peak flow rates resulting in the south-western portion of the catchment.	Low	To be addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Substation delineation to be reviewed in Detailed Design. Refer to section 3.8 of Appendix 11 - Flooding and Hydrology Technical Report.	No Panel accepts the commitment to address the issue in Detailed Design.	Open Action required during detailed design.	To be achieved during detailed design.	
S2G.B.57	Byerra Road	The substation delineation is not accurate in the south-western portion of the catchment.	Possibly a small impact on peak flow rates resulting in the south-western portion of the catchment.	Low	To be addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Substation delineation to be reviewed in Detailed Design. Refer to section 3.8 of Appendix 11 - Flooding and Hydrology Technical Report.	No Panel accepts the commitment to address the issue in Detailed Design.	Open Action required during detailed design.	To be achieved during detailed design.	
S2G.B.58	Byerra Road	The substation delineation is not accurate in the south-western portion of the catchment.	Possibly a small impact on peak flow rates resulting in the south-western portion of the catchment.	Low	To be addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Substation delineation to be reviewed in Detailed Design. Refer to section 3.8 of Appendix 11 - Flooding and Hydrology Technical Report.	No Panel accepts the commitment to address the issue in Detailed Design.	Open Action required during detailed design.	To be achieved during detailed design.	
S2G.B.59	Byerra Road	The substation delineation is not accurate in the south-western portion of the catchment.	Possibly a small impact on peak flow rates resulting in the south-western portion of the catchment.	Low	To be addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Substation delineation to be reviewed in Detailed Design. Refer to section 3.8 of Appendix 11 - Flooding and Hydrology Technical Report.	No Panel accepts the commitment to address the issue in Detailed Design.	Open Action required during detailed design.	To be achieved during detailed design.	
S2G.B.60	Byerra Road	The substation delineation is not accurate in the south-western portion of the catchment.	Possibly a small impact on peak flow rates resulting in the south-western portion of the catchment.	Low	To be addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Substation delineation to be reviewed in Detailed Design. Refer to section 3.8 of Appendix 11 - Flooding and Hydrology Technical Report.	No Panel accepts the commitment to address the issue in Detailed Design.	Open Action required during detailed design.	To be achieved during detailed design.	
S2G.B.61	Byerra Road	The substation delineation is not accurate in the south-western portion of the catchment.	Possibly a small impact on peak flow rates resulting in the south-western portion of the catchment.	Low	To be addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Substation delineation to be reviewed in Detailed Design. Refer to section 3.8 of Appendix 11 - Flooding and Hydrology Technical Report.	No Panel accepts the commitment to address the issue in Detailed Design.	Open Action required during detailed design.	To be achieved during detailed design.	
S2G.B.62	Byerra Road	The substation delineation is not accurate in the south-western portion of the catchment.	Possibly a small impact on peak flow rates resulting in the south-western portion of the catchment.	Low	To be addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Substation delineation to be reviewed in Detailed Design. Refer to section 3.8 of Appendix 11 - Flooding and Hydrology Technical Report.	No Panel accepts the commitment to address the issue in Detailed Design.	Open Action required during detailed design.	To be achieved during detailed design.	
S2G.B.63	Byerra Road	The substation delineation is not accurate in the south-western portion of the catchment.	Possibly a small impact on peak flow rates resulting in the south-western portion of the catchment.	Low	To be addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Substation delineation to be reviewed in Detailed Design. Refer to section 3.8 of Appendix 11 - Flooding and Hydrology Technical Report.	No Panel accepts the commitment to address the issue in Detailed Design.	Open Action required during detailed design.	To be achieved during detailed design.	
S2G.B.64	Byerra Road	The substation delineation is not accurate in the south-western portion of the catchment.	Possibly a small impact on peak flow rates resulting in the south-western portion of the catchment.	Low	To be addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Substation delineation to be reviewed in Detailed Design. Refer to section 3.8 of Appendix 11 - Flooding and Hydrology Technical Report.	No Panel accepts the commitment to address the issue in Detailed Design.	Open Action required during detailed design.	To be achieved during detailed design.	
S2G.B.65	Byerra Road	The substation delineation is not accurate in the south-western portion of the catchment.	Possibly a small impact on peak flow rates resulting in the south-western portion of the catchment.	Low	To be addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Substation delineation to be reviewed in Detailed Design. Refer to section 3.8 of Appendix 11 - Flooding and Hydrology Technical Report.	No Panel accepts the commitment to address the issue in Detailed Design.	Open Action required during detailed design.	To be achieved during detailed design.	
S2G.B.66	Byerra Road	The substation delineation is not accurate in the south-western portion of the catchment.	Possibly a small impact on peak flow rates resulting in the south-western portion of the catchment.	Low	To be addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Accepted subject to AETC committing to Panel's recommendations being addressed in Detailed Design.	Substation delineation to be reviewed in Detailed Design. Refer to section 3.8 of Appendix 11 - Flooding and Hydrology Technical Report.	No Panel accepts the commitment to address the issue in Detailed Design.	Open Action required during detailed design.		

Draft No.	Model Name	Issue	Impact on Design	Level of Consequence	Action Required	Comment	Status	Action Required	Comment	Issue	Action Required	FTN Response
B02.C09	Cattle Creek	The adopted TUFLOW HFC engine was not the latest version.	The change in the hydraulic model cover could result in slightly different peak water levels, velocities and discharges.	Low	To be addressed in Detailed Design	Accepted subject to AHFC committing to Panel's recommendations being addressed in Detailed Design.	Accepted subject to AHFC committing to Panel's recommendations being addressed in Detailed Design.	Adopt the latest version of TUFLOW HFC engine in Detailed Design.	Refer to Section 12.1.1 of Appendix T1 - Flooding and Hydrology Technical Report. AHFC has updated hydraulic models to TUFLOW version 2020-10-40-0P-ade as part of the revised draft ES. A review of the latest releases of TUFLOW will be undertaken at the commencement of Detailed Design to determine whether further updates of the TUFLOW version are required.	2020-10-40 version currently adopted using SCS and Q1. Mentioned in the report that a review of the latest TUFLOW version will be undertaken in Detailed Design.	Open. Action required during Detailed Design.	To be actioned during detailed design
B02.C10	Cattle Creek	Lack of detail in Technical Report.	Without additional detail, there are uncertainties in relation to the level of detail of the modelling not appropriate.	Very High	Responded to in Technical Note	Further technical information required to be provided in the Revised Draft ES.	Accepted subject to additional information in Revised Draft ES	Further technical information required to be provided in the Revised Draft ES.	Additional Detail has been provided in the Hydrology and Hydraulic section of the Technical Report for Cattle Creek (refer to Sections 12.2 and 12.3).	The new technical report contains significantly more detail. The Panel recommends that further detail be provided for the additional modelling items that need to be undertaken for Detailed Design.	Open. Action required during detailed design.	To be actioned during detailed design
B02.C11	Condamine River	An existing culvert design was not consistent with current design standards as required for local Networks.	It is considered necessary for flow upstream and downstream of the culvert structure to result in the over or under estimation of discharge as drainage structure.	Low	Responded to in Technical Note	Open. Further information is required to be provided in the Revised Draft ES.	Open.	Open.	Open.	Open. No further action.	Open. No further action.	
B02.C12	Condamine River	The adopted model generally provides a reasonable match to gauged peak water levels. However, there are discrepancies between the 2015 and 2013 event gauged peak water levels. Furthermore, there are significant differences in recorded versus modelled flows and timing of peaks.	It is considered necessary for flow upstream and downstream of the culvert structure to result in the over or under estimation of discharge as drainage structure.	High	Responded to in Technical Note	Further information is required to be provided in the Revised Draft ES.	Accepted subject to additional information in Revised Draft ES	Refer to Section 12.1.1 of Appendix T1 - Flooding and Hydrology Technical Report.	Refer to Section 12.1.1 of Appendix T1 - Flooding and Hydrology Technical Report.	Refer to Section 12.1.1 of Appendix T1 - Flooding and Hydrology Technical Report.	Open. No further action.	
B02.C13	Condamine River	The comparison between the 2013 Flood Event and modelled levels shows a low towards underestimation of levels. There is also no indication or discussion regarding validation to 2013 historic Flood events.	Verification to historic flood events should also confirm the model calibration. The validation to the 2013 event should also confirm the model calibration. The validation to the 2013 event should also confirm the model calibration.	High	Responded to in Technical Note	Further information is required to be provided in the Revised Draft ES.	Accepted subject to additional information in Revised Draft ES	Refer to Section 12.1.1 of Appendix T1 - Flooding and Hydrology Technical Report.	Refer to Section 12.1.1 of Appendix T1 - Flooding and Hydrology Technical Report.	Refer to Section 12.1.1 of Appendix T1 - Flooding and Hydrology Technical Report.	Open. No further action.	
B02.C14	Condamine River	The proposed pattern for the proposed flow or underestimation flow rates and where the design of hydraulic structures and potential impacts.	It is considered necessary for flow upstream and downstream of the culvert structure to result in the over or under estimation of discharge as drainage structure.	High	Responded to in Technical Note	Further information is required to be provided in the Revised Draft ES.	Accepted subject to additional information in Revised Draft ES	Refer to Section 12.1.1 of Appendix T1 - Flooding and Hydrology Technical Report.	Refer to Section 12.1.1 of Appendix T1 - Flooding and Hydrology Technical Report.	Refer to Section 12.1.1 of Appendix T1 - Flooding and Hydrology Technical Report.	Open. No further action.	
B02.C15	Condamine River	The proposed pattern for the proposed flow or underestimation flow rates and where the design of hydraulic structures and potential impacts.	It is considered necessary for flow upstream and downstream of the culvert structure to result in the over or under estimation of discharge as drainage structure.	High	Responded to in Technical Note	Further information is required to be provided in the Revised Draft ES.	Accepted subject to additional information in Revised Draft ES	Refer to Section 12.1.1 of Appendix T1 - Flooding and Hydrology Technical Report.	Refer to Section 12.1.1 of Appendix T1 - Flooding and Hydrology Technical Report.	Refer to Section 12.1.1 of Appendix T1 - Flooding and Hydrology Technical Report.	Open. No further action.	
B02.C16	Condamine River	The adopted approach for applying internal design flow inputs does not follow standard practice. The current approach requires the routing calculations undertaken within the hydraulic model.	Any influence flood extents and peak flow at critical locations which may require change to design or modelled results.	High	Responded to in Technical Note	Further information is required to be provided in the Revised Draft ES.	Accepted subject to additional information in Revised Draft ES	Refer to Section 12.1.1 of Appendix T1 - Flooding and Hydrology Technical Report.	Refer to Section 12.1.1 of Appendix T1 - Flooding and Hydrology Technical Report.	Refer to Section 12.1.1 of Appendix T1 - Flooding and Hydrology Technical Report.	Open. Action required during Detailed Design.	To be actioned during detailed design
B02.C17	Condamine River	The adopted approach for applying internal design flow inputs does not follow standard practice. The current approach requires the routing calculations undertaken within the hydraulic model.	Any influence flood extents and peak flow at critical locations which may require change to design or modelled results.	High	Responded to in Technical Note	Further information is required to be provided in the Revised Draft ES.	Accepted subject to additional information in Revised Draft ES	Refer to Section 12.1.1 of Appendix T1 - Flooding and Hydrology Technical Report.	Refer to Section 12.1.1 of Appendix T1 - Flooding and Hydrology Technical Report.	Refer to Section 12.1.1 of Appendix T1 - Flooding and Hydrology Technical Report.	Open. Action required during Detailed Design.	To be actioned during detailed design
B02.C18	Condamine River	The adopted approach for applying internal design flow inputs does not follow standard practice. The current approach requires the routing calculations undertaken within the hydraulic model.	Any influence flood extents and peak flow at critical locations which may require change to design or modelled results.	High	Responded to in Technical Note	Further information is required to be provided in the Revised Draft ES.	Accepted subject to additional information in Revised Draft ES	Refer to Section 12.1.1 of Appendix T1 - Flooding and Hydrology Technical Report.	Refer to Section 12.1.1 of Appendix T1 - Flooding and Hydrology Technical Report.	Refer to Section 12.1.1 of Appendix T1 - Flooding and Hydrology Technical Report.	Open. Action required during Detailed Design.	To be actioned during detailed design
B02.C19	Condamine River	The adopted approach for applying internal design flow inputs does not follow standard practice. The current approach requires the routing calculations undertaken within the hydraulic model.	Any influence flood extents and peak flow at critical locations which may require change to design or modelled results.	Low	Responded to in Technical Note	Further information is required to be provided in the Revised Draft ES.	Accepted subject to additional information in Revised Draft ES	Refer to Section 12.1.1 of Appendix T1 - Flooding and Hydrology Technical Report.	Refer to Section 12.1.1 of Appendix T1 - Flooding and Hydrology Technical Report.	Refer to Section 12.1.1 of Appendix T1 - Flooding and Hydrology Technical Report.	Open. Action required during Detailed Design.	To be actioned during detailed design
B02.C20	Condamine River	The adopted approach for applying internal design flow inputs does not follow standard practice. The current approach requires the routing calculations undertaken within the hydraulic model.	Any influence flood extents and peak flow at critical locations which may require change to design or modelled results.	Low	Responded to in Technical Note	Further information is required to be provided in the Revised Draft ES.	Accepted subject to additional information in Revised Draft ES	Refer to Section 12.1.1 of Appendix T1 - Flooding and Hydrology Technical Report.	Refer to Section 12.1.1 of Appendix T1 - Flooding and Hydrology Technical Report.	Refer to Section 12.1.1 of Appendix T1 - Flooding and Hydrology Technical Report.	Open. Action required during Detailed Design.	To be actioned during detailed design
B02.C21	Condamine River	The adopted approach for applying internal design flow inputs does not follow standard practice. The current approach requires the routing calculations undertaken within the hydraulic model.	Any influence flood extents and peak flow at critical locations which may require change to design or modelled results.	High	Responded to in Technical Note	Further information is required to be provided in the Revised Draft ES.	Accepted subject to additional information in Revised Draft ES	Refer to Section 12.1.1 of Appendix T1 - Flooding and Hydrology Technical Report.	Refer to Section 12.1.1 of Appendix T1 - Flooding and Hydrology Technical Report.	Refer to Section 12.1.1 of Appendix T1 - Flooding and Hydrology Technical Report.	Open. Action required during Detailed Design.	To be actioned during detailed design
B02.C22	Condamine River	The adopted approach for applying internal design flow inputs does not follow standard practice. The current approach requires the routing calculations undertaken within the hydraulic model.	Any influence flood extents and peak flow at critical locations which may require change to design or modelled results.	Low	Responded to in Technical Note	Further information is required to be provided in the Revised Draft ES.	Accepted subject to additional information in Revised Draft ES	Refer to Section 12.1.1 of Appendix T1 - Flooding and Hydrology Technical Report.	Refer to Section 12.1.1 of Appendix T1 - Flooding and Hydrology Technical Report.	Refer to Section 12.1.1 of Appendix T1 - Flooding and Hydrology Technical Report.	Open. Action required during Detailed Design.	To be actioned during detailed design
B02.C23	Condamine River	The adopted approach for applying internal design flow inputs does not follow standard practice. The current approach requires the routing calculations undertaken within the hydraulic model.	Any influence flood extents and peak flow at critical locations which may require change to design or modelled results.	High	Responded to in Technical Note	Further information is required to be provided in the Revised Draft ES.	Accepted subject to additional information in Revised Draft ES	Refer to Section 12.1.1 of Appendix T1 - Flooding and Hydrology Technical Report.	Refer to Section 12.1.1 of Appendix T1 - Flooding and Hydrology Technical Report.	Refer to Section 12.1.1 of Appendix T1 - Flooding and Hydrology Technical Report.	Open. Action required during Detailed Design.	To be actioned during detailed design
B02.C24	Condamine River	The adopted approach for applying internal design flow inputs does not follow standard practice. The current approach requires the routing calculations undertaken within the hydraulic model.	Any influence flood extents and peak flow at critical locations which may require change to design or modelled results.	High	Responded to in Technical Note	Further information is required to be provided in the Revised Draft ES.	Accepted subject to additional information in Revised Draft ES	Refer to Section 12.1.1 of Appendix T1 - Flooding and Hydrology Technical Report.	Refer to Section 12.1.1 of Appendix T1 - Flooding and Hydrology Technical Report.	Refer to Section 12.1.1 of Appendix T1 - Flooding and Hydrology Technical Report.	Open. Action required during Detailed Design.	To be actioned during detailed design
B02.C25	Condamine River	The adopted approach for applying internal design flow inputs does not follow standard practice. The current approach requires the routing calculations undertaken within the hydraulic model.	Any influence flood extents and peak flow at critical locations which may require change to design or modelled results.	Medium	Responded to in Technical Note	Further information is required to be provided in the Revised Draft ES.	Accepted subject to additional information in Revised Draft ES	Refer to Section 12.1.1 of Appendix T1 - Flooding and Hydrology Technical Report.	Refer to Section 12.1.1 of Appendix T1 - Flooding and Hydrology Technical Report.	Refer to Section 12.1.1 of Appendix T1 - Flooding and Hydrology Technical Report.	Open. Action required during Detailed Design.	To be actioned during detailed design
B02.C26	Condamine River	The adopted approach for applying internal design flow inputs does not follow standard practice. The current approach requires the routing calculations undertaken within the hydraulic model.	Any influence flood extents and peak flow at critical locations which may require change to design or modelled results.	Medium	Responded to in Technical Note	Further information is required to be provided in the Revised Draft ES.	Accepted subject to additional information in Revised Draft ES	Refer to Section 12.1.1 of Appendix T1 - Flooding and Hydrology Technical Report.	Refer to Section 12.1.1 of Appendix T1 - Flooding and Hydrology Technical Report.	Refer to Section 12.1.1 of Appendix T1 - Flooding and Hydrology Technical Report.	Open. Action required during Detailed Design.	To be actioned during detailed design
B02.C27	Condamine River	The adopted approach for applying internal design flow inputs does not follow standard practice. The current approach requires the routing calculations undertaken within the hydraulic model.	Any influence flood extents and peak flow at critical locations which may require change to design or modelled results.	Low	Responded to in Technical Note	Further information is required to be provided in the Revised Draft ES.	Accepted subject to additional information in Revised Draft ES	Refer to Section 12.1.1 of Appendix T1 - Flooding and Hydrology Technical Report.	Refer to Section 12.1.1 of Appendix T1 - Flooding and Hydrology Technical Report.	Refer to Section 12.1.1 of Appendix T1 - Flooding and Hydrology Technical Report.	Open. Action required during Detailed Design.	To be actioned during detailed design

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Issue No.	Model Name	Issue	Impact on Design	Level of Consequence	Action Required	Comment	Status	Action Required	Comment	Status	Action Required	FRV Response
E05-024	Gowrie Creek ES	Increase in duration of inundation were reported on local roads, with no conclusions drawn as to their acceptability to Council and TfW.	The design may not sufficiently mitigate impacts to roads.	High	Model to be issued to Flood Panel.	No reporting on impacts has been provided for the updated model to date. Report on impacts based off new FICs in Revised Draft ES.	Accepted subject to additional information in Revised Draft ES.	Report on impacts based off FICs in Revised Draft ES.	Reporting of impacts against the new FICs are located in Section 5.3.3 of Appendix T1 - Flooding and Hydrology Technical Report.	Detailed information has been provided in relation to the socioeconomic and their quantum for each FIC. Justification of the socioeconomic and consideration of the potential for mitigation is required.	Open Action required during Detailed Design.	Table 5.34 of Appendix T1 - Flooding and Hydrology Technical Report - Volume 1 has been updated with additional justification and mitigation against high risk roads as identified by the Flood Advisory Panel.
E05-025	Gowrie Creek ES	Only the 2% AEP change in duration of inundation and change in velocity was reported, which is insufficient to draw conclusions on change in duration of inundation and velocity in urbanised areas.	The design may not sufficiently mitigate changes in duration of inundation or changes in velocity, particularly for frequent events.	Medium	Model to be issued to Flood Panel.	No reporting on impacts has been provided for the updated model to date. Report on impacts based off new FICs in Revised Draft ES.	Accepted subject to additional information in Revised Draft ES.	Report on impacts based off FICs in Revised Draft ES.	Reporting of impacts against the new FICs are located in Section 5.3.3 of Appendix T1 - Flooding and Hydrology Technical Report.	Detailed information has been provided in relation to the socioeconomic and their quantum for each FIC. Justification of the socioeconomic and consideration of the potential for mitigation is required.	Open Action required during Detailed Design.	Table 5.25, 5.26, 5.28 and 5.29 of Appendix T1 - Flooding and Hydrology Technical Report - Volume 1 have been updated with additional justification and mitigation against high risk agricultural and residential properties and local and state controlled roads as identified by the Flood Advisory Panel.
E05-026	Gowrie Creek ES	The effects of the change in flood inundation on the design of the road are not known.	The effects of the change in flood inundation on the design of the road are not known.	Medium	Model to be issued to Flood Panel.	No reporting on impacts has been provided for the updated model to date. Report on impacts based off new FICs in Revised Draft ES.	Accepted subject to additional information in Revised Draft ES.	Report on impacts based off FICs in Revised Draft ES.	Reporting of impacts against the new FICs are located in Section 5.3.3 of Appendix T1 - Flooding and Hydrology Technical Report.	Detailed information has been provided in relation to the socioeconomic and their quantum for each FIC. Justification of the socioeconomic and consideration of the potential for mitigation is required.	Open Action required during Detailed Design.	Table 5.25, 5.26, 5.28 and 5.29 of Appendix T1 - Flooding and Hydrology Technical Report - Volume 1 have been updated with additional justification and mitigation against high risk agricultural and residential properties and local and state controlled roads as identified by the Flood Advisory Panel.
E05-081	Macintyre River	Request to be duplication of sub-catchment area in lower and Orriy Creek hydrologic models.	Design flow may be incorrect throughout flood models. GSI flow has not been provided for the Panel to check this.	Medium	Technical notes updated to permit to this being reviewed as part of Revised Draft ES.	Revised Draft ES to adopt the revised hydrological model and document a review of all sub-catchment boundaries.	Accepted subject to additional information in Revised Draft ES.	Revised Draft ES to adopt the revised hydrological model and document a review of all sub-catchment boundaries.	Refer to Sections 2.2.1 and 2.2.2 of Appendix T1 - Flooding and Hydrology Technical Report.	Application still appears to be apparent (sub-catchment 15 of the lower URB1 model appears to be overlapping Orriy Creek sub-catchment 14). It is expected that the overlap will have minimal impacts on flow and can be defined to Detailed Design stage.	Open Action required during Detailed Design.	To be actioned during detailed design
E05-082	Macintyre River	Value of the URB1 sub-model have been used to determine the area of sub-catchment 18.	Incorrect flow rates at the alignment.	Medium	Technical notes updated to permit to this being reviewed as part of Revised Draft ES.	Revised Draft ES to adopt the revised hydrological model and document a review of all sub-catchment boundaries.	Accepted subject to additional information in Revised Draft ES.	Revised Draft ES to adopt the revised hydrological model and document a review of all sub-catchment boundaries.	Refer to Sections 2.2.1 and 2.2.2 of Appendix T1 - Flooding and Hydrology Technical Report.	Application still appears to be apparent (sub-catchment 15 of the lower URB1 model appears to be overlapping Orriy Creek sub-catchment 14). It is expected that the overlap will have minimal impacts on flow and can be defined to Detailed Design stage.	Open Action required during Detailed Design.	To be actioned during detailed design
E05-083	Macintyre River	Value of the URB1 sub-model have been used to determine the area of sub-catchment 18.	Incorrect flow rates at the alignment.	Medium	Technical notes updated to permit to this being reviewed as part of Revised Draft ES.	Revised Draft ES to adopt the revised hydrological model and document a review of all sub-catchment boundaries.	Accepted subject to additional information in Revised Draft ES.	Revised Draft ES to adopt the revised hydrological model and document a review of all sub-catchment boundaries.	Refer to Sections 2.2.1 and 2.2.2 of Appendix T1 - Flooding and Hydrology Technical Report.	Application still appears to be apparent (sub-catchment 15 of the lower URB1 model appears to be overlapping Orriy Creek sub-catchment 14). It is expected that the overlap will have minimal impacts on flow and can be defined to Detailed Design stage.	Open Action required during Detailed Design.	To be actioned during detailed design
E05-084	Macintyre River	The critical duration for the Macintyre River at the lowest flood alignment was the 24-hour event in both lower scenarios in the 2% AEP flood. This critical duration should be confirmed by the designer and sufficient duration in the report added. It is noted that this duration seems relatively short for the scale of the catchment.	Incorrect storm duration could result in incorrect flow rates at the alignment.	Medium	Technical note updated to permit to this being reviewed as part of Revised Draft ES.	Revised Draft ES to review and document the critical durations and whether the hydrologic and hydraulic models are in agreement for the 2% AEP event to show that the hydrologic model's routing is acceptable.	Accepted subject to additional information in Revised Draft ES.	Revised Draft ES to review and document the critical durations and whether the hydrologic and hydraulic models are in agreement for the 2% AEP event to show that the hydrologic model's routing is acceptable.	Refer to Section 17.4.1 of Appendix T1 - Flooding and Hydrology Technical Report.	Critical duration not confirmed with hydraulic modelling which was agreed to be undertaken at this stage. It is considered that more detail is required in the Revised ES.	Open Action required during Detailed Design.	Section 17.4.1 of Appendix T1 - Flooding and Hydrology Technical Report - Volume 1 has been updated with additional detail. Further response below: "The critical duration was selected using the hydrologic model. The adopted approach was considered to be appropriate for the impact assessment, and as a relatively good match for the level of detail, as both the hydrologic and hydraulic models were calibrated to a range of historical flood events. In addition, the overall effect of the floodplain attenuation relative to the magnitude of the combined inflows from the hydrologic models is comparatively minor. The joint calibration of the hydrologic and hydraulic models and the derivation of the critical duration is proposed to be reviewed in future stages."
E05-085	Macintyre River	Value of the URB1 sub-model have been used to determine the area of sub-catchment 18.	Incorrect flow rates at the alignment.	Medium	Technical notes updated to permit to this being reviewed as part of Revised Draft ES.	Revised Draft ES to adopt the revised hydrological model and document a review of all sub-catchment boundaries.	Accepted subject to additional information in Revised Draft ES.	Revised Draft ES to adopt the revised hydrological model and document a review of all sub-catchment boundaries.	Refer to Sections 2.2.1 and 2.2.2 of Appendix T1 - Flooding and Hydrology Technical Report.	Application still appears to be apparent (sub-catchment 15 of the lower URB1 model appears to be overlapping Orriy Creek sub-catchment 14). It is expected that the overlap will have minimal impacts on flow and can be defined to Detailed Design stage.	Open Action required during Detailed Design.	To be actioned during detailed design
E05-086	Macintyre River	A 20% blockage factor has been applied to existing culverts although there are no culverts where no blockage applied.	The flood flows for design events may be underestimated.	Medium	Technical note updated to permit to this being reviewed as part of Revised Draft ES.	Revised Draft ES to include updated and consistent culvert blockage data.	Accepted subject to additional information in Revised Draft ES.	Revised Draft ES to include updated information for all culverts.	Details for all culverts have been provided in the Cross Drainage Register located in Appendix 6 of Appendix T1 - Flooding and Hydrology Technical Report.	Model flow has been checked and it has been confirmed that the majority of the culverts have a 20% blockage factor applied. Culverts with 0.0485, 0.049 and 0.0495 still have zero blockage. As the majority of the culverts have 20% blockage, the impacts of the three remaining is unlikely to be significant. Panel Review Blockage Factor to be included in next of Detailed Design.	Open Action required during Detailed Design.	To be actioned during detailed design
E05-087	Macintyre River	Upstream river boundaries have used source area instead of sub-catchment area. Source area boundaries is slightly downstream of the boundary and some areas not contained over the river channel/flow not cover the flood extent.	Design flow rates could be affected at alignment of upstream boundaries are incorrect.	Low	A sensitivity run has been undertaken to demonstrate the impact of boundary line inflows, which is standard practice in TUFLOW, but have explained that the source area boundaries provide a more realistic flow. Considering all information, the decision is acceptable.	Sensitivity analysis showed that there was no impact on the alignment. APTC have not proposed to adopt 2% boundary line inflows, which is standard practice in TUFLOW, but have explained that the source area boundaries provide a more realistic flow. Considering all information, the decision is acceptable.	Closed	N/A	The Panel accepts the commitment to refine the placement of SA inflows in Detailed Design.	Open Action required during Detailed Design.	To be actioned during detailed design	
E05-088	Macintyre River	The flood extent for design events may be underestimated.	The flood extent for design events may be underestimated.	Medium	Technical notes updated to permit to this being reviewed as part of Revised Draft ES.	Revised Draft ES to include updated and consistent culvert blockage data.	Accepted subject to additional information in Revised Draft ES.	Revised Draft ES to include updated information for all culverts.	Details for all culverts have been provided in the Cross Drainage Register located in Appendix 6 of Appendix T1 - Flooding and Hydrology Technical Report.	Model flow has been checked and it has been confirmed that the majority of the culverts have a 20% blockage factor applied. Culverts with 0.0485, 0.049 and 0.0495 still have zero blockage. As the majority of the culverts have 20% blockage, the impacts of the three remaining is unlikely to be significant. Panel Review Blockage Factor to be included in next of Detailed Design.	Open Action required during Detailed Design.	To be actioned during detailed design
E05-089	Macintyre River	Missing hydraulic structures in existing model.	The absence of these structures in the model may be causing artificial blockage in some areas. However, it is unlikely that this will have significant impact on results from isolated events, such as the 2% AEP, and may only influence frequent events.	Low	Technical notes updated to permit to this being reviewed as part of Revised Draft ES.	Revised Draft ES to include updated and consistent culvert blockage data.	Accepted subject to additional information in Revised Draft ES.	Revised Draft ES to include updated information for all culverts.	Details for all culverts have been provided in the Cross Drainage Register located in Appendix 6 of Appendix T1 - Flooding and Hydrology Technical Report.	Model flow has been checked and it has been confirmed that the majority of the culverts have a 20% blockage factor applied. Culverts with 0.0485, 0.049 and 0.0495 still have zero blockage. As the majority of the culverts have 20% blockage, the impacts of the three remaining is unlikely to be significant. Panel Review Blockage Factor to be included in next of Detailed Design.	Open Action required during Detailed Design.	To be actioned during detailed design
E05-090	Macintyre River	No blockage has been applied to the substructure of bridges to account for pier or any potential debris blockage. No form loss applied to superstructure to account for bridge deck loss.	The design may not properly mitigate flood impacts at all locations.	Medium	Technical notes updated to permit to this being reviewed as part of Revised Draft ES.	Revised Draft ES to include updated and consistent culvert blockage data.	Accepted subject to additional information in Revised Draft ES.	Revised Draft ES to include updated information for all culverts.	Details for all culverts have been provided in the Cross Drainage Register located in Appendix 6 of Appendix T1 - Flooding and Hydrology Technical Report.	Model flow has been checked and it has been confirmed that the majority of the culverts have a 20% blockage factor applied. Culverts with 0.0485, 0.049 and 0.0495 still have zero blockage. As the majority of the culverts have 20% blockage, the impacts of the three remaining is unlikely to be significant. Panel Review Blockage Factor to be included in next of Detailed Design.	Open Action required during Detailed Design.	To be actioned during detailed design
E05-091	Native Dog Creek	Sufficient information is provided in relation to the Macintyre Brook FFA and its use in the validation process.	Flood parameters were to be adjusted. Flow and levels for design events would be affected.	Low	Technical notes updated to permit to this being reviewed as part of Revised Draft ES.	Revised Draft ES to include updated and consistent culvert blockage data.	Accepted subject to additional information in Revised Draft ES.	Revised Draft ES to include updated information for all culverts.	Details for all culverts have been provided in the Cross Drainage Register located in Appendix 6 of Appendix T1 - Flooding and Hydrology Technical Report.	Model flow has been checked and it has been confirmed that the majority of the culverts have a 20% blockage factor applied. Culverts with 0.0485, 0.049 and 0.0495 still have zero blockage. As the majority of the culverts have 20% blockage, the impacts of the three remaining is unlikely to be significant. Panel Review Blockage Factor to be included in next of Detailed Design.	Open Action required during Detailed Design.	To be actioned during detailed design
E05-092	Native Dog Creek	Parameters used in URB1 model.	Flood parameters were to be adjusted. Flow and levels for design events would be affected.	Low	Technical notes updated to permit to this being reviewed as part of Revised Draft ES.	Revised Draft ES to include updated and consistent culvert blockage data.	Accepted subject to additional information in Revised Draft ES.	Revised Draft ES to include updated information for all culverts.	Details for all culverts have been provided in the Cross Drainage Register located in Appendix 6 of Appendix T1 - Flooding and Hydrology Technical Report.	Model flow has been checked and it has been confirmed that the majority of the culverts have a 20% blockage factor applied. Culverts with 0.0485, 0.049 and 0.0495 still have zero blockage. As the majority of the culverts have 20% blockage, the impacts of the three remaining is unlikely to be significant. Panel Review Blockage Factor to be included in next of Detailed Design.	Open Action required during Detailed Design.	To be actioned during detailed design
E05-093	Native Dog Creek	Final loss rates not detailed in Technical Report. Potential variation in loss rates could mask misinterpretation with other URB1 parameters.	The use of alternate loss rates could affect design flow and therefore the required size of drainage structures.	Low	Technical notes updated to permit to this being reviewed as part of Revised Draft ES.	Revised Draft ES to include updated and consistent culvert blockage data.	Accepted subject to additional information in Revised Draft ES.	Revised Draft ES to include updated information for all culverts.	Details for all culverts have been provided in the Cross Drainage Register located in Appendix 6 of Appendix T1 - Flooding and Hydrology Technical Report.	Model flow has been checked and it has been confirmed that the majority of the culverts have a 20% blockage factor applied. Culverts with 0.0485, 0.049 and 0.0495 still have zero blockage. As the majority of the culverts have 20% blockage, the impacts of the three remaining is unlikely to be significant. Panel Review Blockage Factor to be included in next of Detailed Design.	Open Action required during Detailed Design.	To be actioned during detailed design
E05-094	Native Dog Creek	The hydraulic model extent is not set at sufficient distance from the flood extent.	The artificial restriction directly impacts the results generated for the FFA event.	Medium	Technical notes updated to permit to this being reviewed as part of Revised Draft ES.	Revised Draft ES to include updated and consistent culvert blockage data.	Accepted subject to additional information in Revised Draft ES.	Revised Draft ES to include updated information for all culverts.	Details for all culverts have been provided in the Cross Drainage Register located in Appendix 6 of Appendix T1 - Flooding and Hydrology Technical Report.	Model flow has been checked and it has been confirmed that the majority of the culverts have a 20% blockage factor applied. Culverts with 0.0485, 0.049 and 0.0495 still have zero blockage. As the majority of the culverts have 20% blockage, the impacts of the three remaining is unlikely to be significant. Panel Review Blockage Factor to be included in next of Detailed Design.	Open Action required during Detailed Design.	To be actioned during detailed design
E05-095	Native Dog Creek	Adjacent catchment inflows have not been included as in the hydraulic model.	The current flood representation of the alignment, proposed bridge and adjacent catchment inflows, which is standard practice in TUFLOW, but have explained that the source area boundaries provide a more realistic flow. Considering all information, the decision is acceptable.	High	Technical notes updated to permit to this being reviewed as part of Revised Draft ES.	Revised Draft ES to include updated and consistent culvert blockage data.	Accepted subject to additional information in Revised Draft ES.	Revised Draft ES to include updated information for all culverts.	Details for all culverts have been provided in the Cross Drainage Register located in Appendix 6 of Appendix T1 - Flooding and Hydrology Technical Report.	Model flow has been checked and it has been confirmed that the majority of the culverts have a 20% blockage factor applied. Culverts with 0.0485, 0.049 and 0.0495 still have zero blockage. As the majority of the culverts have 20% blockage, the impacts of the three remaining is unlikely to be significant. Panel Review Blockage Factor to be included in next of Detailed Design.	Open Action required during Detailed Design.	Mapping has been provided as requested by the CECs in correspondence received on the 30th September 2022. It was agreed that hard copy mapping would be provided for the 20m and 1% AEP 24h rainfall events for all catchments. Extensive event mapping was agreed to be provided on the Digital Platform, as the Flood Assessment Panel has decided in their response. To add extensive event mapping into Appendix T2 - Safety for Native Dog Creek would result in inconsistencies with other catchments. It is not proposed to update Appendix 2 to Extensive Event mapping is available via the Digital Platform.
E05-096	Native Dog Creek	The downstream HQ boundary is set between 80 m and 250 m downstream of the alignment.	Long section analysis of water levels suggests that the current model boundary may be draining water out of the model which would be improving the design performance.	High	Technical notes updated to permit to this being reviewed as part of Revised Draft ES.	Revised Draft ES to include updated and consistent culvert blockage data.	Accepted subject to additional information in Revised Draft ES.	Revised Draft ES to include updated information for all culverts.	Details for all culverts have been provided in the Cross Drainage Register located in Appendix 6 of Appendix T1 - Flooding and Hydrology Technical Report.	Model flow has been checked and it has been confirmed that the majority of the culverts have a 20% blockage factor applied. Culverts with 0.0485, 0.049 and 0.0495 still have zero blockage. As the majority of the culverts have 20% blockage, the impacts of the three remaining is unlikely to be significant. Panel Review Blockage Factor to be included in next of Detailed Design.	Open Action required during Detailed Design.	To be actioned during detailed design
E05-097	Native Dog Creek	The adaptive TUFLOW HPC engine was not the latest version.	The change in the hydraulic model could result in slightly different peak water levels, velocities and discharges.	Low	Technical notes updated to permit to this being reviewed as part of Revised Draft ES.	Revised Draft ES to include updated and consistent culvert blockage data.	Accepted subject to additional information in Revised Draft ES.	Revised Draft ES to include updated information for all culverts.	Details for all culverts have been provided in the Cross Drainage Register located in Appendix 6 of Appendix T1 - Flooding and Hydrology Technical Report.	Model flow has been checked and it has been confirmed that the majority of the culverts have a 20% blockage factor applied. Culverts with 0.0485, 0.049 and 0.0495 still have zero blockage. As the majority of the culverts have 20% blockage, the impacts of the three remaining is unlikely to be significant. Panel Review Blockage Factor to be included in next of Detailed Design.	Open Action required during Detailed Design.	To be actioned during detailed design
E05-098	Native Dog Creek	Level of detail in Technical Report.	Without additional detail, there are uncertainties in relation to the level of detail of the modelling and its appropriateness.	Very High	Technical notes updated to permit to this being reviewed as part of Revised Draft ES.	Revised Draft ES to include updated and consistent culvert blockage data.	Accepted subject to additional information in Revised Draft ES.	Revised Draft ES to include updated information for all culverts.	Details for all culverts have been provided in the Cross Drainage Register located in Appendix 6 of Appendix T1 - Flooding and Hydrology Technical Report.	Model flow has been checked and it has been confirmed that the majority of the culverts have a 20% blockage factor applied. Culverts with 0.0485, 0.049 and 0.0495 still have zero blockage. As the majority of the culverts have 20% blockage, the impacts of the three remaining is unlikely to be significant. Panel Review Blockage Factor to be included in next of Detailed Design.	Open Action required during Detailed Design.	To be actioned during detailed design
E05-099	Native Dog Creek	Sub-catchment extent details not provided.	No directly accounting for flow gain and downstream of the alignment can result in the over or under estimation of discharge at a drainage structure depending on the point of application of the inflow for a sub-catchment.	High	Technical notes updated to permit to this being reviewed as part of Revised Draft ES.	Revised Draft ES to include updated and consistent culvert blockage data.	Accepted subject to additional information in Revised Draft ES.	Revised Draft ES to include updated information for all culverts.	Details for all culverts have been provided in the Cross Drainage Register located in Appendix 6 of Appendix T1 - Flooding and Hydrology Technical Report.	Model flow has been checked and it has been confirmed that the majority of the culverts have a 20% blockage factor applied. Culverts with 0.0485, 0.049 and 0.0495 still have zero blockage. As the majority of the culverts have 20% blockage, the impacts of the three remaining is unlikely to be significant. Panel Review Blockage Factor to be included in next of Detailed Design.	Open Action required during Detailed Design.	To be actioned during detailed design

Issue No.	Model Name	Issue	Impact on Design	Level of Importance	Action Required	Comment	Status	Action Required	Comments	Notes	Action Required	RTN Response
B2G-N2	Nicot Creek	Insufficient information is provided in relation to the Macintyre Brook FPA and its use in the validation process. Design flow lower than HFC and QFT values.	If model parameters were to be adjusted, flow and levels for design events would be affected.	High	Updated response provided in updated Technical Note, with updated text highlighted.	ATTC has undertaken sensitivity analysis with lower rainfall rates to confirm that these would only have a small increase in flow (4%). To see what the resultant impacts would be with flow increased to match the QFT, ATTC factored up the flow by 1.45 with the resultant levels increasing by 400mm. The headward of the catchment is still maintained. Afflux increase to 220mm but only impacts on 0.2ha area and not at sensitive receptors. ATTC state that overall, the sensitivity tests demonstrate that both the design and FFO outcomes are not influenced by the uncertainties in using ATTC data noting that the URB parameters are already conservative and the otherwise lack of evidence to favour adoption of the regional estimates, the current URB design discharges are considered appropriate for adoption in the Detail Design phase of the Project. The Panel recommend higher flows still be used in Detailed Design phase to ensure higher velocities in particular are accounted for.	Accepted subject to ATTC committing to Panel's recommendations being addressed in Detailed Design	Higher design flow estimates are to be used in Detailed Design phase to ensure higher velocities in particular are accounted for.	Refer to Section 9.2.1 of Appendix T1 - Flooding and Hydrology Technical Report.	Higher flows as recommended by the Panel have not been adopted as yet. The Panel recommends that higher flows be used for Detailed Design phase to ensure higher velocities in particular are accounted for. It is recommended that Section 9.4 of Appendix T1 be modified to reflect this and confirmation be provided by ATTC that the flows will be adjusted as part of Detailed Design.	Open. Action required during detailed design.	To be actioned during detailed design
B2G-N3	Nicot Creek	Parameters used in URB model.	If model parameters were to be adjusted, flow and levels for design events would be affected.	Medium	Updated response provided in updated Technical Note, with updated text highlighted.	Higher design flow estimates are to be used in Detailed Design phase to ensure higher velocities in particular are accounted for.	Accepted subject to ATTC committing to Panel's recommendations being addressed in Detailed Design	Higher design flow estimates are to be used in Detailed Design phase to ensure higher velocities in particular are accounted for.	To be addressed in Detailed Design. Refer Section 9.4 of Appendix T1 - Flooding and Hydrology Technical Report.	Higher flows as recommended by the Panel have not been adopted as yet. The Panel recommends that higher flows be used for Detailed Design phase to ensure higher velocities in particular are accounted for. It is recommended that Section 9.4 of Appendix T1 be modified to reflect this and confirmation be provided by ATTC that the flows will be adjusted as part of Detailed Design.	Open. Action required during detailed design.	To be actioned during detailed design
B2G-N4	Nicot Creek	ATTC has taken not detailed. Potential impact on the design of the bridge structure.	The use of alternate flow rates could affect design flows and therefore the required size of bridge structures.	Low	Responded to in Technical Note.	ATTC have identified flow rates and given information to include in next reports. This is acceptable noting that the ultimate response to B2G-N5 may result in the adoption of different flow values.			Refer to Section 9.2.3.1 of Appendix T1 - Flooding and Hydrology Technical Report.	Closed. No further action.		
B2G-N5	Nicot Creek	The assigned TUFLOW HFC engine is not the latest version.	The change in the hydraulic model after the latest version could result in slightly different peak water levels, velocities and discharges	Low	To be addressed in Detailed Design	Accepted subject to ATTC committing to Panel's recommendations being addressed in Detailed Design.	Accepted subject to ATTC committing to Panel's recommendations being addressed in Detailed Design	Adopt the latest version of TUFLOW HFC engine in Detailed Design.	Refer to Section 9.3.1 of Appendix T1 - Flooding and Hydrology Technical Report. ATTC have updated hydraulic models to TUFLOW version 2020.3A-AD-IP-well as part of the revised draft ES. A review of the latest release of TUFLOW will be undertaken at the commencement of Detailed Design to determine whether further updates of the TUFLOW version are required.	2020-31 HFC version currently adopted using 302 and QFT. Mentioned in the report that a review of the latest TUFLOW version will be undertaken in Detailed Design.	Open. Action required during detailed design.	To be actioned during detailed design
B2G-N6	Nicot Creek	ATTC has taken not detailed. Potential impact on the design of the bridge structure.	The use of alternate flow rates could affect design flows and therefore the required size of bridge structures.	Low	Responded to in Technical Note.	ATTC have identified flow rates and given information to include in next reports. This is acceptable noting that the ultimate response to B2G-N5 may result in the adoption of different flow values.			Refer to Section 9.2.3.1 of Appendix T1 - Flooding and Hydrology Technical Report.	Closed. No further action.		
B2G-N7	Nicot Creek	Location of inflow applied to the hydraulic model are inconsistent.	Inconsistent application of the inflow location will impact on the element timing of the hydrographs from the respective sub-catchments and total inflow.	High	Responded to in Technical Note.	How that subcatchment map has been supplied, most of the inflow are considered to be in a reasonable position except sub-catchment 12. The response states that there is an issue but that the change (splitting the inflow and applying in two locations) was necessary to reflect the catchment and that the impact of the change is minimal.	Closed	N/A	N/A	Closed. No further action.		
B2G-N8	Nicot Creek	No issues applied to bridge "spanning" bridge losses not calibrated or verified against alternate methods.	Larger lateral bridge structures could be required to minimise impacts to satisfy adopted criteria.	Low	To be addressed in Detailed Design	Accepted subject to ATTC committing to Panel's recommendations being addressed in Detailed Design.	Accepted subject to ATTC committing to Panel's recommendations being addressed in Detailed Design	Detailed Design to include bridge losses and to look at alternate approaches.	To be addressed in Detailed Design. Refer Section 9.4 of Appendix T1 - Flooding and Hydrology Technical Report.	No changes to bridge representation as part of the updates. The Panel accepts the commitment to address the issue in Detailed Design.	Open. Action required during detailed design.	To be actioned during detailed design
B2G-N9	Nicot Creek	Level of detail in Technical Report.	Without additional detail, there are uncertainties in relation to the level of detail of the modelling and its appropriateness.	Very High	Responded to in Technical Note.	Seems a reasonable way forward as long the level of detail in the final report is adequate. So far the details in the Technical Note are not of sufficient detail. Though, also note that the commitment is to "have" which is not an over-arching commitment to change the report.	Closed	N/A	Additional Detail has been provided in the Hydrology and Hydraulic section of the Technical Report for Nicot Creek. Refer to Sections 9.2 and 9.3 of Appendix T1 - Flooding and Hydrology Technical Report.	Additional detail has been provided.	Closed. No further action.	
B2G-P1	Parangara Creek	Inflow has been applied to the model via an SA polygon utilising the "All" command.	The inflow approach does not align with the hydrologic model setup and negates the resulting response.	Low	To be addressed in Detailed Design	Accepted subject to ATTC committing to Panel's recommendations being addressed in Detailed Design.	Accepted subject to ATTC committing to Panel's recommendations being addressed in Detailed Design.	Refer to Section 9.2.3 of Appendix T1 - Flooding and Hydrology Technical Report.	It is reported that the inflow application within the hydraulic model will be reviewed in Detailed Design. The Panel recommends that inflow application be committed to in Detailed Design.	Open. Action required during detailed design.	To be actioned during detailed design	
B2G-P2	Parangara Creek	Critical duration analysis and temporal pattern selection has only been undertaken for the hydrologic model outlet, with a single duration and temporal pattern selected.	The current approach of a single temporal pattern and critical duration selection may result in an underestimation of peak flow at key hydrologic infrastructure.	Medium	Responded to in Technical Note.	Sensitivity Test performed and Response provided in updated Technical Note, with additional text highlighted.	Accepted subject to ATTC committing to Panel's recommendations being addressed in Detailed Design.	Revised Draft ES to present results of sensitivity analysis and to present results where impacts are in excess of FFOs.	Sensitivity test results have been presented in Section 13.4.1 of Appendix T1 - Flooding and Hydrology Technical Report. In addition, impacts that exceed the newly updated FFO have been detailed in Section 13.5.3 of Appendix T1 - Flooding and Hydrology Technical Report.	Commitment made to review to section channel inflow in Detailed Design and additional detail has been provided in relation to FFO exceedance. However, justification of the exceedance and consideration of the potential for mitigation is required.	Open. Action required during detailed design.	Open. Action required during detailed design
B2G-P3	Parangara Creek	Downstream catchment inflow and outflow loading has not been accounted for in the model.	The current approach does not account for the potential impact to tailwater conditions which is particularly important in flat areas. Furthermore, the lack of inflow downstream of the alignment is allowing for backwater effects and storage that may not be realistic in a flood event.	Medium	Responded to in Technical Note.	Sensitivity Test performed and Response provided in updated Technical Note, with additional text highlighted.	Accepted subject to ATTC committing to Panel's recommendations being addressed in Detailed Design.	Tailwater changes to be adopted in Detailed Design.	To be addressed in Detailed Design. Refer to Section 13.6 of Appendix T1 - Flooding and Hydrology Technical Report.	It is reported that the tailwater application to be reviewed to account for the effects of Macintyre Brook. Sensitivity analysis shows flood level increases up to 20mm at the Project Alignment. The Panel recommends that the tailwater changes be committed to in Detailed Design.	Open. Action required during detailed design.	To be actioned during detailed design
B2G-P4	Parangara Creek	Both from downstream do not show any design consequences in terms of QFT.	Based on the comparison to the validation results from 6.4.1 (6.4.1.1) the current model estimates are low. As such, the relative impacts of the real alignment may be underestimated.	High	Responded to in Technical Note.	Refer to the comparison to the validation results from 6.4.1 (6.4.1.1) the current model estimates are low. As such, the relative impacts of the real alignment may be underestimated.			N/A	Closed. No further action.		
B2G-P5	Parangara Creek	The assigned TUFLOW HFC engine was not the latest version.	The change in the hydraulic model after the latest version could result in slightly different peak water levels, velocities and discharges	Low	To be addressed in Detailed Design	Accepted subject to ATTC committing to Panel's recommendations being addressed in Detailed Design.	Accepted subject to ATTC committing to Panel's recommendations being addressed in Detailed Design	Adopt the latest version of TUFLOW HFC engine in Detailed Design.	Refer to Section 9.3.1.1. ATTC have updated hydraulic models to TUFLOW version 2020.3A-AD-IP-well as part of the revised draft ES. A review of the latest release of TUFLOW will be undertaken at the commencement of Detailed Design to determine whether further updates of the TUFLOW version are required.	2020-31 HFC version currently adopted using 302 and QFT. Mentioned in the report that a review of the latest TUFLOW version will be undertaken in Detailed Design.	Open. Action required during detailed design.	To be actioned during detailed design
B2G-P6	Parangara Creek	No issues applied to bridge "spanning" bridge loss coefficients not calibrated or verified against alternate methods.	Larger lateral bridge structures could be required to minimise impacts to satisfy adopted criteria.	Low	To be addressed in Detailed Design	Accepted subject to ATTC committing to Panel's recommendations being addressed in Detailed Design.	Accepted subject to ATTC committing to Panel's recommendations being addressed in Detailed Design	Detailed Design to confirm recalibration of bridge losses using alternate approaches.	To be addressed in Detailed Design. Refer to Section 13.6 of Appendix T1 - Flooding and Hydrology Technical Report.	It is reported that a review of bridge losses will be undertaken to verify against alternative methods in Detailed Design.	Open. Action required during detailed design.	To be actioned during detailed design
B2G-P7	Parangara Creek	Level of detail in Technical Report.	Without additional detail, there are uncertainties in relation to the level of detail of the modelling and its appropriateness.	Very High	Responded to in Technical Note.	Further technical information required to be provided in the Revised Draft ES.	Accepted subject to ATTC committing to Panel's recommendations being addressed in Detailed Design.	Further technical information required to be provided in the Revised Draft ES.	Level of detail reviewed in Sections 13.1, 13.2 and 13.3 of Appendix T1 - Flooding and Hydrology Technical Report.	The new technical report contains significantly more detail. The Panel recommends that further detail be provided for the additional modelling items that need to be undertaken for Detailed Design.	Open. Action required during detailed design.	To be actioned during detailed design
B2G-P8	Parangara Creek	Validation for flood impacts.	If an increase is deemed to be excessive, then the design will need to be modified.	Very High	Responded to in Technical Note.	Revised Draft ES to document all impacts against the updated FFOs and justify why any impacts were not within the FFOs.	Accepted subject to ATTC committing to Panel's recommendations being addressed in Detailed Design.	Revised Draft ES to document all impacts against the updated FFOs and justify why any impacts were not within the FFOs.	Validation for FFO exceedance has been provided in relation to the exceedance and their quantification for each FFO. Justification of the exceedance and consideration of the potential for mitigation is required.	Validation information has been provided in relation to the exceedance and their quantification for each FFO. Justification of the exceedance and consideration of the potential for mitigation is required.	Open. Action required during detailed design.	Open. Action required during detailed design
B2G-C1	Woolbrook & Dry Creeks	No climate change modelling has been undertaken.	The result of Climate Change will cause lower increases in peak discharge and associated water throughout the model domain. Based on the 2x ADP results, a slight increase is unlikely to cause additional hydraulic impacts which could result in an acceptable outcome to surrounding properties or infrastructure.	Medium	Responded to in Technical Note.	Analysis is included.			To be addressed in Detailed Design. Refer Section 9.2.3 of Appendix T1 - Flooding and Hydrology Technical Report.	Open. Climate change scenario included.	Closed. No further action.	
B2G-V1	Validation to Ingebrook	Three hydrologic models were adopted for the validation to Ingebrook hydrologic model.	The reported impacts and rat immunity may not be correct.	High	Responded to in Technical Note.	It will be necessary to review the impacts associated with the revised modelling unless there are no significant differences to the hydraulic model results as a result of the revised hydrology.	Accepted subject to ATTC committing to Panel's recommendations being addressed in Detailed Design.	Include the revised hydrology in the Revised Draft ES.	Refer to Section 9.4 of Appendix T1 - Flooding and Hydrology Technical Report.	The model was not subjected to both a set of parameters chosen that a series used for calibration. The model was not subjected to both a set of parameters chosen that a series used for calibration. The model was not subjected to both a set of parameters chosen that a series used for calibration.	Closed. No further action.	
B2G-V2	Validation to Ingebrook	Subcatchment downstream of the Stallbrook Brook (SSA) subcatchment 44 and 45 were not discussed in the Technical Report in high detail, meaning that the lower portion of the Macintyre Brook catchment may not have been properly included in the HFC URB Macintyre Brook model and subsequently not included in the TUFLOW model accurately.	Will be necessary to review the impacts associated with the revised modelling and downstream of validation unless there are no significant differences to the hydraulic model results as a result of the revised hydrology.	Medium	Responded to in Technical Note.	It will be necessary to review the impacts associated with the revised modelling and downstream of validation unless there are no significant differences to the hydraulic model results as a result of the revised hydrology.	Accepted subject to ATTC committing to Panel's recommendations being addressed in Detailed Design.	Include the revised hydrology in the Revised Draft ES.	Refer to Section 9.4 of Appendix T1 - Flooding and Hydrology Technical Report.	The model was not subjected to both a set of parameters chosen that a series used for calibration. The model was not subjected to both a set of parameters chosen that a series used for calibration. The model was not subjected to both a set of parameters chosen that a series used for calibration.	Closed. No further action.	
B2G-V3	Validation to Ingebrook	Local catchment flows were generally not included within the hydraulic model as separate inflows. They were lumped together with larger catchment flows instead.	Lumping of the flow into Macintyre Brook potentially causes local catchment flow peaks and impacts to be unrepresented. There is also no evidence of regional catchment levels being applied as downstream boundary conditions in the local catchment models.	Low	To be addressed in Detailed Design	Local catchment flows to be delineated in Detailed Design.	Accepted subject to ATTC committing to Panel's recommendations being addressed in Detailed Design.	Local catchment flows to be delineated in Detailed Design.	To be addressed in Detailed Design. Refer to Section 13.6 of Appendix T1 - Flooding and Hydrology Technical Report.	The Panel accepts the commitment to address the issue in Detailed Design.	Open. Action required during detailed design.	To be actioned during detailed design
B2G-V4	Validation to Ingebrook	Local catchment flows were generally not included within the hydraulic model as separate inflows. They were lumped together with larger catchment flows instead.	Lumping of the flow into Macintyre Brook potentially causes local catchment flow peaks and impacts to be unrepresented. There is also no evidence of regional catchment levels being applied as downstream boundary conditions in the local catchment models.	Very High	To be addressed in Detailed Design	Local catchment flows to be delineated in Detailed Design.	Accepted subject to ATTC committing to Panel's recommendations being addressed in Detailed Design.	Local catchment flows to be delineated in Detailed Design.	To be addressed in Detailed Design. Refer to Section 13.6 of Appendix T1 - Flooding and Hydrology Technical Report.	The Panel accepts the commitment to address the issue in Detailed Design.	Closed. No further action.	

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Hydrology and Flooding Technical Report—Volume 1

Appendix B Cross-drainage register

BORDER TO GOWRIE REVISED DRAFT ENVIRONMENTAL IMPACT STATEMENT

Rail Cross Drainage Register

Table 1 Catchment Runoff Details

Culvert ID	Chainage	Type	Number	Diameter/Width (m) – Culverts Span (m) - Bridges	Height (m) – Culverts Soffit Level (m AHD) - Bridges	Length (m)	Catchment Area (ha)	Catchment Type
C31.26 (NS2B)	31256 (NS2B)	RCP	10	1.80		32.09	N/A – MacIntyre River Floodplain	Regional
C31.32 (NS2B)	31324 (NS2B)	RCP	10	1.80		30.02	N/A – MacIntyre River Floodplain	Regional
270-BR12	31520 (NS2B)	BRIDGE		140.00	227.46		N/A – MacIntyre River Floodplain	Regional
C31.87 (NS2B)	31875 (NS2B)	RCP	15	0.90		14.29	N/A – MacIntyre River Floodplain	Regional
C32.00 (NS2B)	32000 (NS2B)	RCP	15	0.90		15.13	N/A – MacIntyre River Floodplain	Regional
270-BR13	32550 (NS2B)	BRIDGE		546.00	225.71		N/A - MacIntyre River Floodplain	Regional
C34.64 (NS2B)	34639 (NS2B)	RCBC	9	3.00	1.20	16.03	1296.5	Local
C34.70 (NS2B)	34700 (NS2B)	RCBC	19	3.00	1.50	18.12		Local
C34.75 (NS2B)	34752 (NS2B)	RCBC	10	3.00	1.20	17.24		Local
C34.79 (NS2B)	34785 (NS2B)	RCBC	7	3.00	1.20	14.67		Local
C35.03 (NS2B)	35026 (NS2B)	RCBC	20	1.20	0.90	16.44	320.2	Local
C35.08 (NS2B)	35077 (NS2B)	RCBC	25	1.20	0.90	16.72		Local
C35.21 (NS2B)	35207 (NS2B)	RCBC	20	1.20	0.90	17.79		Local
C35.54 (NS2B)	35535 (NS2B)	RCBC	4	1.20	0.90	18.99		Local
C35.61 (NS2B)	35606 (NS2B)	RCBC	4	1.20	0.90	19.20		Local
C35.7 (NS2B)	35701 (NS2B)	RCBC	4	1.20	0.90	20.34		Local
C35.88 (NS2B)	35879 (NS2B)	RCP	7	0.90		15.34	804.8	Local
C35.91 (NS2B)	35906 (NS2B)	RCP	7	0.90		14.50	94.3	Local
C36.04 (NS2B)	36037 (NS2B)	RCP	6	0.90		13.70		Local
C36.22 (NS2B)	36222 (NS2B)	RCP	7	0.90		15.20		Local
C6.60	6602	RCBC	5	1.80	0.90	8.50	69.8	Local
C8.33	8330	RCBC	8	1.20	1.20	9.66	109.8	Local
C13.00	13001	RCBC	18	2.40	0.90	10.24	630.1	Local
C17.89	17887	RCP	2	1.05		22.21	11.0	Local
C18.51	18507	RCP	2	1.20		20.33	12.1	Local
C18.87	18871	RCP	7	1.05		20.34	43.6	Local
C20.00	19996	RCBC	6	2.10	0.90	11.11	59.3	Local
C22.43	22425	RCBC	11	1.20	0.90	9.09	104.1	Local
C23.05	23047	RCBC	8	1.20	0.90	9.58	36.1	Local
C23.53	23531	RCBC	10	1.20	0.90	8.79	53.4	Local
C24.41	24411	RCBC	3	1.20	0.90	8.92	26.2	Local
C24.84	24840	RCBC	1	1.20	0.90	10.39	13.9	Local
C25.19	25186	RCBC	6	1.20	0.60	12.17	N/A - MacIntyre Brook Floodplain	Regional
C25.46	25459	RCP	21	0.90		10.75	N/A - MacIntyre Brook Floodplain	Regional
C25.50	25504	RCBC	8	2.40	0.90	11.33	N/A - MacIntyre Brook Floodplain	Regional
C25.80	25803	RCBC	24	2.40	0.90	9.26	N/A - MacIntyre Brook Floodplain	Regional

Culvert ID	Chainage	Type	Number	Diameter/Width (m) – Culverts Span (m) - Bridges	Height (m) – Culverts Soffit Level (m AHD) - Bridges	Length (m)	Catchment Area (ha)	Catchment Type
C25.87	25874	RCBC	24	2.40	0.90	9.13	N/A - MacIntyre Brook Floodplain	Regional
C25.95	25946	RCBC	1	3.00	0.50	12.48	N/A - MacIntyre Brook Floodplain	Regional
C25.97	25966	RCBC	1	3.00	0.50	11.85	N/A - MacIntyre Brook Floodplain	Regional
C26.05	26048	RCBC	1	1.20	0.30	16.77	N/A - MacIntyre Brook Floodplain	Regional
C27.05	27051	RCBC	15	1.50	1.20	8.52	N/A - MacIntyre Brook Floodplain	Regional
C27.15	27146	RCBC	15	1.50	1.20	8.58	N/A - MacIntyre Brook Floodplain	Regional
C27.24	27238	RCBC	25	1.50	1.20	9.33	N/A - MacIntyre Brook Floodplain	Regional
C27.33	27332	RCBC	25	1.50	1.20	9.43	N/A - MacIntyre Brook Floodplain	Regional
C27.42	27421	RCBC	20	1.50	1.20	9.76	N/A - MacIntyre Brook Floodplain	Regional
C30.60	30596	RCBC	8	3.00	1.20	12.18	448.3	Local
C31.40	31398	RCP	3	1.20		11.44	16.7	Local
C31.44	31439	RCP	6	1.35		11.27	80.0	Local
C32.03	32027	RCP	13	0.90		10.59	71.0	Local
C32.82	32820	RCP	8	0.90		10.65	12.5	Local
C33.67	33665	RCP	9	1.50		13.07	54.1	Local
C35.18	35176	RCP	16	1.80		13.20	497.9	Local
C41.01	41009	RCP	6	1.20		13.05	348.7	Local
C41.11	41112	RCP	18	1.20		12.30		Local
C41.20	41197	RCP	14	1.20		10.03		Local
C41.32	41318	RCP	8	1.20		11.14		Local
C42.87	42873	RCP	15	0.90		11.97	N/A - MacIntyre Brook Floodplain	Regional
C43.02	43016	RCP	15	1.20		15.66	N/A - MacIntyre Brook Floodplain	Regional
C43.09	43086	RCP	30	1.20		16.20	N/A - MacIntyre Brook Floodplain	Regional
C43.16	43158	RCBC	9	3.00	1.80	15.34	N/A - MacIntyre Brook Floodplain	Regional
C43.35	43346	RCP	45	1.20		14.96	N/A - MacIntyre Brook Floodplain	Regional
C43.56	43555	RCP	10	1.20		15.06	N/A - MacIntyre Brook Floodplain	Regional
C43.66	43660	RCP	15	1.20		14.97	N/A - MacIntyre Brook Floodplain	Regional
C43.77	43767	RCP	15	1.20		15.28	N/A - MacIntyre Brook Floodplain	Regional
C43.86	43864	RCP	15	1.20		15.39	N/A - MacIntyre Brook Floodplain	Regional
C43.97	43970	RCP	15	1.20		14.80	N/A - MacIntyre Brook Floodplain	Regional
C44.32	44323	RCP	15	1.20		15.60	N/A - MacIntyre Brook Floodplain	Regional
C44.67	44670	RCP	15	1.20		25.72	N/A - MacIntyre Brook Floodplain	Regional
C44.78	44781	RCP	30	0.90		17.29	N/A - MacIntyre Brook Floodplain	Regional
C44.88	44878	RCP	30	0.90		17.62	N/A - MacIntyre Brook Floodplain	Regional
C44.96	44962	RCP	15	0.90		16.62	N/A - MacIntyre Brook Floodplain	Regional
C45.01	45005	RCP	35	0.90		16.42	N/A - MacIntyre Brook Floodplain	Regional
C45.10	45095	RCP	30	0.90		16.75	N/A - MacIntyre Brook Floodplain	Regional
C45.24	45237	RCP	35	0.90		15.48	N/A - MacIntyre Brook Floodplain	Regional
C45.30	45300	RCP	35	0.90		16.00	N/A - MacIntyre Brook Floodplain	Regional

Culvert ID	Chainage	Type	Number	Diameter/Width (m) – Culverts Span (m) - Bridges	Height (m) – Culverts Soffit Level (m AHD) - Bridges	Length (m)	Catchment Area (ha)	Catchment Type
C45.40	45395	RCP	40	0.90		16.37	N/A - MacIntyre Brook Floodplain	Regional
C45.45	45452	RCP	40	0.90		16.50	N/A - MacIntyre Brook Floodplain	Regional
C45.53	45525	RCP	40	0.90		16.13	N/A - MacIntyre Brook Floodplain	Regional
C45.64	45642	RCP	7	0.90		15.30	N/A - MacIntyre Brook Floodplain	Regional
C46.46	46464	RCP	11	1.20		20.00	87.1	Local
C48.41	48410	RCP	21	2.70		104.19	796.1	Local
C51.82	51825	RCBC	11	2.70	0.90	15.85	256.0	Local
310-BR03		BRIDGE		207.00	276.02		N/A - Cremascos Road Floodplain	Regional
C53.20	53202	RCP	9	1.20		15.23	32.0	Local
C53.62	53618	RCBC	4	1.20	0.90	10.00	13.7	Local
C54.44	54439	RCBC	3	1.20	0.90	8.50	8.0	Local
C55.06	55056	RCP	7	1.05		50.26	52.9	Local
310-BR04		BRIDGE		207.00	286.96		N/A - Bybera Road Floodplain	Regional
C60.18	60176	RCP	2	1.20		52.65	20.5	Local
C60.44	60443	RCP	18	2.70		89.38	195.4	Local
310-BR51		BRIDGE		75.00	308.59			Local
C61.60	61598	RCP	5	2.70		78.17	225.5	Local
C61.90	61899	RCP	5	2.70		61.98	221.3	Local
C62.94	62940	RCBC	3	2.40	2.10	20.00	51.4	Local
C63.15	63148	RCP	9	2.10		51.00	167.5	Local
C64.42	64415	RCP	6	0.90		10.78	35.0	Local
C64.48	64475	RCP	8	0.90		12.61		Local
C64.50	64496	RCP	12	0.90		13.20		Local
C65.11	65114	RCBC	15	2.10	0.90	9.34	84.5	Local
C66.23	66232	RCBC	8	2.40	1.80	59.54	N/A - Pariagara Creek Floodplain	Regional
C66.81	66814	RCBC	7	2.70	1.20	8.71	88.3	Local
310-BR05		BRIDGE		345.00	285.31		N/A - Pariagara Creek Floodplain	Regional
C67.57	67573	RCP	8	1.20		19.40	N/A - Pariagara Creek Floodplain	Regional
C67.64	67639	RCP	8	1.20		19.55	N/A - Pariagara Creek Floodplain	Regional
C67.70	67704	RCP	8	1.20		18.81	N/A - Pariagara Creek Floodplain	Regional
C67.83	67834	RCP	20	1.20		17.14	N/A - Pariagara Creek Floodplain	Regional
C67.96	67959	RCP	20	1.20		16.97	N/A - Pariagara Creek Floodplain	Regional
C68.09	68089	RCP	20	1.20		16.78	N/A - Pariagara Creek Floodplain	Regional
C68.41	68410	RCP	20	1.20		13.51	N/A - Pariagara Creek Floodplain	Regional
C68.76	68756	RCP	40	2.10		11.80	N/A - Pariagara Creek Floodplain	Regional
C68.89	68890	RCP	2	1.20		11.90	N/A - Pariagara Creek Floodplain	Regional
C69.02	69020	RCP	2	1.20		12.96	N/A - Pariagara Creek Floodplain	Regional
C69.10	69097	RCP	2	1.20		15.01	N/A - Pariagara Creek Floodplain	Regional
C69.14	69143	RCP	2	1.50		14.04	N/A - Pariagara Creek Floodplain	Regional

Culvert ID	Chainage	Type	Number	Diameter/Width (m) – Culverts Span (m) - Bridges	Height (m) – Culverts Soffit Level (m AHD) - Bridges	Length (m)	Catchment Area (ha)	Catchment Type
C69.21	69215	RCP	2	1.80		12.26	N/A - Pariagara Creek Floodplain	Regional
C69.28	69281	RCP	2	1.80		12.71	N/A - Pariagara Creek Floodplain	Regional
C69.41	69410	RCP	5	1.80		13.53	N/A - Pariagara Creek Floodplain	Regional
C69.54	69541	RCP	5	1.80		14.80	N/A - Pariagara Creek Floodplain	Regional
C69.67	69671	RCP	5	1.80		15.36	N/A - Pariagara Creek Floodplain	Regional
C69.80	69801	RCP	5	1.80		15.41	N/A - Pariagara Creek Floodplain	Regional
C70.50	70498	RCBC	3	1.20	1.20	9.75	19.8	Local
C71.51	71512	RCBC	3	2.40	0.90	8.99	31.2	Local
C73.37	73374	RCP	2	1.20		15.54	16.2	Local
C73.44	73441	RCP	1	1.20		17.51	3.9	Local
C73.53	73529	RCP	1	1.35		24.63	9.3	Local
C73.61	73615	RCP	2	0.90		22.50	1.2	Local
C73.68	73685	RCBC	3	3.00	1.80	29.89	167.8	Local
C74.97	74967	RCP	3	1.20		27.00	21.3	Local
C76.57	76572	RCBC	18	1.20	0.90	10.10	53.1	Local
C77.20	77197	RCP	7	1.50		13.66	48.6	Local
C77.47	77466	RCP	5	0.90		10.80	12.2	Local
C77.77	77771	RCBC	8	1.50	1.50	9.20	48.3	Local
C78.28	78280	RCBC	4	2.10	0.90	10.06	16.9	Local
C79.01	79005	RCBC	6	2.40	1.80	27.86	105.0	Local
C79.53	79526	RCP	8	1.35		19.27	35.4	Local
C79.98	79978	RCBC	8	0.90	0.90	7.79	20.3	Local
C80.65	80647	RCBC	6	1.80	1.80	10.55	47.1	Local
C81.18	81184	RCBC	7	2.40	2.40	17.67	82.2	Local
C82.35	82353	RCBC	18	2.40	2.40	16.45	203.6	Local
C83.51	83507	RCBC	12	2.40	1.80	8.44	57.7	Local
C84.38	84381	RCBC	24	3.00	2.70	17.00	419.7	Local
C85.35	85353	RCBC	7	2.40	1.20	9.13	79.8	Local
C87.18	87181	RCP	6	2.10		27.18	N/A - Cattle Creek Floodplain	Regional
C87.37	87368	RCBC	15	2.40	1.80	23.13	N/A - Cattle Creek Floodplain	Regional
C87.54	87542	RCBC	3	1.80	1.80	11.00	24.4	Local
C88.11	88111	RCP	3	1.20		17.50	6.4	Local
310-BR06		BRIDGE		138.00	327.38		N/A - Cattle Creek Floodplain	Regional
C90.96	90963	RCBC	40	2.40	2.40	36.36	477.6	Local
C92.08	92080	RCBC	18	2.40	1.50	9.20	198.8	Local
C92.94	92939	RCP	20	1.50		12.45	137.9	Local
C93.61	93606	RCBC	6	3.00	2.10	11.80	65.9	Local
310-BR07		BRIDGE		184.00	325.42		N/A - Native Dog Creek Floodplain	Regional
C94.91	94910	RCBC	5	2.10	0.90	9.14	38.2	Local

Culvert ID	Chainage	Type	Number	Diameter/Width (m) – Culverts Span (m) - Bridges	Height (m) – Culverts Soffit Level (m AHD) - Bridges	Length (m)	Catchment Area (ha)	Catchment Type
C95.07	95065	RCBC	18	2.40	1.80	10.30	389.8	Local
C96.20	96197	RCBC	14	2.40	1.20	9.82	134.2	Local
C97.01	97008	RCBC	5	1.50	1.20	21.11	28.0	Local
C97.29	97293	RCP	12	0.90		36.19	N/A - Bringalily Creek Floodplain	Regional
C97.38	97382	RCP	12	0.90		41.86	N/A - Bringalily Creek Floodplain	Regional
310-BR08		BRIDGE		299.00	332.79		N/A - Bringalily Creek Floodplain	Regional
C97.84	97836	RCP	8	1.50		34.10	N/A - Bringalily Creek Floodplain	Regional
C98.36	98360	RCP	10	0.90		31.18	N/A - Bringalily Creek Floodplain	Regional
C98.87	98866	RCP	1	1.50		17.50	20.5	Local
C99.37	99373	RCP	17	0.90		24.03	N/A - Bringalily Creek Floodplain	Regional
C99.84	99841	RCP	14	0.90		20.76	N/A - Bringalily Creek Floodplain	Regional
C99.77	99765	RCP	1	1.50		18.21	N/A - Bringalily Creek Floodplain	Regional
C100.00	100002	RCP	8	1.50		18.02	N/A - Bringalily Creek Floodplain	Regional
310-BR10		BRIDGE		621.00	334.54		N/A - Bringalily Creek Floodplain	Regional
C101.48	101485	RCBC	4	1.50	1.20	9.60	5.8	Local
C102.55	102546	RCBC	4	1.50	0.90	8.34	17.9	Local
310-BR11		BRIDGE		92.00	353.90		N/A - Nicol Creek Floodplain	Regional
C104.94	104938	RCP	18	0.90		10.00	N/A - Nicol Creek Floodplain	Regional
C105.09	105093	RCP	6	0.90		15.51	N/A - Nicol Creek Floodplain	Regional
C105.11	105110	RCP	6	0.90		16.33	N/A - Nicol Creek Floodplain	Regional
C105.13	105126	RCP	6	0.90		16.60	N/A - Nicol Creek Floodplain	Regional
C106.54	106541	RCBC	6	1.20	1.20	8.09	30.7	Local
C107.22	107222	RCBC	4	2.40	1.80	16.50	86.4	Local
C107.81	107807	RCBC	3	0.90	0.90	8.50	5.2	Local
C107.96	107964	RCP	4	1.20		14.34	30.8	Local
C108.45	108454	RCP	4	1.50		37.42	22.8	Local
C109.43	109430	RCBC	10	1.80	1.80	16.18	84.8	Local
C110.91	110912	RCBC	2	1.80	1.50	9.46	10.2	Local
C111.16	111165	RCBC	3	1.20	1.20	8.10	3.9	Local
C111.26	111259	RCBC	4	1.20	0.90	8.39	15.9	Local
C112.32	112325	RCBC	2	1.80	1.50	11.10	11.3	Local
C113.00	113001	RCBC	4	1.20	1.20	14.80	12.3	Local
C113.07	113073	RCBC	2	2.40	2.40	13.14	15.8	Local
C113.28	113281	RCBC	12	2.10	0.90	8.92	55.2	Local
C114.27	114269	RCP	11	0.90		12.79	5.8	Local
C114.36	114360	RCBC	10	2.10	1.50	10.50	212.4	Local
C114.90	114900	RCBC	2	1.80	1.50	15.26	12.9	Local
C115.00	115003	RCP	3	0.90		22.00	7.2	Local
C115.32	115322	RCP	3	0.90		29.50	6.4	Local

Culvert ID	Chainage	Type	Number	Diameter/Width (m) – Culverts Span (m) - Bridges	Height (m) – Culverts Soffit Level (m AHD) - Bridges	Length (m)	Catchment Area (ha)	Catchment Type
310-BR28		BRIDGE		75.00	456.88		66.1	Local
C117.40	117401	RCBC	13	3.00	1.80	16.32	156.0	Local
C117.60	117600	RCBC	4	2.70	2.10	11.95	85.0	Local
C117.71	117706	RCBC	1	1.50	1.50	10.52	1.2	Local
C118.10	118099	RCBC	15	2.10	0.90	8.00	47.8	Local
C118.43	118430	RCBC	10	3.00	1.50	8.68	164.5	Local
C118.61	118606	RCBC	10	1.20	1.20	9.56	3.5	Local
C118.90	118903	RCP	10	0.90		17.50	14.9	Local
C119.04	119036	RCBC	2	2.40	1.80	17.00	16.1	Local
C119.30	119298	RCBC	15	1.20	1.20	19.00	5.6	Local
C119.38	119378	RCBC	10	3.00	1.50	19.00	143.0	Local
C119.75	119751	RCBC	2	2.40	1.20	10.60	17.4	Local
C119.88	119875	RCBC	8	1.50	1.20	8.90	18.4	Local
C120.08	120080	RCBC	5	1.50	0.90	10.10	2.6	Local
C120.25	120252	RCBC	26	2.40	1.50	8.80	661.8	Local
C120.31	120310	RCBC	3	2.40	1.80	12.13		Local
C120.77	120765	RCBC	13	2.40	1.80	11.54	198.9	Local
C124.44	124442	RCP	10	1.80		53.00	33.9	Local
C125.48	125483	RCP	10	1.80		72.00	199.7	Local
C125.72	125716	RCP	1	0.90		50.26	4.0	Local
C125.82	125824	RCP	1	1.80		30.50	7.2	Local
C126.77	126767	RCP	12	0.90		45.81	N/A - Back Creek Floodplain	Regional
C126.81	126807	RCP	12	0.90		46.82	N/A - Back Creek Floodplain	Regional
310-BR37		BRIDGE		167.00	417.36		N/A - Back Creek Floodplain	Regional
310-BR38		BRIDGE		230.00	407.41		N/A - Back Creek Floodplain	Regional
C128.89	128893	RCBC	28	1.20	0.90	10.60	114.5	Local
C133.02	133020	RCBC	20	1.20	0.90	15.80	85.8	Local
C135.72	135716	RCBC	10	1.80	0.90	12.66	371.3	Local
C135.79	135790	RCBC	10	1.80	0.90	11.57		Local
C135.91	135910	RCBC	35	1.80	0.90	9.24		Local
C139.13	139129	RCP	8	1.35		23.78	N/A - Condamine River Floodplain	Regional
C139.17	139170	RCP	11	1.65		20.56	N/A - Condamine River Floodplain	Regional
C139.21	139205	RCP	8	1.80		19.27	N/A - Condamine River Floodplain	Regional
310-BR21/22		BRIDGE		1330.00	382.05		N/A - Condamine River Floodplain	Regional
C140.66	140663	RCP	11	1.80		12.58	N/A - Condamine River Floodplain	Regional
C140.73	140730	RCP	8	2.10		13.69	N/A - Condamine River Floodplain	Regional
C140.79	140787	RCP	8	2.10		13.46	N/A - Condamine River Floodplain	Regional
C140.85	140846	RCP	11	1.80		10.98	N/A - Condamine River Floodplain	Regional
C141.00	141001	RCP	9	1.65		16.49	N/A - Condamine River Floodplain	Regional

Culvert ID	Chainage	Type	Number	Diameter/Width (m) – Culverts Span (m) - Bridges	Height (m) – Culverts Soffit Level (m AHD) - Bridges	Length (m)	Catchment Area (ha)	Catchment Type
C141.02	141025	RCBC	4	2.40	2.10	24.23	N/A - Condamine River Floodplain	Regional
C141.07	141068	RCP	10	2.10		19.23	N/A - Condamine River Floodplain	Regional
C141.38	141377	RCP	7	1.80		16.34	N/A - Condamine River Floodplain	Regional
C141.40	141401	RCP	7	1.80		17.19	N/A - Condamine River Floodplain	Regional
C141.46	141460	RCP	6	2.10		12.73	N/A - Condamine River Floodplain	Regional
C141.49	141494	RCP	6	2.10		12.59	N/A - Condamine River Floodplain	Regional
C141.52	141521	RCP	6	2.10		12.64	N/A - Condamine River Floodplain	Regional
C141.54	141541	RCP	6	2.10		12.73	N/A - Condamine River Floodplain	Regional
C141.56	141563	RCP	6	2.10		12.81	N/A - Condamine River Floodplain	Regional
C141.61	141609	RCP	6	2.10		11.31	N/A - Condamine River Floodplain	Regional
C141.67	141665	RCP	6	2.10		11.71	N/A - Condamine River Floodplain	Regional
C141.69	141689	RCP	6	2.10		12.94	N/A - Condamine River Floodplain	Regional
C141.72	141721	RCP	7	1.80		12.67	N/A - Condamine River Floodplain	Regional
C141.75	141753	RCP	5	2.10		12.96	N/A - Condamine River Floodplain	Regional
C141.78	141778	RCP	6	2.10		13.25	N/A - Condamine River Floodplain	Regional
C141.80	141801	RCP	6	2.10		13.21	N/A - Condamine River Floodplain	Regional
C141.84	141842	RCP	5	2.10		13.39	N/A - Condamine River Floodplain	Regional
C141.88	141879	RCP	5	2.10		13.71	N/A - Condamine River Floodplain	Regional
C141.93	141926	RCP	6	2.10		13.30	N/A - Condamine River Floodplain	Regional
C141.96	141963	RCP	5	2.10		13.94	N/A - Condamine River Floodplain	Regional
C142.07	142072	RCP	6	2.10		14.06	N/A - Condamine River Floodplain	Regional
C142.12	142118	RCP	6	2.10		14.16	N/A - Condamine River Floodplain	Regional
C142.16	142164	RCP	4	2.10		13.84	N/A - Condamine River Floodplain	Regional
C142.20	142203	RCP	6	2.10		14.28	N/A - Condamine River Floodplain	Regional
C142.27	142267	RCP	6	2.10		14.27	N/A - Condamine River Floodplain	Regional
C142.32	142320	RCP	4	2.10		14.63	N/A - Condamine River Floodplain	Regional
C142.36	142362	RCP	6	2.10		14.66	N/A - Condamine River Floodplain	Regional
C142.40	142405	RCP	6	2.10		14.87	N/A - Condamine River Floodplain	Regional
C142.49	142489	RCP	6	2.10		14.48	N/A - Condamine River Floodplain	Regional
C142.53	142533	RCP	6	2.10		14.38	N/A - Condamine River Floodplain	Regional
C142.58	142579	RCP	6	2.10		14.53	N/A - Condamine River Floodplain	Regional
C142.61	142614	RCP	4	2.10		15.29	N/A - Condamine River Floodplain	Regional
310-BR24		BRIDGE		658.00	382.05		N/A - Condamine River Floodplain	Regional
C143.29	143288	RCP	6	2.10		14.17	N/A - Condamine River Floodplain	Regional
C143.33	143334	RCP	6	2.10		14.30	N/A - Condamine River Floodplain	Regional
C143.37	143372	RCP	6	2.10		14.37	N/A - Condamine River Floodplain	Regional
C143.42	143417	RCP	6	2.10		13.35	N/A - Condamine River Floodplain	Regional
C143.44	143445	RCP	6	2.10		13.36	N/A - Condamine River Floodplain	Regional
C143.48	143478	RCP	6	2.10		13.26	N/A - Condamine River Floodplain	Regional

Culvert ID	Chainage	Type	Number	Diameter/Width (m) – Culverts Span (m) - Bridges	Height (m) – Culverts Soffit Level (m AHD) - Bridges	Length (m)	Catchment Area (ha)	Catchment Type
C143.51	143509	RCP	6	2.10		13.82	N/A - Condamine River Floodplain	Regional
C143.54	143538	RCP	6	2.10		12.93	N/A - Condamine River Floodplain	Regional
C143.57	143574	RCP	5	2.10		12.65	N/A - Condamine River Floodplain	Regional
C143.65	143654	RCP	6	2.10		12.72	N/A - Condamine River Floodplain	Regional
C143.70	143701	RCP	6	2.10		12.75	N/A - Condamine River Floodplain	Regional
C143.73	143734	RCP	6	2.10		12.58	N/A - Condamine River Floodplain	Regional
C143.77	143768	RCP	6	2.10		12.59	N/A - Condamine River Floodplain	Regional
C143.79	143792	RCP	5	2.10		12.77	N/A - Condamine River Floodplain	Regional
C143.83	143832	RCP	4	2.10		13.07	N/A - Condamine River Floodplain	Regional
C143.87	143875	RCP	5	2.10		13.84	N/A - Condamine River Floodplain	Regional
310-BR25/26		BRIDGE		2577.00	380.60		N/A - Condamine River Floodplain	Regional
C146.45	146452	RCBC	4	1.20	0.90	15.39	N/A - Condamine River Floodplain	Regional
C146.50	146497	RCBC	4	1.20	0.90	14.39	N/A - Condamine River Floodplain	Regional
C146.54	146536	RCBC	4	1.20	0.90	13.52	N/A - Condamine River Floodplain	Regional
C146.61	146610	RCBC	2	1.20	0.90	10.17	N/A - Condamine River Floodplain	Regional
C146.69	146686	RCBC	6	1.20	0.90	10.53	N/A - Condamine River Floodplain	Regional
C147.01	147013	RCBC	10	1.50	0.90	11.06	N/A - Condamine River Floodplain	Regional
C147.12	147116	RCBC	4	1.20	0.90	11.73	N/A - Condamine River Floodplain	Regional
C147.18	147178	RCBC	10	1.50	0.90	11.69	N/A - Condamine River Floodplain	Regional
C147.21	147213	RCBC	4	1.20	0.90	11.72	N/A - Condamine River Floodplain	Regional
C147.27	147265	RCBC	4	1.20	0.90	11.54	N/A - Condamine River Floodplain	Regional
C147.31	147315	RCBC	10	1.50	0.90	11.01	N/A - Condamine River Floodplain	Regional
C147.85	147851	RCBC	6	1.20	0.60	8.97	N/A - Condamine River Floodplain	Regional
C147.91	147914	RCBC	4	1.20	0.60	10.24	N/A - Condamine River Floodplain	Regional
C148.87	148874	RCP	25	1.05		12.19	N/A - Condamine River Floodplain	Regional
C148.92	148915	RCP	25	1.05		13.13	N/A - Condamine River Floodplain	Regional
C148.96	148956	RCP	25	1.05		14.37	N/A - Condamine River Floodplain	Regional
C149.02	149016	RCP	25	1.05		15.55	N/A - Condamine River Floodplain	Regional
310-BR27		BRIDGE		1568.00	383.79		N/A - Condamine River Floodplain	Regional
C150.68	150677	RCP	10	1.35		10.82	N/A - Condamine River Floodplain	Regional
C150.71	150708	RCP	12	1.20		10.76	N/A - Condamine River Floodplain	Regional
C150.74	150744	RCP	3	1.35		11.84	N/A - Condamine River Floodplain	Regional
C151.08	151077	RCP	8	1.20		10.93	N/A - Condamine River Floodplain	Regional
C151.12	151119	RCP	8	1.20		12.51	N/A - Condamine River Floodplain	Regional
C151.14	151142	RCP	8	1.20		12.44	N/A - Condamine River Floodplain	Regional
C151.17	151173	RCP	6	1.35		12.64	N/A - Condamine River Floodplain	Regional
C151.20	151201	RCP	6	1.35		12.21	N/A - Condamine River Floodplain	Regional
C151.27	151269	RCP	8	1.20		11.20	N/A - Condamine River Floodplain	Regional
C151.30	151296	RCP	8	1.05		11.49	N/A - Condamine River Floodplain	Regional

Culvert ID	Chainage	Type	Number	Diameter/Width (m) – Culverts Span (m) - Bridges	Height (m) – Culverts Soffit Level (m AHD) - Bridges	Length (m)	Catchment Area (ha)	Catchment Type
C152.40	152396	RCBC	3	0.90	0.90	9.00	6.6	Local
C153.44	153438	RCBC	2	0.90	0.90	10.80	11.5	Local
C154.51	154508	RCBC	12	2.10	0.90	8.22	236.9	Local
C155.59	155592	RCBC	9	2.10	1.20	9.30	221.3	Local
C159.25	159245	RCBC	5	2.40	1.20	7.44	100.2	Local
C160.41	160414	RCP	5	1.80		16.00	189.5	Local
C161.15	161152	RCP	8	1.20		32.00	32.9	Local
C162.30	162301	RCP	2	1.50		31.00	8.4	Local
310-BR29		BRIDGE		69.00	433.16		48.3	Local
C162.82	162824	RCP	3	1.80		33.00	34.9	Local
C164.16	164160	RCP	6	0.90		86.51	6.5	Local
C164.29	164288	RCP	22	1.20		70.21	166.4	Local
C165.06	165063	RCP	3	1.80		61.23	37.3	Local
C166.10	166104	RCBC	3	2.40	1.20	8.45	23.4	Local
C167.08	167082	RCBC	2	1.20	1.20	9.77	11.1	Local
C168.61	168611	RCBC	4	2.40	0.90	17.40	23.8	Local
C169.01	169013	RCBC	2	1.20	1.20	14.55	10.2	Local
C169.89	169886	RCP	5	2.70		37.35	103.6	Local
C170.55	170548	RCBC	10	2.40	2.40	25.20	162.0	Local
C171.02	171018	RCP	8	1.80		27.04	41.6	Local
C171.90	171902	RCP	15	1.80		70.72	138.2	Local
310-BR30		BRIDGE		69.00	513.94		40.6	Local
C173.54	173544	RCP	3	1.35		61.12	6.5	Local
310-BR40		BRIDGE		75.00	528.95		49.7	Local
C176.89	176892	RCBC	3	2.40	2.10	32.58	60.6	Local
C177.63	177635	RCP	1	2.10		25.27	27.4	Local
C178.02	178017	RCP	4	2.10		41.00	58.0	Local
C178.63	178632	RCP	7	1.80		12.50	83.7	Local
C181.21	181208	RCP	9	2.10		66.79	126.9	Local
C181.78	181784	RCP	4	1.80		48.01	42.9	Local
C182.99	182991	RCP	9	2.40		31.17	165.7	Local
C183.56	183558	RCP	6	2.10		31.00	48.6	Local
310-BR42		BRIDGE		144.00	530.43		527.9	Local
C186.16	186155	RCBC	10	2.40	2.10	14.08	170.9	Local
C187.20	187205	RCP	16	2.10		47.00	248.4	Local
C189.99	189990	RCBC	11	1.80	1.50	74.00	N/A - Westbrook Creek Floodplain	Regional
C192.09	192086	RCP	7	1.80		60.00	62.9	Local
C193.10	193103	RCP	5	3.00		109.86	N/A - Westbrook Creek Floodplain	Regional
C194.66	194659	RCBC	2	1.50	1.20	18.84	N/A - Westbrook Creek Floodplain	Regional

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C194.69	194691	RCP	3	1.05		23.60	N/A - Westbrook Creek Floodplain	Regional
C195.98	195978	RCBC	7	1.50	0.90	9.83	29.4	Local
C196.46	196465	RCP	5	1.20		53.00	26.2	Local
C196.92	196916	RCP	14	1.35		59.73	N/A - Westbrook Creek Floodplain	Regional
C197.18	197181	RCP	2	1.05		47.22	N/A - Westbrook Creek Floodplain	Regional
C197.31	197313	RCP	2	1.05		44.11	N/A - Westbrook Creek Floodplain	Regional
310-BR20		BRIDGE		92.00	440.15		N/A - Westbrook Creek Floodplain	Regional
310-BR31		BRIDGE		230.00	428.95		N/A - Westbrook Creek Floodplain	Regional
C198.68	198684	RCP	15	2.40		14.88	N/A - Westbrook Creek Floodplain	Regional
C198.77	198770	RCP	11	1.50		12.53	N/A - Westbrook Creek Floodplain	Regional
C198.80	198803	RCP	10	1.20		12.15	N/A - Westbrook Creek Floodplain	Regional
C198.98	198983	RCP	17	1.05		23.47	N/A - Westbrook Creek Floodplain	Regional
310-BR32		BRIDGE		184.00	431.25		N/A - Westbrook Creek Floodplain	Regional
C199.53	199533	RCP	15	1.05		65.53	N/A - Westbrook Creek Floodplain	Regional
310-BR33		BRIDGE		75.00	436.21		N/A - Westbrook Creek Floodplain	Regional
C200.52	200519	RCP	5	0.90		15.02	11.0	Local
C200.82	200824	RCBC	13	0.90	0.90	7.75	18.1	Local
C201.23	201233	RCBC	4	0.90	0.90	9.56	7.9	Local
C201.51	201514	RCBC	3	0.90	0.90	8.23	4.6	Local
C201.97	201974	RCP	5	1.20		24.77	23.3	Local
C202.52	202524	RCP	7	1.20		26.62	32.2	Local
C202.80	202802	RCBC	7	2.10	2.10	33.52	69.9	Local
310-BR34		BRIDGE		132.00	481.45		N/A - Gowrie Creek Floodplain	Regional
C204.45	204447	RCP	2	1.05		60.61	N/A - Gowrie Creek Floodplain	Regional
310-BR35		BRIDGE		299.00	463.43		N/A - Gowrie Creek Floodplain	Regional
C206.17	206167	RCP	2	1.05		98.63	N/A - Gowrie Creek Floodplain	Regional
C206.36	206359	RCP	12	1.05		89.11	N/A - Gowrie Creek Floodplain	Regional
C206.42	206421	RCP	2	1.05		88.91	N/A - Gowrie Creek Floodplain	Regional
C206.57	206574	RCP	4	1.05		78.24	N/A - Gowrie Creek Floodplain	Regional
C206.65	206651	RCP	15	1.05		68.62	N/A - Gowrie Creek Floodplain	Regional
C206.75	206746	RCP	5	1.05		46.97	N/A - Gowrie Creek Floodplain	Regional
C206.88	206876	RCP	2	1.05		35.61	N/A - Gowrie Creek Floodplain	Regional
C207.15	207146	RCP	7	1.35		14.73	N/A - Gowrie Creek Floodplain	Regional
C207.70	207705	RCBC	1	1.80	0.90	11.45	7.6	Local
C208.21	208208	RCBC	16	2.40	1.50	21.38	N/A - Gowrie Creek Floodplain	Regional

Table 2 Hydraulic Results

Culvert ID	Chainage	Type	Number	Diameter/Width(m) – Culverts Span (m) – Bridges	Height (m) – Culverts Soffit Level (m AHD) - Bridges	1% AEP Flow Through Structure (m³/s)	1% AEP Upstream Water Level- Design (m AHD)	1% AEP Upstream Headwater Depth- Design (m)	1% AEP Freeboard to Formation (m)	Impacts at Rail Corridor		
										1% AEP Afflux (m)	Existing Time of Inundation (hrs)	Change in Time of Inundation (hrs)
C31.26 (NS2B)	31256 (NS2B)	RCP	10	1.80		5.7	227.20	1.06	5.32	N/A - Regional	N/A - Regional	N/A - Regional
C31.32 (NS2B)	31324 (NS2B)	RCP	10	1.80		5.4	227.04	1.01	5.00	N/A - Regional	N/A - Regional	N/A - Regional
BR12	31520 (NS2B)	BRIDGE		140.00	227.46	67.5	226.76	N/A	4.65	N/A - Regional	N/A - Regional	N/A - Regional
C31.87 (NS2B)	31875 (NS2B)	RCP	15	0.90		0.0	0.00	Dry - location and ground level to be reviewed in DD	229.05	N/A - Regional	N/A - Regional	N/A - Regional
C32.00 (NS2B)	32000 (NS2B)	RCP	15	0.90		0.0	0.00	Dry - location and ground level to be reviewed in DD	229.05	N/A - Regional	N/A - Regional	N/A - Regional
BR13	32550 (NS2B)	BRIDGE		546.00	225.71	112.1	226.52	N/A	2.52	N/A - Regional	N/A - Regional	N/A - Regional
C34.64 (NS2B)	34639 (NS2B)	RCBC	9	3.00	1.20	10.0	225.70	0.75	2.14	0.03	6.90	-0.04
C34.70 (NS2B)	34700 (NS2B)	RCBC	19	3.00	1.50	15.0	225.69	0.86	1.96	0.03	7.04	0.00
C34.75 (NS2B)	34752 (NS2B)	RCBC	10	3.00	1.20	13.5	225.65	0.73	2.08	0.01	6.91	-0.13
C34.79 (NS2B)	34785 (NS2B)	RCBC	7	3.00	1.20	5.8	225.62	0.41	2.07	0.00	6.46	-1.25
C35.03 (NS2B)	35026 (NS2B)	RCBC	20	1.20	0.90	1.6	225.40	0.16	2.23	0.05	4.43	1.24
C35.08 (NS2B)	35077 (NS2B)	RCBC	25	1.20	0.90	3.2	225.42	0.25	2.28	0.07	6.70	0.51
C35.21 (NS2B)	35207 (NS2B)	RCBC	20	1.20	0.90	2.9	225.51	0.32	2.38	0.08	6.86	0.28
C35.54 (NS2B)	35535 (NS2B)	RCBC	4	1.20	0.90	1.5	226.21	0.40	2.15	0.00	7.28	0.25
C35.61 (NS2B)	35606 (NS2B)	RCBC	4	1.20	0.90	1.7	226.27	0.42	2.19	0.01	7.37	0.25
C35.7 (NS2B)	35701 (NS2B)	RCBC	4	1.20	0.90	1.3	226.32	0.33	2.27	0.03	7.51	0.41
C35.88 (NS2B)	35879 (NS2B)	RCP	7	0.90		5.5	227.46	1.19	1.26	0.08	11.31	-0.01
C35.91 (NS2B)	35906 (NS2B)	RCP	7	0.90		5.4	227.46	1.06	1.29	0.09	9.08	0.52
C36.04 (NS2B)	36037 (NS2B)	RCP	6	0.90		2.6	227.44	0.58	1.40	0.08	7.66	1.25
C36.22 (NS2B)	36222 (NS2B)	RCP	7	0.90		1.3	227.57	0.35	1.53	0.03	10.06	0.18
C6.60	6602	RCBC	5	1.80	0.90	7.6	234.07	0.77	0.83	0.03	5.08	1.31
C8.33	8330	RCBC	8	1.20	1.20	14.4	234.85	1.16	0.82	-0.12	6.58	-3.61
C13.00	13001	RCBC	18	2.40	0.90	40.2	237.81	0.84	1.42	0.02	6.66	0.36
C17.89	17887	RCP	2	1.05		3.5	240.28	1.43	1.25	-0.18	6.76	-6.35
C18.51	18507	RCP	2	1.20		4.0	241.66	1.30	0.98	0.00	6.61	-0.21
C18.87	18871	RCP	7	1.05		8.6	241.92	0.96	1.37	0.00	6.85	-3.29
C20.00	19996	RCBC	6	2.10	0.90	9.5	243.82	0.71	1.56	-0.06	6.78	-4.14
C22.43	22425	RCBC	11	1.20	0.90	12.8	244.55	0.86	1.02	0.09	6.85	1.16
C23.05	23047	RCBC	8	1.20	0.90	8.8	244.49	0.83	1.16	0.07	6.86	0.82
C23.53	23531	RCBC	10	1.20	0.90	10.2	244.90	0.78	0.80	0.07	6.82	0.64
C24.41	24411	RCBC	3	1.20	0.90	4.7	245.12	1.12	0.59	0.07	6.85	0.97
C24.84	24840	RCBC	1	1.20	0.90	2.0	244.97	1.39	0.64	0.08	4.82	1.93
C25.19	25186	RCBC	6	1.20	0.60	2.5	244.00	0.49	1.61	N/A - Regional	N/A - Regional	N/A - Regional
C25.46	25459	RCP	21	0.90		4.0	244.21	0.43	1.40	N/A - Regional	N/A - Regional	N/A - Regional
C25.50	25504	RCBC	8	2.40	0.90	6.5	244.20	0.58	1.41	N/A - Regional	N/A - Regional	N/A - Regional
C25.80	25803	RCBC	24	2.40	0.90	1.8	244.39	0.24	1.25	N/A - Regional	N/A - Regional	N/A - Regional

Culvert ID	Chainage	Type	Number	Diameter/Width(m) – Culverts Span (m) – Bridges	Height (m) – Culverts Soffit Level (m AHD) - Bridges	1% AEP Flow Through Structure (m³/s)	1% AEP Upstream Water Level- Design (m AHD)	1% AEP Upstream Headwater Depth- Design (m)	1% AEP Freeboard to Formation (m)	Impacts at Rail Corridor		
										1% AEP Afflux (m)	Existing Time of Inundation (hrs)	Change in Time of Inundation (hrs)
C25.87	25874	RCBC	24	2.40	0.90	5.6	244.42	0.47	1.23	N/A - Regional	N/A - Regional	N/A - Regional
C25.95	25946	RCBC	1	3.00	0.50	1.6	244.50	0.84	1.15	N/A - Regional	N/A - Regional	N/A - Regional
C25.97	25966	RCBC	1	3.00	0.50	1.8	244.52	0.85	1.14	N/A - Regional	N/A - Regional	N/A - Regional
C26.05	26048	RCBC	1	1.20	0.30	0.1	244.73	0.69	0.76	N/A - Regional	N/A - Regional	N/A - Regional
C27.05	27051	RCBC	15	1.50	1.20	7.6	245.56	0.58	1.08	N/A - Regional	N/A - Regional	N/A - Regional
C27.15	27146	RCBC	15	1.50	1.20	7.0	245.63	0.58	1.20	N/A - Regional	N/A - Regional	N/A - Regional
C27.24	27238	RCBC	25	1.50	1.20	9.2	245.91	0.70	1.10	N/A - Regional	N/A - Regional	N/A - Regional
C27.33	27332	RCBC	25	1.50	1.20	8.0	245.97	0.59	1.22	N/A - Regional	N/A - Regional	N/A - Regional
C27.42	27421	RCBC	20	1.50	1.20	9.2	245.91	0.47	1.46	N/A - Regional	N/A - Regional	N/A - Regional
C30.60	30596	RCBC	8	3.00	1.20	45.4	250.62	1.41	1.41	-0.07	6.50	-2.49
C31.40	31398	RCP	3	1.20		3.8	252.26	0.93	1.26	0.03	2.70	0.13
C31.44	31439	RCP	6	1.35		12.0	252.34	1.16	1.22	0.03	2.87	0.00
C32.03	32027	RCP	13	0.90		11.8	253.15	0.91	0.93	0.02	1.99	0.11
C32.82	32820	RCP	8	0.90		4.2	257.96	0.63	1.25	0.00	0.57	0.00
C33.67	33665	RCP	9	1.50		16.3	260.12	1.03	1.89	0.04	0.79	0.08
C35.18	35176	RCP	16	1.80		52.5	257.34	1.36	2.32	-0.09	2.59	-1.81
C41.01	41009	RCP	6	1.20		4.3	266.03	0.63	1.88	-0.25	2.32	-2.09
C41.11	41112	RCP	18	1.20		18.7	266.03	0.82	1.72	-0.17	2.40	0.00
C41.20	41197	RCP	14	1.20		14.7	266.03	0.82	1.58	-0.06	2.36	-1.21
C41.32	41318	RCP	8	1.20		8.7	266.03	0.79	1.40	-0.12	2.29	-1.60
C42.87	42873	RCP	15	0.90		6.0	263.79	0.79	1.30	N/A - Regional	N/A - Regional	N/A - Regional
C43.02	43016	RCP	15	1.20		10.8	263.82	1.72	1.31	N/A - Regional	N/A - Regional	N/A - Regional
C43.09	43086	RCP	30	1.20		9.9	263.82	1.72	1.33	N/A - Regional	N/A - Regional	N/A - Regional
C43.16	43158	RCBC	9	3.00	1.80	16.7	263.82	1.58	1.35	N/A - Regional	N/A - Regional	N/A - Regional
C43.35	43346	RCP	45	1.20		4.2	263.87	1.35	1.35	N/A - Regional	N/A - Regional	N/A - Regional
C43.56	43555	RCP	10	1.20		3.1	263.95	1.50	1.33	N/A - Regional	N/A - Regional	N/A - Regional
C43.66	43660	RCP	15	1.20		2.3	264.00	1.65	1.31	N/A - Regional	N/A - Regional	N/A - Regional
C43.77	43767	RCP	15	1.20		2.4	264.06	1.56	1.28	N/A - Regional	N/A - Regional	N/A - Regional
C43.86	43864	RCP	15	1.20		5.8	264.12	1.54	1.25	N/A - Regional	N/A - Regional	N/A - Regional
C43.97	43970	RCP	15	1.20		8.7	264.20	1.65	1.25	N/A - Regional	N/A - Regional	N/A - Regional
C44.32	44323	RCP	15	1.20		10.8	264.39	1.62	1.11	N/A - Regional	N/A - Regional	N/A - Regional
C44.67	44670	RCP	15	1.20		8.5	264.52	1.28	1.00	N/A - Regional	N/A - Regional	N/A - Regional
C44.78	44781	RCP	30	0.90		4.3	264.60	1.00	0.98	N/A - Regional	N/A - Regional	N/A - Regional
C44.88	44878	RCP	30	0.90		4.4	264.68	0.97	1.14	N/A - Regional	N/A - Regional	N/A - Regional
C44.96	44962	RCP	15	0.90		5.9	264.73	0.80	1.22	N/A - Regional	N/A - Regional	N/A - Regional
C45.01	45005	RCP	35	0.90		4.1	264.74	0.72	1.28	N/A - Regional	N/A - Regional	N/A - Regional
C45.10	45095	RCP	30	0.90		3.0	264.82	0.50	1.34	N/A - Regional	N/A - Regional	N/A - Regional
C45.24	45237	RCP	35	0.90		1.6	264.94	0.24	1.47	N/A - Regional	N/A - Regional	N/A - Regional

Culvert ID	Chainage	Type	Number	Diameter/Width(m) – Culverts Span (m) – Bridges	Height (m) – Culverts Soffit Level (m AHD) - Bridges	1% AEP Flow Through Structure (m³/s)	1% AEP Upstream Water Level- Design (m AHD)	1% AEP Upstream Headwater Depth- Design (m)	1% AEP Freeboard to Formation (m)	Impacts at Rail Corridor		
										1% AEP Afflux (m)	Existing Time of Inundation (hrs)	Change in Time of Inundation (hrs)
C45.30	45300	RCP	35	0.90		2.9	265.06	0.37	1.43	N/A - Regional	N/A - Regional	N/A - Regional
C45.40	45395	RCP	40	0.90		5.9	265.16	0.51	1.48	N/A - Regional	N/A - Regional	N/A - Regional
C45.45	45452	RCP	40	0.90		4.6	265.25	0.45	1.48	N/A - Regional	N/A - Regional	N/A - Regional
C45.53	45525	RCP	40	0.90		2.7	265.36	0.31	1.48	N/A - Regional	N/A - Regional	N/A - Regional
C45.64	45642	RCP	7	0.90		2.4	265.85	1.12	1.47	N/A - Regional	N/A - Regional	N/A - Regional
C46.46	46464	RCP	11	1.20		18.1	271.80	1.02	3.07	0.01	1.52	0.00
C48.41	48410	RCP	21	2.70		75.0	274.44	0.97	19.54	0.02	2.76	0.04
C51.82	51825	RCBC	11	2.70	0.90	30.9	279.43	0.93	0.80	0.10	2.84	0.19
310-BR03		BRIDGE		207.00	276.02	258.7	270.72	N/A	7.30	N/A - Regional	N/A - Regional	N/A - Regional
C53.20	53202	RCP	9	1.20		7.2	276.62	0.62	2.28	0.03	1.18	0.00
C53.62	53618	RCBC	4	1.20	0.90	3.9	280.20	0.75	0.78	0.02	0.98	0.00
C54.44	54439	RCBC	3	1.20	0.90	2.7	283.05	0.70	0.40	0.00	1.31	0.00
C55.06	55056	RCP	7	1.05		10.7	275.39	1.19	8.06	0.18	1.65	0.24
310-BR04		BRIDGE		207.00	286.96	261.7	273.76	N/A	15.20	N/A - Regional	N/A - Regional	N/A - Regional
C60.18	60176	RCP	2	1.20		5.9	308.79	1.92	8.82	0.00	0.00	0.00
C60.44	60443	RCP	18	2.70		40.9	294.25	0.98	20.01	0.00	1.64	0.00
310-BR51		BRIDGE		75.00	308.59	2.1	296.02	N/A	17.02	0.00	1.64	0.00
C61.60	61598	RCP	5	2.70		33.5	289.50	1.67	14.81	0.45	2.50	0.27
C61.90	61899	RCP	5	2.70		34.2	290.54	1.68	11.42	0.08	2.57	0.00
C62.94	62940	RCBC	3	2.40	2.10	12.0	289.71	1.16	4.14	0.00	1.25	0.00
C63.15	63148	RCP	9	2.10		27.1	286.01	1.19	8.72	0.34	2.48	0.19
C64.42	64415	RCP	6	0.90		3.4	298.46	0.61	1.16	0.00	1.25	0.00
C64.48	64475	RCP	8	0.90		3.2	297.40	0.48	1.80	0.02	1.24	0.06
C64.50	64496	RCP	12	0.90		0.4	296.87	0.14	2.19	0.18	0.82	0.24
C65.11	65114	RCBC	15	2.10	0.90	14.8	294.98	0.52	1.43	0.28	2.00	1.00
C66.23	66232	RCBC	8	2.40	1.80	43.0	285.35	2.77	4.84	N/A - Regional	N/A - Regional	N/A - Regional
C66.81	66814	RCBC	7	2.70	1.20	13.8	285.04	0.70	0.90	0.29	2.35	1.17
310-BR05		BRIDGE		345.00	285.31	514.9	285.09	N/A	2.22	N/A - Regional	N/A - Regional	N/A - Regional
C67.57	67573	RCP	8	1.20		0.2	285.34	0.84	1.46	N/A - Regional	N/A - Regional	N/A - Regional
C67.64	67639	RCP	8	1.20		2.4	285.45	0.89	1.43	N/A - Regional	N/A - Regional	N/A - Regional
C67.70	67704	RCP	8	1.20		3.4	285.49	0.80	1.47	N/A - Regional	N/A - Regional	N/A - Regional
C67.83	67834	RCP	20	1.20		8.3	285.60	0.57	1.51	N/A - Regional	N/A - Regional	N/A - Regional
C67.96	67959	RCP	20	1.20		10.3	285.68	0.58	1.57	N/A - Regional	N/A - Regional	N/A - Regional
C68.09	68089	RCP	20	1.20		11.9	285.78	0.58	1.63	N/A - Regional	N/A - Regional	N/A - Regional
C68.41	68410	RCP	20	1.20		8.3	286.41	0.45	1.52	N/A - Regional	N/A - Regional	N/A - Regional
C68.76	68756	RCP	40	2.10		39.5	286.99	1.52	1.49	N/A - Regional	N/A - Regional	N/A - Regional
C68.89	68890	RCP	2	1.20		1.0	287.11	0.61	1.40	N/A - Regional	N/A - Regional	N/A - Regional
C69.02	69020	RCP	2	1.20		2.0	287.25	0.86	1.28	N/A - Regional	N/A - Regional	N/A - Regional

Culvert ID	Chainage	Type	Number	Diameter/Width(m) – Culverts Span (m) – Bridges	Height (m) – Culverts Soffit Level (m AHD) - Bridges	1% AEP Flow Through Structure (m³/s)	1% AEP Upstream Water Level- Design (m AHD)	1% AEP Upstream Headwater Depth- Design (m)	1% AEP Freeboard to Formation (m)	Impacts at Rail Corridor		
										1% AEP Afflux (m)	Existing Time of Inundation (hrs)	Change in Time of Inundation (hrs)
C69.10	69097	RCP	2	1.20		2.9	287.29	1.13	1.26	N/A - Regional	N/A - Regional	N/A - Regional
C69.14	69143	RCP	2	1.50		4.0	287.31	1.28	1.25	N/A - Regional	N/A - Regional	N/A - Regional
C69.21	69215	RCP	2	1.80		5.4	287.33	1.35	1.24	N/A - Regional	N/A - Regional	N/A - Regional
C69.28	69281	RCP	2	1.80		5.4	287.34	1.38	1.27	N/A - Regional	N/A - Regional	N/A - Regional
C69.41	69410	RCP	5	1.80		13.6	287.35	1.53	1.26	N/A - Regional	N/A - Regional	N/A - Regional
C69.54	69541	RCP	5	1.80		13.5	287.36	1.58	1.27	N/A - Regional	N/A - Regional	N/A - Regional
C69.67	69671	RCP	5	1.80		13.8	287.38	1.64	1.27	N/A - Regional	N/A - Regional	N/A - Regional
C69.80	69801	RCP	5	1.80		13.7	287.38	1.64	1.30	N/A - Regional	N/A - Regional	N/A - Regional
C70.50	70498	RCBC	3	1.20	1.20	4.3	290.02	0.97	0.59	0.06	1.26	0.44
C71.51	71512	RCBC	3	2.40	0.90	5.7	297.72	0.74	0.45	0.00	1.77	0.00
C73.37	73374	RCP	2	1.20		4.9	297.42	1.55	1.36	0.00	0.86	-0.01
C73.44	73441	RCP	1	1.20		1.3	295.74	0.95	2.88	0.00	0.72	-0.01
C73.53	73529	RCP	1	1.35		2.7	294.49	1.47	3.93	0.00	1.15	0.00
C73.61	73615	RCP	2	0.90		0.7	294.21	0.46	4.01	0.00	0.13	0.00
C73.68	73685	RCBC	3	3.00	1.80	25.4	293.18	1.85	4.88	0.23	2.72	0.00
C74.97	74967	RCP	3	1.20		5.4	305.15	1.15	3.27	0.00	1.09	0.00
C76.57	76572	RCBC	18	1.20	0.90	15.6	308.02	0.69	0.84	0.25	1.40	0.11
C77.20	77197	RCP	7	1.50		13.0	307.19	0.99	1.72	0.09	1.78	0.09
C77.47	77466	RCP	5	0.90		4.2	308.07	0.84	0.86	0.12	0.57	0.38
C77.77	77771	RCBC	8	1.50	1.50	12.3	308.38	0.88	0.57	0.31	0.88	1.23
C78.28	78280	RCBC	4	2.10	0.90	5.0	309.61	0.61	1.18	0.00	4.39	0.00
C79.01	79005	RCBC	6	2.40	1.80	22.0	307.60	1.10	3.59	0.00	2.69	0.05
C79.53	79526	RCP	8	1.35		9.5	307.71	0.82	3.76	0.27	4.04	0.33
C79.98	79978	RCBC	8	0.90	0.90	6.4	311.38	0.80	0.34	0.00	0.97	0.00
C80.65	80647	RCBC	6	1.80	1.80	12.7	312.02	0.96	1.34	0.24	1.03	0.13
C81.18	81184	RCBC	7	2.40	2.40	18.5	310.73	1.00	3.99	0.28	4.68	0.42
C82.35	82353	RCBC	18	2.40	2.40	52.3	314.08	0.99	3.31	0.26	4.21	0.78
C83.51	83507	RCBC	12	2.40	1.80	14.8	318.72	0.55	1.30	0.01	2.61	0.00
C84.38	84381	RCBC	24	3.00	2.70	71.8	316.83	0.60	3.80	0.44	2.88	0.85
C85.35	85353	RCBC	7	2.40	1.20	16.0	323.72	0.85	0.45	0.16	1.13	0.65
C87.18	87181	RCP	6	2.10		4.3	323.61	0.38	3.26	N/A - Regional	N/A - Regional	N/A - Regional
C87.37	87368	RCBC	15	2.40	1.80	51.2	321.86	1.26	3.52	N/A - Regional	N/A - Regional	N/A - Regional
C87.54	87542	RCBC	3	1.80	1.80	5.1	324.36	0.82	1.60	0.00	2.13	0.00
C88.11	88111	RCP	3	1.20		2.2	325.05	0.62	2.90	0.07	3.66	0.11
310-BR06		BRIDGE		138.00	327.38	239.5	323.91	N/A	5.46	N/A - Regional	N/A - Regional	N/A - Regional
C90.96	90963	RCBC	40	2.40	2.40	110.6	330.47	0.94	5.90	0.45	1.69	0.87
C92.08	92080	RCBC	18	2.40	1.50	45.7	333.09	0.89	0.64	0.01	1.54	0.01
C92.94	92939	RCP	20	1.50		25.4	328.49	0.78	1.91	0.23	3.45	0.73

Culvert ID	Chainage	Type	Number	Diameter/Width(m) – Culverts Span (m) – Bridges	Height (m) – Culverts Soffit Level (m AHD) - Bridges	1% AEP Flow Through Structure (m³/s)	1% AEP Upstream Water Level- Design (m AHD)	1% AEP Upstream Headwater Depth- Design (m)	1% AEP Freeboard to Formation (m)	Impacts at Rail Corridor		
										1% AEP Afflux (m)	Existing Time of Inundation (hrs)	Change in Time of Inundation (hrs)
C93.61	93606	RCBC	6	3.00	2.10	9.1	325.35	0.54	2.11	0.05	3.77	0.03
310-BR07		BRIDGE		184.00	325.42	168.1	321.48	N/A	5.94	N/A - Regional	N/A - Regional	N/A - Regional
C94.91	94910	RCBC	5	2.10	0.90	9.0	324.16	0.78	0.59	0.29	1.24	0.29
C95.07	95065	RCBC	18	2.40	1.80	46.5	323.50	0.90	0.98	0.40	7.60	1.15
C96.20	96197	RCBC	14	2.40	1.20	19.4	325.66	0.60	1.19	0.08	6.87	0.02
C97.01	97008	RCBC	5	1.50	1.20	6.1	328.01	0.76	1.55	0.24	4.97	2.08
C97.29	97293	RCP	12	0.90		6.7	328.35	0.66	4.78	N/A - Regional	N/A - Regional	N/A - Regional
C97.38	97382	RCP	12	0.90		7.9	328.34	0.82	5.64	N/A - Regional	N/A - Regional	N/A - Regional
310-BR08		BRIDGE		299.00	332.79	54.2	328.35	N/A	6.44	N/A - Regional	N/A - Regional	N/A - Regional
C97.84	97836	RCP	8	1.50		6.7	328.65	1.29	5.16	N/A - Regional	N/A - Regional	N/A - Regional
C98.36	98360	RCP	10	0.90		11.6	330.12	0.98	3.45	N/A - Regional	N/A - Regional	N/A - Regional
C98.87	98866	RCP	1	1.50		3.3	332.14	1.54	1.31	0.00	0.08	0.00
C99.37	99373	RCP	17	0.90		4.2	331.67	0.25	2.40	N/A - Regional	N/A - Regional	N/A - Regional
C99.84	99841	RCP	14	0.90		10.2	333.00	0.63	1.63	N/A - Regional	N/A - Regional	N/A - Regional
C99.77	99765	RCP	1	1.50		2.1	333.00	1.18	1.54	N/A - Regional	N/A - Regional	N/A - Regional
C100.00	100002	RCP	8	1.50		7.3	333.11	0.81	1.72	N/A - Regional	N/A - Regional	N/A - Regional
310-BR10		BRIDGE		621.00	334.54	566.1	334.04	N/A	2.51	N/A - Regional	N/A - Regional	N/A - Regional
C101.48	101485	RCBC	4	1.50	1.20	1.4	336.63	0.32	1.30	0.13	1.83	1.83
C102.55	102546	RCBC	4	1.50	0.90	3.9	345.09	0.64	0.64	0.00	1.53	0.00
310-BR11		BRIDGE		92.00	353.90	159.0	353.14	N/A	2.75	N/A - Regional	N/A - Regional	N/A - Regional
C104.94	104938	RCP	18	0.90		3.9	355.13	0.81	0.73	N/A - Regional	N/A - Regional	N/A - Regional
C105.09	105093	RCP	6	0.90		6.4	355.44	1.11	1.77	N/A - Regional	N/A - Regional	N/A - Regional
C105.11	105110	RCP	6	0.90		5.8	355.45	1.14	1.92	N/A - Regional	N/A - Regional	N/A - Regional
C105.13	105126	RCP	6	0.90		5.9	355.48	1.09	2.03	N/A - Regional	N/A - Regional	N/A - Regional
C106.54	106541	RCBC	6	1.20	1.20	9.6	369.53	1.05	0.33	0.10	1.02	0.01
C107.22	107222	RCBC	4	2.40	1.80	19.7	373.28	1.38	2.19	0.38	1.24	0.35
C107.81	107807	RCBC	3	0.90	0.90	1.9	381.35	0.67	0.40	0.00	3.31	0.00
C107.96	107964	RCP	4	1.20		9.6	382.25	1.53	1.46	0.08	2.93	0.09
C108.45	108454	RCP	4	1.50		8.3	383.83	1.33	6.02	0.00	1.29	0.00
C109.43	109430	RCBC	10	1.80	1.80	17.2	399.53	0.83	2.62	0.13	1.58	0.00
C110.91	110912	RCBC	2	1.80	1.50	3.6	420.01	0.86	0.67	0.00	1.94	0.04
C111.16	111165	RCBC	3	1.20	1.20	1.6	423.00	0.50	0.84	0.00	0.32	0.00
C111.26	111259	RCBC	4	1.20	0.90	3.7	424.48	0.73	0.53	0.00	1.16	0.00
C112.32	112325	RCBC	2	1.80	1.50	4.5	435.99	0.99	0.52	0.00	2.97	0.00
C113.00	113001	RCBC	4	1.20	1.20	4.9	433.97	0.87	2.16	0.00	1.42	0.00
C113.07	113073	RCBC	2	2.40	2.40	4.7	434.52	0.92	1.57	0.00	0.91	0.00
C113.28	113281	RCBC	12	2.10	0.90	12.8	435.56	0.55	1.14	0.00	1.20	0.00
C114.27	114269	RCP	11	0.90		2.9	446.31	0.41	2.05	0.00	0.00	0.00

Culvert ID	Chainage	Type	Number	Diameter/Width(m) – Culverts Span (m) – Bridges	Height (m) – Culverts Soffit Level (m AHD) - Bridges	1% AEP Flow Through Structure (m³/s)	1% AEP Upstream Water Level- Design (m AHD)	1% AEP Upstream Headwater Depth- Design (m)	1% AEP Freeboard to Formation (m)	Impacts at Rail Corridor		
										1% AEP Afflux (m)	Existing Time of Inundation (hrs)	Change in Time of Inundation (hrs)
C114.36	114360	RCBC	10	2.10	1.50	35.7	448.54	1.28	0.77	0.32	3.73	0.55
C114.90	114900	RCBC	2	1.80	1.50	4.6	452.64	1.02	2.21	0.00	0.59	0.00
C115.00	115003	RCP	3	0.90		2.6	453.54	0.84	2.41	0.00	0.34	0.00
C115.32	115322	RCP	3	0.90		2.6	454.53	0.86	4.66	0.00	0.25	0.00
310-BR28		BRIDGE		75.00	456.88	19.6	452.48	N/A	8.85	0.00	1.15	0.00
C117.40	117401	RCBC	13	3.00	1.80	29.8	457.82	0.72	2.93	0.13	3.09	0.36
C117.60	117600	RCBC	4	2.70	2.10	14.8	458.51	1.01	1.48	0.00	2.52	0.00
C117.71	117706	RCBC	1	1.50	1.50	0.6	458.51	0.56	1.08	0.00	2.02	0.00
C118.10	118099	RCBC	15	2.10	0.90	10.6	457.11	0.41	1.00	0.00	1.66	0.00
C118.43	118430	RCBC	10	3.00	1.50	27.1	455.70	0.80	1.16	0.00	2.54	0.00
C118.61	118606	RCBC	10	1.20	1.20	1.5	454.73	0.21	1.47	0.00	1.71	0.03
C118.90	118903	RCP	10	0.90		5.4	454.14	0.54	2.68	0.00	0.72	0.00
C119.04	119036	RCBC	2	2.40	1.80	4.0	454.32	0.77	2.91	0.00	0.89	0.00
C119.30	119298	RCBC	15	1.20	1.20	1.8	454.08	0.18	3.91	0.00	0.60	0.00
C119.38	119378	RCBC	10	3.00	1.50	20.6	454.87	0.67	3.36	0.14	2.74	0.69
C119.75	119751	RCBC	2	2.40	1.20	4.2	457.96	0.79	1.35	0.00	1.39	0.00
C119.88	119875	RCBC	8	1.50	1.20	5.1	458.28	0.48	1.25	0.10	1.13	0.36
C120.08	120080	RCBC	5	1.50	0.90	0.9	458.33	0.20	1.34	0.00	0.21	0.00
C120.25	120252	RCBC	26	2.40	1.50	55.0	458.69	0.79	1.08	0.24	3.73	0.65
C120.31	120310	RCBC	3	2.40	1.80	16.3	458.69	1.51	1.12	0.08	3.71	0.00
C120.77	120765	RCBC	13	2.40	1.80	44.9	458.98	1.08	1.11	0.00	3.23	0.00
C124.44	124442	RCP	10	1.80		6.3	437.44	0.59	10.09	0.04	1.60	0.17
C125.48	125483	RCP	10	1.80		26.1	425.14	1.19	9.97	0.05	3.82	0.00
C125.72	125716	RCP	1	0.90		1.6	425.72	1.60	6.58	0.00	0.46	0.00
C125.82	125824	RCP	1	1.80		2.7	424.91	1.06	6.08	0.00	0.69	0.00
C126.77	126767	RCP	12	0.90		0.2	412.65	0.34	9.88	N/A - Regional	N/A - Regional	N/A - Regional
C126.81	126807	RCP	12	0.90		4.4	412.57	0.60	9.72	N/A - Regional	N/A - Regional	N/A - Regional
310-BR37		BRIDGE		167.00	417.36	74.4	411.77	N/A	10.03	N/A - Regional	N/A - Regional	N/A - Regional
310-BR38		BRIDGE		230.00	407.41	350.4	407.05	N/A	2.36	N/A - Regional	N/A - Regional	N/A - Regional
C128.89	128893	RCBC	28	1.20	0.90	21.0	406.88	0.63	1.63	0.35	3.38	1.13
C133.02	133020	RCBC	20	1.20	0.90	12.0	396.37	0.54	0.59	0.33	3.03	0.01
C135.72	135716	RCBC	10	1.80	0.90	1.5	386.11	0.17	1.89	0.17	3.64	0.53
C135.79	135790	RCBC	10	1.80	0.90	5.2	386.11	0.38	1.66	0.24	3.64	0.06
C135.91	135910	RCBC	35	1.80	0.90	28.6	386.11	0.51	1.25	0.22	3.35	1.63
C139.13	139129	RCP	8	1.35		1.7	380.66	0.92	1.57	N/A - Regional	N/A - Regional	N/A - Regional
C139.17	139170	RCP	11	1.65		2.8	380.65	1.09	1.53	N/A - Regional	N/A - Regional	N/A - Regional
C139.21	139205	RCP	8	1.80		10.6	380.63	1.15	1.51	N/A - Regional	N/A - Regional	N/A - Regional
310-BR21/22		BRIDGE		1330.00	382.05	1169.4	380.55	N/A	2.50	N/A - Regional	N/A - Regional	N/A - Regional

Culvert ID	Chainage	Type	Number	Diameter/Width(m) – Culverts Span (m) – Bridges	Height (m) – Culverts Soffit Level (m AHD) - Bridges	1% AEP Flow Through Structure (m³/s)	1% AEP Upstream Water Level- Design (m AHD)	1% AEP Upstream Headwater Depth- Design (m)	1% AEP Freeboard to Formation (m)	Impacts at Rail Corridor		
										1% AEP Afflux (m)	Existing Time of Inundation (hrs)	Change in Time of Inundation (hrs)
C140.66	140663	RCP	11	1.80		9.9	380.54	1.30	1.69	N/A - Regional	N/A - Regional	N/A - Regional
C140.73	140730	RCP	8	2.10		11.7	380.54	1.39	1.69	N/A - Regional	N/A - Regional	N/A - Regional
C140.79	140787	RCP	8	2.10		12.0	380.55	1.47	1.68	N/A - Regional	N/A - Regional	N/A - Regional
C140.85	140846	RCP	11	1.80		13.3	380.55	1.13	1.68	N/A - Regional	N/A - Regional	N/A - Regional
C141.00	141001	RCP	9	1.65		13.6	380.56	1.05	1.67	N/A - Regional	N/A - Regional	N/A - Regional
C141.02	141025	RCBC	4	2.40	2.10	14.5	380.57	2.45	1.66	N/A - Regional	N/A - Regional	N/A - Regional
C141.07	141068	RCP	10	2.10		12.1	380.56	1.62	1.79	N/A - Regional	N/A - Regional	N/A - Regional
C141.38	141377	RCP	7	1.80		10.1	380.56	0.89	1.78	N/A - Regional	N/A - Regional	N/A - Regional
C141.40	141401	RCP	7	1.80		10.2	380.55	1.27	1.68	N/A - Regional	N/A - Regional	N/A - Regional
C141.46	141460	RCP	6	2.10		9.6	380.50	1.26	1.73	N/A - Regional	N/A - Regional	N/A - Regional
C141.49	141494	RCP	6	2.10		8.3	380.49	1.25	1.74	N/A - Regional	N/A - Regional	N/A - Regional
C141.52	141521	RCP	6	2.10		8.4	380.49	1.26	1.74	N/A - Regional	N/A - Regional	N/A - Regional
C141.54	141541	RCP	6	2.10		8.2	380.49	1.26	1.74	N/A - Regional	N/A - Regional	N/A - Regional
C141.56	141563	RCP	6	2.10		8.4	380.49	1.23	1.74	N/A - Regional	N/A - Regional	N/A - Regional
C141.61	141609	RCP	6	2.10		9.5	380.51	1.19	1.72	N/A - Regional	N/A - Regional	N/A - Regional
C141.67	141665	RCP	6	2.10		9.1	380.50	1.26	1.73	N/A - Regional	N/A - Regional	N/A - Regional
C141.69	141689	RCP	6	2.10		10.5	380.51	1.27	1.72	N/A - Regional	N/A - Regional	N/A - Regional
C141.72	141721	RCP	7	1.80		10.4	380.50	1.11	1.73	N/A - Regional	N/A - Regional	N/A - Regional
C141.75	141753	RCP	5	2.10		8.8	380.50	1.25	1.73	N/A - Regional	N/A - Regional	N/A - Regional
C141.78	141778	RCP	6	2.10		10.4	380.50	1.31	1.73	N/A - Regional	N/A - Regional	N/A - Regional
C141.80	141801	RCP	6	2.10		11.0	380.50	1.31	1.73	N/A - Regional	N/A - Regional	N/A - Regional
C141.84	141842	RCP	5	2.10		9.8	380.49	1.30	1.74	N/A - Regional	N/A - Regional	N/A - Regional
C141.88	141879	RCP	5	2.10		10.7	380.49	1.43	1.74	N/A - Regional	N/A - Regional	N/A - Regional
C141.93	141926	RCP	6	2.10		13.6	380.48	1.35	1.75	N/A - Regional	N/A - Regional	N/A - Regional
C141.96	141963	RCP	5	2.10		11.9	380.48	1.42	1.75	N/A - Regional	N/A - Regional	N/A - Regional
C142.07	142072	RCP	6	2.10		16.9	380.46	1.52	1.77	N/A - Regional	N/A - Regional	N/A - Regional
C142.12	142118	RCP	6	2.10		16.1	380.46	1.57	1.77	N/A - Regional	N/A - Regional	N/A - Regional
C142.16	142164	RCP	4	2.10		11.0	380.46	1.46	1.77	N/A - Regional	N/A - Regional	N/A - Regional
C142.20	142203	RCP	6	2.10		16.5	380.45	1.55	1.78	N/A - Regional	N/A - Regional	N/A - Regional
C142.27	142267	RCP	6	2.10		16.5	380.45	1.58	1.78	N/A - Regional	N/A - Regional	N/A - Regional
C142.32	142320	RCP	4	2.10		11.2	380.45	1.65	1.78	N/A - Regional	N/A - Regional	N/A - Regional
C142.36	142362	RCP	6	2.10		16.9	380.43	1.64	1.80	N/A - Regional	N/A - Regional	N/A - Regional
C142.40	142405	RCP	6	2.10		16.6	380.43	1.57	1.80	N/A - Regional	N/A - Regional	N/A - Regional
C142.49	142489	RCP	6	2.10		14.8	380.42	1.53	1.81	N/A - Regional	N/A - Regional	N/A - Regional
C142.53	142533	RCP	6	2.10		14.6	380.40	1.52	1.83	N/A - Regional	N/A - Regional	N/A - Regional
C142.58	142579	RCP	6	2.10		11.0	380.38	1.51	1.85	N/A - Regional	N/A - Regional	N/A - Regional
C142.61	142614	RCP	4	2.10		6.5	380.35	1.67	1.88	N/A - Regional	N/A - Regional	N/A - Regional
310-BR24		BRIDGE		658.00	382.05	273.9	380.39	N/A	2.66	N/A - Regional	N/A - Regional	N/A - Regional

Culvert ID	Chainage	Type	Number	Diameter/Width(m) – Culverts Span (m) – Bridges	Height (m) – Culverts Soffit Level (m AHD) - Bridges	1% AEP Flow Through Structure (m³/s)	1% AEP Upstream Water Level- Design (m AHD)	1% AEP Upstream Headwater Depth- Design (m)	1% AEP Freeboard to Formation (m)	Impacts at Rail Corridor		
										1% AEP Afflux (m)	Existing Time of Inundation (hrs)	Change in Time of Inundation (hrs)
C143.29	143288	RCP	6	2.10		5.6	380.33	1.46	1.90	N/A - Regional	N/A - Regional	N/A - Regional
C143.33	143334	RCP	6	2.10		8.8	380.35	1.38	1.88	N/A - Regional	N/A - Regional	N/A - Regional
C143.37	143372	RCP	6	2.10		9.9	380.35	1.36	1.88	N/A - Regional	N/A - Regional	N/A - Regional
C143.42	143417	RCP	6	2.10		9.3	380.35	1.21	1.88	N/A - Regional	N/A - Regional	N/A - Regional
C143.44	143445	RCP	6	2.10		9.1	380.35	1.23	1.88	N/A - Regional	N/A - Regional	N/A - Regional
C143.48	143478	RCP	6	2.10		9.7	380.35	1.21	1.88	N/A - Regional	N/A - Regional	N/A - Regional
C143.51	143509	RCP	6	2.10		9.8	380.35	1.31	1.88	N/A - Regional	N/A - Regional	N/A - Regional
C143.54	143538	RCP	6	2.10		9.3	380.35	1.17	1.88	N/A - Regional	N/A - Regional	N/A - Regional
C143.57	143574	RCP	5	2.10		8.7	380.36	1.19	1.87	N/A - Regional	N/A - Regional	N/A - Regional
C143.65	143654	RCP	6	2.10		10.1	380.37	1.15	1.86	N/A - Regional	N/A - Regional	N/A - Regional
C143.70	143701	RCP	6	2.10		9.0	380.37	1.16	1.86	N/A - Regional	N/A - Regional	N/A - Regional
C143.73	143734	RCP	6	2.10		9.5	380.38	1.17	1.85	N/A - Regional	N/A - Regional	N/A - Regional
C143.77	143768	RCP	6	2.10		9.3	380.38	1.14	1.85	N/A - Regional	N/A - Regional	N/A - Regional
C143.79	143792	RCP	5	2.10		7.5	380.37	1.13	1.86	N/A - Regional	N/A - Regional	N/A - Regional
C143.83	143832	RCP	4	2.10		6.8	380.39	1.21	1.84	N/A - Regional	N/A - Regional	N/A - Regional
C143.87	143875	RCP	5	2.10		5.5	380.38	1.20	1.85	N/A - Regional	N/A - Regional	N/A - Regional
310-BR25/26		BRIDGE		2577.00	380.60	1517.8	380.44	N/A	2.61	N/A - Regional	N/A - Regional	N/A - Regional
C146.45	146452	RCBC	4	1.20	0.90	2.4	380.84	0.51	1.77	N/A - Regional	N/A - Regional	N/A - Regional
C146.50	146497	RCBC	4	1.20	0.90	2.4	380.93	0.39	1.35	N/A - Regional	N/A - Regional	N/A - Regional
C146.54	146536	RCBC	4	1.20	0.90	2.1	380.98	0.32	1.24	N/A - Regional	N/A - Regional	N/A - Regional
C146.61	146610	RCBC	2	1.20	0.90	0.8	381.06	0.31	1.18	N/A - Regional	N/A - Regional	N/A - Regional
C146.69	146686	RCBC	6	1.20	0.90	0.8	381.09	0.19	1.16	N/A - Regional	N/A - Regional	N/A - Regional
C147.01	147013	RCBC	10	1.50	0.90	2.3	381.17	0.31	1.15	N/A - Regional	N/A - Regional	N/A - Regional
C147.12	147116	RCBC	4	1.20	0.90	0.0	381.19	0.49	1.14	N/A - Regional	N/A - Regional	N/A - Regional
C147.18	147178	RCBC	10	1.50	0.90	3.1	381.18	0.45	1.18	N/A - Regional	N/A - Regional	N/A - Regional
C147.21	147213	RCBC	4	1.20	0.90	0.3	381.20	0.43	1.16	N/A - Regional	N/A - Regional	N/A - Regional
C147.27	147265	RCBC	4	1.20	0.90	0.2	381.23	0.38	1.14	N/A - Regional	N/A - Regional	N/A - Regional
C147.31	147315	RCBC	10	1.50	0.90	3.0	381.25	0.38	1.14	N/A - Regional	N/A - Regional	N/A - Regional
C147.85	147851	RCBC	6	1.20	0.60	0.1	381.75	0.23	0.75	N/A - Regional	N/A - Regional	N/A - Regional
C147.91	147914	RCBC	4	1.20	0.60	0.2	381.77	0.20	0.74	N/A - Regional	N/A - Regional	N/A - Regional
C148.87	148874	RCP	25	1.05		4.3	382.46	0.98	1.26	N/A - Regional	N/A - Regional	N/A - Regional
C148.92	148915	RCP	25	1.05		5.4	382.46	0.96	1.33	N/A - Regional	N/A - Regional	N/A - Regional
C148.96	148956	RCP	25	1.05		7.6	382.46	1.01	1.41	N/A - Regional	N/A - Regional	N/A - Regional
C149.02	149016	RCP	25	1.05		9.0	382.44	1.02	1.51	N/A - Regional	N/A - Regional	N/A - Regional
310-BR27		BRIDGE		1568.00	383.79	201.3	382.55	N/A	2.24	N/A - Regional	N/A - Regional	N/A - Regional
C150.68	150677	RCP	10	1.35		3.8	382.66	0.86	1.31	N/A - Regional	N/A - Regional	N/A - Regional
C150.71	150708	RCP	12	1.20		3.8	382.67	0.84	1.29	N/A - Regional	N/A - Regional	N/A - Regional
C150.74	150744	RCP	3	1.35		1.4	382.68	0.89	1.29	N/A - Regional	N/A - Regional	N/A - Regional

Culvert ID	Chainage	Type	Number	Diameter/Width(m) – Culverts Span (m) – Bridges	Height (m) – Culverts Soffit Level (m AHD) - Bridges	1% AEP Flow Through Structure (m³/s)	1% AEP Upstream Water Level- Design (m AHD)	1% AEP Upstream Headwater Depth- Design (m)	1% AEP Freeboard to Formation (m)	Impacts at Rail Corridor		
										1% AEP Afflux (m)	Existing Time of Inundation (hrs)	Change in Time of Inundation (hrs)
C151.08	151077	RCP	8	1.20		2.8	382.73	0.88	1.24	N/A - Regional	N/A - Regional	N/A - Regional
C151.12	151119	RCP	8	1.20		2.2	382.73	0.86	1.24	N/A - Regional	N/A - Regional	N/A - Regional
C151.14	151142	RCP	8	1.20		2.3	382.73	0.84	1.24	N/A - Regional	N/A - Regional	N/A - Regional
C151.17	151173	RCP	6	1.35		2.5	382.73	0.91	1.24	N/A - Regional	N/A - Regional	N/A - Regional
C151.20	151201	RCP	6	1.35		2.6	382.73	0.90	1.24	N/A - Regional	N/A - Regional	N/A - Regional
C151.27	151269	RCP	8	1.20		1.7	382.73	0.76	1.24	N/A - Regional	N/A - Regional	N/A - Regional
C151.30	151296	RCP	8	1.05		0.9	382.73	0.50	1.24	N/A - Regional	N/A - Regional	N/A - Regional
C152.40	152396	RCBC	3	0.90	0.90	1.1	387.01	0.48	0.71	0.00	3.63	0.00
C153.44	153438	RCBC	2	0.90	0.90	1.2	389.19	0.66	0.51	0.00	0.00	0.00
C154.51	154508	RCBC	12	2.10	0.90	15.7	390.18	0.63	1.22	0.33	4.35	2.63
C155.59	155592	RCBC	9	2.10	1.20	17.2	393.03	0.80	1.10	0.16	5.60	2.82
C159.25	159245	RCBC	5	2.40	1.20	15.5	403.23	1.03	0.47	0.13	5.47	1.62
C160.41	160414	RCP	5	1.80		24.7	406.85	1.75	2.43	0.10	5.23	0.15
C161.15	161152	RCP	8	1.20		7.7	411.37	0.75	6.73	0.15	1.73	0.47
C162.30	162301	RCP	2	1.50		3.2	425.49	0.90	6.36	0.00	0.00	0.00
310-BR29		BRIDGE		69.00	433.16	13.2	426.83	N/A	8.33	0.00	0.05	0.00
C162.82	162824	RCP	3	1.80		8.9	429.11	1.09	6.74	0.19	1.24	0.27
C164.16	164160	RCP	6	0.90		2.3	435.13	0.00	14.77	-0.12	1.19	4.39
C164.29	164288	RCP	22	1.20		33.3	436.16	0.96	15.22	0.15	1.81	2.56
C165.06	165063	RCP	3	1.80		12.3	451.51	1.41	9.11	2.33	0.85	5.49
C166.10	166104	RCBC	3	2.40	1.20	8.9	472.27	0.99	0.63	0.00	0.00	0.00
C167.08	167082	RCBC	2	1.20	1.20	3.2	484.00	1.05	0.64	0.00	0.36	0.00
C168.61	168611	RCBC	4	2.40	0.90	10.3	501.97	0.93	0.84	0.00	0.00	0.00
C169.01	169013	RCBC	2	1.20	1.20	4.5	504.53	1.39	0.93	0.00	0.00	0.00
C169.89	169886	RCP	5	2.70		25.2	498.20	1.29	7.15	0.00	1.48	0.00
C170.55	170548	RCBC	10	2.40	2.40	37.7	498.75	1.12	5.32	0.01	5.22	0.02
C171.02	171018	RCP	8	1.80		11.2	497.72	0.82	5.43	0.00	1.31	0.00
C171.90	171902	RCP	15	1.80		29.3	499.71	0.91	12.73	0.39	2.61	0.92
310-BR30		BRIDGE		69.00	513.94	11.2	506.67	N/A	9.27	0.18	0.34	0.20
C173.54	173544	RCP	3	1.35		2.2	521.78	0.56	8.71	0.00	0.29	0.01
310-BR40		BRIDGE		75.00	528.95	14.1	520.35	N/A	13.05	0.00	1.40	4.45
C176.89	176892	RCBC	3	2.40	2.10	18.1	558.40	1.56	4.31	0.00	1.25	0.00
C177.63	177635	RCP	1	2.10		8.1	564.48	2.28	1.94	0.00	0.95	0.00
C178.02	178017	RCP	4	2.10		15.6	563.59	1.04	4.74	0.00	1.31	0.00
C178.63	178632	RCP	7	1.80		18.1	569.68	1.13	1.73	0.12	1.77	0.01
C181.21	181208	RCP	9	2.10		33.2	547.69	1.25	13.18	0.38	1.42	0.02
C181.78	181784	RCP	4	1.80		17.3	545.77	1.47	8.24	0.17	0.53	0.03
C182.99	182991	RCP	9	2.40		27.4	532.92	1.15	6.93	0.27	3.31	0.53

Culvert ID	Chainage	Type	Number	Diameter/Width(m) – Culverts Span (m) – Bridges	Height (m) – Culverts Soffit Level (m AHD) - Bridges	1% AEP Flow Through Structure (m³/s)	1% AEP Upstream Water Level- Design (m AHD)	1% AEP Upstream Headwater Depth- Design (m)	1% AEP Freeboard to Formation (m)	Impacts at Rail Corridor		
										1% AEP Afflux (m)	Existing Time of Inundation (hrs)	Change in Time of Inundation (hrs)
C183.56	183558	RCP	6	2.10		14.8	533.95	0.96	5.78	0.29	1.13	0.29
310-BR42		BRIDGE		144.00	530.43	76.4	516.94	N/A	17.94	0.00	2.65	0.00
C186.16	186155	RCBC	10	2.40	2.10	30.2	521.37	0.99	2.45	0.00	2.82	0.00
C187.20	187205	RCP	16	2.10		40.2	518.94	1.06	8.11	0.13	4.83	0.19
C189.99	189990	RCBC	11	1.80	1.50	43.3	511.89	1.43	14.87	N/A - Regional	N/A - Regional	N/A - Regional
C192.09	192086	RCP	7	1.80		13.0	489.99	0.89	10.93	0.00	1.29	0.00
C193.10	193103	RCP	5	3.00		55.0	465.40	2.44	22.92	N/A - Regional	N/A - Regional	N/A - Regional
C194.66	194659	RCBC	2	1.50	1.20	6.4	453.60	0.00	17.52	N/A - Regional	N/A - Regional	N/A - Regional
C194.69	194691	RCP	3	1.05		4.3	470.57	1.65	1.18	N/A - Regional	N/A - Regional	N/A - Regional
C195.98	195978	RCBC	7	1.50	0.90	6.5	453.61	0.62	1.19	0.00	0.00	0.00
C196.46	196465	RCP	5	1.20		5.8	439.50	0.82	9.42	0.00	1.17	0.00
C196.92	196916	RCP	14	1.35		8.5	433.39	1.03	10.11	N/A - Regional	N/A - Regional	N/A - Regional
C197.18	197181	RCP	2	1.05		0.7	433.39	0.48	7.94	N/A - Regional	N/A - Regional	N/A - Regional
C197.31	197313	RCP	2	1.05		0.6	433.30	0.49	8.03	N/A - Regional	N/A - Regional	N/A - Regional
310-BR20		BRIDGE		92.00	440.15	57.6	432.78	N/A	9.37	N/A - Regional	N/A - Regional	N/A - Regional
310-BR31		BRIDGE		230.00	428.95	566.5	426.69	N/A	4.26	N/A - Regional	N/A - Regional	N/A - Regional
C198.68	198684	RCP	15	2.40		104.5	426.46	2.58	1.31	N/A - Regional	N/A - Regional	N/A - Regional
C198.77	198770	RCP	11	1.50		27.2	426.42	1.33	1.01	N/A - Regional	N/A - Regional	N/A - Regional
C198.80	198803	RCP	10	1.20		13.0	426.44	0.97	0.95	N/A - Regional	N/A - Regional	N/A - Regional
C198.98	198983	RCP	17	1.05		16.3	426.37	0.93	2.79	N/A - Regional	N/A - Regional	N/A - Regional
310-BR32		BRIDGE		184.00	431.25	196.5	425.49	N/A	7.76	N/A - Regional	N/A - Regional	N/A - Regional
C199.53	199533	RCP	15	1.05		12.6	427.08	0.79	8.37	N/A - Regional	N/A - Regional	N/A - Regional
310-BR33		BRIDGE		75.00	436.21	11.7	429.83	N/A	10.83	N/A - Regional	N/A - Regional	N/A - Regional
C200.52	200519	RCP	5	0.90		4.3	438.59	0.84	1.26	0.00	0.21	0.00
C200.82	200824	RCBC	13	0.90	0.90	6.0	442.20	0.55	0.52	0.00	0.60	0.00
C201.23	201233	RCBC	4	0.90	0.90	3.0	446.68	0.76	0.86	0.00	0.18	0.00
C201.51	201514	RCBC	3	0.90	0.90	2.1	450.33	0.73	0.50	0.00	0.22	0.00
C201.97	201974	RCP	5	1.20		8.6	451.31	1.08	4.93	0.00	0.63	0.00
C202.52	202524	RCP	7	1.20		10.0	457.92	0.92	4.79	0.00	0.76	0.00
C202.80	202802	RCBC	7	2.10	2.10	17.7	460.26	0.96	5.75	0.16	1.89	0.13
310-BR34		BRIDGE		132.00	481.45	3.4	473.33	N/A	10.62	N/A - Regional	N/A - Regional	N/A - Regional
C204.45	204447	RCP	2	1.05		0.6	471.86	0.46	10.96	N/A - Regional	N/A - Regional	N/A - Regional
310-BR35		BRIDGE		299.00	463.43	13.4	449.97	N/A	15.46	N/A - Regional	N/A - Regional	N/A - Regional
C206.17	206167	RCP	2	1.05		1.1	447.48	0.62	14.49	N/A - Regional	N/A - Regional	N/A - Regional
C206.36	206359	RCP	12	1.05		6.0	447.09	0.58	12.79	N/A - Regional	N/A - Regional	N/A - Regional
C206.42	206421	RCP	2	1.05		1.8	447.31	0.41	12.02	N/A - Regional	N/A - Regional	N/A - Regional
C206.57	206574	RCP	4	1.05		2.3	447.77	0.63	9.67	N/A - Regional	N/A - Regional	N/A - Regional
C206.65	206651	RCP	15	1.05		6.7	447.77	0.55	8.75	N/A - Regional	N/A - Regional	N/A - Regional

Culvert ID	Chainage	Type	Number	Diameter/Width(m) – Culverts Span (m) – Bridges	Height (m) – Culverts Soffit Level (m AHD) - Bridges	1% AEP Flow Through Structure (m³/s)	1% AEP Upstream Water Level- Design (m AHD)	1% AEP Upstream Headwater Depth- Design (m)	1% AEP Freeboard to Formation (m)	Impacts at Rail Corridor		
										1% AEP Afflux (m)	Existing Time of Inundation (hrs)	Change in Time of Inundation (hrs)
C206.75	206746	RCP	5	1.05		1.2	448.56	0.41	6.84	N/A - Regional	N/A - Regional	N/A - Regional
C206.88	206876	RCP	2	1.05		2.0	449.29	0.87	4.50	N/A - Regional	N/A - Regional	N/A - Regional
C207.15	207146	RCP	7	1.35		8.6	450.21	1.26	0.63	N/A - Regional	N/A - Regional	N/A - Regional
C207.70	207705	RCBC	1	1.80	0.90	2.2	455.15	1.05	0.35	0.00	0.60	0.00
C208.21	208208	RCBC	16	2.40	1.50	46.6	457.02	1.62	2.57	N/A - Regional	N/A - Regional	N/A - Regional

N/A – Refer to Flood Study Report for Afflux in floodplains

Table 3 Proposed outlet scour protection works

Culvert ID	Chainage	Type	Number	Diameter/Width(m) – Culverts Span (m) – Bridges	Height (m) – Culverts Soffit Level (m AHD) - Bridges	1% AEP Peak Outlet Velocity (m/s)	Downstream Scour Protection				Fish Passageway	
							Material	Scour protection required (Y/N) ¹	Length	Rock d50	WWBW zoning colour	Fish passage required (Y/N)
C31.26 (NS2B)	31256 (NS2B)	RCP	10	1.80		1.18	Vertosol	N				N
C31.32 (NS2B)	31324 (NS2B)	RCP	10	1.80		1.23	Vertosol	N				N
BR12	31520 (NS2B)	BRIDGE		140.00	227.46	0.51	N/A	N/A				N
C31.87 (NS2B)	31875 (NS2B)	RCP	15	0.90		0.00	Vertosol	N				N
C32.00 (NS2B)	32000 (NS2B)	RCP	15	0.90		0.00	Vertosol	N				N
BR13	32550 (NS2B)	BRIDGE		546.00	225.71	0.30	N/A	N/A			Amber	Y
C34.64 (NS2B)	34639 (NS2B)	RCBC	9	3.00	1.20	0.64	Dermosol	N				N
C34.70 (NS2B)	34700 (NS2B)	RCBC	19	3.00	1.50	0.63	Dermosol	N			Amber	Y
C34.75 (NS2B)	34752 (NS2B)	RCBC	10	3.00	1.20	0.61	Dermosol	N				N
C34.79 (NS2B)	34785 (NS2B)	RCBC	7	3.00	1.20	0.47	Dermosol	N				N
C35.03 (NS2B)	35026 (NS2B)	RCBC	20	1.20	0.90	0.89	Dermosol	N				N
C35.08 (NS2B)	35077 (NS2B)	RCBC	25	1.20	0.90	0.91	Dermosol	N				N
C35.21 (NS2B)	35207 (NS2B)	RCBC	20	1.20	0.90	0.70	Dermosol	N				N
C35.54 (NS2B)	35535 (NS2B)	RCBC	4	1.20	0.90	1.59	Sodosol	N				N
C35.61 (NS2B)	35606 (NS2B)	RCBC	4	1.20	0.90	1.64	Sodosol	N				N
C35.7 (NS2B)	35701 (NS2B)	RCBC	4	1.20	0.90	1.53	Sodosol	N				N
C35.88 (NS2B)	35879 (NS2B)	RCP	7	0.90		1.64	Sodosol	N				N
C35.91 (NS2B)	35906 (NS2B)	RCP	7	0.90		1.65	Sodosol	N				N
C36.04 (NS2B)	36037 (NS2B)	RCP	6	0.90		1.56	Sodosol	N				N
C36.22 (NS2B)	36222 (NS2B)	RCP	7	0.90		1.07	Sodosol	N				N
C6.60	6602	RCBC	5	1.80	0.90	1.09	Vertosol	N				N
C8.33	8330	RCBC	8	1.20	1.20	1.29	Sodosol	Y	7.2	100		N
C13.00	13001	RCBC	18	2.40	0.90	1.11	Sodosol	Y	5.4	100		N
C17.89	17887	RCP	2	1.05		2.53	Sodosol	Y	6.3	200		N
C18.51	18507	RCP	2	1.20		2.91	Sodosol	Y	9.6	300		N
C18.87	18871	RCP	7	1.05		1.48	Sodosol	N				N
C20.00	19996	RCBC	6	2.10	0.90	1.06	Sodosol	Y	5.4	100		N
C22.43	22425	RCBC	11	1.20	0.90	1.12	Sodosol	Y	5.4	100		N
C23.05	23047	RCBC	8	1.20	0.90	1.11	Sodosol	N				N
C23.53	23531	RCBC	10	1.20	0.90	1.09	Sodosol	N				N
C24.41	24411	RCBC	3	1.20	0.90	1.46	Sodosol	N				N
C24.84	24840	RCBC	1	1.20	0.90	2.50	Sodosol	Y	2.7	200		N
C25.19	25186	RCBC	6	1.20	0.60	1.56	Sodosol	N				N
C25.46	25459	RCP	21	0.90		1.08	Sodosol	N				N
C25.50	25504	RCBC	8	2.40	0.90	0.91	Sodosol	N				N
C25.80	25803	RCBC	24	2.40	0.90	0.46	Sodosol	N				N

Culvert ID	Chainage	Type	Number	Diameter/Width(m) – Culverts Span (m) – Bridges	Height (m) – Culverts Soffit Level (m AHD) - Bridges	1% AEP Peak Outlet Velocity (m/s)	Downstream Scour Protection				Fish Passageway	
							Material	Scour protection required (Y/N) ¹	Length	Rock d50	WWBW zoning colour	Fish passage required (Y/N)
C25.87	25874	RCBC	24	2.40	0.90	0.50	Sodosol	N				N
C25.95	25946	RCBC	1	3.00	0.50	1.11	Sodosol	N				N
C25.97	25966	RCBC	1	3.00	0.50	1.88	Sodosol	N				N
C26.05	26048	RCBC	1	1.20	0.30	0.82	Sodosol	N				N
C27.05	27051	RCBC	15	1.50	1.20	0.74	Sodosol	N				N
C27.15	27146	RCBC	15	1.50	1.20	0.75	Sodosol	N				N
C27.24	27238	RCBC	25	1.50	1.20	2.07	Sodosol	Y	7.2	200		N
C27.33	27332	RCBC	25	1.50	1.20	1.26	Sodosol	N				N
C27.42	27421	RCBC	20	1.50	1.20	0.95	Sodosol	N				N
C30.60	30596	RCBC	8	3.00	1.20	1.58	Sodosol	Y	7.2	200		N
C31.40	31398	RCP	3	1.20		1.35	Sodosol	N				N
C31.44	31439	RCP	6	1.35		1.53	Sodosol	N				N
C32.03	32027	RCP	13	0.90		1.43	Sodosol	Y	5.4	200		N
C32.82	32820	RCP	8	0.90		1.12	Sodosol	Y	5.4	100		N
C33.67	33665	RCP	9	1.50		1.40	Sodosol	Y	9.0	200		N
C35.18	35176	RCP	16	1.80		1.60	Sodosol	Y	10.8	200		N
C41.01	41009	RCP	6	1.20		1.17	Kurosol	Y	7.2	100		N
C41.11	41112	RCP	18	1.20		1.27	Kurosol	Y	7.2	100		N
C41.20	41197	RCP	14	1.20		1.27	Kurosol	Y	7.2	100		N
C41.32	41318	RCP	8	1.20		1.38	Kurosol	Y	7.2	200		N
C42.87	42873	RCP	15	0.90		0.92	Kurosol	N				N
C43.02	43016	RCP	15	1.20		0.85	Kurosol	N				N
C43.09	43086	RCP	30	1.20		0.78	Kurosol	N				N
C43.16	43158	RCBC	9	3.00	1.80	0.55	Kurosol	N			Amber	Y
C43.35	43346	RCP	45	1.20		0.33	Kurosol	N				N
C43.56	43555	RCP	10	1.20		0.57	Kurosol	Y	7.2	100		N
C43.66	43660	RCP	15	1.20		0.48	Kurosol	N				N
C43.77	43767	RCP	15	1.20		0.19	Kurosol	N				N
C43.86	43864	RCP	15	1.20		0.46	Kurosol	N				N
C43.97	43970	RCP	15	1.20		0.68	Kurosol	N				N
C44.32	44323	RCP	15	1.20		0.86	Kurosol	Y	7.2	100		N
C44.67	44670	RCP	15	1.20		1.40	Kurosol	Y	7.2	200		N
C44.78	44781	RCP	30	0.90		0.99	Kurosol	N				N
C44.88	44878	RCP	30	0.90		1.01	Kurosol	N				N
C44.96	44962	RCP	15	0.90		0.97	Kurosol	N				N
C45.01	45005	RCP	35	0.90		1.11	Kurosol	N				N
C45.10	45095	RCP	30	0.90		1.12	Kurosol	N				N
C45.24	45237	RCP	35	0.90		0.92	Kurosol	N				N

Culvert ID	Chainage	Type	Number	Diameter/Width(m) – Culverts Span (m) – Bridges	Height (m) – Culverts Soffit Level (m AHD) - Bridges	1% AEP Peak Outlet Velocity (m/s)	Downstream Scour Protection				Fish Passageway	
							Material	Scour protection required (Y/N) ¹	Length	Rock d50	WWBW zoning colour	Fish passage required (Y/N)
C45.30	45300	RCP	35	0.90		1.09	Kurosol	N				N
C45.40	45395	RCP	40	0.90		0.86	Kurosol	N				N
C45.45	45452	RCP	40	0.90		0.79	Kurosol	N				N
C45.53	45525	RCP	40	0.90		0.98	Kurosol	N				N
C45.64	45642	RCP	7	0.90		1.34	Kurosol	N				N
C46.46	46464	RCP	11	1.20		1.62	Kurosol	Y	7.2	200		N
C48.41	48410	RCP	21	2.70		2.08	Rudosol	Y	16.2	100	Amber	Y
C51.82	51825	RCBC	11	2.70	0.90	1.16	Rudosol	Y	5.4	100		N
310-BR03		BRIDGE		207.00	276.02	1.21	Rudosol	N/A			Red	Y
C53.20	53202	RCP	9	1.20		1.36	Kurosol	Y	7.2	200		N
C53.62	53618	RCBC	4	1.20	0.90	1.09	Kurosol	Y	5.4	100		N
C54.44	54439	RCBC	3	1.20	0.90	1.06	Kurosol	N				N
C55.06	55056	RCP	7	1.05		1.77	Kurosol	Y	6.3	200		N
310-BR04		BRIDGE		207.00	286.96	0.62	Kurosol	N/A			Purple	Y
C60.18	60176	RCP	2	1.20		2.62	Rudosol	Y	9.6	300		N
C60.44	60443	RCP	18	2.70		1.89	Rudosol	Y	16.2	100	Green	Y
310-BR51		BRIDGE		75.00	308.59	0.58	Rudosol	N/A				N
C61.60	61598	RCP	5	2.70		2.21	Rudosol	Y	16.2	100	Green	Y
C61.90	61899	RCP	5	2.70		2.23	Rudosol	Y	16.2	100	Amber	Y
C62.94	62940	RCBC	3	2.40	2.10	1.44	Rudosol	Y	16.8	300	Green	Y
C63.15	63148	RCP	9	2.10		1.92	Rudosol	Y	16.8	300	Green	Y
C64.42	64415	RCP	6	0.90		1.34	Rudosol	Y	5.4	200		N
C64.48	64475	RCP	8	0.90		1.24	Rudosol	N				N
C64.50	64496	RCP	12	0.90		0.57	Rudosol	N				N
C65.11	65114	RCBC	15	2.10	0.90	0.90	Rudosol	Y	5.4	100		N
C66.23	66232	RCBC	8	2.40	1.80	3.19	Sodosol	Y	14.4	400	Amber	Y
C66.81	66814	RCBC	7	2.70	1.20	1.05	Sodosol	Y	7.2	100		N
310-BR05		BRIDGE		345.00	285.31	0.67	Sodosol	N/A			Purple	Y
C67.57	67573	RCP	8	1.20		0.23	Sodosol	N				N
C67.64	67639	RCP	8	1.20		0.47	Sodosol	N				N
C67.70	67704	RCP	8	1.20		0.72	Sodosol	N				N
C67.83	67834	RCP	20	1.20		0.83	Sodosol	N				N
C67.96	67959	RCP	20	1.20		1.09	Sodosol	N				N
C68.09	68089	RCP	20	1.20		1.24	Sodosol	N				N
C68.41	68410	RCP	20	1.20		1.27	Sodosol	N				N
C68.76	68756	RCP	40	2.10		0.54	Sodosol	N				N
C68.89	68890	RCP	2	1.20		1.27	Sodosol	N				N
C69.02	69020	RCP	2	1.20		1.93	Sodosol	N				N

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							Material	Scour protection required (Y/N) ¹	Length	Rock d50	WWBW zoning colour	Fish passage required (Y/N)
C69.10	69097	RCP	2	1.20		1.89	Sodosol	Y	7.2	200		N
C69.14	69143	RCP	2	1.50		1.75	Sodosol	Y	9.0	200		N
C69.21	69215	RCP	2	1.80		1.79	Sodosol	Y	10.8	200		N
C69.28	69281	RCP	2	1.80		1.70	Sodosol	Y	10.8	200		N
C69.41	69410	RCP	5	1.80		1.51	Sodosol	Y	10.8	200		N
C69.54	69541	RCP	5	1.80		1.46	Sodosol	N				N
C69.67	69671	RCP	5	1.80		1.46	Sodosol	N				N
C69.80	69801	RCP	5	1.80		1.44	Sodosol	N				N
C70.50	70498	RCBC	3	1.20	1.20	1.24	Sodosol	Y	7.2	100		N
C71.51	71512	RCBC	3	2.40	0.90	1.08	Sodosol	Y	5.4	100		N
C73.37	73374	RCP	2	1.20		2.15	Sodosol	Y	7.2	200		N
C73.44	73441	RCP	1	1.20		1.31	Sodosol	N				N
C73.53	73529	RCP	1	1.35		1.92	Sodosol	N				N
C73.61	73615	RCP	2	0.90		1.16	Sodosol	Y	5.4	100		N
C73.68	73685	RCBC	3	3.00	1.80	1.88	Sodosol	N			Green	Y
C74.97	74967	RCP	3	1.20		1.60	Sodosol	N				N
C76.57	76572	RCBC	18	1.20	0.90	1.04	Sodosol	Y	5.4	100		N
C77.20	77197	RCP	7	1.50		1.51	Sodosol	Y	9.0	200		N
C77.47	77466	RCP	5	0.90		1.37	Sodosol	N				N
C77.77	77771	RCBC	8	1.50	1.50	1.18	Sodosol	Y	9.0	200	Green	Y
C78.28	78280	RCBC	4	2.10	0.90	0.98	Sodosol	N				N
C79.01	79005	RCBC	6	2.40	1.80	1.39	Sodosol	Y	10.8	200	Green	Y
C79.53	79526	RCP	8	1.35		1.30	Sodosol	Y	8.1	200		N
C79.98	79978	RCBC	8	0.90	0.90	1.10	Sodosol	Y	5.4	100		N
C80.65	80647	RCBC	6	1.80	1.80	1.23	Sodosol	N			Green	Y
C81.18	81184	RCBC	7	2.40	2.40	1.26	Sodosol	Y	14.4	200	Green	Y
C82.35	82353	RCBC	18	2.40	2.40	1.26	Sodosol	Y	14.4	200	Green	Y
C83.51	83507	RCBC	12	2.40	1.80	0.94	Sodosol	Y	10.8	200	Green	Y
C84.38	84381	RCBC	24	3.00	2.70	1.17	Sodosol	Y	16.2	100	Green	Y
C85.35	85353	RCBC	7	2.40	1.20	1.12	Sodosol	N			Green	Y
C87.18	87181	RCP	6	2.10		1.32	Sodosol	N				N
C87.37	87368	RCBC	15	2.40	1.80	2.05	Sodosol	Y	14.4	300	Red	Y
C87.54	87542	RCBC	3	1.80	1.80	1.16	Sodosol	N				N
C88.11	88111	RCP	3	1.20		1.28	Sodosol	N				N
310-BR06		BRIDGE		138.00	327.38	1.42	Sodosol	N/A			Purple	Y
C90.96	90963	RCBC	40	2.40	2.40	1.23	Sodosol	Y	14.4	200	Amber	Y
C92.08	92080	RCBC	18	2.40	1.50	1.19	Sodosol	Y	9.0	200	Amber	Y
C92.94	92939	RCP	20	1.50		1.37	Sodosol	Y	9.0	200		N

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							Material	Scour protection required (Y/N) ¹	Length	Rock d50	WWBW zoning colour	Fish passage required (Y/N)
C93.61	93606	RCBC	6	3.00	2.10	0.94	Sodosol	N			Green	Y
310-BR07		BRIDGE		184.00	325.42	1.02	Sodosol	N/A			Red	Y
C94.91	94910	RCBC	5	2.10	0.90	1.10	Sodosol	N				N
C95.07	95065	RCBC	18	2.40	1.80	1.20	Sodosol	Y	10.8	200	Amber	Y
C96.20	96197	RCBC	14	2.40	1.20	0.97	Sodosol	Y	7.2	100	Amber	Y
C97.01	97008	RCBC	5	1.50	1.20	1.08	Sodosol	N			Amber	Y
C97.29	97293	RCP	12	0.90		1.55	Sodosol	N				N
C97.38	97382	RCP	12	0.90		1.61	Sodosol	Y	5.4	200		N
310-BR08		BRIDGE		299.00	332.79	0.41	Sodosol	N/A			Amber	Y
C97.84	97836	RCP	8	1.50		0.66	Sodosol	Y	9.0	100		N
C98.36	98360	RCP	10	0.90		2.43	Sodosol	Y	5.4	200		N
C98.87	98866	RCP	1	1.50		1.89	Sodosol	N				N
C99.37	99373	RCP	17	0.90		1.17	Sodosol	N				N
C99.84	99841	RCP	14	0.90		1.61	Sodosol	Y	5.4	200		N
C99.77	99765	RCP	1	1.50		2.40	Sodosol	Y	12.0	300		N
C100.00	100002	RCP	8	1.50		1.34	Sodosol	Y	9.0	200		N
310-BR10		BRIDGE		621.00	334.54	0.58	Sodosol	N/A			Purple	Y
C101.48	101485	RCBC	4	1.50	1.20	0.72	Vertosol	N				N
C102.55	102546	RCBC	4	1.50	0.90	1.01	Vertosol	N				N
310-BR11		BRIDGE		92.00	353.90	1.86	Sodosol	N/A			Red	Y
C104.94	104938	RCP	18	0.90		1.77	Sodosol	N				N
C105.09	105093	RCP	6	0.90		2.23	Sodosol	Y	5.4	200		N
C105.11	105110	RCP	6	0.90		2.04	Sodosol	Y	5.4	200		N
C105.13	105126	RCP	6	0.90		2.14	Sodosol	Y	5.4	200		N
C106.54	106541	RCBC	6	1.20	1.20	1.27	Sodosol	Y	7.2	100		N
C107.22	107222	RCBC	4	2.40	1.80	1.49	Vertosol	Y	10.8	200	Amber	Y
C107.81	107807	RCBC	3	0.90	0.90	1.03	Vertosol	N				N
C107.96	107964	RCP	4	1.20		2.13	Vertosol	Y	7.2	200		N
C108.45	108454	RCP	4	1.50		1.83	Vertosol	Y	9.0	200	Green	Y
C109.43	109430	RCBC	10	1.80	1.80	1.15	Vertosol	N			Amber	Y
C110.91	110912	RCBC	2	1.80	1.50	1.17	Vertosol	Y	9.0	200	Green	Y
C111.16	111165	RCBC	3	1.20	1.20	0.91	Vertosol	N				N
C111.26	111259	RCBC	4	1.20	0.90	1.07	Vertosol	N				N
C112.32	112325	RCBC	2	1.80	1.50	1.25	Vertosol	N			Green	Y
C113.00	113001	RCBC	4	1.20	1.20	1.18	Vertosol	Y	7.2	100		N
C113.07	113073	RCBC	2	2.40	2.40	1.22	Vertosol	Y	14.4	200	Green	Y
C113.28	113281	RCBC	12	2.10	0.90	0.93	Vertosol	Y	5.4	100		N
C114.27	114269	RCP	11	0.90		0.94	Vertosol	N				N

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							Material	Scour protection required (Y/N) ¹	Length	Rock d50	WWBW zoning colour	Fish passage required (Y/N)
C114.36	114360	RCBC	10	2.10	1.50	1.42	Vertosol	N			Amber	Y
C114.90	114900	RCBC	2	1.80	1.50	1.26	Vertosol	N			Green	Y
C115.00	115003	RCP	3	0.90		1.41	Vertosol	Y	5.4	200		N
C115.32	115322	RCP	3	0.90		1.41	Vertosol	N				N
310-BR28		BRIDGE		75.00	456.88	1.86	Vertosol	N/A			Green	Y
C117.40	117401	RCBC	13	3.00	1.80	1.07	Vertosol	Y	10.8	200	Red	Y
C117.60	117600	RCBC	4	2.70	2.10	1.36	Vertosol	N			Amber	Y
C117.71	117706	RCBC	1	1.50	1.50	0.95	Vertosol	N			Amber	Y
C118.10	118099	RCBC	15	2.10	0.90	0.83	Vertosol	Y	5.4	100		N
C118.43	118430	RCBC	10	3.00	1.50	1.13	Vertosol	Y	9.0	200	Amber	Y
C118.61	118606	RCBC	10	1.20	1.20	0.61	Vertosol	Y	7.2	100		N
C118.90	118903	RCP	10	0.90		1.36	Vertosol	Y	5.4	200		N
C119.04	119036	RCBC	2	2.40	1.80	1.10	Vertosol	N			Green	Y
C119.30	119298	RCBC	15	1.20	1.20	0.55	Vertosol	Y	7.2	100		N
C119.38	119378	RCBC	10	3.00	1.50	1.03	Vertosol	Y	9.0	200	Red	Y
C119.75	119751	RCBC	2	2.40	1.20	1.12	Vertosol	N				N
C119.88	119875	RCBC	8	1.50	1.20	0.89	Vertosol	Y	7.2	100		N
C120.08	120080	RCBC	5	1.50	0.90	0.58	Vertosol	Y	5.4	100		N
C120.25	120252	RCBC	26	2.40	1.50	1.12	Vertosol	Y	9.0	200		N
C120.31	120310	RCBC	3	2.40	1.80	1.51	Vertosol	Y	10.8	200	Amber	Y
C120.77	120765	RCBC	13	2.40	1.80	1.35	Vertosol	Y	10.8	200	Green	Y
C124.44	124442	RCP	10	1.80		1.31	Vertosol	N			Amber	Y
C125.48	125483	RCP	10	1.80		1.73	Vertosol	Y	10.8	200	Green	Y
C125.72	125716	RCP	1	0.90		2.47	Vertosol	Y	2.7	200		N
C125.82	125824	RCP	1	1.80		1.83	Vertosol	Y	7.2	300		N
C126.77	126767	RCP	12	0.90		0.60	Sodosol	Y	5.4	100		N
C126.81	126807	RCP	12	0.90		1.15	Sodosol	Y	5.4	100		N
310-BR37		BRIDGE		167.00	417.36	0.86	Sodosol	N/A			Purple	Y
310-BR38		BRIDGE		230.00	407.41	1.07	Sodosol	N/A			Purple	Y
C128.89	128893	RCBC	28	1.20	0.90	0.99	Sodosol	N				N
C133.02	133020	RCBC	20	1.20	0.90	0.93	Sodosol	Y	5.4	100		N
C135.72	135716	RCBC	10	1.80	0.90	0.53	Sodosol	N				N
C135.79	135790	RCBC	10	1.80	0.90	0.78	Sodosol	N				N
C135.91	135910	RCBC	35	1.80	0.90	0.90	Sodosol	Y	5.4	100		N
C139.13	139129	RCP	8	1.35		1.03	Vertosol	N				N
C139.17	139170	RCP	11	1.65		1.08	Vertosol	N				N
C139.21	139205	RCP	8	1.80		1.62	Vertosol	Y	10.8	200		N
310-BR21/22		BRIDGE		1330.00	382.05	1.15	Vertosol	N/A			Amber	Y

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							Material	Scour protection required (Y/N) ¹	Length	Rock d50	WWBW zoning colour	Fish passage required (Y/N)
C140.66	140663	RCP	11	1.80		0.95	Vertosol	N				N
C140.73	140730	RCP	8	2.10		0.68	Vertosol	N				N
C140.79	140787	RCP	8	2.10		0.74	Vertosol	N				N
C140.85	140846	RCP	11	1.80		0.84	Vertosol	N				N
C141.00	141001	RCP	9	1.65		1.02	Vertosol	N				N
C141.02	141025	RCBC	4	2.40	2.10	1.09	Vertosol	N			Amber	Y
C141.07	141068	RCP	10	2.10		1.53	Vertosol	Y	16.8	300		N
C141.38	141377	RCP	7	1.80		1.67	Vertosol	Y	10.8	200		N
C141.40	141401	RCP	7	1.80		1.67	Vertosol	Y	10.8	200		N
C141.46	141460	RCP	6	2.10		1.06	Vertosol	N				N
C141.49	141494	RCP	6	2.10		1.19	Vertosol	N				N
C141.52	141521	RCP	6	2.10		1.14	Vertosol	N				N
C141.54	141541	RCP	6	2.10		1.13	Vertosol	N				N
C141.56	141563	RCP	6	2.10		1.42	Vertosol	N				N
C141.61	141609	RCP	6	2.10		1.31	Vertosol	N				N
C141.67	141665	RCP	6	2.10		1.07	Vertosol	N				N
C141.69	141689	RCP	6	2.10		0.84	Vertosol	N				N
C141.72	141721	RCP	7	1.80		0.84	Vertosol	N				N
C141.75	141753	RCP	5	2.10		0.81	Vertosol	N				N
C141.78	141778	RCP	6	2.10		0.80	Vertosol	N				N
C141.80	141801	RCP	6	2.10		0.84	Vertosol	N				N
C141.84	141842	RCP	5	2.10		0.93	Vertosol	N				N
C141.88	141879	RCP	5	2.10		1.14	Vertosol	N				N
C141.93	141926	RCP	6	2.10		1.41	Vertosol	N				N
C141.96	141963	RCP	5	2.10		1.50	Vertosol	Y	16.8	300		N
C142.07	142072	RCP	6	2.10		1.90	Vertosol	Y	16.8	300		N
C142.12	142118	RCP	6	2.10		1.85	Vertosol	Y	16.8	300		N
C142.16	142164	RCP	4	2.10		1.86	Vertosol	Y	16.8	300		N
C142.20	142203	RCP	6	2.10		1.85	Vertosol	Y	16.8	300		N
C142.27	142267	RCP	6	2.10		1.81	Vertosol	Y	16.8	300		N
C142.32	142320	RCP	4	2.10		1.78	Vertosol	Y	16.8	300		N
C142.36	142362	RCP	6	2.10		1.83	Vertosol	Y	16.8	300		N
C142.40	142405	RCP	6	2.10		1.84	Vertosol	Y	16.8	300		N
C142.49	142489	RCP	6	2.10		1.70	Vertosol	Y	16.8	300		N
C142.53	142533	RCP	6	2.10		1.78	Vertosol	Y	16.8	300		N
C142.58	142579	RCP	6	2.10		1.49	Vertosol	Y	16.8	300		N
C142.61	142614	RCP	4	2.10		1.65	Vertosol	Y	16.8	300		N
310-BR24		BRIDGE		658.00	382.05	1.23	Vertosol	N/A				N

Culvert ID	Chainage	Type	Number	Diameter/Width(m) – Culverts Span (m) – Bridges	Height (m) – Culverts Soffit Level (m AHD) - Bridges	1% AEP Peak Outlet Velocity (m/s)	Downstream Scour Protection				Fish Passageway	
							Material	Scour protection required (Y/N) ¹	Length	Rock d50	WWBW zoning colour	Fish passage required (Y/N)
C143.29	143288	RCP	6	2.10		1.16	Vertosol	N				N
C143.33	143334	RCP	6	2.10		1.28	Vertosol	N				N
C143.37	143372	RCP	6	2.10		1.37	Vertosol	N				N
C143.42	143417	RCP	6	2.10		1.31	Vertosol	N				N
C143.44	143445	RCP	6	2.10		1.26	Vertosol	N				N
C143.48	143478	RCP	6	2.10		1.39	Vertosol	N				N
C143.51	143509	RCP	6	2.10		1.34	Vertosol	N				N
C143.54	143538	RCP	6	2.10		1.41	Vertosol	N				N
C143.57	143574	RCP	5	2.10		1.52	Vertosol	Y	16.8	300		N
C143.65	143654	RCP	6	2.10		1.49	Vertosol	N				N
C143.70	143701	RCP	6	2.10		1.37	Vertosol	N				N
C143.73	143734	RCP	6	2.10		1.41	Vertosol	N				N
C143.77	143768	RCP	6	2.10		1.34	Vertosol	N				N
C143.79	143792	RCP	5	2.10		1.35	Vertosol	N				N
C143.83	143832	RCP	4	2.10		1.43	Vertosol	N				N
C143.87	143875	RCP	5	2.10		1.29	Vertosol	N				N
310-BR25/26		BRIDGE		2577.00	380.60	1.32	Vertosol	N/A			Purple	Y
C146.45	146452	RCBC	4	1.20	0.90	1.73	Vertosol	Y	5.4	200		N
C146.50	146497	RCBC	4	1.20	0.90	1.93	Vertosol	Y	5.4	200		N
C146.54	146536	RCBC	4	1.20	0.90	1.53	Vertosol	Y	5.4	200		N
C146.61	146610	RCBC	2	1.20	0.90	1.51	Vertosol	Y	5.4	200		N
C146.69	146686	RCBC	6	1.20	0.90	1.10	Vertosol	N				N
C147.01	147013	RCBC	10	1.50	0.90	1.30	Vertosol	N				N
C147.12	147116	RCBC	4	1.20	0.90	0.42	Vertosol	N				N
C147.18	147178	RCBC	10	1.50	0.90	1.44	Vertosol	N				N
C147.21	147213	RCBC	4	1.20	0.90	0.80	Vertosol	N				N
C147.27	147265	RCBC	4	1.20	0.90	0.74	Vertosol	N				N
C147.31	147315	RCBC	10	1.50	0.90	1.36	Vertosol	N				N
C147.85	147851	RCBC	6	1.20	0.60	0.61	Vertosol	N				N
C147.91	147914	RCBC	4	1.20	0.60	0.85	Vertosol	N				N
C148.87	148874	RCP	25	1.05		0.35	Vertosol	N				N
C148.92	148915	RCP	25	1.05		0.38	Vertosol	N				N
C148.96	148956	RCP	25	1.05		0.68	Vertosol	N				N
C149.02	149016	RCP	25	1.05		0.90	Vertosol	N				N
310-BR27		BRIDGE		1568.00	383.79	0.97	Vertosol	N/A			Purple	Y
C150.68	150677	RCP	10	1.35		0.83	Vertosol	N				N
C150.71	150708	RCP	12	1.20		0.80	Vertosol	N				N
C150.74	150744	RCP	3	1.35		0.97	Vertosol	N				N

Culvert ID	Chainage	Type	Number	Diameter/Width(m) – Culverts Span (m) – Bridges	Height (m) – Culverts Soffit Level (m AHD) - Bridges	1% AEP Peak Outlet Velocity (m/s)	Downstream Scour Protection				Fish Passageway	
							Material	Scour protection required (Y/N) ¹	Length	Rock d50	WWBW zoning colour	Fish passage required (Y/N)
C151.08	151077	RCP	8	1.20		0.74	Vertosol	N				N
C151.12	151119	RCP	8	1.20		0.58	Vertosol	N				N
C151.14	151142	RCP	8	1.20		0.57	Vertosol	N				N
C151.17	151173	RCP	6	1.35		0.63	Vertosol	N				N
C151.20	151201	RCP	6	1.35		0.64	Vertosol	N				N
C151.27	151269	RCP	8	1.20		0.40	Vertosol	N				N
C151.30	151296	RCP	8	1.05		0.61	Vertosol	N				N
C152.40	152396	RCBC	3	0.90	0.90	0.88	Vertosol	N				N
C153.44	153438	RCBC	2	0.90	0.90	1.02	Vertosol	N				N
C154.51	154508	RCBC	12	2.10	0.90	0.99	Vertosol	Y	5.4	100		N
C155.59	155592	RCBC	9	2.10	1.20	1.13	Vertosol	Y	7.2	100		N
C159.25	159245	RCBC	5	2.40	1.20	1.26	Vertosol	N				N
C160.41	160414	RCP	5	1.80		1.96	Vertosol	Y	14.4	300		N
C161.15	161152	RCP	8	1.20		1.30	Vertosol	N				N
C162.30	162301	RCP	2	1.50		1.45	Vertosol	N				N
310-BR29		BRIDGE		69.00	433.16	0.66	Vertosol	N/A				N
C162.82	162824	RCP	3	1.80		1.92	Vertosol	Y	14.4	300		N
C164.16	164160	RCP	6	0.90		1.31	Vertosol	Y	5.4	200		N
C164.29	164288	RCP	22	1.20		1.56	Vertosol	Y	7.2	200		N
C165.06	165063	RCP	3	1.80		1.94	Vertosol	Y	14.4	300		N
C166.10	166104	RCBC	3	2.40	1.20	1.24	Vertosol	Y	7.2	100		N
C167.08	167082	RCBC	2	1.20	1.20	1.26	Vertosol	Y	7.2	100		N
C168.61	168611	RCBC	4	2.40	0.90	1.19	Vertosol	Y	5.4	100		N
C169.01	169013	RCBC	2	1.20	1.20	1.56	Vertosol	Y	7.2	200		N
C169.89	169886	RCP	5	2.70		2.09	Vertosol	Y	16.2	100	Green	Y
C170.55	170548	RCBC	10	2.40	2.40	1.40	Vertosol	Y	19.2	300	Green	Y
C171.02	171018	RCP	8	1.80		1.66	Vertosol	Y	10.8	200	Green	Y
C171.90	171902	RCP	15	1.80		1.75	Vertosol	Y	10.8	200	Green	Y
310-BR30		BRIDGE		69.00	513.94	0.59	Vertosol	N/A			Green	Y
C173.54	173544	RCP	3	1.35		1.34	Vertosol	N				N
310-BR40		BRIDGE		75.00	528.95	0.65	Vertosol	N/A				N
C176.89	176892	RCBC	3	2.40	2.10	1.61	Vertosol	Y	16.8	300	Green	Y
C177.63	177635	RCP	1	2.10		2.35	Vertosol	Y	8.4	300		N
C178.02	178017	RCP	4	2.10		1.68	Vertosol	Y	16.8	300		N
C178.63	178632	RCP	7	1.80		1.55	Vertosol	Y	10.8	200		N
C181.21	181208	RCP	9	2.10		1.93	Vertosol	Y	16.8	300		N
C181.78	181784	RCP	4	1.80		1.95	Vertosol	Y	14.4	300		N
C182.99	182991	RCP	9	2.40		1.56	Vertosol	Y	19.2	300	Green	Y

Culvert ID	Chainage	Type	Number	Diameter/Width(m) – Culverts Span (m) – Bridges	Height (m) – Culverts Soffit Level (m AHD) - Bridges	1% AEP Peak Outlet Velocity (m/s)	Downstream Scour Protection				Fish Passageway	
							Material	Scour protection required (Y/N) ¹	Length	Rock d50	WWBW zoning colour	Fish passage required (Y/N)
C183.56	183558	RCP	6	2.10		1.62	Vertosol	Y	16.8	300		N
310-BR42		BRIDGE		144.00	530.43	2.52	Vertosol	N/A			Amber	Y
C186.16	186155	RCBC	10	2.40	2.10	1.27	Vertosol	Y	12.6	200	Green	Y
C187.20	187205	RCP	16	2.10		1.81	Vertosol	Y	16.8	300	Green	Y
C189.99	189990	RCBC	11	1.80	1.50	3.06	Vertosol	Y	12.0	300	Amber	Y
C192.09	192086	RCP	7	1.80		1.76	Vertosol	Y	10.8	200	Green	Y
C193.10	193103	RCP	5	3.00		2.70	Vertosol	Y	18.0	100	Amber	Y
C194.66	194659	RCBC	2	1.50	1.20	2.00	Vertosol	Y	7.2	200	Amber	Y
C194.69	194691	RCP	3	1.05		2.65	Vertosol	Y	6.3	200		N
C195.98	195978	RCBC	7	1.50	0.90	1.00	Vertosol	N				N
C196.46	196465	RCP	5	1.20		1.58	Vertosol	Y	7.2	200		N
C196.92	196916	RCP	14	1.35		1.93	Vertosol	Y	8.1	200	Green	Y
C197.18	197181	RCP	2	1.05		1.23	Vertosol	N				N
C197.31	197313	RCP	2	1.05		1.50	Vertosol	Y	6.3	200		N
310-BR20		BRIDGE		92.00	440.15	2.37	Vertosol	N/A			Amber	Y
310-BR31		BRIDGE		230.00	428.95	2.40	Vertosol	N/A			Purple	Y
C198.68	198684	RCP	15	2.40		2.05	Vertosol	Y	19.2	300		N
C198.77	198770	RCP	11	1.50		2.17	Vertosol	Y	9.0	200		N
C198.80	198803	RCP	10	1.20		1.86	Vertosol	Y	7.2	200		N
C198.98	198983	RCP	17	1.05		2.17	Vertosol	Y	6.3	200		N
310-BR32		BRIDGE		184.00	431.25	2.85	Vertosol	N/A			Red	Y
C199.53	199533	RCP	15	1.05		1.68	Vertosol	Y	6.3	200		N
310-BR33		BRIDGE		75.00	436.21	0.79	Vertosol	N/A				N
C200.52	200519	RCP	5	0.90		1.41	Vertosol	Y	5.4	200		N
C200.82	200824	RCBC	13	0.90	0.90	0.93	Vertosol	N				N
C201.23	201233	RCBC	4	0.90	0.90	1.08	Vertosol	Y	5.4	100		N
C201.51	201514	RCBC	3	0.90	0.90	1.07	Vertosol	Y	5.4	100		N
C201.97	201974	RCP	5	1.20		1.61	Vertosol	Y	7.2	200		N
C202.52	202524	RCP	7	1.20		1.58	Vertosol	Y	7.2	200		N
C202.80	202802	RCBC	7	2.10	2.10	1.26	Vertosol	Y	12.6	200	Green	Y
310-BR34		BRIDGE		132.00	481.45	0.72	Vertosol	N/A				N
C204.45	204447	RCP	2	1.05		1.20	Vertosol	N				N
310-BR35		BRIDGE		299.00	463.43	1.47	Vertosol	N/A			Green	Y
C206.17	206167	RCP	2	1.05		1.45	Vertosol	N				N
C206.36	206359	RCP	12	1.05		1.40	Vertosol	N				N
C206.42	206421	RCP	2	1.05		1.71	Vertosol	Y	6.3	200		N
C206.57	206574	RCP	4	1.05		1.47	Vertosol	Y	6.3	200		N
C206.65	206651	RCP	15	1.05		1.35	Vertosol	Y	6.3	200		N

Culvert ID	Chainage	Type	Number	Diameter/Width(m) – Culverts Span (m) – Bridges	Height (m) – Culverts Soffit Level (m AHD) - Bridges	1% AEP Peak Outlet Velocity (m/s)	Downstream Scour Protection				Fish Passageway	
							Material	Scour protection required (Y/N) ¹	Length	Rock d50	WWBW zoning colour	Fish passage required (Y/N)
C206.75	206746	RCP	5	1.05		1.13	Vertosol	N				N
C206.88	206876	RCP	2	1.05		1.77	Vertosol	Y	6.3	200		N
C207.15	207146	RCP	7	1.35		1.96	Vertosol	Y	8.1	200	Green	Y
C207.70	207705	RCBC	1	1.80	0.90	1.36	Vertosol	Y	2.7	200		N
C208.21	208208	RCBC	16	2.40	1.50	1.64	Vertosol	Y	9.0	200	Amber	Y

¹ Where areas immediately downstream of culvert outlets have been identified within the revised draft EIS to experience FIO velocity exceedances, additional scour protection has been provided in these locations beyond those locations identified within the revised reference design. Refer to Section 22.4.1.2 for further detail. The scour protection design will be reviewed at the Detailed Design stage in conjunction with various forms of further assessment / testing (soil analysis / testing, geomorphic review, etc). Accordingly what is presented in this report for the revised reference design scour protection is preliminary only, and subject to change at future design stages. This is with a view to enhancing the overall design performance and compliance of the rail infrastructure.

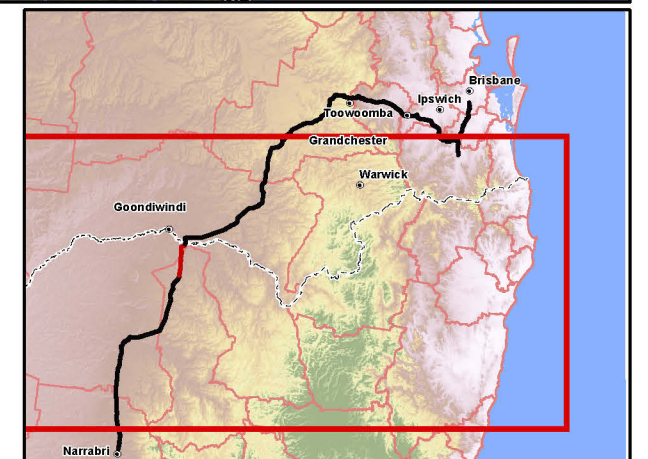
N/A – bridge scour assessment has not been undertaken for feasibility design

T1

Hydrology and Flooding Technical Report—Volume 1

Appendix C Macintyre River hydrologic model update mapping

BORDER TO GOWRIE REVISED DRAFT ENVIRONMENTAL IMPACT STATEMENT

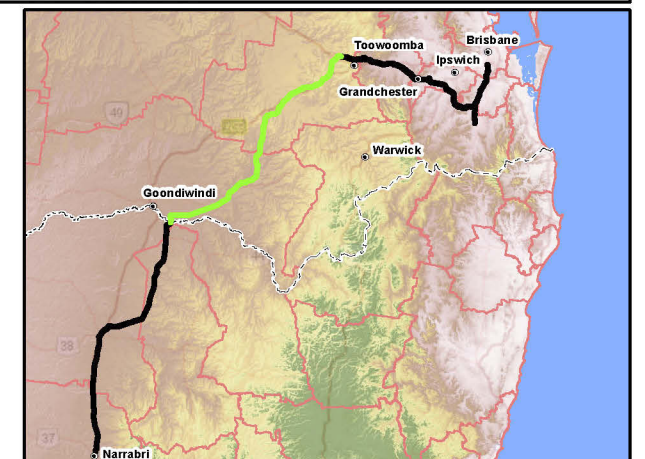
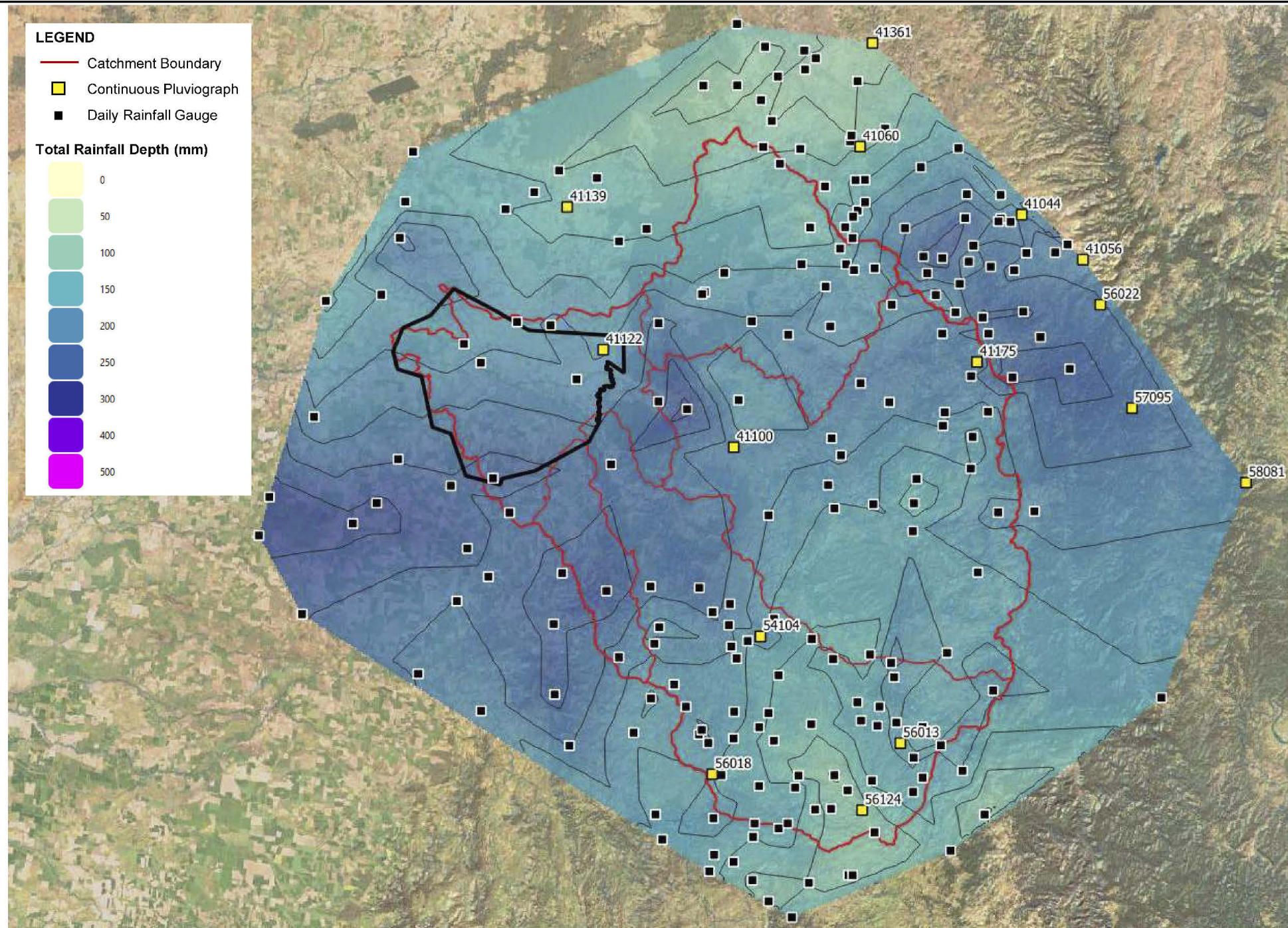


- Localities
- Existing rail (operational)
- + Existing rail (non-operational)
- North Star to NSW/QLD border alignment
- Adjoining alignments
- - - NSW/QLD border
- Watercourses
- NS2B upstream dams
- TUFLOW model boundary

- Macintyre Brook
- Dumaresq River
- Macintyre River
- Ottleys Creek
- Lower floodplain

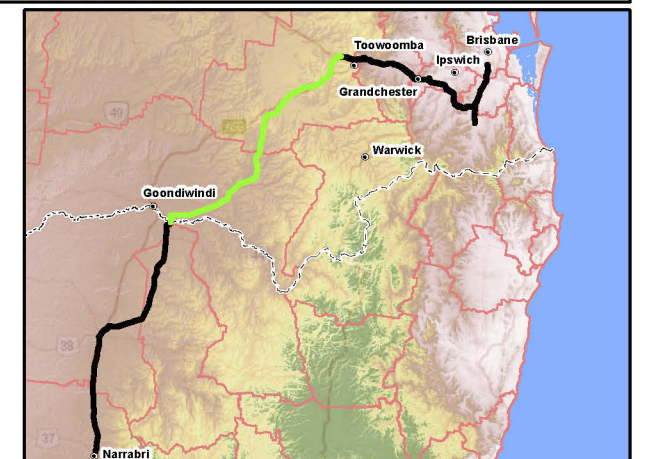
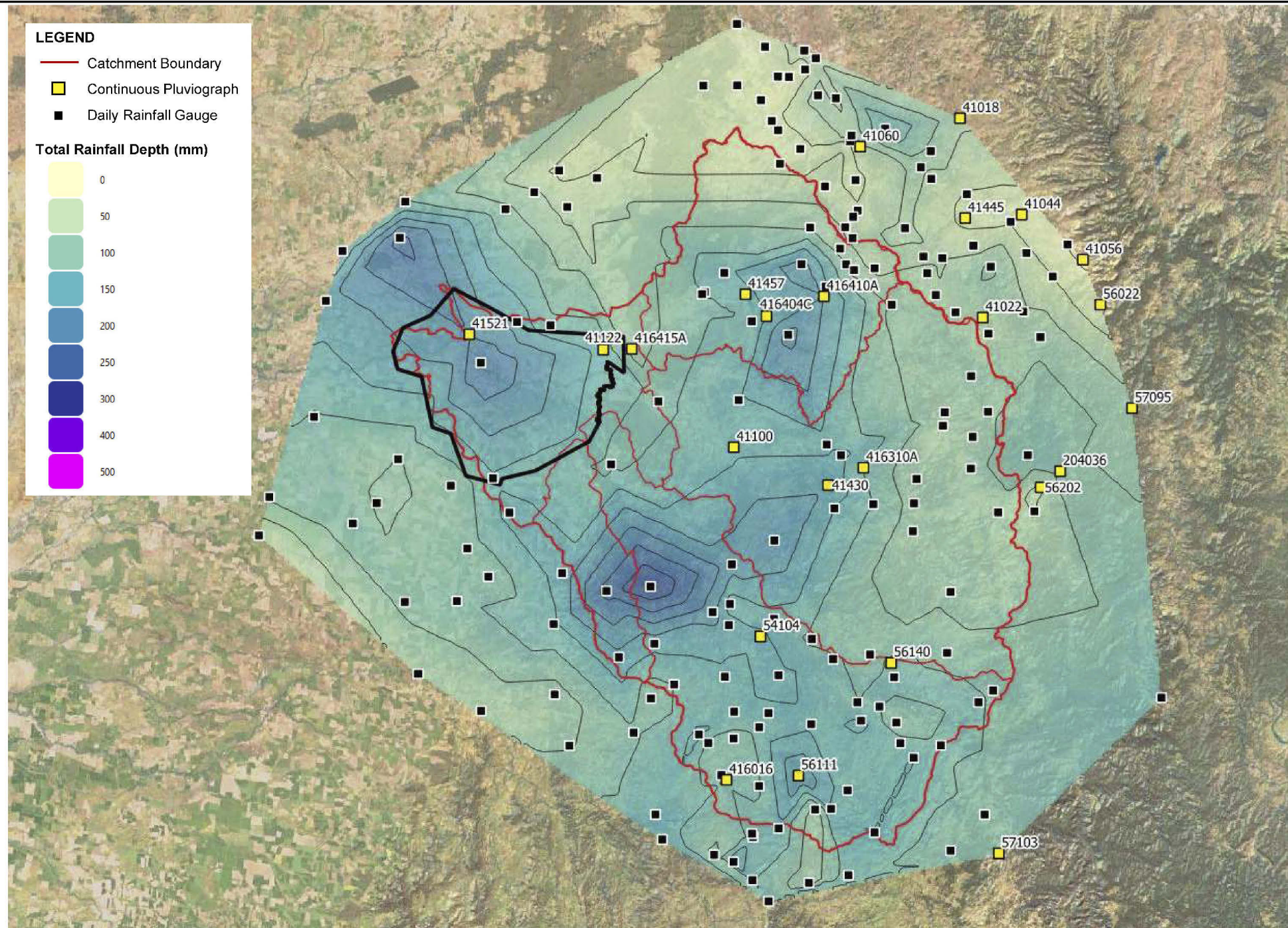


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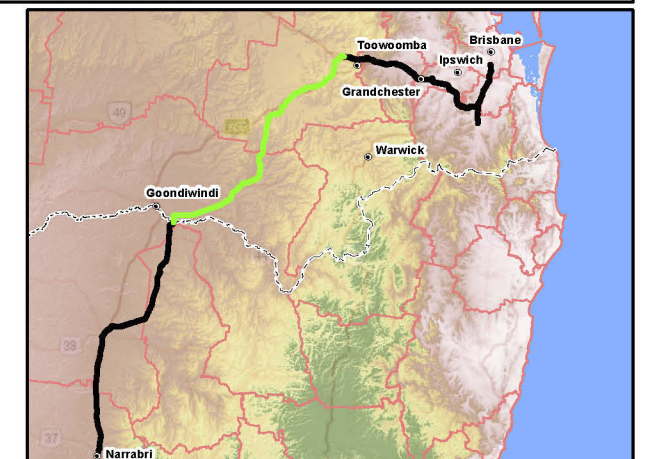
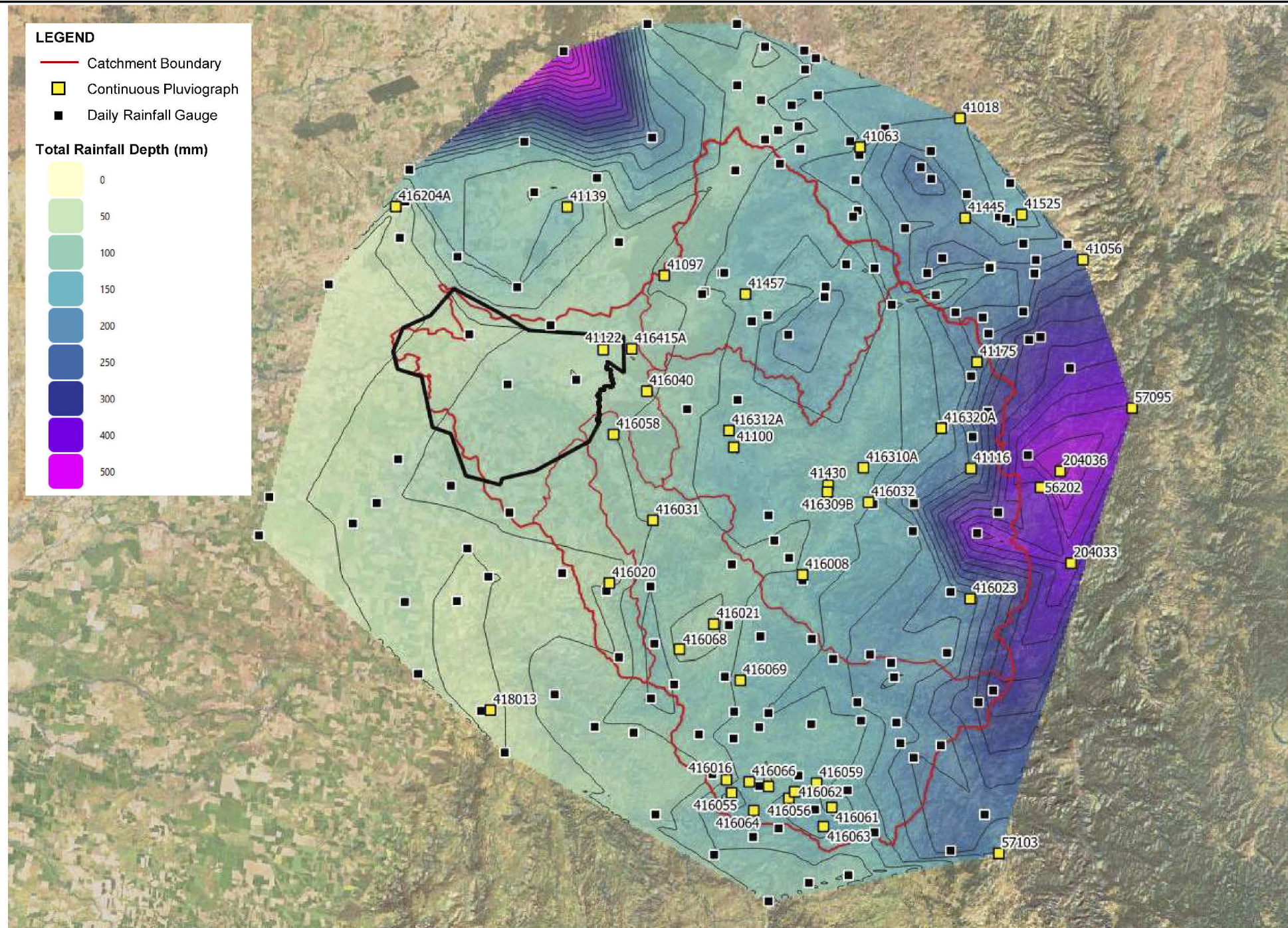
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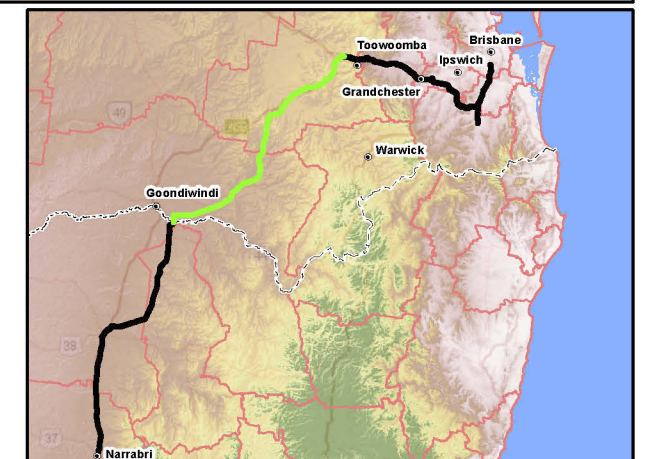
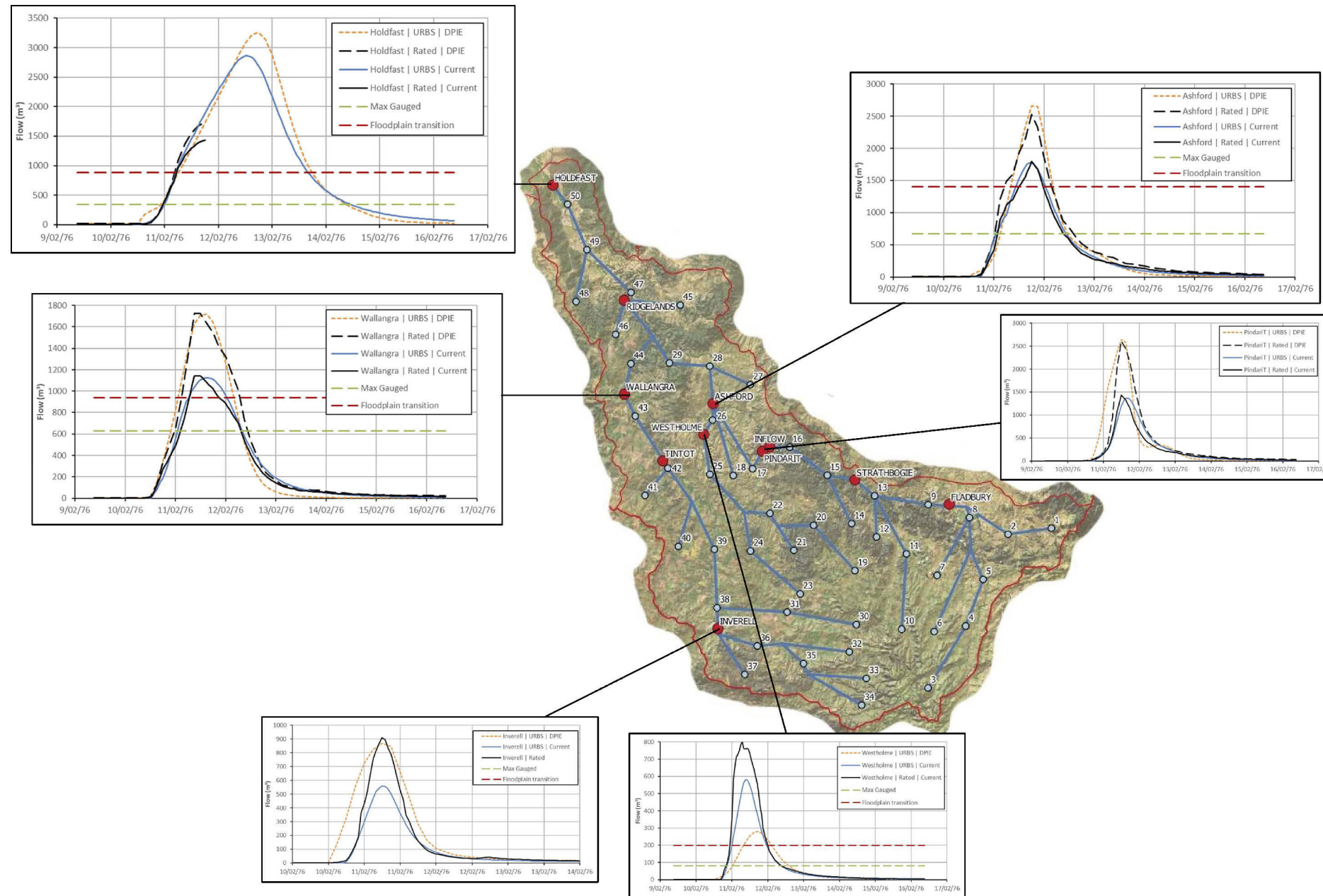
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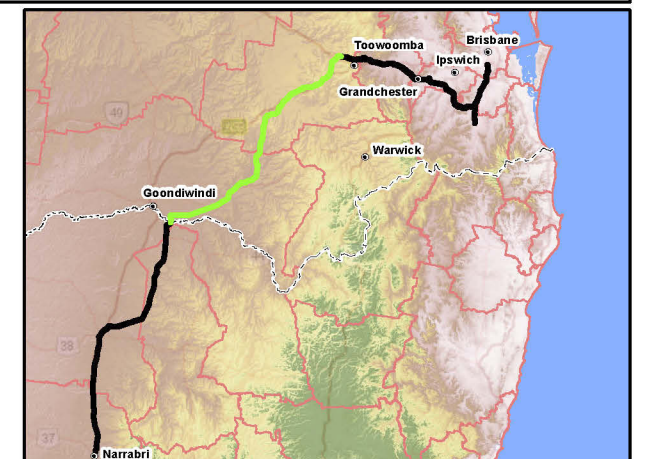
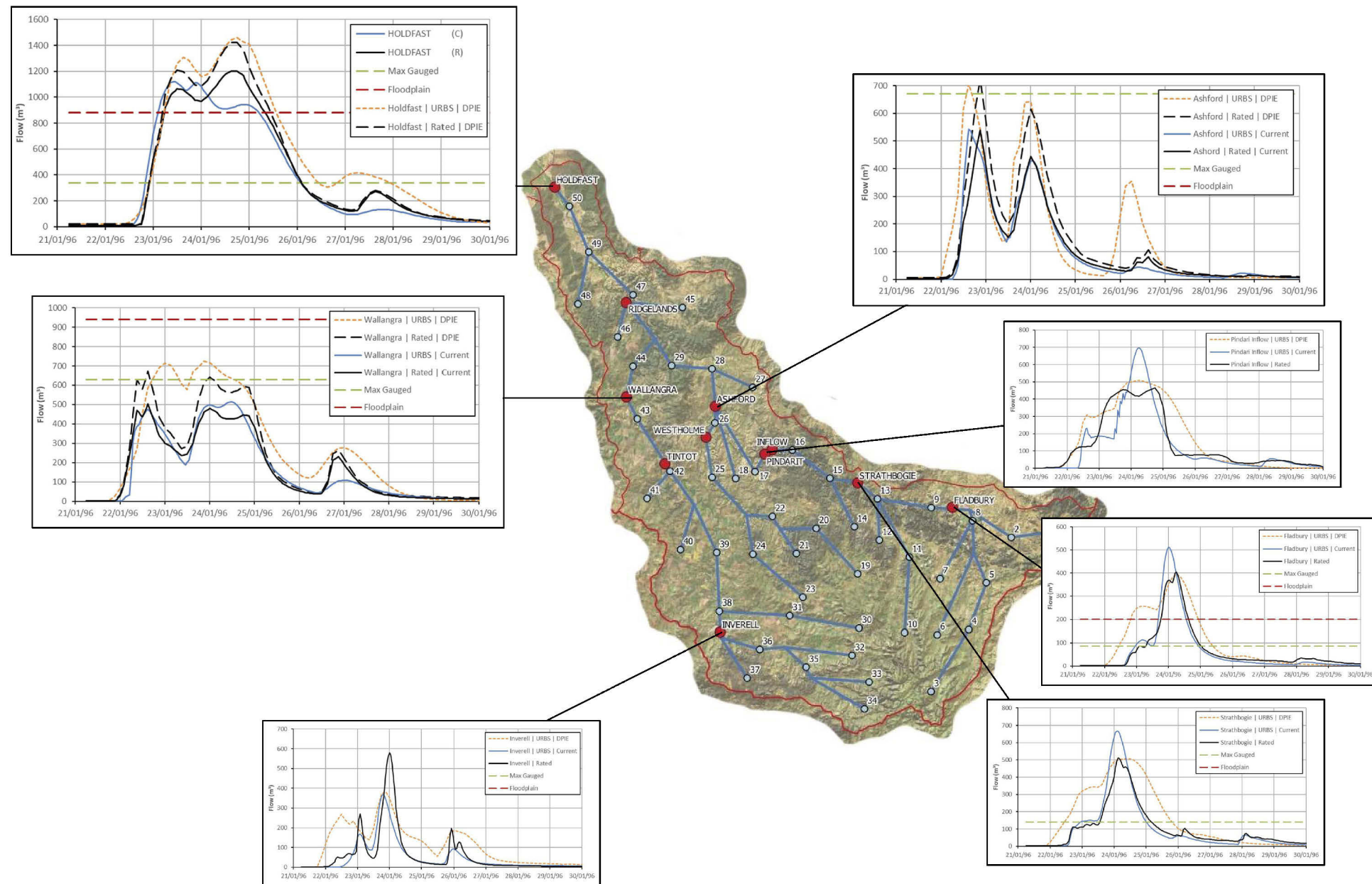
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MACINTYRE RIVER TO HOLDFAST – 1976



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MACINTYRE RIVER TO HOLDFAST – 1996

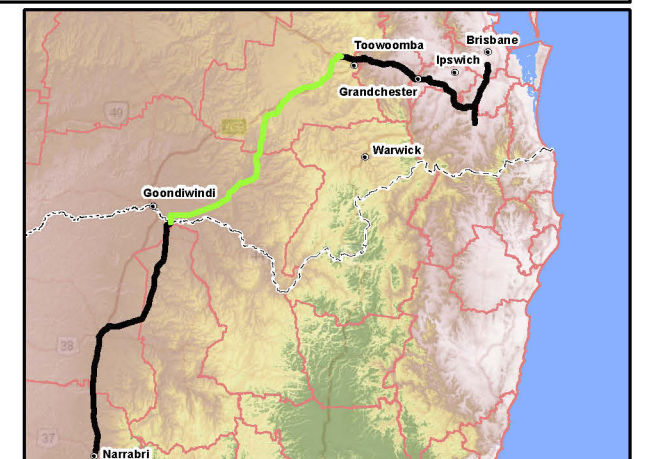
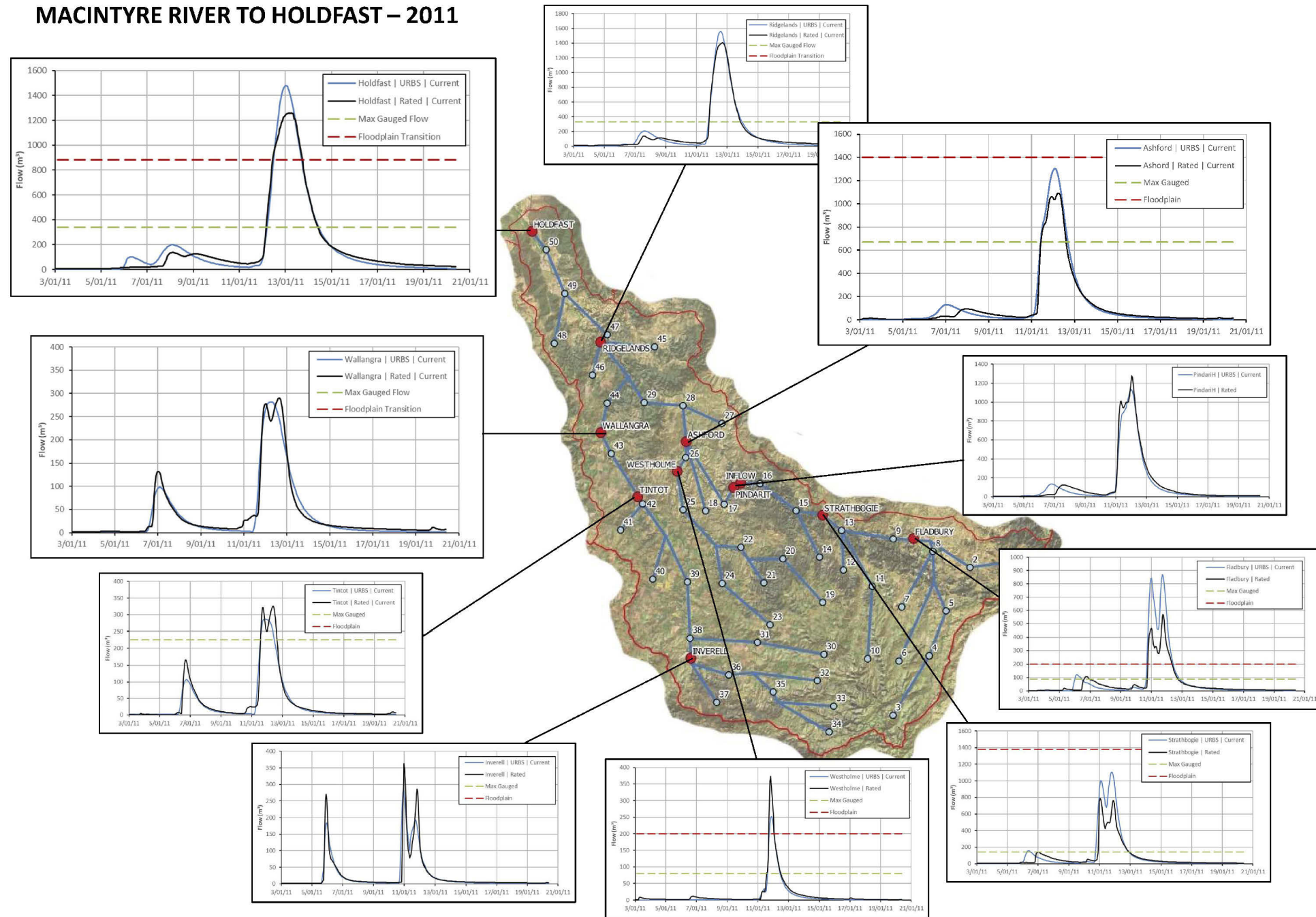


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MACINTYRE RIVER TO HOLDFAST – 2011



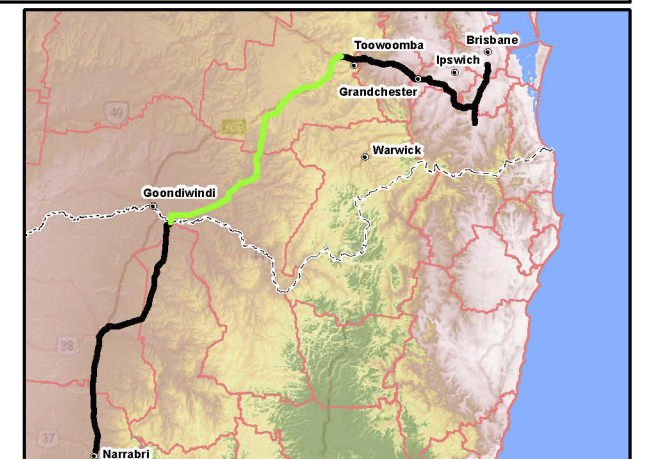
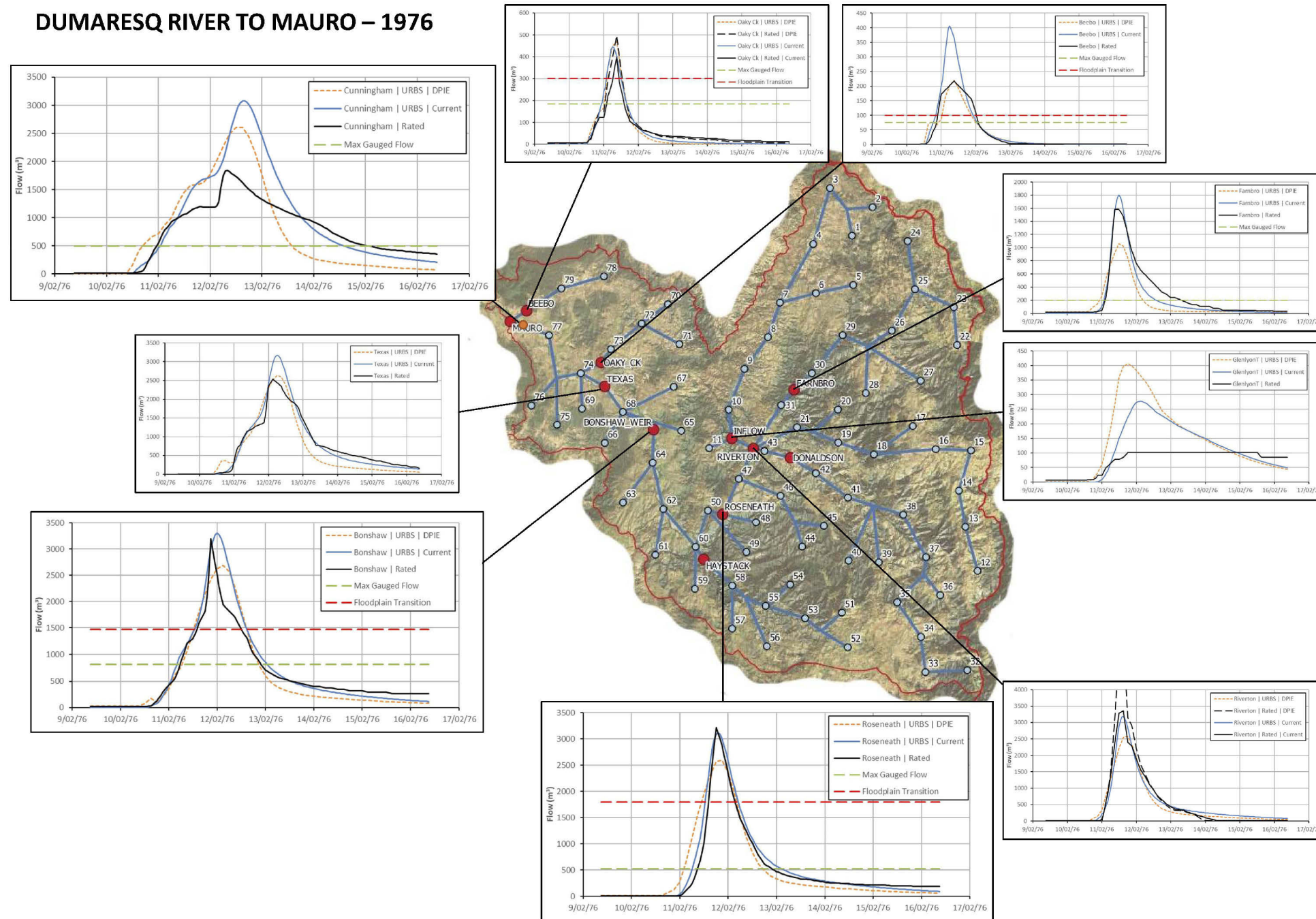
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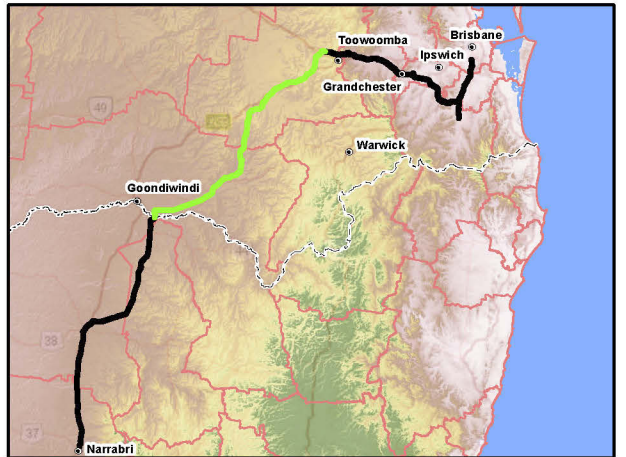
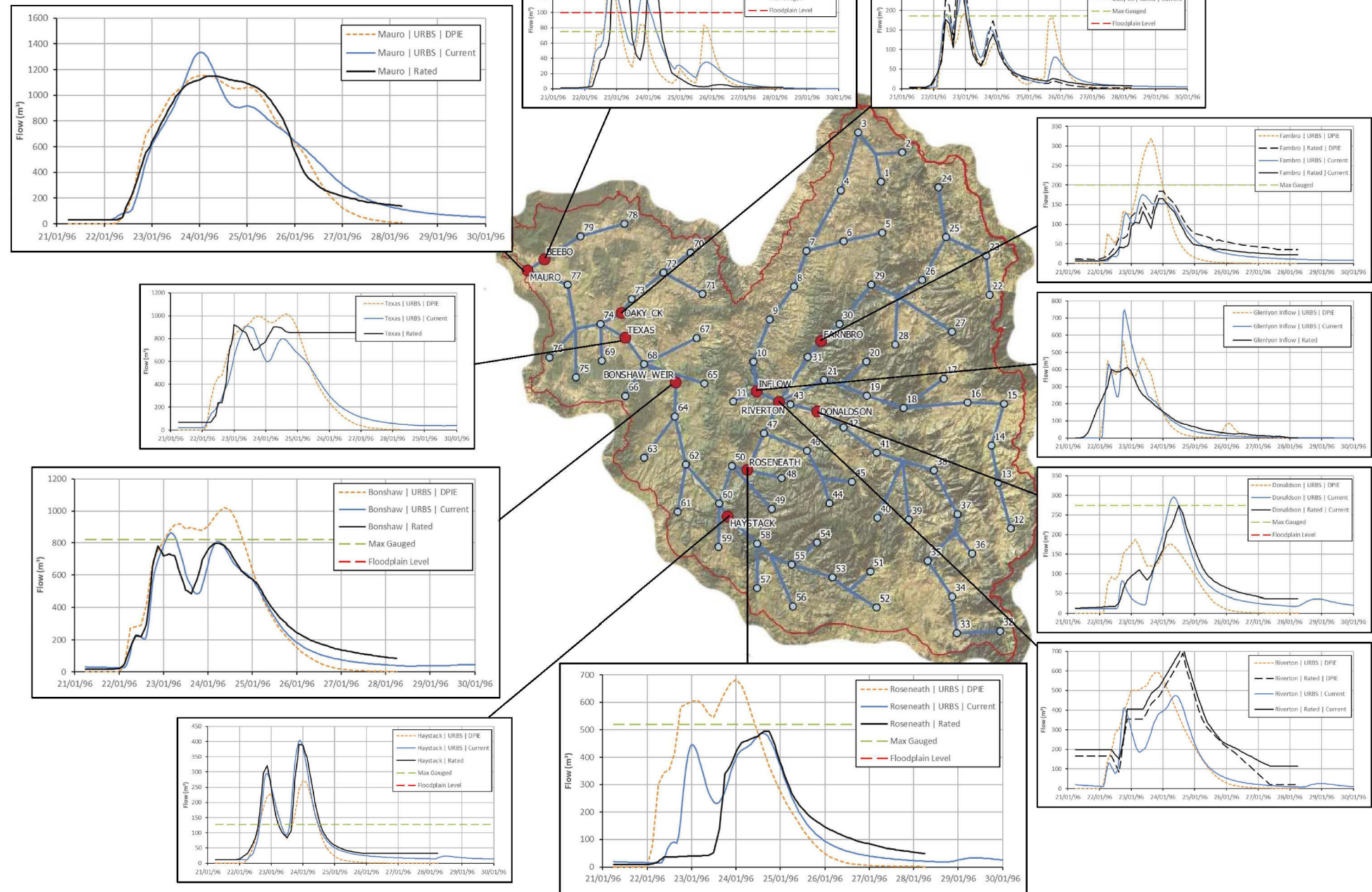
DUMARESQ RIVER TO MAURO – 1976



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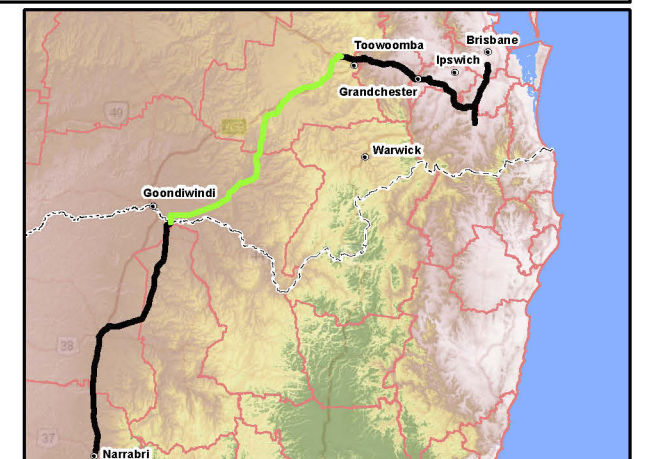
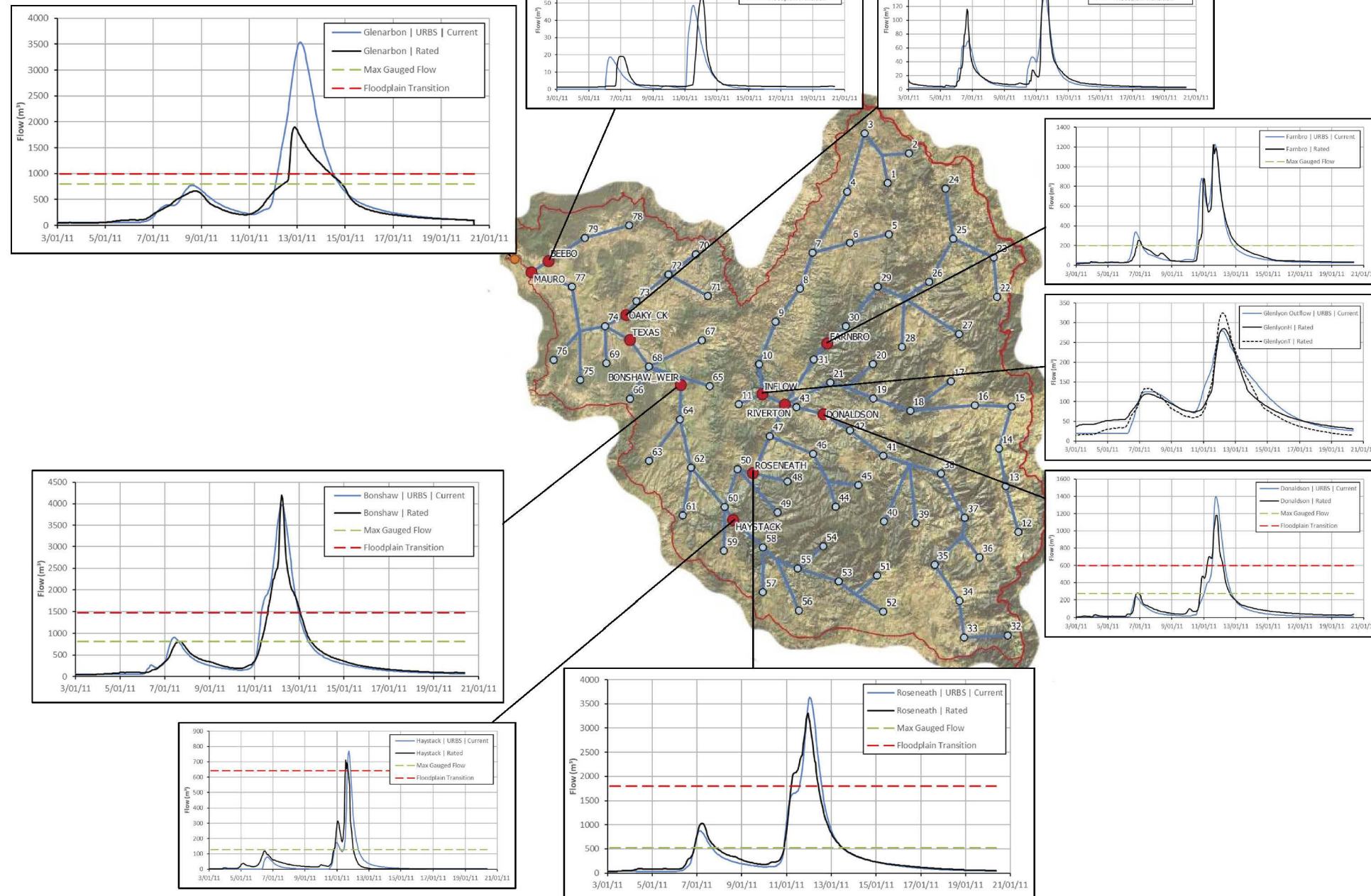
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DUMARESQ RIVER TO MAURO – 1996



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DUMARESQ RIVER TO MAURO – 2011

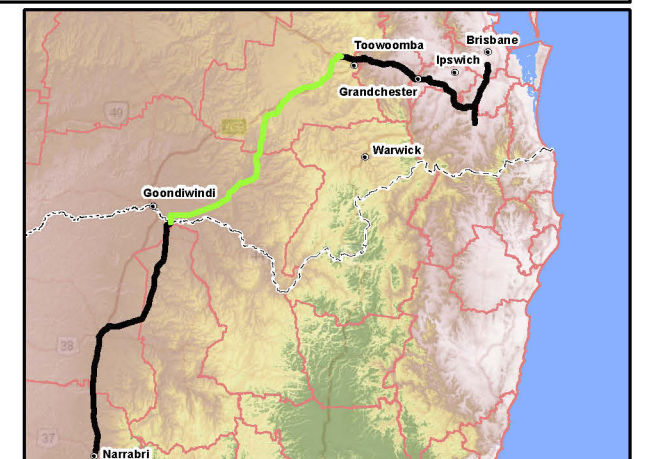
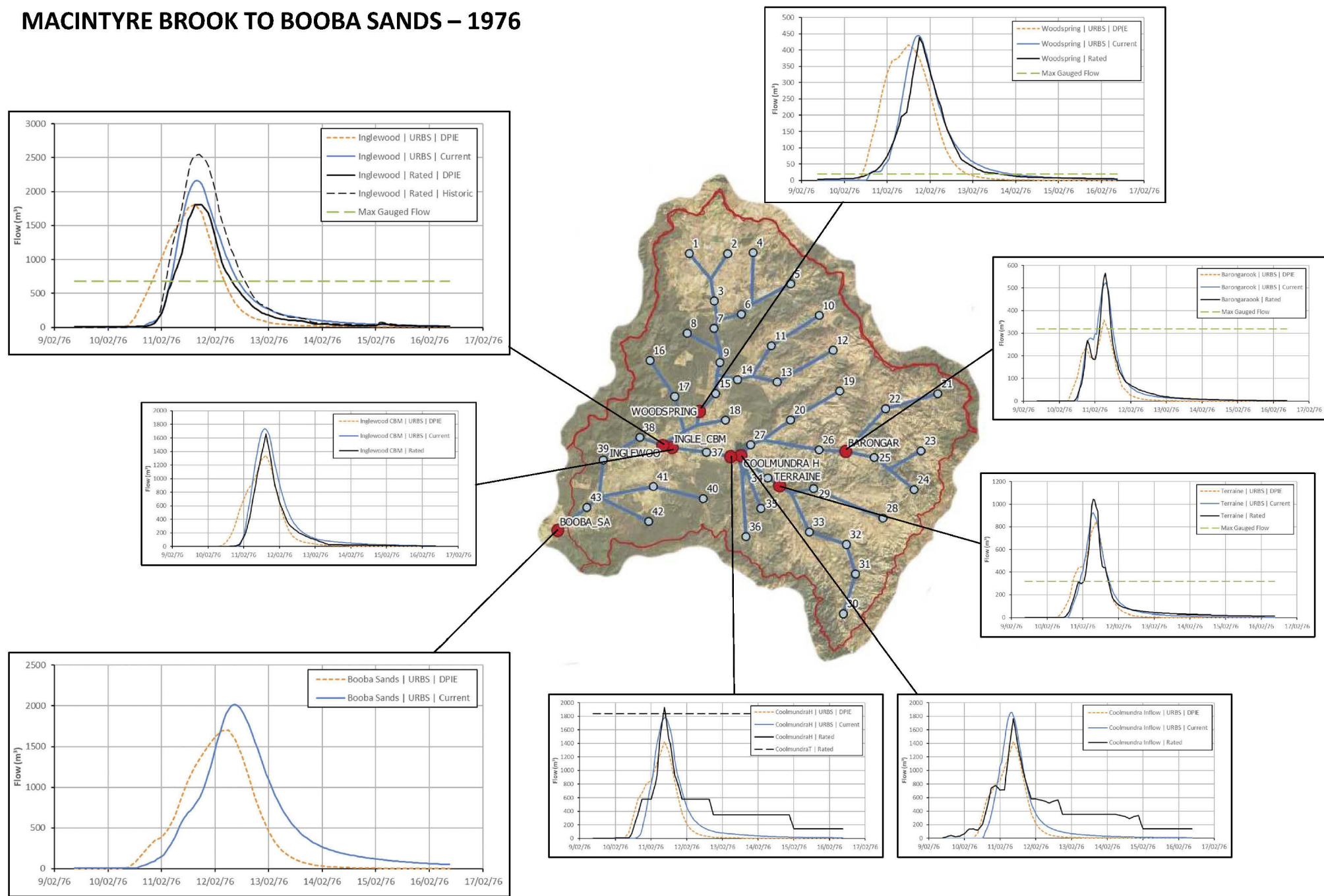


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MACINTYRE BROOK TO BOOBA SANDS – 1976

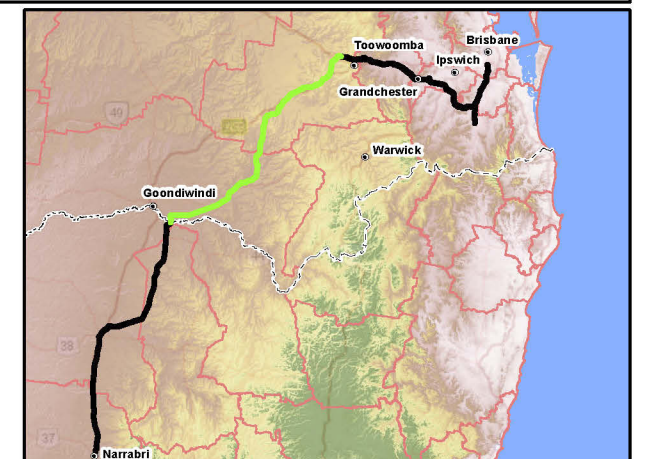
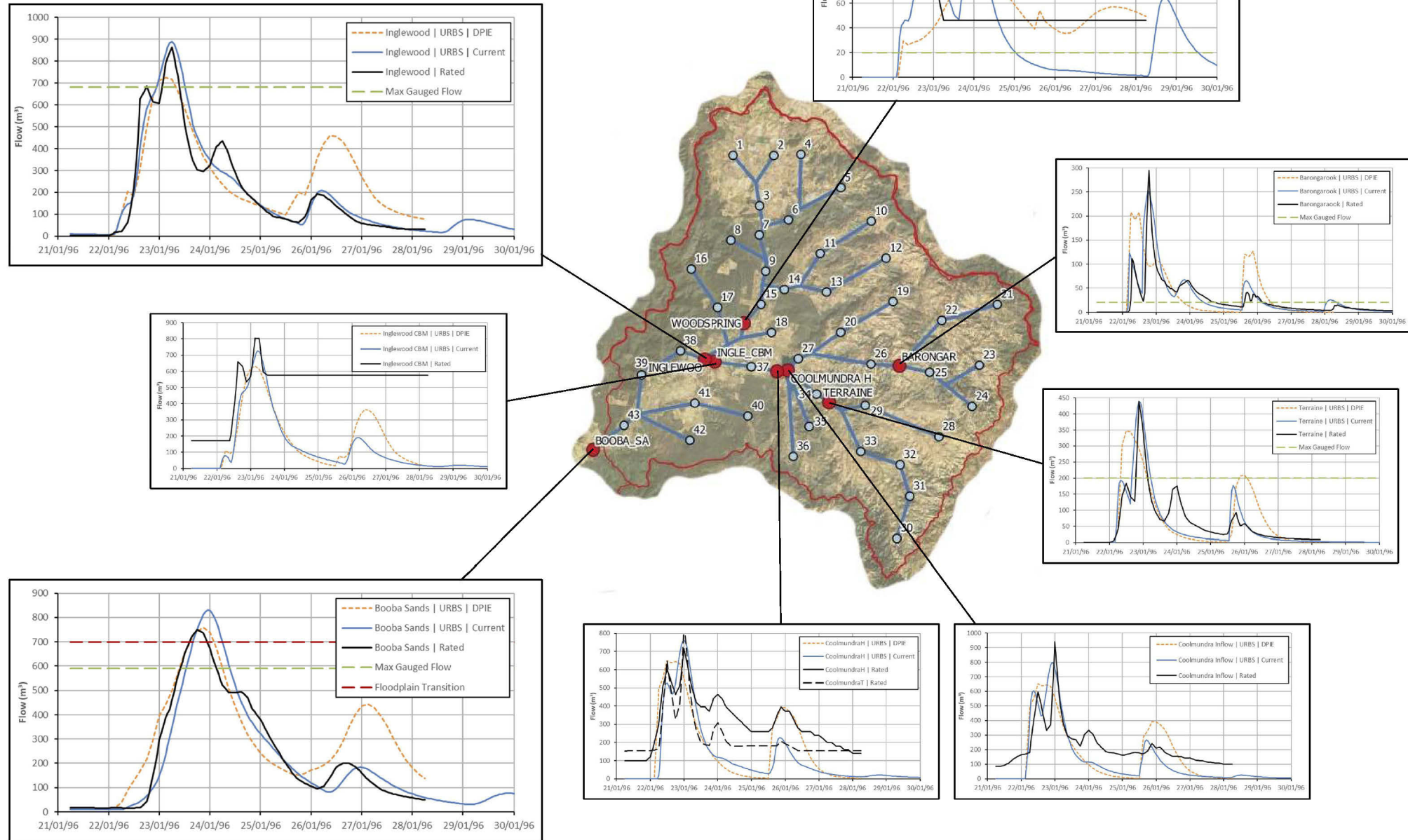


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MACINTYRE BROOK TO BOOBA SANDS – 1996

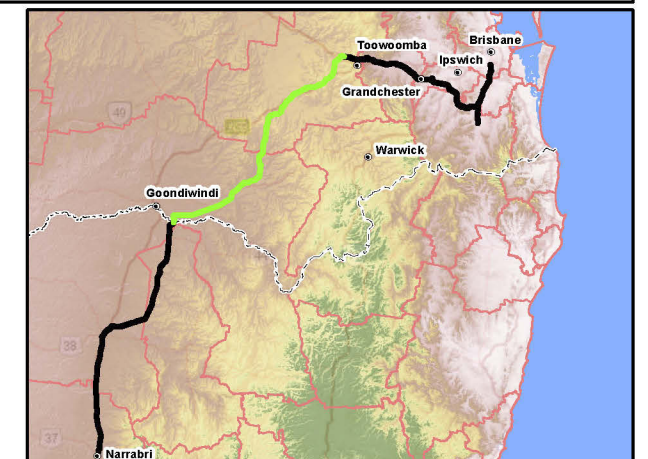
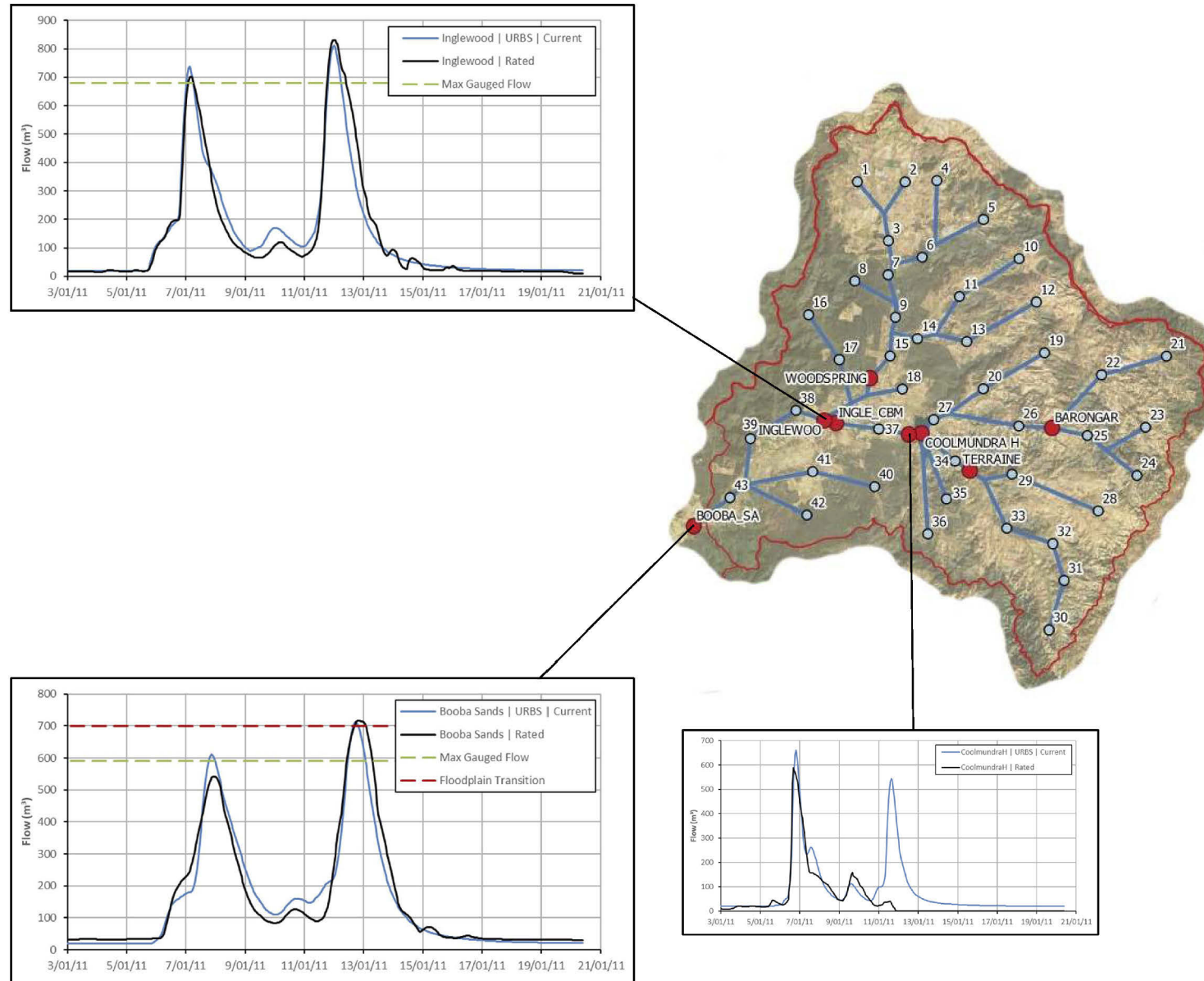


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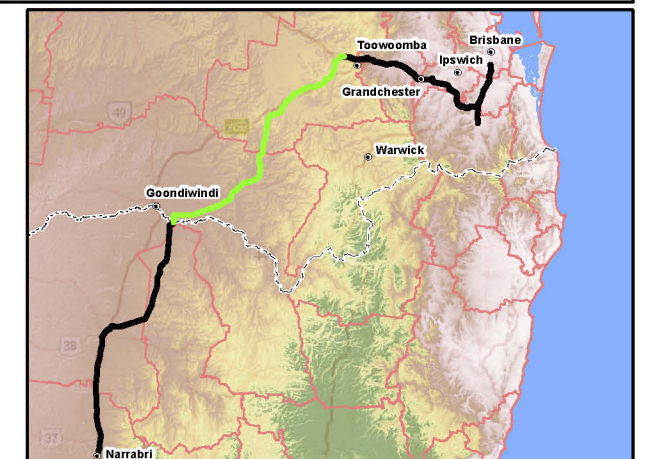
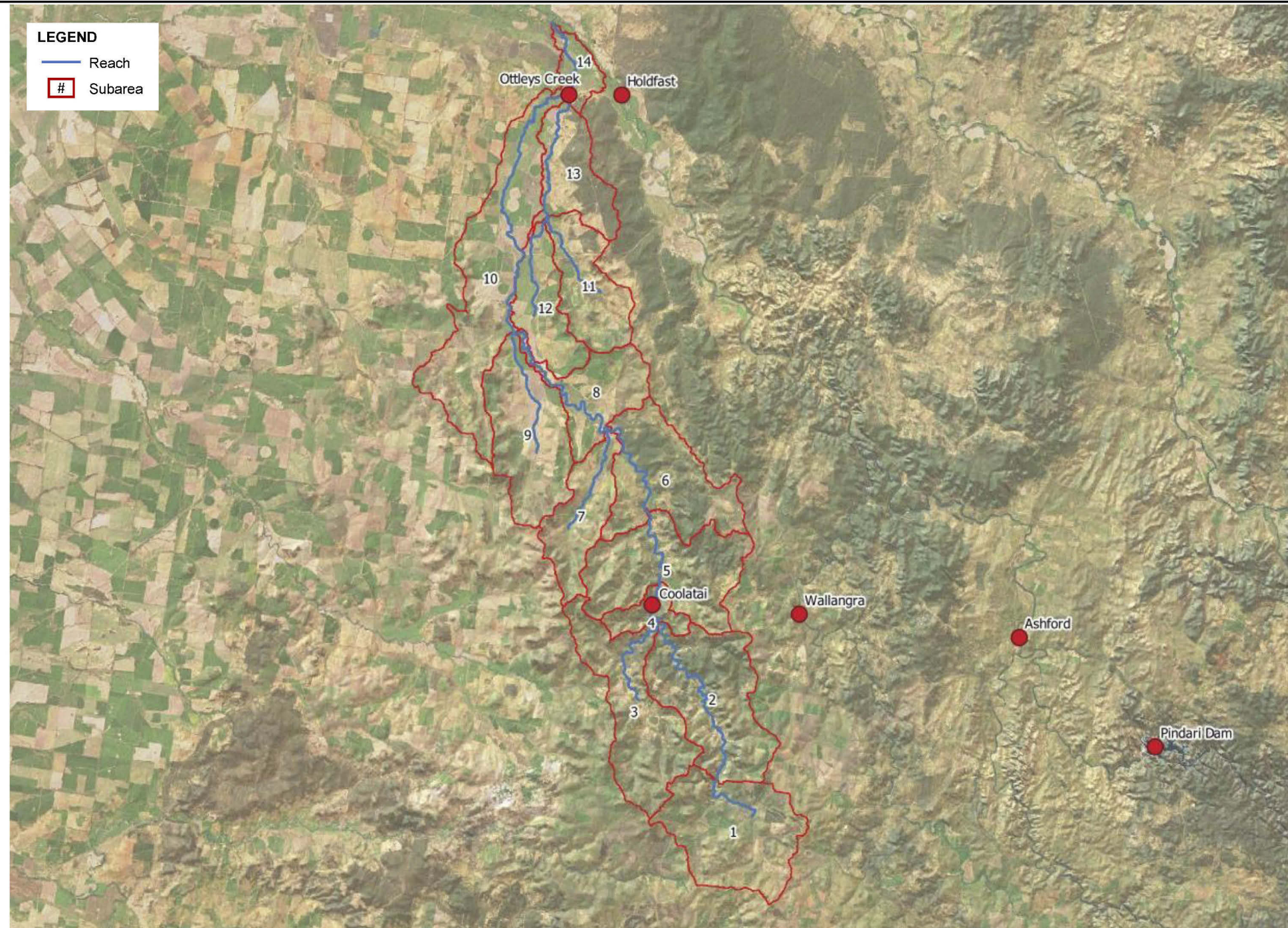
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MACINTYRE BROOK TO BOOBA SANDS – 2011



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A3 scale: 1:19,440
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Future Freight
Integrating Community, Environment and Engineering

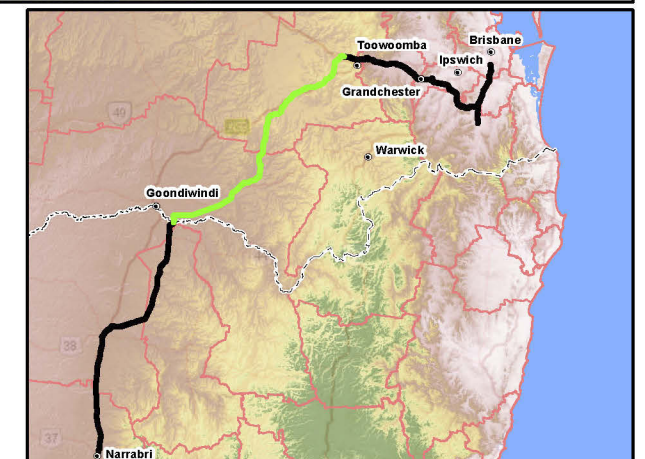
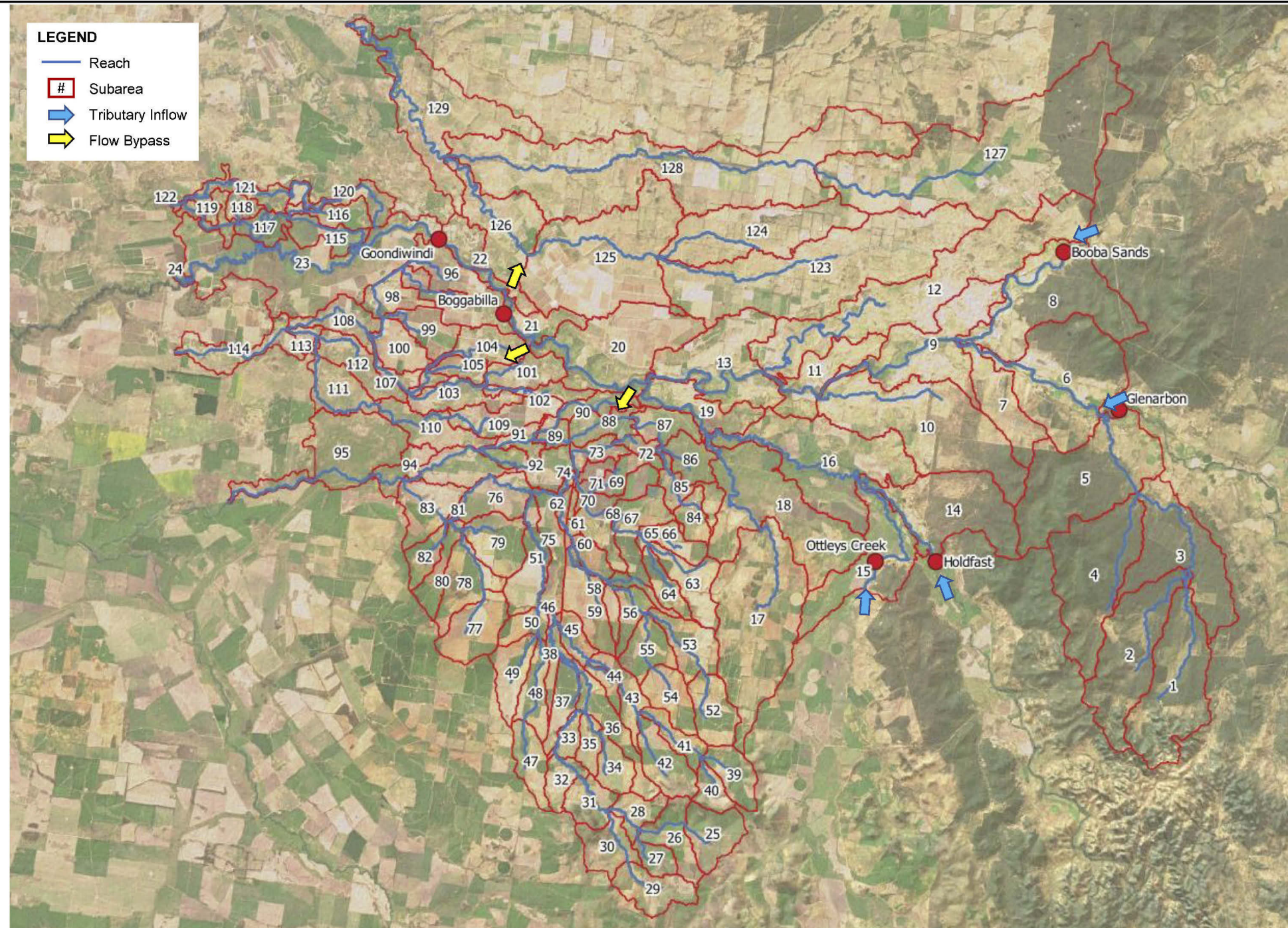
Issue date: 12/12/2022 Version: 1
Coordinate System: GDA 1994 MGA Zone 56

Border to Gowrie

Figure C8:

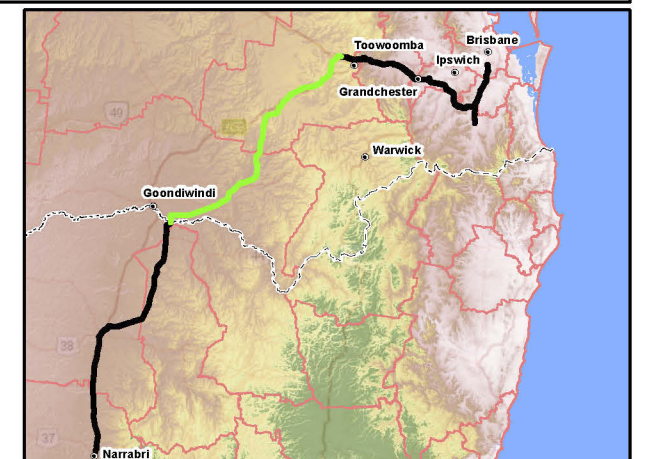
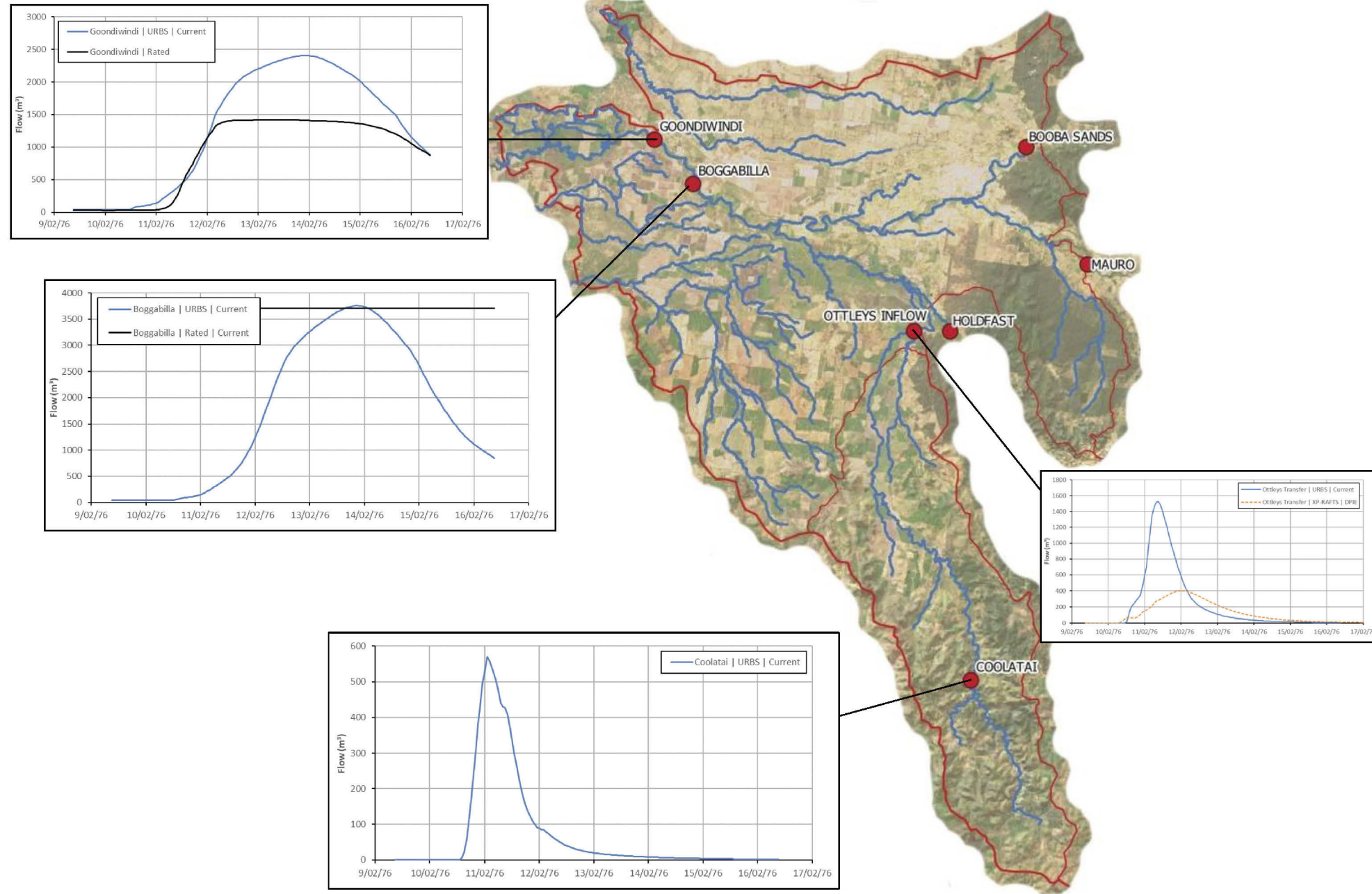
Macintyre River - Ottleys Creek URBS Model Layout

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A3 scale: 1:19,440
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OTTLEYS CREEK AND LOWER MACINTYRE TO GOONDIWINDI – 1976



Map by: JA.Z:\GIS\GIS_270_NS2B\Task\270-HY-202102151444_Hydrology_and_Flooding_PIR\270-HY-202102151444_FF\IV_FigCX_HydrologicModelCalibration_ddimages2_A4L_v0.mxd Date: 12/12/2022 14:58



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Future Freight
Integrating Community, Environment and Engineering

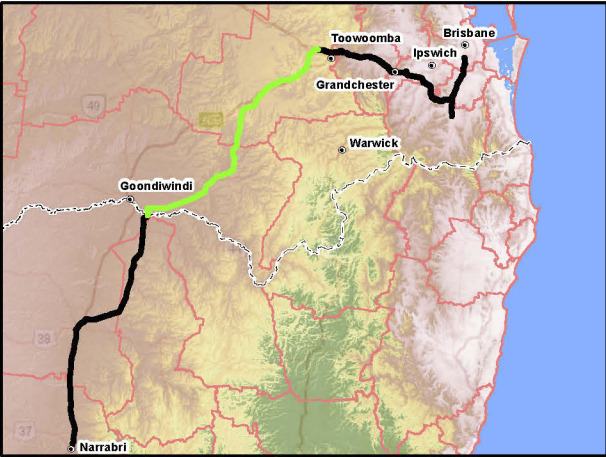
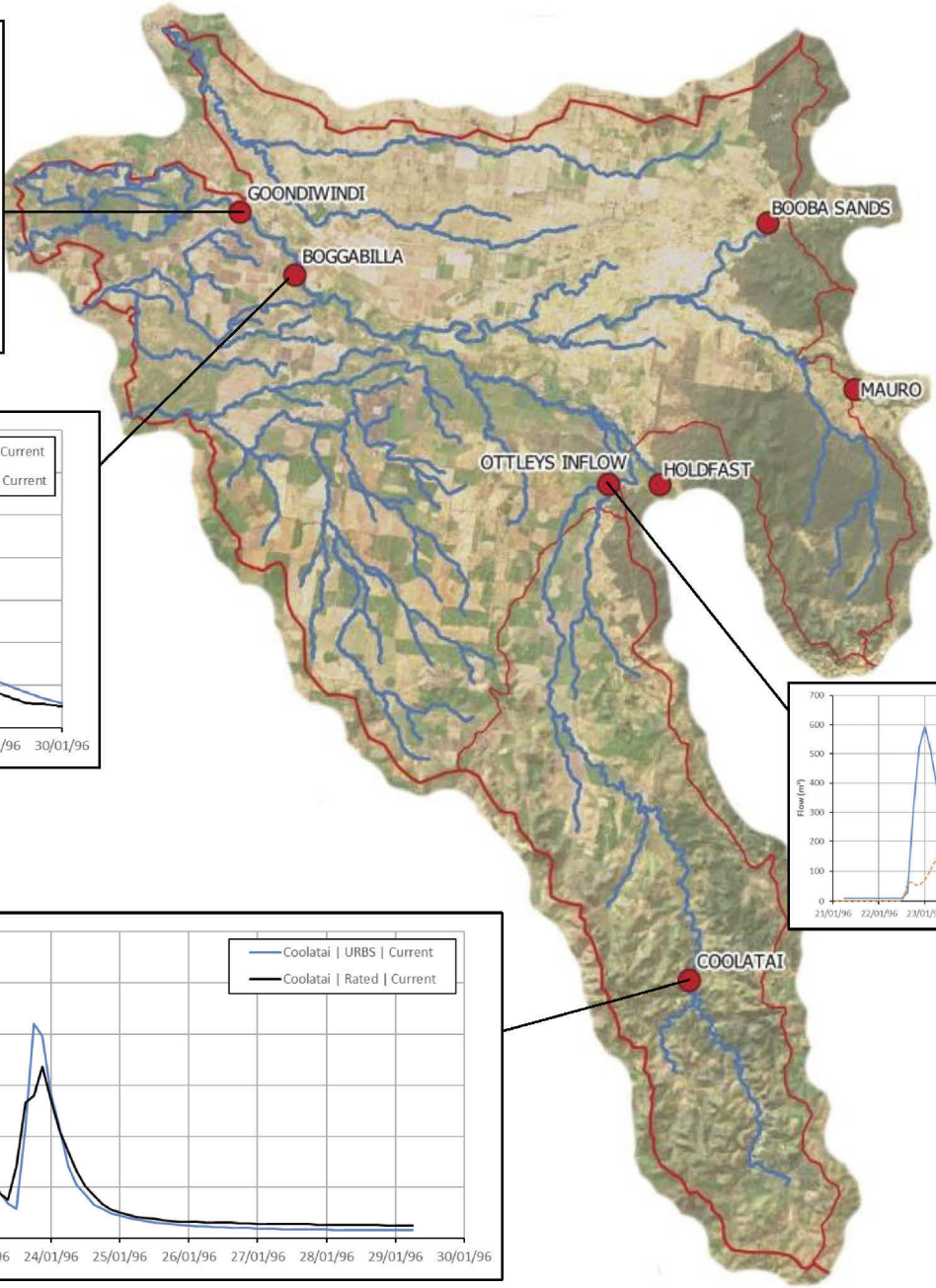
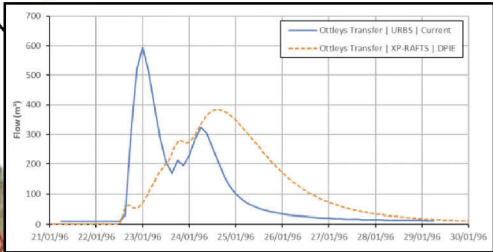
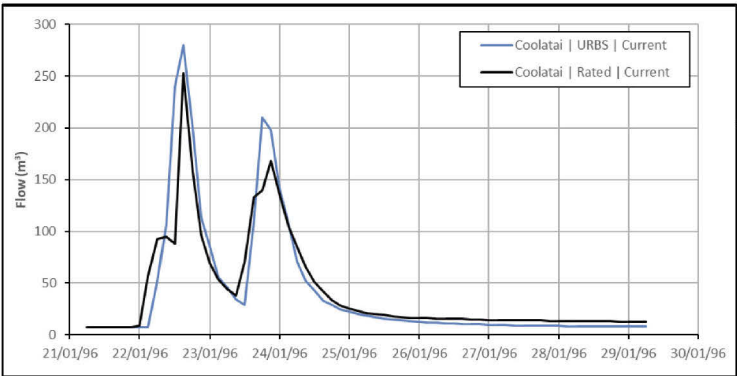
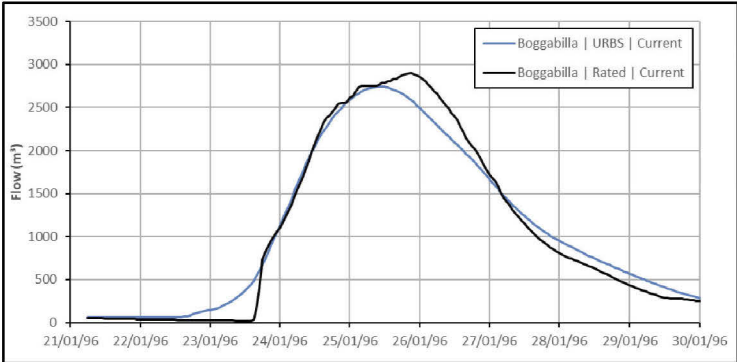
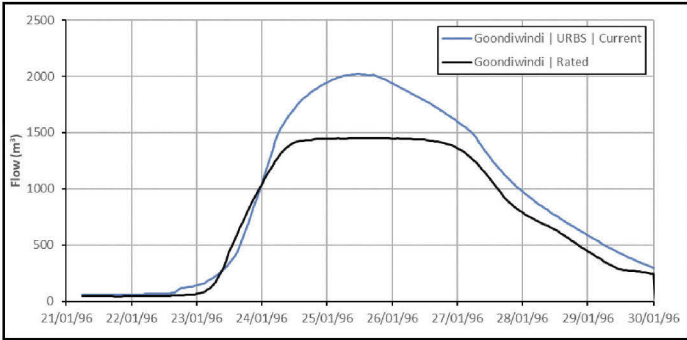
Issue date: 12/12/2022 Version: 1
Coordinate System: GDA 1994 MGA Zone 56

Border to Gowrie

Figure C10-A:

Macintyre River - Ottleys Creek and Lower Macintyre to Goondiwindi 1976 Calibration

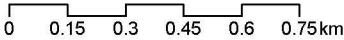
OTTLEYS CREEK AND LOWER MACINTYRE TO GOONDIWINDI – 1996



Map by: JA.Z:\GIS\GIS_270_NS2B\Task\270-IHY-202102151444_Hydrology_and_Flooding_PIR\270-IHY-202102151444_FF\IV_FigC10-HydrologicalModelCalibration_ddimages2_A4L_v0.mxd Date: 12/12/2022 14:58



A3 scale: 1:19,550



Future Freight
Integrating Community, Environment and Engineering

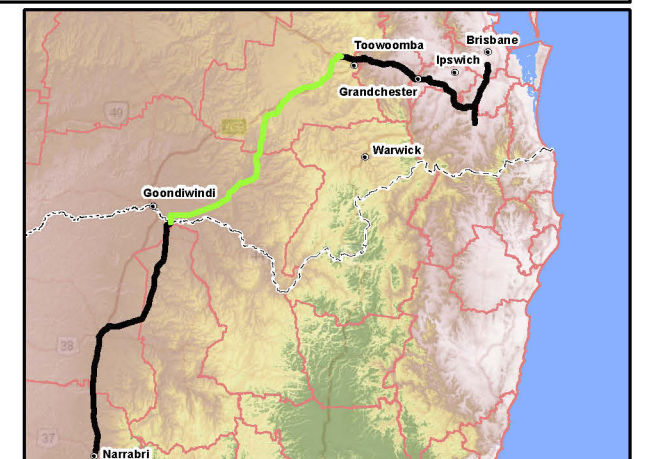
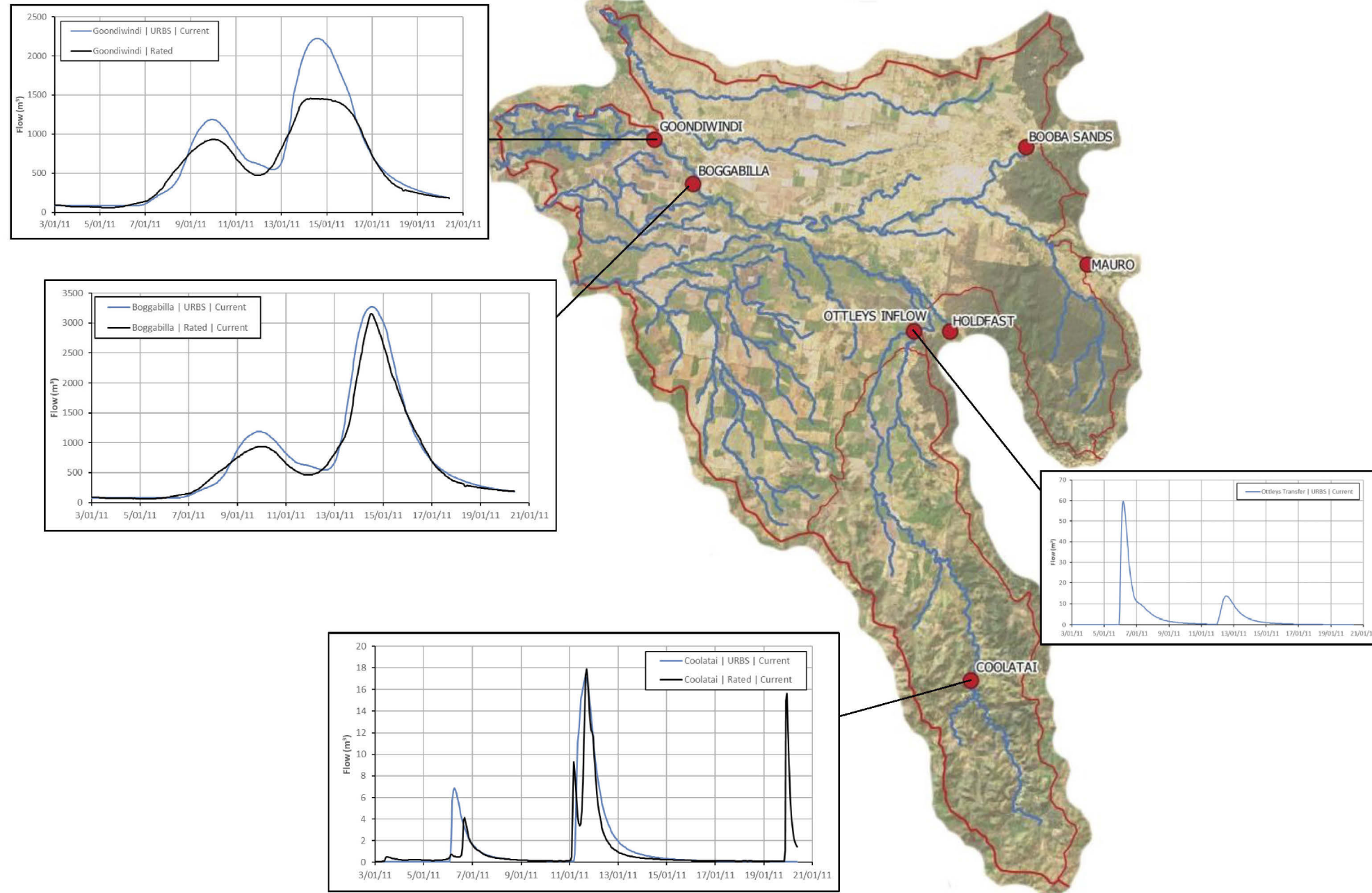
Issue date: 12/12/2022 Version: 1
Coordinate System: GDA 1994 MGA Zone 56

Border to Gowrie

Figure C10-B:

Macintyre River - Ottleys Creek and Lower Macintyre to Goondiwindi 1996 Calibration

OTTLEYS CREEK AND LOWER MACINTYRE TO GOONDIWINDI – 2011



Map by: JA.Z:\GIS\GIS_270_NS2B\Task\270-IHY-202102151444_Hydrology_and_Flooding_PIR\270-IHY-202102151444_FF\IV_FigC10_HydrologicalModelCalibration_ddimages2_A4_v0.mxd Date: 12/12/2022 14:58



A3 scale: 1:19,550
0 0.15 0.3 0.45 0.6 0.75km



Future Freight
Integrating Community, Environment and Engineering

Issue date: 12/12/2022 Version: 1
Coordinate System: GDA 1994 MGA Zone 56

Border to Gowrie

Figure C10-C:
Macintyre River - Ottleys Creek and Lower Macintyre to Goondiwindi 2011 Calibration

APPENDIX T1

Hydrology and Flooding Technical Report—Volume 1

Appendix D SGM Environmental— Erosion Threshold Velocity technical note

BORDER TO GOWRIE REVISED DRAFT ENVIRONMENTAL IMPACT STATEMENT



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14 November 2022

Ron Dela Pena
Principal Environmental Consultant
AECOM Australia
8/540 Wickham St
Fortitude Valley Qld 4006

Re: | 22B004 — Inland Rail Border to Gowrie (B2G) Erosion Threshold Velocities

Dear Ron

I.0 Introduction

SGM environmental Pty Limited (SGME) was engaged by the Australian Rail Track Corporation (ARTC) to assess the erodibility of soil management units (SMUs) for the Inland Rail Project: Border to Gowrie (the Project) and assign erosion threshold velocities (ETVs) to each SMU. These ETVs will assist ARTC and any subcontractors in the selection of culvert and channel linings along the Project corridor.

I.1 Erodibility

Erodibility describes a soil's response to rainfall and runoff erosivity (the potential for rainfall / runoff to detach and erode a soil) and is influenced by inherent soil properties such as:

- texture;
- structure;
- sodicity;
- salinity;
- organic matter;
- permeability / drainage;
- soil depth;
- coarse fragments;
- hardsetting surface;
- waterlogging; and
- water repellence.



2.0 Method

2.1 Soil review

A desktop review of the *Soil Assessment and Management Plan* (Aurecon 2022) was done to compile typical soil properties for each of the Project's SMUs.

2.1.1 Further testing

The erodibility assessment required further laboratory analysis to define the Atterberg limits (ie liquid limit / plastic index) of representative surface samples.

2.2 Erosion threshold velocities

There is a lack of research in assessing the erodibility of Queensland soils. Due to the complexity in accurately weighting various soil properties to develop erodibility assessment criteria that is applicable to the Project, the following method has been adopted:

1. Using laboratory information from the soil assessment and the unified soil classification (USC) system to classify each SMU and / or soil type as per the USC soil groups.
 - a. Using the soil group classification criteria from Table I (*ASTM Standard D2487 – 06*).
 - b. If a SMU classification is uncertain (ie cannot be classified into a single USC soil group), either the more erodible soil group will be chosen (worst case) or the SMU will be divided into subgroups.
2. Assign erodibility class based on USC soil group and Table A23 of the *Best Practice in Erosion and Sediment Control* (BPESC) guidelines (IECA 2008).
 - a. Verify erodibility class by comparing USC soil groups to K-factors from Table F3 of the BPESC guidelines.
3. Create a shapefile of the SMUs that contains erodibility class and maximum permissible velocities.

2.2.1 USC soil group classification

The USC system was developed to describe the engineering properties of a soil based on the size of the particles, the amounts of the various sizes and the characteristics of the very fine grains. It can be carried out by either field examination or laboratory testing.

Classification criteria for the USC soil groups is in Table I.

2.2.1.1 Coefficients of gradation

The uniformity coefficient (C_U) is defined as the ratio of D_{60} (diameter where 60% of particles are finer) to D_{10} (EQN1). A value of C_U greater than 4 to 6 classifies the soil as well graded. When C_U is less than 4, it is classified as poorly graded or uniformly graded soil.

Coefficient of curvature (C_C) is a measure of gradation of particles and is given by EQN2. For a soil to be well graded, the value of C_C must range between 1 and 3.

$$C_U = D_{60} / D_{10} \quad (\text{EQN1})$$

$$C_C = (D_{30})^2 / (D_{10} \times D_{60}) \quad (\text{EQN2})$$

Where: D_{60} = diameter where 60% of particles are finer; D_{30} = diameter where 30% of particles are finer; and D_{10} = diameter where 10% of particles are finer.



Table I USC soil group classification criteria — ASTM Standard D2487 – 06

Classification criteria				Soil Group	
				Group symbol	Group name
Coarse grained soils (> 50% retained by No. 200 sieve)	Gravels (> 50% retained by No. 4 sieve)	Clean gravels (< 5% fines)	$C_u \geq 4$; and $I \leq C_c \leq 3$	GW	Well-graded gravel
			$C_u < 4$; and / or $I > C_c > 3$	GP	Poorly graded gravel
		Gravel with fines (> 12% fines)	Fines classify as ML or MH	GM	Silty gravel
			Fines classify as CL or CH	GC	Clayey gravel
	Sands (> 50% passes No. 4 sieve)	Clean sands (< 5% fines)	$C_u \geq 6$; and $I \leq C_c \leq 3$	SW	Well graded sand
			$C_u < 6$; and / or $I > C_c > 3$	SP	Poorly graded sand
		Sands with fines (> 12% fines)	Fines classify as ML or MH	SM	Silty sand
			Fines classify as CL or CH	SC	Clayey sand
		Inorganic (LL (oven-dried) ≥ 0.75 of LL (not dried))	$PI > 7$; and plots on or above "A" line	CL	Lean clay
			$PI < 4$; or plots below "A" line	ML	Silt
			$PI \geq 4$; and plots on or above "A" line	OL	Organic clay
			$PI < 4$; or plots below "A" line	OH	Organic silt
Fine grained soils (> 50% passes No. 200 sieve)	Silt and clays (LL < 50)	Inorganic (LL (oven-dried) ≥ 0.75 of LL (not dried))	PI plots on or above "A" line	CH	Fat clay
			PI plots below "A" line	MH	Elastic silt
		Organic (LL (oven-dried) < 0.75 of LL (not dried))	PI plots on or above "A" line	OH	Organic clay
			PI plots below "A" line	OH	Organic silt
	Highly organic soils	Primarily organic matter, dark in colour and organic odour		PT	Peat

1. Based on the material passing the 3-in. (75-mm) sieve.
2. If soil contains $\geq 15\%$ sand, add "with sand" to group name.
3. Gravels with 5 to 12 % fines require dual symbols: GW-GM = well-graded gravel with silt; GW-GC = well-graded gravel with clay; GP-GM = poorly graded gravel with silt; and GP-GC = poorly graded gravel with clay.
4. If fines classify as CL-ML, use dual symbol GC-GM, or SC-SM.
5. If fines are organic, add "with organic fines" to group name.
6. If soil contains $\geq 15\%$ gravel, add "with gravel" to group name.
7. Sands with 5 to 12 % fines require dual symbols: SW-SM well-graded sand with silt; SW-SC well-graded sand with clay; SP-SM poorly graded sand with silt; and SP-SC poorly graded sand with clay.
8. If Atterberg limits plot in hatched area, soil is a CL-ML, silty clay.
9. If soil contains 15 to 29 % plus No. 200, add "with sand" or "with gravel," whichever is predominant.
10. If soil contains $\geq 30\%$ plus No. 200, predominantly sand, add "sand" to group name.
11. If soil contains $\geq 30\%$ plus No. 200, predominantly gravel, add "gravelly" to group name.



2.2.2 Best Practice Erosion and Sediment Control

The BPESC guidelines (IECA 2008) provide industry standards for the design, construction and monitoring of drainage, erosion and sediment controls.

Texture and erodibility classes (based on USC soil groups) have been given allowable flow velocities for various channel linings. These velocities are from Table A23 of the BPESC guidelines and are in Table 2.

Table 2 Allowable flow velocity for various soil descriptions

Type	Description	Allowable velocity (metres per second (m/s))
Open earth (unlined channels)	Extremely erodible soils	0.30
	Sandy soils	0.45
	Highly erodible soils	0.40-0.50
	Sandy loam soils	0.50
	Moderately erodible soils	0.60
	Silty loam soils	0.60
	Erosion-resistant soils	0.70
	Firm loam soils	0.70
	Stiff clay very colloidal soils	1.10

1. Dispersive clays are extremely erodible and must be treated (ie gypsum) or covered with a minimum of 100 mm of stable soil.
2. Highly erodible soils: lithosols, alluvials, podzols, siliceous sands, some black earths and soil groups ML and CL.
3. Moderately erodible soils: red earths, podzolics, some black earths, grey of brown clays and soil groups SW, SP, SM and SC.
4. Erosion-resistant soils: xanthozem, euchrozem, krasnozems some red earths and soil groups GW GP, GM, GC, MH and CH.

Allowable velocities have also been assigned based on channel gradient and the percentage of stable vegetation cover in Table A28 of the BPESC guidelines. A summary of these velocities is in Table 3.

Table 3 Allowable flow velocity for various vegetation covers

Channel gradient (%)	Percent of stable vegetation cover (%) ¹			
	0 ²	50	70	100
<i>Erosion resistant soils</i>				
1	0.7	1.6	2.1	2.8
2	0.6	1.4	1.8	2.5
3	0.5	1.3	1.7	2.4
4	-	1.3	1.6	2.3
5	-	1.2	1.6	2.2
<i>Easily eroded soils</i>				



Channel gradient (%)	Percent of stable vegetation cover (%) ¹			
1	0.5	1.2	1.5	2.1
2	0.5	1.1	1.4	1.9
3	0.4	1.0	1.3	1.8
4	-	1.0	1.2	1.7
5	-	0.9	1.2	1.6

1. Designers should assess the percentage of stable vegetation cover likely to persist under design flow conditions.

2. Applies to surface that are consolidated, but not cultivated.

2.2.3 Validation

The Revised Universal Soil Loss Equation (RUSLE) model has been used extensively to estimate soil loss across a broad range of industries and land uses. The erodibility factor (K-factor) is one of the five inputs into the RUSLE model and arguably the most impactful. K-factors are generally estimated based on the following attributes (Wischmeier et al 1971):

- particle size distribution (percent clay, silt and sand);
- organic matter content;
- soil structure; and
- permeability.

2.2.3.1 Best Practice Erosion and Sediment Control

The BPESC guidelines (IECA 2008) provide a range of typical K-factors for each of the USC soil groups. This information (Table 4) was used along with available laboratory data to validate the erodibility classes that were assigned using the criteria in Table 1.

Table 4 Nominal K-factors based on USC (Table F3 — IECA 2008)

Description	Group	K-factor (typical range (default ¹))
Silty gravels, poorly graded gravel-sand-silt	GM	0.000-0.060 (0.053)
Clayey gravels, poorly graded gravel-sand-clay	GC	0.000-0.050 (0.042)
Well graded sands, gravelly sands, little fines	SW	0.000-0.040 (0.036)
Poorly graded sands, gravelly sands, few fines	SP	0.000-0.030 (0.027)
Silty sands, poorly graded sand-clay mixtures	SM	0.010-0.050 (0.043)
Clayey sands, poorly graded sand-clay mixtures	SC	0.020-0.050 (0.044)



Description	Group	K-factor (typical range (default'))
Inorganic silts, clayey sands with slight plasticity	ML	0.030-0.070 (0.062)
Inorganic clays of low to medium plasticity	CL	0.020-0.060 (0.058)
Organic silts and organic silt-clay of low plasticity	OL	0.010-0.040 (0.033)
Inorganic silts, fine sands or silty soils, elastic silts	MH	0.020-0.070 (0.066)
Inorganic clays of high plasticity, elastic soils	CH	0.000-0.050 (0.047)

1. Default values were developed from a statistical analysis of NSW soil data (Landcom 2004) and may be used in absence of local data.

2.2.3.2 K-factor mapping

Erodibility classes were further validated using K-factor mapping from the work of Brough et al (2004). This mapping was used to assess the spatial distribution of soil erodibility values derived from historical soil sampling programs and compare the values to the range of K-factors that are typical for each of the SMUs (Table 4).

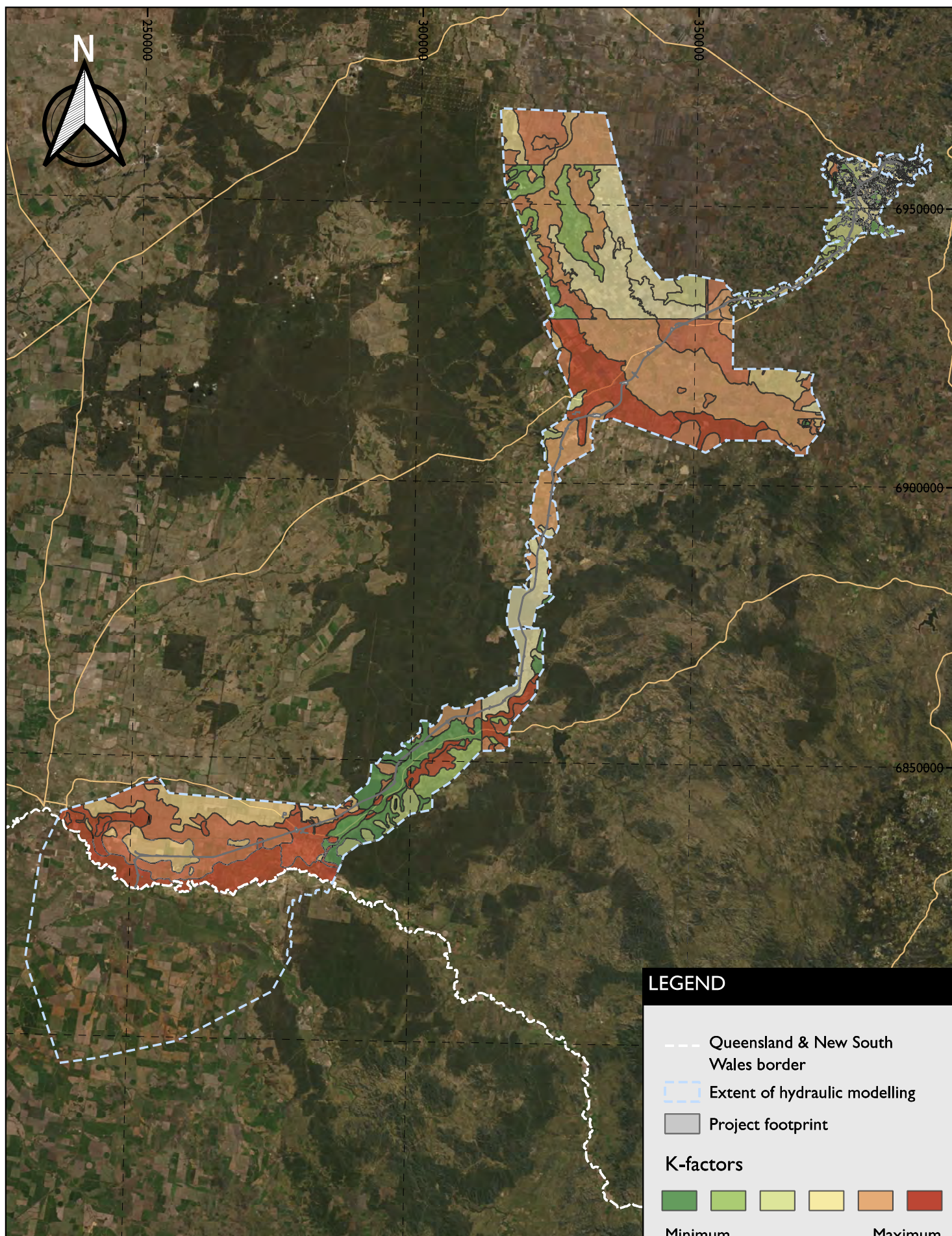
The Project crosses the Condamine Floodplain, which has undergone detailed hydraulic modelling. For interpretive purposes, K-factor mapping has been provided (Figure 1) for the spatial extent of the hydraulic modelling, which includes areas covered by the Condamine floodplain that are within ~ 50 km of the Project footprint.

This mapping will provide a useful tool during initial construction works by assisting the identification of erodible soils and allowing proper management measures to be used.

3.0 Soil review

A summary of the SMUs mapped along the Project is in Table 5.





LEGEND

- Queensland & New South Wales border
- Extent of hydraulic modelling
- Project footprint

K-factors



Scale 1 : 1 000 000

0 10 20 30 km



Project		Client	
Erosion Threshold Velocities		Australian Rail Track Corporation	
Figure	Tide	Design:	SGM Environmental Pty Ltd 24.05.2022
I	K-Factor Mapping	Scale:	1 : 1,000,000
		CRS:	GDA94 / MGA zone 56



Table 5 Summary of Project SMUs

SMU	Mapped soil units (MSU)	Australian soil classification (ASC) ¹	Key soil features
A	1.1.1.1a	Grey / Black Vertosol	- Clayey soils found: - adjacent to major watercourses (subject to flooding); - drainage depressions, slopes, ridges and crests (may be subject to occasional flooding); and - elevated and gently undulating plains (subject to flooding). - Topsoil: - light medium to heavy clays; - moderately acid to strongly alkaline; - generally low to moderate salinity; - generally non-sodic but can be sodic; and - eutrophic with moderate to high fertility. - Subsoil: - neutral to very strongly alkaline; - very low to extreme salinity; - non-sodic to strongly sodic (at depth) with deficient to low Ca:Mg; - low to high CEC; and - EAT classes 1, 2, 3a, 3b, 4, 5, 6, 7 and 8.
	1.2.1.1a	Black / Brown Vertosol	
	1.3.1.1a	Black Vertosol	
	2.1.1.1a	-	
	2.2.1.1a	Epicalcareous Self-mulching Black / Brown Vertosol	
	2.3.1.1a	Epicalcareous Crusty Black Vertosol	
	3.1.1.1a	Haplic Self-mulching Black Vertosol	
	3.2.1.1a	Self-mulching Black / Brown Vertosol	
	3.3.1.1a	Self-mulching Black Vertosol	
	3.4.1.1a	-	
	3.6.1.1a	Self-mulching Black Vertosol	
	4.1.1.1a	Episodic Crusty Black / Grey / Brown Vertosol	
	5.1.1.1a	-	
	5.2.1.1a	Epicalcareous Endohypersodic Crusty Black Vertosol	
	5.3.1.1a	-	
B	1.1.1.1b	Grey / Black Vertosol	- Clayey soils found: - adjacent to major watercourses (subject to flooding); and - may have moderate to deep melonhole gilgai with strong shrink / swell properties. - Topsoil: - light to medium heavy clays; - moderately acid (depression) to neutral (mound); - very low to moderate salinity; - non-sodic to sodic with deficient to low Ca:Mg; and - eutrophic with moderate fertility. - Subsoil: - moderately to very strongly alkaline; - moderately to extremely saline; - strongly sodic with deficient to low Ca:Mg; - moderate to very high CEC; and - EAT classes 1, 2, 3b and 6.
	1.2.1.1b	-	
	4.1.1.1b	-	

SMU	Mapped soil units (MSU)	Australian soil classification (ASC) ¹	Key soil features
C	1.1.1.2	Sodic Brown / Black / Red Dermosol	- Clayey soils found: - adjacent to major watercourses (subject to flooding). - Topsoil: - clay loam to light medium clay; - slightly acid to mildly alkaline; - very low to low salinity; - non-sodic to strongly sodic with deficient to low Ca:Mg; and - eutrophic with moderate fertility. - Subsoil: - moderately to very strongly alkaline; - moderately to very highly saline; - sodic to strongly sodic with deficient to low Ca:Mg; - very low to high CEC; and - EAT classes 1, 2, 3a, 3b, 4, 5 and 8.
	1.2.1.2	Sodic Brown / Black Dermosol	
	4.1.1.2	Acidic Sodic Eutrophic Brown Dermosol	
D	3.2.1.2	Manganic Eutrophic Red Dermosol	- Clayey soils found: - on plains (subject to flooding); and - gently undulating plains and rises (unlikely to flood). - Topsoil: - clay loam to light clay; - moderately acid to moderately alkaline; - very low to low salinity; - non-sodic to sodic with low Ca:Mg; and - eutrophic with moderate fertility. - Subsoil: - slightly acid to strongly alkaline; - very low to very high salinity; - non-sodic to strongly sodic with deficient to low Ca:Mg; - very low to high CEC; and - EAT classes 1, 3b, 4, 5, 7 and 8.
	3.3.1.2	Haplic Eutrophic Red Dermosol	
	3.6.1.2	Eutrophic Black Dermosol	
	5.1.1.2	Mottled Sodic Hypocalcic Black Dermosol	
	5.2.1.2	Mottled Sodic Eutrophic Yellow Dermosol	
	5.3.1.2	-	

SMU	Mapped soil units (MSU)	Australian soil classification (ASC) ¹	Key soil features
E	3.1.1.3	Eutrophic Red / Brown Ferrosol	- Clayey soils found: - on level and very gently inclined areas adjacent to drainage depressions (may be subject to flooding); and - on slopes, ridges and crests (not subject to flooding). - Topsoil: - light to medium clay; - strongly acid to slightly alkaline; - very low to low salinity; - non-sodic with low Ca:Mg; and - eutrophic with moderate to high fertility. - Subsoil: - neutral to moderately alkaline; - very low to moderate salinity; - non-sodic to sodic with deficient to low Ca:Mg; - moderate to very high CEC; and - EAT classes 3a, 3b, 5, 7 and 8.
	3.2.1.3	-	
	3.3.1.3	Eutrophic Black / Brown Ferrosol	
	3.4.1.3	-	
	3.6.1.3	Eutrophic Red / Brown Ferrosol	
F	1.1.2.2	-	- Deep to very deep sands, loams and clay loams: - found adjacent to major watercourses (may be subject to flooding); - have very low to low fertility; - high infiltration; and - EAT classes 2 (lower subsoil) and 5 (majority of profile). - Very shallow to shallow sands, loams and clay loams: - found on moderately steep to steep slopes (not subject to flooding); - have moderate fertility; - low to moderate infiltration (steep slopes); and - EAT classes 3b (topsoil) and 8 (subsoil). - Both soils: - moderately to strongly acid; - clay fraction of subsoils (typically a very small fraction) is dispersive; and - low dust potential (low fraction of fines).
	1.2.2.2	Basic Regolithic Brown Tenosol	
	5.4.2.2	-	
	5.5.2.2	-	
	5.6.2.2	Acidic Lithic Brown Tenosol	

SMU	Mapped soil units (MSU)	Australian soil classification (ASC) ¹	Key soil features
G	5.1.3.1b 5.2.3.1b 5.3.3.1b 5.4.3.1b 5.6.3.1a 5.6.3.1b	- Mottled Natric Grey Kurosol Brown / Grey / Yellow Kurosol - Mottled Natric Brown Kurosol -	- Texture contrast soils found: - on elevated level plains, and gently undulating plains and rises (unlikely to flood). - Topsoil: - sandy to loam; - moderately to strongly acid; - very low salinity; - sodic to strongly sodic with deficient to low Ca:Mg; and - mesotrophic to eutrophic with moderate fertility. - Subsoil: - sandy to medium clay; - strongly acid; - very low to low salinity; - sodic to strongly sodic with deficient Ca:Mg; - very low to low CEC; and - EAT classes 3a, 3b, 5, 7 and 8.
H	1.1.3.2a 1.1.3.2b 1.2.3.2b	- Mottled Yellow / Brown / Grey Sodosol Eutrophic Mottled Mesonatric Grey Sodosol	- Texture contrast soils found: - on alluvial plains and terraces adjacent to watercourses (subject to flooding). - Topsoil: - sand to loam; - very strongly acid to neutral; - very low salinity; - non-sodic to strongly sodic with low to balanced Ca:Mg; and - eutrophic with moderate fertility. - Subsoil: - light to medium, sandy clay; - slightly acid to very strongly alkaline; - very low to high salinity; - sodic to strongly sodic with deficient to low Ca:Mg; - low to moderate CEC; and - EAT classes 1, 2, 3b, 4 and 8.

SMU	Mapped soil units (MSU)	Australian soil classification (ASC) ¹	Key soil features
I	4.2.3.2b 5.1.3.2a 5.1.3.2b 5.2.3.2a 5.2.3.2b 5.3.3.2b 5.4.3.2b 5.6.3.2b	Hypocalcric Mottled Hypernatric Brown Sodosol Hypocalcric Mottled Hypernatric Brown Sodosol Mottled Hypernatric Brown Sodosol Eutrophic Mottled Subnatric Brown Sodosol Mottled Grey / Brown / Yellow Sodosol Eutrophic Mottled Hypernatric Grey Sodosol - -	• Texture contrast soils found: - on elevated plains, and gently undulating plains and rises (not subject to flooding). • Topsoil: - sand to loam; - very strongly acid to moderately alkaline; - very low salinity; - non-sodic to strongly sodic with low Ca:Mg; and - eutrophic with low fertility. • Subsoil: - loam to medium clays; - moderately acid to very strongly alkaline; - very low to very high salinity; - strongly sodic with deficient to low Ca:Mg; and - EAT classes 1, 2, 3a, 3b, 5, 6, 7, and 8.
J	5.1.3.3b 5.2.3.3b 5.3.3.3b	- Mottled Sodic Hypocalcic Red Chromosol -	• Texture contrast soils found: - on level to gently inclined slopes of plains and gently undulating rises (not subject to flooding). • Topsoil: - sand; - strongly acid; - very low ECEC with very low to low fertility; and - low Ca:Mg. • Subsoil: - sandy to medium clay; - upper subsoil: > slightly to moderately alkaline; > low salinity; > moderate ECEC; and > non-sodic with deficient Ca:Mg. - lower subsoil: > strongly alkaline; > moderate salinity; > moderate ECE; and > sodic with deficient Ca:Mg.

1. ASCs have been adjusted so that only descriptors that are shared across all (observed) soil profiles in the MSU are shown (with exception to colour).



4.0 Results

4.1 Gradation of soil

An analysis of the gradation of the coarse fraction of the SMUs is in Table 6.

Table 6 Particle size analysis — coarse fraction

SMU	D ₆₀ ¹	D ₃₀ ¹	D ₁₀ ¹	C _u	C _c	Comments
A	0.151	0.003	4.90E-06	30,917	10	Well graded (fine-grained)
B	0.022	1.41E-04	4.59E-08	487,798	19	Well graded (fine-grained)
C	0.172	0.012	1.75E-04	981	5	Poorly graded (coarse-grained)
D	0.262	0.053	0.004	62	3	Well graded (coarse-grained)
E	0.584	0.035	4.01E-04	1,456	5	Poorly graded (coarse-grained)
F	1.203	0.266	0.024	50	2	Well graded (coarse-grained)
G	1.287	0.133	0.004	350	4	Poorly graded (coarse-grained)
H	0.565	0.107	0.008	73	3	Well graded (coarse-grained)
I	0.570	0.086	0.004	131	3	Well graded (coarse-grained)
J	0.405	0.071	0.005	89	3	Well graded (coarse-grained)

1. Diameters are in millimetres (mm).

4.2 Atterberg limits

Results from the additional laboratory testing of Atterberg limits are in Table 7.

Table 7 Atterberg limits

SMU	Liquid Limit	Plastic Limit	Plasticity Index	Comments
A	41	12	29	Clay (highly plastic)
B	41	19	22	Clay (highly plastic)



SMU	Liquid Limit	Plastic Limit	Plasticity Index	Comments
C	24	12	12	Silty clay (medium plastic)
D	50	48	2	Gravelly clayey sand (slightly plastic)
E	39	20	19	Gravelly clay (highly plastic)
F	N/A	N/P	N/P	Sand (non-plastic)
G	N/A	N/P	N/P	Sand (non-plastic)
H	N/A	N/P	N/P	Sand (non-plastic)
I	18	16	2	Clayey sand (slightly plastic)
J	17	15	2	Sandy clay (slightly plastic)

4.3 Erosion threshold velocities

Using the classification criteria for USC soil groups (Table 1) and the BPESC guidelines, erodibility classes and ETVs have been assigned to each SMU. The USC soil groups, erodibility classes and ETVs are in Table 8.

It should be noted, all SMUs (with exception to D, E and F) contain soils that are classified as sodic (ie Sodic, Endohypersodic, Natric, Subnatric and Hypernatric). Since sodic soils are generally also dispersive, it is advised that Project staff be familiar with these SMUs in order to observe any variances from the provided soil maps or obvious signs of dispersive soils (see Section 5.1.1).



Table 8 Erosion threshold velocities

SMU	USC soil group	Erodibility class ^{1,2}	Bare soil ETV ³ (m/s)	Allowable flow velocity — 50% cover ⁴	Allowable flow velocity — 70% cover ⁴	Allowable flow velocity — 100% cover ⁴
A	CL – sandy lean clay	Highly erodible	0.40-0.50	0.9-1.2	1.2-1.5	1.6-2.1
B	CL – lean clay with sand	Highly erodible	0.40-0.50	0.9-1.2	1.2-1.5	1.6-2.1
C	SC – clayey sand	Moderately erodible	0.60	0.9-1.2	1.2-1.5	1.6-2.1
D	SM – silty sand	Moderately erodible	0.60	0.9-1.2	1.2-1.5	1.6-2.1
E	SC – clayey sand	Moderately erodible	0.60	0.9-1.2	1.2-1.5	1.6-2.1
F	SC – clayey sand	Moderately erodible	0.60	0.9-1.2	1.2-1.5	1.6-2.1
G	SC – clayey sand	Moderately erodible	0.60	0.9-1.2	1.2-1.5	1.6-2.1
H	SC – clayey sand	Moderately erodible	0.60	0.9-1.2	1.2-1.5	1.6-2.1
I	SM – silty sand	Moderately erodible	0.60	0.9-1.2	1.2-1.5	1.6-2.1
J	SM – silty sand	Moderately erodible	0.60	0.9-1.2	1.2-1.5	1.6-2.1

1. Where a SMU classification was uncertain (ie more than one soil group), the more erodible (conservative) class was chosen.

2. Where a SMU contained shallow (<30 cm) or waterlogged soils, the erodibility class was conservatively increased.

3. Based on Table A23 from the BPESC guidelines (IECA 2008).

4. Allowable flow velocity for each vegetation cover has been estimated from Table 3 for slopes up to 5%. **Estimate is not suitable for design and construction purposes.**



4.3.1 Validation

The soil groups and ETVs assigned in Table 8 have been validated through spatial comparison to RUSLE K-factor mapping derived from historical soil sampling programs (Brough et al 2004). Generally, the K-factor mapping agreed with descriptions in the soil assessment (Aurecon 2022).

The following observations were made and should be considered during construction:

- High K-factor (sodic soil) near Inglewood:
 - hard-setting fine sands to clay loams over yellowish-brown or brown clays (solodics, solodized solonetz);
 - northeast of Inglewood, where the rail intersects with Millmerran Inglewood Road;
 - the potential for sodic soils means nearby soils should be assumed to be extremely erodible (worst-case).
- Moderate K-factor (old alluvial plain) near Millmerran:
 - mapped over footprint, from before Millmerran (where the rail crosses the Millmerran to Inglewood Road) to Yandilla (where the rail crosses the Millmerran to Leyburn Road);
 - alluvial soils are typically associated with highly erodible soils (Table 2 – note 2).
- Moderate K-factor (recent alluvial soil association) near NSW border:
 - within an approximately 3 km buffer of the border;
 - alluvial soils are typically associated with highly erodible soils (Table 2 – note 2).

5.0 Conclusions

Erosion threshold velocities (Table 8) were assigned for each of the Project SMUs using the USC soil classification criteria (ASTM 2006), BPESC guidelines (IECA 2008) and soil samples collected during the soil assessment (Aurecon 2022).

As a practicable tool, the Project SMUs shapefile created during the soil assessment has been updated to include the USC soil groups, erodibility classes and ETV values that were assigned during the assessment described in this letter. This shapefile will provide a practicable tool in selecting culvert designs for each section of the Project that will meet the ETV values and as a result protect the receiving environment.

The erodibility assessment and ETVs that have been assigned for each SMU should be used to identify liner requirements for each of the culverts along the Project corridor. If a culvert's design velocity exceeds the ETV that has been mapped over the area, then a liner (or equivalent) will be required to reduce velocities to below the ETV.

5.1 Further studies

The information contained herein cannot be applied to detailed design of the Project. During the detailed design, further studies should be done to ensure appropriate identification and management of dispersive soils (Section 5.1.1) and other site constraints that have the potential to influence the erodibility and the subsequent ETVs (Section 5.1.2). This may take the form of a soil / erosion hazard checklist done by the site supervisor, to confirm the SMUs present and account for any site conditions that may affect erodibility.

5.1.1 Identifying dispersive soils

Dispersive soils present one of the greatest challenges regarding the soil management of the Project. Since it is impracticable to sample the entire Project footprint to test for dispersive soil, it is essential to be able to quickly identify them in the field.



Dispersive soils may be identified by distinctive dribble patterns (or 'worm channels') that form following exposure to rain or low electrolyte runoff. These patterns are considered to be a reliable indicator of moderately to highly dispersive soils. Where topsoil has been removed by erosion or disturbance, pitting and pocketing can also be observed in subsoils exposed to rainfall or runoff (DPIW 2009). Cloudy pools of standing water (ie dams and puddles) can be another simple indicator that a soil is dispersive.

If a soil is suspected to be dispersive, a simple field test can be performed by submerging soil aggregates in distilled water and observing any dispersion or slaking behaviour. The method for performing this field test is described in Catchment and Creek's *On-site Soil Testing — General Construction Practice* (Appendix C).

If soil is found to be dispersive (based on field tests, ESP, Emerson class and / or Dispersion Index), then it should be considered extremely erodible (ETV of 0.3 m/s) even at low velocities (ie chemically dispersive) and must be either treated (ie gypsum) or covered with a minimum of 100 mm of stable soil (IECA 2008).

5.1.2 Site constraints

The following site constraints should be identified and accounted for by conservatively increasing the assigned erodibility class (and subsequent ETV) of the area.

- slopes >10%;
- unstable terrain;
- flood inundation lines;
- rock outcrops; and
- existing erosion.

5.2 Assumptions

The ETVs provided in this letter (Table 8) have been derived for use in culverts and constructed channels. Historic hydrological studies have measured maximum velocities in Project watercourses that greatly exceed the provided ETVs. Due to the complex nature of these systems and little research being available for allowable velocities within Queensland's drainage lines it is recommended that the ETVs are not applied to Project watercourses.

Section 2.4 of the Austroads *Guide to Road Design Part 5B: Drainage— Open Channels, Culverts and Floodways* (Austroads 2013) derives erosive velocities in natural streams for a variety of stream bed types (ie silt, sand, gravel, clay or rock) that may be used, along with the information in this letter, to inform construction decisions.

Yours sincerely



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TMR 2019a, *Road Drainage Manual, Transport and Main Roads — Chapter 2: General Design Requirements*, Department of Transport and Main Roads, Queensland Government, Brisbane.

TMR 2019b, *Road Drainage Manual, Transport and Main Roads — Chapter 9: Culvert Design*, Department of Transport and Main Roads, Queensland Government, Brisbane.



Appendix A

D 2487 – 06: Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System)





Designation: D 2487 – 06

Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System)¹

This standard is issued under the fixed designation D 2487; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

This standard has been approved for use by agencies of the Department of Defense.

1. Scope*

1.1 This practice describes a system for classifying mineral and organo-mineral soils for engineering purposes based on laboratory determination of particle-size characteristics, liquid limit, and plasticity index and shall be used when precise classification is required.

NOTE 1—Use of this standard will result in a single classification group symbol and group name except when a soil contains 5 to 12 % fines or when the plot of the liquid limit and plasticity index values falls into the crosshatched area of the plasticity chart. In these two cases, a dual symbol is used, for example, GP-GM, CL-ML. When the laboratory test results indicate that the soil is close to another soil classification group, the borderline condition can be indicated with two symbols separated by a slash. The first symbol should be the one based on this standard, for example, CL/CH, GM/SM, SC/CL. Borderline symbols are particularly useful when the liquid limit value of clayey soils is close to 50. These soils can have expansive characteristics and the use of a borderline symbol (CL/CH, CH/CL) will alert the user of the assigned classifications of expansive potential.

1.2 The group symbol portion of this system is based on laboratory tests performed on the portion of a soil sample passing the 3-in. (75-mm) sieve (see Specification E 11).

1.3 As a classification system, this standard is limited to naturally occurring soils.

NOTE 2—The group names and symbols used in this test method may be used as a descriptive system applied to such materials as shale, claystone, shells, crushed rock, etc. See Appendix X2.

1.4 This standard is for qualitative application only.

NOTE 3—When quantitative information is required for detailed designs of important structures, this test method must be supplemented by laboratory tests or other quantitative data to determine performance characteristics under expected field conditions.

1.5 This standard is the ASTM version of the Unified Soil Classification System. The basis for the classification scheme is the Airfield Classification System developed by A. Casa-

grande in the early 1940's.² It became known as the Unified Soil Classification System when several U.S. Government Agencies adopted a modified version of the Airfield System in 1952.

1.6 *This standard does not purport to address all of the safety concerns, if any, associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.*

1.7 *This practice offers a set of instructions for performing one or more specific operations. This document cannot replace education or experience and should be used in conjunction with professional judgment. Not all aspects of this practice may be applicable in all circumstances. This ASTM standard is not intended to represent or replace the standard of care by which the adequacy of a given professional service must be judged, nor should this document be applied without consideration of a project's many unique aspects. The word "Standard" in the title of this document means only that the document has been approved through the ASTM consensus process.*

2. Referenced Documents

2.1 ASTM Standards:³

C 117 Test Method for Materials Finer than 75-μm (No. 200) Sieve in Mineral Aggregates by Washing

C 136 Test Method for Sieve Analysis of Fine and Coarse Aggregates

C 702 Practice for Reducing Samples of Aggregate to Testing Size

D 420 Guide to Site Characterization for Engineering Design and Construction Purposes

D 422 Test Method for Particle-Size Analysis of Soils

D 653 Terminology Relating to Soil, Rock, and Contained Fluids

¹ This standard is under the jurisdiction of ASTM Committee D18 on Soil and Rock and is the direct responsibility of Subcommittee D18.07 on Identification and Classification of Soils.

Current edition approved May 1, 2006. Published June 2006. Originally approved in 1966. Last previous edition approved in 2000 as D 2487 – 00.

² Casagrande, A., "Classification and Identification of Soils," *Transactions, ASCE*, 1948, p. 901.

³ For referenced ASTM standards, visit the ASTM website, www.astm.org, or contact ASTM Customer Service at service@astm.org. For *Annual Book of ASTM Standards* volume information, refer to the standard's Document Summary page on the ASTM website.

*A Summary of Changes section appears at the end of this standard.



- D 1140** Test Method for Amount of Material in Soils Finer Than the No. 200 (75- μ m) Sieve
- D 2216** Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass
- D 2217** Practice for Wet Preparation of Soil Samples for Particle-Size Analysis and Determination of Soil Constants
- D 2488** Practice for Description and Identification of Soils (Visual-Manual Procedure)
- D 3740** Practice for Minimum Requirements for Agencies Engaged in the Testing and/or Inspection of Soil and Rock as Used in Engineering Design and Construction
- D 4083** Practice for Description of Frozen Soils (Visual-Manual Procedure)
- D 4318** Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils
- D 4427** Classification of Peat Samples by Laboratory Testing
- D 6913** Test Methods for Particle-Size Distribution (Gradation) of Soils Using Sieve Analysis
- E 11** Specification for Wire Cloth and Sieves for Testing Purposes

3. Terminology

3.1 *Definitions*—Except as listed below, all definitions are in accordance with Terminology **D 653**.

NOTE 4—For particles retained on a 3-in. (75-mm) U.S. standard sieve, the following definitions are suggested:

Cobbles—particles of rock that will pass a 12-in. (300-mm) square opening and be retained on a 3-in. (75-mm) U.S. standard sieve, and

Boulders—particles of rock that will not pass a 12-in. (300-mm) square opening.

3.1.1 *clay*—soil passing a No. 200 (75- μ m) U.S. standard sieve that can be made to exhibit plasticity (putty-like properties) within a range of water contents and that exhibits considerable strength when air dry. For classification, a clay is a fine-grained soil, or the fine-grained portion of a soil, with a plasticity index equal to or greater than 4, and the plot of plasticity index versus liquid limit falls on or above the “A” line.

3.1.2 *gravel*—particles of rock that will pass a 3-in. (75-mm) sieve and be retained on a No. 4 (4.75-mm) U.S. standard sieve with the following subdivisions:

Coarse—passes 3-in. (75-mm) sieve and retained on $\frac{3}{4}$ -in. (19-mm) sieve, and

Fine—passes $\frac{3}{4}$ -in. (19-mm) sieve and retained on No. 4 (4.75-mm) sieve.

3.1.3 *organic clay*—a clay with sufficient organic content to influence the soil properties. For classification, an organic clay is a soil that would be classified as a clay except that its liquid limit value after oven drying is less than 75 % of its liquid limit value before oven drying.

3.1.4 *organic silt*—a silt with sufficient organic content to influence the soil properties. For classification, an organic silt is a soil that would be classified as a silt except that its liquid limit value after oven drying is less than 75 % of its liquid limit value before oven drying.

3.1.5 *peat*—a soil composed of vegetable tissue in various stages of decomposition usually with an organic odor, a dark-brown to black color, a spongy consistency, and a texture ranging from fibrous to amorphous.

3.1.6 *sand*—particles of rock that will pass a No. 4 (4.75-mm) sieve and be retained on a No. 200 (75- μ m) U.S. standard sieve with the following subdivisions:

Coarse—passes No. 4 (4.75-mm) sieve and retained on No. 10 (2.00-mm) sieve,

Medium—passes No. 10 (2.00-mm) sieve and retained on No. 40 (425- μ m) sieve, and

Fine—passes No. 40 (425- μ m) sieve and retained on No. 200 (75- μ m) sieve.

3.1.7 *silt*—soil passing a No. 200 (75- μ m) U.S. standard sieve that is nonplastic or very slightly plastic and that exhibits little or no strength when air dry. For classification, a silt is a fine-grained soil, or the fine-grained portion of a soil, with a plasticity index less than 4 or if the plot of plasticity index versus liquid limit falls below the “A” line.

3.2 *Definitions of Terms Specific to This Standard:*

3.2.1 *coefficient of curvature, C_c* —the ratio $(D_{30})^2 / (D_{10} \times D_{60})$, where D_{60} , D_{30} , and D_{10} are the particle sizes corresponding to 60, 30, and 10 % finer on the cumulative particle-size distribution curve, respectively.

3.2.2 *coefficient of uniformity, C_u* —the ratio D_{60}/D_{10} , where D_{60} and D_{10} are the particle diameters corresponding to 60 and 10 % finer on the cumulative particle-size distribution curve, respectively.

4. Summary

4.1 As illustrated in **Table 1**, this classification system identifies three major soil divisions: coarse-grained soils, fine-grained soils, and highly organic soils. These three divisions are further subdivided into a total of 15 basic soil groups.

TABLE 1 Soil Classification Chart

Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests ^A				Soil Classification	
				Group Symbol	Group Name ^B
COARSE-GRAINED SOILS	Gravels	Clean Gravels	$C_u \geq 4$ and $1 \leq C_c \leq 3^C$	GW	Well-graded gravel ^D
More than 50 % retained on No. 200 sieve	More than 50 % of coarse fraction retained on No. 4 sieve	Less than 5 % fines ^E	$C_u < 4$ and/or $1 > C_c > 3^C$	GP	Poorly graded gravel ^D
		Gravels with Fines	Fines classify as ML or MH	GM	Silty gravel ^{D, F, G}

TABLE 1 *Continued*

Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests ^A			Soil Classification	
			Group Symbol	Group Name ^B
	More than 12 % fines ^E	Fines classify as CL or CH	Gravel; 1 > GC	Clayey gravel ^{D, F, G}
Sands	Clean Sands	$Cu \geq 6$ and $1 \leq Cc \leq 3^C$	SW	Well-graded sand ^H
50 % or more of coarse fraction passes No. 4 sieve	Less than 5 % fines ^I	$Cu < 6$ and/or $1 > Cc > 3^C$	SP	Poorly graded sand ^H
	Sands with Fines	Fines classify as ML or MH	SM	Silty sand ^{F, G, H}
	More than 12 % fines ^I	Fines classify as CL or CH	SC	Clayey sand ^{F, G, H}
FINE-GRAINED SOILS	Silts and Clays	inorganic	CL	Lean clay ^{K, L, M}
50 % or more passes the No. 200 sieve	Liquid limit less than 50	PI < 4 or plots below "A" line ^J	ML	Silt ^{K, L, M}
	organic	Liquid limit – oven dried > < 0.75	OL	Organic clay ^{K, L, M, N}
		Liquid limit – not dried	OL	Organic silt ^{K, L, M, O}
	Silts and Clays	inorganic	CH	Fat clay ^{K, L, M}
		PI plots on or above "A" line	CH	Fat clay ^{K, L, M}
	Liquid limit 50 or more	PI plots below "A" line	MH	Elastic silt ^{K, L, M}
	organic	Liquid limit – oven dried > < 0.75	OH	Organic clay ^{K, L, M, P}
		Liquid limit – not dried	OH	Organic silt ^{K, L, M, O}
HIGHLY ORGANIC SOILS	Primarily organic matter, dark in color, and organic odor		PT	Peat

^A Based on the material passing the 3-in. (75-mm) sieve.

^B If field sample contained cobbles or boulders, or both, add "with cobbles or boulders, or both" to group name.

^C $Cu = D_{60}/D_{10}$ $Cc = (D_{30})^2 / D_{10} \times D_{60}$

^D If soil contains ≥ 15 % sand, add "with sand" to group name.

^E Gravels with 5 to 12 % fines require dual symbols:

GW-GM well-graded gravel with silt
GW-GC well-graded gravel with clay
GP-GM poorly graded gravel with silt
GP-GC poorly graded gravel with clay

^F If fines classify as CL-ML, use dual symbol GC-GM, or SC-SM.

^G If fines are organic, add "with organic fines" to group name.

^H If soil contains ≥ 15 % gravel, add "with gravel" to group name.

^I Sands with 5 to 12 % fines require dual symbols:

SW-SM well-graded sand with silt
SW-SC well-graded sand with clay
SP-SM poorly graded sand with silt
SP-SC poorly graded sand with clay

^J If Atterberg limits plot in hatched area, soil is a CL-ML, silty clay.

^K If soil contains 15 to 29 % plus No. 200, add "with sand" or "with gravel," whichever is predominant.

^L If soil contains ≥ 30 % plus No. 200, predominantly sand, add "sand" to group name.

^M If soil contains ≥ 30 % plus No. 200, predominantly gravel, add "gravelly" to group name.

^N $PI \geq 4$ and plots on or above "A" line.

^O $PI < 4$ or plots below "A" line.

^P PI plots on or above "A" line.

^Q PI plots below "A" line.

4.2 Based on the results of visual observations and prescribed laboratory tests, a soil is catalogued according to the basic soil groups, assigned a group symbol(s) and name, and thereby classified. The flow charts, Fig. 1 for fine-grained soils, and Fig. 3 for coarse-grained soils, can be used to assign the appropriate group symbol(s) and name.

5. Significance and Use

5.1 This standard classifies soils from any geographic location into categories representing the results of prescribed laboratory tests to determine the particle-size characteristics, the liquid limit, and the plasticity index.

5.2 The assigning of a group name and symbol(s) along with the descriptive information required in Practice D 2488 can be used to describe a soil to aid in the evaluation of its significant properties for engineering use.

5.3 The various groupings of this classification system have been devised to correlate in a general way with the engineering behavior of soils. This standard provides a useful first step in any field or laboratory investigation for geotechnical engineering purposes.

5.4 This standard may also be used as an aid in training personnel in the use of Practice D 2488.

5.5 This standard may be used in combination with Practice D 4083 when working with frozen soils.

NOTE 5—Notwithstanding the statements on precision and bias contained in this standard: The precision of this test method is dependent on the competence of the personnel performing it and the suitability of the equipment and facilities used. Agencies that meet the criteria of Practice D 3740 are generally considered capable of competent and objective testing. Users of this test method are cautioned that compliance with Practice D 3740 does not in itself assure reliable testing. Reliable testing

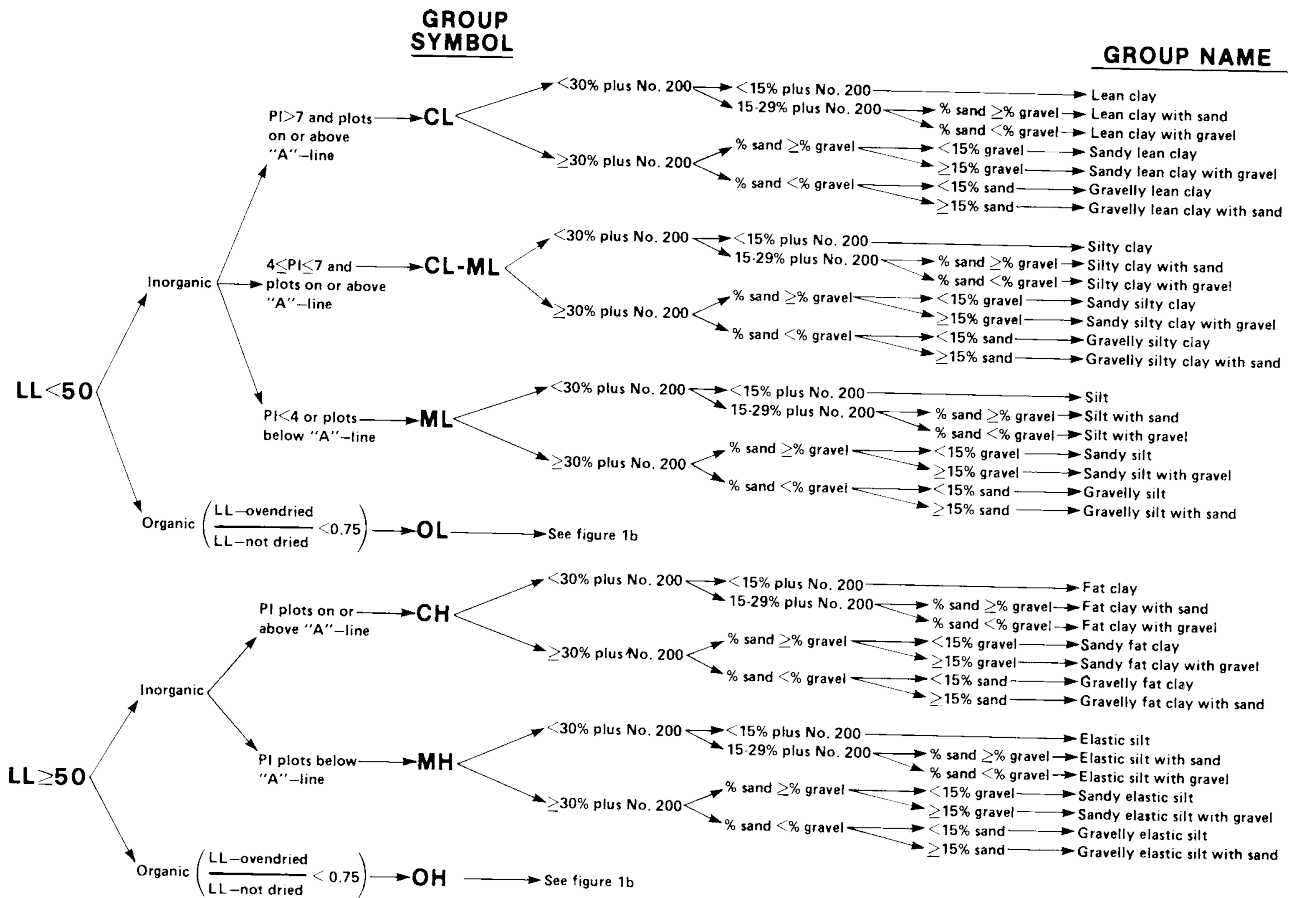


FIG. 1 Flow Chart for Classifying Fine-Grained Soil (50 % or More Passes No. 200 Sieve)

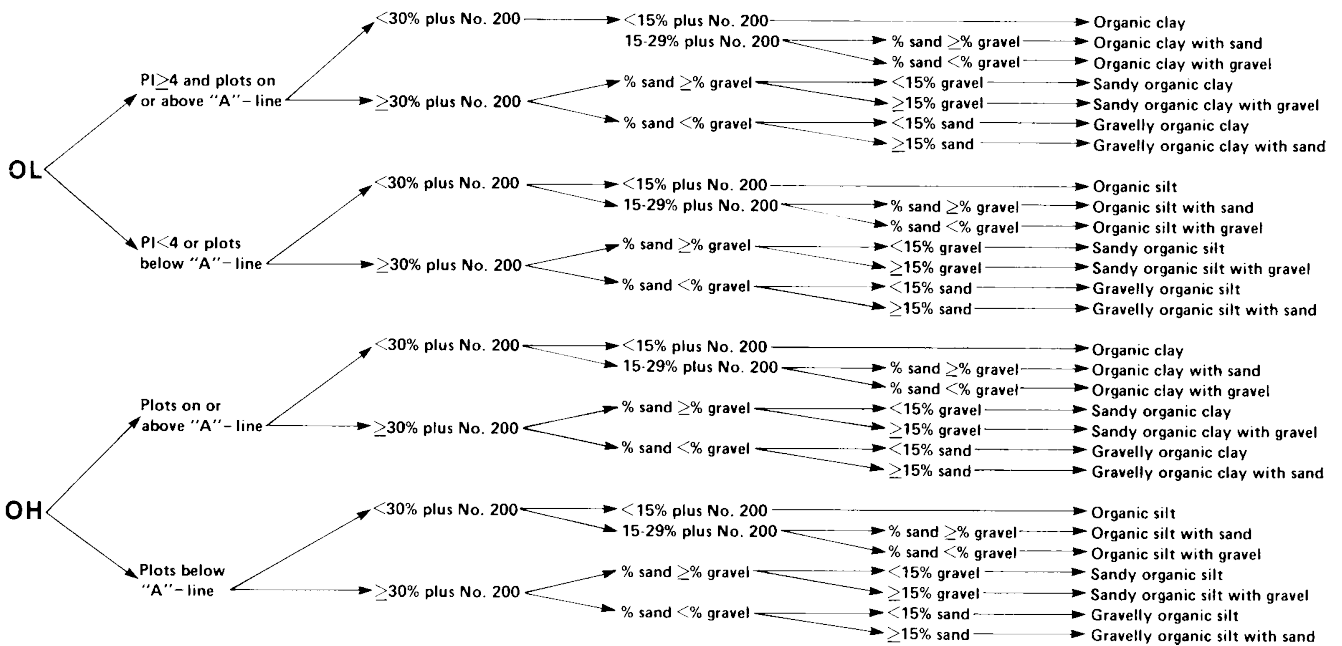
GROUP SYMBOL**GROUP NAME**

FIG. 2 Flow Chart for Classifying Organic Fine-Grained Soil (50 % or More Passes No. 200 Sieve)

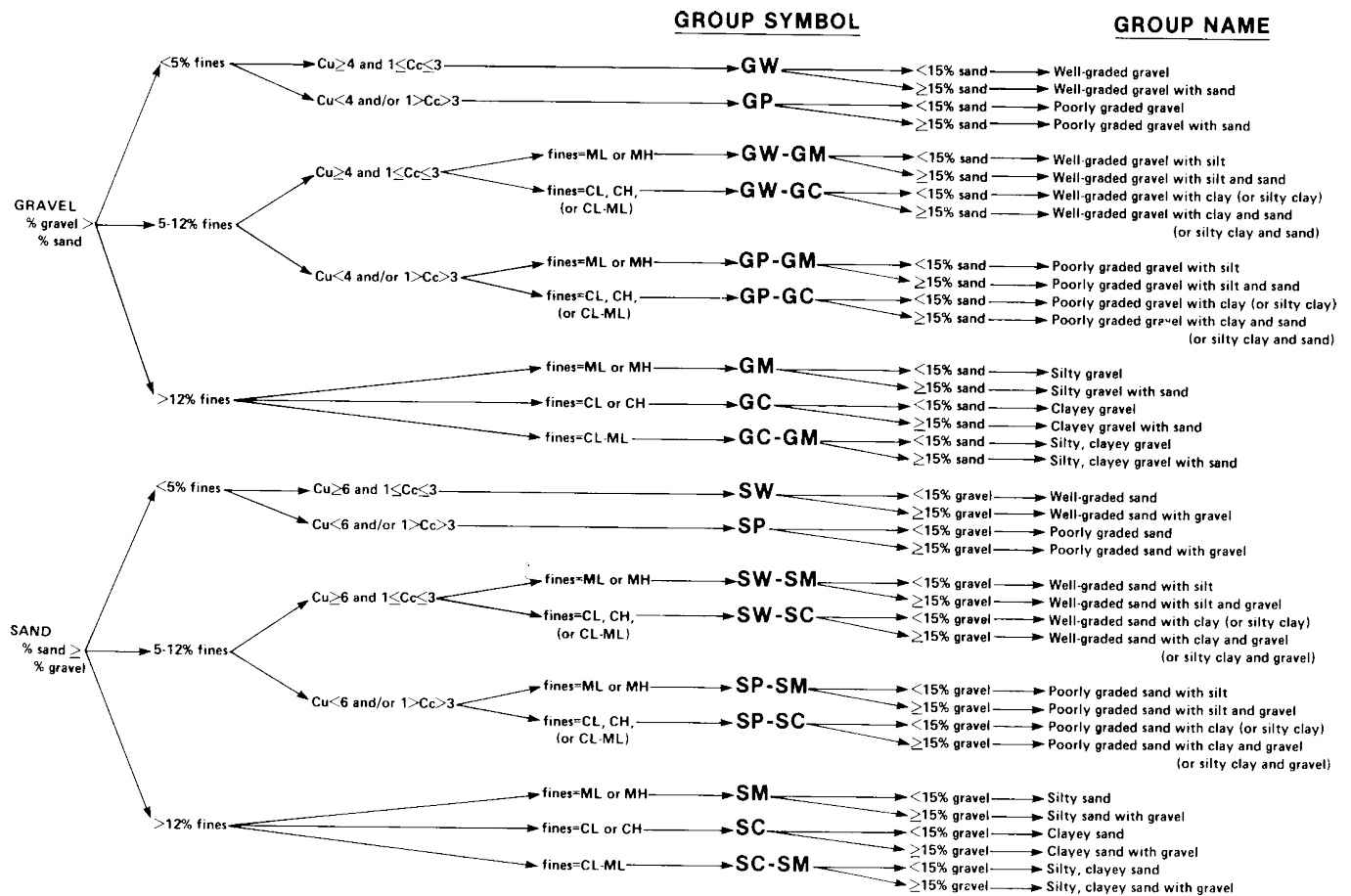


FIG. 3 Flow Chart for Classifying Coarse-Grained Soils (More Than 50 % Retained on No. 200 Sieve)

depends on several factors; Practice D 3740 provides a means for evaluating some of those factors.

6. Apparatus

6.1 In addition to the apparatus that may be required for obtaining and preparing the samples and conducting the prescribed laboratory tests, a plasticity chart, similar to Fig. 4, and a cumulative particle-size distribution curve, similar to Fig. 5, are required.

NOTE 6—The “U” line shown on Fig. 4 has been empirically determined to be the approximate “upper limit” for natural soils. It is a good check against erroneous data, and any test results that plot above or to the left of it should be verified.

7. Sampling

7.1 Samples shall be obtained and identified in accordance with a method or methods, recommended in Guide D 420 or by other accepted procedures.

7.2 Test Methods D 6913 provides guidance on selecting size of specimen. Two test methods are provided in this standard. The methods differ in the significant digits recorded and the size of the specimen (mass) required. The method to be used may be specified by the requesting authority; otherwise Method A shall be performed. Whenever possible, the field samples should have weights two to four times larger than shown.

7.3 If the field sample or test specimen is smaller than the minimum recommended amount, the report shall include an appropriate remark.

8. Classification of Peat

8.1 A sample composed primarily of vegetable tissue in various stages of decomposition and has a fibrous to amorphous texture, a dark-brown to black color, and an organic odor should be designated as a highly organic soil and shall be classified as peat, PT, and not subjected to the classification procedures described hereafter.

8.2 If desired, classification of type of peat can be performed in accordance with Classification D 4427.

9. Preparation for Classification

9.1 Before a soil can be classified according to this standard, generally the particle-size distribution of the minus 3-in. (75-mm) material and the plasticity characteristics of the minus No. 40 (425- μ m) sieve material must be determined. See 9.8 for the specific required tests.

9.2 The preparation of the soil specimen(s) and the testing for particle-size distribution and liquid limit and plasticity index shall be in accordance with accepted standard procedures. Two procedures for preparation of the soil specimens for testing for soil classification purposes are given in Appendixes X3 and X4. Appendix X3 describes the wet preparation method

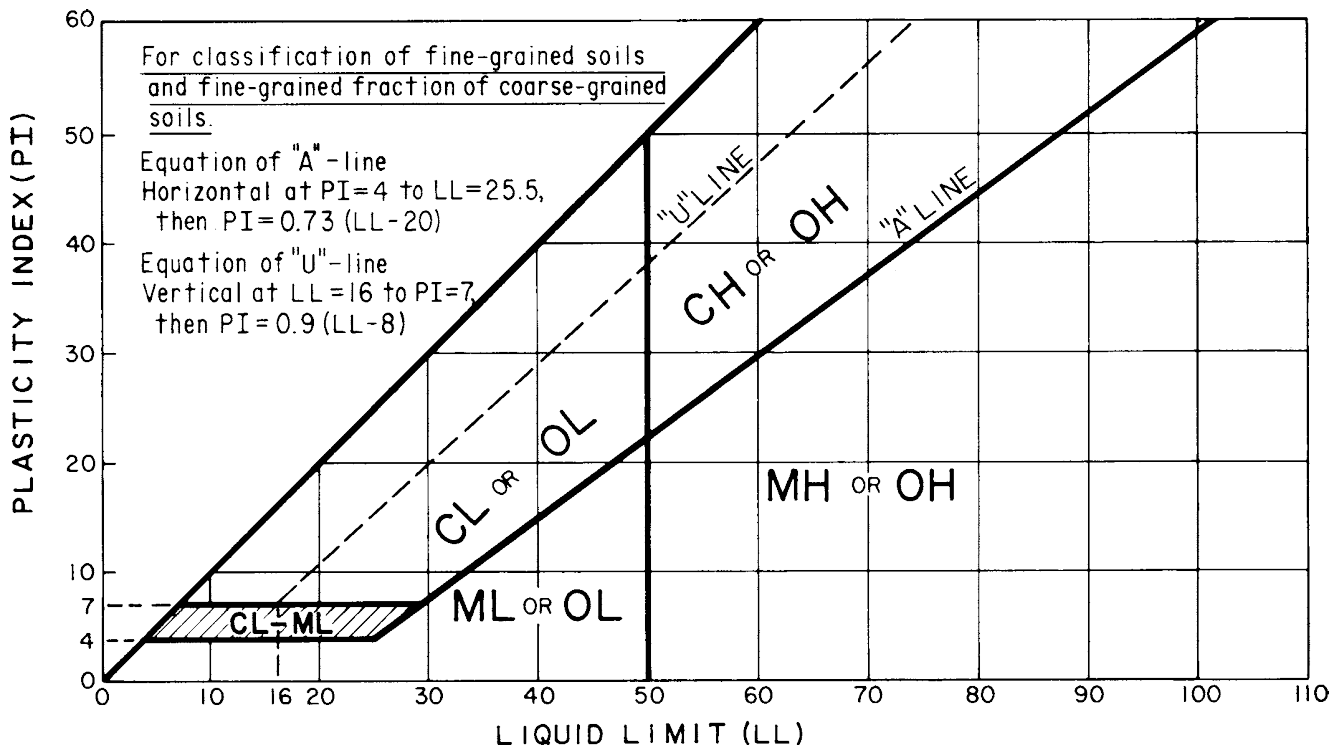
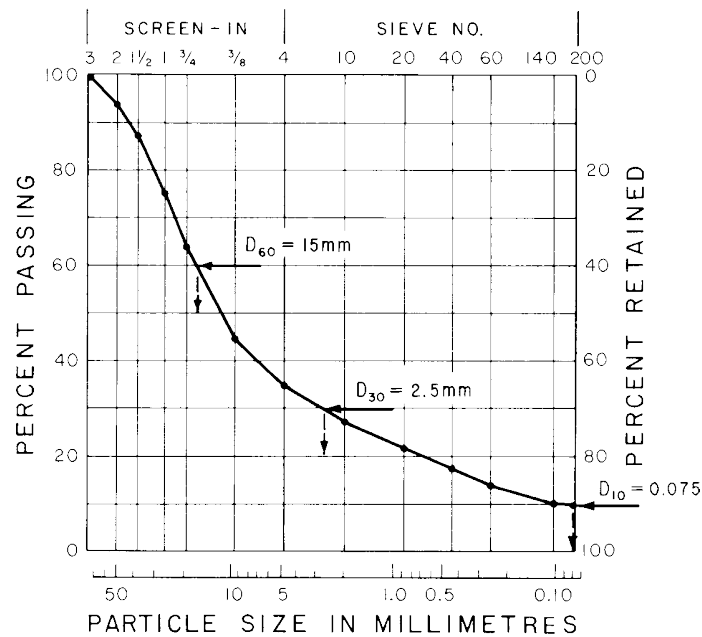


FIG. 4 Plasticity Chart

SIEVE ANALYSIS



$$C_u = \frac{D_{60}}{D_{10}} = \frac{15}{0.075} = 200$$

$$C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}} = \frac{(2.5)^2}{0.075 \times 15} = 5.6$$

FIG. 5 Cumulative Particle-Size Plot

and is the preferred method for cohesive soils that have never dried out and for organic soils.

9.3 When reporting soil classifications determined by this standard, the preparation and test procedures used shall be reported or referenced.

9.4 Although the test procedure used in determining the particle-size distribution or other considerations may require a hydrometer analysis of the material, a hydrometer analysis is not necessary for soil classification.

9.5 The percentage (by dry weight) of any plus 3-in. (75-mm) material must be determined and reported as auxiliary information.

9.6 The maximum particle size shall be determined (measured or estimated) and reported as auxiliary information.

9.7 When the cumulative particle-size distribution is required, a set of sieves shall be used which include the following sizes (with the largest size commensurate with the maximum particle size) with other sieve sizes as needed or required to define the particle-size distribution:

3-in. (75-mm)
 ¾-in. (19.0-mm)
 No. 4 (4.75-mm)
 No. 10 (2.00-mm)
 No. 40 (425-µm)
 No. 200 (75-µm)

9.8 The tests required to be performed in preparation for classification are as follows:

9.8.1 For soils estimated to contain less than 5 % fines, a plot of the cumulative particle-size distribution curve of the fraction coarser than the No. 200 (75-µm) sieve is required. A semi-log plot of percent passing versus particle-size or sieve size/sieve number is plotted as shown in Fig. 5.

9.8.2 For soils estimated to contain 5 to 15 % fines, a cumulative particle-size distribution curve, as described in 9.8.1, is required, and the liquid limit and plasticity index are required.

9.8.2.1 If sufficient material is not available to determine the liquid limit and plasticity index, the fines should be estimated to be either silty or clayey using the procedures described in Practice D 2488 and so noted in the report.

9.8.3 For soils estimated to contain 15 % or more fines, a determination of the percent fines, percent sand, and percent gravel is required, and the liquid limit and plasticity index are required. For soils estimated to contain 90 % fines or more, the percent fines, percent sand, and percent gravel may be estimated using the procedures described in Practice D 2488 and so noted in the report.

10. Preliminary Classification Procedure

10.1 Class the soil as fine-grained if 50 % or more by dry weight of the test specimen passes the No. 200 (75-µm) sieve and follow Section 3.1.2.

10.2 Class the soil as coarse-grained if more than 50 % by dry weight of the test specimen is retained on the No. 200 (75-µm) sieve and follow Section 12.

11. Procedure for Classification of Fine-Grained Soils

(50 % or more by dry weight passing the No. 200 (75-µm) sieve)

11.1 The soil is an inorganic clay if the position of the plasticity index versus liquid limit plot, Fig. 4, falls on or above the “A” line, the plasticity index is greater than 4, and the presence of organic matter does not influence the liquid limit as determined in 11.3.2.

NOTE 7—The plasticity index and liquid limit are determined on the minus No. 40 (425 µm) sieve material.

11.1.1 Classify the soil as a *lean clay*, CL, if the liquid limit is less than 50. See area identified as CL on Fig. 4.

11.1.2 Classify the soil as a *fat clay*, CH, if the liquid limit is 50 or greater. See area identified as CH on Fig. 4.

NOTE 8—In cases where the liquid limit exceeds 110 or the plasticity index exceeds 60, the plasticity chart may be expanded by maintaining the same scale on both axes and extending the “A” line at the indicated slope.

11.1.3 Classify the soil as a *silty clay*, CL-ML, if the position of the plasticity index versus liquid limit plot falls on or above the “A” line and the plasticity index is in the range of 4 to 7. See area identified as CL-ML on Fig. 4.

11.2 The soil is an inorganic silt if the position of the plasticity index versus liquid limit plot, Fig. 4, falls below the “A” line or the plasticity index is less than 4, and presence of organic matter does not influence the liquid limit as determined in 11.3.2.

11.2.1 Classify the soil as a *silt*, ML, if the liquid limit is less than 50. See area identified as ML on Fig. 4.

11.2.2 Classify the soil as an *elastic silt*, MH, if the liquid limit is 50 or greater. See area identified as MH on Fig. 4.

11.3 The soil is an organic silt or clay if organic matter is present in sufficient amounts to influence the liquid limit as determined in 11.3.2.

11.3.1 If the soil has a dark color and an organic odor when moist and warm, a second liquid limit test shall be performed on a test specimen which has been oven dried at $110 \pm 5^\circ\text{C}$ to a constant weight, typically over night.

11.3.2 The soil is an organic silt or organic clay if the liquid limit after oven drying is less than 75 % of the liquid limit of the original specimen determined before oven drying (see Procedure B of Practice D 2217).

11.3.3 Classify the soil as an *organic silt* or *organic clay*, OL, if the liquid limit (not oven dried) is less than 50 %. Classify the soil as an *organic silt*, OL, if the plasticity index is less than 4, or the position of the plasticity index versus liquid limit plot falls below the “A” line. Classify the soil as an *organic clay*, OL, if the plasticity index is 4 or greater and the position of the plasticity index versus liquid limit plot falls on or above the “A” line. See area identified as OL (or CL-ML) on Fig. 4.

11.3.4 Classify the soil as an *organic clay* or *organic silt*, OH, if the liquid limit (not oven dried) is 50 or greater. Classify the soil as an *organic silt*, OH, if the position of the plasticity index versus liquid limit plot falls below the “A” line. Classify the soil as an *organic clay*, OH, if the position of the plasticity index versus liquid-limit plot falls on or above the “A” line. See area identified as OH on Fig. 4.

11.4 If less than 30 % but 15 % or more of the test specimen is retained on the No. 200 (75- μ m) sieve, the words “with sand” or “with gravel” (whichever is predominant) shall be added to the group name. For example, lean clay with sand, CL; silt with gravel, ML. If the percent of sand is equal to the percent of gravel, use “with sand.”

11.5 If 30 % or more of the test specimen is retained on the No. 200 (75- μ m) sieve, the words “sandy” or “gravelly” shall be added to the group name. Add the word “sandy” if 30 % or more of the test specimen is retained on the No. 200 (75- μ m) sieve and the coarse-grained portion is predominantly sand. Add the word “gravelly” if 30 % or more of the test specimen is retained on the No. 200 (75- μ m) sieve and the coarse-grained portion is predominantly gravel. For example, sandy lean clay, CL; gravelly fat clay, CH; sandy silt, ML. If the percent of sand is equal to the percent of gravel, use “sandy.”

12. Procedure for Classification of Coarse-Grained Soils (more than 50 % retained on the No. 200 (75- μ m) sieve)

12.1 Class the soil as gravel if more than 50 % of the coarse fraction [plus No. 200 (75- μ m) sieve] is retained on the No. 4 (4.75-mm) sieve.

12.2 Class the soil as sand if 50 % or more of the coarse fraction [plus No. 200 (75- μ m) sieve] passes the No. 4 (4.75-mm) sieve.

12.3 If 12 % or less of the test specimen passes the No. 200 (75- μ m) sieve, plot the cumulative particle-size distribution, Fig. 5, and compute the coefficient of uniformity, C_u , and coefficient of curvature, C_c , as given in Eqs 1 and 2.

$$C_u = D_{60}/D_{10} \quad (1)$$

$$C_c = (D_{30})^2 / (D_{10} \times D_{60}) \quad (2)$$

where:

D_{10} , D_{30} , and D_{60} = the particle-size diameters corresponding to 10, 30, and 60 %, respectively, passing on the cumulative particle-size distribution curve, Fig. 5.

NOTE 9—It may be necessary to extrapolate the curve to obtain the D_{10} diameter.

12.3.1 If less than 5 % of the test specimen passes the No. 200 (75- μ m) sieve, classify the soil as a *well-graded gravel*, GW, or *well-graded sand*, SW, if C_u is greater than or equal to 4.0 for gravel or greater than 6.0 for sand, and C_c is at least 1.0 but not more than 3.0.

12.3.2 If less than 5 % of the test specimen passes the No. 200 (75- μ m) sieve, classify the soil as *poorly graded gravel*, GP, or *poorly graded sand*, SP, if either the C_u or the C_c criteria for well-graded soils are not satisfied.

12.4 If more than 12 % of the test specimen passes the No. 200 (75- μ m) sieve, the soil shall be considered a coarse-grained soil with fines. The fines are determined to be either clayey or silty based on the plasticity index versus liquid limit plot on Fig. 4. (See 9.8.2.1 if insufficient material available for testing) (see Note 7).

12.4.1 Classify the soil as a *clayey gravel*, GC, or *clayey sand*, SC, if the fines are clayey, that is, the position of the plasticity index versus liquid limit plot, Fig. 4, falls on or above the “A” line and the plasticity index is greater than 7.

12.4.2 Classify the soil as a *silty gravel*, GM, or *silty sand*, SM, if the fines are silty, that is, the position of the plasticity index versus liquid limit plot, Fig. 4, falls below the “A” line or the plasticity index is less than 4.

12.4.3 If the fines plot as a silty clay, CL-ML, classify the soil as a *silty, clayey gravel*, GC-GM, if it is a gravel or a *silty, clayey sand*, SC-SM, if it is a sand.

12.5 If 5 to 12 % of the test specimen passes the No. 200 (75- μ m) sieve, give the soil a dual classification using two group symbols.

12.5.1 The first group symbol shall correspond to that for a gravel or sand having less than 5 % fines (GW, GP, SW, SP), and the second symbol shall correspond to a gravel or sand having more than 12 % fines (GC, GM, SC, SM).

12.5.2 The group name shall correspond to the first group symbol plus “with clay” or “with silt” to indicate the plasticity characteristics of the fines. For example, well-graded gravel with clay, GW-GC; poorly graded sand with silt, SP-SM (See 9.8.2.1 if insufficient material available for testing).

NOTE 10—If the fines plot as a *silty clay*, CL-ML, the second group symbol should be either GC or SC. For example, a poorly graded sand with 10 % fines, a liquid limit of 20, and a plasticity index of 6 would be classified as a poorly graded sand with silty clay, SP-SC.

12.6 If the specimen is predominantly sand or gravel but contains 15 % or more of the other coarse-grained constituent, the words “with gravel” or “with sand” shall be added to the group name. For example, poorly graded gravel with sand, clayey sand with gravel.

12.7 If the field sample contained any cobbles or boulders or both, the words “with cobbles,” or “with cobbles and boulders” shall be added to the group name. For example, silty gravel with cobbles, GM.

13. Report

13.1 The report should include the group name, group symbol, and the results of the laboratory tests. The particle-size distribution shall be given in terms of percent of gravel, sand, and fines. The plot of the cumulative particle-size distribution curve shall be reported if used in classifying the soil. Report appropriate descriptive information according to the procedures in Practice D 2488. A local or commercial name or geologic interpretation for the material may be added at the end of the descriptive information if identified as such. The test procedures used shall be referenced.

NOTE 11—Example: *Clayey Gravel with Sand and Cobbles* (GC)—46 % fine to coarse, hard, subrounded gravel; 30 % fine to coarse, hard, subrounded sand; 24 % clayey fines, LL = 38, PI = 19; weak reaction with HCl; original field sample had 4 % hard, subrounded cobbles; maximum dimension 150 mm.

In-Place Conditions—firm, homogeneous, dry, brown,
Geologic Interpretation—alluvial fan.

NOTE 12—Other examples of soil descriptions are given in Appendix X1.

14. Precision and Bias

14.1 Criteria for acceptability depends on the precision and bias of Test Methods D 422, D 1140 and D 4318.

15. Keywords

15.1 Atterberg limits; classification; clay; gradation; gravel; laboratory classification; organic soils; sand; silt; soil classification; soil tests

APPENDIXES

(Nonmandatory Information)

X1. EXAMPLES OF DESCRIPTIONS USING SOIL CLASSIFICATION

X1.1 The following examples show how the information required in 13.1 can be reported. The appropriate descriptive information from Practice D 2488 is included for illustrative purposes. The additional descriptive terms that would accompany the soil classification should be based on the intended use of the classification and the individual circumstances.

X1.1.1 *Well-Graded Gravel with Sand (GW)*—73 % fine to coarse, hard, subangular gravel; 23 % fine to coarse, hard, subangular sand; 4 % fines; $C_c = 2.7$, $C_u = 12.4$.

X1.1.2 *Silty Sand with Gravel (SM)*—61 % predominantly fine sand; 23 % silty fines, $LL = 33$, $PI = 6$; 16 % fine, hard, subrounded gravel; no reaction with HCl; (field sample smaller than recommended). *In-Place Conditions*—Firm, stratified and contains lenses of silt 1 to 2 in. thick, moist, brown to gray; in-place density = 106 lb/ft³ and in-place moisture = 9 %.

X1.1.3 *Organic Clay (OL)*—100 % fines, LL (not dried) = 32, LL (oven dried) = 21, PI (not dried) = 10; wet, dark brown, organic odor, weak reaction with HCl.

X1.1.4 *Silty Sand with Organic Fines (SM)*—74 % fine to coarse, hard, subangular reddish sand; 26 % organic and silty dark-brown fines, LL (not dried) = 37, LL (oven dried) = 26, PI (not dried) = 6, wet, weak reaction with HCl.

X1.1.5 *Poorly Graded Gravel with Silt, Sand, Cobbles and Boulders (GP-GM)*—78 % fine to coarse, hard, subrounded to subangular gravel; 16 % fine to coarse, hard, subrounded to subangular sand; 6 % silty (estimated) fines; moist, brown; no reaction with HCl; original field sample had 7 % hard, subrounded cobbles and 2 % hard, subrounded boulders with a maximum dimension of 18 in.

X2. USING SOIL CLASSIFICATION AS A DESCRIPTIVE SYSTEM FOR SHALE, CLAYSTONE, SHELLS, SLAG, CRUSHED ROCK, ETC.

X2.1 The group names and symbols used in this standard may be used as a descriptive system applied to materials that exist in situ as shale, claystone, sandstone, siltstone, mudstone, etc., but convert to soils after field or laboratory processing (crushing, slaking, etc.).

X2.2 Materials such as shells, crushed rock, slag, etc., should be identified as such. However, the procedures used in this standard for describing the particle size and plasticity characteristics may be used in the description of the material. If desired, a classification in accordance with this standard may be assigned to aid in describing the material.

X2.3 If a classification is used, the group symbol(s) and group names should be placed in quotation marks or noted with some type of distinguishing symbol. See examples.

X2.4 Examples of how soil classifications could be incorporated into a description system for materials that are not naturally occurring soils are as follows:

X2.4.1 *Shale Chunks*—Retrieved as 2- to 4-in. pieces of shale from power auger hole, dry, brown, no reaction with HCl.

After laboratory processing by slaking in water for 24 h, material classified as “Sandy Lean Clay (CL)” —61 % clayey fines, $LL = 37$, $PI = 16$; 33 % fine to medium sand; 6 % gravel-size pieces of shale.

X2.4.2 *Crushed Sandstone*—Product of commercial crushing operation; “Poorly Graded Sand with Silt (SP-SM)” —91 % fine to medium sand; 9 % silty (estimated) fines; dry, reddish-brown, strong reaction with HCl.

X2.4.3 *Broken Shells*—65 % gravel-size broken shells; 31 % sand and sand-size shell pieces; 4 % fines; $C_c = 2.4$, $C_u = 1.9$; would be classified as “Poorly Graded Gravel with Sand (GP)”.

X2.4.4 *Crushed Rock*—Processed gravel and cobbles from Pit No. 7; “Poorly Graded Gravel (GP)” —89 % fine, hard, angular gravel-size particles; 11 % coarse, hard, angular sand-size particles, dry, tan; no reaction with HCl; $C_c = 2.4$, $C_u = 0.9$.

X3. PREPARATION AND TESTING FOR CLASSIFICATION PURPOSES BY THE WET METHOD

X3.1 This appendix describes the steps in preparing a soil sample for testing for purposes of soil classification using a wet-preparation procedure.

X3.2 Samples prepared in accordance with this procedure should contain as much of their natural water content as possible and every effort should be made during obtaining, preparing, and transporting the samples to maintain the natural moisture.

X3.3 The procedures to be followed in this standard assume that the field sample contains fines, sand, gravel, and plus 3-in. (75-mm) particles and the cumulative particle-size distribution plus the liquid limit and plasticity index values are required (see 9.8). Some of the following steps may be omitted when they are not applicable to the soil being tested.

X3.4 If the soil contains plus No. 200 (75- μ m) particles that would degrade during dry sieving, use a test procedure for determining the particle-size characteristics that prevents this degradation.

X3.5 Since this classification system is limited to the portion of a sample passing the 3-in. (75-mm) sieve, the plus 3-in. (75-mm) material shall be removed prior to the determination of the particle-size characteristics and the liquid limit and plasticity index.

X3.6 The portion of the field sample finer than the 3-in. (75-mm) sieve shall be obtained as follows:

X3.6.1 Separate the field sample into two fractions on a 3-in. (75-mm) sieve, being careful to maintain the natural water content in the minus 3-in. (75-mm) fraction. Any particles adhering to the plus 3-in. (75-mm) particles shall be brushed or wiped off and placed in the fraction passing the 3-in. (75-mm) sieve.

X3.6.2 Determine the air-dry or oven-dry weight of the fraction retained on the 3-in. (75-mm) sieve. Determine the total (wet) weight of the fraction passing the 3-in. (75-mm) sieve.

X3.6.3 Thoroughly mix the fraction passing the 3-in. (75-mm) sieve. Determine the water content, in accordance with Test Method D 2216, of a representative specimen with a minimum dry weight as required in 7.2. Save the water-content specimen for determination of the particle-size analysis in accordance with X3.8.

X3.6.4 Compute the dry weight of the fraction passing the 3-in. (75-mm) sieve based on the water content and total (wet) weight. Compute the total dry weight of the sample and calculate the percentage of material retained on the 3-in. (75-mm) sieve.

X3.7 Determine the liquid limit and plasticity index as follows:

X3.7.1 If the soil disaggregates readily, mix on a clean, hard surface and select a representative sample by quartering in accordance with Practice C 702.

X3.7.1.1 If the soil contains coarse-grained particles coated with and bound together by tough clayey material, take extreme care in obtaining a representative portion of the No. 40 (425- μ m) fraction. Typically, a larger portion than normal has to be selected, such as the minimum weights required in 7.2.

X3.7.1.2 To obtain a representative specimen of a basically cohesive soil, it may be advantageous to pass the soil through a 3/4-in. (19-mm) sieve or other convenient size so the material can be more easily mixed and then quartered or split to obtain the representative specimen.

X3.7.2 Process the representative specimen in accordance with Procedure B of Practice D 2217.

X3.7.3 Perform the liquid-limit test in accordance with Test Method D 4318, except the soil shall not be air dried prior to the test.

X3.7.4 Perform the plastic-limit test in accordance with Test Method D 4318, except the soil shall not be air dried prior to the test, and calculate the plasticity index.

X3.8 Determine the particle-size distribution as follows:

X3.8.1 If the water content of the fraction passing the 3-in. (75-mm) sieve was required (X3.6.3), use the water-content specimen for determining the particle-size distribution. Otherwise, select a representative specimen in accordance with Practice C 702 with a minimum dry weight as required in 7.2.

X3.8.2 If the cumulative particle-size distribution including a hydrometer analysis is required, determine the particle-size distribution in accordance with Test Method D 422. See 9.7 for the set of required sieves.

X3.8.3 If the cumulative particle-size distribution without a hydrometer analysis is required, determine the particle-size distribution in accordance with Test Method C 136. See 9.7 for the set of required sieves. The specimen should be soaked until all clayey aggregations have softened and then washed in accordance with Test Method C 117 prior to performing the particle-size distribution.

X3.8.4 If the cumulative particle-size distribution is not required, determine the percent fines, percent sand, and percent gravel in the specimen in accordance with Test Method C 117, being sure to soak the specimen long enough to soften all clayey aggregations, followed by Test Method C 136 using a nest of sieves which shall include a No. 4 (4.75-mm) sieve and a No. 200 (75- μ m) sieve.

X3.8.5 Calculate the percent fines, percent sand, and percent gravel in the minus 3-in. (75-mm) fraction for classification purposes.

X4. AIR-DRIED METHOD OF PREPARATION OF SOILS FOR TESTING FOR CLASSIFICATION PURPOSES

X4.1 This appendix describes the steps in preparing a soil sample for testing for purposes of soil classification when air-drying the soil before testing is specified or desired or when the natural moisture content is near that of an air-dried state.

X4.2 If the soil contains organic matter or mineral colloids that are irreversibly affected by air drying, the wet-preparation method as described in Appendix X3 should be used.

X4.3 Since this classification system is limited to the portion of a sample passing the 3-in. (75-mm) sieve, the plus 3-in. (75-mm) material shall be removed prior to the determination of the particle-size characteristics and the liquid limit and plasticity index.

X4.4 The portion of the field sample finer than the 3-in. (75-mm) sieve shall be obtained as follows:

X4.4.1 Air dry and weigh the field sample.

X4.4.2 Separate the field sample into two fractions on a 3-in. (75-mm) sieve.

X4.4.3 Weigh the two fractions and compute the percentage of the plus 3-in. (75-mm) material in the field sample.

X4.5 Determine the particle-size distribution and liquid

limit and plasticity index as follows (see 9.8 for when these tests are required):

X4.5.1 Thoroughly mix the fraction passing the 3-in. (75-mm) sieve.

X4.5.2 If the cumulative particle-size distribution including a hydrometer analysis is required, determine the particle-size distribution in accordance with Test Method D 422. See 9.7 for the set of sieves that is required.

X4.5.3 If the cumulative particle-size distribution without a hydrometer analysis is required, determine the particle-size distribution in accordance with Test Method D 1140 followed by Test Method C 136. See 9.7 for the set of sieves that is required.

X4.5.4 If the cumulative particle-size distribution is not required, determine the percent fines, percent sand, and percent gravel in the specimen in accordance with Test Method D 1140 followed by Test Method C 136 using a nest of sieves which shall include a No. 4 (4.75-mm) sieve and a No. 200 (75-μm) sieve.

X4.5.5 If required, determine the liquid limit and the plasticity index of the test specimen in accordance with Test Method D 4318.

X5. ABBREVIATED SOIL CLASSIFICATION SYMBOLS

X5.1 In some cases, because of lack of space, an abbreviated system may be useful to indicate the soil classification symbol and name. Examples of such cases would be graphical logs, databases, tables, etc.

X5.2 This abbreviated system is not a substitute for the full name and descriptive information but can be used in supplementary presentations when the complete description is referenced.

X5.3 The abbreviated system should consist of the soil classification symbol based on this standard with appropriate lower case letter prefixes and suffixes as:

Prefix

Suffix

s = sandy
g = gravelly

s = with sand
g = with gravel
c = cobbles
b = boulders

X5.4 The soil classification symbol is to be enclosed in parentheses. Some examples would be:

Group Symbol and Full Name	Abbreviated
CL, Sandy lean clay	s(CL)
SP-Sm, Poorly graded sand with silt and gravel	(SP-SM)g
GP, poorly graded gravel with sand, cobbles, and boulders	(GP)scb
ML, gravelly silt with sand and cobbles	g(ML)sc



SUMMARY OF CHANGES

In accordance with Committee D18 policy, this section identifies the location of changes to this standard since the last edition (2000) that may impact the use of this standard.

- (1) Added Test Method **D 6913** to Section **2** and **7.2**.
- (2) Corrected **7.2** in selecting size of a sample.
- (3) Corrected Example **X2.4.3**.

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Appendix B

Certificates of Analyses





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East West Enviroag Pty Limited
82 Plain Street Tamworth NSW 2340

ABN 82 125 442 382 124 ph 02 6762 1733 fax 02 6765 9109

SOIL CLASSIFICATION TEST REPORT

AUSTRALIAN STANDARDS METHODS

CLIENT:	SGM Environmental
CLIENT ADDRESS:	PO Box 5622, STAFFORD HEIGHTS QLD 4053
PROJECT:	Material Evaluation
SITE LOCATION:	B2G Soil Samples
DATE OF TESTING:	29/04/2022
TECHNICIAN:	M.C
REPORT NO:	EW220912-1
PROJECT NO:	EW220912
DATE OF REPORT:	6/05/2022
DATE SAMPLED:	-

Sampled By Client: The results apply to samples as received

SAMPLE No:	1	2	3
SAMPLE LOCATION:	310-03-AH3137	310-03-AH3005	310-03-AH3220
DEPTH:	0.0 - 0.1m	0.0 - 0.1m	0.0 - 0.1m
MATERIAL DESCRIPTION:	CLAY	CLAY	Silty CLAY
WASHED/UNWASHED			
AS1289.3.8.1 Water Type & Temperature:			
TEST METHOD	TEST DESCRIPTION		
AS1289.3.6.1	75.0mm		
	63.0mm		
	53.0mm		
	37.5mm		
	26.5mm		
	19.0mm		
	13.2mm		
	9.5mm		
	6.7mm		
	4.75mm		
	2.36mm		
	1.18mm		
	600µm		
	425µm		
	300µm		
	150µm		
	75µm		
AS1289.3.8.1	EMERSON CLASS NO:		
AS1289.2.1.1	MOISTURE CONTENT: %		

CODE					
AS1289.3.1.1	LIQUID LIMIT	%			
AS1289.3.1.2	LIQUID LIMIT	%	2,4	41	41
AS1289.3.2.1	PLASTIC LIMIT	%	2,4	12	19
AS1289.3.3.1	PLASTICITY INDEX	%	2,4	29	22
AS1289.3.4.1	LINEAR SHRINKAGE	%			

CODES USED

Sample history for plasticity tests	
Air Dried	1
Low temperature oven (<50°) dried	2
Other/Unknown	3

Method of preparation for plasticity tests	
Dry sieved	4
Wet sieved	5
Natural	6



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Approved Signatory

Signatory Name: M Coleman

Document ID: REP-102

Issue No: 2

Date of Issue: 24-Apr-13

Page 1 of 1

SOIL CLASSIFICATION TEST REPORT

AUSTRALIAN STANDARDS METHODS

CLIENT:	SGM Environmental		
CLIENT ADDRESS:	PO Box 5622, STAFFORD HEIGHTS QLD 4053	REPORT NO:	EW220912-2
PROJECT:	Material Evaluation	PROJECT NO:	EW220912
SITE LOCATION:	B2G Soil Samples		
DATE OF TESTING:	29/04/2022	DATE OF REPORT:	6/05/2022
TECHNICIAN:	M.C	DATE SAMPLED:	-

Sampled By Client: The results apply to samples as received

		4	5	6
SAMPLE No:		4	5	6
SAMPLE LOCATION:		310-03-AH3571	310-03-AH3639	310-03-AH3266
DEPTH:		0.0 - 0.1m	0.0 - 0.1m	0.0 - 0.1m
MATERIAL DESCRIPTION:		Gravelly Clayey SAND	Gravelly CLAY	SAND
WASHED/UNWASHED				
AS1289.3.8.1 Water Type & Temperature:				
TEST METHOD	TEST DESCRIPTION			
AS1289.3.6.1	75.0mm			
	63.0mm			
	53.0mm			
	37.5mm			
	26.5mm			
	19.0mm			
	13.2mm			
	9.5mm			
SAMPLING METHOD	% Passing Sieve Analysis			
	6.7mm			
	4.75mm			
	2.36mm			
	1.18mm			
	600µm			
	425µm			
	300µm			
AS1289.1.2.1.6.4 AS1289.1.2.1.6.5.1 AS1289.1.2.1.6.5.3 AS1289.1.2.1.6.5.4	150µm			
	75µm			
	EMERSON CLASS NO:			
AS1289.2.1.1	MOISTURE CONTENT: %			

		CODE			
AS1289.3.1.1	LIQUID LIMIT %				
AS1289.3.1.2	LIQUID LIMIT %	2,4	50	39	N/A
AS1289.3.2.1	PLASTIC LIMIT %	2,4	48	20	N/P
AS1289.3.3.1	PLASTICITY INDEX %	2,4	2	19	N/P
AS1289.3.4.1	LINEAR SHRINKAGE %				

CODES USED

Sample history for plasticity tests	
Air Dried	1
Low temperature oven (<50°) dried	2
Other/Unknown	3

Method of preparation for plasticity tests	
Dry sieved	4
Wet sieved	5
Natural	6



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SOIL CLASSIFICATION TEST REPORT

AUSTRALIAN STANDARDS METHODS

CLIENT:	SGM Environmental	REPORT NO:	EW220912-3
CLIENT ADDRESS:	PO Box 5622, STAFFORD HEIGHTS QLD 4053	PROJECT NO:	EW220912
PROJECT:	Material Evaluation	DATE OF REPORT:	6/05/2022
SITE LOCATION:	B2G Soil Samples	DATE SAMPLED:	-
DATE OF TESTING:	29/04/2022		
TECHNICIAN:	M.C		

Sampled By Client: The results apply to samples as received

SAMPLE No:	7	8	9
SAMPLE LOCATION:	310-03-AH3198	310-03-AH3348	310-03-AH3135
DEPTH:	0.0 - 0.1m	0.0 - 0.1m	0.0 - 0.1m
MATERIAL DESCRIPTION:	SAND	SAND	Clayey SAND
WASHED/UNWASHED			
AS1289.3.8.1 Water Type & Temperature:			
TEST METHOD	TEST DESCRIPTION		
AS1289.3.6.1	75.0mm		
	63.0mm		
	53.0mm		
	37.5mm		
	26.5mm		
	19.0mm		
	13.2mm		
	9.5mm		
SAMPLING METHOD	% Passing Sieve Analysis		
	6.7mm		
	4.75mm		
	2.36mm		
	1.18mm		
	600µm		
	425µm		
	300µm		
AS1289.1.2.1.6.4 AS1289.1.2.1.6.5.1 AS1289.1.2.1.6.5.3 AS1289.1.2.1.6.5.4	150µm		
	75µm		
	EMERSON CLASS NO:		
	MOISTURE CONTENT: %		

CODE				
AS1289.3.1.1	LIQUID LIMIT	%		
AS1289.3.1.2	LIQUID LIMIT	%	2,4	N/A
AS1289.3.2.1	PLASTIC LIMIT	%	2,4	N/P
AS1289.3.3.1	PLASTICITY INDEX	%	2,4	N/P
AS1289.3.4.1	LINEAR SHRINKAGE	%		

CODES USED

Sample history for plasticity tests	
Air Dried	1
Low temperature oven (<50°) dried	2
Other/Unknown	3

Method of preparation for plasticity tests	
Dry sieved	4
Wet sieved	5
Natural	6



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SOIL CLASSIFICATION TEST REPORT

AUSTRALIAN STANDARDS METHODS

CLIENT: SGM Environmental			
CLIENT ADDRESS: PO Box 5622, STAFFORD HEIGHTS QLD 4053		REPORT NO:	EW220912-4
PROJECT: Material Evaluation		PROJECT NO:	EW220912
SITE LOCATION: B2G Soil Samples			
DATE OF TESTING: 29/04/2022		DATE OF REPORT: 6/05/2022	
TECHNICIAN: M.C		DATE SAMPLED: -	

Sampled By Client: The results apply to samples as received

SAMPLE No:		10		
SAMPLE LOCATION:		310-03-AH3286		
DEPTH:		0.0 - 0.1m		
MATERIAL DESCRIPTION:		Sandy CLAY		
WASHED/UNWASHED				
AS1289.3.8.1 Water Type & Temperature:				
TEST METHOD	TEST DESCRIPTION			
AS1289.3.6.1		75.0mm		
		63.0mm		
		53.0mm		
		37.5mm		
		26.5mm		
		19.0mm		
		13.2mm		
		9.5mm		
SAMPLING METHOD	% Passing Sieve Analysis	6.7mm		
		4.75mm		
		2.36mm		
		1.18mm		
		600µm		
		425µm		
		300µm		
		150µm		
AS1289.1.2.1.6.4 AS1289.1.2.1.6.5.1 AS1289.1.2.1.6.5.3 AS1289.1.2.1.6.5.4		75µm		
AS1289.3.8.1	EMERSON CLASS NO:			
AS1289.2.1.1	MOISTURE CONTENT:	%		

CODE				
AS1289.3.1.1	LIQUID LIMIT	%		
AS1289.3.1.2	LIQUID LIMIT	%	2,4	17
AS1289.3.2.1	PLASTIC LIMIT	%	2,4	15
AS1289.3.3.1	PLASTICITY INDEX	%	2,4	2
AS1289.3.4.1	LINEAR SHRINKAGE	%		

CODES USED

Sample history for plasticity tests	
Air Dried	1
Low temperature oven (<50°) dried	2
Other/Unknown	3

Method of preparation for plasticity tests	
Dry sieved	4
Wet sieved	5
Natural	6



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Page 1 of 1

Appendix C

On-site Soil Testing — General Construction Practices
(Catchment and Creeks 2010)



On-site Soil Testing

GENERAL CONSTRUCTION PRACTICE

Various **non-scientific** soil tests can be conducted on a construction site. These are 'indicator' tests that can be used to identify potential soil issues. If the tests indicate a potential problem, then a representative soil sample should be sent away for **official** scientific testing.

Soil texture

A **general soil description** may be inferred from the following soil properties.

Table 1 – Typical indicator properties for various soil textures

Soil texture	Indicator properties
Sandy soil/gravel	A coarse grained soil Soil grains are clearly visible Easy to shovel and break-up when compacted Very difficult to form a clod of soil when compressed in the hand
Sandy loam	Soils have a rough texture in the hand Easy to shovel and break-up when compacted Moist soils may form a clod when compressed in the hand Usually a well drained soil
Clay loam	Soils feel smooth in the hand, but some soil grains can be felt Difficult to break-up when compacted Easy to form a clod when moist soil is compressed in the hand Slightly sticky when wet
Clay	Soils feel very smooth and sticky when wet Very difficult the shovel and break-up when compacted Readily forms a clod when compressed in the hand Usually a poorly drained soil

Soil pH

Soil pH is an indicator of potential revegetation problems. A representative soil sample is formed by mixing at least five samples from holes dug in different part of an area of interest. If the soil being tested is a topsoil, then collect each sample such that it is representative of the top 10cm of soil, but do not include any material that is obviously a subsoil.

After mixing the collective soil sample well, place a level teaspoon of the mixture on a test plate (supplied with the pH test kit). Add drops of the indicator liquid until the sample can be stirred into a thick paste. Then dust the sample with the white powder supplied in the test kit, wait one minute, and read from the colour card the soil pH based on the colour nearest to the sample.



Photo 1 – Soil pH test kit



Photo 2 – Tested soil sample

Acid sulfate soils

Soil pH is **not** a good indicator of potential acid sulfate soils. In in-risk areas (i.e. soils obtained from land with a surface elevation less than 5m AHD), professional soil testing and the adoption of strict soil management procedures are essential.

Soil moisture

Ensuring the correct soil moisture is important when mechanically working a soil. If the soil is too dry it will pulverise the soil, if too wet it may lead to clodding or hardsetting—particularly if the soil has a high silt or clay content. Prior to conducting earth works, the soil should be wet enough to form a clump when squeezed by hand, but not wet enough to drip water while being squeezed.

Settling properties

The 'jar settling test' is primarily used to indicate the settling qualities of a soil, but it can also identify a soil that may be dispersive. The test involves placing a crushed soil sample in a clear jar containing distilled (de-ionised) water. The sealed contents are shaken vigorously to further break-up the soil sample. The jar is then left undisturbed for up to 5 days. If the suspended soil settles leaving the water clear, then there is a low risk that the soil is dispersive (Photo 5).

If the settled soil sample leaves the water 'slightly' cloudy after 5 days, then the soil may be dispersive. If the water remains significantly cloudy after 5 days (Photo 6), then there is a high risk that the soil is dispersive.

The test can also be used as an indicator of the percentage fraction of fine and coarse sediments within a soil sample (Photos 3 & 4). A soil sample that settles very slowly (Photo 4) indicates that the clay may not be dispersive, but instead has a very fine grain size that settles very slowly—these soil may or may not respond favourably to chemical coagulation.



Photo supplied by Catchments & Creeks Pty Ltd

Photo 3 – Separate layers of clay, silt and sand visible in settled sample



Photo supplied by Catchments & Creeks Pty Ltd

Photo 4 – Slow settlement after 1-day indicates a very fine clay



Photo supplied by Catchments & Creeks Pty Ltd

Photo 5 – Good settlement indicates soil is unlikely dispersive



Photo supplied by Catchments & Creeks Pty Ltd

Photo 6 – Poor settlement indicates a high risk of dispersive properties

Soil dispersion

There are two on-site tests that can identify if dispersive soils may be present. The most common and reliable test is the *Aggregate Immersion Test*. The other less reliable test is the *Jar Settling Test* (discussed above).

The testing procedure involves filling a dish or jar with distilled water to a depth sufficient to cover the soil samples. Several dry, hard clumps of soil (about 5mm square) are gently placed in the water (don't put the soil in the dish before the water is added). The sample is then observed after being left undisturbed for an hour.

Non-dispersive: Water remains clear though particles may slightly collapse. The boundary of clumps remains clearly defined (Photo 7).

Slightly dispersive: Discolouration surrounding particles or distinct cloudiness surrounding some. Boundary of clumps vaguely defined (Photo 8).

Dispersive: Dispersive and cloudiness surround most or all particles (extending vertically). Boundary of clumps not able to be defined.

Highly dispersive: Discolouration extending vertically throughout most or all water. Highly dispersive or slaking soils will collapse in less than 10 minutes. A cloudy ring will be seen around the collapsed soil when it is dispersive (Photo 9).

Slightly slaking: Water remains clear. Boundary of clumps vaguely defined.

Slaking: Water remains clear. Boundary of clumps not able to be defined. The clumps completely collapse and spread horizontally (Photo 10).

If the soil sample collapses as air rushes out of the sample, then this does not necessarily indicate a slaking soil. Such a response is typical when testing disturbed, poorly compacted soil clumps. What is more important is what happens to the clumps **after** all the air has escaped.

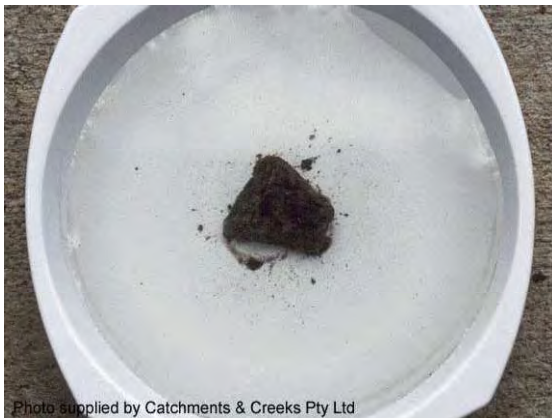


Photo 7 – Typical response of a non slaking, non dispersive soil



Photo 8 – Typical response of a slightly dispersive soil



Photo 9 – Typical response of a dispersive or highly dispersive soil



Photo 10 – Typical response of a slaking soil