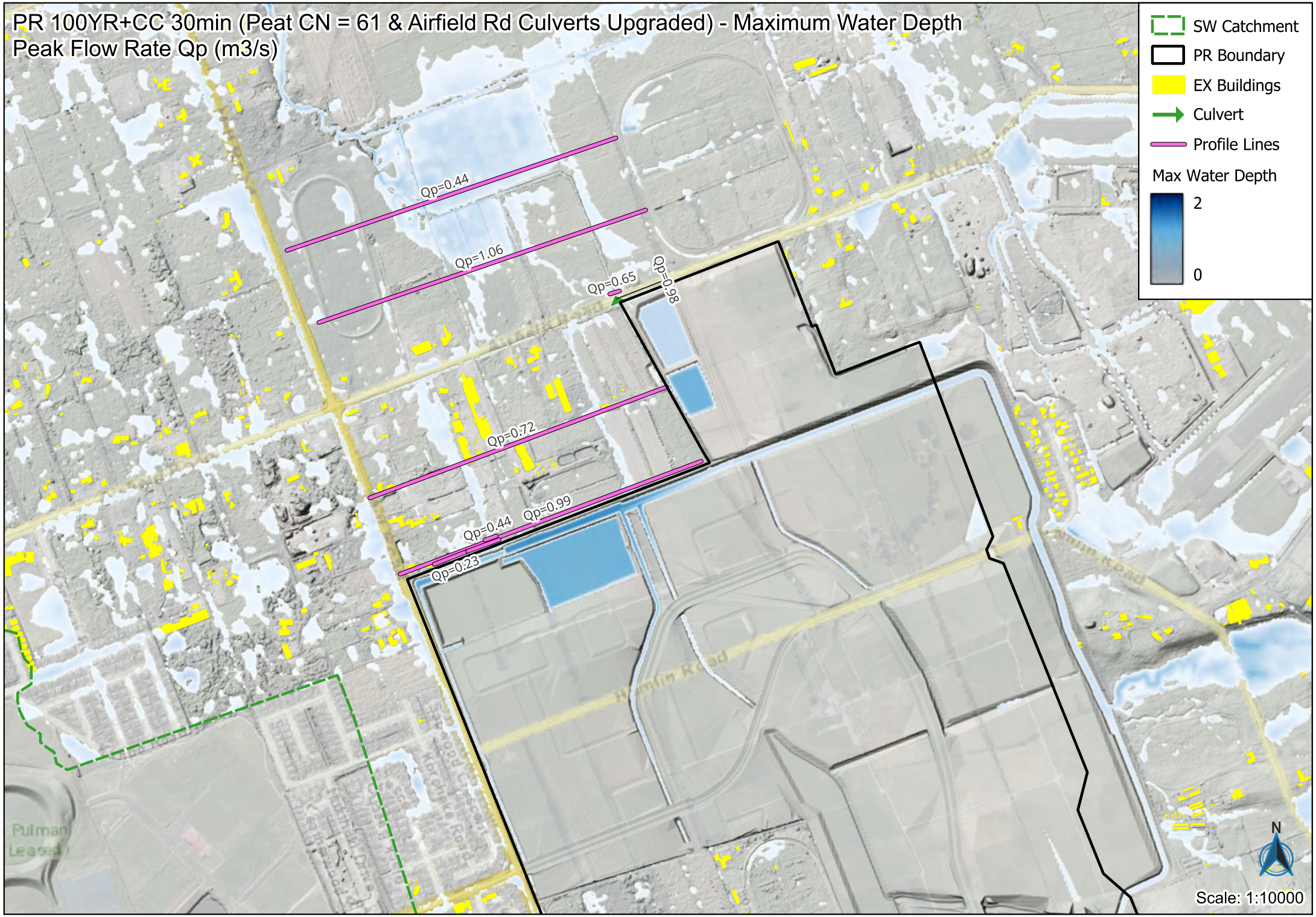
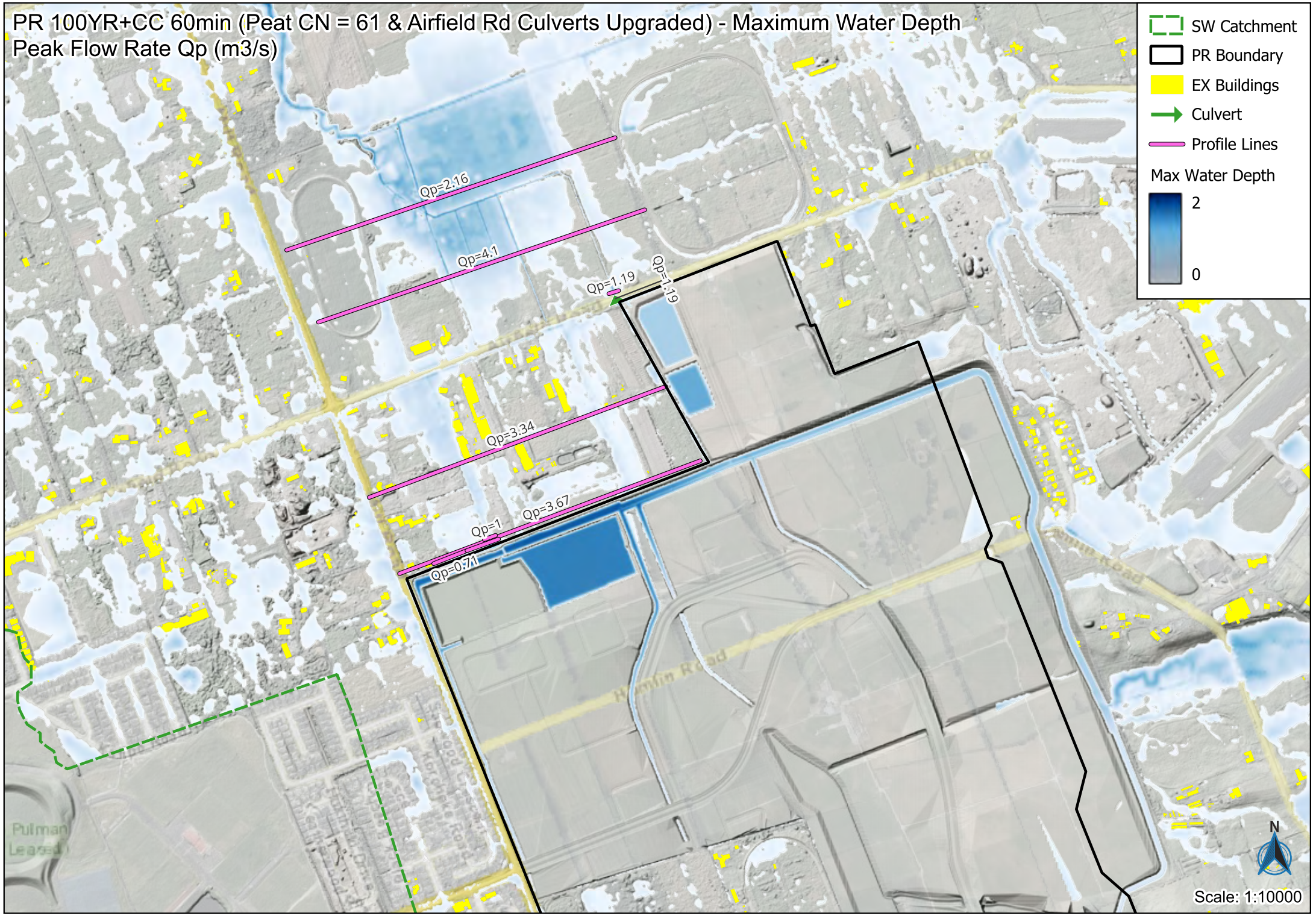
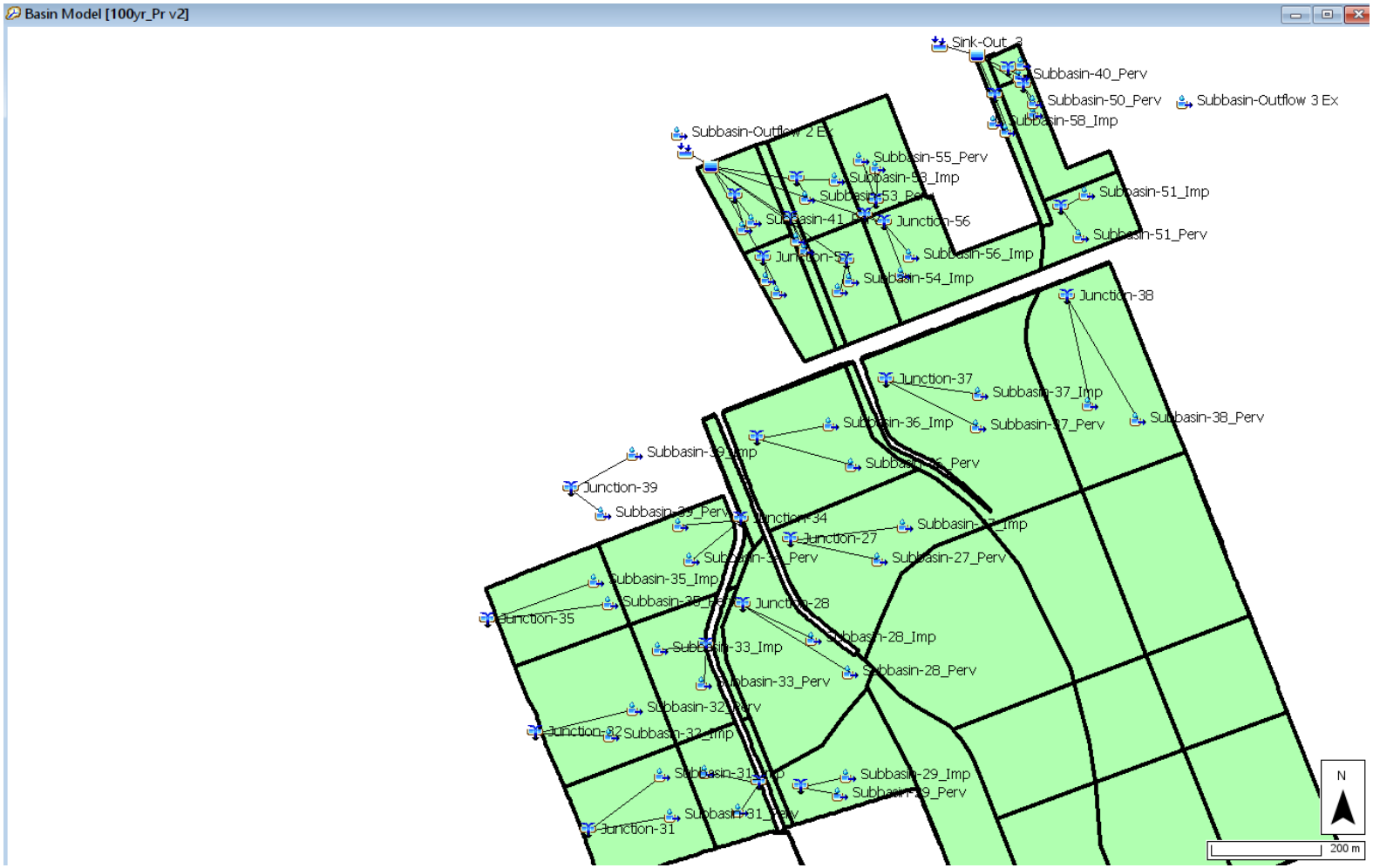

PR 100YR+CC 30min (Peat CN = 61 & Airfield Rd Culverts Upgraded) - Maximum Water Depth
Peak Flow Rate Qp (m3/s)
SW Catchment
PR Boundary
EX Buildings
Culvert
Profile Lines
Max Water Depth
2
0
Qp=0.44
Qp=1.06
Qp=0.65
Qp=0.98
Qp=0.72
Qp=0.44
Qp=0.99
Qp=0.23
Hamlin Road
N
Scale: 1:10000


PR 100YR+CC 60min (Peat CN = 61 & Airfield Rd Culverts Upgraded) - Maximum Water Depth
Peak Flow Rate Qp (m3/s)
SW Catchment
PR Boundary
EX Buildings
Culvert
Profile Lines
Max Water Depth
2
0
Qp=2.16
Qp=4.1
Qp=1.19
Qp=1.19
Qp=3.34
Qp=3.67
Qp=1
Qp=0.71
Hamlin Road
N
Scale: 1:10000

APPENDIX 8 – HMS EASTERN INFLOW HYDROGRAPHS

Eastern Catchment Proposed Scenario– HMS model setup for Inflow hydrograph



Eastern Catchment Proposed Scenario– HMS Inflow hydrograph summary 2yr Storm

Project: FAB_Swale_Sizing Simulation Run: 2yr_FAB v2

Start of Run: 01Jan2000, 00:00
End of Run: 03Jan2000, 00:00
Compute Time: DATA CHANGED, RECOMPUTE

Basin Model: 2yr_Pr v2
Meteorologic Model: TP108_2yr_86mm
Control Specifications: 48hr

Show Elements: All Elements Volume Units: ○ MM ◉ 1000 M3 Sorting: Alphabetic

Hydrologic Element	Drainage Area (KM2)	Peak Discharge (M3/S)	Time of Peak	Volume (1000 M3)
Junction-27	0.06420	0.65636	1 January 2000, 12:16	4.16871
Junction-28	0.06297	0.70676	1 January 2000, 12:14	4.08864
Junction-29	0.03433	0.41057	1 January 2000, 12:13	2.22892
Junction-30	0.03111	0.37784	1 January 2000, 12:12	2.02031
Junction-31	0.04699	0.53929	1 January 2000, 12:13	3.05104
Junction-32	0.05554	0.62268	1 January 2000, 12:14	3.60631
Junction-33	0.03631	0.41885	1 January 2000, 12:13	2.35798
Junction-34	0.04128	0.46899	1 January 2000, 12:14	2.68062
Junction-35	0.03693	0.43006	1 January 2000, 12:13	2.39777
Junction-36	0.06601	0.70589	1 January 2000, 12:15	4.28632
Junction-37	0.10365	1.27654	1 January 2000, 12:17	7.96574
Junction-38	0.07211	0.96316	1 January 2000, 12:14	5.54229
Junction-39	0.26627	1.06932	1 January 2000, 12:49	13.66252
Junction-40	0.00353	0.02959	1 January 2000, 12:12	0.13637
Junction-41	0.01849	0.12701	1 January 2000, 12:16	0.71343
Junction-50	0.01729	0.24395	1 January 2000, 12:13	1.32914
Junction-51	0.01933	0.29190	1 January 2000, 12:12	1.48554
Junction-52	0.00799	0.11110	1 January 2000, 12:13	0.59688
Junction-53	0.02187	0.32847	1 January 2000, 12:12	1.68102
Junction-54	0.02226	0.33106	1 January 2000, 12:12	1.71057
Junction-55	0.02434	0.36321	1 January 2000, 12:12	1.87081
Junction-56	0.04020	0.52847	1 January 2000, 12:15	3.08960
Junction-57	0.01759	0.26408	1 January 2000, 12:12	1.35148
Junction-58	0.00673	0.09327	1 January 2000, 12:13	0.50259
Reservoir-Swale_Storage	0.65143	1.24773	1 January 2000, 13:14	44.14202
Reservoir-SW_Pond_2	0.15274	0.05980	1 January 2000, 18:15	7.03891
Reservoir-SW_Pond_3	0.02755	0.07489	1 January 2000, 13:03	1.68934
Sink-Outflow_1	0.65143	1.24773	1 January 2000, 13:14	44.14202
Sink-Outflow_2	0.15274	0.05980	1 January 2000, 18:15	7.03891
Sink-Out_3	0.02755	0.07489	1 January 2000, 13:03	1.68934
Subbasin-Outflow 2 Ex	0.15274	0.82457	1 January 2000, 12:25	5.88644
Subbasin-Outflow 3 Ex	0.02755	0.17724	1 January 2000, 12:18	1.06193
Subbasin-27_Imp	0.03980	0.54490	1 January 2000, 12:15	3.22852
Subbasin-27_Perv	0.02440	0.13368	1 January 2000, 12:24	0.94019
Subbasin-28_Imp	0.03904	0.58008	1 January 2000, 12:13	3.16651
Subbasin-28_Perv	0.02393	0.14677	1 January 2000, 12:20	0.92213
Subbasin-29_Imp	0.02128	0.33416	1 January 2000, 12:12	1.72622
Subbasin-29_Perv	0.01304	0.08734	1 January 2000, 12:17	0.50270
Subbasin-30_Imp	0.01929	0.30585	1 January 2000, 12:12	1.56466
Subbasin-30_Perv	0.01182	0.08114	1 January 2000, 12:16	0.45565
Subbasin-31_Imp	0.02913	0.44223	1 January 2000, 12:13	2.36293
Subbasin-31_Perv	0.01786	0.11292	1 January 2000, 12:19	0.68812

Project: FAB_Swale_Sizing Simulation Run: 2yr_FAB v2

Start of Run: 01Jan2000, 00:00
End of Run: 03Jan2000, 00:00
Compute Time: DATA CHANGED, RECOMPUTE

Basin Model: 2yr_Pr v2
Meteorologic Model: TP108_2yr_86mm
Control Specifications: 48hr

Show Elements: All Elements Volume Units: ○ MM ◉ 1000 M3 Sorting: Alphabetic

Hydrologic Element	Drainage Area (KM2)	Peak Discharge (M3/S)	Time of Peak	Volume (1000 M3)
Subbasin-32_Imp	0.03443	0.51165	1 January 2000, 12:13	2.79296
Subbasin-32_Perv	0.02110	0.12911	1 January 2000, 12:20	0.81335
Subbasin-33_Imp	0.02251	0.34283	1 January 2000, 12:13	1.82617
Subbasin-33_Perv	0.01380	0.08786	1 January 2000, 12:18	0.53181
Subbasin-34_Imp	0.02560	0.38465	1 January 2000, 12:13	2.07604
Subbasin-34_Perv	0.01569	0.09806	1 January 2000, 12:19	0.60457
Subbasin-35_Imp	0.02289	0.35065	1 January 2000, 12:13	1.85699
Subbasin-35_Perv	0.01403	0.09055	1 January 2000, 12:18	0.54078
Subbasin-36_Imp	0.04093	0.58353	1 January 2000, 12:14	3.31961
Subbasin-36_Perv	0.02508	0.14468	1 January 2000, 12:22	0.96672
Subbasin-37_Imp	0.09328	1.23271	1 January 2000, 12:16	7.56629
Subbasin-37_Perv	0.01036	0.05418	1 January 2000, 12:26	0.39945
Subbasin-38_Imp	0.06490	0.92911	1 January 2000, 12:14	5.26436
Subbasin-38_Perv	0.00721	0.04181	1 January 2000, 12:22	0.27792
Subbasin-39_Imp	0.07988	0.64850	1 January 2000, 12:40	6.47924
Subbasin-39_Perv	0.18639	0.54936	1 January 2000, 13:13	7.18328
Subbasin-40_Imp	0.00000	0.00006	1 January 2000, 12:10	0.00029
Subbasin-40_Perv	0.00353	0.02953	1 January 2000, 12:12	0.13608
Subbasin-41_Imp	0.00002	0.00029	1 January 2000, 12:12	0.00150
Subbasin-41_Perv	0.01847	0.12677	1 January 2000, 12:16	0.71193
Subbasin-50_Imp	0.01557	0.23473	1 January 2000, 12:13	1.26249
Subbasin-50_Perv	0.00173	0.01084	1 January 2000, 12:19	0.06665
Subbasin-51_Imp	0.01740	0.27977	1 January 2000, 12:11	1.41105
Subbasin-51_Perv	0.00193	0.01358	1 January 2000, 12:15	0.07449
Subbasin-52_Imp	0.00679	0.10427	1 January 2000, 12:13	0.55070
Subbasin-52_Perv	0.00120	0.00776	1 January 2000, 12:18	0.04617
Subbasin-53_Imp	0.01969	0.31476	1 January 2000, 12:12	1.59673
Subbasin-53_Perv	0.00219	0.01523	1 January 2000, 12:16	0.08430
Subbasin-54_Imp	0.02003	0.31760	1 January 2000, 12:12	1.62479
Subbasin-54_Perv	0.00223	0.01522	1 January 2000, 12:16	0.08578
Subbasin-55_Imp	0.02191	0.34839	1 January 2000, 12:12	1.77700
Subbasin-55_Perv	0.00243	0.01670	1 January 2000, 12:16	0.09381
Subbasin-56_Imp	0.03618	0.50956	1 January 2000, 12:15	2.93467
Subbasin-56_Perv	0.00402	0.02275	1 January 2000, 12:23	0.15493
Subbasin-57_Imp	0.01583	0.25306	1 January 2000, 12:12	1.28371
Subbasin-57_Perv	0.00176	0.01224	1 January 2000, 12:16	0.06777
Subbasin-58_Imp	0.00572	0.08756	1 January 2000, 12:13	0.46371
Subbasin-58_Perv	0.00101	0.00651	1 January 2000, 12:18	0.03888

Eastern Catchment Proposed Scenario – HMS Inflow hydrograph summary 10yr Storm

Project: FAB_Swale_Sizing Simulation Run: 10yr_FAB v2

Start of Run: 01Jan2000, 00:00 Basin Model: 10yr_Pr v2
End of Run: 03Jan2000, 00:00 Meteorologic Model: TP108_10yr_170mm_CoPv4
Compute Time: 24Jan2025, 20:51:53 Control Specifications: 48hr

Show Elements: All Elements Volume Units: ○ MM ◉ 1000 M3 Sorting: Alphabetic

Hydrologic Element	Drainage Area (KM2)	Peak Discharge (M3/S)	Time of Peak	Volume (1000 M3)
Junction-27	0.06374	1.40175	1 January 2000, 12:16	9.11297
Junction-28	0.06002	1.45357	1 January 2000, 12:14	8.58151
Junction-29	0.03304	0.85538	1 January 2000, 12:13	4.72400
Junction-30	0.02764	0.72596	1 January 2000, 12:12	3.95179
Junction-31	0.04300	1.06547	1 January 2000, 12:14	6.14812
Junction-32	0.05227	1.26401	1 January 2000, 12:14	7.47309
Junction-33	0.03284	0.81733	1 January 2000, 12:13	4.69582
Junction-34	0.03798	0.93207	1 January 2000, 12:14	5.42957
Junction-35	0.03445	0.86639	1 January 2000, 12:13	4.92561
Junction-36	0.06341	1.46027	1 January 2000, 12:15	9.06586
Junction-37	0.10002	2.49703	1 January 2000, 12:17	15.92133
Junction-38	0.07191	1.94717	1 January 2000, 12:14	11.44670
Junction-39	0.26627	2.57331	1 January 2000, 12:52	33.13708
Junction-41	0.01849	0.35950	1 January 2000, 12:16	1.98117
Junction-50	0.01729	0.49487	1 January 2000, 12:13	2.75294
Junction-52	0.00799	0.22763	1 January 2000, 12:13	1.24836
Junction-53	0.02187	0.66694	1 January 2000, 12:12	3.48177
Junction-54	0.02226	0.67202	1 January 2000, 12:12	3.54296
Junction-55	0.02434	0.73730	1 January 2000, 12:12	3.87485
Junction-55_56_Culv...	0.06454	1.76968	1 January 2000, 12:13	10.27408
Junction-56	0.04020	1.07148	1 January 2000, 12:15	6.39923
Junction-57	0.01759	0.53619	1 January 2000, 12:12	2.79921
Reservoir-Swale_Stor...	0.88660	16.04138	1 January 2000, 12:16	124.17540
Reservoir-SW_Pond_2	0.15274	0.63895	1 January 2000, 13:10	17.37989
Sink-Outflow_1	0.88660	16.04138	1 January 2000, 12:16	124.17540
Sink-Outflow_2	0.15274	0.63895	1 January 2000, 13:10	17.37989
Subbasin-27_Imp	0.03952	1.07932	1 January 2000, 12:15	6.51934
Subbasin-27_Perv	0.02422	0.37727	1 January 2000, 12:23	2.59363
Subbasin-28_Imp	0.03721	1.10292	1 January 2000, 12:13	6.13913
Subbasin-28_Perv	0.02281	0.39732	1 January 2000, 12:19	2.44238
Subbasin-29_Imp	0.02049	0.64139	1 January 2000, 12:12	3.37950
Subbasin-29_Perv	0.01256	0.23799	1 January 2000, 12:17	1.34450
Subbasin-30_Imp	0.01714	0.54174	1 January 2000, 12:12	2.82707
Subbasin-30_Perv	0.01050	0.20414	1 January 2000, 12:16	1.12472
Subbasin-31_Imp	0.02666	0.80710	1 January 2000, 12:13	4.39831
Subbasin-31_Perv	0.01634	0.29336	1 January 2000, 12:18	1.74981
Subbasin-32_Imp	0.03241	0.96046	1 January 2000, 12:13	5.34617
Subbasin-32_Perv	0.01986	0.34493	1 January 2000, 12:19	2.12692
Subbasin-33_Imp	0.02036	0.61833	1 January 2000, 12:13	3.35934
Subbasin-33_Perv	0.01248	0.22551	1 January 2000, 12:18	1.33648
Subbasin-34_Imp	0.02355	0.70570	1 January 2000, 12:13	3.88426
Subbasin-34_Perv	0.01443	0.25578	1 January 2000, 12:19	1.54531

Project: FAB_Swale_Sizing Simulation Run: 10yr_FAB v2

Start of Run: 01Jan2000, 00:00
End of Run: 03Jan2000, 00:00
Compute Time: 24Jan2025, 20:51:53

Basin Model: 10yr_Pr v2
Meteorologic Model: TP108_10yr_170mm_CoPv4
Control Specifications: 48hr

Show Elements: All Elements Volume Units: ○ MM ◉ 1000 M3 Sorting: Alphabetic

Hydrologic Element	Drainage Area (KM2)	Peak Discharge (M3/S)	Time of Peak	Volume (1000 M3)
Subbasin-35_Imp	0.02136	0.65233	1 January 2000, 12:13	3.52374
Subbasin-35_Perv	0.01309	0.23940	1 January 2000, 12:18	1.40187
Subbasin-36_Imp	0.03931	1.11818	1 January 2000, 12:14	6.48564
Subbasin-36_Perv	0.02410	0.39476	1 January 2000, 12:21	2.58023
Subbasin-37_Imp	0.09002	2.37344	1 January 2000, 12:16	14.85029
Subbasin-37_Perv	0.01000	0.14868	1 January 2000, 12:25	1.07104
Subbasin-38_Imp	0.06472	1.84809	1 January 2000, 12:14	10.67667
Subbasin-38_Perv	0.00719	0.11847	1 January 2000, 12:21	0.77003
Subbasin-39_Imp	0.07988	1.29502	1 January 2000, 12:40	13.17798
Subbasin-39_Perv	0.18639	1.56246	1 January 2000, 13:11	19.95911
Subbasin-40_Imp	0.00000	0.00012	1 January 2000, 12:10	0.00058
Subbasin-40_Perv	0.00353	0.08278	1 January 2000, 12:11	0.37811
Subbasin-41_Imp	0.00002	0.00059	1 January 2000, 12:12	0.00305
Subbasin-41_Perv	0.01847	0.35904	1 January 2000, 12:16	1.97812
Subbasin-50_Imp	0.01557	0.46814	1 January 2000, 12:13	2.56775
Subbasin-50_Perv	0.00173	0.03074	1 January 2000, 12:18	0.18519
Subbasin-52_Imp	0.00679	0.20791	1 January 2000, 12:13	1.12006
Subbasin-52_Perv	0.00120	0.02197	1 January 2000, 12:18	0.12830
Subbasin-53_Imp	0.01969	0.62752	1 January 2000, 12:12	3.24754
Subbasin-53_Perv	0.00219	0.04305	1 January 2000, 12:16	0.23422
Subbasin-54_Imp	0.02003	0.63325	1 January 2000, 12:12	3.30462
Subbasin-54_Perv	0.00223	0.04311	1 January 2000, 12:16	0.23833
Subbasin-55_Imp	0.02191	0.69461	1 January 2000, 12:12	3.61419
Subbasin-55_Perv	0.00243	0.04731	1 January 2000, 12:16	0.26066
Subbasin-56_Imp	0.03618	1.01637	1 January 2000, 12:15	5.96875
Subbasin-56_Perv	0.00402	0.06470	1 January 2000, 12:22	0.43048
Subbasin-57_Imp	0.01583	0.50451	1 January 2000, 12:12	2.61091
Subbasin-57_Perv	0.00176	0.03461	1 January 2000, 12:16	0.18830
Subbasin-58_Imp	0.00572	0.17460	1 January 2000, 12:13	0.94313
Subbasin-58_Perv	0.00101	0.01845	1 January 2000, 12:18	0.10804

Eastern Catchment Proposed Scenario – HMS Inflow hydrograph summary 100yr Storm

Project: FAB_Swale_Sizing Simulation Run: 100yr_FAB v2

Start of Run: 01Jan2000, 00:00 Basin Model: 100yr_Pr v2
End of Run: 03Jan2000, 00:00 Meteorologic Model: TP108_100yr_298mm
Compute Time: 24Jan2025, 20:52:13 Control Specifications: 48hr

Show Elements: All Elements Volume Units: ○ MM ◉ 1000 M3 Sorting: Alphabetic

Hydrologic Element	Drainage Area (KM2)	Peak Discharge (M3/S)	Time of Peak	Volume (1000 M3)
Junction-27	0.06374	2.44536	1 January 2000, 12:20	17.01497
Junction-28	0.06002	2.56700	1 January 2000, 12:17	16.02267
Junction-29	0.03304	1.52967	1 January 2000, 12:15	8.82025
Junction-30	0.02764	1.30844	1 January 2000, 12:14	7.37845
Junction-31	0.04300	1.89389	1 January 2000, 12:16	11.47925
Junction-32	0.05227	2.23305	1 January 2000, 12:17	13.95312
Junction-33	0.03284	1.45642	1 January 2000, 12:16	8.76764
Junction-34	0.03798	1.65293	1 January 2000, 12:16	10.13763
Junction-35	0.03445	1.54495	1 January 2000, 12:15	9.19667
Junction-36	0.06341	2.55474	1 January 2000, 12:18	16.92701
Junction-37	0.10002	4.16941	1 January 2000, 12:20	28.61318
Junction-38	0.07191	3.29250	1 January 2000, 12:17	20.57156
Junction-39	0.26627	4.66047	1 January 2000, 13:08	65.25941
Junction-40	0.00353	0.17270	1 January 2000, 12:12	0.79407
Junction-41	0.01849	0.72248	1 January 2000, 12:18	4.15429
Junction-50	0.01729	0.84576	1 January 2000, 12:15	4.94748
Junction-51	0.01933	1.03749	1 January 2000, 12:13	5.52964
Junction-52	0.00799	0.39422	1 January 2000, 12:15	2.25776
Junction-53	0.02187	1.16086	1 January 2000, 12:13	6.25729
Junction-54	0.02226	1.16768	1 January 2000, 12:14	6.36726
Junction-55	0.02434	1.28036	1 January 2000, 12:14	6.96372
Junction-55_56_Culvert	0.06454	2.99123	1 January 2000, 12:15	18.46417
Junction-56	0.04020	1.80127	1 January 2000, 12:18	11.50045
Junction-57	0.01759	0.93641	1 January 2000, 12:13	5.03064
Junction-58	0.00673	0.33098	1 January 2000, 12:15	1.90113
Reservoir-SW_Pond_2	0.15274	4.14206	1 January 2000, 12:28	33.62547
Reservoir-SW_Pond_3	0.02755	0.98821	1 January 2000, 12:23	7.24933
Sink-Outflow_2	0.15274	4.14206	1 January 2000, 12:28	33.62547
Sink-Out_3	0.02755	0.98821	1 January 2000, 12:23	7.24933
Subbasin-Outflow 2 Ex	0.15274	4.65679	1 January 2000, 12:28	34.30427
Subbasin-Outflow 3 Ex	0.02755	1.00483	1 January 2000, 12:20	6.18860
Subbasin-27_Imp	0.03952	1.79219	1 January 2000, 12:18	11.57514
Subbasin-27_Perv	0.02422	0.74896	1 January 2000, 12:28	5.43983
Subbasin-28_Imp	0.03721	1.86195	1 January 2000, 12:16	10.90008
Subbasin-28_Perv	0.02281	0.79248	1 January 2000, 12:22	5.12260
Subbasin-29_Imp	0.02049	1.09726	1 January 2000, 12:14	6.00033
Subbasin-29_Perv	0.01256	0.47768	1 January 2000, 12:19	2.81992
Subbasin-30_Imp	0.01714	0.93200	1 January 2000, 12:13	5.01948
Subbasin-30_Perv	0.01050	0.41031	1 January 2000, 12:18	2.35896
Subbasin-31_Imp	0.02666	1.36656	1 January 2000, 12:15	7.80923
Subbasin-31_Perv	0.01634	0.58695	1 January 2000, 12:21	3.67002

Project: FAB_Swale_Sizing Simulation Run: 100yr_FAB v2

Start of Run: 01Jan2000, 00:00 Basin Model: 100yr_Pr v2
End of Run: 03Jan2000, 00:00 Meteorologic Model: TP108_100yr_298mm
Compute Time: 24Jan2025, 20:52:13 Control Specifications: 48hr

Show Elements: All Elements Volume Units: ○ MM ◉ 1000 M3 Sorting: Alphabetic

Hydrologic Element	Drainage Area (KM2)	Peak Discharge (M3/S)	Time of Peak	Volume (1000 M3)
Subbasin-31_Perv	0.01634	0.58695	1 January 2000, 12:21	3.67002
Subbasin-32_Imp	0.03241	1.61675	1 January 2000, 12:16	9.49217
Subbasin-32_Perv	0.01986	0.69012	1 January 2000, 12:22	4.46095
Subbasin-33_Imp	0.02036	1.05068	1 January 2000, 12:15	5.96454
Subbasin-33_Perv	0.01248	0.45178	1 January 2000, 12:21	2.80310
Subbasin-34_Imp	0.02355	1.19395	1 January 2000, 12:15	6.89654
Subbasin-34_Perv	0.01443	0.51143	1 January 2000, 12:22	3.24110
Subbasin-35_Imp	0.02136	1.11193	1 January 2000, 12:15	6.25642
Subbasin-35_Perv	0.01309	0.48020	1 January 2000, 12:20	2.94025
Subbasin-36_Imp	0.03931	1.86492	1 January 2000, 12:17	11.51529
Subbasin-36_Perv	0.02410	0.78603	1 January 2000, 12:25	5.41172
Subbasin-37_Imp	0.09002	3.92267	1 January 2000, 12:20	26.36681
Subbasin-37_Perv	0.01000	0.29451	1 January 2000, 12:30	2.24638
Subbasin-38_Imp	0.06472	3.08930	1 January 2000, 12:17	18.95652
Subbasin-38_Perv	0.00719	0.23613	1 January 2000, 12:25	1.61505
Subbasin-39_Imp	0.07988	2.04910	1 January 2000, 12:52	23.39760
Subbasin-39_Perv	0.18639	3.00143	1 January 2000, 13:26	41.86181
Subbasin-40_Imp	0.00000	0.00022	1 January 2000, 12:10	0.00104
Subbasin-40_Perv	0.00353	0.17249	1 January 2000, 12:12	0.79304
Subbasin-41_Imp	0.00002	0.00101	1 January 2000, 12:13	0.00542
Subbasin-41_Perv	0.01847	0.72164	1 January 2000, 12:18	4.14888
Subbasin-50_Imp	0.01557	0.79219	1 January 2000, 12:15	4.55907
Subbasin-50_Perv	0.00173	0.06145	1 January 2000, 12:21	0.38841
Subbasin-51_Imp	0.01740	0.96717	1 January 2000, 12:13	5.09552
Subbasin-51_Perv	0.00193	0.07761	1 January 2000, 12:17	0.43412
Subbasin-52_Imp	0.00679	0.35438	1 January 2000, 12:15	1.98868
Subbasin-52_Perv	0.00120	0.04407	1 January 2000, 12:20	0.26908
Subbasin-53_Imp	0.01969	1.08318	1 January 2000, 12:13	5.76604
Subbasin-53_Perv	0.00219	0.08675	1 January 2000, 12:18	0.49125
Subbasin-54_Imp	0.02003	1.08944	1 January 2000, 12:13	5.86738
Subbasin-54_Perv	0.00223	0.08667	1 January 2000, 12:18	0.49988
Subbasin-55_Imp	0.02191	1.19631	1 January 2000, 12:13	6.41702
Subbasin-55_Perv	0.00243	0.09509	1 January 2000, 12:18	0.54670
Subbasin-56_Imp	0.03618	1.68947	1 January 2000, 12:18	10.59756
Subbasin-56_Perv	0.00402	0.12885	1 January 2000, 12:26	0.90288
Subbasin-57_Imp	0.01583	0.87396	1 January 2000, 12:13	4.63569
Subbasin-57_Perv	0.00176	0.06974	1 January 2000, 12:18	0.39495
Subbasin-58_Imp	0.00572	0.29761	1 January 2000, 12:15	1.67454
Subbasin-58_Perv	0.00101	0.03701	1 January 2000, 12:20	0.22659

Stormwater Pond 2 Sizing

PWL = 19.40 mRL

Outlets

SMAF Outlet = ø180mm

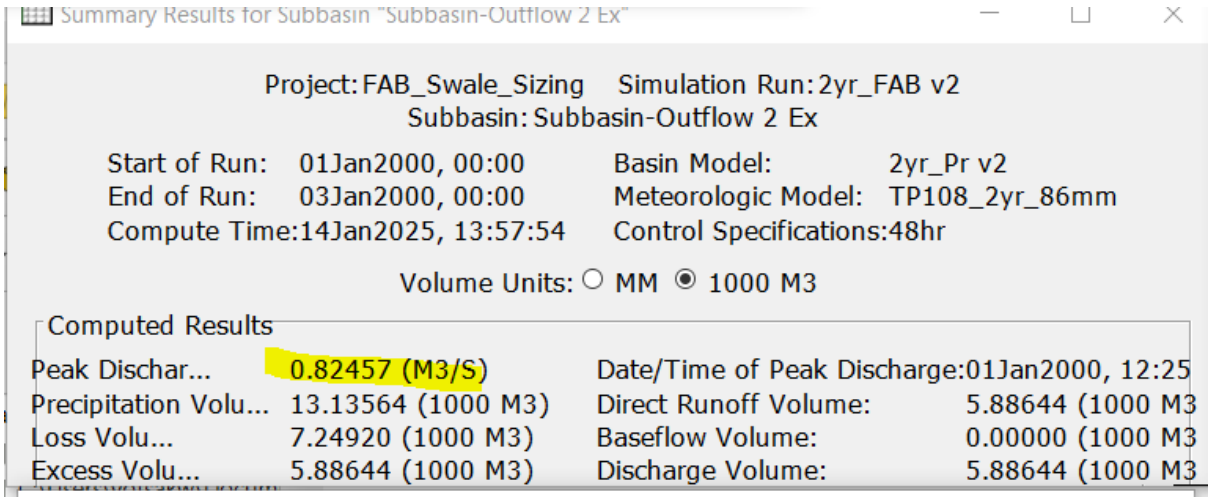
2yr/10yr Outlet = 2m cutout in DN1200 Scruffy dome @ 20.40 mRL

100yr Spillway -

100yr Emergency Spillway @ 21.50 mRL 70m Long

2yr

Ex



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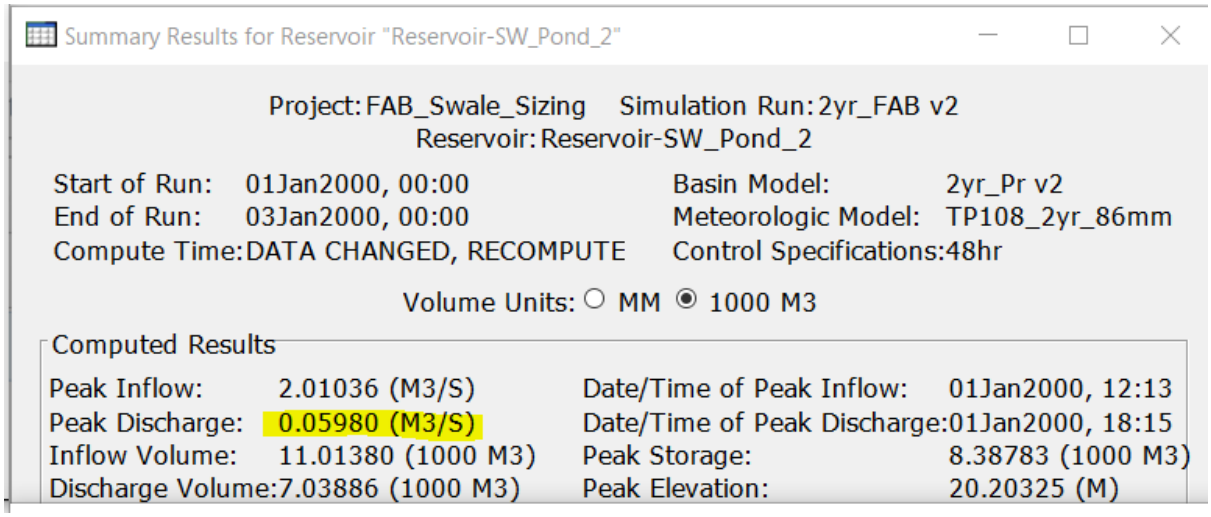
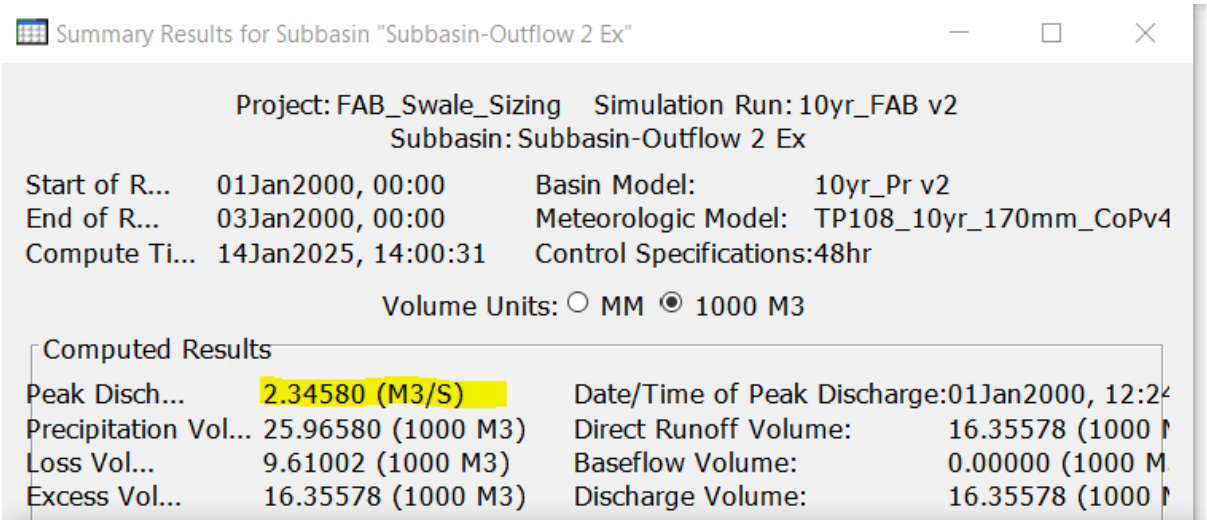
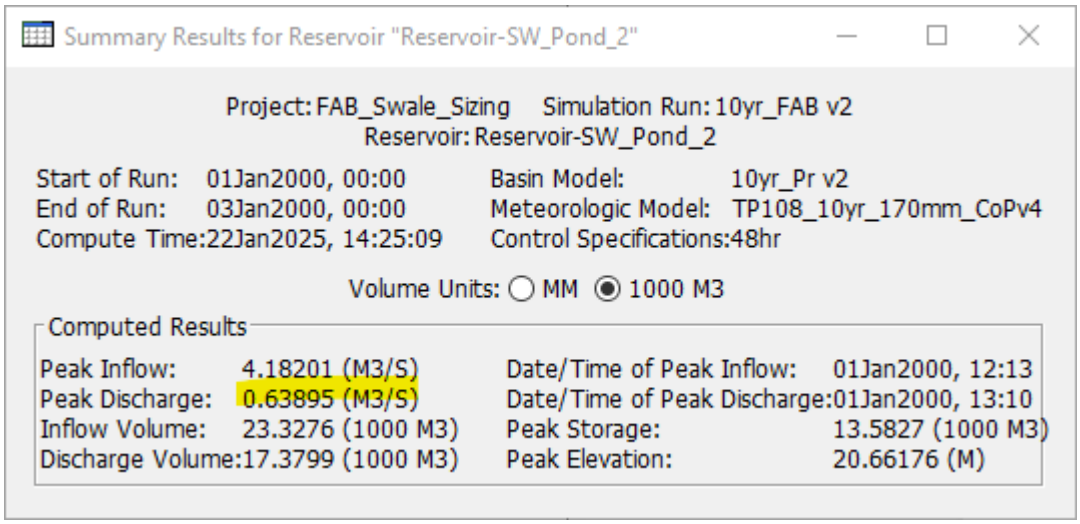


10yr

Ex

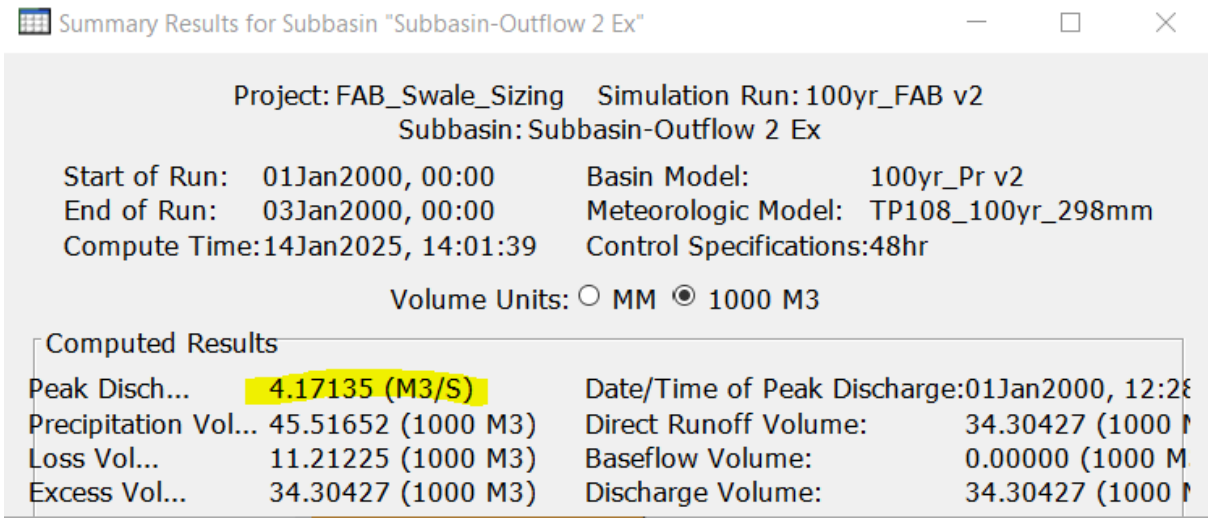

Summary Results for Subbasin "Subbasin-Outflow 2 Ex"
Project: FAB_Swale_Sizing Simulation Run: 10yr_FAB v2
Subbasin: Subbasin-Outflow 2 Ex
Start of R... 01Jan2000, 00:00
End of R... 03Jan2000, 00:00
Compute Ti... 14Jan2025, 14:00:31
Basin Model: 10yr_Pr v2
Meteorologic Model: TP108_10yr_170mm_CoPv4
Control Specifications:48hr
Volume Units: MM 1000 M3
Computed Results
Peak Disch... 2.34580 (M3/S)
Precipitation Vol... 25.96580 (1000 M3)
Loss Vol... 9.61002 (1000 M3)
Excess Vol... 16.35578 (1000 M3)
Date/Time of Peak Discharge:01Jan2000, 12:2
Direct Runoff Volume: 16.35578 (1000
Baseflow Volume: 0.00000 (1000 M
Discharge Volume: 16.35578 (1000

Pr

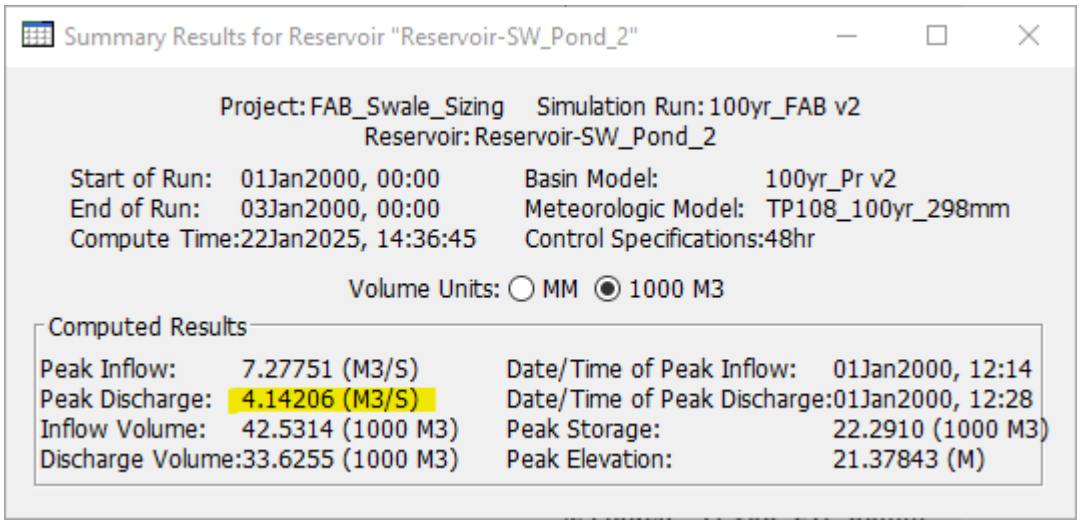

Summary Results for Reservoir "Reservoir-SW_Pond_2"
Project: FAB_Swale_Sizing Simulation Run: 10yr_FAB v2
Reservoir: Reservoir-SW_Pond_2
Start of Run: 01Jan2000, 00:00
End of Run: 03Jan2000, 00:00
Compute Time:22Jan2025, 14:25:09
Basin Model: 10yr_Pr v2
Meteorologic Model: TP108_10yr_170mm_CoPv4
Control Specifications:48hr
Volume Units: MM 1000 M3
Computed Results
Peak Inflow: 4.18201 (M3/S)
Peak Discharge: 0.63895 (M3/S)
Inflow Volume: 23.3276 (1000 M3)
Discharge Volume:17.3799 (1000 M3)
Date/Time of Peak Inflow: 01Jan2000, 12:13
Date/Time of Peak Discharge:01Jan2000, 13:10
Peak Storage: 13.5827 (1000 M3)
Peak Elevation: 20.66176 (M)

100yr

Ex

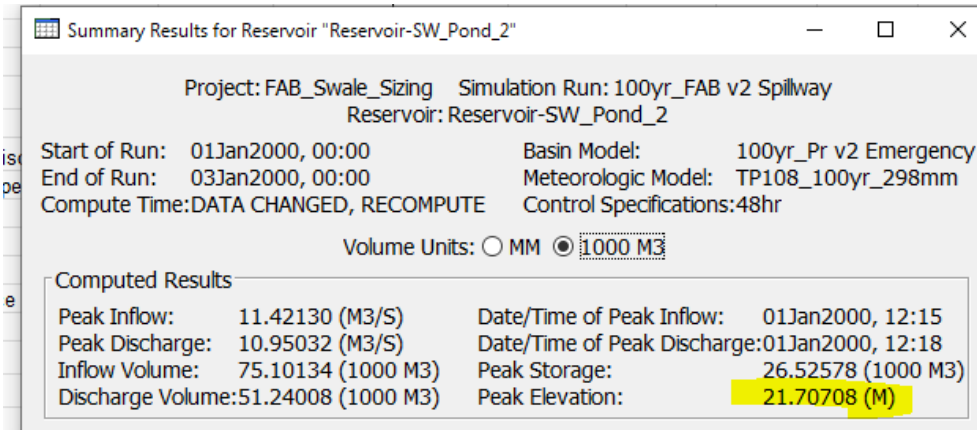

Summary Results for Subbasin "Subbasin-Outflow 2 Ex"
Project: FAB_Swale_Sizing Simulation Run: 100yr_FAB v2
Subbasin: Subbasin-Outflow 2 Ex
Start of Run: 01Jan2000, 00:00
End of Run: 03Jan2000, 00:00
Compute Time: 14Jan2025, 14:01:39
Basin Model: 100yr_Pr v2
Meteorologic Model: TP108_100yr_298mm
Control Specifications: 48hr
Volume Units: MM 1000 M3
Computed Results
Peak Disch... 4.17135 (M3/S)
Precipitation Vol... 45.51652 (1000 M3)
Loss Vol... 11.21225 (1000 M3)
Excess Vol... 34.30427 (1000 M3)
Date/Time of Peak Discharge: 01Jan2000, 12:28
Direct Runoff Volume: 34.30427 (1000
Baseflow Volume: 0.00000 (1000 M
Discharge Volume: 34.30427 (1000

Pr


Summary Results for Reservoir "Reservoir-SW_Pond_2"
Project: FAB_Swale_Sizing Simulation Run: 100yr_FAB v2
Reservoir: Reservoir-SW_Pond_2
Start of Run: 01Jan2000, 00:00
End of Run: 03Jan2000, 00:00
Compute Time: 22Jan2025, 14:36:45
Basin Model: 100yr_Pr v2
Meteorologic Model: TP108_100yr_298mm
Control Specifications: 48hr
Volume Units: MM 1000 M3
Computed Results
Peak Inflow: 7.27751 (M3/S)
Peak Discharge: 4.14206 (M3/S)
Inflow Volume: 42.5314 (1000 M3)
Discharge Volume: 33.6255 (1000 M3)
Date/Time of Peak Inflow: 01Jan2000, 12:14
Date/Time of Peak Discharge: 01Jan2000, 12:28
Peak Storage: 22.2910 (1000 M3)
Peak Elevation: 21.37843 (M)

100yr

Pr Emergency



Stormwater Pond 3 Sizing

PWL = 25.40 mRL

Outlets

SMAF Outlet = ø68mm

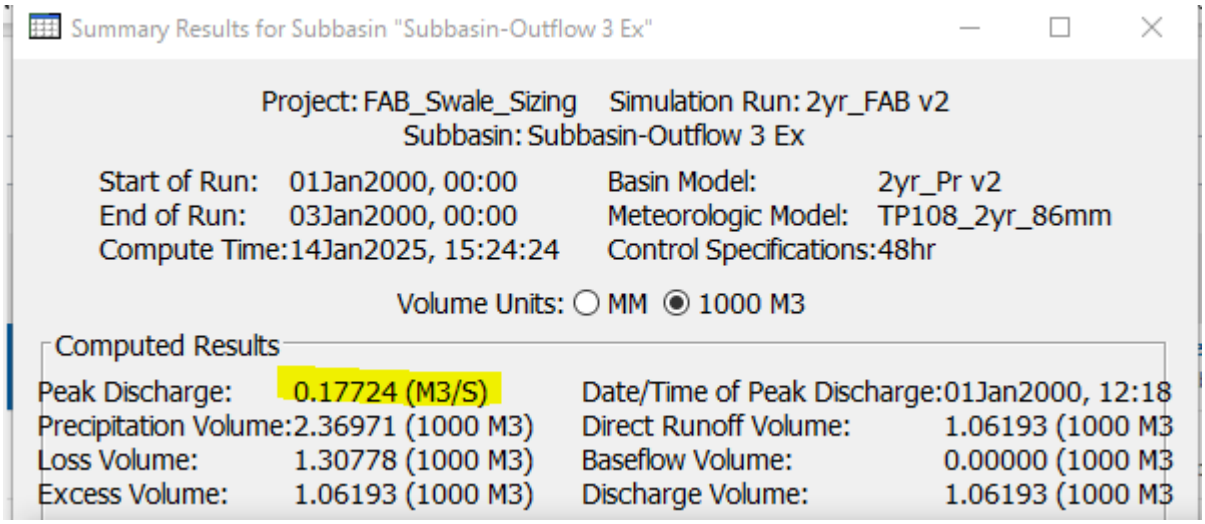
2yr/10yr/100yr Outlet 0.7m long Manhole Cutout @ 26.16 mRL (2yr tailwater)

100yr Emergency Spillway @ 27.00 mRL 20m Long

Freeboard Top of bund = 27.30 mRL

2yr

Ex



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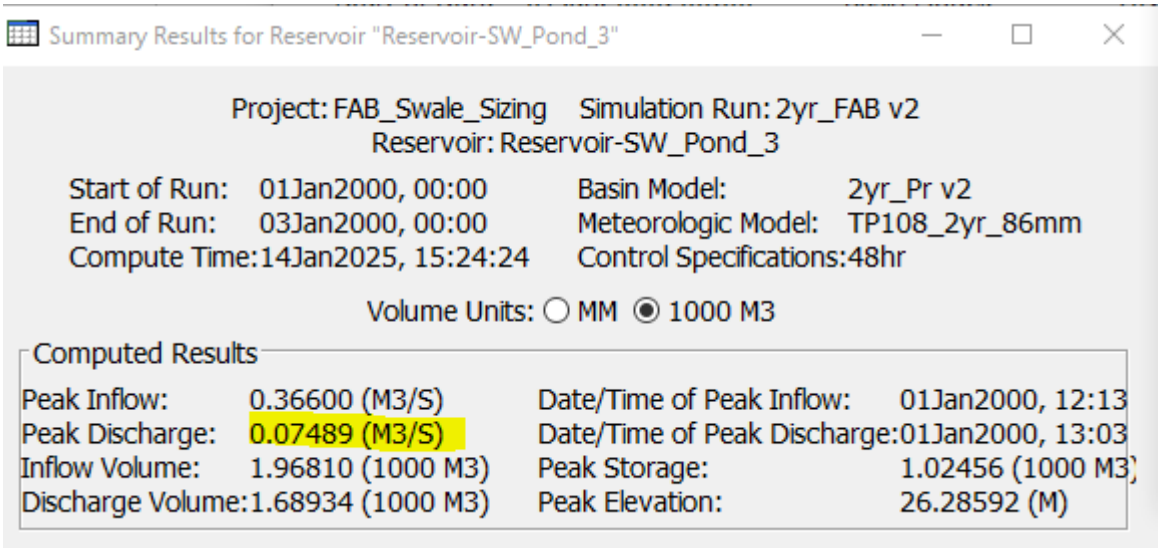
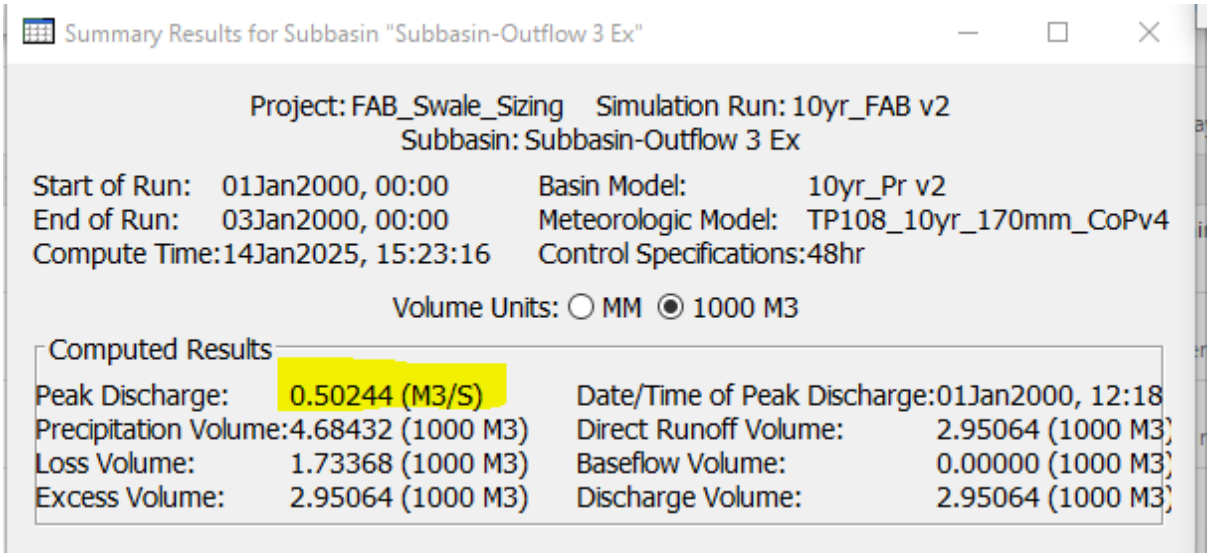
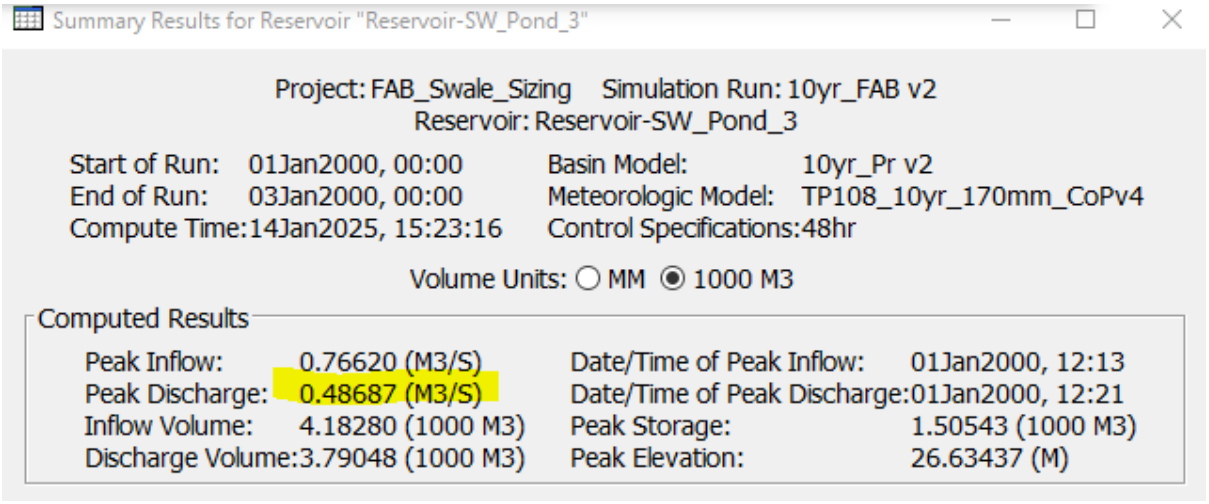


10yr

Ex

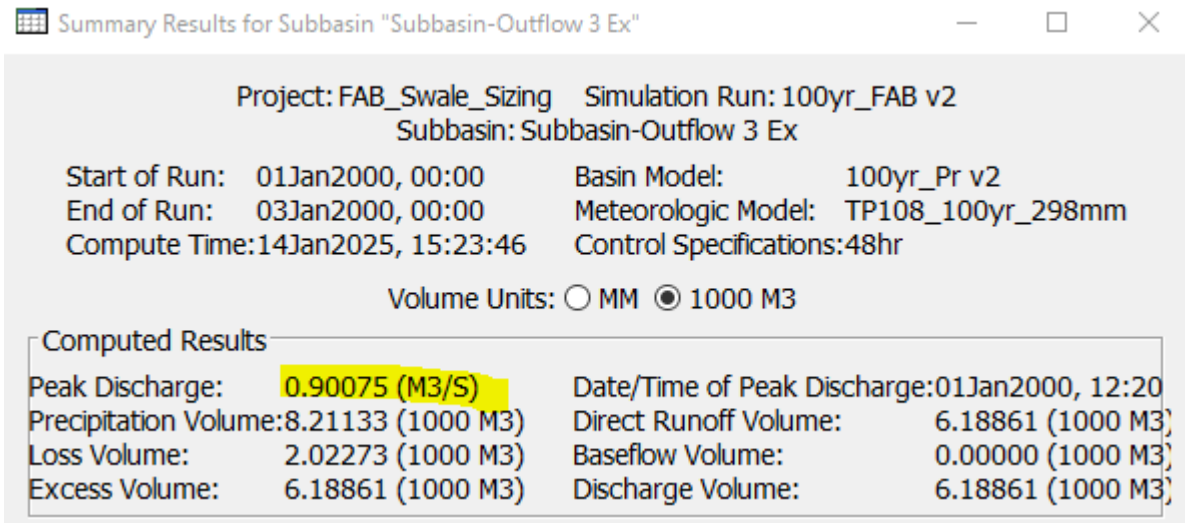

Summary Results for Subbasin "Subbasin-Outflow 3 Ex"
Project: FAB_Swale_Sizing Simulation Run: 10yr_FAB v2
Subbasin: Subbasin-Outflow 3 Ex
Start of Run: 01Jan2000, 00:00
End of Run: 03Jan2000, 00:00
Compute Time: 14Jan2025, 15:23:16
Basin Model: 10yr_Pr v2
Meteorologic Model: TP108_10yr_170mm_CoPv4
Control Specifications: 48hr
Volume Units: MM 1000 M3
Computed Results
Peak Discharge: 0.50244 (M3/S)
Precipitation Volume: 4.68432 (1000 M3)
Loss Volume: 1.73368 (1000 M3)
Excess Volume: 2.95064 (1000 M3)
Date/Time of Peak Discharge: 01Jan2000, 12:18
Direct Runoff Volume: 2.95064 (1000 M3)
Baseflow Volume: 0.00000 (1000 M3)
Discharge Volume: 2.95064 (1000 M3)

Pr

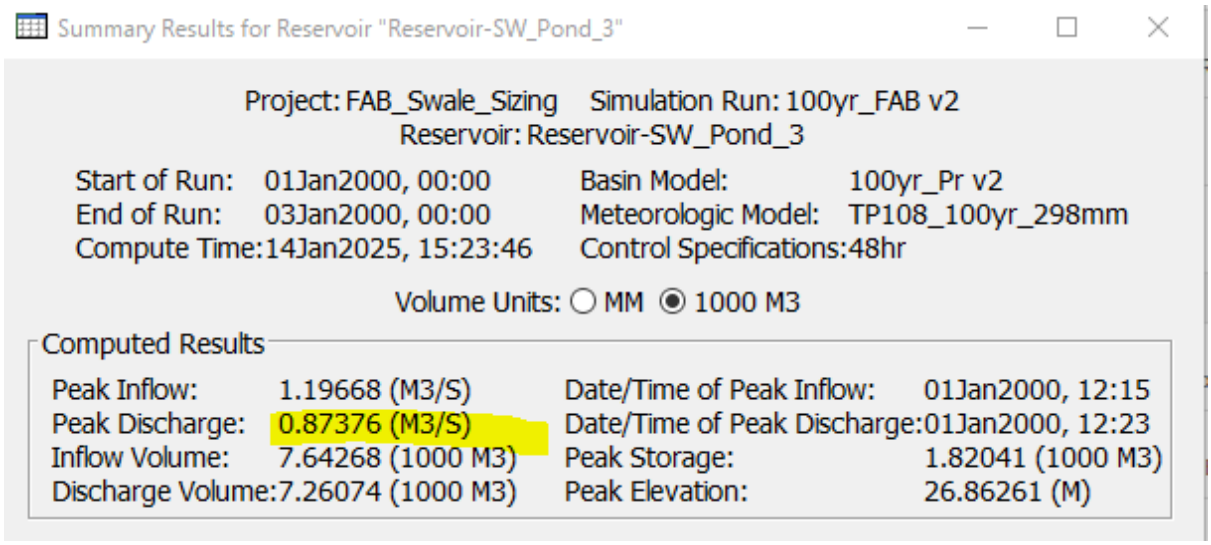

Summary Results for Reservoir "Reservoir-SW_Pond_3"
Project: FAB_Swale_Sizing Simulation Run: 10yr_FAB v2
Reservoir: Reservoir-SW_Pond_3
Start of Run: 01Jan2000, 00:00
End of Run: 03Jan2000, 00:00
Compute Time: 14Jan2025, 15:23:16
Basin Model: 10yr_Pr v2
Meteorologic Model: TP108_10yr_170mm_CoPv4
Control Specifications: 48hr
Volume Units: MM 1000 M3
Computed Results
Peak Inflow: 0.76620 (M3/S)
Peak Discharge: 0.48687 (M3/S)
Inflow Volume: 4.18280 (1000 M3)
Discharge Volume: 3.79048 (1000 M3)
Date/Time of Peak Inflow: 01Jan2000, 12:13
Date/Time of Peak Discharge: 01Jan2000, 12:21
Peak Storage: 1.50543 (1000 M3)
Peak Elevation: 26.63437 (M)

100yr

Ex

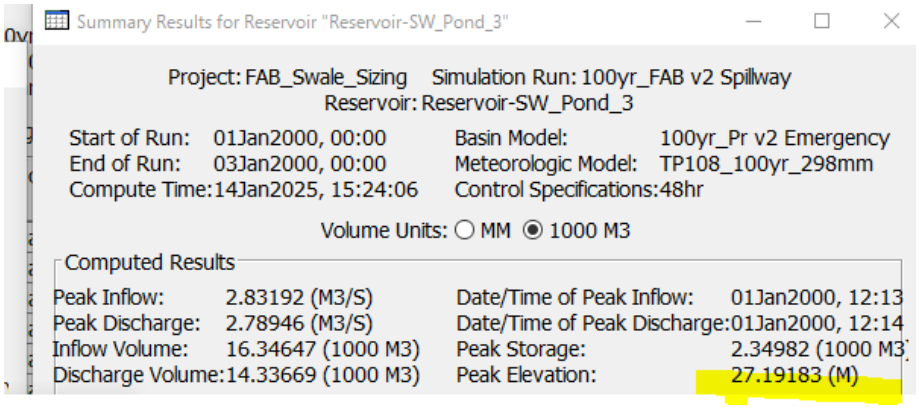

Summary Results for Subbasin "Subbasin-Outflow 3 Ex"
Project: FAB_Swale_Sizing Simulation Run: 100yr_FAB v2
Subbasin: Subbasin-Outflow 3 Ex
Start of Run: 01Jan2000, 00:00
End of Run: 03Jan2000, 00:00
Compute Time: 14Jan2025, 15:23:46
Basin Model: 100yr_Pr v2
Meteorologic Model: TP108_100yr_298mm
Control Specifications: 48hr
Volume Units: MM 1000 M3
Computed Results
Peak Discharge: 0.90075 (M3/S)
Precipitation Volume: 8.21133 (1000 M3)
Loss Volume: 2.02273 (1000 M3)
Excess Volume: 6.18861 (1000 M3)
Date/Time of Peak Discharge: 01Jan2000, 12:20
Direct Runoff Volume: 6.18861 (1000 M3)
Baseflow Volume: 0.00000 (1000 M3)
Discharge Volume: 6.18861 (1000 M3)

Pr


Summary Results for Reservoir "Reservoir-SW_Pond_3"
Project: FAB_Swale_Sizing Simulation Run: 100yr_FAB v2
Reservoir: Reservoir-SW_Pond_3
Start of Run: 01Jan2000, 00:00
End of Run: 03Jan2000, 00:00
Compute Time: 14Jan2025, 15:23:46
Basin Model: 100yr_Pr v2
Meteorologic Model: TP108_100yr_298mm
Control Specifications: 48hr
Volume Units: MM 1000 M3
Computed Results
Peak Inflow: 1.19668 (M3/S)
Peak Discharge: 0.87376 (M3/S)
Inflow Volume: 7.64268 (1000 M3)
Discharge Volume: 7.26074 (1000 M3)
Date/Time of Peak Inflow: 01Jan2000, 12:15
Date/Time of Peak Discharge: 01Jan2000, 12:23
Peak Storage: 1.82041 (1000 M3)
Peak Elevation: 26.86261 (M)

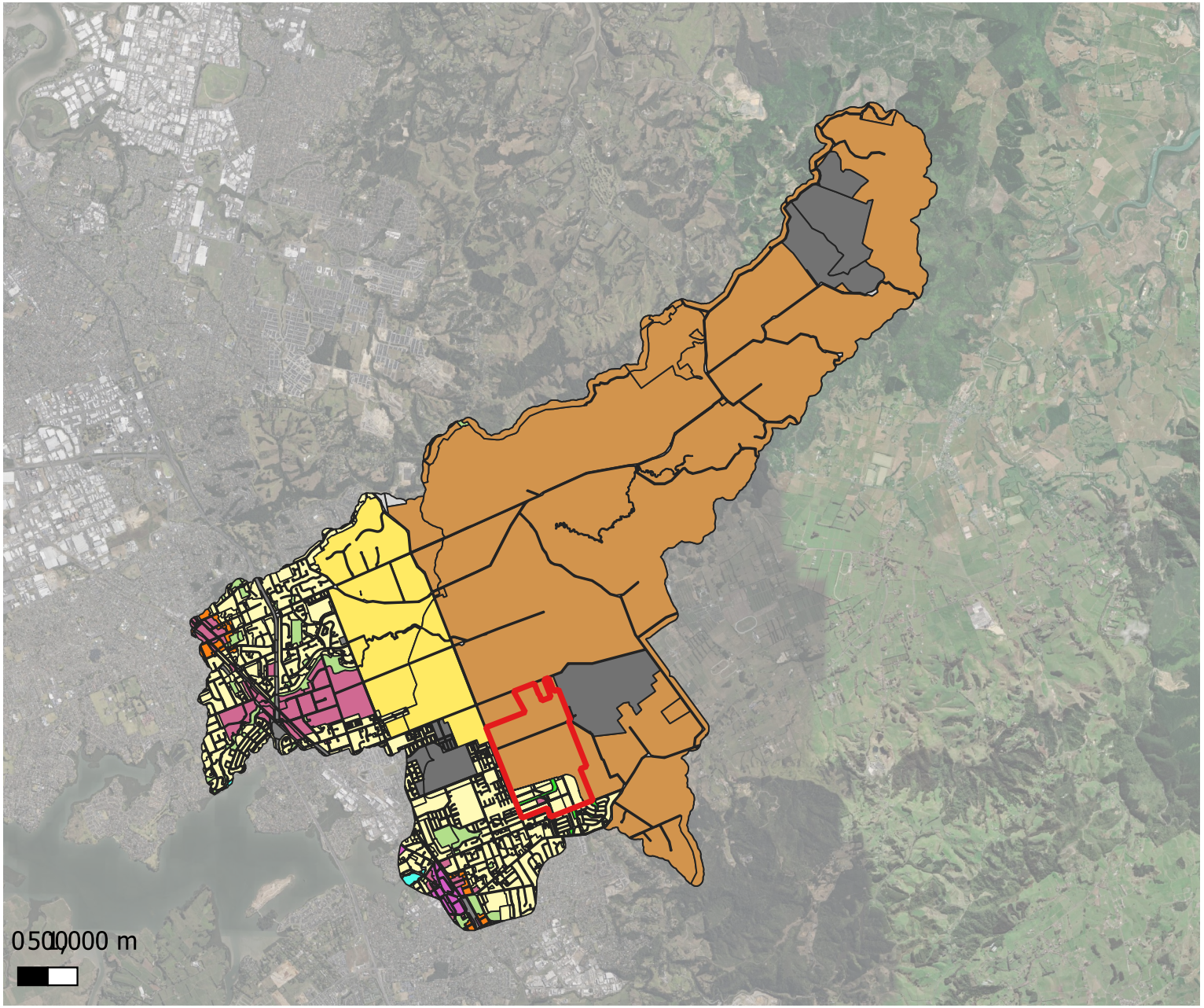
100yr

Pr Emergency



APPENDIX 9 – Zoning

Pre development Zoning



Legend

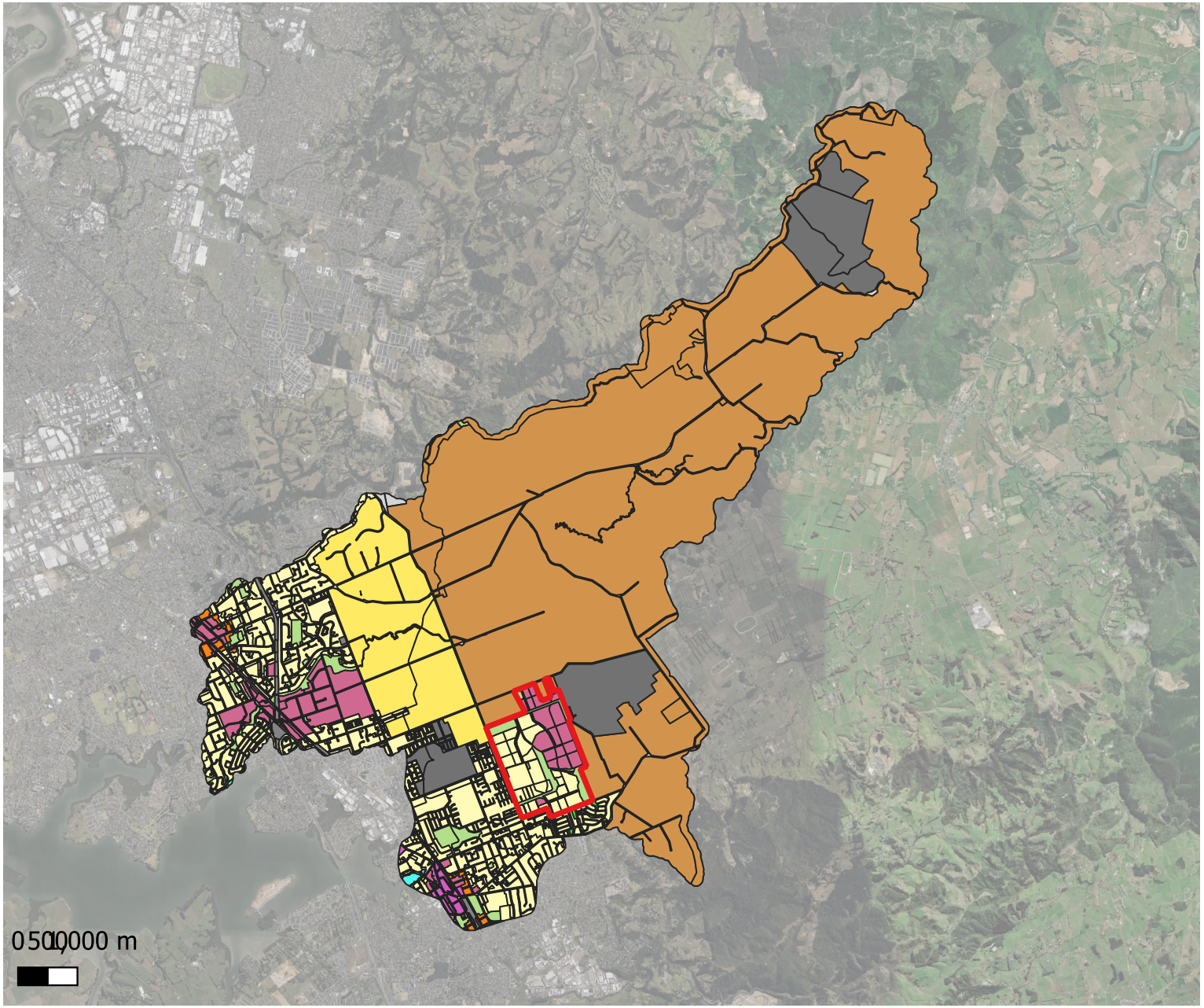
Pr-Bdy

Pre development Zoning

- Business - General Business Zone
- Business - Town Centre Zone
- Future Urban Zone
- Residential - Mixed Housing Suburban Zone
- Residential - Mixed Housing Urban Zone
- Residential - Single House Zone
- Road
- Rural - Countryside Living Zone
- Rural - Mixed Rural Zone
- Special Purpose - Airports and Airfields Zone
- Special Purpose - Major Recreation Facility Zone
- Special Purpose - Quarry Zone
- Special Purpose - School Zone
- Water
- Open Space

SK033
REV 002

Post development Zoning



Legend

Pr-Bdy

Post development Zoning

- Business - General Business Zone
- Business - Town Centre Zone
- Future Urban Zone
- Residential - Mixed Housing Suburban Zone
- Residential - Mixed Housing Urban Zone
- Residential - Single House Zone
- Road
- Rural - Countryside Living Zone
- Rural - Mixed Rural Zone
- Special Purpose - Airports and Airfields Zone
- Special Purpose - Major Recreation Facility Zone
- Special Purpose - Quarry Zone
- Special Purpose - School Zone
- Water
- Open Space

SK032
REV 002

Rain on Grid Infiltration layer Parameters

Zone Name	Impervious %	CN	Ia	S	Ab_Ratio
Business - General Business Zone	0.9	95.6	0.5	11.7	0.043
Business - Heavy Industry Zone	0.9	95.6	0.5	11.7	0.043
Business - Light Industry Zone	0.9	95.6	0.5	11.7	0.043
Business - Local Centre Zone	0.9	95.6	0.5	11.7	0.043
Business - Mixed Use Zone	0.9	95.6	0.5	11.7	0.043
Business - Neighbourhood Centre Zone	0.9	95.6	0.5	11.7	0.043
Business - Town Centre Zone	1	98	0	5.2	0.001
Coastal - Coastal Transition Zone	0.6	88.4	2	33.3	0.06
Coastal - General Coastal Marine Zone	1	98	0	5.2	0.001
Future Urban Zone	0.6	88.4	2	33.3	0.06
Open Space	0.1	76.4	4.5	78.5	0.057
Open Space - Community Zone	0.1	76.4	4.5	78.5	0.057
Open Space - Informal Recreation Zone	0.1	76.4	4.5	78.5	0.057
Open Space - Sport and Active Recreation Zone	0.4	83.6	3	49.8	0.06
Residential - Mixed Housing Suburban Zone	0.6	88.4	2	33.3	0.06
Residential - Mixed Housing Urban Zone	0.6	88.4	2	33.3	0.06
Residential - Single House Zone	0.6	88.4	2	33.3	0.06
Residential -Terrace Housing and Apartment Buildings Zone	0.7	90.8	1.5	25.7	0.058
Road	0.85	94.4	0.75	15.1	0.05
Rural - Countryside Living Zone	0.1	76.4	4.5	78.5	0.057
Rural - Mixed Rural Zone	0.1	76.4	4.5	78.5	0.057
Special Purpose - Airports and Airfields Zone	0.9	95.6	0.5	11.7	0.043
Special Purpose - Major Recreation Facility Zone	0.5	86	2.5	41.3	0.06
Special Purpose - Quarry Zone	0.9	95.6	0.5	11.7	0.043
Special Purpose - School Zone	0.7	90.8	1.5	25.7	0.058
Special Purpose Zone	0.7	90.8	1.5	25.7	0.058
Strategic Transport Corridor Zone	0.9	95.6	0.5	11.7	0.043
Water	1	98	0	5.2	0.001

APPENDIX 10 – STAGE 2 & 3 AWAKERI WETLANDS REQUIREMENTS

AWAKERI WETLANDS STAGE 2

DESIGN REQUIREMENTS

Healthy Waters

Design Office

Awakeri Wetlands – Stage 2
Design Requirements

Project	AWAKERI WETLANDS STAGE 2
Document	DESIGN REQUIREMENTS
Issue and Revision Record	

Status	Issued
Version	1
Date of Issue	27/11/2023
Author and Designer	Jesse Peeters Senior Healthy Waters Specialist
Reviewed	Amelia Cunningham Design Office Team Manager
Approved for Issue	Amelia Cunningham Design Office Team Manager

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Awakeri Wetlands – Stage 2
Design Requirements

Table of Contents

1.0 Project extent and staging

The project area for the Awakeri Wetlands is shown in Figure 1below and is broken into three stages by location:

- Stage 1: Awakeri Wetlands between Grove Road, Cosgrave Road and Walters Road
- Stage 2: Culvert crossing Cosgrave Road and connection to Stage 1
- Stage 3: Awakeri Wetlands between Cosgrave Road, Old Wairoa Road and the pond upstream of Old Wairoa Road

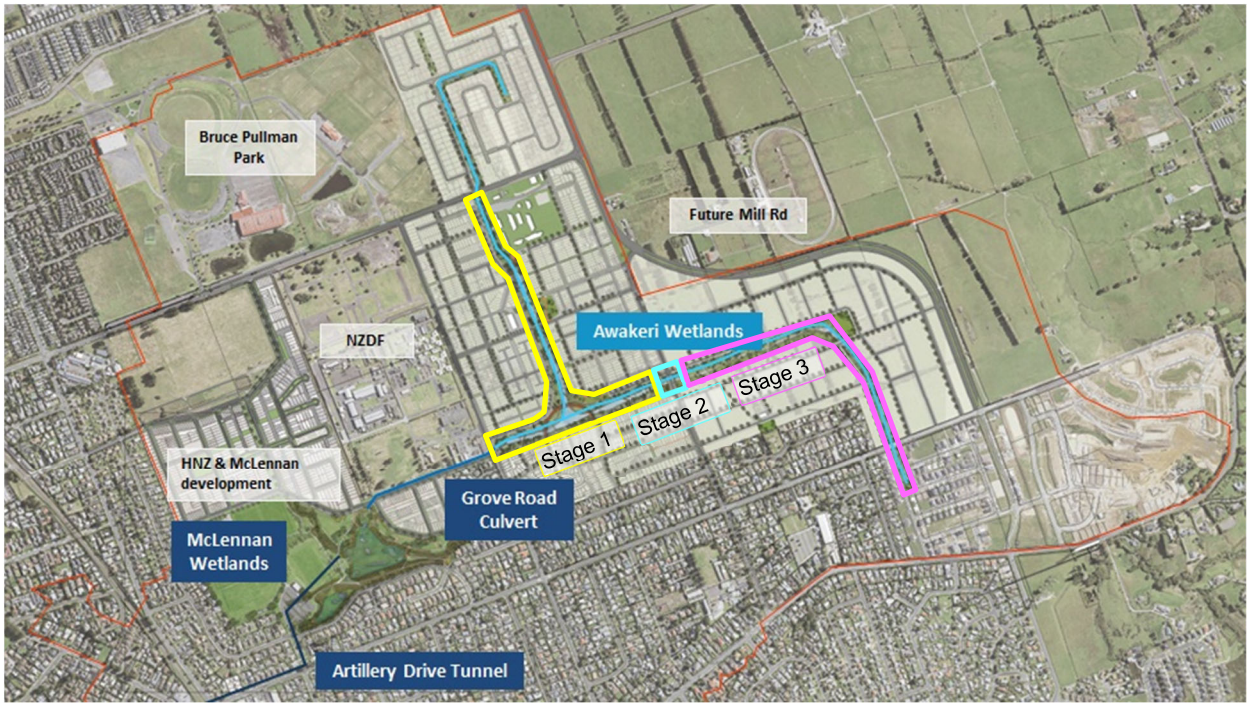


Figure 1 Awakeri Wetlands Staging

2.0 Design criteria

2.1 Standards, manuals and publications

The design and physical works shall be done in accordance with the version current at the time of the work of the following standards, manuals and publications except where amended by these Principal's Requirements:

a) Auckland Council Standard Specifications

b) The Building Act

c) Health and Safety in Employment Act

d) Resource Management Act

e) Maritime Safety Regulation

f) New Zealand Standards and codes

g) Auckland Council Code of Practice for Land Development

h) Auckland Transport Code of Practice

i) NZ Transport Agency Standards and Guidelines Manual

j) NZ Transport Agency Standard Specifications and Publications.

k) NZ Transport Agency Bridge Manual

Where the above does not explicitly cover all parts or issues relating to the design or construction, other codes or standards, such British, Australian or American that are applicable in respect of a part or issue shall apply where agreed by the Engineer.

Standards, manuals and publications shall be read in the following order of priority:

a) Acts of Parliament

b) The Principal's Requirements

c) Auckland Council Standard Specifications

d) Auckland Council Code of Practice for Land Development

e) Auckland Transport Code of Practice

f) Regulatory authority standards, specifications and guidelines

g) Australian/New Zealand Standards and guidelines

h) NZ Transport Agency specifications, standards and guidelines

i) British Standards

j) United States Standards.

Where a guideline document allows for different options or where engineering judgement is required, a design report or technical memorandum shall be provided.

2.2 Safety in Design

Safety in design must be considered throughout all stags of design and shall include a register, reporting and workshop to discuss and document the options considered for each element of the design.

Safety in design shall include input from Auckland Council Healthy Waters Operations, Community Facilities, Auckland Transport, Watercare and any other parties who will be involved with the asset throughout its design life.

Safety in design considerations shall be made for all key features including, but not limited to:

- Culvert
- Wingwall/headwalls
- Road, footpath and berm
- Pedestrian crossing

- Debris clearance
- Fall protection
- Stormwater connections
- Groundwater cutoff barrier installation
- Watermain protection methodology

2.3 Mana whenua partnership

A partnership was formed between mana whenua and the Awakeri Wetlands project design team during the design of Stage 1 of the Awakeri Wetlands. This partnership must be transitioned to the new designer to ensure that mana whenua continued to be partners of the project.

Regular huis/meetings are expected as part of this, an initial hui shall be arranged by the new designer and mana whenua to agree on hui frequency and level of involvement. The iwi who were part of the partnership are listed below. Key staff / contacts may have changed since and it is the designers responsibility to identify the current representatives of the iwi.

Table 1 Mana whenua representatives

Iwi group	Representatives	Contact
Ngati Tamaoho	Lucie Rutherford Hero Potini Zachary Sirett Edith Tuhimata (previously Ngati Te Ata Waiohua)	lucierutherfurd@gmail.com zac@tamaoho.maori.nz
Te Ākitai Waiohua	Nigel Denny Karen Wilson	kaitiaki@teakitai.com
Ngai Tai ki Tamaki	Jonathan Billington James Brown	kaitiaki@ngaitaitamaki.iwi.nz
Ngāti Te Ata Waiohua	Karl Flavell	karl.flavell@ngatiteata.iwi.nz

2.4 Watercare consultation and approvals

Watercare's Waikato No.1 watermain is in close proximity to the project and Watercare approval will therefore be needed prior to any work taking place. Previous correspondence with Watercare has indicated the following:

- Any shutdowns of the Waikato No.1 Watermain would need to be planned in advance with up to 2 years notice given to Watercare. This requirement may change depending on Watercares scheduled shutdowns so communications should be made with Watercare during the design phase to confirm.
- Finite Element Analysis (FEA) will be required and provided to Watercare to demonstrate that effects on the Waikato No.1 Watermain can be managed.
- Any design of temporary support or ground improvements shall be reviewed and approved by Watercare.

2.5 Local board consultation

The designer shall contact the local board to provide regular updates and accommodate feedback from the local board as required. The frequency of updates shall be determined based on an initial meeting with the local board to agree on frequencies.

2.6 Consents

All required consents (resource, discharge, landowner, etc.) and approvals (Watercare EPA and Works Over Approval, Auckland Transport EPA, etc) shall be obtained by the designer.

2.7 Existing services

The designer shall be responsible for liaising with utility providers and designing protection for of all known services which conflict with the proposed work, including but not limited to:

- Waikato No.1 Transmission Watermain (1200mm diameter CLS pipe)
- Fibre optic cable for Watercare treatment plant controls (critical watercare infrastructure)
- Wastewater pipes and wastewater rising mains (225mm dia. rising main)
- Local watermains
- Overhead power lines
- Underground power cables
- Underground communications cables
- Fibre optic cables
- Roads

Approval from Watercare shall be obtained by the designer for the proposed works around the Waikato No.1 Watermain. This shall include approval of any short term (during construction) and long-term (post construction) protection methods and/or support required for the Waikato No.1 Watermain.

All underground and overhead services shall be protected and/or diverted during the works, with approvals gained from the relevant service providers where required.

2.8 Minimum design requirements

General design features of the Cosgrave Road Culvert (Stage 2 of the Awakeri Wetlands) are shown in the Specimen Design Drawings. The Specimen Design Drawings shall be referred to and significant deviations from the key features shown in these drawings shall be documented and approved by Auckland Council.

It is the designer's responsibility to determine the final design criteria, however the following section provides minimum requirement and sets out Auckland Councils expectations.

2.8.1 Crossing type

Stage 2 of the Awakeri Wetlands is proposed to be a multi-barrel culvert to convey water under Cosgrave Road.

The preferred culvert type is to use box culverts due to the following benefits:

- Larger air gap between low flow water level and soffit of culvert, as the flat top provides an increased width of air gap compared to a circular culvert. This will allow more debris to float through the culvert without getting impinged at the entrance.
- Larger capacity per width of cross section compared to circular culverts, therefore higher capacity, lower velocities and less erosion protection needed.

Sizing has been calculated in the *Awakeri Wetlands Stage 2 Specimen Design Report (Auckland Council, 2019)* as twin 2m (H) x 3m (W) box culverts.

Another size that would be acceptable is three 1.5m (H) x 2.5m (W) box culverts. The reduction in height and width per culvert unit means a shallower permanent water depth and larger air gap could potentially be achieved due to a reduced thickness of roof slab on the smaller culverts.

Final sizing of the box culverts is to be provided by the designer.

2.8.2 Culvert alignment (vertical/horizontal)

The vertical and horizontal alignment of the culverts shall be such that they do not adversely affect the integrity of any existing structures (i.e. Waikato No.1 Watermain, other services) and considers the safety, operation and maintenance considerations and risks described in the *Awakeri Wetlands Stage 2 Specimen Design Report (Auckland Council, 2019).*

The low flow depth of water in the culvert is controlled by a downstream weir within Stage 1 of the Awakeri Wetlands. The low flow water level is 22.25 m RL. The design shall consider the safety, operational and maintenance aspects associated with the depth of water in the culvert. An air gap shall be provided in the culvert between the low flow water level and the soffit of the culvert to allow small debris to flow through.

The length of the culvert shall be confirmed based on discussions with Auckland Transport in regards to any future road modifications planned and other constraints determined by the designer.

2.8.3 Design Life

A design life of not less than 100 years shall be allowed for, taking into consideration the low pH / aggressive ground conditions such as potential acid sulphate soils. Further information is available in the *Hydrogeology Assessment of Effects (GHD, 2016)* and the *Acid Sulfate Soils Management Plan (GHD, 2017).*

2.8.4 Design flow

The design of the culvert shall be able to convey up to the 1% AEP storm event without the immediate upstream water level surcharging above RL23.80m RL. Catchment flows for the culvert are outlined in Table 2, provided the catchment, development and impervious area assumptions from the Awakeri Wetlands Stage 1 Detailed Design Report are met.

Table 2 Target Hydraulic Capacity Requirements

Storm Event	Peak flow (m^3/s)
50% AEP (2yr ARI)	5.7
10% AEP (10yr ARI)	14.6
1% AEP (100yr ARI)	23.0

2.8.5 Blockage assessment

A blockage assessment shall be provided which outlines the likelihood and consequence of various blockage scenarios. The final allowance for blockage must be agreed with Auckland Council prior to finalization of the design.

2.8.6 Hydraulic design parameters

Hydraulic design parameters are described in the *Awakeri Wetlands Stage 2 Specimen Design Report (Auckland Council, 2019).* This report provides a hydraulic design for various options, however the designer is not limited to these options.

The designer will need to provide an updated hydraulic design for any solutions outside the options considered in the *Awakeri Wetlands Stage 2 Specimen Design Report (Auckland Council, 2019).* Table 3 outlines the hydraulic design parameters that shall be applicable to any option. These performance requirements must be met for Stages 1 and 3 of the Awakeri Wetlands to perform as intended.

Table 3 Hydraulic design parameters

Assumption	Value	Source
Low flow water level	22.25 m RL	Awakeri Wetlands Scheme Design (GHD, 2016)
1% AEP tailwater level	23.25 m RL	Awakeri Wetlands Scheme Design Hydraulic model (GHD, 2017).
Maximum 1% AEP upstream water level	23.80 m RL	Selected based on not increasing flood levels upstream in comparison to the Awakeri Wetlands Scheme Design model. Value of the 1% AEP water level at the upstream weir used.
Invert level of Waikato No.1 Watermain	23.25 m RL	Watercare As-built (at centre of proposed infrastructure alignment). Level to be confirmed by the designer.
Awakeri Wetlands channel invert U/S and D/S end	21.45 m RL	Awakeri Wetlands Scheme Design (GHD, 2016)

2.8.7 Minimum external design loads

The designer shall determine the design load parameters for the crossing with Auckland Transport.

Minimum design load parameters are available in Table 4.

Table 4 Live loads

Item	Load allowance	Reference

HN vehicle loads	3.5 kPa x 3 m wide uniform load *plus* 2 x 120 kN axle loads at 5 m ctrs, or 12 kPa surcharge pressure (as appropriate)	Bridge Manual (SP/M/022) – Section 3.2.2 and Figure 3.1 and Section 3.4.12
HO vehicle loads	3.5 kPa x 3 m wide uniform load *plus* 2 x 240 kN axle loads at 5 m ctrs, or 24 kPa surcharge pressure (as appropriate)	

2.8.8 Inlet and outlet structures

An inlet / outlet structure shall be provided at each end of the culverts. The headwall/wingwalls shall be similar to the Grove Road culvert outlet wingwalls and shall include:

- Wingwalls/headwall shall be parallel with the road (ie. straight concrete retaining walls)
- Material shall be concrete with an exposed aggregate finish (sandblasted or similar)
- Height of the headwall shall be minimised to minimise the fall height from above.
- Mitigation to discourage access by the public, such as planting.
- Access to be provided for clearing blockages
- Safety barrier / fence to mitigate fall height. Safety fence to match the fence on top of the Grove Road Culvert outlet structure at the McLennan Wetland.
- Erosion / scour protection. Type and extent of erosion protection to minimise visual impact on surrounding environment and align with materials used in Stage 1 of the Awakeri Wetlands.

Figure 2 Culvert headwall/wingwalls

2.8.9 Buoyancy

The culvert shall be designed for the effects of buoyancy for suitable scenarios determined by the designer based on nearby groundwater monitoring data and Awakeri Wetlands design information.

2.8.10 Dewatering

Any dewatering systems used shall be designed and operated so that related settlement does not exceed limits prescribed by the Resource Consent Conditions in both long term and short term (during construction) scenarios.

2.8.11 Groundwater cut-off barrier

Previous assessments, as described in the *Hydrogeology Assessment of Effects (GHD, 2016)* have indicated that a vertical groundwater cut-off barrier is required around the perimeter of the culvert excavation, and the base of the excavation should be lined with a low permeability material to reduce groundwater inflows during construction.

The vertical groundwater cut-off barrier is also required around permanent excavations at either end of the culvert to manage long-term groundwater drawdown and associated settlement effects.

Groundwater barrier requirements

- The cut-off barrier is required to have a maximum permeability of 1 x 10^{-8} in order to mitigate groundwater drawdown to an acceptable level.
- The base of the excavation during culvert construction shall also have a maximum permeability of 1 x 10^{-8}.
- The vertical groundwater cut-off barrier was proposed as a bentonite-cement slurry wall. The slurry wall was proposed as minimum 600mm wide and 7.0m deep.
- The mix design for the slurry wall shall be provided by the designer, with lab testing and in-situ field testing to demonstrate that the required permeability can be met and sufficient curing will be achieved.
- Final design of the slurry wall shall be provided by the designer.

Quality assurance

- In-situ QA coring and sampling shall be undertaken during construction to confirm that the required permeability requirements have been met for the installed wall.
- For every hundred meters of the slurry wall, the proper curing of the slurry wall shall be checked as follows:
    - Coring should be done no earlier than 14 days in at least 4 locations in the central axis of the cut off wall and all the way to the full depth of the cut off wall.
    - Coring will enable checking of cut off wall consistency, depth, verticality and width. Holes to be grouted back
    - Number of locations to be drilled will be reviewed and may change in view of the encountered results, as further coring could be necessary.
- Where QA coring or sampling shows non-compliance, the slurry wall shall be repaired to achieve compliance. This may involve re-excavating and reinstalling the slurry wall in some sections.

Contingency plan for obstructions

A contingency plan is required where the slurry wall installation encounters underground obstructions. The Engineer shall be consulted when an obstruction is encountered to determine an appropriate action. Actions could include:

- Investigating the extent of the obstruction through coring, or additional excavating
- Coring through timber obstructions using an excavator mounted coring tool.
- Cutting and removing the obstruction.
- Realigning the cut-off barrier to go around the obstruction.

Figure 3 Slurry wall photo

2.8.12 Culvert groundwater ingress and settlement considerations

The following groundwater and settlement considerations shall be made when designing the culvert:

- The infrastructure conveying water under the road shall be fully sealed to prevent leakage of groundwater and sediments into the culvert over the design life of the structure. This requirement is important for mitigating the risk of groundwater drawdown and settlement of the surrounding peat soils.
- Post tensioning shall be considered if the structure has joints that are at risk of leaking due to ground movement.
- Settlement of the structure (due to the weight of the structure and/or backfill) shall be assessed and the impact of any predicted settlement on adjacent structures and services shall be considered and mitigated. Use of lightweight backfill shall be considered where/if appropriate.
- Settlement of the ground around the structure (due to any anticipated effects on adjacent groundwater and soil) shall be assessed and the impact of any predicted settlement on adjacent structures and services shall be considered and mitigated.

- Lightweight backfill such as polyrock may be required to reduce the load on subsoils and manage ground settlement.
- Stiffening of the culvert bedding may also be required by including a geogrid raft or flowable fill to mitigate differential settlement across the culvert.

2.8.13 Road reinstatement

The design shall include details for road reinstatement after the culvert is installed which shall be agreed with Auckland Transport, but at a minimum shall include:

- A pedestrian crossing for Cosgrave Road shall be designed and approval shall be obtained by Auckland Transport and any other relevant parties. The designer shall liaise with Auckland Transport and any other relevant parties to determine a suitable crossing detail (ie. signalised, island, zebra crossing).
- The Cosgrave Road corridor shall be upgraded to align with the future road cross section. The designer shall liaise with Auckland Transport to determine the future road cross section details.
- The extent of road corridor upgrade shall include along the full frontage property boundary of the Awakeri Wetlands designation.
- Preference is to avoid the need for road safety barriers (crash barriers). Providing a setback for inlet/outlet structures is a preferred method for managing the risk of collisions with the culvert or associated structures rather than installing crash barriers.
- Consultation with Auckland Transport is required to confirm whether a Traffic Impact Assessment is required and to plan any road closures, diversions or traffic management required during the work.

2.8.14 Scour and erosion protection

The design shall include the 2 year, 10 year and 100 year ARI peak flow scenarios for assessment of scour and erosion potential by carrying out a permissible shear stress analysis. Scour protection shall be designed and installed at the upstream and downstream ends of the structure to adequately mitigate scour and erosion.

Large riprap and concrete shall be avoided where possible for scour protection. Naturalised methods of scour protection such as planting is preferred. Oversizing culverts to minimise velocity, energy and shear stress is preferred over providing scour protection to dissipate high energy. Geosynthetic materials to reinforce plant roots such as geoweb and enkamat are preferred over hard engineering solutions.

Consideration to lining the underwater base of the wetland with geotextile filter fabric, enkamat and gravel shall be made to avoid scour, discharge of sediment and soft exposed peat which can be a safety hazard. This also provides a distinct base that can be identified during maintenance or desilting.

Figure 4 Geoweb scour protection, enkamat and gravel lining for underwater wetland base

2.8.15 Planting

Planting is required between the footpath and the culvert headwall to discourage access. Planting shall meet the following requirements:

- Species to match those used in Stage 1 of the Awakeri Wetlands.
- Zones, mixes and planting layouts/clumps to match those in Stage 1 of the Awakeri Wetlands.
- Planting areas to include 100mm of aged arbor mulch. Processed wood chips are not acceptable.
- Planting areas shall include a minimum of 300mm topsoil or local peat soil.
- All planted areas below the 10% AEP flood level shall be covered with 100% biodegradable coconut matting or similar with jute mesh (no plastic mesh).
- All planted areas above the 10% AEP flood event shall be covered with a minimum of 100mm aged arbor mulch.

2.8.16 Obstruction Management Plan

The designer shall prepare an obstruction management plan which outlines the approach that needs to be taken if an obstruction is encountered during construction. It is highly likely that large buried kauri logs will be discovered when excavating. The management plan shall include the following response actions for the discovery of an obstruction:

- Determine the nature of the obstruction.
- Determine whether the obstruction clashes with the proposed work.
- Recommend a response action and seek approval from Auckland Council before proceeding.

- Options for managing buried obstruction/kauri log that clashes with the proposed infrastructure include, in order of preference:
    - Leave the kauri log in place if it doesn't clash with any key infrastructure such as weirs, boardwalks, footpaths or culverts.
    - Leave the kauri log in place and realign the proposed infrastructure to avoid the log.
    - Cut and remove part of the kauri log to avoid the proposed infrastructure, leaving the remainder in the ground.
    - Complete removal of the obstruction/kauri log, stockpile on site or place the kauri log in an approved location within the wetland.
    - Other options may be identified and proposed by the designer.

2.8.17 Reinstatement of permanent surface water drainage features

The reinstatement of surface drainage systems shall be designed and constructed such that the existing conveyance and inlet capacities are maintained or improved where they have been disrupted by the Contract Works.

2.8.18 Geotechnical design criteria

Recent geotechnical investigations are provided in the *Geotechnical Investigations Report (GHD 2016)* and the *Geotechnical and Ground Settlement Effects Report (GHD, 2016).* This is for information only and the designer is responsible for preparing updated geotechnical investigations and assessments to support their design.

2.8.18.1 Ground conditions

The ground conditions for the project area are described in the *Geotechnical Investigations Report (GHD 2016)* and the *Geotechnical and Ground Settlement Effects Report (GHD, 2016).* This is for information only and the designer is responsible for preparing updated geotechnical investigations and assessments to support their design.

2.8.18.2 Groundwater

Groundwater information is provided in the *Hydrogeology Assessment of Effects (GHD, 2016).* This is for information only and the designer is responsible for gathering updated groundwater information and preparing an updated hydrogeology assessment.

2.8.18.3 Seismic design

The designer shall consider seismic hazards and liquefaction risks in the design of the project.

Some information is provided in the *Geotechnical and Ground Settlement Effects Report (GHD, 2016)*. This is for information only and the designer is responsible for preparing updated seismic hazard and liquefaction assessments..

2.8.18.4 Geotechnical Baseline Report (GBR)

The designer shall prepare a GBR for the project.

The purpose of the GBR is to provide a single source contract document containing measurable contractual descriptions of the geotechnical conditions to be anticipated or to be assumed to be anticipated during construction. In the event of the project running into difficulties due to ground conditions, the GBR can be used to decide if the conditions are unforeseen and therefore create potential for a claim or fall within the conditions expected at the site. This does not present any ambiguous interpretation of conditions or any uncertainty. Only measurable, quantitative terms used.

The GBR shall present a very concise contractual summary of the ground model that the Contractor should allow for when tendering for the construction.

The GBR shall be prepared in general accordance with the Essex, R.J., 1996, Geotechnical Baseline Reports for Underground Construction: Guidelines and Practices published by the American Society of Civil Engineers, as amended by this contract.

The GBR shall contain:

- A brief summary description of the material types expected to be encountered.
- The estimated amounts and distribution of different materials along the alignment, typically presented as an estimate of the percentage of the project (for example, linear metres of tunnel) that each material type will make up. This shall be given as a predicted range to allow for geological uncertainty.
- Geotechnical and groundwater parameters, and expected behaviours, for each of these materials, given as a predicted range to allow for geological uncertainty. Include strength, permeability, grain size, mineralogy, predicted pumping rates, predicted settlement and any other aspects which could impact on construction. Wherever possible, these shall be expressed in quantitative terms, and should present the expected distribution envelope of each parameter within the range.
- Descriptions of geotechnical and man-made sources of potential difficulty or hazards that could impact the construction process (such as boulders, bedrock variability, contaminated groundwater, subsurface obstructions, unstable slopes, adjacent activities).
- A description of the anticipated construction methodology with which the baselines are associated. The baseline statements should be clear that the ground can be expected to behave differently with alternative tools, methods, sequences and equipment.

The GBR shall not contain:

- Ambiguous or vague interpretations
- Descriptions or parameters that cannot be easily measured or assessed and recorded during construction
- Qualitative terms such as ‘large’ or ‘major’ unless these are clearly defined.

2.8.19 Ground improvements

The designer shall identify in its Detailed Design Report:

- What ground improvements, if any, are proposed
- The methods used to quantify their extent and effectiveness, and

- The precedent that has been followed in their development.

The methods and precedent shall be referenced in the Detailed Design Report. The effectiveness of ground improvement shall be demonstrated by field testing.

2.8.20 Operation and Maintenance

A Draft Operations and Maintenance (O&M) Manual shall be supplied with the proposed design solution to enable the proposed solution to be fully assessed and understood by the asset owner. The Draft O&M Manual will include maintenance of fittings, plant and machinery requirements, traffic management, isolation, dewatering (and treatment/disposal of this water), pipe inspection, sediment/debris removal and decommissioning. Auckland Council may engage a third party operations expert to review the acceptability of the solution and the O&M Manual.

The design shall provide for safe personnel access to enable a walk-through of the full length of the new infrastructure for operational and maintenance purposes.

The designer shall finalise O&M manuals, containing such information and details as are necessary for Auckland Council to carry out the operation and cost-effective maintenance of the system. The contents and format of these O&M manuals shall be subject to the approval of Auckland Council and a third party expert.

2.9 Handover

A plan for handover of the asset to Auckland Council shall be prepared by the designed and reviewed / approved by Auckland Council during the design phase. The handover plan shall include:

- Confirm timeframes for handover, allowing for a minimum 12 month defects liability period.
- Confirm which departments will own and maintain each asset.
- Duration of planting maintenance by the contractor prior to handover to Auckland Council.
- Responsibility for completing resource consent conditions including groundwater and settlement monitoring.

3.0 Design deliverables

3.1 Preliminary Design

3.1.1 Definition

In the Preliminary Design the drawings and technical documentation are developed to the point where a resource consent application may be lodged if required.

It is expected that no significant changes to line, level, sizing or construction techniques will be required at detailed design. The purpose of detailed design will be to finalize structural calculations and produce construction drawings.

3.1.2 Safety in design

The Contractor shall conduct a safety in design process for the works.

At the Preliminary Design Stage, the Contractor shall take account within the design of:

- Provision and maintenance of a safe work environment for whole of asset life
- Provision and maintenance of safe plant and structures
- Provision and maintenance of safe systems of work
- Safe use, handling, and storage of plant, substances, and structures
- Provision of adequate facilities for the welfare at work of workers.

At Preliminary Design Phase, a Safety in Design Risk Assessment Register is required to identify current risk exposure in construction, operation, maintenance and decommissioning, plus the proposed risk treatment or improvement opportunity for the preferred option and any residual risks with commentary on how they should be managed.

3.1.3 Preliminary design requirements

As a minimum, the designer shall undertake the following tasks:

- Develop options for variations of the concept design including, but not limited to, construction methodology, vertical and horizontal alignments, exact locations of features, materials. The purpose of this exercise is for the designer to ensure that an optimum solution is developed for detailed design that has the lowest whole-of-life cost whilst still meeting all other project objectives and providing a safe working environment throughout the life of the asset.
- Identifying and scoping any additional investigations the designer considers necessary to properly complete the design
- Preliminary design report. The designer shall prepare a brief Preliminary design report as described below.
- Preliminary design drawings. The drawings shall be sufficiently detailed to support the required consent applications. The designer shall allow for all drawings and supporting documents required for a consent but they shall include as a minimum:
    - General arrangement drawings
    - Long sections and elevations
    - Working areas
    - Preliminary erosion and sediment control plans
    - Preliminary traffic management plans.

3.1.4 Preliminary design report and drawings

The Contractor shall submit a Preliminary Design Report covering the following:

- Hazards identified and mitigated through the safety in design process followed (either appended with a summary within the report or a detailed section within the report).
- Outline specifications for all key components including all structures and materials.

- Key assumptions made.
- Utility diversions (if required).
- Key risks and risk management.
- Recommendations including confirmation that the recommended option meets the project objectives or otherwise.
- Commentary on hydraulic performance.
- A clear statement of the internal quality control and quality assurance procedures adopted in developing the design options.
- Stakeholder and end-user requirements and confirmation that the preferred option meets agreed asset owner requirements.
- Benefits of the option, and how they can be measured during or after project implementation.

3.2 Detailed design

3.2.1 Definition

In the Detailed Design Stage, the design is finalised and complete construction drawings, specifications and any monitoring, QC plans etc are produced.

3.2.2 Safety in design

A safety in design assessment is required for the Detailed Design. This safety in design process should be continued from the Preliminary design phase. The designer should define the methodology used for the risk assessment during this phase for agreement with Auckland Council. Auckland Council will identify the stakeholders that will be involved in the Safety in Design process.

At the end of the detailed design, a Safety in Design Risk Register and Report is required. All residual health and safety risks in construction, operation, design for exceedance, maintenance and demolition or disposal shall be clearly described and conveyed. Where relevant, these should be noted on the construction drawings if they are to be managed during construction.

A stand-alone safety in design report is required. All other reports will require a safety in design section as part of the design reporting.

3.2.3 Detailed design requirements

As a minimum, the Contractor shall undertake the following tasks:

- Scope confirmation workshop. The designers project manager and lead technical staff shall all attend this workshop. The purpose of the workshop will be to confirm the preferred solution and determine any preferences for final details. The designer shall prepare minutes of the workshop.
- Review the preliminary design including; confirmation of the findings of the preliminary design report costings, benefits, etc.
- Carry out all calculations required to finalize materials, structural strengths etc.

- Prepare fully detailed Construction Drawings including:
    - Geotechnical information (bore logs) and existing underground and overhead services shall be shown on the long sections.
    - Detailed construction site access, storage and site office areas.
    - Locations of existing utility services including details of diversions where required.
    - Waikato No.1 Watermain protection details.
    - Groundwater and settlement monitoring plan.
    - Structural drawings.
    - Any H&S issues and design mitigation assumptions that may affect or be affected by construction methodology.
- Review Standard Specifications and prepare Particular Specifications.
- Update safety in design and prepare report suitable for hand over to Constructors.
- Prepare a Geotechnical Baseline Report and agree with Auckland Council.
- Submit draft drawings, specifications and plans to Auckland Council for review.
- Finalize pre-construction risk register.
- Detailed design report as below.
- Address comments on the detailed design report and drawings. The designer shall ensure that the drawings and documents are properly reviewed and a QA undertaken by the designer prior to submission. Where any errors or omissions are found that Auckland Council considers to be a failure of QA or review, the drawings and documents shall be returned to the designer without further comment. The formal review will only be undertaken once the QA issues have been resolved. Auckland Council may appoint a peer review in addition to internal reviews. The designer shall allow for collating and assessing the reviews from Auckland Council and shall produce a Construction Documents and Drawing set that adequately addresses the comments.

3.2.4 Detailed design report and drawings

The designer shall prepare and submit a Detailed Design Report. The Final Design Report need not repeat all the detail of the Preliminary design report but should cover the following:

- Details of the final design and proposed construction methodology
- Safety in design considerations in the design process
- The Contractor's assessment of hazards and risks (including H&S) to be managed, if appropriate, a summary of risks eliminated or minimised through design can also be included
- Significant changes to the design or departures from the approved Preliminary Design Report
- Summary of residual risks at contract stage and for whole of life highlighting any remaining high risk items

- Updates to any technical or specialist reports as a result of amendments since the Preliminary design report
- Advice on any variations that may be required to Resource Consents as a result of such changes.
- Any departures from the Auckland Council Code of Practice and Standard Specifications
- Detailed advice on producer statement requirements and the level of construction monitoring required by the designer in order to be able to deliver the producer statements.

Construction drawings, labelled “For Construction”, shall be prepared on the standard Auckland Council title block and include the same information as the Preliminary Design drawings plus the following:

- Intended purpose of the asset (if appropriate)
- Geotechnical information (bore logs) and existing underground and overhead services shall be shown on the long sections
- Updated and more detailed construction site access, storage and site office areas
- Utility services diversions
- Groundwater and settlement monitoring plan.
- Structural drawings.
- Safety in design considerations to be addressed during construction, operation, maintenance and demolition or disposal.

The designer shall facilitate a meeting with Auckland Council to present and discuss the detailed design and the draft construction documentation. The objective of the design meeting is to agree what changes (if any) are required to the Draft Construction Documentation before a final version of the documentation is signed off.

Any review does not remove any responsibility from the designer for the correctness or appropriateness of the design or construction documents.

3.2.5 Quality assurance requirements

The designer shall prepare a project review and audit schedule for the Detailed Design and Construction Documentation. This will identify:

- One or more named reviewers accountable for reviewing technical outcomes, and technical reviews planned. Unless otherwise agreed with Auckland Council, the reviewers shall be those named in the Proposal. Where Auckland Council considers that less qualified staff than those shown in the Proposal are offered, revised rates shall be agreed.
- The designers named Project Director or Sponsor will be accountable for reviewing overall project delivery, and project outcome reviews. Auckland Council has an expectation that there will be a scope review (10%), a progress review (50%) and an outcome review (90%) as a minimum

- All documents will have a QA review before being delivered to Auckland Council, including draft reports. The report should include a QA and document control page to identify author, reviewer and version control for drafts.

4.0 Hold points

In addition to reviews of the design packages and elements described above, specific review of the following design items is also required by Auckland Council as they become available:

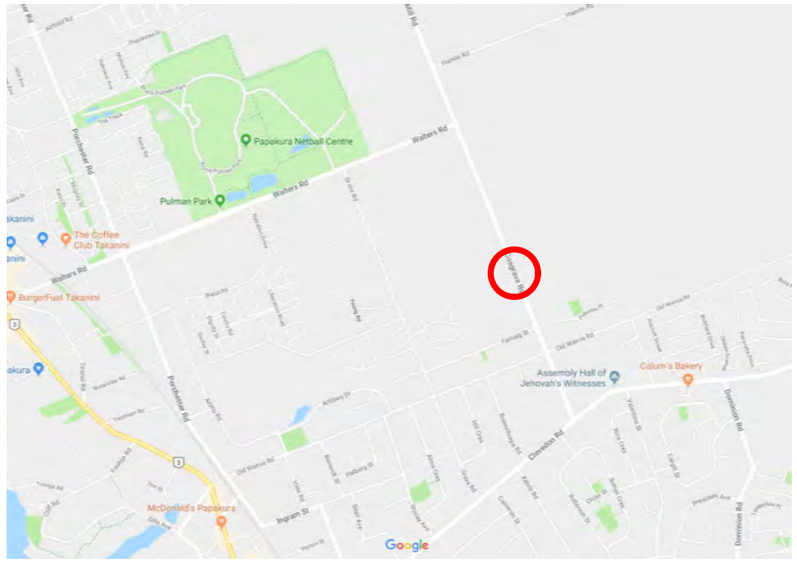
- Construction methodology.
- Staging of Stage 2 and 3.
- Groundwater management design.
- Peer review for groundwater drawdown assessment and settlement assessment (long term and short term), buoyancy assessment, culvert foundation/bedding design and road backfill design.
- Waikato No.1 Watermain effects assessment and protection design.
- Groundwater and Settlement Monitoring and Contingency Plan.
- Material specifications including shop drawings for pre-cast concrete structures.
- Groundwater cut-off barrier / slurry wall mix design and testing documentation.
- Erosion and scour protection design.
- Obstructions management plan (approach for managing clashes with buried kauri logs etc).

Additional hold points will be required for the construction phase and these will be outlined by Auckland Council prior to construction.

AWAKERI WETLANDS - STAGE 2

SPECIMEN DESIGN DRAWINGS

COSGRAVE ROAD, TAKANINI



LOCALITY PLAN
NTS

DRAWING REGISTER	
DESIGN DRAWINGS	
008325.01.200	COVER SHEET
008325.01.201	GREATER STORMWATER SCHEME
008325.01.203	CONTRACTORS WORKING AREA
008325.01.204	GROUNDWATER AND SETTLEMENT MONITORING
008325.01.210	PLAN VIEW
008325.01.211	LONGSECTION
008325.01.212	SECTION DETAILS

SPECIMEN DESIGN

Plot Date: 10-Oct-23 1:53 pm
File path: U:\COO\IES\STW\D&D\4. Projects\N\008325 Awakeri Stage 2\9. SDO Technical\CAD\Detailed Design Stage 2

REVISION	DATE	AMENDMENT	BY	APPD.
B	10.10.23	SPECIMEN DESIGN	JP	AC
A	04.06.19	DRAFT	JP	AC

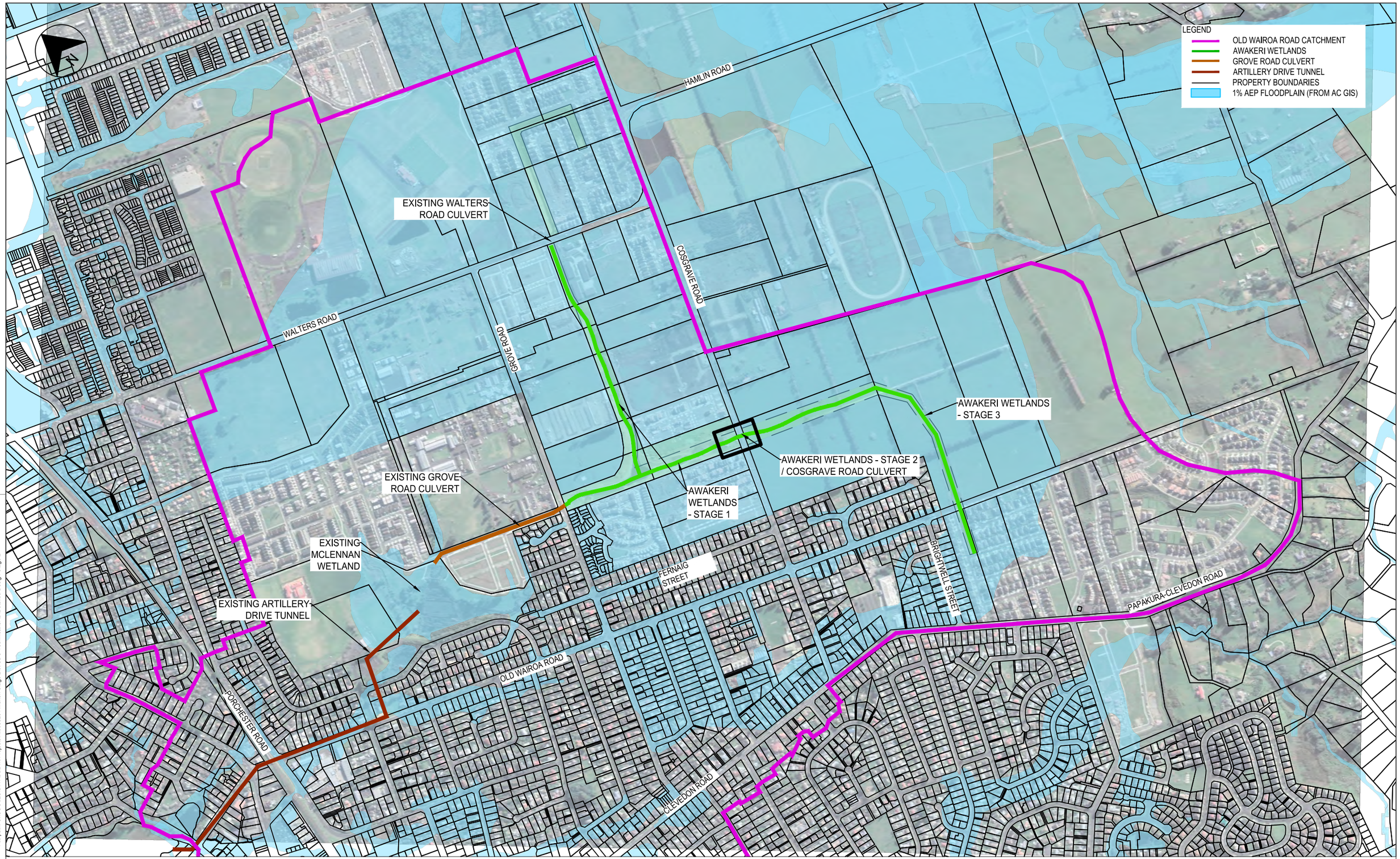
DRAWN BY: J PEETERS — SIGNED: — DATE: 10.10.23
DESIGNED BY: J PEETERS — SIGNED: — DATE: 10.10.23
CHECKED BY: A CUNNINGHAM — SIGNED: — DATE: 10.10.23
APPROVED FOR RELEASE BY: A CUNNINGHAM — SIGNED: — DATE: 10.10.23

CONSULTANT'S LOGO

Auckland Council
Te Kaunihera o Tāmaki Makaurau

TITLE:
AWAKERI WETLANDS
COSGRAVE ROAD CULVERT - SPECIMEN DESIGN
COVER SHEET

ORIGINAL SCALE A3: **NTS**
WBS No.: **008325**
DRAWING No.: **008325.01.200**
REVISION: **B**



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SPECIMEN DESIGN

REVISION	DATE	AMENDMENT	BY	APPD.
B	10.10.23	SPECIMEN DESIGN	JP	AC
A	16.04.19	DRAFT	JP	AC

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TITLE:

AWAKERI WETLANDS - STAGE 2

COSGRAVE ROAD CULVERT - SPECIMEN DESIGN

GREATER SCHEME LOCATION PLAN

ORIGINAL SCALE A3	WBS No.
1:10,000	008325
DRAWING No.	REVISION
008325.01.201	B

EXTENT OF WORKING AREA TO BE CONFIRMED

PROPOSED 7m DEEP SLURRY WALL

PROPOSED 3m DEEP SLURRY WALL

PROPOSED 5m WIDE HAUL ROAD

SPECIFIC LEGEND
CONTRACTORS WORKING AREA

EXISTING STAGE 1 EXCAVATION

PROPOSED CHANNEL EXCAVATION (STAGE 2)

PROPOSED TEMPORARY BATTER (STAGE 2)

PROPOSED COSGRAVE ROAD CULVERTS

CONTRACTORS WORKING AREA

COSGRAVE ROAD

EXISTING STORMWATER POND (TO FORM PART OF AWAKERI WETLANDS STAGE 3)

EXISTING OLD WAIROA ROAD CULVERT

OLD WAIROA ROAD

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DESIGN LEGEND
- EXISTING STORMWATER CULVERT
- EXISTING FENCE
- EXISTING FARM RACE
- EXISTING WATER PIPE
- EXISTING GROUNDWATER CUT-OFF WALL (7m DEEP).
- EXISTING SHALLOW CUT-OFF WALL (3m DEEP).
- EXISTING PROPERTY BOUNDARIES
- WORKS AREA
- EXISTING STAGE 1 WORKS CONTOURS
- DESIGNATION BOUNDARY
- EXISTING GAS TRANSMISSION PIPELINE
- EXISTING FARM DRAIN
- EXISTING HAUL ROAD

SPECIMEN DESIGN

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TITLE:
AWAKERI WETLANDS - STAGE 2
COSGRAVE ROAD CULVERT - SPECIMEN DESIGN
CONTRACTORS WORKING AREA

ORIGINAL SCALE A3: 1:2000
WBS No.: 008325
DRAWING No.: 008325.01.203
REVISION: B

Plot Date: 10-Oct-23 1:56 pm
File path: U:\CO\OMES\ST\WD&D\4. Projects\N.008325 Awakeri Stage 2\9. SDO Technical\CAD\Detailed Design Stage 2

NOTE:
INDICATIVE SETTLEMENT VALUES (mm) BASED ON PREDICTED RESIDUAL GROUNDWATER DRAWDOWN BELOW SEASONAL LOW (2014-2015 MONITORING) WITH MITIGATION

- 20-25 — POTENTIAL SETTLEMENT CONTOURS IN MILLIMETERS (mm)
- 20-25 — POTENTIAL SETTLEMENT CONTOURS IN MILLIMETERS ASSOCIATED WITH OTHER STAGES
- POTENTIAL DRAWDOWN MITIGATION
- STRUCTURES PROPOSED FOR MONITORING
- INDICATIVE BUILDING SETTLEMENT MONITORING PINS
- AREA OF PROPOSED PIEZOMETER AND GROUND SETTLEMENT PIN MONITORING FOR SCOPE OF THIS PLAN
- AREA OF PROPOSED PIEZOMETER AND GROUND SETTLEMENT PIN MONITORING COVERED BY SEPARATE PLAN

LEGEND

- PIEZOMETER
- DAMAGED / DESTROYED PIEZOMETERS
- PROPOSED PIEZOMETERS
- SETTLEMENT MONITORING PINS
- SETTLEMENT MONITORING PINS ON BOREHOLES
- DAMAGED / DESTROYED SETTLEMENT MONITORING PINS

EXISTING WALTERS ROAD CULVERT
WALTERS ROAD
COSGRAVE ROAD
GROVE ROAD
AWAKERI WETLANDS - STAGE 1
AWAKERI WETLANDS - STAGE 3
PROPOSED COSGRAVE ROAD CULVERT
EXISTING GROVE ROAD CULVERT
DEWATERING TRIAL PINS REFER 51-3217404-Q022 BHMP-38 - BHMP-42 AND MP36-MP-38
OLD WAIROA ROAD
BRIGHTWELL STREET
FERNAIG STREET

SPECIMEN DESIGN

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Auckland Council
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TITLE:
AWAKERI WETLANDS - STAGE 2
COSGRAVE ROAD CULVERT - SPECIMEN DESIGN
GROUNDWATER AND SETTLEMENT MONITORING

ORIGINAL SCALE A3: 1:5,000
WBS No.: 008325
DRAWING No.: 008325.01.204
REVISION: B

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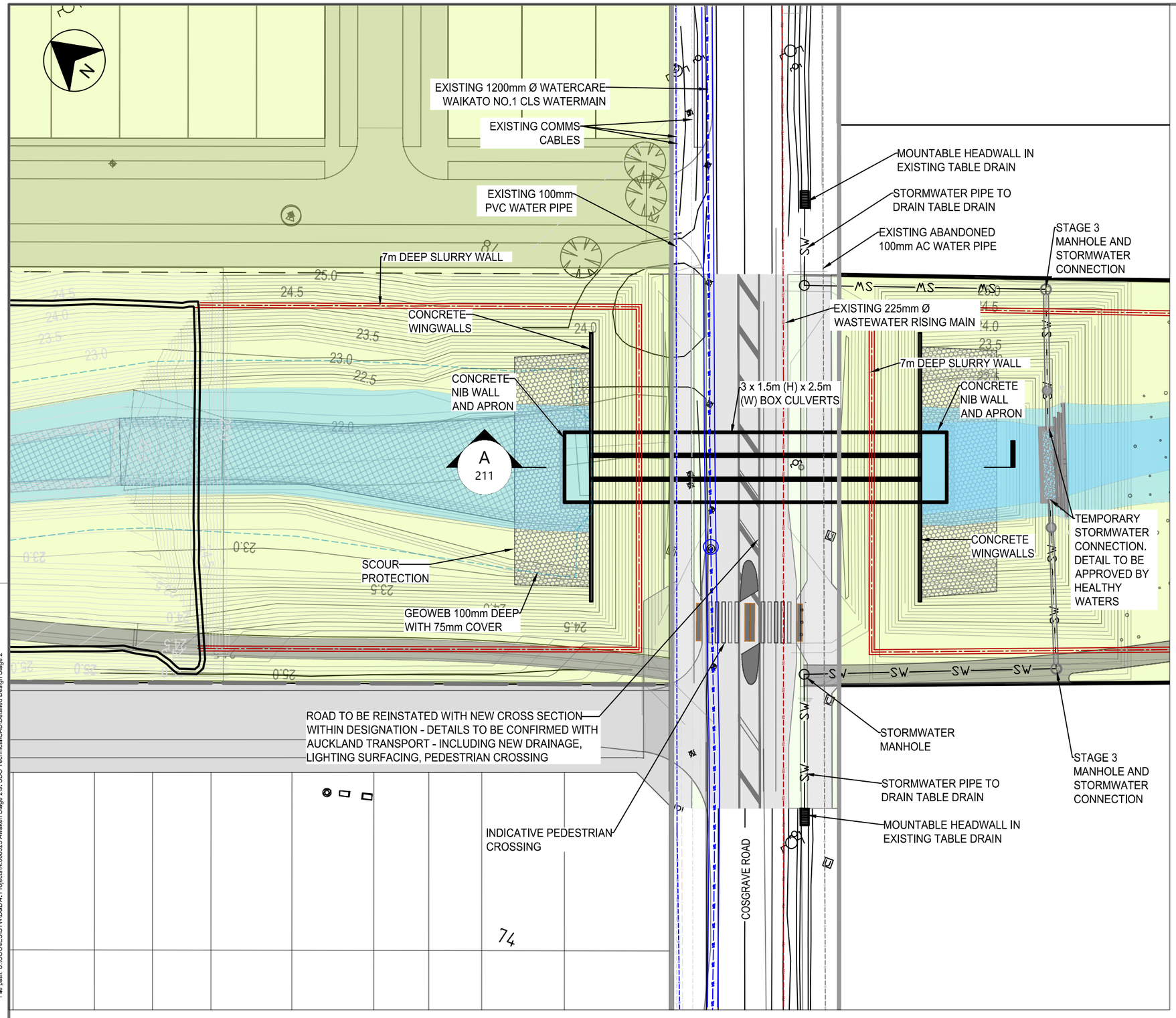

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7. SET-OUT LEVELS AND CO-ORDINATES TO BE BASED OFF THE BENCHMARK ORIGIN POINTS NOTED ON DRAWING 51-33411-V001.

DESIGN LEGEND

- PROPERTY BOUNDARIES (EXISTING)
- DESIGN CONTOURS (0.5m)
- DESIGN CONTOURS (0.1m)
- EXISTING GROUNDWATER CUT-OFF WALL (7m DEEP)
- EXISTING SHALLOW CUT-OFF WALL (3m DEEP)
- FUTURE POSSIBLE CUT-OFF WALL
- EROSION PROTECTION FOR CRITICAL LOCATIONS
- LOW FLOW CHANNEL SCOUR PROTECTION
- PERMANENT WATER LEVEL
- WORKS AREA
- TEMPORARY POND BATTERS (WORKS BY OTHERS) - REPRESENT DESIGN CONTOURS ONLY AND DO NOT REFLECT ACTUAL COMPLETED WORKS.
- INFILTRATION SWALE
- DESIGNATION BOUNDARY
- 10 YEAR ARI FLOOD LEVEL

SERVICES LEGEND

- SW — EXISTING STORMWATER PIPE
- S — EXISTING WASTEWATER PIPE
- RM — EXISTING WASTEWATER RISING MAIN
- W — EXISTING WATER PIPE
- WSL — EXISTING BULK WATERMAIN
- T — EXISTING CHORUS COMMUNICATION CABLES
- G — EXISTING GAS PIPE
- VF — EXISTING VODAFONE
- E — EXISTING UNDERGROUND POWER
- FO — EXISTING FIBRE OPTIC

SPECIMEN DESIGN

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TITLE:

AWAKERI WETLANDS - STAGE 2

COSGRAVE ROAD CULVERT - SPECIMEN DESIGN

PLAN VIEW

ORIGINAL SCALE A3	WBS No.
1:500	008325
DRAWING No.	REVISION
008325.01.210	B

Plot Date: 10-Oct-23 1:57 pm
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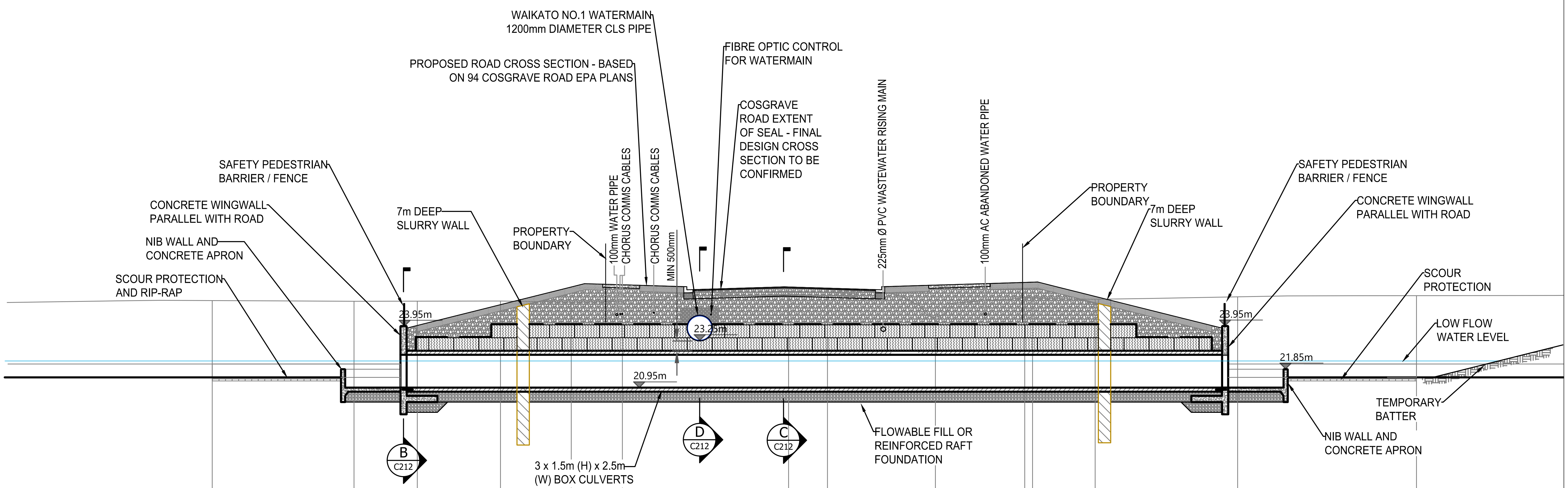

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A SECTION - COSGRAVE ROAD CULVERT LONGSECTION

Scale: 1:100

SPECIMEN DESIGN

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TITLE:
AWAKERI WETLANDS - STAGE 2
COSGRAVE ROAD CULVERT - SPECIMEN DESIGN
SECTIONS

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DRAWING No.: 008325.01.211
REVISION: B

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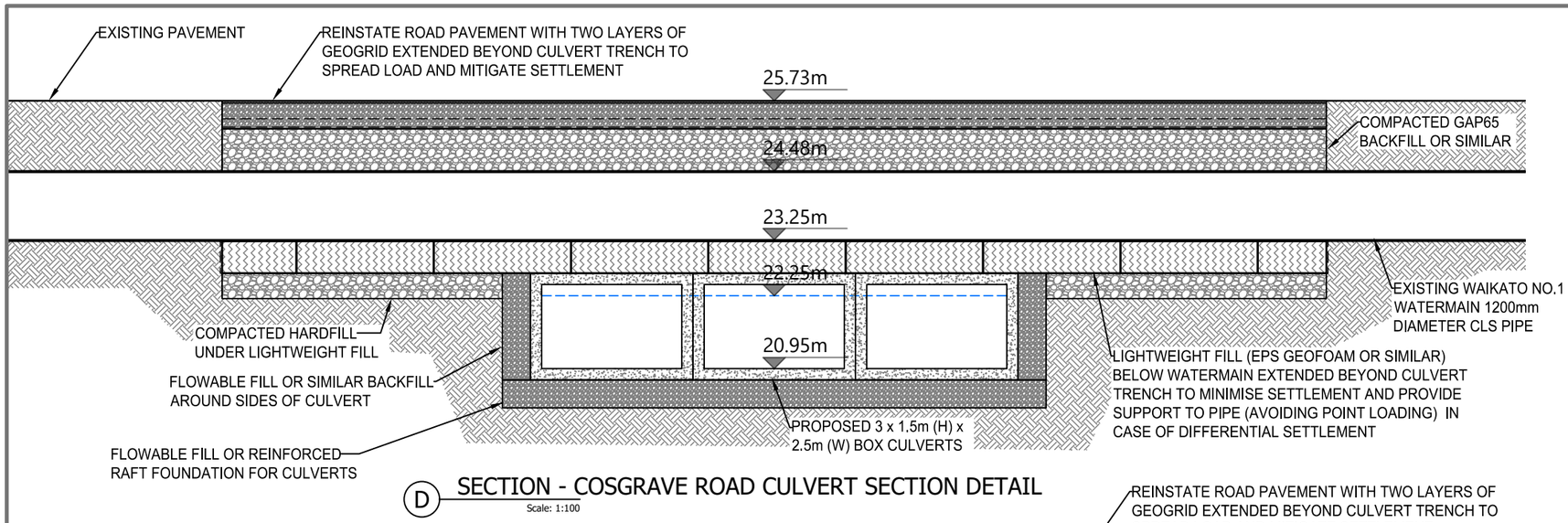

D SECTION - COSGRAVE ROAD CULVERT SECTION DETAIL

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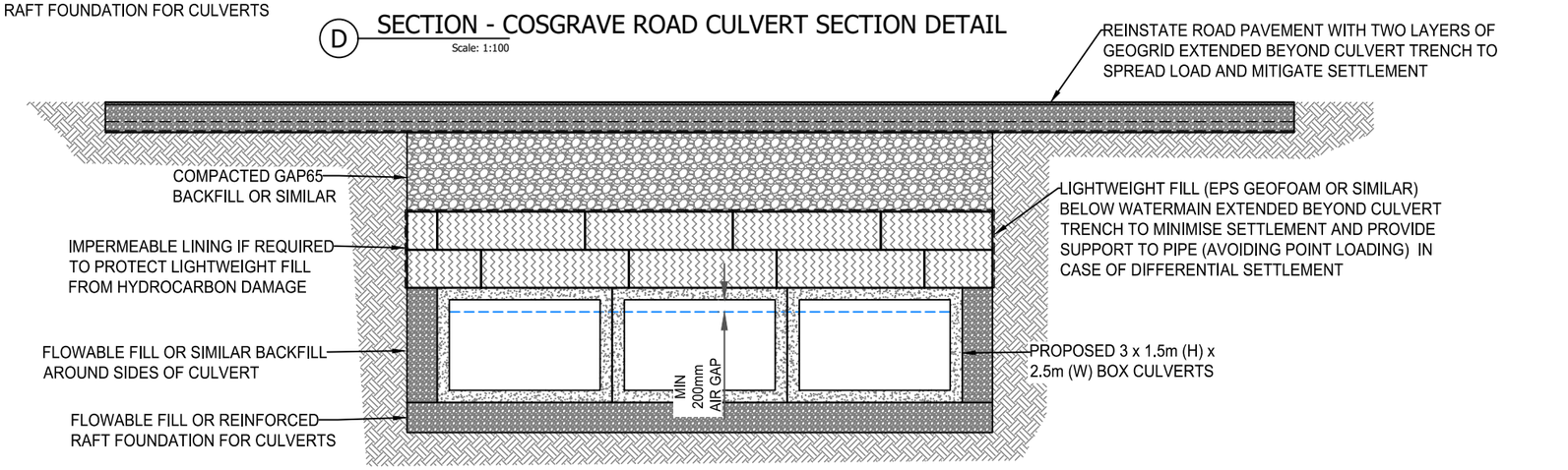

C SECTION - COSGRAVE ROAD CULVERT SECTION DETAIL

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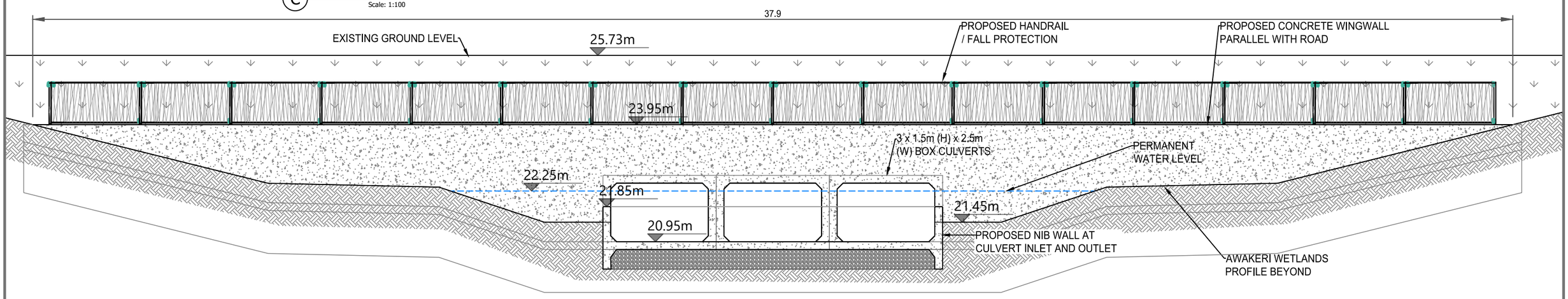

B SECTION - COSGRAVE ROAD CULVERTS - TRENCHLESS DETAIL

Scale: 1:100

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COSGRAVE ROAD CULVERT - SPECIMEN DESIGN
SECTION DETAILS

ORIGINAL SCALE A3	WBS No.
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AWAKERI WETLANDS STAGE 3

DESIGN REQUIREMENTS

Healthy Waters

Design Office

Awakeri Wetlands – Stage 3
Design Requirements

Project	AWAKERI WETLANDS STAGE 3
Document	DESIGN REQUIREMENTS
Issue and Revision Record	

Status	Issued
Version	1
Date of Issue	27/11/2023
Author and Designer	Jesse Peeters Senior Healthy Waters Specialist
Reviewed	Amelia Cunningham Design Office Team Manager
Approved for Issue	Amelia Cunningham Design Office Team Manager

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Awakeri Wetlands – Stage 3
Design Requirements

Table of Contents

1.0 Project extent and staging

The project area for the Awakeri Wetlands is shown in Figure 1 below and is broken into three stages by location:

- Stage 1: Awakeri Wetlands between Grove Road, Cosgrave Road and Walters Road
- Stage 2: Culvert crossing Cosgrave Road and connection to Stage 1
- Stage 3: Awakeri Wetlands between Cosgrave Road, Old Wairoa Road and the pond upstream of Old Wairoa Road

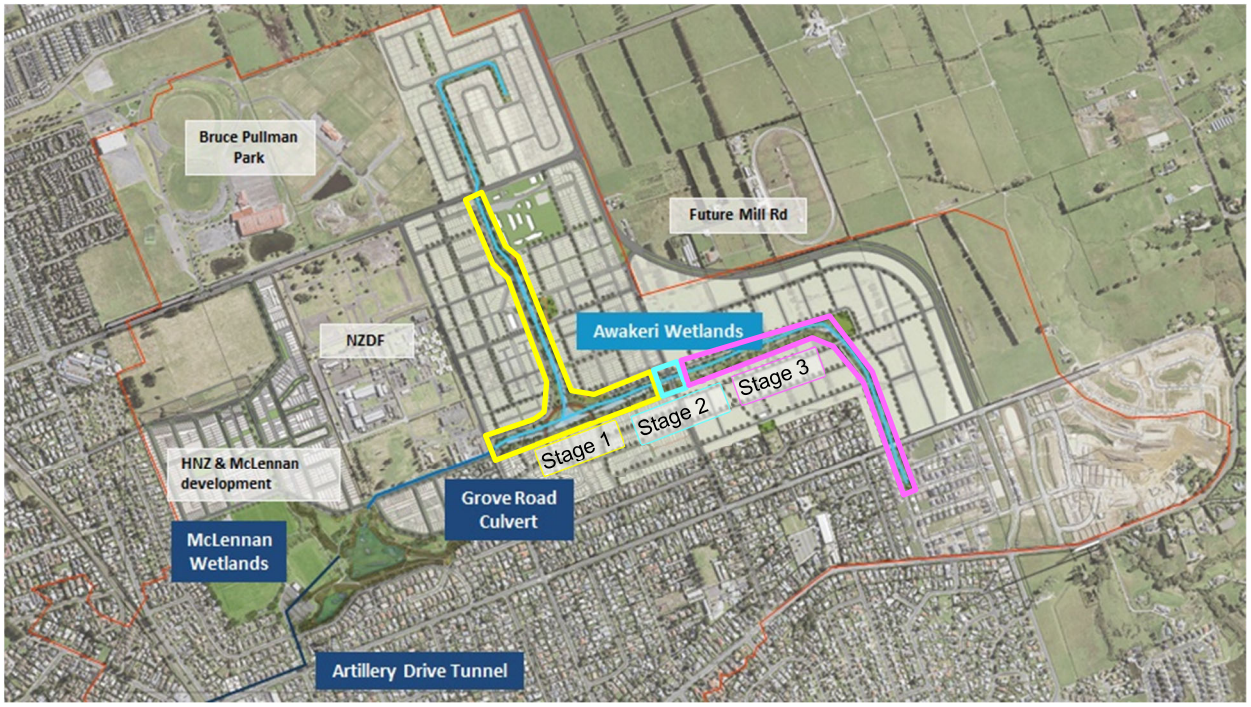

Figure 1 Awakeri Wetlands Staging

2.0 Design criteria

2.1 Standards, manuals and publications

The design and physical works shall be done in accordance with the version current at the time of the work of the following standards, manuals and publications except where amended by these Design Requirements:

a) Auckland Council Standard Specifications

b) The Building Act

c) Health and Safety in Employment Act

d) Resource Management Act

e) Maritime Safety Regulation

f) New Zealand Standards and codes

g) Auckland Council Code of Practice for Land Development

h) Auckland Transport Code of Practice

i) NZ Transport Agency Standards and Guidelines Manual

j) NZ Transport Agency Standard Specifications and Publications.

k) NZ Transport Agency Bridge Manual

Where the above does not explicitly cover all parts or issues relating to the design or construction, other codes or standards, such British, Australian or American that are applicable in respect of a part or issue shall apply where agreed by the Engineer.

Standards, manuals and publications shall be read in the following order of priority:

a) Acts of Parliament

b) The Design Requirements

c) Auckland Council Standard Specifications

d) Auckland Council Code of Practice for Land Development

e) Auckland Transport Code of Practice

f) Regulatory authority standards, specifications and guidelines

g) Australian/New Zealand Standards and guidelines

h) NZ Transport Agency specifications, standards and guidelines

i) British Standards

j) United States Standards.

Where a guideline document allows for different options or where engineering judgement is required, a design report or technical memorandum shall be provided.

2.2 Safety in Design

Safety in design must be considered throughout all stags of design and shall include a register, reporting and workshop to discuss all of the options which are to be considered for each element of the design. Safety in design considerations shall be made for all key features including but not limited to those described in Section 1.3 of this document.

Safety in design shall include input from Auckland Council Healthy Waters Operations, Community Facilities, Auckland Transport, Watercare and any other parties who will be involved with the asset throughout its design life.

2.3 Mana whenua partnership

A partnership was formed between mana whenua and the Awakeri Wetlands project design team during the design of Stage 1 of the Awakeri Wetlands. This partnership must be transitioned to the new designer to ensure that mana whenua continued to be partners of the project.

Regular huis/meetings are expected as part of this, an initial hui shall be arranged by the new designer and mana whenua to agree on hui frequency and level of involvement. The iwi who were part of the partnership are listed below. Key staff / contacts may have changed since and it is the designers responsibility to identify the current representatives of the iwi.

Table 1 Mana whenua representatives

Iwi group	Representatives	Contact
Ngati Tamaoho	Lucie Rutherford Hero Potini Zachary Sirett Edith Tuhimata (previously Ngati Te Ata Waiohua)	lucierutherfurd@gmail.com zac@tamaoho.maori.nz
Te Ākitai Waiohua	Nigel Denny Karen Wilson	kaitiaki@teakitai.com
Ngai Tai ki Tamaki	Jonathan Billington James Brown	kaitiaki@ngaitaitamaki.iwi.nz
Ngāti Te Ata Waiohua	Karl Flavell	karl.flavell@ngatiteata.iwi.nz

2.4 Watercare consultation and approvals

Watercare's Waikato No.1 watermain is in close proximity to the project and Watercare approval will therefore be needed prior to any work taking place. Previous correspondence with Watercare has indicated the following:

- Any shutdowns of the Waikato No.1 Watermain would need to be planned in advance with up to 2 years notice given to Watercare. This requirement may change depending on Watercares scheduled shutdowns so communications should be made with Watercare during the design phase to confirm.
- Finite Element Analysis (FEA) will be required and provided to Watercare to demonstrate that effects on the Waikato No.1 Watermain can be managed.
- Any design of temporary support or ground improvements shall be reviewed and approved by Watercare.

2.5 Local board consultation

The designer shall contact the local board to provide regular updates and accommodate feedback from the local board as required. The frequency of updates shall be determined based on an initial meeting with the local board to agree on frequencies.

2.6 Consents

All required consents (resource, discharge, landowner, etc.) and approvals (Watercare EPA and Works Over Approval, Auckland Transport EPA, etc) shall be obtained by the designer.

2.7 Services

The designer shall be responsible for incorporating the design of any protection or relocation of any known existing services or proposed future services which conflict or cross the proposed work, including but not limited to:

- Fibre optic cables
- Wastewater pipes and wastewater rising mains
- Local watermains
- Overhead power lines
- Underground power cables
- Underground communications cables
- Roads

Approvals shall be obtained from all relevant utility providers where required and clearances shall be provided as per the relevant standards and utility provider requirements.

2.8 Minimum design requirements

General design features of the Awakeri Wetlands Stage 3 scope are shown in the Awakeri Wetlands Stage 3 Specimen Design Drawings. The Specimen Design Drawings shall be referred to and significant deviations from the key features shown in these drawings shall be documented and approved by Auckland Council.

It is the designer's responsibility to determine the final design criteria, however the following section provides minimum requirement and sets out Auckland Councils expectations.

2.8.1 Key features of design

Stage 3 of the Awakeri Wetlands must include the following features:

- Low flow channel and wetland bench
- Erosion protection
- 2.5m wide shared path
- Boardwalks
- Staircases
- Informal stepping logs
- Removable bollards
- Overland flowpaths
- Weirs
- Stormwater connections
- Road culvert crossing
- Debris screen

- Groundwater cut-off barrier
- Planting, mulching and erosion control matting
- In-situ swamp kauri

2.8.2 Layout, framework plan and wider context

A framework plan (shown in Figure 2 was prepared by Auckland Council for development adjacent to the Awakeri Wetlands to ensure co-ordination between the Awakeri Wetlands layout and the adjacent development layout.

A park and neighbourhood centre is proposed at the eastern end of the Awakeri Wetlands and interaction between the wetland, park and neighbourhood centre is required. The designer shall communicate with Auckland Council Parks to ensure the park and wetland are well planned and co-ordinated.

Figure 2 Awakeri Wetlands Stage 3 Framework Plan

2.8.3 Design Life

A design life of not less than 100 years shall be allowed for infrastructure assets unless agreed otherwise with Auckland Council, taking into consideration the low pH / aggressive ground conditions such as potential acid sulphate soils. Further information is available in the *Hydrogeology Assessment of Effects (GHD, 2016)* and the *Acid Sulfate Soils Management Plan (GHD, 2017).*

2.8.4 Design flow

The design of the culvert shall be able to convey up to the 50% AEP, 10% AEP and 1% AEP storm event without resulting in:

- Flooding of the proposed shared paths or boardwalks in the 50% AEP event
- Surcharging of the pipe network in the developments beyond the Awakeri Wetland boundary during the 10% AEP event.
- Surcharging of overland flowpaths of developments within the catchment during the 1% AEP event.

Catchment flows for Stage 3 of the Awakeri Wetlands are outlined in Table 2, provided the catchment, development and impervious area assumptions from the Awakeri Wetlands Stage 1 Detailed Design Report are met. If these assumptions are altered in the developers proposal, then the flows shall be recalculated based on the updated assumptions.

Table 2 Peak flows in the Awakeri Wetlands (Stage 3)

	MIKE11 modelled peak flow (m^3/s)		
Chainage (m)	50% AEP	10% AEP	1% AEP
500	5.7	14.6	23
600	5.6	14.3	22.6
700	5.2	13.5	21.3
800	4.9	12.6	19.9
900	4.5	11.6	18.5
950	4.3	11.2	18.0
1000	1.9	5.8	9.6
1100	1.8	5.6	9.2
1200	1.6	5.3	8.7
1300	1.5	5.1	8.2
1400	1.4	5.0	7.9
1500	1.4	3.2	4.7

2.8.5 Awakeri Wetlands low flow channel and wetland bench

The Awakeri Wetlands low flow channel is typically 800mm deep and varies in width. The invert of the channel is flat, with a step in elevation at each weir location. Some localised deeper areas are proposed.

On the edges of the low flow channel is a wetland bench where the water level varies from 200mm to 0mm (refer to Figure 3). The wetland bench provides a safety warning prior to reaching the deeper water, and includes wetland planting for shade, habitat and water quality benefits.

The low flow channel and wetland bench shall be as per the Specimen Design Drawings unless modified and approved via the resource consent process. In particular levels shall not be modified as these form the basis of the groundwater and settlement effects assessment.

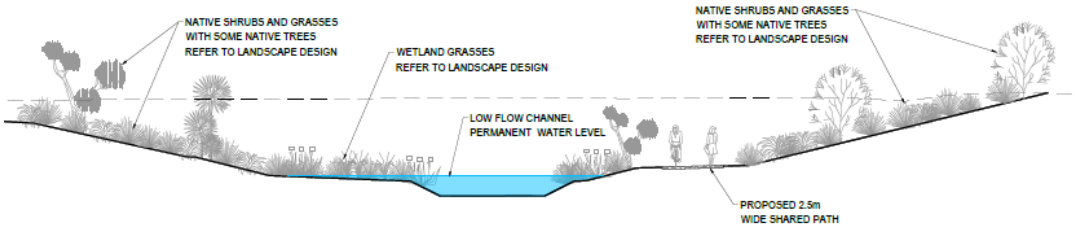

Figure 3 Typical cross section

2.8.6 Scour and erosion protection

The design shall include the 2 year, 10 year and 100 year ARI peak flow scenarios for assessment of scour and erosion potential by carrying out a permissible shear stress analysis. Scour protection shall be designed and installed at key areas along the channel where the applied shear stress exceeds the permissible shear stress of the surface and a structure or boundary is at risk of undercutting or damage.

Large riprap and concrete shall be avoided where possible for scour protection. Naturalised methods of scour protection such as planting is preferred. Oversizing culverts to minimise velocity, energy and shear stress is preferred over providing scour protection to dissipate high energy. Geosynthetic materials to reinforce plant roots such as geoweb and enkamat are preferred over hard engineering solutions.

Consideration to lining the underwater base of the wetland with geotextile filter fabric, enkamat and gravel shall be made to avoid scour, discharge of sediment and soft exposed peat which can be a safety hazard. This also provides a distinct base that can be identified during maintenance or desilting.

The specimen design drawings provide a high level indication of where erosion protection materials are likely to be required based on protecting hard assets. Monitoring of unreinforced areas is an acceptable approach where scour does not risk undermining structures or properties.

Figure 4 Geoweb scour protection, enkamat and gravel lining for underwater wetland base

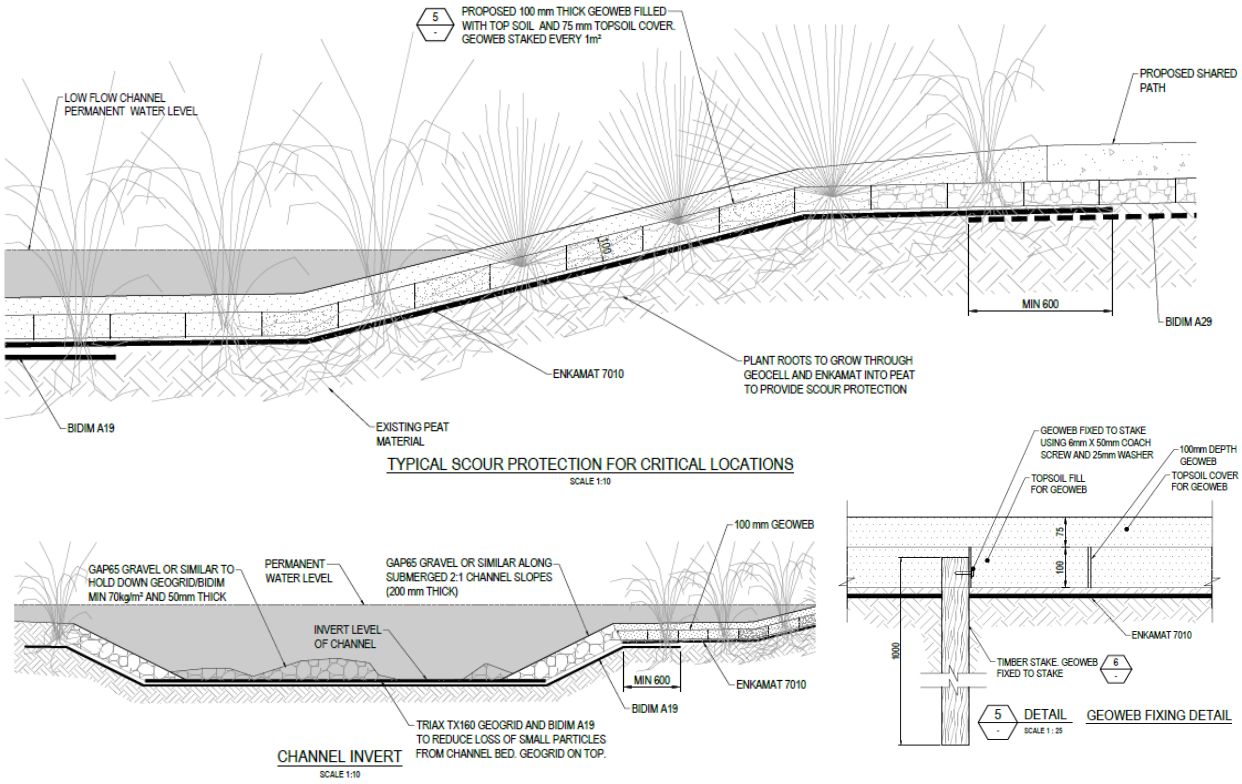

Figure 5 Scour protection details for Stage 1 of the Awakeri Wetlands

2.8.7 Shared path

A shared path must be designed to follow the approximate alignment shown in the Specimen Design Drawings. Final alignments may be adjusted to align with the final development lot layout. The shared path shall be a similar detail to the shared path in the Awakeri Wetlands Stage 1. The structural design of the path shall be provided by the designer, with the following minimum requirements:

- Minimum 2.5m width
- Maximum gradients in accordance with Auckland Transport and Auckland Council Shared Path standards.
- Design loading to be determined by structural engineer and shall include allowance for light maintenance vehicle loading, defined as:
    - A vehicle or combination of vehicles having a gross vehicle weight not exceeding 7.2 kN consisting of 3 axle loads of 2.4 kN each, spaced 1500 mm apart. Each axle load shall consist of two wheel loads of 1.2 kN each spaced at 500 mm centre to centre. Each wheel load shall be applied over a square not greater than 150 mm x 150 mm. Typical examples include a power carrier or a 4-wheel motorcycle towing a trailer.
- Exposed aggregate finish using the same river pebble aggregate (Longburn Pebble)
- Red oxide (Peter Fell 468 or similar) shall be added to the concrete mix at key areas to act as a warning for pedestrians / cyclists at path intersections and at entry/exits to boardwalks. Red coloured strips shall be created in a similar way to the Awakeri Wetlands Stage 1 design. Final location of red strips to be agreed with Auckland Council.

- Concrete strength to be specified by designer.
- River stone drainage channel on the uphill side of the path to prevent groundwater flowing across the path surface
- Foundation consisting of geoweb filled with drainage metal to allow flow of water underneath path without loss of material (similar to permeable paving basecourse)
- Geotextile under geoweb to prevent drainage metal mixing with subsoils
- Control joints at 3m spacing
- Dowel bars at control joints to minimise movement

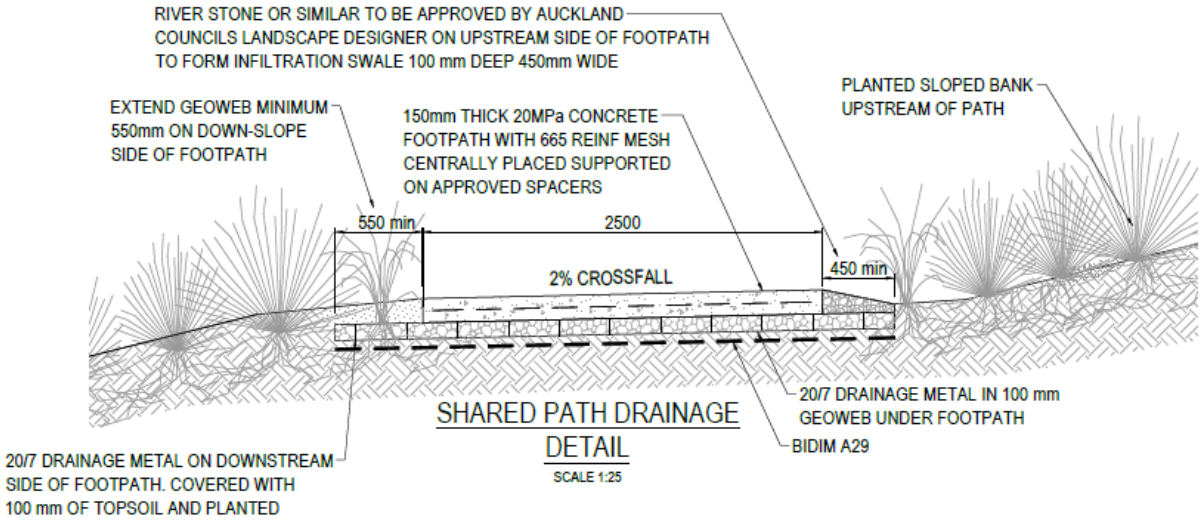

Figure 6 Footpath detail from Awakeri Wetlands Stage 1

Figure 7 Footpath photo from Awakeri Wetlands Stage 1

2.8.8 Staircases

Staircases may be required where the maximum gradient of a path exceeds allowance longitudinal slope for shared paths. If a staircase is proposed, there must be an alternative accessible route

available. Staircases shall be a similar detail to the staircases in the Awakeri Wetlands Stage 1. Minimum requirements for the staircases are:

- Red coloured concrete to match the warning strips on shared path.
- Exposed aggregate tread.
- Hand rail matching the Awakeri Wetland Stage 1 staircases.
- Cycle ramp to match Awakeri Wetlands Stage 1 staircases.
- Minimum 2.5m wide to match shared path.
- Tread depth to be 360mm and tread height to be 120mm.

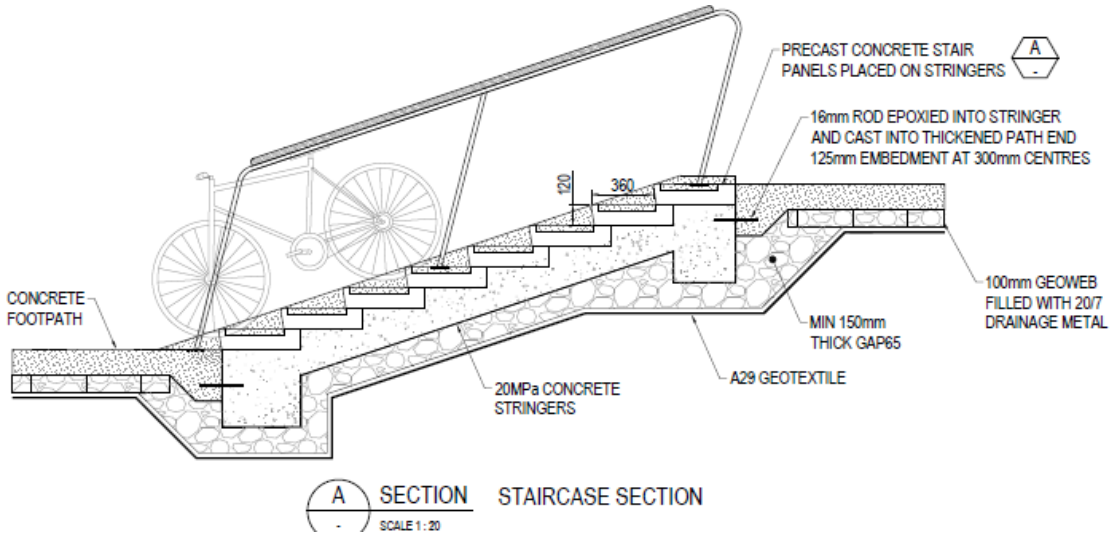

Figure 8 Staircase detail from Awakeri Wetlands Stage 1

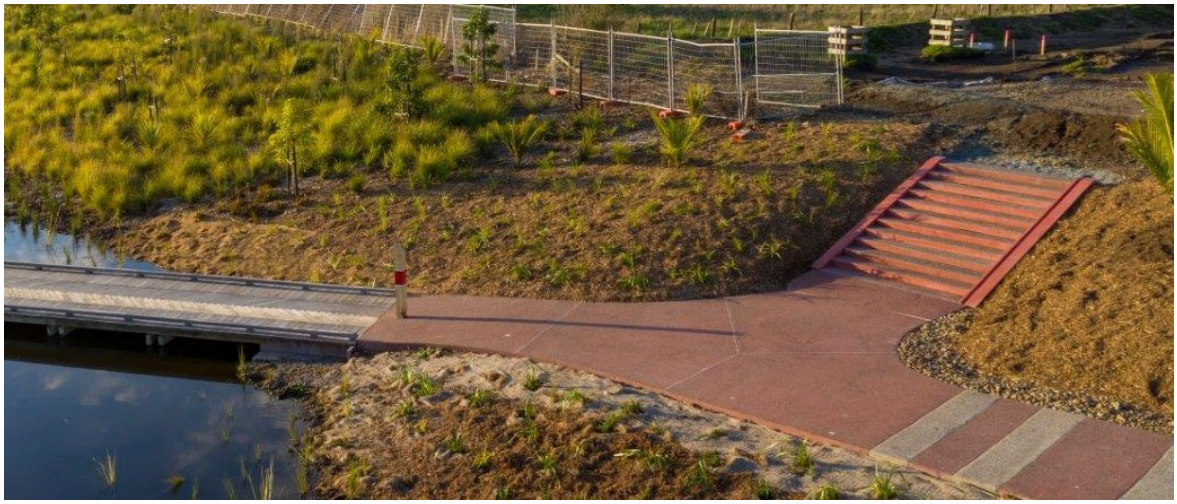

Figure 9 Photo of staircase from Awakeri Wetlands Stage 1

2.8.9 Informal stepping logs

Informal stepping logs shall be designed to provide informal access from the shared path to useful connections within the development or points of interest within the Awakeri Wetlands, where a formal

shared path or staircase is not required. The informal stepping logs shall use a similar detail to the informal stepping logs in Stage 1 of the Awakeri Wetlands, including:

- Two sizes of stepping logs to be used to create variability:
    - Size one: 1500mm (L) x 250mm (W) x 150mm (D)
    - Size two: 2000mm (L) x 300mm (W) x 200mm (D)
- Leading edge and face needs to be refined with a 15-20mm chamfer around top face.
- Timber for stepping log to be Eucalyptus or similar approved.
- Stepping logs to have concrete footings which shall be specified by the designer.

Figure 10 Informal stepping logs photo from Awakeri Wetlands Stage 1

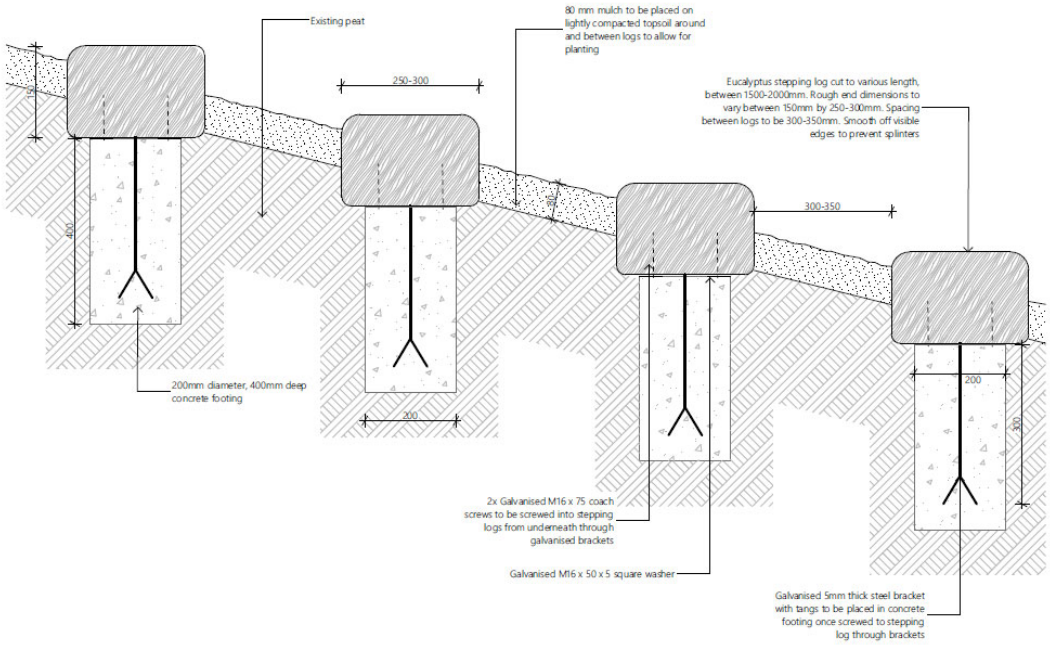

Figure 11 Stepping log detail

2.8.10 Boardwalks

Boardwalks shall be designed at the approximate locations shown in the Specimen Design Drawings. Final localities/alignments may be adjusted to align with the final development lot layout. The boardwalks shall use a similar detail to the Awakeri Wetlands Stage Boardwalks including:

- Straight decking shall be 140mm x 45mm decking.
- Weaving pattern decking shall be 90mm x 45mm decking.
- Timber decking pattern with the direction weaving pattern, same Tonka hardwood timber species, stainless steel plate running longitudinally between the decking pattern.
- Timber kerb.
- Shallow concrete pad foundations designed by a structural engineer.
- Width to be 2.5m between inside of kerbs.

Figure 12 Boardwalk decking (left) and Boardwalk foundations (right)

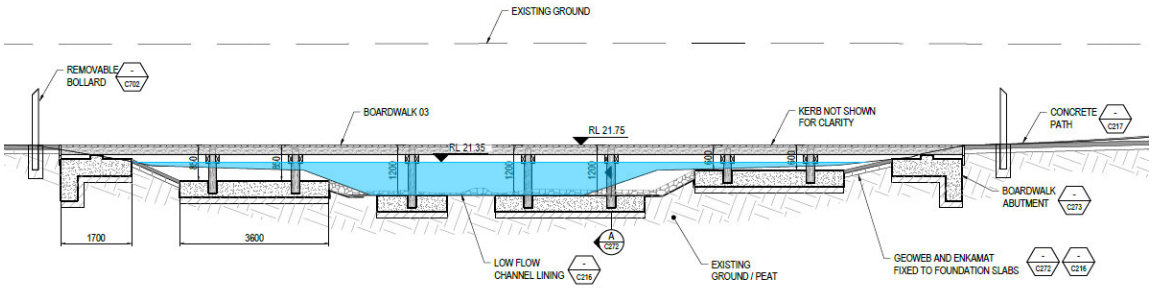

Figure 13 Boardwalk detail from Awakeri Stage 1

2.8.11 Weirs

Weirs shall be designed at the locations and to the levels shown in the Specimen Design Drawings. No changes must be made to this unless the effects are assessed and approved as part of a resource consent.

Weirs shall use a similar detail to the weirs installed in Awakeri Wetlands Stage 1 including:

- Minimum 6m deep PVC sheetpiles, final depth to be specified by the designer based on geotechnical and/or hydrogeological advice.
- Scour pool with 4m deep PVC sheetpiles around perimeter and concrete base.

- Hardwood (Tonka) timber capping on all sheetpiles.
- Fish passage to be designed with input from a qualified freshwater ecologist and to have a similar design to those in Awakeri Wetlands Stage 1.
- Weirs shall be water tight to maintain an upstream water level at the crest of the timber capping.
- Allowance for stormwater connections through the scour pool sheet piles.

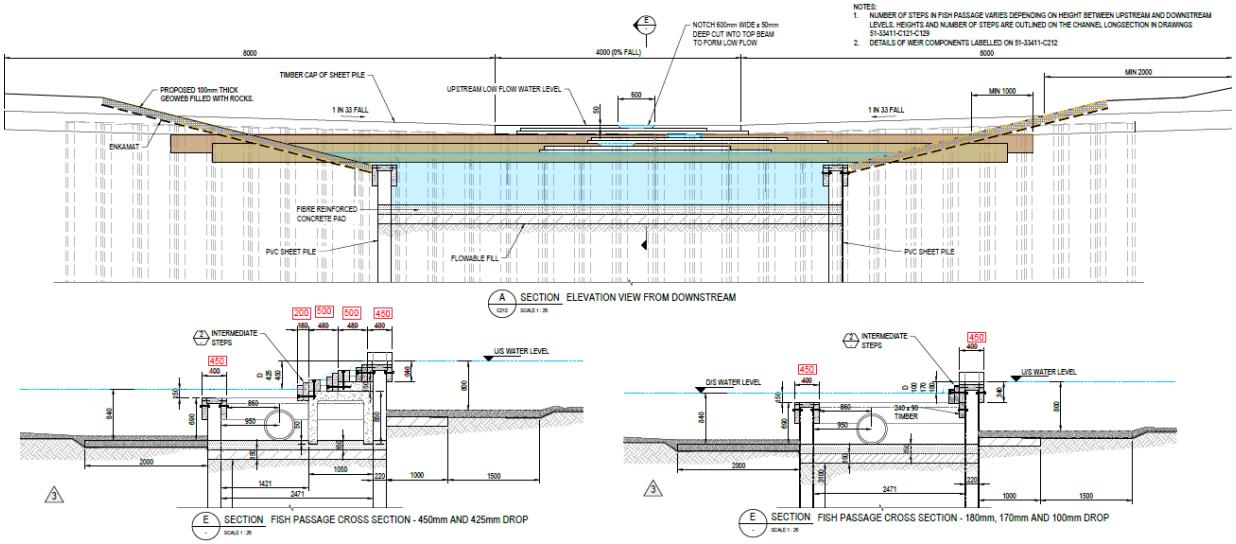

Figure 14 Weir detail from Awakeri Wetlands Stage 1

Figure 15 Photo of weirs from Awakeri Wetlands Stage 1

2.8.12 Stormwater Connections

Stormwater pipes shall only be connected into the Awakeri Wetlands at weir locations. One stormwater pipe connection shall discharge into each side of each weir scour pool. The stormwater connection shall use a similar detail to the stormwater connections in Awakeri Wetlands Stage 1 including:

- Low flow pipe connection to the scour pool to have a maximum size of 500mm OD PE100 SDR17. Slope of pipe to be determined by designer.

- Overflow manhole with scruffy dome at the upstream end of the low flow pipe. The lip level of the manhole shall be set below the 10 year ARI event level where possible. Final levels to be determined by the designer.
- Stormwater outfall pipe upstream of the overflow manhole to be made of PE100 SDR17. Slope and pipe diameter to be determined by the designer. This pipe will penetrate the slurry wall and a specific detail is required to reseal the slurry wall around the pipe. Refer to Awakeri Wetlands Stage 1 detail.
- Pipe bedding and support to be determined by the designer, considering soft peat soils and other geotechnical ground conditions.
- Manhole specifications to be determined by the designer, considering soft peat soils, low pH, potential acid sulphate soils. This could include using micro-silica concrete additives, increased concrete strength, Hydura products etc.

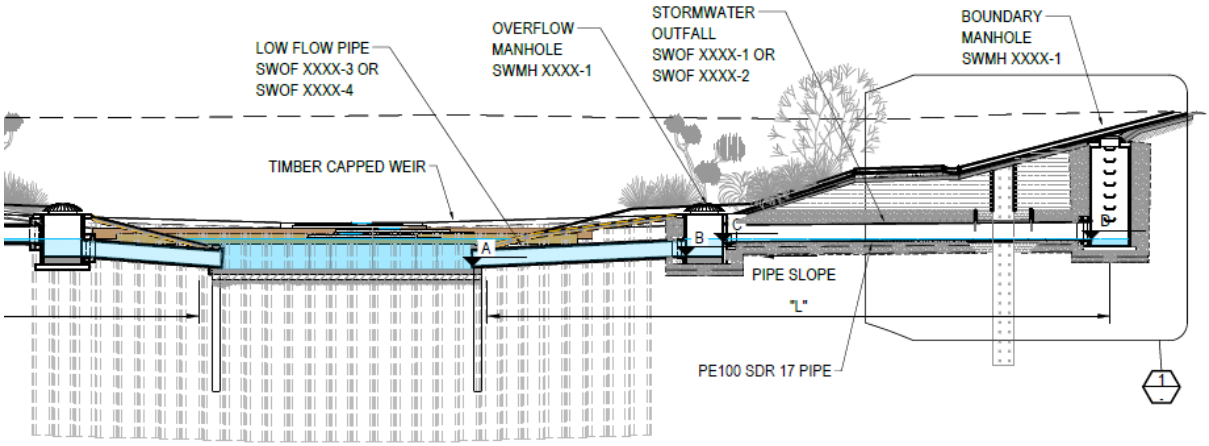

Figure 16 Stormwater connection detail from Awakeri Wetlands Stage 1

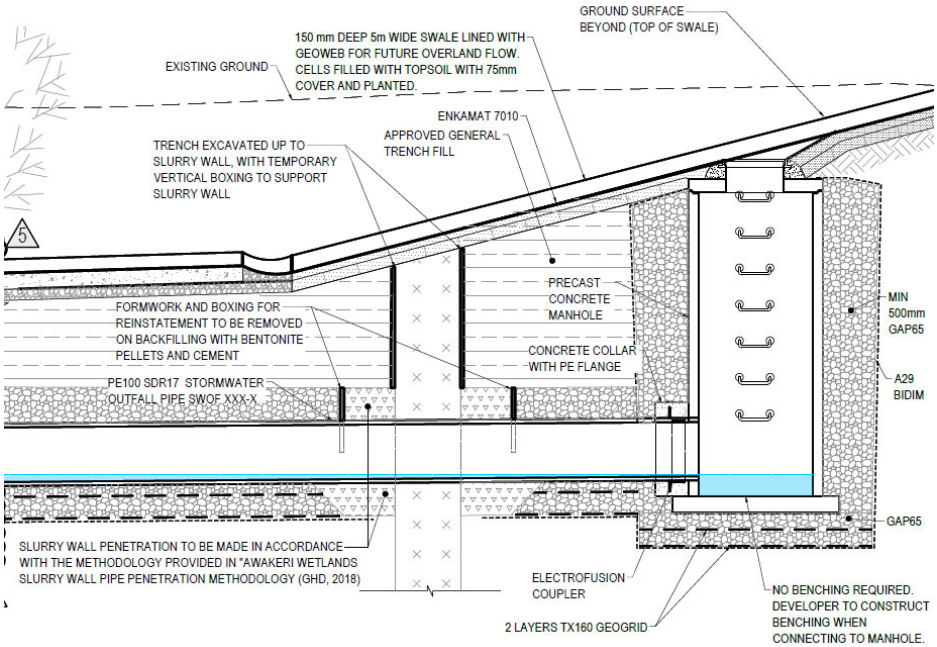

Figure 17 Slurry wall penetration detail

2.8.13 Overland flowpaths

Overland flowpaths within the adjacent development shall be designed as per the Auckland Council Stormwater Code of Practice. Overland flowpath locations shall be co-ordinated with the development lot and road layouts and connections shall generally be located to align with the weirs such that water flows towards the bubble up manholes of the stormwater connections. This allows the scour protection around the bubble up manholes to be utilised for overland flow.

Overland flowpath connections shall be designed to match the Awakeri Wetlands Stage 1 design including:

- A dip in the shared path with a maximum gradient of 1:10
- Minimum base width of 2m, to be confirmed by designed based on flow rate.
- Erosion protection along the overland flowpath to be specified by the designer but to generally included planting of native grasses as the main erosion control method. Geoweb and enkamat can be used in conjunction with native grass roots to provide additional protection.

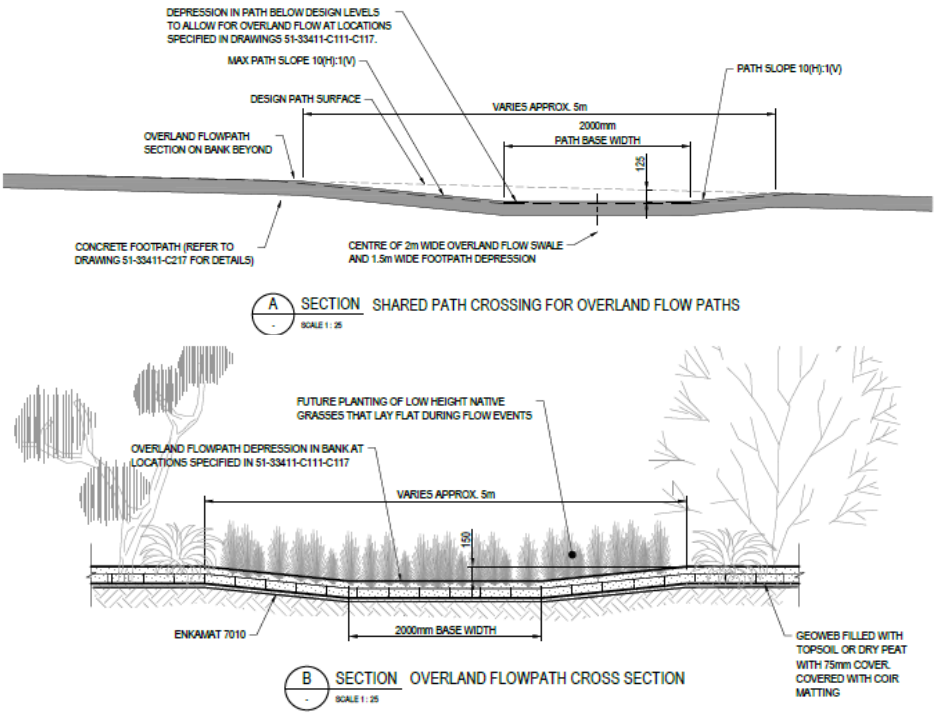

Figure 18 Overland flowpath detail from Awakeri Wetlands Stage 1

2.8.14 Wastewater crossings

Wastewater crossings may be required to convey wastewater within the adjacent developments. The designer shall determine any wastewater pipe crossing locations and design these to avoid future excavations within the Awakeri Wetlands area.

2.8.15 Wastewater connections

A toilet block is likely to be required at the adjacent park. A wastewater connection shall be designed for the toilet block and any other facilities that require a wastewater connection within the project or adjacent park.

2.8.16 Culverts

A road culvert crossing shall be designed at the approximate location shown in the Specimen Design Drawings. The following section provides minimum requirements for the design of the road crossing culvert.

2.8.16.1 Blockage assessment

A blockage assessment shall be provided which outlines the likelihood and consequence of various blockage scenarios for the culverts. The final allowance for blockage must be agreed with Auckland Council prior to finalization of the design.

Auckland Councils preliminary view is that a safety grill on the inlet and outlet of the culvert would not be required due to its expected short length, large height/width and location in the upper part of the catchment, however the requirement of a safety grill would need to be considered as part of the design.

2.8.16.2 Minimum external design loads

The designer shall determine the design load parameters for the crossing with Auckland Transport.

Minimum design load parameters are available in Table 3.

Table 3 Live loads

Item	Load allowance	Reference
HN vehicle loads	3.5 kPa x 3 m wide uniform load *plus* 2 x 120 kN axle loads at 5 m ctrs, or 12 kPa surcharge pressure (as appropriate)	Bridge Manual (SP/M/022) – Section 3.2.2 and Figure 3.1 and Section 3.4.12
HO vehicle loads	3.5 kPa x 3 m wide uniform load *plus* 2 x 240 kN axle loads at 5 m ctrs, or 24 kPa surcharge pressure (as appropriate)	

2.8.16.3 Culvert alignment (vertical/horizontal)

The vertical and horizontal alignment of the culverts shall be such that they do not adversely affect the integrity of any existing structures and considers safety, operation and maintenance considerations.

The design shall consider the safety, operational and maintenance aspects associated with the depth of water in the culvert. The culvert shall generally be placed with its invert level matching the bed level of the wetland, and therefore will include 800mm of permanent water within it. The remainder will be an air gap to allow debris to flow through.

2.8.16.4 Culvert groundwater ingress and settlement considerations

The following groundwater and settlement considerations shall be made when designing culverts within Stage 3 of the Awakeri Wetlands:

- The infrastructure conveying water under the road shall be fully sealed to prevent leakage of groundwater and sediments into the culvert over the design life of the structure. This requirement is important for mitigating the risk of groundwater drawdown and settlement of the surrounding peat soils.
- Post tensioning shall be considered if the structure has joints that are at risk of leaking due to ground movement.
- Settlement of the structure (due to the weight of the structure and/or backfill) shall be assessed and the impact of any predicted settlement on adjacent structures and services shall be considered and mitigated. Use of lightweight backfill shall be considered where/if appropriate.
- Settlement of the ground around the structure (due to any anticipated effects on adjacent groundwater and soil) shall be assessed and the impact of any predicted settlement on adjacent structures and services shall be considered and mitigated.
- Lightweight backfill such as polyrock may be required to reduce the load on subsoils and manage ground settlement.
- Stiffening of the culvert bedding may also be required by including a geogrid raft or flowable fill to mitigate differential settlement across the culvert.

2.8.16.5 Inlet and outlet structures

An inlet / outlet structure shall be provided at each end of the culverts. The headwall/wingwalls shall be similar to the Grove Road culvert outlet wingwalls and shall include:

- Wingwalls/headwall shall be parallel with the road (ie. straight concrete retaining walls)
- Material shall be concrete with an exposed aggregate finish (sandblasted or similar)
- Height of the headwall shall be minimised to minimise the fall height from above.
- Mitigation to discourage access by the public, such as planting.
- Access to be provided for clearing blockages
- Safety barrier / fence to mitigate fall height. Safety fence to match the fence on top of the Grove Road Culvert outlet structure at the McLennan Wetland.
- Erosion / scour protection. Type and extent of erosion protection to minimise visual impact on surrounding environment and align with materials used in Stage 1 of the Awakeri Wetlands.

Figure 19 Culvert headwall/wingwalls

2.8.16.6 Buoyancy

The culvert shall be designed for the effects of buoyancy for suitable scenarios determined by the designer based on nearby groundwater monitoring data and Awakeri Wetlands design information.

2.8.16.7 Road reinstatement

The design shall include details for road reinstatement after the culvert is installed which shall be agreed with Auckland Transport, but at a minimum shall include:

- A pedestrian crossing shall be designed and approval shall be obtained by Auckland Transport any other relevant parties. The designer shall liaise with Auckland Transport and any other relevant parties to determine a suitable crossing detail (ie. signalised, island, zebra crossing).
- The road corridor shall be upgraded to align with the future road cross section. The designer shall liaise with Auckland Transport to determine the future road cross section details.

2.8.17 Dewatering

Any dewatering systems used shall be designed and operated so that related settlement does not exceed limits prescribed by the Resource Consent Conditions in both long term (post-construction) and short term (during construction) scenarios.

The designer shall also consider how groundwater will be managed during construction.

2.8.18 Groundwater cut-off barrier

Previous assessments, as described in the *Hydrogeology Assessment of Effects (GHD, 2016)* have indicated that a vertical groundwater cut-off barrier (7m deep) is required around the perimeter of certain areas of the wetland excavation.

A shallower slurry wall (3m deep) was also proposed along the perimeter of remaining (lower risk) areas to manage groundwater during construction. The location of the proposed slurry wall is shown in the Specimen Design Drawings.

Groundwater barrier requirements

- The cut-off barrier is required to have a maximum permeability of 1 x 10^{-8} in order to mitigate groundwater drawdown to an acceptable level.
- The base of the excavation during culvert construction shall also have a maximum permeability of 1 x 10^{-8}.
- The vertical groundwater cut-off barrier was proposed as a bentonite-cement slurry wall. The slurry wall was proposed as minimum 600mm wide and 7.0m deep.
- The mix design for the slurry wall shall be provided by the designer, with lab testing and in-situ field testing to demonstrate that the required permeability can be met and sufficient curing will be achieved.
- Final design of the slurry wall shall be provided by the designer.

Quality assurance

- In-situ QA coring and sampling shall be undertaken during construction to confirm that the required permeability requirements have been met for the installed wall.
- For every hundred meters of the slurry wall, the proper curing of the slurry wall shall be checked as follows:
    - Coring should be done no earlier than 14 days in at least 4 locations in the central axis of the cut off wall and all the way to the full depth of the cut off wall.
    - Coring will enable checking of cut off wall consistency, depth, verticality and width. Holes to be grouted back
    - Number of locations to be drilled will be reviewed and may change in view of the encountered results, as further coring could be necessary.
- Where QA coring or sampling shows non-compliance, the slurry wall shall be repaired to achieve compliance. This may involve re-excavating and reinstalling the slurry wall in some sections.

Contingency plan for obstructions

A contingency plan is required where the slurry wall installation encounters underground obstructions. The Engineer shall be consulted when an obstruction is encountered to determine an appropriate action. Actions could include:

- Investigating the extent of the obstruction through coring, or additional excavating
- Coring through timber obstructions using an excavator mounted coring tool.
- Cutting and removing the obstruction.
- Realigning the cut-off barrier to go around the obstruction.

Figure 20 Slurry wall photo

2.8.19 Debris Screen

A debris screen shall be designed immediately upstream of the Cosgave Road culvert. The purpose of the debris screen is to catch large objects that float down the network during large storms which could block the culvert inlet such as mattresses, cars, woody debris, vegetation or other large items.

The debris screen shall use a similar detail to the debris screen in Awakeri Wetlands Stage 1 including:

- Dead hardwood gum trees or similar embedded into the ground in an array designed to capture large debris.
- A maximum opening size of 800mm shall be achieved along the screen in regards to distance between each log post.
- Suitable embedment and a concrete ring for support shall be designed to ensure the logs are stable during operation.
- Dead trees shall also be designed to be easy to replace after damage or at the end of their design life.
- Additional dead trees shall be placed around the screen to achieve a natural aesthetic, rather than installing the minimum number of dead trees to achieve performance.
- Ends of dead trees shall be charred to represent a burnt forest appearance to align with the Awakeri Wetlands Stage 1 design.

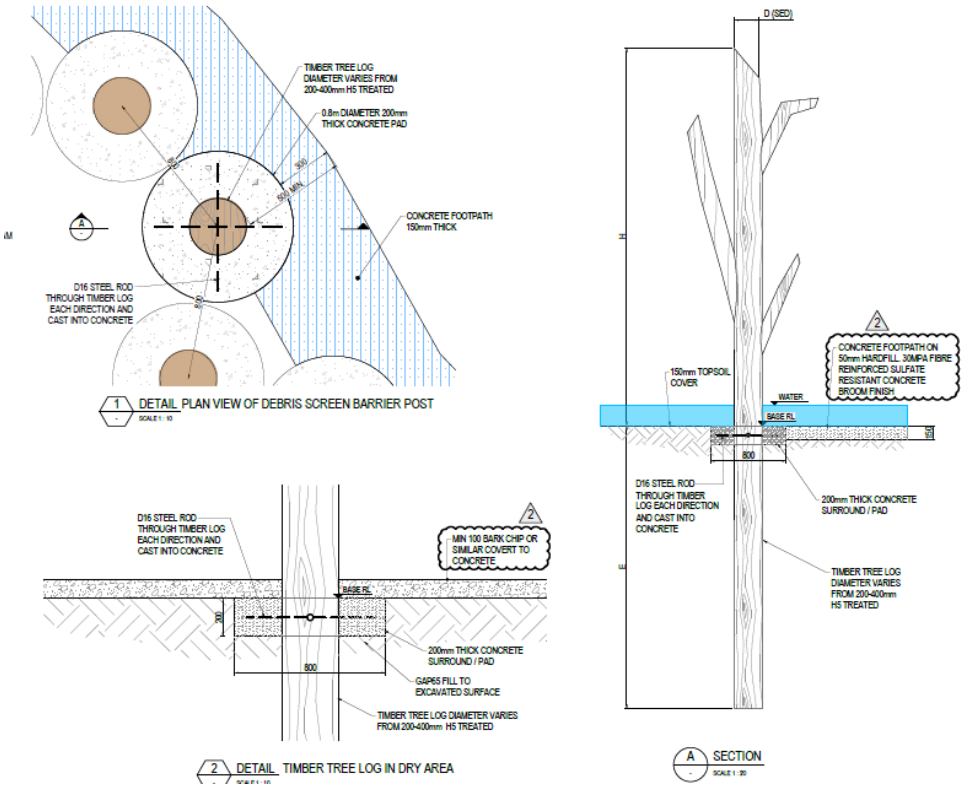

Figure 21 Debris screen detail from Awakeri Wetlands Stage 1

Figure 22 Debris screen from Awakeri Wetlands Stage 1

2.8.20 Water supply

A water supply pipeline shall be designed within the project to service drinking water fountains, toilets and any other assets within the project or adjacent park area that require water. This shall include at least one water meter connection at the boundary, or possibly multiple if multiple connection points achieves a better design outcome.

2.8.21 Lighting

Lighting design for the project shall be provided, including electrical design. This shall include for the adjacent park area.

2.8.22 Signage and wayfinding

Signage and wayfinding shall be designed to match the signage and wayfinding design for Awakeri Wetlands Stage 1. Locations and quantity of signs shall be determined by the landscape designer and reviewed/approved by Auckland Council. Signage and Wayfinding shall include:

- Entry plinth signs
- Entry blade signs
- Directional bollards
- Flood warning steel plaques
- Information boards

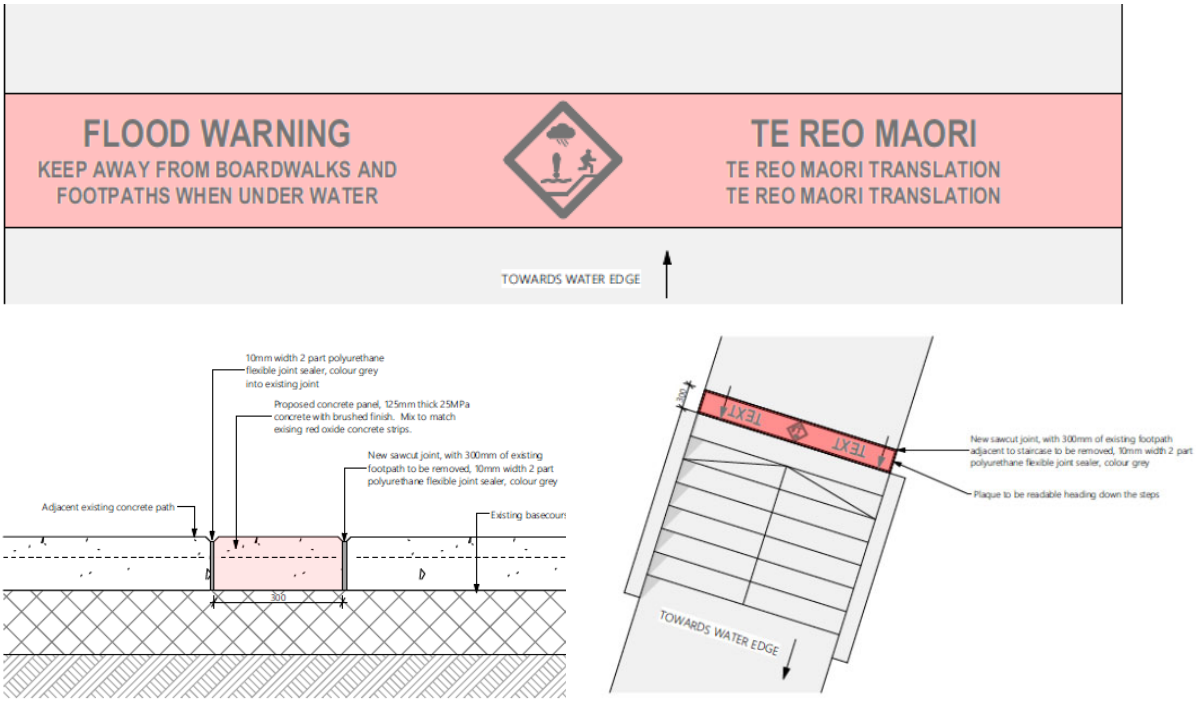

Figure 23 Flood warning plaque detail from Awakeri Wetlands Stage 1

2.8.23 Furniture

Furniture shall be designed to match the furniture design for Awakeri Wetlands Stage 1. Locations and quantity of furniture shall be determined by the landscape designer and reviewed/approved by Auckland Council. Furniture includes:

- Streetscape Statesman seats and benches
- Streetscape Mondo Accessible Picnic Set
- Streetscape Pan Bin
- Scope Cycle Rack

- Blok Drinking Fountain

Figure 24 Furniture details from Awakeri Wetlands Stage 1

2.8.24 Removable bollards

Removable bollards shall be designed to match the Awakeri Wetlands Stage 1 design. Removable bollards shall be located at each shared path entrance and at each end of each boardwalk. Refer to the Awakeri Wetlands Landscape Design Drawings for more details.

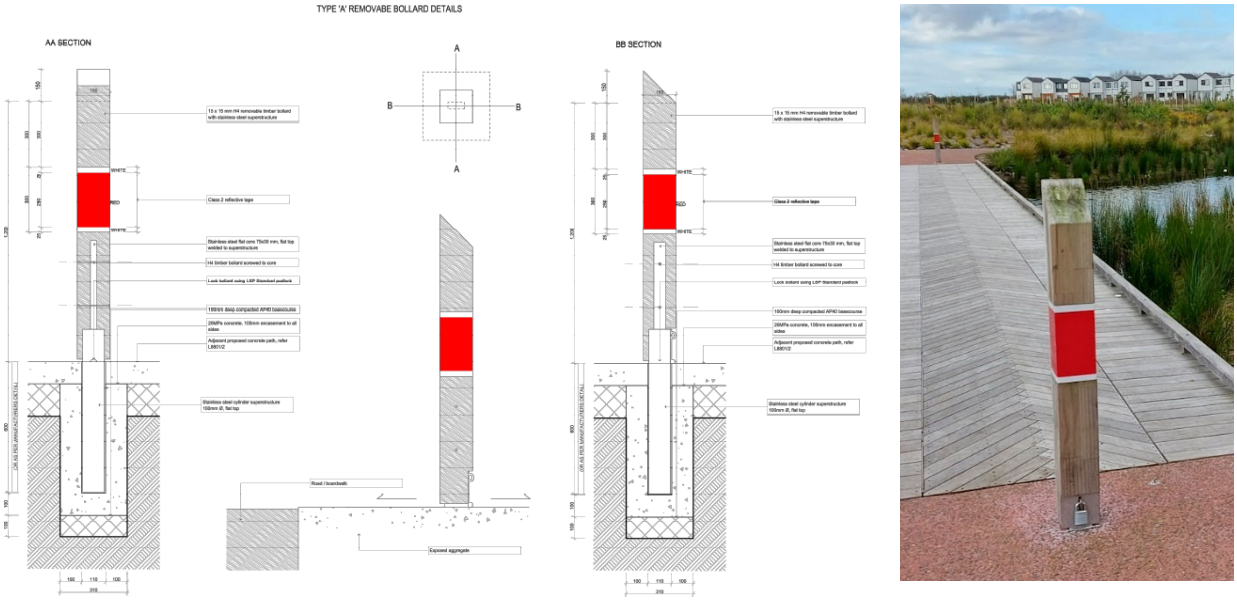

Figure 25 Removable bollard detail from Awakeri Wetlands Stage 1

2.8.25 Planting

Planting shall meet the following requirements:

- Species to match those used in Stage 1 of the Awakeri Wetlands.
- Zones, mixes and planting layouts/clumps to match those in Stage 1 of the Awakeri Wetlands.
- A CPTED analysis shall be used when designing planting zones and shall include the following:
    - Clear sightlines to be maintained along boardwalk crossings, entry points and street interfaces
    - Dense, high growth planting zones to be limited to areas where viewshafts are not required.
    - Refer to planting zones of the Awakeri Stage 1 planting plan for examples.
- All plants shall be eco-sourced from the appropriate ecological district.
- Planting areas to include 100mm of aged arbor mulch. Processed wood chips are not acceptable.
- Planting areas shall include a minimum of 300mm topsoil or local peat soil.
- All planted areas below the 10% AEP flood level shall be covered with 100% biodegradable coconut matting or similar with jute mesh (no plastic mesh).
- All planted areas above the 10% AEP flood event shall be covered with a minimum of 100mm aged arbor mulch.

2.9 Handover

A plan for handover of the asset to Auckland Council shall be prepared by the designed and reviewed / approved by Auckland Council during the design phase. The handover plan shall include:

- Confirm timeframes for handover, allowing for a minimum 12 month defects liability period.
- Confirm which departments will own and maintain each asset.
- Duration of planting maintenance by the contractor prior to handover to Auckland Council.
- Responsibility for completing resource consent conditions including groundwater and settlement monitoring.

3.0 Design deliverables

3.1 Preliminary Design

3.1.1 Definition

In the Preliminary Design the drawings and technical documentation are developed to the point where a resource consent application may be lodged if required.

It is expected that no significant changes to line, level, sizing or construction techniques will be required at detailed design. The purpose of detailed design will be to finalize structural calculations and produce construction drawings.

3.1.2 Safety in design

The Contractor shall conduct a safety in design process for the works.

At the Preliminary Design Stage, the Contractor shall take account within the design of:

- Provision and maintenance of a safe work environment for whole of asset life
- Provision and maintenance of safe plant and structures
- Provision and maintenance of safe systems of work
- Safe use, handling, and storage of plant, substances, and structures
- Provision of adequate facilities for the welfare at work of workers.

At Preliminary Design Phase, a Safety in Design Risk Assessment Register is required to identify current risk exposure in construction, operation, maintenance and decommissioning, plus the proposed risk treatment or improvement opportunity for the preferred option and any residual risks with commentary on how they should be managed.

3.1.3 Preliminary design requirements

As a minimum, the designer shall undertake the following tasks:

- Develop options for variations of the concept design including, but not limited to, construction methodology, vertical and horizontal alignments, exact locations of features, materials. The purpose of this exercise is for the designer to ensure that an optimum solution is developed for detailed design that has the lowest whole-of-life cost whilst still meeting all other project objectives and providing a safe working environment throughout the life of the asset.
- Identifying and scoping any additional investigations the designer considers necessary to properly complete the design
- Preliminary design report. The designer shall prepare a brief Preliminary design report as described below.
- Preliminary design drawings. The drawings shall be sufficiently detailed to support the required consent applications. The designer shall allow for all drawings and supporting documents required for a consent but they shall include as a minimum:
    - General arrangement drawings
    - Long sections and elevations
    - Working areas
    - Preliminary erosion and sediment control plans
    - Preliminary traffic management plans.

3.1.4 Preliminary design report and drawings

The Contractor shall submit a Preliminary Design Report covering the following:

- Hazards identified and mitigated through the safety in design process followed (either appended with a summary within the report or a detailed section within the report).
- Outline specifications for all key components including all structures and materials.

- Key assumptions made.
- Utility diversions (if required).
- Key risks and risk management.
- Recommendations including confirmation that the recommended option meets the project objectives or otherwise.
- Commentary on hydraulic performance.
- A clear statement of the internal quality control and quality assurance procedures adopted in developing the design options.
- Stakeholder and end-user requirements and confirmation that the preferred option meets agreed asset owner requirements.
- Benefits of the option, and how they can be measured during or after project implementation.

3.2 Detailed design

3.2.1 Definition

In the Detailed Design Stage, the design is finalised and complete construction drawings, specifications and any monitoring, QC plans etc are produced.

3.2.2 Safety in design

A safety in design assessment is required for the Detailed Design. This safety in design process should be continued from the Preliminary design phase. The designer should define the methodology used for the risk assessment during this phase for agreement with Auckland Council. Auckland Council will identify the stakeholders that will be involved in the Safety in Design process.

At the end of the detailed design, a Safety in Design Risk Register and Report is required. All residual health and safety risks in construction, operation, design for exceedance, maintenance and demolition or disposal shall be clearly described and conveyed. Where relevant, these should be noted on the construction drawings if they are to be managed during construction.

A stand-alone safety in design report is required. All other reports will require a safety in design section as part of the design reporting.

3.2.3 Detailed design requirements

As a minimum, the Contractor shall undertake the following tasks:

- Scope confirmation workshop. The designers project manager and lead technical staff shall all attend this workshop. The purpose of the workshop will be to confirm the preferred solution and determine any preferences for final details. The designer shall prepare minutes of the workshop.
- Review the preliminary design including; confirmation of the findings of the preliminary design report costings, benefits, etc.
- Carry out all calculations required to finalize materials, structural strengths etc.

- Prepare fully detailed Construction Drawings including:
    - Geotechnical information (bore logs) and existing underground and overhead services shall be shown on the long sections.
    - Detailed construction site access, storage and site office areas.
    - Locations of existing utility services including details of diversions where required.
    - Groundwater and settlement monitoring plan.
    - Structural drawings.
    - Any H&S issues and design mitigation assumptions that may affect or be affected by construction methodology.
- Review Standard Specifications and prepare Particular Specifications.
- Update safety in design and prepare report suitable for hand over to Constructors.
- Prepare a Geotechnical Baseline Report and agree with Auckland Council.
- Submit draft drawings, specifications and plans to Auckland Council for review.
- Finalize pre-construction risk register.
- Detailed design report as below.
- Address comments on the detailed design report and drawings. The designer shall ensure that the drawings and documents are properly reviewed and a QA undertaken by the designer prior to submission. Where any errors or omissions are found that Auckland Council considers to be a failure of QA or review, the drawings and documents shall be returned to the designer without further comment. The formal review will only be undertaken once the QA issues have been resolved. Auckland Council may appoint a peer review in addition to internal reviews. The designer shall allow for collating and assessing the reviews from Auckland Council and shall produce a Construction Documents and Drawing set that adequately addresses the comments.

3.2.4 Detailed design report and drawings

The designer shall prepare and submit a Detailed Design Report. The Final Design Report need not repeat all the detail of the Preliminary design report but should cover the following:

- Details of the final design and proposed construction methodology
- Safety in design considerations in the design process
- The Contractor's assessment of hazards and risks (including H&S) to be managed, if appropriate, a summary of risks eliminated or minimised through design can also be included
- Significant changes to the design or departures from the approved Preliminary Design Report
- Summary of residual risks at contract stage and for whole of life highlighting any remaining high risk items
- Updates to any technical or specialist reports as a result of amendments since the Preliminary design report

- Advice on any variations that may be required to Resource Consents as a result of such changes.
- Any departures from the Auckland Council Code of Practice and Standard Specifications
- Detailed advice on producer statement requirements and the level of construction monitoring required by the designer in order to be able to deliver the producer statements.

Construction drawings, labelled “For Construction”, shall be prepared on the standard Auckland Council title block and include the same information as the Preliminary Design drawings plus the following:

- Intended purpose of the asset (if appropriate)
- Geotechnical information (bore logs) and existing underground and overhead services shall be shown on the long sections
- Updated and more detailed construction site access, storage and site office areas
- Utility services diversions
- Groundwater and settlement monitoring plan.
- Structural drawings.
- Safety in design considerations to be addressed during construction, operation, maintenance and demolition or disposal.

The designer shall facilitate a meeting with Auckland Council to present and discuss the detailed design and the draft construction documentation. The objective of the design meeting is to agree what changes (if any) are required to the Draft Construction Documentation before a final version of the documentation is signed off.

Any review does not remove any responsibility from the designer for the correctness or appropriateness of the design or construction documents.

3.2.5 Ground improvements

The designer shall identify in its Detailed Design Report:

- What ground improvements, if any, are proposed
- The methods used to quantify their extent and effectiveness, and
- The precedent that has been followed in their development.

The methods and precedent shall be referenced in the Detailed Design Report. The effectiveness of ground improvement shall be demonstrated by field testing.

3.2.6 Quality assurance requirements

The designer shall prepare a project review and audit schedule for the Detailed Design and Construction Documentation. This will identify:

- One or more named reviewers accountable for reviewing technical outcomes, and technical reviews planned. Unless otherwise agreed with Auckland Council, the reviewers shall be those named in the Proposal. Where Auckland Council considers that less qualified staff than those shown in the Proposal are offered, revised rates shall be agreed.
- A named Project Director or Sponsor accountable for reviewing overall project delivery, and project outcome reviews planned. Auckland Council has an expectation that there will be a scope review (10%), a progress review (50%) and an outcome review (90%) as a minimum
- All documents will have a QA review before being delivered to the Auckland Council, including draft reports. The report should include a QA and document control page to identify author, reviewer and version control for drafts.

3.3 Geotechnical design criteria

Recent geotechnical investigations are provided in the *Geotechnical Investigations Report (GHD 2016)* and the *Geotechnical and Ground Settlement Effects Report (GHD, 2016).* This is for information only and the designer is responsible for preparing updated geotechnical investigations and assessments to support their design.

3.3.1 Ground conditions

The ground conditions for the project area are described in the *Geotechnical Investigations Report (GHD 2016)* and the *Geotechnical and Ground Settlement Effects Report (GHD, 2016)* This is for information only and the designer is responsible for preparing updated geotechnical investigations and assessments to support their design.

3.3.2 Groundwater

Groundwater information is provided in the *Hydrogeology Assessment of Effects (GHD, 2016).* This is for information only and the designer is responsible for gathering updated groundwater information and preparing an updated hydrogeology assessment.

3.3.3 Seismic design

The designer shall consider seismic hazards and liquefaction risks in the design of the project.

Some information is provided in the *Geotechnical and Ground Settlement Effects Report (GHD, 2016)*. This is for information only and the designer is responsible for preparing updated seismic hazard and liquefaction assessments..

3.3.4 Geotechnical Baseline Report (GBR)

The designer shall prepare a GBR for the project.

The purpose of the GBR is to provide a single source contract document containing measurable contractual descriptions of the geotechnical conditions to be anticipated or to be assumed to be anticipated during construction. In the event of the project running into difficulties due to ground conditions, the GBR can be used to decide if the conditions are unforeseen and therefore create

potential for a claim or fall within the conditions expected at the site. This does not present any ambiguous interpretation of conditions or any uncertainty. Only measurable, quantitative terms used.

The GBR shall present a very concise contractual summary of the ground model that the Contractor should allow for when tendering for the construction.

The GBR shall be prepared in general accordance with the Essex, R.J., 1996, Geotechnical Baseline Reports for Underground Construction: Guidelines and Practices published by the American Society of Civil Engineers, as amended by this contract.

The GBR shall contain:

- A brief summary description of the material types expected to be encountered.
- The estimated amounts and distribution of different materials along the alignment, typically presented as an estimate of the percentage of the project (for example, linear metres of tunnel) that each material type will make up. This shall be given as a predicted range to allow for geological uncertainty.
- Geotechnical and groundwater parameters, and expected behaviours, for each of these materials, given as a predicted range to allow for geological uncertainty. Include strength, permeability, grain size, mineralogy, predicted pumping rates, predicted settlement and any other aspects which could impact on construction. Wherever possible, these shall be expressed in quantitative terms, and should present the expected distribution envelope of each parameter within the range.
- Descriptions of geotechnical and man-made sources of potential difficulty or hazards that could impact the construction process (such as boulders, bedrock variability, contaminated groundwater, subsurface obstructions, unstable slopes, adjacent activities).
- A description of the anticipated construction methodology with which the baselines are associated. The baseline statements should be clear that the ground can be expected to behave differently with alternative tools, methods, sequences and equipment.

The GBR shall not contain:

- Ambiguous or vague interpretations
- Descriptions or parameters that cannot be easily measured or assessed and recorded during construction
- Qualitative terms such as ‘large’ or ‘major’ unless these are clearly defined.

3.4 Operation and Maintenance

A Draft Operations and Maintenance (O&M) Manual shall be supplied with the proposed design solution to enable the proposed solution to be fully assessed and understood by the asset owner. The Draft O&M Manual will include maintenance of fittings, plant and machinery requirements, traffic management, isolation, dewatering (and treatment/disposal of this water), pipe inspection, sediment/debris removal and decommissioning. Auckland Council may engage a third party operations expert to review the acceptability of the solution and the O&M Manual.

The design shall provide for safe personnel access to enable a walk-through of the full length of the new infrastructure for operational and maintenance purposes.

The designer shall finalise O&M manuals, containing such information and details as are necessary for Auckland Council to carry out the operation and cost-effective maintenance of the system. The contents and format of these O&M manuals shall be subject to the approval of Auckland Council and a third party expert.

4.0 Hold points

In addition to reviews of the design packages and elements described above, specific review of the following design items is also required by Auckland Council as they become available:

- Construction methodology.
- Staging of Stage 2 and 3.
- Groundwater management design.
- Peer review for groundwater drawdown assessment and settlement assessment (long term and short term), buoyancy assessment, culvert foundation/bedding design and road backfill design.
- Groundwater and Settlement Monitoring and Contingency Plan.
- Material specifications including shop drawings for pre-cast concrete structures.
- Groundwater cut-off barrier / slurry wall mix design and testing documentation.
- Erosion and scour protection design.
- Obstructions management plan (approach for managing clashes with buried kauri logs etc).
- Landscape design drawings and report.
- Planting plans.
- Preliminary design (Report and Drawings)
- Detailed design (Report and Drawings)

Additional hold points will be required for the construction phase and these will be outlined by Auckland Council prior to construction.

APPENDIX 11 – STAGE 1 AWAKERI WETLANDS DESIGN REPORT

Auckland Council

Awakeri Wetlands - Stage 1A

Detailed Design Report

September 2017

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1. Introduction

The Awakeri Wetlands, also known as Takanini Stormwater Conveyance Channel (TSCC) or Takanini Cascades, forms the fourth stage of a greater scheme to provide stormwater servicing for the Takanini south-east area. The Awakeri Wetlands will pass forward flows from Old Wairoa Road, Cosgrave Road, Walters Road and Grove Road, for which there is currently no formal drainage system, to a box culvert at Grove Road. The Grove Road Box Culvert conveys flows from the Awakeri Wetlands to the McLennan Wetland. During large storm events, flow is attenuated in the McLennan Wetland before being discharged to the Pahurehure inlet via the proposed Artillery Drive tunnel. Refer to Drawing 51-33411-C001 (in Appendix C) for an overview of the greater scheme. The Awakeri Wetlands construction will take approximately 2-3 years to complete.

The Awakeri Wetlands consists of approximately 2.3 km of open waterway that will contain the existing 1% AEP floodplain, allowing the surrounding land to be comprehensively developed.

Resource Consent was granted in September 2016. A Notice of Requirement was approved in October 2016 for the designation of land to allow the development of the TSCC.

The designation corridor will allow for the construction of the Wetlands which will convey low flow and the full 1% AEP flow from the catchment. It will deliver an open public space with the provision for cycleways and footpaths that will increase the connectivity between new urban areas and allow for the development of the Special Housing Takanini Strategic Areas (including Special Housing Areas 2A, 2B and Wallace) and area 2B4 which is currently zoned Future Urban.

The Awakeri Wetlands are proposed to be constructed in three discrete stages, these are shown in Figure 1 below. Stage 1 is split into Stages 1A and 1B; where 1A includes bulk earthworks and hydraulic structures for the wetlands to operate for stormwater management. Stage 1B incudes additional structures and landscaping features. This detailed design report is for Stage 1A of the Awakeri Wetlands.

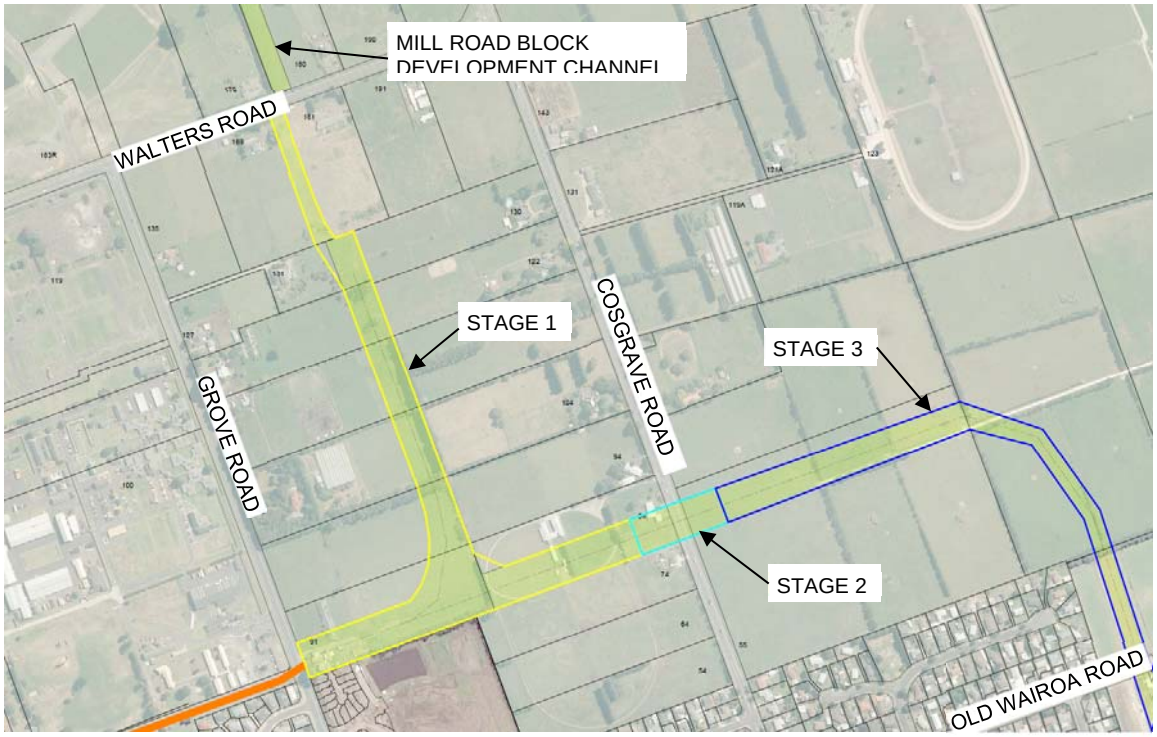


Figure 1 Awakeri Wetlands Staging

1.1 Purpose

The purpose of this report is to document the detailed design of Stage 1A of the Awakeri Wetlands.

1.2 Scope

The scope of this report is to:

- Document the detailed design of Stage 1A of the Awakeri Wetlands.
- Document the design philosophy and design practices relating to the detailed design.
- Provide a record of the key decisions and Safety in Design provisions.
- Document the anticipated maintenance requirements and project risks.

The scope of Stage 1A of the Awakeri Wetlands project includes design of the features required for the channel to operate for stormwater management, primarily as a stormwater conveyance system. This includes:

- Bulk earthworks.
- Erosion and scour protection.
- Weirs.
- Footpaths.
- Boardwalks.
- Culverts.
- Groundwater cut-off barriers.
- Other works required for construction to make the stormwater system operational.

The scope of Stage 1A of the Awakeri Wetlands excludes additional structures such as high level pedestrian bridges or urban design / landscaping features which will be included in Stage 1B. Planting will happen concurrently during Stage 1A, however the specifics relating to planting are documented separately by Auckland Council.

The planting forms an essential part of the erosion protection regime. Discussions with Auckland Council have been undertaken to coordinate the type of plants, as discussed further in Section 4.7.1 and in Section 6.5. As part of this liaison Auckland Council have confirmed that the plants being selected will have extensive root systems. The root systems of the planting are relied on as one strategy for mitigating scour within the channel as discussed within this report and is an integral part of the design.

1.3 Assumptions and Limitations

This report has been prepared by GHD for Auckland Council and may only be used and relied on by Auckland Council for the purpose agreed between GHD and the Auckland Council as set out in this report.

GHD otherwise disclaims responsibility to any person other than Auckland Council arising in connection with this report. GHD also excludes implied warranties and conditions, to the extent legally permissible.

The services undertaken by GHD in connection with preparing this report were limited to those specifically detailed in the report and are subject to the scope limitations set out in the report.

The opinions, conclusions and any recommendations in this report are based on conditions encountered and information reviewed at the date of preparation of the report. GHD has no responsibility or obligation to update this report to account for events or changes occurring subsequent to the date that the report was prepared.

The opinions, conclusions and any recommendations in this report are based on assumptions made by GHD described in this report. GHD disclaims liability arising from any of the assumptions being incorrect.

GHD has prepared this report on the basis of information provided by Auckland Council and others who provided information to GHD (including Government authorities), which GHD has not independently verified or checked beyond the agreed scope of work. GHD does not accept liability in connection with such unverified information, including errors and omissions in the report which were caused by errors or omissions in that information.

2. Project Overview

2.1 Awakeri Wetlands

The proposed Awakeri Wetlands will extend from 989-999 Papakura-Clevedon Road in the south-east to 91 Grove Road in the west. A northern branch will extend northwards towards Walters Road.

In general the Awakeri Wetlands will provide stormwater servicing for future development of Areas 2A, 2B and part of Area 4 (2B4) of the Takanini Structure Plan. At present the area is significantly impacted by the 1% AEP (Annual Exceedance Probability) floodplain, restricting development of the area.

The proposed channel will:

- Provide for the full 1% AEP flows, effectively removing the floodplain from surrounding land.
- Offer an ecological corridor (both terrestrial and aquatic) that would otherwise not be provided.
- Deliver stormwater servicing for development within the catchment area that is not currently presented.
- Afford an open space with significant amenity value and the provision for pedestrian linkages and cycleways.

The Awakeri Wetlands consists of two main branch channels; the main channel and the northern branch channel.

Main channel

The main channel has a length of 1.55 km of open waterway, ranging in depth between 2 m and 4 m below ground level. The channel has an approximate gradient of 0.28% and a total width (at the 1% AEP water level) ranging from 20 m to 37 m. The low flow water width is typically 14 m wide but varies substantially in width and depth.

Northern channel

The northern channel has a length of 0.7 km of open waterway, ranging in depth between 2.4 m and 3.8 m below ground level. The channel has an approximate gradient of 0.20% and a total width (at the 1% AEP water level) of approximately 25 m. The low flow water width is typically 14 m wide but varies substantially in width and depth.

The Awakeri Wetlands is designed with a meandering low flow series of discrete water bodies or wetlands, with a permanent water depth of about 0.8 m controlled by weirs at approximately 100 m centres longitudinally along the base of the channel. These provide an ecological benefit and limit groundwater drawdown. Generally the low flow channel base width varies between 3-6 m and has side slopes of 2H:1V, with an intermediate flat wetland bench. Above the wetland bench are riparian planted channel banks with slope batters 4H:1V integrated into landscape features such as shared paths and play areas.

Figure 2 provides a typical cross section of the Awakeri Wetlands.

Figure 2 Typical section of the Awakeri Wetlands

2.2 Catchment area

The Awakeri Wetlands stormwater catchment (shown in Figure 3) represents the area to be serviced by the Awakeri Wetlands for stormwater conveyance.

The area is approximately 162 hectares (ha) and consists of areas 2A (50.3 ha), 'Mill Road Block' (16.4 ha), 2B4 (57.3 ha) and 2B (38.0 ha) as shown as a dotted purple line in Figure 3.

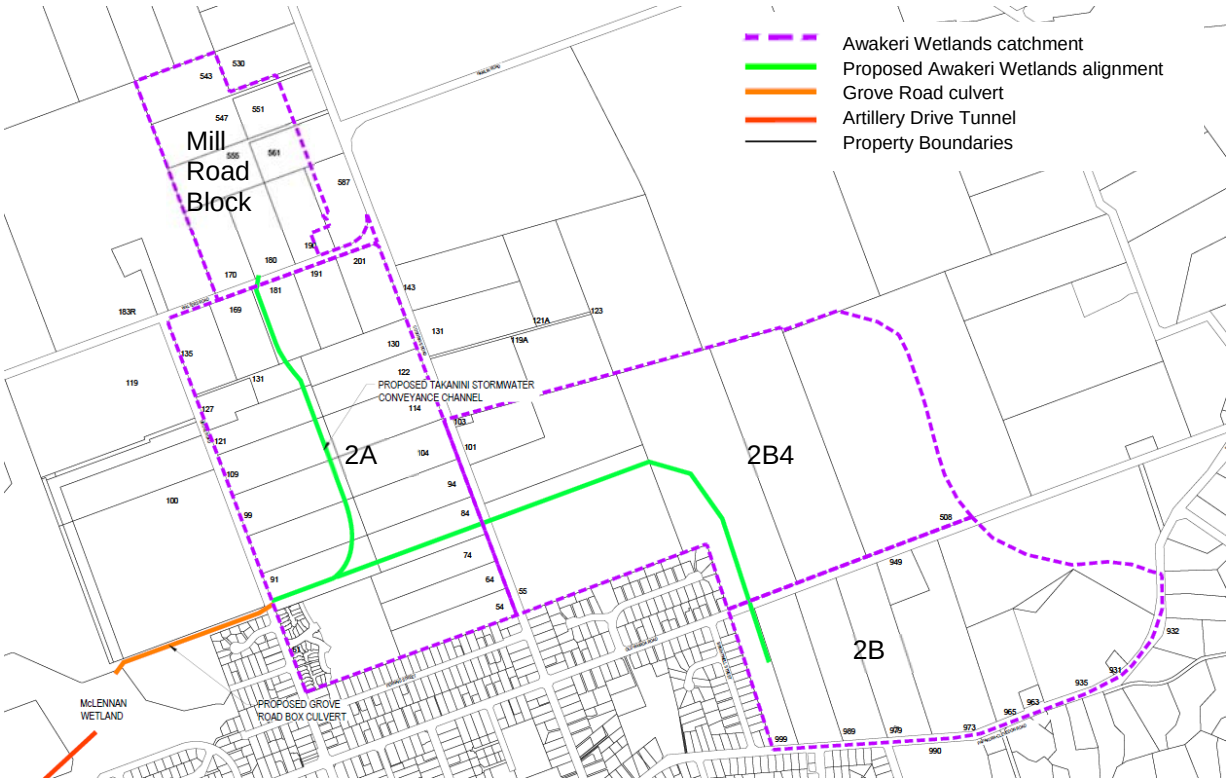


Figure 3 Awakeri Wetlands catchment

2.3 Takanini Stormwater Scheme

The Awakeri Wetlands is part of the Takanini Stormwater Scheme (refer Drawing 51-33411-C001) to reduce flooding for events up to the 1% AEP and provide servicing for the greater Old Wairoa Road catchment. The Takanini Stormwater Scheme is comprised of four sections including:

Part 1 - Artillery Drive Tunnel

A new 2.5 m diameter tunnel that will extend over approximately 1.1 km from the McLennan Wetland to the Pahurehure Inlet. This effectively forms the downstream outlet for the stormwater scheme. The Artillery Drive Tunnel project is currently under construction (2017).

Part 2 - McLennan Wetland

Constructed in 2002, this wetland already receives stormwater from the Housing New Zealand development and Papakura Military Camp through to Bruce Pulman Park in the north; and Willis Road and Clevedon Road to the south. The wetland provides attenuation and treatment for the greater catchment before discharge. Currently the wetland passes forward flows to the Gills Road pond and will continue to do so in the future with only high flows being conveyed through the new Artillery Drive tunnel.

The McLennan Wetland is designed to accept flows from the Old Wairoa Road catchment, which includes the catchment area of the Awakeri Wetlands. The McLennan Wetland has been included in a hydrological model, built and held by Auckland Council. The model indicates that there is enough storage to attenuate flows to an acceptable level which the Artillery Drive Tunnel has been designed in accordance with.

Part 3 - Grove Road Culvert

A new culvert that will convey flows from the Awakeri Wetlands catchment to the McLennan Wetland.

The location of the Grove Road Culvert was altered from the location shown in the Grove Road Structure Plan. The structure plan showed the channel running through the middle of 61 Grove Road and connecting to the proposed Grove Road Culvert at Matheson Street.

The property at 61 Grove Road has subdivision consent and physical works on site are near completion for Stage 1 of their development. As a consequence; the route defined in the Structure Plan is no longer viable. The optimal location for the box culvert connection was therefore to the north of the northern boundary of 61 Grove Road. This allows minimal dissection of private properties and optimises the drainage potential of the surrounding land.

The Grove Road Culvert has been designed by Jacobs (NZ) Ltd and is a separate project to the Awakeri Wetlands.

Construction is currently underway with completion expected April / May 2018.

Part 4 - Awakeri Wetlands

As outlined in this report, a new 2.3 km open channel that will convey flows from part of the Old Wairoa Road catchment (Old Wairoa Road in the south-west to Walters Road in the north) to the Grove Road Culvert. The construction of the channel will take 2-3 years for completion of all stages.

2.4 Zoning and Special Housing Areas

The zoning of the catchment is based on the Unitary Plan zoning. Area 2B4 is currently zoned Future Urban, and therefore a similar level of development has been assumed to the surrounding areas and assumptions made based on existing information from Auckland Council, as described in this report.

2.5 Network Discharge Consent

The Old Wairoa Road Catchment Management Plan (CMP) (PDC, 2004) defines the catchment boundary for the McLennan Wetland. In 2010 the boundary shown in the CMP increased to include part of the Takanini South Catchment through CMP Variation 33738 (2010). This additional area is shown as the ‘Wallace’ area.

A “trunk stormwater conveyance system to serve areas 2A, 2B and 2B4” is consented under the Network Discharge Consent 34887 (NDC). The Awakeri Wetlands is the proposed infrastructure for servicing these areas and the Wallace area to the north.

2.6 Draft Central Papakura ICMP

The Draft Central Papakura Integrated Catchment Management Plan ICMP (PDC, 2007) documents the overarching stormwater conveyance approach for the catchment. The ICMP outlines a potential alignment for the Awakeri Wetlands.

The ICMP alignment is similar to the main channel alignment proposed in this report; with the main difference at the eastern end where the ICMP alignment splits into two channels. The ICMP channel excludes the proposed Northern Branch channel and services part of the 2A catchment using a piped stormwater system.

2.7 Concept design

The concept design was developed by GHD in July 2014 as part of the Notice of Requirement process and is described in the Takanini Stormwater Conveyance Channel Infrastructure Report (GHD, 2014). The Concept Design concluded that a conveyance channel was the most beneficial and recommended stormwater solution for the catchment, compared to a piped solution, or piped / pond hybrid system.

Refer to the Plan amendment 48 – Takanini stormwater conveyance corridor (Auckland Council, 2014) for more detail.

2.8 Scheme design

The scheme design was developed by GHD in July 2016 as part of the Resource Consent process and is described in the Takanini Stormwater Conveyance Channel Stormwater Report (GHD, 2016a). The Scheme Design outlined the key features, effects and mitigation of effects for the TSCC.

3. Existing Environment

The following section provides a brief description of known future works and a general description of how the existing environment will be affected by the proposed works.

3.1 Site setting

3.1.1 Land use

The majority of land within the conveyance catchment has historically been pastoral with large lifestyle blocks and a relatively low intensive nature. Recently, large areas of land have begun developing into residential areas to a high density.

Consents have already been obtained for development of sites within the catchment, subject to temporary stormwater solutions, on the proviso that once the Awakeri Wetlands are built, these sites will be connected to it. These include:

- The Grove at 61 Grove Road (Equinox Group).
- 54, 64, 74 and 94 Cosgrave Road.
- Kauri Flats School at 181 and 191 Walters Road.
- 201 Walters Road.
- Twin Parks Estate at 989 to 999 Papakura-Clevedon Road (Cappella Papakura Developments Ltd).
- Papakura Residential at 965 Old Wairoa Road and 965 to 973 Papakura-Clevedon Road (Cabra Investments Ltd).
- Part of the Montgomery development at 881 to 899 Papakura-Clevedon Road.

These sites are at different stages of development, from concept stage to bulk earthworks. Houses have been established at 61 Grove Road (The Grove) and at the Cappella development (Twin Parks Estate). Additional houses are currently still under construction within both of these developments.

The developments above are shown on drawing 51-33411-C006 (see Appendix C).

Other properties that have expressed their intention to develop within the next 12 months include:

- 169 Walters Road.
- 122 Cosgrave Road.
- 130 Cosgrave Road.
- 99 Grove Road.

3.1.2 Topography

The catchment is essentially flat in nature; except for the eastern portion where it falls from approximately 67 m over a distance of 0.8 km to 26 m; with an average slope of about 3 %.

From here; the catchment falls from an RL of 26 m over 1.7 km to an RL of 22 m at Grove Road. This provides an average slope for the flat portion of about 0.24 %.

3.1.3 Existing stormwater and features

There is no formalised drainage across the catchment with small dissected channels and farm drains connecting to roadside table drains. The existing natural streams in the region are very short and have little to nil baseflow during the summer months (PDC, 2007).

The roadside table drains along Cosgrave Road and Walters Road collect overland flow and have limited conveyance capability. These roadside drains are deeply incised, up to about 2 to 2.5 m in depth. Generally, the roadside drains store water and discharge to ground soakage when water tables are low over summer. Figure 4 shows the table drain on Cosgrave Road.

Figure 4 Cosgrave Road table drain

To the west of Grove Road and south of Fernaig Street and Pukeroa Place stormwater is reticulated. Most of these flows are directed to the wetland located in McLennan Park. This wetland (the McLennan wetland) is designed to attenuate and treat flows from the Old Wairoa Road catchment before discharge via Gills Pond to the Pahurehure Inlet and is discussed further in Section 3.2.

Figure 5 McLennan wetland

3.1.4 Existing flooding

The vast majority of the Awakeri Wetlands catchment area and a portion of the Takanini South catchment to the north-west are predicted to be inundated in a 1% AEP storm event to a depth of 300 to 500 mm. Extensive ponding has been observed during rainfall events, particularly in winter when the groundwater table is high. This is primarily a result of ineffective stormwater drainage, but is also due to flat topography, high groundwater tables and limited soakage capacity of the peat fields.

3.1.5 Geological setting and extent of peat

The geotechnical investigation conducted by GHD in 2016 confirms that the ground beneath the area is predominantly made up of peats, organic silts and sands. Further details of the peat are discussed in the Takanini Stormwater Conveyance Channel Geotechnical Investigations Report (GHD, 2016c).

The organic peat typically extends to a depth of 20 m below ground level and is extensive throughout the entire Stage 1A of the Awakeri Wetlands. An approximate extent of the peat is shown in Figure 6 below.

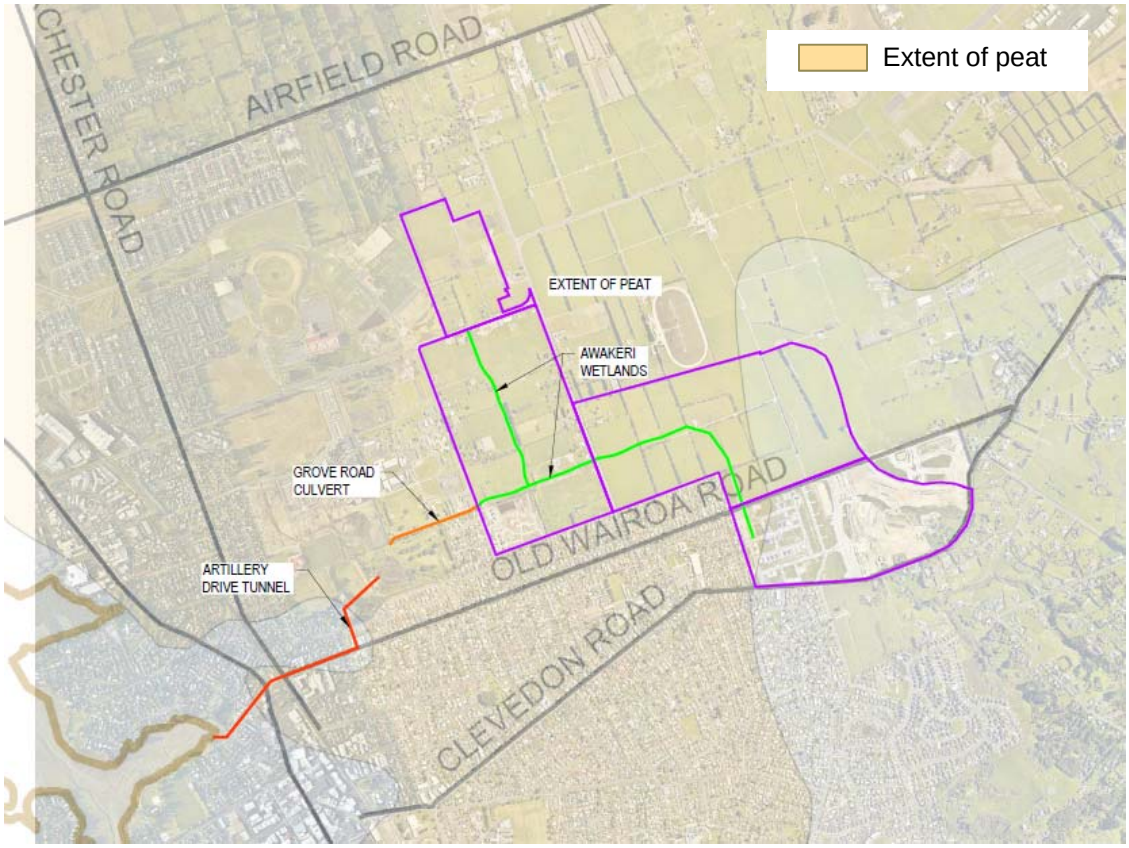


Figure 6 Extent of peat (PDP, 2006)

The Takanini area is known to be underlain by a significant peat aquifer.

Geological units described generally as peat in this area consist of a material that ranges from humic, fibrous peat to amorphous organic clay and are generally horizontally stratified. This is further discussed in the Takanini Stormwater Conveyance Channel Geotechnical Investigations Report (GHD, 2016c).

3.1.6 Surface water and discharge to ground

The majority of stormwater in the undeveloped areas of the Awakeri Wetlands catchment and surrounding rural areas enters the ground via direct infiltration. Impervious surfaces in areas designated as rural discharge to ground soakage or open channels. Soakage test results indicate some of the highest soakage rates were found within peat areas. However, sample testing indicated the peat also had low permeability.

The stormwater from developed areas are generally conveyed via pipe networks or swales and will generally be piped into the Awakeri Wetlands at the weir locations.

3.1.7 Groundwater

Groundwater level monitoring data has been collected over the past 33 months (depending on location) to establish seasonal variation in groundwater levels. This data is included in Takanini Stormwater Conveyance Channel Geotechnical Investigation Report (GHD, 2016c) and in the Takanini Stormwater Conveyance Channel Stage 1 Groundwater and Settlement Monitoring and Contingency Plan (GSMCP) (GHD, 2017). The latest data can be provided on request.

Depths to groundwater in the shallow unconfined aquifer system range from 0.0 m in the eastern part of the subject site to 1.0 m to 1.5 m near Cosgrove Road and are >1.5 m depth in the south western part of the site near Grove Road.

3.1.8 Existing utilities

Existing services are outlined in Drawing 51-33411-C008 which include:

Stormwater

As already noted, a large portion of the Awakeri Wetlands catchment area is not serviced by a formal stormwater network. The developed and developing areas of the catchment typically include stormwater attenuation which discharge at predevelopment levels to the roadside table drains or existing stormwater networks at the catchment extents.

Water

Watercare Services Limited (WSL) through Veolia Water provides reticulated potable water to residential properties within the Awakeri Wetlands catchment area along Cosgrave Road, Walters Road and Grove Road. The following water assets are known to be within the area:

- 1200 mm diameter CLS water pipe along the west side of Cosgrave Road (Waikato No.1 Trunk Watermain discussed further below)
- 100 mm diameter PVC pipe along the west side of Cosgrave Road.
- 100 mm diameter AC abandoned water pipe along the east side of Cosgrave Road.
- 250 mm diameter PE pipe along the east side of Grove Road.
- 100 mm diameter PVC pipe along the west side of Grove Road.
- 175 mm diameter CLS and 100 mm diameter AC abandoned water pipes along the east side of Grove Road.
- 100 mm diameter AC pipe along the south side of Walters Road.
- PE pipes within the development at 61 Grove Road (along Saddleback Crescent, Bellbird Street and Stitchbird Crescent).

New water pipes are proposed along Walters Road by developers and these are discussed further in the Walters Road culvert Detailed Design Report (GHD,2017).

Waikato No. 1 trunk watermain

A 1,200 mm diameter watermain owned by Watercare Services Ltd runs along the western side of Cosgrave Road and has an estimated depth to invert varying between approximately 2.5 m to 3.0 m. This is a strategic main, supplying the bulk of potable water to Auckland.

There is a fibre optic cable above the watermain for communication purposes with a direct link to the Waikato Treatment Plant.

Wastewater

The Takanini Sewer which runs through Bruce Pulman Park is the proposed wastewater discharge location for developments within the Awakeri Wetlands catchment. The closest current (2017) connection point to the Awakeri Wetlands is at 169 Walters Road which is at a 525 RCRRJ pipe and manhole.

Currently, there are two known rising mains in the area which discharge to the Takanini Sewer at 169 Walters Road. One is along Grove Road from the 61 Grove Road development, and the other along Cosgrave Road from the Cappella and Cabra developments. These rising mains discharge to the north at the watercare trunk line near Walters Road, which is the proposed connection for future wastewater.

There is no existing wastewater servicing for the undeveloped areas within the catchment. As development of the catchment commences, wastewater servicing is being constructed by developers. The wastewater is owned and operated by Veolia.

The residential areas adjacent to the catchment such as Fernaig Street and Corkill Place are reticulated with wastewater and water services. Refer to Drawing 51-33411-C008.

Gas

A 356 OD PE Vector high pressure gas transmission pipeline traverses through areas 2B and 2B4 with an average depth of cover of 900 mm and has a 12 m wide designation. The gas main travels in a north-south direction between Settlement Road and Hamlin Road, as shown in Drawing 51-33411-C008.

Power

Historically, power has been transmitted in overhead lines. Some new developments such as at Old Wairoa Road are installing underground power systems. Hence there is a mixture of overhead and underground power throughout the area.

There are no significant known high voltage feeds in this area.

Telecom and Vodafone

There are existing Telecom and Vodafone services along Cosgrave Road, Grove Road and the local roads adjacent to the Awakeri Wetlands catchment.

3.2 McLennan wetland

Existing and consented wetland

The McLennan wetland was constructed in 2002, this wetland already receives stormwater from the Housing New Zealand development and Papakura Military Camp through to Bruce Pulman Park in the north; and Willis Road and Clevedon Road to the south. The wetland provides attenuation and treatment for of the Old Wairoa Road catchment as per Figure 7.

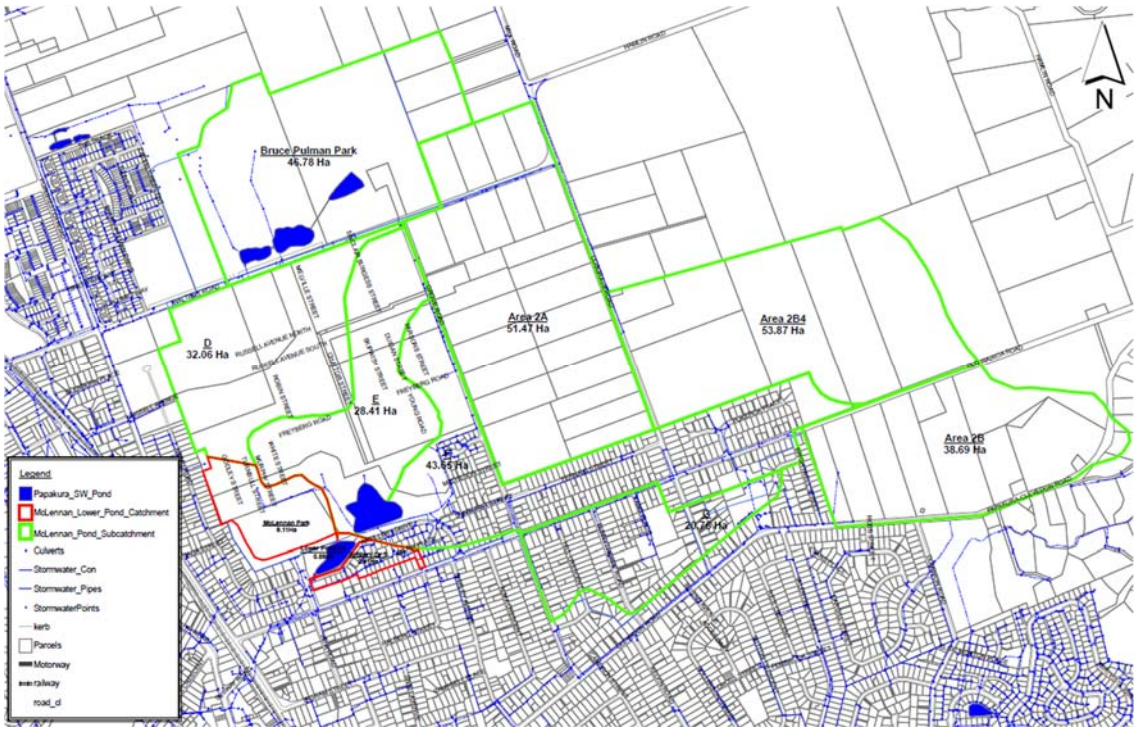


Figure 7 McLennan wetland sub-catchment map (Old Wairoa Road CMP Variations, 2009)

The wetland currently has an embankment top level of RL 16.00 m and an emergency spillway level of RL 15.1 m.

Network Discharge Consent 37205, 33738 and 33538 specify that prior to any further development commencing in areas 2A, 2B or 2B4 (ie. The construction of the Awakeri Wetlands) the following works will be undertaken:

- Increase of embankment level from RL 16.0 m to RL 16.2 m.
- Increase of spillway level from RL 15.1 m to RL 15.4 m.

3.3 Capacity downstream

The capacity of the downstream network has been considered and discussed in the *Takanini Stormwater Conveyance Channel Stormwater Report (GHD, 2016a)*.

3.4 Water quality

For the pre-developed scenario, during the Water Quality rainfall event (1/3 50% AEP event), rainfall onto the Awakeri Wetlands catchment is expected to soak through the soil, with little runoff being produced.

For the developed areas adjacent to the proposed Awakeri Wetlands catchment; water quality treatment is provided by the McLennan Wetland discussed in Section 3.2. The efficiency of the upper McLennan Wetland has been estimated at 72% (PDC, 2004).

There is another stormwater treatment pond at the downstream end of the Old Wairoa Road catchment; the Gills Road Pond. The Gills Road Pond provides stormwater treatment for the Old Wairoa Road catchment prior to discharging to the Pahurehure Inlet.

There is a requirement for developments in the area to discharge stormwater into soakage devices, which will mitigate some contaminants from entering the downstream receiving environment during small rainfall events (<15mm).

4. Methodology and Design Parameters

4.1 Design requirements

The Awakeri Wetlands has been designed to accommodate the following elements:

1. Convey the 1% AEP flows that are conveyed to the designation boundary wholly within the channel extent and subsequently within the designation. The design does not include earthworks outside the designation within private developments that would be required to get overland flow from the adjacent land into the channel. The design assumes that these works will be undertaken by the developers in accordance with their own designs.
2. Provide a permanent water level to support the development of a natural aquatic ecosystem.
3. Provide low flow operation levels of the channel at a suitable depth to allow piped flow from adjacent catchment areas to flow with a free discharge at low flows (not drowned) where practical.
4. Provide suitable 1% AEP flow levels in the channel to allow properties at the catchment extents to design overland flow paths with sufficient capacity and grade to discharge to the channel.
5. Provide a safe environment for the community and for those staff undertaking the operation and maintenance of the channel.
6. Provide for additional amenity value within the designated area where possible.
7. Make provision of the development of footpaths and cycleways.

4.1.1 Design standards

The design requirements for relevant components of the Awakeri Wetlands are discussed in this section. The relevant design guides and reference material that have been referred in this report include, but are not limited to:

Wetland / channel and hydraulic structures (culverts, pipes, weirs) design

- TP108 (Auckland Regional Council, 1999)
- TP10 (Auckland Regional Council, 2003)
- Queensland Urban Drainage Manual (Department of Energy and Water Supply, 2013)
- Auckland Council Stormwater Code of Practice (Auckland Council, 2015)
- Hydraulics of Precast Concrete Conduits (Concrete Pipe Association of Australasia, 1997)

Shared paths and boardwalks

- Auckland Transport Code of Practice (Auckland Transport, 2017)
- Tracks and Outdoor Visitor Structures SNZ HB 8630:2004 (SNZ, 2004)

Design criteria for the Awakeri Wetlands has been summarised in Table 1 below, based generally on the above publications.

Table 1 Design requirements and considerations

Constraint		Design principle	Discussion
Hydrology	Conveyance	Allow for conveying up to the 1% AEP flow from the catchment.	Section 4.5
	Soakage / first 15mm rainfall in catchment goes to soakage	Low flow water level to accommodate reduced recharge from small rainfall events.	Section 4.4
	Climate change	Hydrological parameters to allow for climate change to 2090 values.	Section 4.4
	Scour	Control scour of channel bed and banks to acceptable levels.	Section 6
Hydrogeology	Peat soils (permeability)	Consider high permeability of peat soils.	Sections 4.3.1 and 7
	Groundwater levels	Consider high groundwater levels during construction, seepage and floatation risk for structures.	Section 10
	Groundwater levels (drawdown)	Risk of lowering groundwater levels and inducing settlement of adjacent land. Consider effect of works on groundwater levels and settlement risk.	Section 4.3.1
	Groundwater/soil chemistry	Low pH of groundwater/soil and potential sulphates in groundwater/soil. Consider low pH and sulphate impact on structures and materials.	Section 4.3
	Groundwater/soil chemistry (drawdown)	Consider impact of groundwater drawdown on chemistry of groundwater and soils. Risk of lowering pH further and Acid Sulphate Soils.	Section 4.3
Geotechnical	Peat soils (strength)	Peat to ~20 m depth. Consider impact of low strength peat in terms of erodibility, settlement, bearing and difficulties with piling.	Sections 5,6,7 and 10
	Slope stability	Consider low strength of soils and wetland batters. Maximum slope of batters typically 4:1 (or 2:1 where low height, underwater and stabilised with aggregate).	Section 5
Development	Primary drainage	Allow for stormwater connections into the channel at levels which allows the extent of the catchment to be drained by gravity.	Section 8
	Secondary drainage / overland flow	Allow for overland flow connections into the channel at levels which allows the extent of the catchment to be drained by gravity.	Section 8
Ecological	Temperature	Consider temperature of low flow water during consideration of depth, width, shading, materials and operation.	Section 5
	Fish passage	Allow for fish passage at structures that may cause barrier such as culverts and weirs.	Sections 7 and 10

	Evaporation	Consider evaporation effects and lowering of water level / drying out of wetland benches.	Section 5.1.5
	Habitat	Consider habitat of wetlands including planting, water quality, flow velocities, materials.	Section 5
Planning	Designation	All permanent works to fit within the designation boundary or land owned by Auckland Council. No earthworks outside the designation, unless agreed with landowners.	Section 5
	Resource consent	Works to be in accordance with resource consent conditions	Section 5
Public space	Public access	Shared paths provided along the wetlands corridor. Boardwalks to provide access between both sides of the wetlands. Paths and boardwalks to be designed in accordance with ATCOP (Auckland Transport, 2017).	Sections 11 and 12
Operational	Debris	Blockage risk of culverts to consider impact on flooding and conveyance of flow.	Section 10
	Sediment deposition	Consider build-up of sediment in the wetland and how this will be inspected and cleaned out.	Section 6.7
	Maintenance	Access to be provided for inspections and maintenance of weirs, culverts, boardwalks, paths and wetland.	Sections 11 and 12
Cultural	Design philosophy	Involve Mana Whenua in the design process and incorporate iwi philosophy in the stormwater design where possible.	At hui's and incorporated throughout design
Safety	Public safety	Safety in design for areas that could be accessed by public: low flow water body, paths, culverts and boardwalks.	Section 13
	Operational safety	Safety in design for areas accessed by operations staff: low flow water body, weirs, paths, culverts and boardwalks.	Section 13
	Construction safety	Safety in design for the construction of the wetlands and associated structures.	Section 13

4.2 Geotechnical background

Geotechnical parameters have been derived from the investigations carried out as part of the Awakeri Wetlands Scheme Design.

Table 2 from the geotechnical investigation describes the geotechnical parameters for the site. This includes key soil parameters. Additional information in relation to the depth of peat, groundwater levels and geological logs is available in the Takanini Stormwater Conveyance Channel Geotechnical Investigations Report – Technical Report C (GHD, 2016c).

Table 2 Range of geotechnical parameters

Geological Unit	Bulk Unit Weight (kN/m^3)	Undrained Shear Strength S_u (kPa)	Effective Strength Parameters		Young Modulus (MPa)		Poisson's Ratio		Estimated Permeability k (m/sec)[Note 1]
			Effective Cohesion c' (kPa)	Effective Friction Angle Φ' (degrees)	Undrained (E_u)	Drained (E')	Undrained v_u	Drained v'	
Cohesive Fill	14	6 - 12	3 - 6	26 - 30	0.6	0.4	0.5	-	10^{-4} to 10^{-5}
Puketoka Formation Organic soils / Peat	11	0 - 20	0 - 5	25 - 36	0.5 - 0.8	0.4 - 0.6	0.5	0.1-0.15	$k_v = 10^{-4}$ to 10^{-6} $k_h = 10^{-5}$ to 10^{-7}
Puketoka Formation Alluvial clays/silts	16 - 20	30 - 60	5 - 10	27 - 34	4 - 10	2 - 8	0.5	0.2 - 0.3	$k_v = 10^{-4}$ to 10^{-6} $k_h = 10^{-5}$ to 10^{-7}
Kaawa Formation Sands	18 - 22	-	-	30 - 34	-	40 - 90	0.5	0.2 - 0.4	$k_v = 10^{-3}$ to 10^{-6} $k_h = 10^{-4}$ to 10^{-7}
ECBF Residual Soils	17 - 20	100 - 230	10 - 13	26 - 34	-	45 - 75	0.5	0.2 – 0.3	$k_v = 10^{-4}$ to 10^{-6} $k_h = 10^{-5}$ to 10^{-7}
ECBF Rock (Sandstone)	20 - 22		20	34 - 36	-	90 - 150	0.5	0.25	$k_v = 10^{-3}$ to 10^{-5}

4.3 Hydrogeology and settlement

The hydrogeology of the area and effect of the Awakeri Wetlands are discussed in the Takanini Stormwater Conveyance Channel Hydrogeology Assessment of Effects (GHD, 2016d).

4.3.1 Groundwater dewatering

The Awakeri Wetlands are constructed below the seasonal low groundwater level in some areas. This has the potential to cause dewatering of the groundwater within adjacent land. Dewatering of the adjacent peat can cause ground settlement and potentially damage adjacent structures.

Slurry wall

To mitigate potential dewatering, a slurry wall was installed at critical locations during the enabling works for the Awakeri Wetlands. The design of the slurry wall is further discussed in the Takanini Stormwater Conveyance Channel Groundwater and Ground Settlement Effects Report (GHD, 2016e) and the extents of the slurry walls are shown in Figure 8 below.

The slurry wall is 7 m deep and a minimum of 600 mm wide, consisting of a cement / bentonite mix. The slurry wall was installed by excavating a trench under slurry and replacing the peat material with the cement-bentonite mix.

The top 1 m of the slurry wall will be excavated through when forming the Awakeri Wetlands final contours. This is considered acceptable and part of the design. The top 150 mm of the slurry wall will be covered with topsoil to protect the top of the wall from damage.

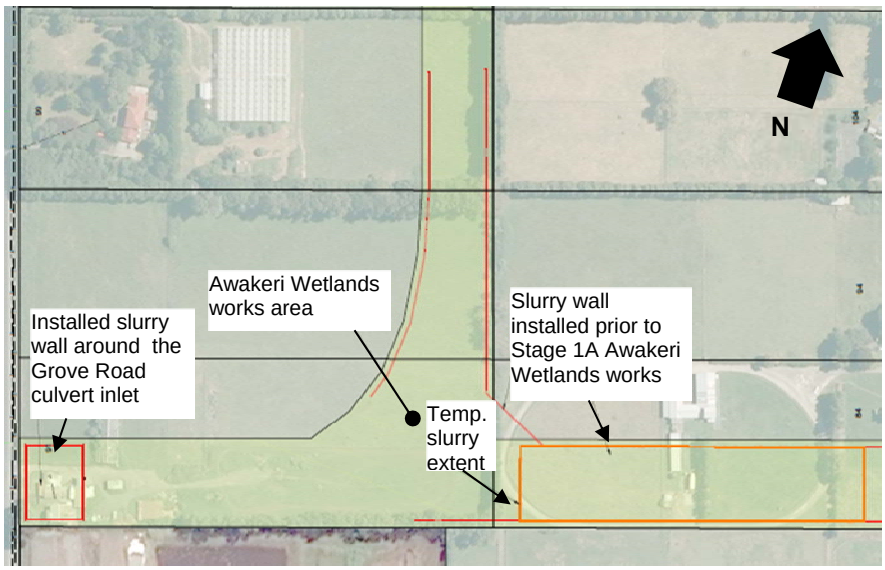


Figure 8 Slurry wall extent

4.3.2 Settlement effects

As discussed above, the Awakeri Wetlands has the potential to cause settlement of adjacent land. This can have some positive effects prior to development of land, but can also have adverse effects to existing housing and infrastructure.

This risk has been considered in the Takanini Stormwater Conveyance Channel Groundwater and Ground Settlement Effects Report (GHD, 2016e) as part of the resource consent assessment.

Groundwater and settlement will be monitored in accordance with the GSMCP (GHD, 2017) during construction.

4.3.3 Downstream water chemistry

The downstream receiving environment is the McLennan wetland. The McLennan wetland environment is an acidic environment due to the nature of the area. The peat soils generate low pH groundwater and hence the pH of the water in the wetland has been measured as low as 5.

The flow from the channel has the potential to have a low pH due to the inflow of groundwater and the nature of the soils in the area. The risk of causing adverse effects on the downstream environment is considered to be low given than the downstream environment is currently subject to low pH.

An Acid Sulphate Soils Management Plan (GHD, 2017n) has been prepared to address this risk.

4.4 Hydrological parameters

The following section outlines the hydrological parameters assumed for the catchment. TP108 (Auckland Regional Council, 1999) is the general approach used for the hydrological assessment. The TP108 methodology has been used in the modelling software MIKE11 for calculation of flows and channel flow.

4.4.1 Prescribed catchment

The proposed catchment area outlines the area that the stormwater conveyance channel can service for the 1% AEP event as described and outlined in Section 2.2. This is controlled by the

channel depth, capacity and the topography of the catchment. Refer to Figure 9 for an outline of the catchment.

4.4.2 Design rainfall and climate change

24 hour rainfall

For this project the design rainfall has been derived from Auckland Council's TP108 (Auckland Regional Council, 1999) with a 24-hour storm profile. The 24-hour total rainfall for each of the design storms without climate change allowances are presented in Table 3 below:

Table 3 Design rainfall

Rainfall event	24 hr rainfall (mm)
1% AEP	220
2% AEP	200
5% AEP	165
10% AEP	140
20% AEP	110
50% AEP	70

Climate change

The adopted climate change scenario for this project is to year 2090, as per the AC Stormwater COP (Auckland Council, 2015). The MfE Guidance for local government (New Zealand Climate Change Office, 2008) recommends a warming value of 2.1°C for the 2090 A1B mid-range scenario.

Based upon a 24-hour storm, the effect on rainfall per degree rise is set out in Table 4 (New Zealand Climate Change Office, 2008).

Table 4 Adopted climate change scenarios

Rainfall event	Percentage increase in rainfall
1% AEP	8.0 % increase per 1ºC rise
2% AEP	8.0 % increase per 1ºC rise
5% AEP	7.2 % increase per 1ºC rise
10% AEP	6.3 % increase per 1ºC rise
20% AEP	5.4 % increase per 1ºC rise
50% AEP	4.3 % increase per 1ºC rise

Design rainfall values

The adopted 24-hour design rainfall with climate change to 2090 used in the design is as shown in Table 5.

Table 5 Adopted design rainfall

Rainfall event	24 hr rainfall (not including climate change) (mm)	24 hr design rainfall including climate change (mm)
1% AEP	220	256
2% AEP	200	234
5% AEP	165	190
10% AEP	140	158
20% AEP	110	122
50% AEP	70	76

4.4.3 Modelling and hydrological parameters

Impervious areas

The Maximum Probable Development (MPD) impervious areas for the catchment have been assumed using the Draft Papakura Central ICMP as a base. The impervious areas in the ICMP are generally equal to or greater than the maximum allowable in the Proposed Auckland Unitary Plan zoning. The Proposed Auckland Unitary Plan allows for 60% maximum impervious area in catchment 2A and 2B. Area 2B4 is currently zoned Future Urban in the Proposed Auckland Unitary Plan, however it is expected that this land will be rezoned in the near future.

The impervious areas from the ICMP have been adjusted to account for additional impervious area from the Mill Road Block, as discussed below.

The Mill Road Corridor is proposed to run through areas 2B and 2B4, as shown in Figure 9. The alignment and size of the Mill Road Corridor has not been confirmed; however, for the purpose of this report, a possible route has been assumed which allows for a corridor approximately 1 km long, 20 m wide and 100% impervious. This additional impervious area will slightly increase the maximum impervious area (MPD) scenario as per the values in Figure 9. The three sub-catchments that Mill Road runs through will have impervious areas increased from 60% to 63%. This has been allowed for in our design flow for the Awakeri Wetlands.

Figure 9 outlines the impervious area assumptions used for calculation of design flows for the Awakeri Wetlands.

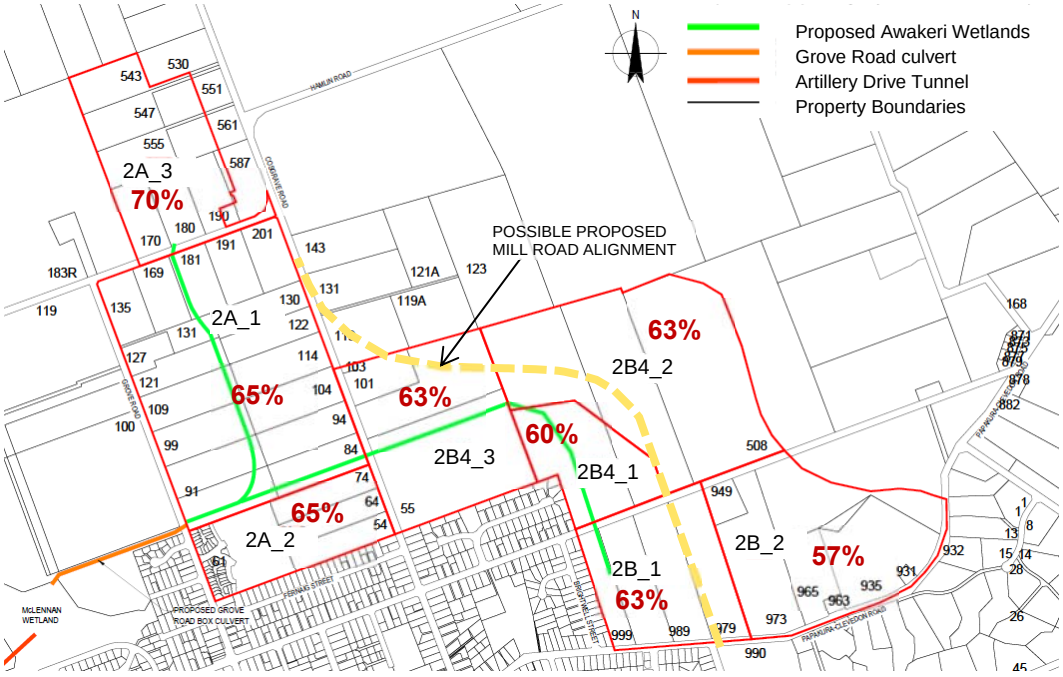


Figure 9 Impervious areas

Design curve numbers

An SCS Curve Number (CN) of 74 has been used for peat soils for the predevelopment scenario as per the Papakura ICMP, as per TP108. The post-developed scenario also uses a CN of 74 for pervious areas based on likely imported fill characteristics or existing peat soils as per above.

This aligns with the curve numbers being used by developers in the catchment.

Geotechnical observations indicate that the top crust of the soil can harden when exposed to oxygen and sheds water. This gives further support to using a curve number of 74.

An SCS Curve Number (CN) of 98 has been used for impervious areas as per the Papakura ICMP, this aligns with TP108 and other industry standards.

Channelisation factor

Channelisation factors as per Table 6 below were used.

Table 6 Channelisation factors

Surface	Factor	
Impervious areas	0.8	This is considered appropriate due to the fact that developers are required to implement recharge pits which will increase the time of concentration as water needs to pass through the granular material before discharging through a pipe. In addition, the catchment is very flat and overland flow to the channel for events greater than the 10% AEP event does not follow direct routes to the channel. Overland flow is expected to pass through “green corridors” in some areas.
Pervious areas	1.0	This is considered appropriate as the pervious areas in the catchment are expected to sheet flow onto the impervious areas once saturated with no formalised drainage pathways. In small events, water will likely soak into the ground before reaching the impervious areas. In larger events, the water will be slowed by grass / vegetation before sheet flowing onto the impervious areas.

The channelisation factors in Table 6 were used for the 50%, 10% and 1% AEP events. A sensitivity check was carried out by changing the channelisation factor for impervious areas to 0.6 for the 10% AEP model. This resulted in an increase in flow of less than 5% in the 10% AEP. This is expected to have a negligible effect on the water level in the channel, and therefore using a channelisation factor of 0.8 for impervious areas for all storm events has been considered reasonable; given the flat catchment, possible use of open stormwater systems for some areas of the catchment and recharge pits / soakage devices.

Time of concentration

The values for flow length and time of peak flow have been derived from calculations based on the TP108 methodology. The slopes and catchment lengths consider the developed slopes of the catchment draining to the proposed channel and therefore in some cases are slightly steeper than the existing gradient. These consider:

- Channel flow in the main channel.
- Pervious and impervious flow over the reduced length.

Depression storage

The significant area of flat land within the catchment area currently has the ability to store significant volumes of runoff.

Post development with the Awakeri Wetlands in place, the flow path lengths and depression storage will be significantly reduced due to filling and grading of the land towards the channel. GHD has used reduced channel lengths to reflect the geometric layout of the proposed conveyance channel layout within the catchment.

For impervious and pervious areas; depression storage of 0 and 5 mm respectively, has been used. These are the recommended values in Auckland Council’s TP108 (Auckland Regional Council, 1999).

Recharge pits

The development controls have a requirement for storage and soakage to ground for the first 15 mm of rainfall through the installation of recharge pits. We consider that the soakage will have negligible effect on the peak flows from larger events such as the 50%, 10% and 1% AEP events (which have been modelled). Therefore the 15mm soakage criteria has not been explicitly considered in the model, however, some representation is present in the consideration of channelisation factors. The presence of soakage devices has only been considered in the model for selection of channelisation factors to account for drainage pathways.

Attenuation

Generally there is limited attenuation in the catchment, as the proposed Awakeri Wetlands will convey post-development flows.

The exception is for the sub-catchment which is currently under development by Cabra Investments Ltd (refer to Figure 10). A permanent stormwater pond has been consented to attenuate flows from the Cabra development up to the 1% AEP event to pre-development levels.

The effect of the pond has been flow routed by GHD and incorporated into the hydraulic model. The peak discharge from the pond in the 1% AEP event has been modelled as 3.6 m^3/s.

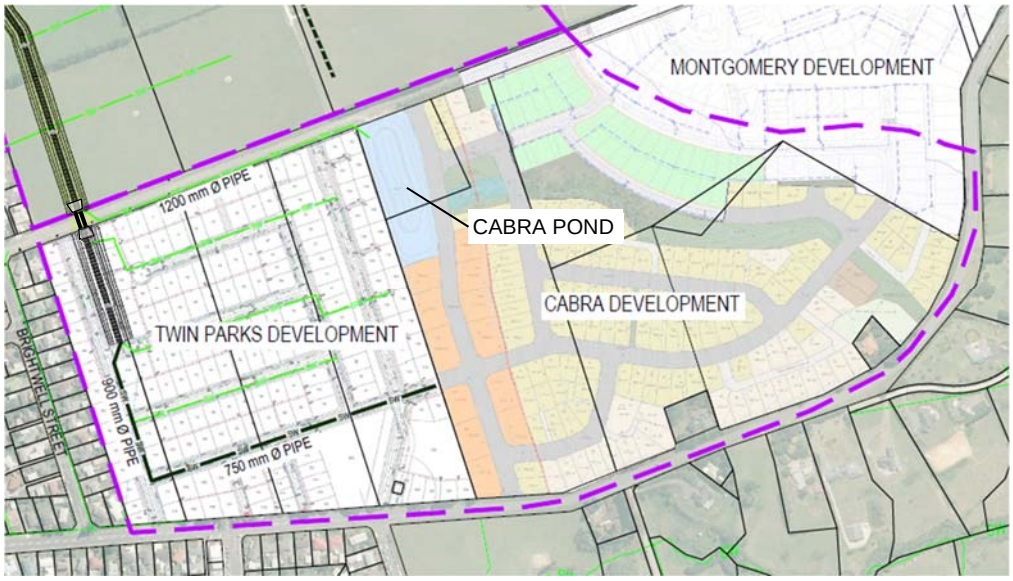


Figure 10 Cabra development and attenuation pond

4.5 Design flows

4.5.1 GHD 1D / 2D coupled model

The catchment and scheme design channel have been modelled in a 1D / 2D coupled model to determine peak flow in the catchment and flood levels within the catchment and channel. The channels were modelled using MIKE11 (1D model) and the surface has been modelled in MIKE 21 (2D model).

The sub-catchment runoff was computed by the model; using the parameters outlined in Section 4.4.

The model predicts a peak flow at the downstream end of the conveyance channel of 37.9 m^3/s for the 1% AEP storm event.

Sub-catchment loading

The sub-catchments were loaded into the Awakeri Wetlands in the GHD model as per Table 7 below (refer to Figure 9 for sub-catchment boundaries).

Table 7 Loading of sub-catchments

Sub - catchment	Loading	Explanation	Impervious area
2A_1	Distributed load along the northern branch channel.	Represents multiple incoming pipes and overland flow paths as per the expected development.	65%
2A_2	Distributed load along the main channel.	Represents multiple incoming pipes and overland flow paths as per the expected development.	65%
2A_3	Point load at top of northern branch channel.	The Mill Road Block is expected to discharge to the top of the branch channel.	70%
2B4_1	Distributed load along the channel.	Represents multiple incoming pipes and overland flow paths as per the expected development.	60%
2B4_2	Point load at CH 950.	Represents an incoming pipe or open channel connection. This sub-catchment is large and it is expected that the developer will need to construct an open channel to service their development which will connect into the Awakeri Wetlands at CH 950.	63%
2B4_3	Distributed load along the main channel.	Represents multiple incoming pipes and overland flow paths as per the expected development.	63%
2B_2	Point load downstream of the Old Wairoa Road Culvert.	Represents the proposed connection location of the Cabra Pond discharge pipe.	57%
2B_1	Point load at top (upstream end) of the main channel.	Assumes the development discharge to the top of the channel via a pipe or overland flowpath.	63%

The *modelled flow and hydraulic grade line are plotted on the channel longsections on drawings 51-33411-C121-C129.*

MIKE11 model outputs

Drawings 51-33411-C111-C117 shows the modelled Awakeri Wetlands and the chainage along the channel. Refer to Table 8 for MIKE11 model outputs. Further model outputs are included in Appendix A.

Table 8 MIKE11 model outputs - design peak flows

	MIKE11 modelled peak flow (m^3/s)		
Chainage (m)	50% AEP	10% AEP	1% AEP
Main Channel			
0	10.2	25.2	40.2
100	10.1	25.0	39.9
150	10.0	24.9	39.7
200	6.0	15.4	24.5
300	5.9	15.2	24.0
400	5.8	14.8	23.5

	MIKE11 modelled peak flow (m^3/s)		
Chainage (m)	**50% AEP**	**10% AEP**	**1% AEP**
500	5.7	14.6	23
600	5.6	14.3	22.6
700	5.2	13.5	21.3
800	4.9	12.6	19.9
900	4.5	11.6	18.5
950	4.3	11.2	18.0
1000	1.9	5.8	9.6
1100	1.8	5.6	9.2
1200	1.6	5.3	8.7
1300	1.5	5.1	8.2
1400	1.4	5.0	7.9
1500	1.4	3.2	4.7
Northern Channel			
60	4.1	9.4	15.0
200	3.7	8.5	14.4
300	3.0	7.0	11.5
400	2.4	5.5	9.2
500	2.0	4.4	7.4
600	1.6	3.6	6.0
700	1.2	2.8	4.6

4.5.1 Validation of flows

A HEC-HMS model was prepared by GHD to compare and confirm the predicted flows from the MIKE11 modelling. The peak flow predicted by the HEC-HMS model in the Awakeri Wetlands at Grove Road is 39.1 m^3/s.

The channel was represented in HEC-HMS as a series of reaches linked together with junctions. Lag time for each reach was based on expected flow velocities along the length of each reach. Velocities and corresponding lag times for each reach have been assumed as per Table 9.

Table 9 Lag times and flow velocity

Reach	Velocity (m/s)	Lag time (min)
Main channel		
CH 0 - 160	1.50	1.8
CH 160 - 550	1.00	6.5
CH 550 - 950	1.00	6.7
CH 950 - 1400	0.80	9.4
CH 1400 - 1540	0.50	4.7
Northern branch		
CH 0 - 300	0.75	6.7
CH 300 - 550	0.90	4.6

The catchment was represented by a series of sub-catchments which were split into separate impervious and pervious catchments, with the catchment parameters as per Section 4.4. Each sub-catchment was loaded into the channel at junction points. This is expected to give a good

representation of the flow at each junction. However between junctions the flow rate could be deduced from interpolation.

The Cabra pond has been represented in HEC-HMS by a reservoir linked to an Elevation-Area Function and an Elevation-Discharge Function which was derived from the pond routing carried out by GHD.

The flow predicted by the HEC-HMS model matches the MIKE11 modelling and confirms that the peak flow predictions are within acceptable levels of accuracy suitable for the purpose of this design.

Refer to Appendix B for HEC-HMS model outputs.

4.6 Sensitivity analysis

A sensitivity analysis was carried out on the impervious area assumptions for the catchment. The 1% AEP model was run using a base of 70% impervious area for each sub-catchment, adjusted further as above for the Mill Road Corridor (+3% for the three sub-catchments that Mill Road runs through). This resulted in a less than 3% increase in flow for the 1% AEP event which is expected to have a negligible effect on the water level in the channel.

4.7 Hydraulic modelling

The Awakeri Wetlands Scheme design was modelled in MIKE11 to determine the hydraulic grade line in the channel for the 50%, 10% and 1% AEP events. The model was checked using spreadsheet calculations based on Bernoulli's energy principle and Manning's flow equation (using Flowmaster).

Channel cross sections were input into the model at 20 m spacing. Channel cross sections, roughness, culverts and catchment parameters were used to match the values described in Section 4.4 and of this report.

The model confirms that the channel design is adequate for conveying the 1% AEP event with adequate freeboard. In addition, the hydraulic grade line is maintained at a low enough level to provide drainage of the surrounding land developments; this is further discussed in Section 8. Refer to Drawing 51-33411-C121-C127.

Refer to Appendix A for the MIKE11 model outputs.

4.7.1 Channel hydraulic parameters

Manning's numbers

The adopted Manning's numbers for the Awakeri Wetlands align with the recommended values in Christchurch City Council's Waterways, Wetlands and Drainage Guide (Christchurch City Council, 2003). The above publication was used as it contains Manning's numbers for stream surfaces that are similar to the proposed vegetation and channel profiles of the proposed Awakeri Wetlands.

Adopted Manning's n numbers

The following Manning's numbers have been used for the hydraulic analysis. These have been selected assuming:

1. The low flow channel is maintained to keep clear of obstructions and prevent excessive weed growth. n = 0.030
2. The wetland plants are lay flat species and will flatten during flood events. n = 0.045

3. The flax and native grasses on the channel bank are maintained to keep clear of excessive weeds. The plant species are assumed as a mixture of those that can flatten during flood events with some heavier shrubs less than 1 m tall. n = 0.060

4. The grass is maintained at a short length and specimen trees are scattered throughout the floodplain. n = 0.045

Table 10 Manning's numbers for conveyance channel design

Section	Surface Cover	Manning's number (n)
Low flow channel	Naturalised channel with pools and slight channel meander	0.030
Wetland bench	Wetland grasses	0.045
Channel bank	Flax and native grasses (<1 m tall)	0.060
Floodplain	Mowed grass with footpath and specimen trees	0.045

4.7.2 Culverts

There are two culverts within Stage 1 of the Awakeri Wetlands.

Losses through the culverts were checked using Bernoulli's Energy equation and the losses were determined to be sufficiently low to not significantly impact the hydraulic grade line of the channel. This is further discussed and detailed under Section 10.

5. Channel Design

5.1 Design basis

The design of the Awakeri Wetlands has been driven by a number of factors. These are recorded below along with a brief commentary of the effects of each on other aspects of the design.

- The design philosophy in having weirs along the channel length is to maintain low flow water as high as is practical in order to limit the groundwater drawdown and provide for the development of aquatic habitats.
- A second parameter is that the weirs should not cause more than a modest rise in the 1% AEP flow profile.
- The design has considered the ability to drain all of the catchment with minimal site filling to maintain minimum freeboard to habitable floor levels.
- The setting of the 1% AEP flood level has been determined at sufficient depth to allow the channel to operate as an open waterway whilst minimising the overall depth and allowing overland flow from the catchment extents to flow by gravity to the channel and be unaffected by backwater effects from the flood level in the channel.
- During low flow there will be a series of discrete water bodies or wetlands. Each water body will be nominally 100 m long and be separated by a weir structure to maintain a permanent water surface.
- The wetland bench channel is important for flow, ecological, aesthetic and safety reasons. The wetland bench will contain plants, whereas the low flow channel will be deep enough to prevent or limit plant growth.

5.1.1 Channel geometry

Defined zones

The channel has been designed to allow for the following zones:

1. **Low flow channel**

   A meandering low flow channel with a permanent water depth varying between 0.5 m – 1.0 m (typically 0.8 m) controlled by the weirs at 100 m centres longitudinally along the base of the channel. The base of the low flow channel typically varies between 3 -6 m wide with slope batters 2H:1V. The 2H:1V batters are generally only 0.6 m high, below water level and are lined with a granular material, hence it is considered acceptable from a safety and slope stability perspective to have these greater than 4H:1V.

2. **Wetland bench**

   A slightly meandering wetland bench above the low flow channel that varies in width as the low flow channel meanders within it. The wetland bench is part of the permanent flow channel and the intention is for this zone to be within the permanent water level provided for by the weirs. The wetland bench will be planted with wetland species, is nominally flat and has a permanent water depth of 0-0.2 m. The wetland bench provides ecological, water quality and safety benefits.

3. **10% AEP water level**

   The channel bank is battered at 4H: 1V or flatter to a height between 0.70 m and 1.5 m to allow for conveyance of the 10% AEP. The batters will incorporate riparian planting, as per

the planting plan in the Urban and Landscape Design Analysis Report (GHD, 2014) and as specified by Auckland Council. Paths and boardwalks extend through this area.

4. **1% AEP water level**

   The 1% AEP flood area is above the 10% AEP flood level and includes a mix of planted areas, paths, play space, grass and trees.

Flooded areas

The extent and depth of flooding from the 50%, 10% and 1% AEP events has been extensively discussed with Auckland Council. Auckland Council have adopted a design that has paths and boardwalks that will be submerged in events greater than the 50% AEP event and as such will not be available for public use.

In general the velocities in the channel are relatively low (<1 m/s) except at weir locations and 40 m upstream of the Grove Road culvert inlet.

Side slopes / channel batters

Generally, slope batters have been designed at 4H:1V or flatter to fit in with the landscape design and as per the recommendations from the Geotechnical Investigations Report (Technical Report C).

Steeper batters (2H:1V) in the low flow channel have been considered suitable for the following reasons:

- Being fully submerged improves slope stability from a geotechnical perspective and discourages access by pedestrians – hence improving safety.
- Having a granular lining improves stability from an erosion perspective and provides a stable / traversible surface if accessed by pedestrians.
- Low height (approx. 0.6 m) allows the slopes to be traversed by pedestrians who may enter the channel.

The channel sections have been modelled in the Geotechnical Investigations Report (Technical Report E).

Overall depth and width

The overall depth of the channel has been designed as shallow as possible for the following reasons:

- Minimise groundwater drawdown and associated potential ground settlement of adjacent land and structures.
- Maintain stable side slopes / channel batters.
- Minimise excavation volumes.

The main channel ranges in depth from between 1.9 m to 4.0 m bgl to the base of the channel. The overall total width of the main channel at the 1% AEP water level ranges from 20 m to 37 m.

The northern branch channel ranges in depth between 2.4 m to 3.8 m bgl to the base of the channel. The total overall width of the northern branch channel at the 1% AEP level ranges from 12 m to 27 m.

Planting

Planting has been selected by Auckland Council and generally consists of native grass species and sedges that would lay flat during large flow events. Tree species will have most of their mass above the 1% AEP event and therefore would not have a significant impact on the channel roughness. These include cabbage tree and kahikatea.

Paths

There are shared paths and boardwalks within the Awakeri Wetlands which allows public access along the corridor. Parts of the paths are expected to flood occasionally, with paths and boardwalks closer to the water surface flooding more frequently. Paths within the 10 year ARI event flow typically have alternative routes which would allow public access around flooded areas.

5.1.2 Channel alignment

The overall alignment of the corridor is linear, however the low flow channel varies in width, depth and direction to create variation in habitat. Refer to Drawing 51-33411-C211 for typical sections of the channel.

5.1.3 Channel bed slope

The overall gradient of the main channel from Old Wairoa Road at IL 23.97 m at the top of the channel falls to IL 19.80 m at Grove Road over a distance of approximately 1.55 km. This is an approximate overall gradient of 0.28%.

The overall gradient of the northern branch channel from Walters Road at IL 21.48 m at the top of the channel falls to IL 20.10 m at the junction with the main branch over a distance of approximately 0.70 km. This is an approximate overall gradient of 0.20%.

There are 9 major weirs designed along both channels. At very low flow, the hydraulic gradient is flat. The bed of the channels between each weir is also flat except where there is a variation in depth.

5.1.4 Low flow channel

The low flow channel depth has been selected based on a combination of water quality, flow characteristics, safety and industry guidelines.

The width of the low flow channel varies significantly along the alignment of the Awakeri Wetlands. The low flow channel for the Awakeri wetlands is unique in that it operates as both a conveyance channel and a wetland, therefore typical design guidelines for channels **or** wetlands cannot be applied directly – however the design has been based on the principles of a number of different design guidelines as discussed below.

The permanent water level in the channel varies throughout its length with a depth ranging from 800 mm to 1200 mm for the deeper sections and 200 mm for the wetland bench areas. These depths align with the principles for design of wetlands in Auckland Council's TP10.

No design recommendations for low flow channel depths or widths have been found in any Auckland Council or New Zealand design standards for similar channel designs.

The Queensland Urban Drainage Manual (Department of Energy and Water Supply, 2013) recommends a depth of 0.45 m for a low flow channel with a base width of 2.0 m.

The width of the Awakeri Wetlands is within this order of magnitude and has similar proportions, however it is typically deeper and wider due the specific project requirements including:

- Sufficient flow capacity required for conveyance of large events.

- Low velocities during low flow to minimise erosion.
- Fixed water levels to manage groundwater drawdown impacts.
- Safety (shallow / 200 mm around the perimeter of the open water and maximum depth that allows an adult to walk through).
- Water quality (sufficient volume/depth to manage temperature fluctuations).
- Variety (varying depths for ecological purposes).
- Aligns with the principles for design of wetlands in TP10.

5.1.5 Evaporation

There is a risk of lowering of the water level in the low flow channel due to evaporation. Lowering of the water level in the low flow channel could result in the following key issues:

- Drying out of the wetland bench areas.
- Die off of wetland plants.
- Odour issues.

It is expected that the wetland grasses in the wetland bench adjacent to the low flow channel, and some of the larger plant and tree species in the riparian margin (cabbage tree, kahikatea) will provide shading to the channel. This will help control temperature and evaporation while also providing additional ecological benefit.

As discussed above, planting will provide some mitigation for this risk, however additional management is recommended for the operation and maintenance of the channel, including:

- Monitoring of water levels.
- If evaporation issues are found to be an issue, additional mitigation can be installed to recharge water into the wetlands. This could include pumping from a nearby water source into the wetlands.

An assessment of historical rainfall and evaporation rates in the area has been undertaken to assess the likelihood and scale of this risk (provided in Appendix D), however there are a number of factors that cannot be modelled accurately and hence monitoring should be the key tool for assessing this risk.

5.1.6 Water balance

Main channel - above Cosgrave Road

Based on ground water balance models above Cosgrave Road we expect the dry summer low flow to have a surplus of water and a base flow in excess of 3 l/s. Historically, groundwater levels rise above the channel level, and hence there is expected to be flow from groundwater into the channel.

Main channel - Cosgrave to Grove Road

Groundwater has been observed in this area to drop lower than the proposed low flow water level during dry periods. Due to this, and from evaporation, we expect that there could be a net loss of water in the lower part of this area (near Grove Road) during extended dry periods.

Northern channel

The northern channel low flow water level is set close to the seasonal low groundwater level. For most of the year, the groundwater is typically expected to be above this level and therefore

a baseflow is likely to be achieved in the channel for most of the year. This flow could decrease during extended dry periods. Monitoring of low flow water levels throughout the Awakeri Wetlands is recommended.

5.1.7 Operational water levels

The operational water levels for the 10% AEP and 1% AEP flows vary along the channel/wetland but typically are in the order of those shown in Table 11.

Table 11 Operational water levels

Channel zone	Typical water level above channel invert (m)
Low flow	0.80-1.20
10% AEP	1.40
1% AEP	1.70

6. Scour protection

Scour and erosion potential is an important consideration for the design of the Awakeri Wetlands. Scour and erosion of the channel could potentially result in poor amenity, discharge of sediment into the downstream receiving environment and bank stability issues for adjacent structures.

The peat soils which the Awakeri Wetlands will be constructed in are particularly soft and susceptible to scour and undercutting. Evidence of this can be observed at the McLennan Wetlands, the Bruce Pulman Park ponds and the table drains on Cosgrave Road and Walters Road, where the channel banks are being undercut by the open water surface.

Potentially high velocities and shear stresses in the channel pose the biggest risk of scour and erosion to the channel banks. Velocities are expected to be low during small rainfall events and scour and erosion is not considered to be an issue. In larger events, such as the 1% and 10% AEP, velocity and shear stress is higher and scour and erosion protection has been incorporated in the design to address this.

6.1 Scour and erosion protection design philosophy

Two approaches have been considered for the scour and erosion protection design.

1. **Hard engineered, fully mitigated approach**

   This approach would include providing scour and erosion protection along the entire channel banks within the 10% AEP flow area. This would consist of a mixture of granular material / rip rap and other proprietary devices such as Geoweb, blown bags and Reno mattresses applied extensively throughout the channel.

   The hard engineering solution would provide a reduced chance that retrospective scour and erosion protection would need to be installed post-construction, however it would also require a significantly higher up-front cost and overall would result in a less attractive asset from a landscaping, ecological and public amenity perspective.

2. **Risk based approach**

   This approach would consist of providing scour and erosion protection at critical areas only where the consequence of scour and erosion could cause damage to key structures and would impact directly on the performance of the channel as a hydraulic asset and amenity feature. This would consist of installing granular material and other proprietary devices such as Geoweb and Reno-mattresses at locations around key structures and at high velocity locations in the channel such as:

   - Where the paths are in proximity to the low flow channel (key structures).
   - At weir locations (high velocities).
   - Around boardwalk or bridge locations (key structures).
   - At culvert inlet and outlets (high velocities).
   - Along steep slopes (high risk).
   - Within the low flow channel (high risk).

   The remainder of the areas within the 10% AEP flow area are at risk of scour and erosion, however the immediate consequence is considered low because any issues can be detected during inspections and remediated if required. This risk based approach would

be adopted, coupled with on-going monitoring of scour and erosion and that this should form part of the operation and maintenance manual for the Awakeri Wetlands.

If scour does become an issue in these areas and it is left to continue over a period of time (ie. monitoring and maintenance is not undertaken), it is possible that the consequence could become critical such that:

- Deep vertical channel slopes are formed.
- Channel batters approach adjacent properties and/or cause slope instabilities.

This philosophy has been discussed with Auckland Council who have indicated that this is the preferred approach and that there is a strong driver to keep the Awakeri Wetlands as natural as possible.

6.1.1 Discussion with Auckland Council

As mentioned above, Auckland Council have requested that the Awakeri Wetlands are naturalised as much as possible and that exposed granular material is kept at a minimum. The scour risks have been communicated and it has been agreed that a higher level of scour risk will be accepted by Auckland Council in conjunction with monitoring to keep a natural finish to the channel batters.

Monitoring will be undertaken in accordance with the Operation and Maintenance Manual for the channel which has been prepared by Auckland Council.

Auckland Council have advised that the selected plants will have extensive root systems that are expected to provide reinforcement to the soil and on this basis have adopted that the risk based approach is their preferred option.

6.1.2 Discussion of recommended approach

The risk based approach is the recommended option. The risk based approach has the following key benefits over the hard engineered solution:

- **Cost effective:** Reduced construction costs due to reduced quantities of imported construction materials. This approach requires special attention to monitoring and retrospective remediation for repairing any identified areas, however it allows economical use of resources as additional mitigation can be applied to known areas of scour that are not possible to predict in advance such as:
  - Soft, loose ground conditions encountered at low flow channel level.
  - Obstructions / logs encountered and left in place during construction.
- **Aesthetics:** The risk based approach allows increased areas of planting in natural soils to give more vegetation cover and less exposed granular material.
- **Environmental:** The risk based approach allows greater density and extent of planting in natural soils. This provides more wildlife habitat, shading for the low flow channel and materials are less prone to heating up to help manage temperature of the low flow channel.

6.1.3 Adoption of risk based approach in design

The primary method of scour protection throughout the channel is reliance on the root systems of the plants to reinforce the channel banks and soils. At areas within the Awakeri Wetlands that are considered at high risk of scour or where scour will have a high consequence, planting is proposed to be coupled with Geoweb and Enkamat to reinforce the plant roots.

In areas where velocities and shear stresses are especially high, the Geoweb will be filled with granular material or a mixture of granular material and topsoil for planting.

Granular material by itself has only been accepted by Auckland Council where it is permanently submerged and not visible. Where there is a high risk of scour above water, such as around the weir locations; a mixture of granular material and topsoil for planting is proposed within the Geoweb cells.

Further detailed of how this approach has been implemented are provided in the following sections.

6.2 Channel velocity

The peak 1% AEP flow velocity has been calculated and is approximately 1.3 m/s just upstream of Grove Road before dropping into the fish passage leading down to the Grove Road culvert inlet structure (Refer to Table 12). At Cosgrave Road the peak 1% AEP velocity is 1 m/s. The northern channel typically has velocities less than 1.0 m/s. This excludes the peak localised velocity of water flowing over the weir sections, where there is an expected increase.

Average velocities have been calculated along the channel and are noted in Table 12.

Table 12 Average channel velocities

	10% AEP		1% AEP	
Chainage (m)	Q (m3/s)	V (m/s)	Q (m3/s)	V (m/s)
Main Channel				
0	25.2	1.34	40.2	1.71
100	25.0	1.01	39.9	1.2
200	15.4	0.84	24.5	1.06
300	15.2	0.84	24.0	1.06
400	14.8	0.81	23.5	1.02
500	14.6	0.44	23.0	0.6
600	14.3	0.79	22.6	0.93
700	13.5	0.75	21.3	0.89
800	12.6	0.71	19.9	0.88
900	11.6	0.78	18.5	1.00
1000	5.8	0.68	9.6	0.83
1100	5.6	0.64	9.2	0.81
1200	5.3	0.62	8.7	0.79
1300	5.1	0.45	8.2	0.55
1400	5.0	0.31	7.9	0.93
1500	3.2	0.42	4.7	0.43
Northern Channel				
60	9.4	0.78	15.0	0.87
200	8.5	0.63	14.4	0.76
300	7.0	0.59	11.5	0.73
400	5.5	0.54	9.2	0.74
500	4.4	0.49	7.4	0.67
600	3.6	0.78	6.0	0.80
700	2.8	0.55	4.6	0.31

The velocities in Table 12 represent the average velocities over the full cross sectional flow area. These velocities are low and are generally less than 1 m/s. In storm events smaller than the 1% and 10% AEP, velocities are expected to be lower.

It is estimated that the velocity at the downstream end of the main channel is approximately 0.6 m/s in the 50% AEP and 0.3 m/s in the 100% AEP. These velocities are low and are not expected to cause significant scour or erosion in the channel.

6.3 Shear stress

Bed shear stress in the channel has been extracted from the MIKE11 model as per Table 13 below.

Table 13 Average channel shear stress

	10% AEP		1% AEP		
Chainage (m)	Q (m^3/s)	Bed Shear Stress (N/m^2)	Q (m^3/s)	Bed Shear Stress (N/m^2)	Comments
Main Channel					
0	25.2	17.51	40.2	27.40	
100	21.9	9.15	37.6	12.65	
200	14.0	9.86	24.2	10.61	
300	13.7	6.99	23.6	10.73	
400	13.4	6.42	23.2	9.81	
500	13.1	1.43	22.7	8.26	
600	12.9	6.21	22.3	8.37	
700	12.2	5.56	21	7.77	
800	11.5	5.05	19.8	7.51	
900	10.7	6.18	18.5	9.80	
1000	5.9	4.70	9.6	6.86	
1100	5.7	4.12	9.2	6.52	
1200	5.4	3.89	8.7	6.18	
1300	5.1	2.04	8.2	3.02	
1500	2.8	1.04	4.7	1.02	
Northern Branch					
60	9.4	6.42	15.0	7.82	
200	8.5	4.78	14.4	5.82	
300	7.0	4.30	11.5	5.90	
400	5.5	3.33	9.2	4.95	
500	4.4	8.55	7.4	12.13	500 mm deep channel drops into 800 mm deep channel
600	3.6	3.67	6.0	4.90	
700	2.8	0.65	4.6	0.87	

Erosion occurs when the shear stress of the flow exceeds the strength of the soil particles along the surface of the bed. Peat soil is highly erodible and the bed shear stresses from the channel flow is expected to surpass the critical value for exposed peat. The critical shear stress values for peat is typically highly variable depending on the level of decomposition and disturbance, and can range from <1 N/m^2 up to 5 N/m^2 (Tuukkanen, T., H. Marttila, and B. Klove, 2014)

The bed shear stress experienced in the Awakeri Wetlands peaks at approximately 27-30 N/m^2. The higher values are predicted mostly at the downstream end of the main channel, at the confluence of the two channels and at the weir locations.

The peat soil in this catchment is therefore susceptible to sediment transport and as a result, the channel bed is proposed to be protected with a Bidim+Geogrid composite material held down with 70kg/m^2 of GAP 65 hardfill. This will minimise the degradation and the aggradation of the channel invert.

6.4 Scour and erosion risk

The surface cover of different zones within the channel provides varying levels of resistance against scour and erosion. Table 14 outlines the surface cover types and the expected performance in regards to scour and erosion due to flow in the channel.

Table 14 Scour and erosion risk for channel zones

Zone	Surface Cover	Risk of scour / erosion	Protection measures
Typical channel areas			
Low Flow Channel	Naturalised channel with pools and slight channel meander	High susceptibility to scour and erosion.	Geogrid+Bidim composite with gravel lining along base of the channel and 1(V):2(H) submerged slopes.
Wetland Bench	Wetland grasses	Low risk. Wetland grasses will slow velocities and roots will strengthen soils.	No additional protection required.
Channel bank	Sedges and native grasses. Small unrestrictive trees with mass of branches above 1% AEP.	Low risk. Roots of grasses and trees will strengthen channel banks.	No additional protection required.
Floodplain	Sedges, with footpath and specimen trees	Low risk. Roots of grasses and sedges will naturally protect from scour and erosion.	No additional protection required.
Critical areas			
Between paths and water body	Sedges and native grasses. Small unrestrictive trees with mass of branches above 1% AEP.	Low risk. Roots of grasses and trees will strengthen channel banks. High consequence of undercutting paths.	Geoweb along slope with soil / gravel infill and planted.
Weirs	Sedges and native grasses. Small unrestrictive trees with mass of branches above 1% AEP.	High risk. Velocities high as water flows over weir.	Reno Mattress and Gabion basket immediately downstream of weir. Geoweb with GAP80 and Geoweb with planting and topsoil around weir.

Boardwalks	Boardwalk abutments and adjacent ground	High risk, high consequence. Undercutting of different materials at boardwalk and bridge locations.	GAP65 backfill immediately around structure and Geoweb with a mixture of granular material and planted topsoil

The low flow channel side slopes are proposed to be lined with a granular material to prevent scouring of the sides of the low flow channel and to provide solid side slopes which would facilitate safety aspects if someone were to enter the channel. The base of the channel will be protected with a Geogrid / Bidim composite, weighed down with GAP65 at 70kg/m^2.

6.5 Planting

Wetland plants, sedges, native grasses and small trees have an ability to withstand the generally expected velocities (<1.4 m/s) without adverse effect. The planting of the channel will provide stability to the soils to resist against scour and erosion once plants are established.

In early years before plants and roots are fully established, the channel will be more susceptible to erosion. However it is unlikely that the catchment will be fully developed within this time, and therefore peak runoff and velocities are expected to be much less, hence mitigating this risk.

Plants are being relied on to provide the bulk of the erosion and scour protection for the channel and the ability of their roots to provide this function has been confirmed by the planting designer at Auckland Council. The channel slopes will need to be protected temporarily while the plants are establishing. The planting designer indicates that within 1-2 years the roots will have grown extensively through the soils and will aid greatly in stability of the slopes. Coir matting is proposed to provide temporary protection to the channel slopes during the 1-2 year establishment phase. Coir matting will slowly biodegrade over time but typically lasts for a period of 2-3 years.

Full development of the catchment is not expected to be completed for some years after the construction of the channel. As such peak flow rates will be less than the MPD scenario during this time. This will allow additional time for the wetland plants to become established and grow.

As discussed in Table 14, soil filled Geoweb with Enkamat underneath will be installed at critical areas where the consequence of scour is high (such as around footpaths and structures).

6.5.1 Geoweb

Geoweb is frequently used in slope protection and stormwater channel applications. It provides structural confinement of topsoil/vegetation and granular materials such as sand, gravel and larger rock or stone. An example is shown in Figure 11 below.

Figure 11 Geoweb (https://www.geofabrics.co)

The Geoweb proposed in the Awakeri Wetlands project will typically extend from the lower side of footpaths down and across the wetland bench of the low flow. Hence it will be partially submerged. Native grasses and shrubs will be planting into the Geoweb cells in accordance with the planting plan. The Geoweb will provide confinement for the topsoil and some support for the planting across the surface of the channel. The cells will also aid in capturing sediments and material that may be lost due to erosion, before these materials are discharged downstream.

6.5.2 Enkamat

Enkamat is proposed beneath the Geoweb to provide additional support for the plant roots. Native grasses and shrubs that are planted within the Geoweb cells and above the Enkamat layer will be small and will take time to establish. During the establishment phase, the roots will grow through the Enkamat, providing additional reinforcement to the plants. This will reduce the risk of plants being lost during storm events and is expected to reduce the long-term maintenance costs of the channel.

Enkamat is usually installed close (25 mm) to the ground surface, because typically the surface above the Enkamat is grassed with a turf grass with shallow roots. Having the Enkamat at a shallow depth provides support to the shallow roots. Given that the planting in the Awakeri Wetlands consists of larger native grass species with deeper roots, the Enkamat has been proposed deeper, and to be laid beneath the Geoweb (100 mm deep).

Enkamat is shown in Figure 12 below:

Figure 12 Enkamat example

As a proof of concept, a high level trial was undertaken which consisted of planting two plants within a container on top of a layer of Enkamat (MacMat R) with approximately 100 mm of topsoil above it.

6.5.3 Proof of concept trial

Two plants were selected which were readily available from a local store which represented an approximation of the type of plants that would be used within the relevant areas of the TSCC. The two plants used were:

- *Carex secta* (Purei)
- *Phorium tenax* (New Zealand Flax)

MacMat R was used which is Enkamat with a layer of steel wire reinforcement. A schematic is shown in Figure 13.

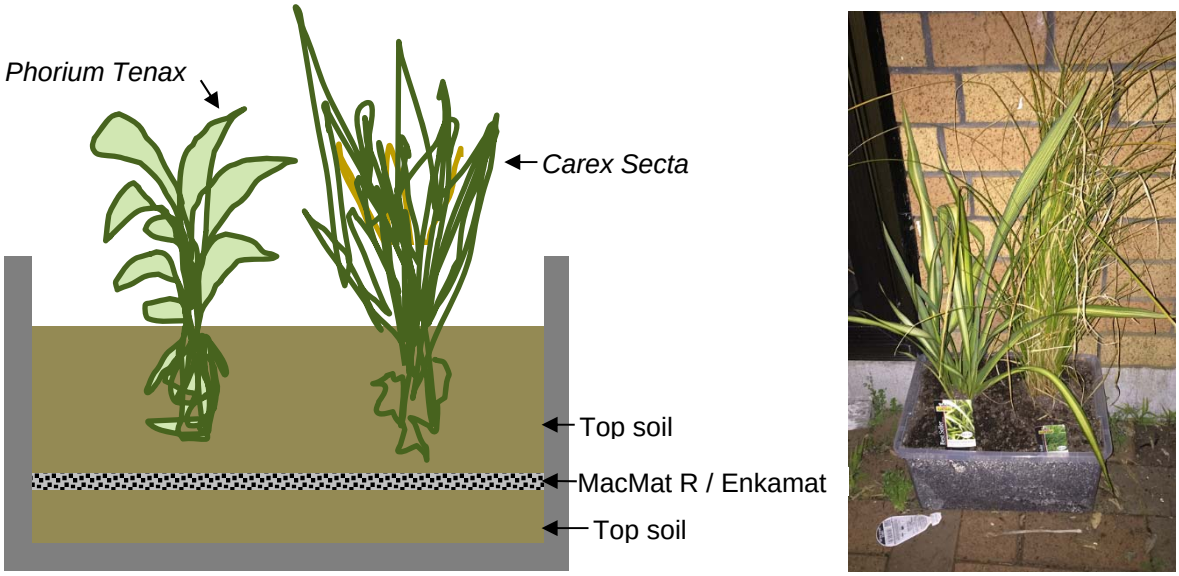


Figure 13 Trial set up 05/06/2017

After 2 months, the plants were removed to observe whether the roots had grown through the Enkamat.

Figure 14 shows some photos after 2 months indicating that the roots had grown through and interwoven with the Enkamat, hence indicating that the Enkamat will provide some support to the plants.

Figure 14 End of trial 05/08/2017

6.6 Confluence main channel and northern branch (Ch 200)

At the confluence of the main channel and northern branch there is a large body of water. This body of water is expected to provide energy dissipation as the two channels come together. Given the low velocities in each channel and the large volume of water at this confluence, the erosion and scour potential of the flow is expected to be low. The expected bed shear stresses are expected to be resisted by the bidim+geogrid composite with the aggregate covering.

The purpose of the aggregate is to hold down the bidim+geogrid on the bed of the channel. The geogrid by itself is neutrally buoyant, and therefore requires aggregate to reduce the chance of it coming loose.

No additional or special scour protection is proposed at this location, other than what is already proposed in the previous discussion.

6.7 Sediment deposition

6.7.1 Estimated sediment deposition

The typical runoff from a developed Auckland catchment will be in the order of 0.5 t/ha/annum. This is based on soil types generally consisting of Waitemata clays and would occur when all bulk earthwork development has been completed and individual housing sites are developed. In the case of this development there is expected to be areas of recent peat alluvium as per the existing soils, in addition, there is expected to be imported fill from developers. Slopes in this catchment are very flat and therefore it is expected that the runoff will be towards the lower range of any variance around 0.5 t/ha/annum.

The steep portion of the 2B catchment will drain to a stormwater pond at the Cabra Development site, and therefore sediment removal is expected for this area.

We can also expect that a portion of sediment will be entrained and passed through the system down to the McLennan wetland and Pahurehure Inlet during high flow events. We therefore expect the residual sediment deposition in the channel to be in the order of 0.25 t/ha/annum. If this deposition is evenly distributed along the channel, then the catchment area/channel length (155 ha / 2,100 m = 0.74 ha / lineal meter) relates to an annual deposition of 18 kg per lineal meter of channel per annum. We would expect some of this to be deposited below the permanent water level.

6.7.2 Maintenance

The annual estimated deposition rate is between 1.0 - 1.5 mm/annum. At this rate, it would take between 60-100 years for 100 mm of sediment to build up along the channel. This may not be distributed evenly, and would likely be distributed along the wetland planting area, the main low flow channel and behind the weirs. It is expected that maintenance to remove sediment would be required approximately every 20-50 years. This has been allowed for by provided access to key areas.

This has been discussed and agreed with Auckland Council Healthy Waters Operations Team.

6.8 Other channel features

6.8.1 Swamp Kauri and obstructions

There is a high likelihood of digging up timber including Swamp Kauri tree logs, stumps and trunks during the excavation. In accordance with the Curatorial Framework, any Kauri that is dug up and removed from the channel shall be retained on site and stockpiled for potential re-use. If the Kauri cannot be excavated without damaging the channel batters then it shall be left in place unaltered and flagged within the design team for potential utilisation in-situ within the channel design.

Logs or obstructions cannot be excavated where they penetrate the channel profile as backfilling to form the channel profiles is not practical.

An assessment will need to be made on a case-by-case basis on how to utilise the Swamp Kauri and other timber. Hydraulic, geotechnical and structural risks will also need to be considered.

Options for managing logs or obstructions include:

- Leaving the obstruction in place within the channel profile.
- Cutting the obstruction and leaving the remainder in place.

These options will need to consider how the obstructions will integrate into the final form of the channel, including assessment of any hydraulic issues, scour and erosion issues and landscaping elements.

7. Weirs

In order to maintain a permanent waterbody within the wetland channel, a series of weirs at notional 100 m centres will be used to maintain these bodies of water. The depth of water behind each weir is 800 mm with a depth of 200 mm along the wetland bench. As well as providing for aquatic habitat, the permanent water level will assist in reducing groundwater drawdown and associated potential settlement by maintaining the groundwater at a level higher than the channel invert.

The top surface of the weirs ranges between 9 m to 14 m across.

The step between each weir varies from 0.18 m to 0.45 m to give an overall average gradient along the full channel length and to facilitate fish passage and to provide hydraulic controls. At medium and high flows these weirs will be totally drowned. The depth of the 1% AEP event flow above the top of the weir level has been calculated as up to 1 m deep.

As the flow increases (during a flood event), the flow over the weir increases and the flow in the channel downstream of the weir raises at a faster rate until the weir is almost drowned. Prior to the weir being drowned the flow becomes critical over the weir and the velocities will be at a maximum. The extent of increase will depend on the difference in water level above and below the weir.

7.1 Main structure

The main structure of the weir consists of PVC sheetpiles down to 6 m below ground. This is required to create an impermeable hydraulic cut-off and to maintain stability of the weir structure. An impermeable cut-off is required to maintain the permanent body of water behind the weir to reduce permanent groundwater lowering within adjacent land which can cause ground settlement.

PVC sheetpiles were selected because they provide the following advantages:

- Durable and resistant to acid / low pH (compared to steel sheetpiles).
- Easy to install along the channel following excavation of the ground profile.
- Reduced risk of ground settlement (compared to a concrete structure due to weight).
- Top surface of the weir can be readjusted to account for any movement by alterations to the timber facing on top of the weir.

The proposed method of installation is to drive the sheetpiles and leave approximately 200 mm high to allow any negative skin friction forces to dissipate. Following this rest period, the PVC sheets will be cut to length and capped with a hardwood timber beam. The beam has a low point in the middle and slopes up gradually towards the channel batters. This provides hydraulic benefits in larger storm events by concentrating flow towards the centre of the channel.

Stability calculations for the weir are provided in Appendix D.

7.1.1 PVC sheet pile stability

The stability of the sheet pile walls have been checked based on kick-out calculations, as attached in Appendix D. The calculation shows that the sheet pile is stable under the proposed conditions.

The Wallap software was also used to determine displacement of the sheet pile. Displacement of the sheet pile under the proposed conditions is less than 50 mm at the top of the sheet pile. This is considered acceptable.

7.2 Fish passage

Fish passage is provided for at each weir. A series of timber beams and notches downstream of the main weir provides a series of 50 mm steps which water will flow down to create a passage for fish to climb. The overall drop varies per weir between 180-450 mm, but with the largest individual step of 50 mm.

Refer to drawings 51-33411-C213 for fish passage details.

During drier periods when flows reduce below 5 l/s, the fish passage will be restricted to eels and other good climbing species. A very low flow fish passage during dry periods will not be available unless make up water is introduced at the top of the channel.

7.3 Erosion and scour protection

A Reno Mattress and gabion basket at the downstream end of the weirs is proposed for dissipating energy from water flowing over the weirs. This also provides support and energy dissipation for the proposed incoming stormwater pipes from adjacent developments.

Rock and soil filled Geoweb around the weirs provides additional erosion protection at critical locations.

Refer to drawing 51-33411-C215 for details of the erosion and scour protection at the weir locations.

8. Stormwater Connections

Development within the catchment of the Awakeri Wetlands are expected to discharge primary and secondary flows into the channel. Primary flows (10% AEP) are expected to enter the channel via piped networks. Secondary flows (1% AEP) are expected to enter the channel via overland flow.

8.1 Development connections to channel

The channel has been designed with a shallow depth to reduce potential for groundwater drawdown and ground settlement. The channel therefore requires a wide, shallow flow depth to allow connections for servicing the 10% AEP. Swales or multiple small diameter shallow pipes would be favourable for draining the catchment once developed due to the shallow channel.

Lateral connections to allow properties to drain have been assumed as piped flow, where practical, for events up to the 10% AEP. Overland flow paths will be required to convey flows up to the 1% AEP event.

Drawings 51-33411-C218-C219 shows the proposed outlet detail for connections to the channel.

Piped connections to the channel will typically enter at the permanent water level. Piped connections are required to discharge at the base of the 4H:1V channel banks, typically downstream of the proposed weirs.

Key benefits of discharging downstream of the weir locations are:

- Limit outlet structures and associated energy dissipation to areas where energy dissipation is already required to control flow over the weirs.
- Allows maximum steepness of the hydraulic gradient of the piped flow and as such limiting pipe sizes to their respective minimum size.
- Increased cover over the discharging pipe.
- Visually less prominent within the riparian and wetland planting between the weir structures.

8.2 Pipe connections

PE stormwater pipe outfalls will be installed as part of the Stage 1A works with one upstream manhole for developers to connect into.

Manholes are located on the outside of the slurry wall compared to the channel where relevant. This allows developers to connect into the manholes with their stormwater discharges without excavating through the slurry wall. This reduces the risk of the slurry wall being compromised in the future which could result in lowering of groundwater and the associated long-term settlement issues.

Stormwater outfalls have been sized based on an indicative development layout plan for the catchment. The indicative layout plan was provided by Auckland Council and is titled “Takanini Cascades Development Framework Plan July 2017”. Based on this plan, possible road levels were determined from which possible overland flow and stormwater catchments have been proposed as per the GHD Drawings 51-33411-C601-C613 attached in Appendix C.

The possible catchments were used for calculating pipe sizes for the stormwater discharge pipes. These sizes would need to be checked prior to developers connecting into these

locations, and alterations to the outfalls may be required depending on the final catchment draining to each area.

The proposed pipe sizes and details are outlined in Table 15 below:

Table 15 Pipe outfalls

Pipe	Pipe diameter (mm)	Pipe slope (m/m)	10% AEP flow (m^3/s)	10% HGL slope (m/m)	Velocity (m/s)	Location
SWOF-80A-1	630	0.01	0.565	0.011	2.34	D/S Weir
SWOF-80A-2	710	0.01	0.678	0.008	2.21	D/S Weir
SWOF-180A-1	500	0.01	0.187	0.004	1.23	D/S Weir
SWOF-180A-2	500	0.01	0.372	0.016	2.45	D/S Weir
SWOF-330A-1	500	0.01	0.213	0.005	1.40	D/S Weir
SWOF-330A-2	710	0.01	0.812	0.012	2.65	D/S Weir
SWOF-440A-1	630	0.01	0.518	0.009	2.15	D/S Weir
SWOF-440A-1	560	0.01	0.495	0.015	2.60	D/S Weir
SWOF-100B-1	500	0.01	0.309	0.011	2.03	D/S Weir
SWOF-260B-1	400	0.01	0.138	0.007	1.42	D/S Weir
SWOF-260B-2	500	0.01	0.231	0.006	1.52	D/S Weir
SWOF-300B-1	400	0.01	0.146	0.008	1.50	Culvert wall
SWOF-300B-2	800	0.01	0.909	0.008	2.34	Culvert wall
SWOF-340B-1	560	0.01	0.483	0.015	2.53	D/S Weir
SWOF-340B-2	630	0.01	0.684	0.016	2.83	D/S Weir
SWOF-480B-1	630	0.01	0.672	0.015	2.78	D/S Weir
SWOF-480B-2	560	0.01	0.353	0.014	2.32	D/S Weir
SWOF-570B-1	355	0.01	0.107	0.008	1.41	Low flow channel
SWOF-680B-1	710	0.01	0.680	0.008	2.16	Culvert wall

8.2.1 Pipe outfall support

Pipe outfalls typically enter the channel 200 mm below the low flow water level and will therefore be partially drowned (less than half) during low flow. This assists with energy dissipation and reduces the visual impact of the pipes while still allowing suitable maintenance access.

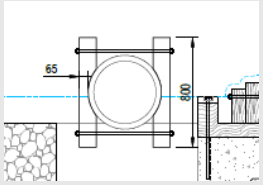
The philosophy for the pipe outfall design includes:

- Minimising the visual impact of the outlets.

- Managing scour and erosion of material around the pipes.
- Managing loss of material via seepage through backfill.
- Connecting into Geoweb material.
- Allowing practical maintenance and access.

A number of options have been considered for the detail around the pipe outlets. Table 16 outlines the options considered.

Table 16 Pipe outfall options

Option	Diagram	Advantages	Disadvantages
Standard concrete headwalls structure		• Effective management of seepage, scour and erosion	• High visual impact • Heavy
No facing around pipe		• Low visual impact	• Risk of scour and erosion around outfall • Risk of seepage and loss of material around pipe
Rip rap surround		• Medium visual impact • Effective management of seepage, scour and erosion	• Un-natural materials for the area
Mitred concrete headwall		• Effective management of seepage, scour and erosion	• Medium visual impact
Hessian bag headwall		• Effective management of seepage, scour and erosion	• Medium visual impact
Timber frame		• Effective management of scour and erosion	• Medium visual impact • Moderate risk of scour and erosion
PE flange fixed to Geo web		• Low visual impact • Reasonable management of scour and erosion when combined with Geoweb	• Moderate risk of scour and erosion

8.2.2 Selected option

Auckland Council have advised that the visual impact of the pipe outfalls is a key requirement and should be reduced as much as possible. The timber frame and rip rap surround options were not considered suitable by the landscape architect.

The PE flange fixed to Geoweb option is the selected option.

The PE flange option provides a reduced visual impact of the pipe outfalls as they can be integrated and hidden by the surrounding landscape features. Geoweb will surround the pipe outfalls and will be fixed to the PE pipe.

The flange of the pipe will overlap the Geoweb to prevent it from coming loose around the pipe. The flange is proposed to be 75 mm wide from the outer wall of the PE pipe to the outer diameter of the flange.

The Geoweb will be filled with a mixture of rock and soil and planted. Rock will be placed around the pipe outfall to a distance of at least 300 mm to minimise scour around the pipe outlet.

8.2.3 Timing

The benefits of installing the outfalls as part of the Awakeri Wetlands project are:

- Pipes are installed through the slurry wall in a controlled manner, and reinstatement of the slurry wall hydraulic barrier can be monitored and achieved to a high quality.
- Auckland Council can control the appearance of the outfalls, including size, material and headwall structures.
- Auckland Council can control the locations of the outfalls to align with the design of the Awakeri Wetlands (i.e. typically downstream of weirs where practical.

8.3 Overland flow

Overland flow will need to be conveyed to the channel via secondary overland flow paths from development within the adjacent land. The design of these flow paths will be undertaken by the developers of the land. Overland flow paths for developments are usually designed along walkways or roads. This will be done by individual developers as and when infrastructure for particular development is implemented.

The channel has been designed with a depth to allow sufficient hydraulic grade from the furthermost extent of the catchment to the channel. Some areas will require fill by the developer due to the existing topography sloping away from the catchment. The possible drainage solution considered uses pipes to convey the primary flow (10% AEP). Developers may use swales and water sensitive design rather than piped networks, however assuming pipes provides a conservative assessment.

The Awakeri Wetlands design includes bank protection at locations where overland flow is expected down the banks. Scruffy dome manholes are provided upstream of the pipe outfalls to allow the capture of some overland flow where overland flow paths are expected to align with pipe outfalls.

Given that development of the catchments and alignment of overland flow within the catchment is highly dependent on developers, the locations allowed for in the Awakeri Wetlands project are indicative, and alternative locations may be installed in the future by developers.

9. Grove Road Culvert Inlet

The Grove Road Box Culvert and the inlet structure for it has been designed by Jacobs, who have provided an invert level of the culvert of 17.5 m. The culvert entry has a tapered mouth to provide more efficient inlet conditions. The culvert mouth has an invert level of 17.6 m. The mouth transitions into an apron which slopes up to RL19.1 m.

The downstream weir of the Awakeri Wetlands has an RL of 20.55 m (lowest point). Therefore a 1.5 m vertical transition is required between the inlet structure/apron and the last weir of the Awakeri Wetlands. This section outlines the design of this transition. Drawings 51-33411-C221-C223 outlines the concept.

Design principle

The key considerations for the design of the transition between the Awakeri Wetlands and the Grove Road Box Culvert inlet structure include:

- Lower velocities to control erosion / scour.
- Flood level to achieve suitable freeboard for Grove Road.
- Fish passage.
- Controlling groundwater drawdown.

The key design features include:

- A series of discrete pools formed using PVC sheet piles and timber facing to assist fish passage between the Grove Road culvert and the TSCC.
- There is a 150 mm step between each pool allowing each pool to cascade into each other via low points created using the weirs. Each 150 mm step is broken up into three 50 mm steps which is formed using timber. This provides a 50 mm maximum jump for fish travelling up the passage.
- The average longitudinal slope of the fish passage is approximately 13.3H:1V.
- The pools are approximately 2.4 m wide allowing some shading from adjacent planting of native grasses and shrubs.
- This defined channel has capacity up to 1 m^3/s before water spills over other sections of the weir and flows across the adjacent ground slope.
- The ground adjacent to the fish passage has an approximate slope of 5% with a 0.5 m drop over a timber faced concrete wall at the base of the slope. The purpose of the wall is to allow a reduction of the overall slope leading down to the culvert headwall to reduce velocities and shear stresses along the slope. This will allow the slope to be planted with native grasses and shrubs, which will hide the concrete and improve the aesthetics at this location.
- Geoweb and Enkamat is proposed across the slope to reduce the risk of scour during storm events and to help stabilise the plants and soil along the slope.
- The last weir of the Awakeri Wetlands is located at the top of the slope and is approximately 35 m long with an RL of 20.55 m at the centre (lowest point). The level of the weir varies across its length with areas of RL 20.80 m and RL 20.90 m to control the flow regime and manage scour risk of the downstream slope.
- This weir sets the permanent water level in the upstream channel, which is maintained to control the groundwater level.

- This last weir outside of the controlled fish passage incorporates a drop of RL 20.80 to RL 20.50. This drop concentrates the energy dissipation where there is a high level of erosion protection prior to flowing down the overall slope to the culvert mouth.

9.1.1 Scour and erosion

High flow events up to the 1% AEP event are not expected to produce the highest velocities, as the flow will be drowned out at the culvert entry; rather, the smaller events will produce the critical velocities for erosion and scour. Velocities are expected to reach up to 3-4 m/s for the critical storm events along the surface leading down to the Grove Road culvert inlet structure. These velocities are expected to be acceptable for planting and will be dissipated using a strategically placed concrete wall and apron at the downstream end of the slope and hardfill immediately downstream of the weir.

A combination of Geoweb and Enkamat is proposed along the slope down to the inlet. The Geoweb will be filled with:

- A mixture of 75% soil and 25% GAP65. This will allow the surface to be planted and will provide some reinforcement to the plant roots as discussed in Section 6.5.3.
- GAP65 strip 1 m wide downstream of the last weir at the top of the slope leading down to the culvert mouth to manage sufficient energy dissipation over this weir.

9.1.2 Groundwater drawdown

The weir at the top of the slope will maintain the permanent water level in the channel. Downstream of this weir, the proposed ground level will drop into the Grove Road Culvert Inlet. To prevent groundwater drawdown due to the deeper cut; a physical groundwater cut-off barrier has been constructed to surround the entire inlet structure, as per Drawing 51-33411-C221.

The barrier is a 7 m deep slurry wall and has been installed. A similar barrier has also been installed upstream near Cosgrave Road to mitigate groundwater drawdown due to the deep cut of the channel.

10. Crossings

The Awakeri Wetlands includes a number of proposed vehicular crossings. The Cosgrave Road culvert and Old Wairoa culvert are part of Stages 2 and 3 and therefore have not been discussed in this report.

Crossings within the Stage 1 Awakeri Wetlands include

- A proposed culvert on the northern channel at Chainage 300.
- A proposed culvert on the northern channel at Chainage 700.

The proposed culverts are standard culvert units and the suppliers shall provide the structural design details based on the requirements discussed within this section.

This report, along with the specification outlines the design requirements for each of the culverts.

Table 17 Culvert details

Culvert location	Internal dimensions	Length	Description
Northern Channel CH300	1.5 m (H) x 2 m (W)	20 m	Provides a channel vehicle crossing for developments on adjacent sides of the channel. Allows for a standard 20 m wide road corridor including 2 lanes, footpaths and berms.
Northern Channel CH700	1.5 m (H) x 2 m (W)	8 m	Provides a one-way entry or exit for the Kauri Flats School. Single lane and footpath.

Key features for the culverts include:

- The culvert has a notionally flat gradient and operates under a hydraulic grade line above culvert invert.
- The design hydraulic grade line is such that the culvert does not result in any noticeable increase in flow depth in the channel upstream of each of the culverts.
- The units will be post-tensioned together to resist differential settlement.
- The culvert is designed to allow for continuation of the low flow channel beneath the road to extend the ecological corridor past this constraint.
- Sufficient height and width to allow for safe maintenance and inspections (1.5 m high).
- Flows partially full with low velocity during all storm events for safety.
- Shallow low flow water depth within the culvert for safety (0.8 m depth).
- Earth / gravel fill on top of the wing wall structures to continue channel profile up to the culvert and allow planting to hide the concrete wing wall structures.
- Allowance for up to 40% blockage without significant increase in headwater depth.
    - Based on the principles of the Stormwater CoP and a best practicable approach, the Stormwater Code of Practice recommends 50% blockage, but given the environment and level of risk, 40% has been considered appropriate.

10.1.1 Northern Channel Chainage 300 Crossing

This crossing is shown on Drawing 51-33411-C230. This crossing is proposed as a twin 1.5 m (H) x 2 m (W) x 20 m (L) concrete box culvert with 45 degree wingwalls.

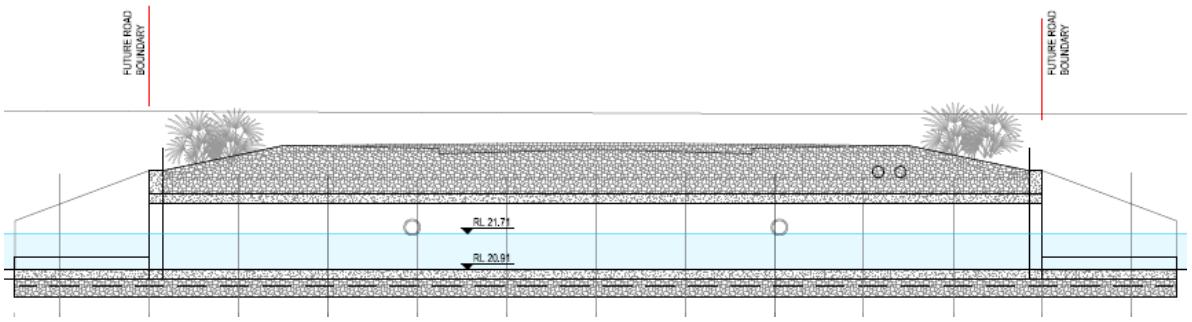


Figure 15 Northern Channel CH 300 culvert crossing

10.1.2 Northern Channel Chainage 700 Crossing

This crossing is shown on Drawing 51-33411-C240. This crossing is proposed as a twin 1.5 m (H) x 2 m (W) x 8 m (L) concrete box culvert with 45 degree wingwalls.

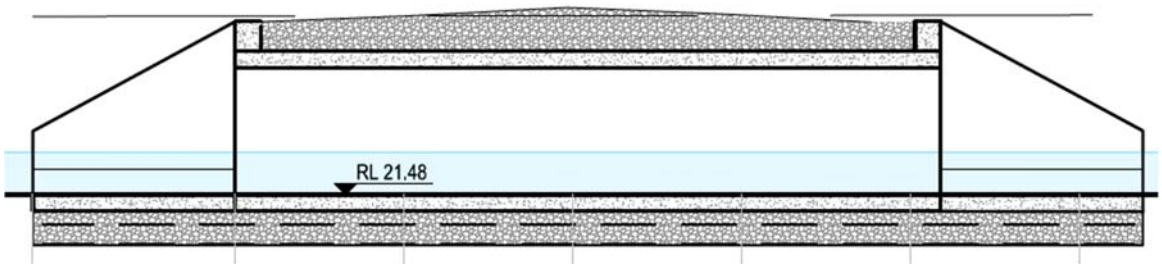


Figure 16 Northern Channel CH 700 culvert

10.2 Hydraulic design

10.2.1 Hydraulic losses

Losses through the culverts were checked using Bernoulli's Energy equation and the losses were determined to be sufficiently low to not significantly impact the hydraulic grade line of the channel. Calculations are provided in Appendix D and summarised Table 18 below.

Table 18 Culvert losses

Culvert	Exit loss (m) 1% AEP	Friction loss (m) 1% AEP	Entry loss (m) 1% AEP	Total loss through culvert (m) 1% AEP	Upstream water level 1% AEP (m RL)
CH300B	0.067	0.045	0.067	0.179	22.64
CH700B	0.019	0.003	0.019	0.041	23.00

10.2.2 Blockage

A blockage assessment for the culverts was undertaken, where it was assumed that the inlet of the culverts were blocked by 0%, 25% and 40% to determine the impact on hydraulic operation of the Awakeri Wetlands system. The results are outlined in Table 19 below

Table 19 Culvert blockage

Culvert	Upstream water level / (in brackets) = increase in headwater depth (m RL)			Upstream water level from culvert calculation (m RL)
	0% blockage	25% blockage	40% blockage	
CH300B	22.64 (0.000)	22.71 (0.070)	23.14 (0.500)	22.64
CH700B	23.00 (0.000)	23.00 (0.000)	23.00 (0.000)	23.00

The blockage assessment indicates that the culvert at chainage 300B can tolerate up to 25% blockage without any noticeable adverse effects on upstream water levels. At blockages greater than 25%, upstream water levels would increase. At 40% blockage the water level upstream of the culvert could increase up to 0.5 m. The water level would still be maintained 440 mm below the road level. This increase could create a backwater effect up the Awakeri Wetlands with water levels approximately 0.1 m below the road level at chainage 700B (the MoE crossing). This is considered acceptable as no significant flooding of floor levels is anticipated at this level of blockage.

The blockage assessment indicates that the culvert at Chainage 700B can tolerate up to 40% blockage of the inlet area without any adverse effect on upstream water levels.

Blockage of up to 40% is considered unlikely for culverts of this size based on the likely type of debris that may be floating down the corridor during large storm events such as wooden pallets, logs, mattresses, containers, shrink-wrap or car bodies.

10.3 Structural design requirements

As discussed above, the supplier shall provide a structural design for the culverts.

The structural design should be in accordance, but not limited to the requirements in drawings 51-33411-S001-S002 and Table 20 below.

Table 20 Culvert design requirements

Feature	Note
Culvert internal dimensions	1.5 m (H) x 2 m (W) twin box culverts
Design working life	100 years
Exposure classification	XA2
Minimum concrete cover to reinforcement	Minimum 65 mm
Minimum concrete strength	50 MPa
Concrete specification	Refer to B610 Concrete Construction
Importance level	2
Site subsoil class for seismic design	D
Surcharge loads on proposed ground level	HN-HO-72 traffic loads
Longitudinal post-tensioning	Longitudinal post-tensioning cables are to be provided within the ducts in the corners of the culverts as per the Contact Drawings.
Cross-bolting of twin culvert units	Each twin culvert unit shall be bolted to the adjacent unit as per the drawings.

10.4 Geotechnical design parameters for culverts

A typical design philosophy for the culverts has been chosen to provide consistency throughout the corridor and ease of construction, maintenance and aesthetics.

Ground support and fill

GAP65 granular hardfill is proposed for underneath the culverts. Two layers of TX160 Geogrid are typically proposed within the GAP 65 layer to create a stiffened raft. This will reduce the potential differential settlement of the culvert units.

GAP65 granular hardfill is also proposed for around the sides of the culverts and above the culverts as backfill.

Geotechnical unit weight parameters

Given the variable nature of the geotechnical testing results, a range of unit weights have been considered for the existing peat soil for each fill scenario; 11 kN/m^3 and 13 kN/m^3, respectively.

The assumed unit weight parameters for other materials are outlined in Table 21 below.

Table 21 Material unit weight parameters

Material	Unit weight (kN/m^3)
GAP65 fill	20
Concrete	25
Water	10

10.5 Settlement assessment

A settlement analysis for the culvert has been undertaken based on a range of soil parameters and site conditions to determine the likely risk of settlement issues during construction and long-term post construction.

Groundwater level

Two possible groundwater level assumptions have been considered to provide a long-term scenario estimate and a construction scenario estimate:

Long-term scenario: assumes the weirs in the Awakeri Wetlands are operating effectively and the groundwater level is maintained at the weir level.

Construction scenario: The water level in the culvert is empty and the groundwater is at the invert level of the culvert.

Settlement predictions

The calculations are provided in Appendix D and summarised in Table 22.

Table 22 Predicted settlement

Scenario	Possible range of settlement (mm)	
	Culvert CH300B	Culvert CH700B
During construction	80-110	100-125
Long-term settlement Design Scenario	40-75	70-90
Long-term settlement Worst case	55-90	75-100

Based on the settlement assessment discussed above, it is recognised that some ground settlement will occur as a result of installing and operating the culvert. This is likely to occur mostly in the short term (during construction), where the dewatering and excavation of the existing peat soils will cause the most settlement. During the construction stage, the settlement should be closely monitored and any difference in the final levels can be made good by increasing the level of backfill to reinstate the affected area back up to the design level.

In the long-term, further minor settlement is also likely to occur. This is not expected to be a significant issue as the effect on hydraulics of the channel (if the invert levels drops by 100-200 mm) is minimal, and there are no services currently proposed above or below the culverts. Future services crossing above or below the culvert could include pipes for wastewater, water, power ducts, fibre and a road. These services will need to be designed to tolerate the predicted settlement of each of the culverts.

10.6 Buoyancy assessment

A buoyancy assessment of the proposed culvert designs has been undertaken to ensure that the design will be sufficient to prevent floatation.

10.6.1 Assessment area

Buoyancy has been estimated by calculating the weight of water displaced by the culvert (uplift force) and the weight of the culvert structure itself (resisting downward force). The factor of safety against buoyancy is determined by dividing the resisting force (mass) by the uplift force. The culvert assessment area includes:

- The culvert structure itself.
- GAP65 between the top of the culvert and the assumed design surface level (cover).
- Any permanent water in the culvert (see groundwater level scenarios below).

The geotechnical unit weight parameters for water, concrete and backfill are as presented in Table 21.

10.6.2 Water level

Two possible water level scenarios have been considered to assess buoyancy in different worst case scenarios:

Flooding scenario: groundwater level is at the ground surface level and the water level in the culvert is at the permanent water level (800 mm above the culvert invert, as set by the downstream weir). Any water depth above ground surface level (during floods) does not create any greater buoyancy risk as the weight of the water cancels out the buoyancy force.

Poor operation scenario: groundwater level is at the top of the culvert and the culvert is empty i.e. the permanent water level in the culvert is at the invert level. This case assumes the downstream weir is ineffective at maintaining a permanent water level within the culvert. This is unlikely to occur during wet weather / flood events, therefore the backfill above the culvert has been considered dry / above the groundwater level for this scenario.

10.6.3 Applied factors of safety

Construction soils can be variable and it is good practice to apply a factor of safety (FoS) to decrease the downward force of backfill. Generally, if the weight of the structure is the primary force resisting flotation, then a FoS of 1.0 is adequate. If friction or cohesion of the backfill are the primary forces resisting floatation, then it would be appropriate apply a safety factor to account for the variability of the soil properties. Therefore, the following factors of safety have been applied for the following scenarios:

Flooding scenario: FoS of 1.0. The backfill above the culvert is saturated so the resisting force is largely provided by the culvert structure rather than the backfill. Friction and cohesion of the soil has been ignored and only the self-weight of the backfill has been considered. Therefore, the backfill does not need an additional factor of safety.

Poor operation scenario: FoS of 0.9. The dry backfill above the culvert exerts a significant downward force compared with the weight of the concrete culvert. Given the variability of the soil, it is appropriate to multiply the backfill downward force by a factor of safety, which essentially gives a more conservative estimate of the total downward force.

10.6.4 Buoyancy predictions

Buoyancy along the length of the culvert has been calculated at 2 m intervals. The units have been considered as individual unconnected units, which is the conservative scenario given that they are expected to be tensioned together. In all cases, the culvert structure (including backfill) is sufficiently weighted to prevent floatation. The buoyancy factor of safety predicted for the different water level scenarios is presented in Table 23 below:

Table 23 Predicted buoyancy factor of safety

Design scenarios	Buoyancy Factor of Safety	
	Culvert CH300B	Culvert CH700B
Flooding – groundwater is at ground level and the water level in the culvert is at the permanent water level.	1.8-2.2	1.4-1.5
Poor operation – groundwater level is at the top of the culvert and the culvert is empty.	1.4-1.9	1.2-1.4

An additional ‘worst case’ scenario has been considered as a sensitivity check which assumes groundwater is at the ground level and the culvert is empty. This provides a minimum FoS greater than 1.0 which is considered acceptable, hence the floatation risk for this culvert is low. This scenario could be possible during construction when the water in the channel is being pumped.

This assessment is conservative as friction and cohesion of the surrounding soils is ignored.

Refer to Appendix D for the buoyancy assessment.

11. Paths

Stage 1 of the Awakeri Wetlands includes approximately 1,300 m of shared path within the works area. Paths within the Stage 1 works area will be constructed as part of this stage and are discussed in the sections below.

11.1 Path alignment and levels

The alignment and levels of the paths were designed and provided by Auckland Council. The alignment and levels were incorporated into the contour design of the Awakeri Wetlands by GHD. Slight adjustments were made where required and these have been confirmed by Auckland Council.

The resulting path alignments are available on drawings 51-33411-C111-C117 and long sections on drawings 51-33411-C151-C161.

Details of the paths are shown on drawing 51-33411-C217.

11.2 Path details

11.2.1 Typical section

The width of the paths is typically 2.5 m of formalised / paved area, within a 4 m wide corridor that slopes at 4% towards the low flow channel to provide natural drainage to the channel.

The paths generally consist of 100 mm thick concrete, with a 100 mm thick layer of Geoweb filled with 20/7 drainage metal underlying the concrete slab. This detail allows groundwater which is expected to seep out of the upstream slope to pass beneath the concrete footpath. This will minimise staining of the paths and minimise build up of slime and debris on the paths.

11.2.2 Drainage

The paths are generally cut into the slope batters of the channel, and therefore will potentially have surface water and/or groundwater flowing towards them from the channel batters. The flow rate is generally expected to be low. A 750 mm wide shallow drainage channel is proposed along the upstream side of the footpath to capture surface water from small rainfall events and any groundwater that seeps out of the upstream slope. This will minimise staining and slippage as discussed above.

The drainage channel will comprise of river stones / pebbles restrained in Geoweb cells, which connect with the drainage metal beneath the footpath. This allows water to be collected in the drainage channel on the upstream side of the footpath where it can soak through the drainage metal and the perforated Geoweb cells as well as soaking into the ground.

11.2.3 Foundation support for the paths

As discussed above, a 100 mm thick layer of Geoweb filled with 20/7 drainage metal will underlie the concrete footpath. While 20/7 drainage metal is not typically relied on for its strength, the confinement provided by the Geoweb cells will provide suitable strength for supporting the footpath. A similar detail is used in the design of permeable pavements where storage of water within pavement base-course is required.

11.2.4 Concrete reinforcement

As discussed above, the footpaths will be 100 mm thick, 20 MPa concrete using sulphate resisting cement (SR type). There is high risk of cracking to the footpaths due to the soft ground

and potential settlement and movement of the ground underneath. To mitigate this risk, the concrete is proposed to have macro-synthetic fibre reinforcement.

Synthetic fibre reinforcement, unlike steel, is resistant to low pH / acidic conditions, which is present in the groundwater and soils of the site. Using fibre reinforcing is also expected to provide reduced construction timeframes compared to standard steel mesh reinforcing. Therefore, providing an overall saving in cost, time and risk.

The proposed application of macro-synthetic reinforcement is proposed as 3.0 kg/m^3 of concrete, but this will depend on the product of fibre reinforcement that the Contractor selects, and therefore is subject to the manufacturers requirements. The 3.0 kg/m^3 is based on Figure 17 below.

	Mesh type (all 150mm centres) standard wire sizes			
	668	**665**	**500E**	**663**
	Typical application Residential driveway or parking area, footpaths, etc.	Typical application Heavy duty residential driveway, residential slab on grade (single storey) or light commercial.	Typical application Residential slab on grade (compliance with code)	Typical application Commercial or industrial warehouse or loading area (external)
Slab Thickness	Forta Ferro dosage (kg per cubic metre of concrete)			
100mm	2.5kg/m³	3.0kg/m³	3.5kg/m³	4.5kg/m³
125mm	2.0kg/m³	2.5kg/m³	3.0kg/m³	3.5kg/m³
150mm	2.0kg/m³	2.0kg/m³	2.5kg/m³	3.0kg/m³

Figure 17 Fibre reinforcement dosage (http://fbsltd.co.nz)

11.2.5 Control joints

Control joints will be required as per Auckland Transports Code of Practice. It is expected that these will be formed through a hit and miss pouring methodology of the footpath, but could be achieved in other ways such as saw cutting. The contractor will confirm the proposed methodology.

11.2.6 Surfacing

Two different paving types are proposed as per the Landscape Plan:

1. Stevensons 'Riviera' exposed aggregate concrete with Peter Fell 468 oxide added to the mix.
2. Stevensons 'Harvest' exposed aggregate concrete with 5% black oxide added to the mix.

These paving types have been provided by Auckland Council and the locations for use are specified in the Takanini Cascades General Arrangement Plans – Hardworks, drawings L8102 – L8117 (Auckland Council, 2017).

An F5E exposed aggregate surfacing is proposed for the finishing of these pavement types.

This is in line with the curatorial framework considerations that were put together by the Auckland Council landscape designer, and Iwi representatives. The curatorial framework requests that the footpaths acknowledge the 'red earth' definition of the name Papakura.

Stevensons 'Riviera' aggregate gives a 'red earth' appearance as shown in Figure 18 below.

Figure 18 Stevensons Riviera exposed aggregate concrete

This style of surfacing has the following benefits:

- Aligns with curatorial framework.
- The footpath will be subject to channel flow and therefore is at risk of staining, the expected staining colour would be orange / brown, and hence would be less noticeable with this finishing.
- Given that the area is wet and is subject to flood flows, there is a risk of the pavement becoming slippery. An exposed aggregate finishing will help to mitigate this by providing grip, however maintenance / cleaning of the footpath will be the primary mitigation for this risk.
- Suitable for walking and cycling.

11.3 Taupo ash layer

Ash layers are present throughout the soil in Takanini. The level of the Taupo ash layer is proposed to be marked where possible, however this has not yet been incorporated into the design. This requirement should be considered by the Contractor to determine what the most effective way of marking this within the works. This should be agreed with Auckland Council and the Engineer.

12. Boardwalks

Stage 1 of the Awakeri Wetlands includes approximately 290 m of boardwalks within the works area. Boardwalks within the Stage 1 works area will be constructed as part of this stage and are discussed in the sections below.

12.1 Boardwalk alignment

The alignment of boardwalks were designed and provided by Auckland Council. The alignments were incorporated into the contour design of the Awakeri Wetlands by GHD. Slight adjustments were made where required which were checked by Auckland Council.

The resulting boardwalk alignments are available on drawings 51-33411-C261-C264.

Details of the boardwalks are shown drawings 51-33411-C265-C266.

12.2 Boardwalk details

12.2.1 Curatorial framework

A curatorial framework has been provided by Auckland Council which collates the aspiration of Mana Whenua, the AC Landscaping Team and other stakeholders.

Key points for the boardwalks are:

- Boardwalk construction to have environmental sensitivity design and prioritise a 'light touch' on the landscape.
- The timber boardwalk decking will acknowledge the 'red earth' meaning of the name Papakura.
- Timber used is to be environmentally sensitive. Local or native timber is to be prioritised.

12.2.2 Typical section

The boardwalks are typically 2.23 m wide between the kerbs, but with a total width of the decking of 2.7 m. The structure consists of timber kerbs, timber decking, timber joists, timber bearers and timber posts which attach to a concrete footing to spread the load of the boardwalk onto the soft peat ground below.

12.2.3 Decking

The decking typically consists of 45 mm x 140 mm timber decking panels with each panel 2.7 m long. Hardwood is proposed for the decking that is resistant to low pH and frequent wetting.

The Hardwood Jarrah is proposed for the decking timber. Native timbers to NZ were considered as per the curatorial framework, however none of the native timbers were suitable to achieve sufficient durability or the 'red earth' look. While not native to New Zealand Jarrah presents the following advantages:

- Very durable.
- Dark red colour.
- Resistant to acid / low pH.

Jarrah is available in 150 x 50 nominal size, which is suitable for the decking timber. This same species is proposed for the kerb and packers underneath the kerbs.

Figure 19 shows Jarrah being used in a marine setting for a waterfront platform.

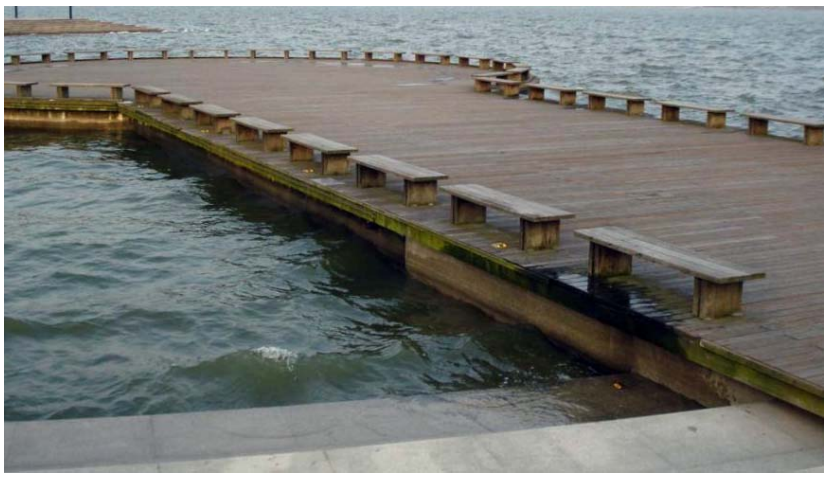


Figure 19 Jarrah waterfront platform (http://www.fqtimber.com)

12.2.4 Support structure

The structure beneath the decking consists of:

- 190 mm x 90 mm joists to support the decking.
- 190 mm x 90 mm bearers to support the joists.
- The bearers are bolted onto 200 mm SED posts.
- The posts are connected to a concrete footing below ground.

The joists, bearers and posts are proposed as treated pine, in accordance with the specification and drawings. The treatment requirements for each component is outlined in Table 24 below.

Table 24 Timber boardwalk components

Component	Location	Material	Grade	Treatment	Species
Decking	Frequently in contact with water	Hardwood	F11	H4	Jarrah
Joists, bearers	In contact with water (low pH)	Sawn timber	SG8(wet) / G8	H6	Pinus Radiata
Posts	In contact with aggressive soils and water	SED posts	NZS 3605	H6	Pinus Radiata

12.2.5 Footing

A 400 mm thick concrete footing is proposed to support the boardwalk and spread loads onto the peat soils below.

The 400 mm thick concrete slab will typically be 3.6 m x 3.0 m for four posts, or 1.8 m x 3.0 m slab for two posts at the ends of the boardwalk.

This footing has been designed to spread the load across a sufficient surface area to accommodate the strength of the peat below.

The concrete footing shall meet the requirements outlined in Table 25 below.

Table 25 Boardwalk footing requirements

Component	Specification
Concrete type	Sulphate Resistant (SR type)
Strength	50 MPa
Reinforcement	668 steel mesh
Exposure classification	XA-2
Cover	Min 65 mm

12.2.6 Foundation

The concrete footing is proposed to be laid a 150 mm thick layer of flowable fill poured directly onto the excavated peat surface.

The Bidim+Geogrid composite material will tie into the foundation to mitigate scour around the footings.

13. Safety in design

Safety has been considered throughout the design process. Each component of the Awakeri Wetlands has been designed with safety as a key consideration.

The following section provides a summary of the safety considerations for the channel design.

13.1 Low flow channel

The low flow channel has been designed to discourage entry by the public through dense wetland planting on the edges of the water body. If someone were to enter the low flow channel, the key features below have been incorporated into the design to reduce safety risks:

- Flow velocity very low.
- Shallow depth maintained by weirs (0.5-1.2 m).
- 2:1 side slopes lined with granular material. As such, the ability for someone to walk up this submerged slope without slipping is enhanced.
- Wetland bench of varying width provides warning of imposing deep water. The wetland bench also acts as a safety bench to assist anyone climbing out of the channel and reduces the chance of people falling into the deeper section.
- Riparian margin and wetland planting creates barrier to entry.

13.2 Weirs

The water level drop between weirs varies from 0.18 m - 0.45 m. This drop is into 800 mm deep water. This is a relatively small drop and a safe falling height, however given that there is water either side of the weirs, there is an associated safety risk. Key safety features and considerations for the weirs include:

- Small drop height between weirs.
- Wetland bench and planting on both sides of the weirs discourages access to weirs by public.
- Timber capping provides a lip at the weir surface that could be held on to if required, likewise with the fish passage structure.
- Low flow channel safety features on both sides of the weir as described in Section 13.1 above.

13.3 Paths

The paths within the channel provide a key amenity feature for the public. As with any public asset, there are some associated risks as outlined below:

- **Falls and trips:** The shared paths will be standard surfacing, that would be familiar to most users, hence minimising fall and trip hazards. This consideration should form part of the operation and maintenance plan to allow frequent maintenance and clearing of the paths, as if plant debris, dirt or slime is allowed to build up on the paths, then the risk of falls and trips would be increased. A gravel drainage strip on the upstream side of the paths is proposed to minimise groundwater seepage or surface flows from frequently flowing across the paths. This will reduce the chance of slime build up and slippery paths.
- **Sight:** Generally, sight distances should not be an issue with the alignment of the paths, as provided by Auckland Council, given that the proposed planting is generally less than

1 m tall grasses and shrubs with some largely amenity trees. Furthermore the alignment of the paths and the nature of the environment is expected to make cyclists ride cautiously and be aware of their surroundings, given the natural environment, proximity to open water, vertical and horizontal curvature of paths, planting and reduced width of the shared path.

- **Proximity to water:** The path alignments occasionally run alongside and over open water. The wetland bench and planting provides a shallow depth of water and a natural barrier to the deeper water which would restrict anyone who veers off the paths from falling into the deeper water.
- **Flooding:** The paths levels are designed above the 50% AEP water level as per ATCOP. Signage is proposed to warn the public of flooding. In these circumstances alternative routes are available which bypass flooded areas. For larger events where the paths are flooded, alternative routes will be available to give access throughout the Awakeri Wetlands alignment. Furthermore, flow velocities in the channel are low, and therefore the safety risks associated with flooding of the paths is considered low.

13.4 Crime Prevention through Environmental Design (CPTED)

The urban and landscape designer has carried out a CPTED analysis as part of a separate report for the TSCC.

13.5 Culverts

13.5.1 Northern Channel Chainage 300 Crossing

This culvert is approximately 18 m long and will have a permanent water depth of 0.8 m within the 2 m high box culvert as a continuation of the low flow channel. The permanent water body and the planting at each end of the culvert will discourage interaction with the culvert by the public.

13.5.2 Northern Channel Chainage 700 Crossing

This culvert is approximately 10 m long and will have a permanent water depth of 0.5 m within the 2 m high box culvert as a continuation of the low flow channel. The permanent water body and the planting at each end of the culvert will discourage interaction with the culvert by the public.

13.5.3 Safety in design features

Key safety in design features and considerations for the culverts are:

- Entry into the culvert is discouraged by planting in the channel at each end and a permanent water level that is continuous between the channel and the culvert.
- Low velocity and low turbulence during low flow conditions.
- Shallow depth of water within the culvert.
- Fencing mitigates falls from the top of the headwalls. Planting behind the headwalls and fencing reduces the risk of anyone accessing the top of the headwall and being in a position where falling is possible.
- No inlet or outlet grills to eliminate risk of people getting stuck against them.

14. Project risks

A number of risks have been identified in the design of the Awakeri Wetlands. These risks sit within the design, construction and operation phases of the project and are outlined in Table 26 below with the proposed management strategies for each risk.

Table 26 has been provided at the end of the design phase and addresses identified design risks and anticipated construction risks. It is expected that these would be incorporated into a risk register and updated as new risks are identified. At the completion of the construction phase it is expected that the risk register will be managed by Auckland Council as asset owner.

Table 26 Project risks

Risk	Description	Management
Design risks		
Flooding	Flooding risks are possible if the channel is planted with excessive planting which could reduce the capacity of the channel.	Low height shrubs and native grasses than can lay flat during storm events are proposed for the channel planting. Landscaping designer has been made aware of these constraints.
Service crossings	Channel could create a barrier to services in the area.	A typical detail has been provided to allow services to cross at the weir locations (upstream of the weirs at the channel invert level). Auckland Council should guide developers to implement this detail where required.
Stormwater connections to channel	Poor choice of stormwater connection locations can have an adverse effect on the channel visually and/or in terms of erosion.	The design recommends stormwater connections to enter channel immediately downstream of the weirs and a typical detail has been provided. Auckland Council should guide developers to implement this detail where required.
Settlement of weirs	Settlement of the weirs could alter the permanent water level in the channel, potentially resulting in groundwater drawdown induced settlement or drying out of the wetland areas.	Considered in design of the weirs. Using sheet piles reduces this risk. The top of the sheet pile can be retrofitted to readjust the top level if future settlement is encountered.
Scour and erosion	Risk of undercutting structures or de-stabilising channel banks and channel invert due to scour and erosion.	Managed through implementing protection measures in critical areas and recommending monitoring and maintenance to promptly address problem areas.
Culvert blockages	There is a risk of culvert blockages and potential upstream flooding as a result.	Managed through design of the culverts as discussed in Section 10. Culverts are typically outlet controlled up until 40% blockage, which is considered an unlikely scenario for this size of the culverts and considering the culverts typically have two barrels.
Culvert settlement	Risk of culvert settlement and damage to services.	Managed through design of culverts and protection of services as discussed in Section 10.

Risk	Description	Management
Culvert floatation	Risk of culvert flotation and damage to adjacent services.	Managed through design of culverts. Suitable factor of safety has been achieved.
Construction risks		
Soft ground	Potential for soft ground.	Contractor's safety plan to include risk of soft ground and management options. Contingency plan to allow remediation if ground is softer than assumed.
Obstructions	Chance of hitting buried tree trunks / logs as observed in the area.	Contractor to allow for contingency plan if obstructions are encountered. Designer has considered this risk and have contingency measures to manage this this outcome.
Flooding	Risk of heavy rainfall event during construction and excavation flooding	Contractor to include a contingency plan in their construction management plan on how to address risk of flooding during excavation. Erosion and Sediment Control plan includes a bund around the excavation to mitigate this.
Access	Construction access issues	Contractor to liaise with Auckland Council regarding access, to understand the access constraints and include in their methodology how these will be considered.
Operation risks		
Safety	A number of safety risks exist within the Awakeri Wetlands corridor – open water, trips and fall hazards.	Mitigated through design as per Section 13 and proposed corridor maintenance as part of the O&M.
Flooding	Flooding risks are possible if the channel planting is not maintained; as overgrown vegetation can reduce the capacity of the channel.	Include regular maintenance of plants within the Operation and Maintenance Plan.
Culvert blockage	Risk of culverts blocking.	The culverts should be inspected and maintained in accordance with Auckland Council's Operation and Maintenance schedule to remove any small blockages or material deposited within the culvert that could accumulate and increase the blockage potential compared to the design assumptions.
Channel scour and erosion	There is a risk of scour and erosion in the channel, undercutting of structures and instability of slopes.	Managed through a risk based approach and monitoring as discussed in Section 6.
Long term settlement	Effect of settlement upon structures, adjacent land, property and buildings.	Risk of settlement post construction due to groundwater dewatering has been considered as part of the Awakeri Wetlands Northern Extension resource consent application. This considers long-term groundwater dewatering as part of the overall scheme. Monitoring and mitigation if required will be carried out in accordance with the GSMCP.

Risk	Description	Management
Water balance	Risk of no flow through fish passage and water level dropping during extended dry periods.	On-going monitoring of water levels, especially during dry periods. Make up water can be designed to recharge the system if required.

15. Conclusion and monitoring

The proposed Stage 1 Awakeri Wetlands will extend from 181 Walters Road in the north to 91 Grove Road in the south (Northern channel), and includes part of the main channel between Cosgrave Road and Grove Road.

In general the conveyance channel will provide stormwater servicing for future development of Areas 2A, 2B and part of Area 4 (2B4) of the Takanini Structure Plan and the Mill Road Block area. At present the area is significantly impacted by the 1% AEP (Annual Exceedance Probability) floodplain, restricting development of the area. The Awakeri Wetlands will reduce the extent of the floodplain within the Awakeri Wetlands catchment to facilitate development of the land.

Development of the Awakeri Wetlands catchment area will increase peak flows from the catchment. The proposed Awakeri Wetlands will direct the increased flows up to the 1% AEP event to the discharge location at the proposed Grove Road Box Culvert.

Stage 1 of the Awakeri Wetlands will consist of:

- 1.2 km of open waterway.
- Depth of 1.9 m to 4.0 m below ground level.
- Notional overall gradient of the channel invert of approximately 0.2%.
- Overall total width (of the 1% AEP level) ranging from 13 m to 39 m.
- 1.3 km of footpath.
- 290 m of boardwalk.

The channel is designed with a meandering low flow series of discrete water bodies or wetlands with a permanent water depth of about 0.2-1.2 m controlled by sheet pile weirs at notional 100 m centres longitudinally along the base of the channel. These provide an ecological benefit and limit the ground water drawdown. Generally the low flow channel will have a of 3-6 m wide base with slope batters 2H:1V, with an intermediate wetland bench and upper 4H:1V riparian planted slopes.

There are two existing future crossings included:

- Twin 3 m x 2 m box culverts on the Northern Channel at Chainage 300.
- Twin 2 m x 1.5 m box culverts on the Northern Channel at Chainage 700.

The proposed Awakeri Wetlands will provide an effective drainage solution for the Awakeri Wetlands catchment.

15.1 Monitoring

The following recommendations are proposed.

Table 27 outlines the monitoring recommendations for the Awakeri Wetlands.

Table 27 Monitoring recommendations

Monitoring	Details	Frequency
Scour and erosion	Monitoring of the channel banks during construction to determine areas of fibrous peat or particularly soft areas within the 10% AEP extent. Scour protection as per the typical details on drawing 51-33411-C216 should be installed in these areas during construction.	On-going during construction.
	Monitoring of the channel banks post-construction, particularly around the edges of the low flow channel should be carried out to determine whether any areas are degrading over time. If scour is observed, then these areas should be remediated with the typical details on drawing 51-33411-C216. Budget should be allowed for retrofitting some areas of the channel.	6 monthly following construction of the channel for 5 years and after storm events, then on-going as part of the standard channel maintenance as per the O&M Manual.
Water level monitoring	Monitoring of the low flow water level in the channel. The water level in the channel should be maintained at the weir level to provide a healthy environment for wetland plants, aquatic life and to control groundwater levels.	On-going as part of the standard channel maintenance as per the O&M Manual.
Water chemistry	The water chemistry of the channel should be monitored as per the Acid Sulphate Soils (ASS) Management Plan.	As per the ASS management plan.
Groundwater and settlement	Groundwater and settlement monitoring should be carried out in accordance with the GSMCP before, during and after construction.	As per the GSMCP.

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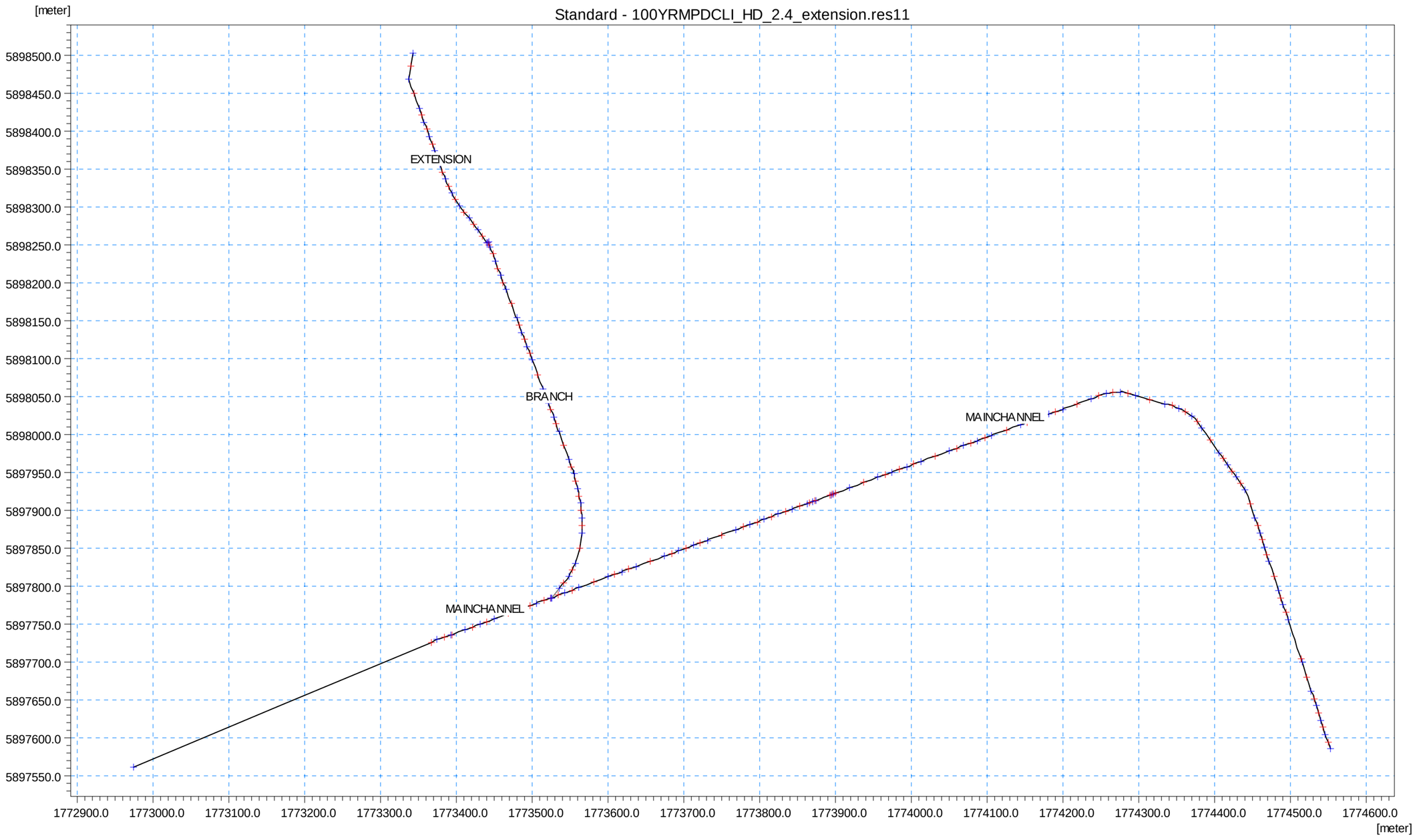
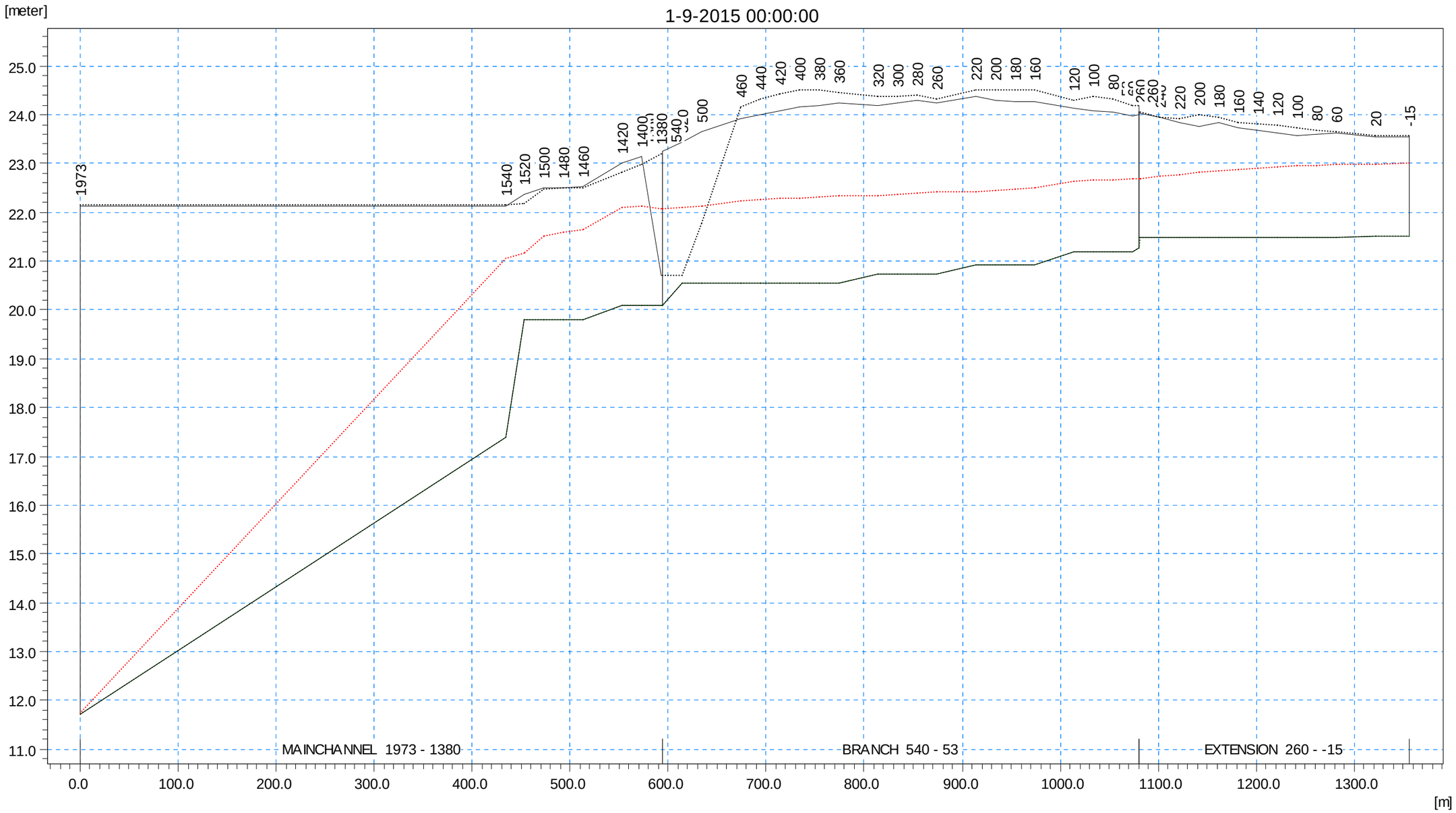
Appendices

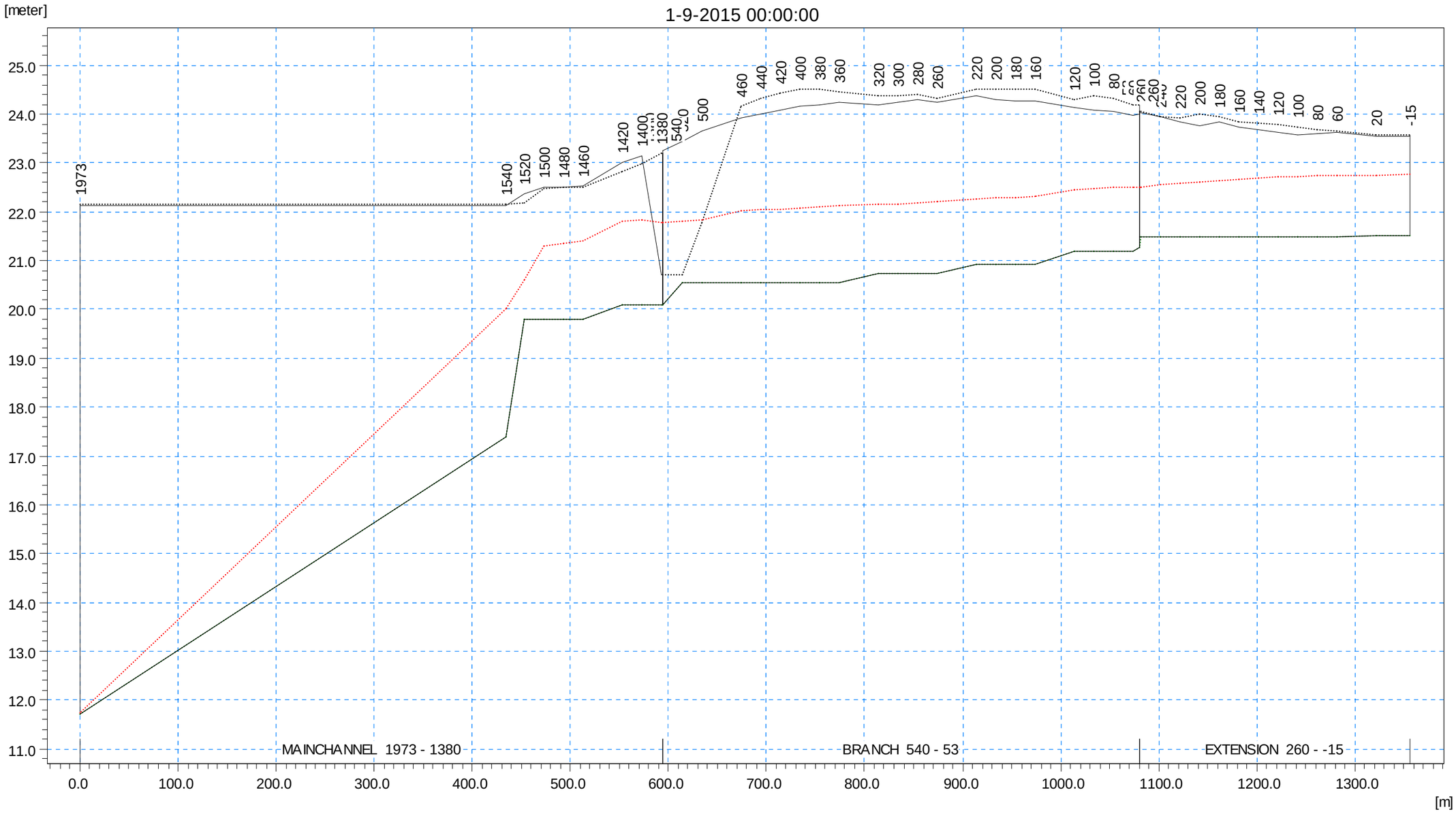
Appendix A - (MIKE11 Model)



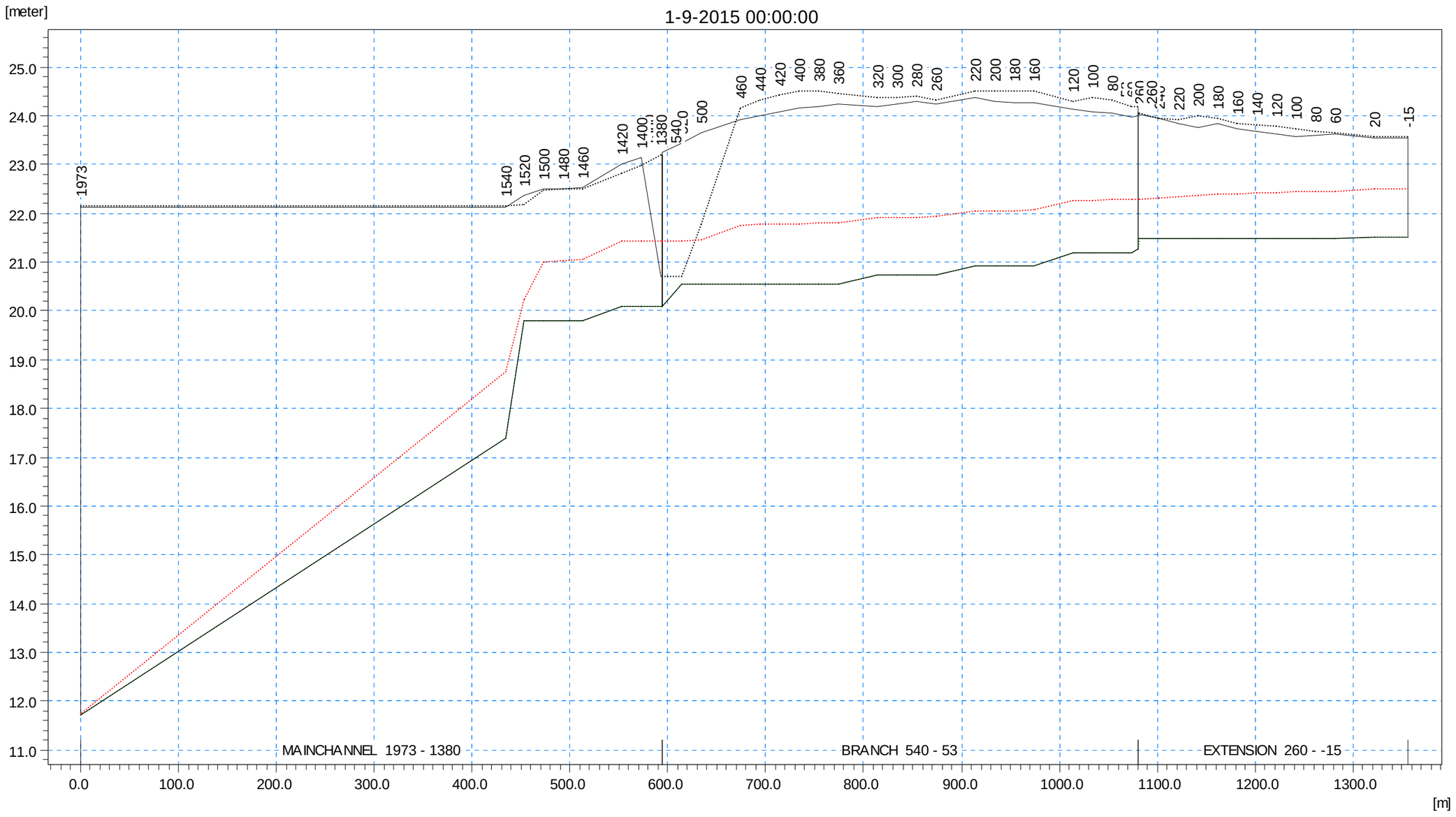
MIKE11 Model alignment



1% AEP – Northern Branch and Extension

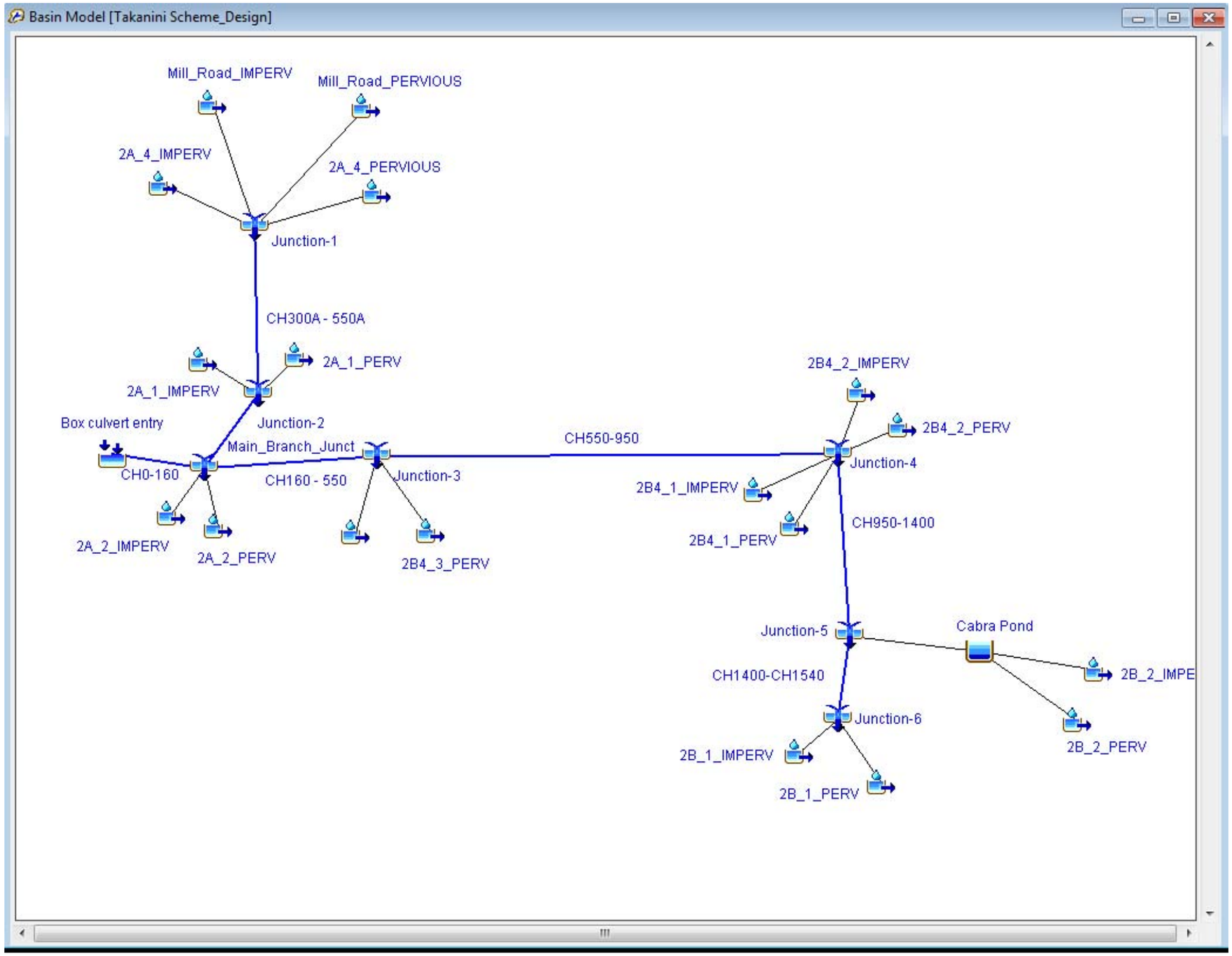


10% AEP – Northern Branch and Extension



50% AEP – Northern Branch and Extension

Appendix B - (HEC-HMS Model)



HEC-HMS – Model Alignment

Global Summary Results for Run "Run 1"

Project: Takanini - Northern Extensi Simulation Run: Run 1

Start of Run: 01Jan2000, 00:00
End of Run: 02Jan2000, 00:00
Compute Time: 27Nov2016, 18:30:32
Basin Model: Takanini Scheme_Design
Meteorologic Model: Met 1_100yr
Control Specifications: Control 1

Show Elements: All Elements Volume Units: MM 1000 M3 Sorting: Hydrologic

Hydrologic Element	Drainage Area (KM2)	Peak Discharge (M3/S)	Time of Peak	Volume (MM)
2B_2_IMPERV	0.1245000	4.8	01Jan2000, 12:05	255.73
2B_2_PERV	0.0939550	2.4	01Jan2000, 12:11	182.36
Cabra Pond	0.2184550	3.5	01Jan2000, 12:25	199.11
2B_1_IMPERV	0.0948250	3.5	01Jan2000, 12:06	255.62
2B_1_PERV	0.0556910	1.4	01Jan2000, 12:13	184.25
Junction-6	0.1505160	4.7	01Jan2000, 12:07	229.21
CH1400-CH1540	0.1505160	4.7	01Jan2000, 12:11	228.97
Junction-5	0.3689710	8.1	01Jan2000, 12:11	211.29
CH950-1400	0.3689710	8.1	01Jan2000, 12:20	210.67
2B4_2_IMPERV	0.1806022	6.3	01Jan2000, 12:08	255.46
2B4_2_PERV	0.1060700	2.4	01Jan2000, 12:16	183.98
2B4_1_IMPERV	0.0506018	2.0	01Jan2000, 12:05	255.76
2B4_1_PERV	0.0337345	0.9	01Jan2000, 12:10	184.50
Junction-4	0.7399795	17.3	01Jan2000, 12:13	219.67
CH550-950	0.7399795	17.3	01Jan2000, 12:19	219.27
2B4_3_IMPERV	0.1249192	4.6	01Jan2000, 12:06	255.63
2B4_3_PERV	0.0733652	1.8	01Jan2000, 12:13	184.27
Junction-3	0.9382639	21.8	01Jan2000, 12:18	221.37
CH160 - 550	0.9382639	21.8	01Jan2000, 12:24	220.97
Mill_Road_IMPERV	0.1143000	3.5	01Jan2000, 12:12	255.03
2A_4_IMPERV	0.0694320	2.5	01Jan2000, 12:07	255.54
Mill_Road_PERVIOUS	0.0490000	1.1	01Jan2000, 12:18	183.84
2A_4_PERVIOUS	0.0297560	0.7	01Jan2000, 12:15	184.09
Junction-1	0.2624880	7.4	01Jan2000, 12:11	233.83
CH300A - 550A	0.2624880	7.4	01Jan2000, 12:15	233.57
2A_1_IMPERV	0.2058660	7.3	01Jan2000, 12:07	255.52
2A_1_PERV	0.0882280	2.1	01Jan2000, 12:15	184.08
Junction-2	0.5565820	16.0	01Jan2000, 12:11	233.85
CH0A-CH300A	0.5565820	16.0	01Jan2000, 12:17	233.45
2A_2_IMPERV	0.0724208	3.0	01Jan2000, 12:03	255.84
2A_2_PERV	0.0389958	1.1	01Jan2000, 12:08	184.67
Main_Branch_Junct	1.6062625	39.1	01Jan2000, 12:18	225.98
CH0-160	1.6062625	39.1	01Jan2000, 12:19	225.92
Box culvert entry	1.6062625	39.1	01Jan2000, 12:19	225.92

HEC-HMS results – 1% AEP event

APPENDIX 12 – Auckland Council 2019 McLennan Wetland Spillway Assessment

Official Height Standard Change

From 1 July 2024, Auckland Council adopts the official height standard for New Zealand called New Zealand Vertical Datum 2016 (NZVD2016).

This model was carried out prior to the height standard change.

All levels included in this modelling report are in Auckland Vertical Datum 1946 (AUK1946/AVD1946).

Levels in this report can be transformed from Auckland Vertical Datum 1946 into New Zealand Vertical Datum 2016 by applying an offset value of 0.282 m.

For example:

$$H_{NZVD2016} = H_{AVD1946} - \text{Offset Value}$$

A single offset value for the catchment has been taken from the Land Information New Zealand (LINZ) Auckland 1946 to NZVD2016 Conversion Raster therefore this offset should be taken as an approximation only for the catchment.

A more accurate height transformation value can be derived by downloading the conversion raster available on the LINZ website below:
https://data.linz.govt.nz/layer/103953-auckland-1946-to-nzvd2016-conversion-raster/

REPORT

Tonkin+Taylor

McLennan wetland spillway options modelling

Prepared for
Auckland Council
Prepared by
Tonkin & Taylor Ltd
Date
June 2021
Job Number
1012030.1040

Exceptional thinking together
www.tonkintaylor.co.nz

Document Control

Title: McLennan wetland spillway options modelling					
Date	Version	Description	Prepared by:	Reviewed by:	Authorised by:
03.06.2021	1	DRAFT report	JMOR	SGB	-
29.06.2021	2	Final report	JMOR	SGB	TSRF

Distribution:

Auckland Council	1 electronic copy
Tonkin & Taylor Ltd (FILE)	1 electronic copy

Table of contents

1 Introduction and background

Tonkin & Taylor Ltd (T+T) was engaged by Auckland Council to identify an optimum spillway level at McLennan wetland, to enable the Artillery Drive Stormwater Tunnel (ADST) to perform as per design. The McLennan wetland spillway in this study refers to the above ground spillway from the upper to the lower wetland. The McLennan wetland sub-catchment is located within the Pahurehure inlet stormwater administrative catchment.

The ADST was built in 2017 to facilitate growth in the catchment upstream of McLennan wetland without increased flood risk to downstream properties. One of the design objectives of the ADST was to prevent the spillway from the upper McLennan wetland storage area being activated in a 1% Annual Exceedance Probability (AEP) rainfall event, including allowance for climate change (CC) and Maximum Probable Development (MPD)[1]. The ADST was designed assuming a wetland spillway level of 15.4 m RL, but the current crest level of the spillway is 15.1 to 15.2 m RL[2].

All levels reported (RL) in this study are in terms of Auckland Vertical Datum 1946.

1.1 Study objectives and scope

The objective of this study is to inform the required upper McLennan wetland spillway height and the resulting flood effects from any raising of the spillway.

The scope of this study was as follows:

- Build a flood model of the McLennan wetland sub-catchment, to an appropriate level of detail to meet the study objective. Representing an MPD scenario in the catchment, incorporating best available data on constructed and planned upstream works (conveyance structures and future land use).
- Assess the MPD baseline scenario at the McLennan wetland including sensitivity analysis on two hydraulic parameters.
- Determine an appropriate upper McLennan wetland spillway height including assessment of flood effects associated with raising the spillway.

1.2 Background

1.2.1 Catchment

A catchment map is presented in Figure Appendix A.1. The catchment upstream of Grove Road is zoned 'Residential – mixed housing suburban zone', 'Future Urban Zone' and 'Residential – Single House Zone'[3]. To facilitate the anticipated growth in these areas numerous stormwater infrastructure projects have been completed or are being designed including the Awakeri wetland conveyance channels, The Grove Road Culvert, and the ADST and associated works at McLennan wetland.

The catchment topography is very flat, particularly upstream of McLennan wetland and therefore raising of the spillway at the wetland has potential to incur backwater flood effects.

1.2.2 McLennan wetland

Figure 1.1 shows the layout of McLennan wetland and the key hydraulic structures. Flows are discharged to the wetland through numerous stormwater pipes, the largest being the Grove Road

[1] Artillery Drive Stormwater Tunnel, Detail design report for client review. Jacobs. 14 November 2014.
[2] McLennan Dam Survey crest levels "topo160517_nztm.shp", Provided by Auckland Council.
[3] Auckland Unitary Plan Operative in part (15 November 2016) Update 9 April 2021 planning maps viewer. https://unitaryplanmaps.aucklandcouncil.govt.nz/upviewer/

culvert which discharges flows from the Awakeri wetland conveyance channels in the upstream catchment.

The upper McLennan wetland is connected to the lower wetland by a 1350 mm diameter pipe. It is understood from discussion with Auckland Council that the existing 950 mm orifice at this pipe will be further throttled to a 200 mm orifice, with the permanent water level in the pond being maintained at 11.30 m RL.

Flood flows are attenuated within the upper wetland and drained by the ADST which has two inlet structures[4]:

- A low-flow 1050 mm diameter scruffy dome at 11.7 m RL. Connected to the ADST with a 450 mm diameter 3.5 m length pipe.
- A bellmouth weir scruffy dome at 12.7 m RL into the 2500 mm diameter tunnel.

At 14.2 m RL flood flows spill into the adjacent sports field which provides further attenuation volume to the upper wetland. The spillway conveys any flows exceeding the total storage volume of the upper wetland and sports field to the lower wetland. The lower wetland is drained by two 900 mm diameter pipes.

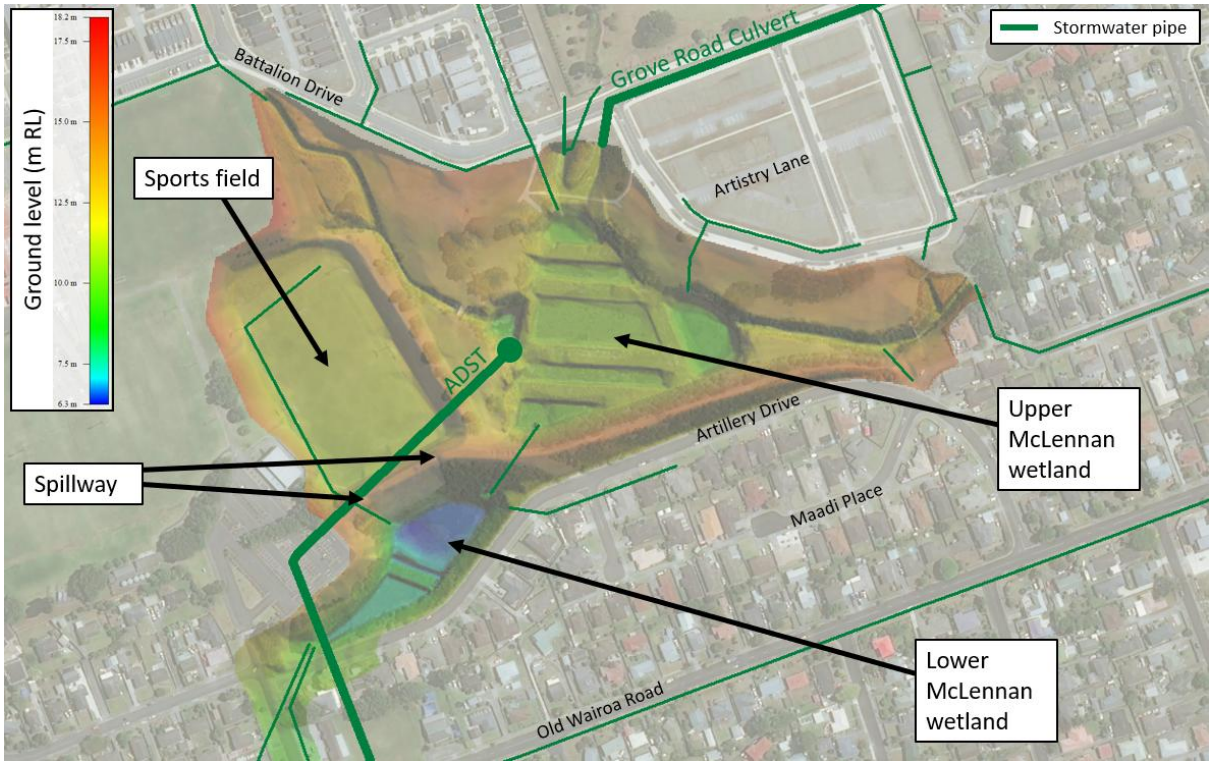


Figure 1.1: McLennan wetland key hydraulic structures

1.2.2.1 McLennan spillway and embankment

Figure 1.2 shows the surveyed crest levels[5] of the spillway and embankment at McLennan wetland. The crest levels can be divided into three distinct sections:

[4] Artillery Drive Stormwater Tunnel: Operations and Maintenance Manual. Auckland Council Healthy Waters Design Office. Final Version 1.0. 19/07/2019.

[5] McLennan Dam Survey crest levels "topo160517_nztm.shp", Provided by Auckland Council.

1 The McLennan wetland spillway: Elevations across the spillway range from 15.07 to 15.39 m RL and then tie into the high ground at 15.7 m RL to the west of the spillway.

2 Embankment along Artillery Drive: Elevations range from 15.98 to 16.31 m RL.

3 Dip in Embankment / overland flowpath into the wetland at the junction of Artillery Drive and Maadi Place: Elevations range from 15.68 to 16.28 m RL.

Figure 1.2: McLennan wetland spillway and embankment crest levels

2 Methodology

2.1 Flood model

A flood model of the catchment was built in Mike Flood (Mike Urban, Mike 21, Mike 11)[6]. Details of the flood model build, and input data are recorded in the model review documentation in Appendix B. An ArcGIS map package is also provided with the flood model deliverables which contains the model schematisation and data record.

The flood model was reviewed by Auckland Council and approved for the purposes of this study after the initial review comments were addressed.

2.1.1 Boundary conditions

The hydrological inflows to the flood model are derived using the TP108 methodology. All simulations in this study include Maximum Probable Development (MPD) within the catchment and climate change (CC) applied to rainfall, as per the Stormwater Code of Practice[7]. The MPD impervious coverages were assigned using the latest Auckland Council modelling recommendations[8] and the Auckland Unitary Plan Operative in part[9].

[6] Model built and simulated in DHI software 2017 release.
[7] Auckland Council, November 2015. Code of Practice for Land Development and subdivision. Chapter 4 – Stormwater.
[8] Land use Zone Imperviousness for Hydraulic Modelling based on the Auckland Unitary Plan Operative in Part (AUP OiP), Auckland Council Memorandum 04/09/2019.
[9] Auckland Unitary Plan Operative in part (15 November 2016) Update 9 April 2021 planning maps viewer. https://unitaryplanmaps.aucklandcouncil.govt.nz/upviewer/

A constant downstream boundary of 2.34 m RL has been applied as requested by Auckland Council. This is the Highest Astronomical Tide (HAT) condition used in the design of the ADST[10].

2.1.2 Flood model assumptions and limitations

All model build assumptions are recorded in the review documentation in Appendix B and the ArcGIS map package. The main assumptions of note are:

- The flood model has been built as per the Auckland Council modelling specification where applicable, but it is not a detailed catchment model appropriate for floodplain mapping. The model has been schematised to represent an appropriate flood hydrograph and hydraulic detail at McLennan wetland to assess local flood effects. Therefore, the model only includes the primary trunklines of the stormwater network in the upper catchment.
- Hydrological soil groups D and C have been used to derive the pervious area curve numbers in the catchment. Auckland Council requested these soil groups were applied with regards to the high soil moisture content and peat.
- Soakage is present within the catchment but has not been included in the flood model following agreement with Auckland Council. It is understood that soakage in the catchment is primarily for peat recharge purposes, and it does not provide mitigation in high magnitude flood events.
- The proposed 200 mm orifice throttle on the 1350 mm diameter pipe connecting the upper and lower wetland has been included in the model as requested by Auckland Council during the peer review process.
- Assumptions associated with the representation of the ADST structures, as described in section 2.1.2.1 below.
- No debris blockage has been included in the upstream catchment stormwater system or the ADST structures. Debris blockage at the ADST has potential to reduce the efficiency of the structure and increase water levels in the upper Wetland.

A limitation of the flood model is that the majority of overland flowpaths are modelled using the 2016 LiDAR (unless specified) and these ground levels are subject to change as development in the catchment occurs. Modification to overland flowpaths in the catchment could impact the timing and shape of the flood hydrograph at McLennan wetland.

2.1.2.1 Artillery Drive Stormwater Tunnel representation

The ADST and inlet structures have been modelled using a discharge-stage (QH) relationship derived using spreadsheet calculations. The QH includes allowances for the tailwater condition and hydraulic losses at the inlets, outlet, pipe bends and roughness. A new QH relationship has been developed due to the following differences observed between the ADST as-built[11] and design drawings:

- The as-built drawings show that a 2500 mm internal diameter tunnel has been installed. The detailed design report, drawings, and calculations showed a 2470 mm internal diameter. The as-built tunnel therefore has an increased capacity compared with design.
- The as-built drawings and photos show that a low flow 1050 mm diameter scruffy dome has been installed at 11.7 m RL instead of the designed low flow slot in the main inlet structure, as shown in Figure 2.1.

[10] Artillery Drive Stormwater Tunnel, Detail design report for client review. Jacobs. 14 November 2014
[11] Artillery Drive Stormwater Tunnel: Operations and Maintenance Manual. Auckland Council Healthy Waters Design Office. Final Version 1.0. 19/07/2019

- The as-built drawings show that four 250 mm wide raised separator blocks are included on the bellmouth weir structure at 12.7 m RL (assumed to be for structural reasons). These separators impact the effective weir length of the structure, as shown in Figure 2.1.

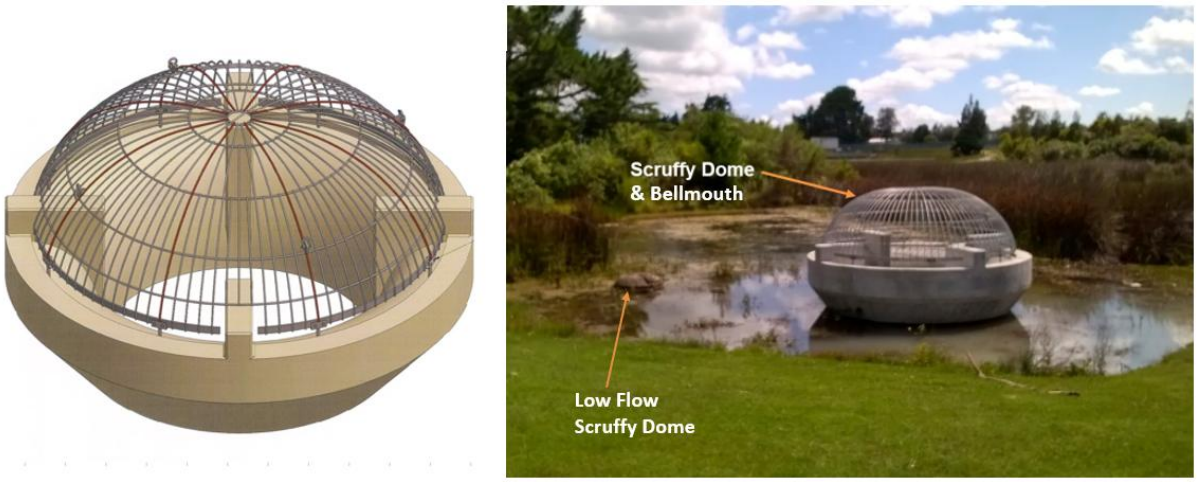


Figure 2.1: ADST inlet structures. Images from Artillery Drive Stormwater Tunnel: Operations and Maintenance Manual.

Figure 2.2 shows the new QH relationship curve derived, and the QH curve from the previous 2016 assessment of the McLennan wetland[12]. The previous curve was based on dimensions in the design drawings.

The updated QH curve includes the low flow scruffy dome at 11.7 m RL and is shown to be more efficient (conveys more flow) at water levels greater than 13.75 m RL where the capacity of the tunnel dominates over the weir control at the inlet. The main reason for this improved efficiency is the increased internal diameter size (2470 to 2500 mm) of the 1.1 km length tunnel. The key stages of the baseline QH curve are described in Table 2.1

Ideally the hydraulics of the ADST would be verified through computational fluid dynamic (CFD) or physical modelling as they are complex, but this was not within the scope of this work. Sensitivity scenarios have been undertaken on the structures, as described in section 2.2.1.

[12] Assessment of McLennan Upper Wetland and Artillery Drive Tunnel Design Performance, 2016-09-21, Auckland Council.

Table 2.1: Key stages of the updated QH curve for the ADST and inlet structures

Water level / H (m RL)	Hydraulics / Q
11.3 – 11.7	No flow entering ADST
11.7 – 12.7	Flow entering ADST through low flow scruffy dome only (Weir control up to 11.94, then pipe control)
12.7 -13.8	Flow enters ADST via Bellmouth weir and low flow scruffy dome (both under Weir control). The as-built drawings of the ADST show that four 250 mm wide raised separator blocks are included on the bellmouth weir structure at 12.7 m RL (as shown in Figure 2.1). These separators reduce the effective weir length of the structure. The separators have been included at all elevations above 12.7 m RL (where weir control is dominant) in the QH. In reality the hydraulics become very complex when the water level exceeds the top of the separators (13.0 m RL) as multiple weir structures of different type, crest level, and orientation will become active. A sensitivity scenario was set up with the separators completely removed from the structure (sensitivity scenario 1), as described in section 2.2.1.
13.8 – 13.91	At 13.8 m RL the low flow scruffy dome and connection is fully drowned and becomes ineffective / negligible. The bellmouth weir remains under weir control.
13.91 – 15.6	The capacity of the ADST becomes the dominant control. The bellmouth weir is transitioning from weir to orifice control.
15.6 >	The bellmouth weir inlet structure is under full orifice flow conditions. The hydraulic losses at the inlet are adjusted accordingly to account for orifice flow throttling and an additional bend loss under orifice conditions.

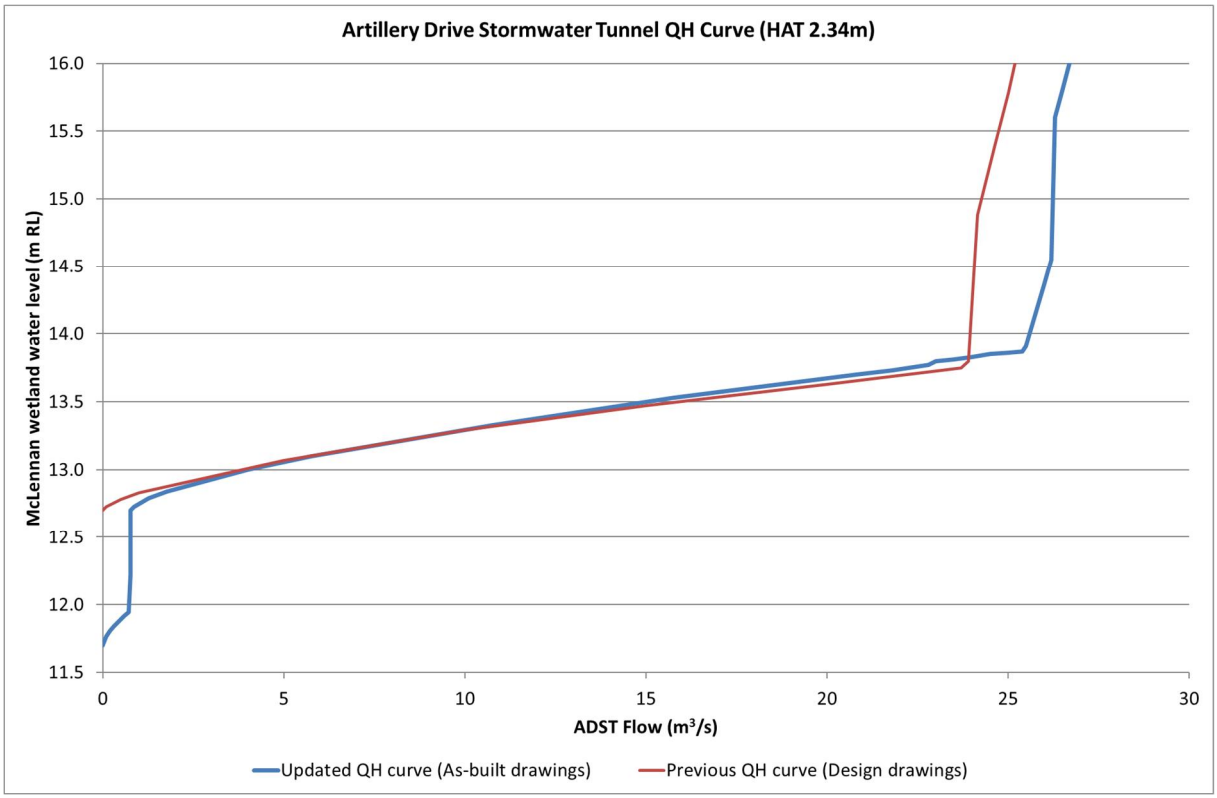


Figure 2.2: Updated and previous QH curves for ADST and inlet structures.

2.2 Scenarios modelled

2.2.1 Baseline and sensitivity

Baseline scenarios were modelled for the 10 and 100 year ARI MPD CC events. Sensitivity scenarios on the ADST and the wetland spillway are described in Table 2.2 below. Sensitivity scenarios 1 and 2 impact the QH curve used to represent the ADST, as shown in Figure 2.3.

Table 2.2: Sensitivity scenarios modelled

Scenario	Description
1	Removal of 250 mm separator structures from Bellmouth weir structure: The as-built drawings show that four 250 mm wide raised separator blocks are included on the bellmouth weir structure at 12.7 m RL. These separators reduce the effective weir length of the structure. The hydraulics are complex when the water level exceeds the top of the separators (13.0 m RL) as multiple weir structures of different type, crest level, and orientation will become active. In the updated baseline QH curve the separators are included at all elevations above 12.7 m RL (where weir control is dominant). A sensitivity analysis has therefore been completed on the QH curve where these separator structures are fully removed (full diameter of bellmouth at 12.7 m RL is used in weir equation) to understand the uncertainty of the complex hydraulics described above.
2	Manning's roughness of Artillery tunnel increased from 0.012 to 0.015. Deterioration of pipe wall roughness values can occur overtime due to slime/grime growth, barnacles, sedimentation, weathering, and debris accumulation.
3	Weir coefficient on McLennan spillway reduced to 1.28 (20% reduction to 1.6 value used in baseline).
4	Weir coefficient on McLennan spillway increased to 1.92 (20% increase to 1.6 value used in baseline).

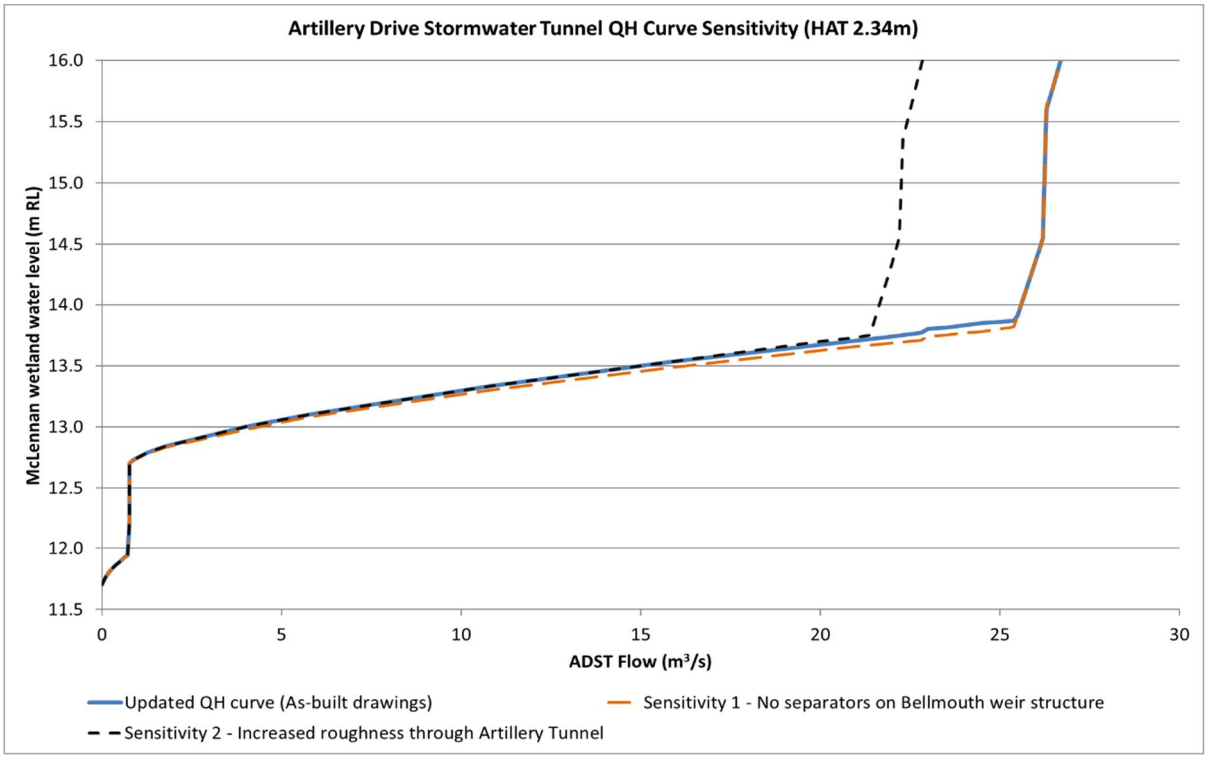


Figure 2.3: QH curves for ADST: sensitivity scenarios 1 and 2.

2.2.2 Options

Raised spillway options modelled are described in Table 2.3.

Table 2.3: Option scenarios modelled

Option scenario	Description
1	All three sections of the spillway and embankment shown in Figure 1.2 raised to a high value of 25 m RL. This is a 'modelled elevation' rather than a proposed spillway height, to establish the peak water level within the wetland when flow over the spillway and embankment is restricted.
1b	Option scenario 1 described above with sensitivity scenario 2 applied (Manning's roughness of Artillery tunnel increased from 0.012 to 0.015).
2	Crest levels raised to 15.68 m RL at the spillway. This is the maximum level the spillway could be raised to without causing an obstruction or backwater effects to the overland flowpath into the wetland at the junction of Artillery Drive and Maadi Place (section 3 in Figure 1.2).
2b	Option scenario 2 described above with sensitivity scenario 2 applied (Manning's roughness of Artillery tunnel increased from 0.012 to 0.015).

3 Results

Results are summarised in Table 3.1. Flood extent figures are shown in Appendix C. Time series of modelled water levels in the upper McLennan wetland are shown in Figure 3.1 and Figure 3.2.

Baseline results show that 0.48 m^3/s overtops the spillway in the 100 year ARI event (no freeboard to the existing spillway level). The ADST inlet structure and spillway coefficient sensitivity (scenarios 1, 3, and 4) resulted in only minor differences in water level in the upper wetland (≤ 0.01 m).

The performance of the ADST is shown to be sensitive to hydraulic roughness (sensitivity scenario 2) and this highlights the importance of regular maintenance of this asset to ensure good hydraulic conditions are retained. Figure Appendix C.3 shows increased flood levels occur in McLennan Park and on the Artillery Drive road, but no increases in flood level > 0.05 m are observed on private properties.

The overland flowpath into the wetland at the junction of Artillery Drive and Maadi Place has a peak flow of 2.1 m^3/s and 1.1 m^3/s in the 100 and 10 year ARI events respectively. In option scenario 1 this overland flowpath is obstructed by the raised embankment. The obstruction to this flowpath results in a lower peak water level (compared to baseline) of 15.06 m RL in the upper wetland as approximately 8,300 m^3 volume of flow cannot discharge into the wetland. The obstruction to the flowpath results in negative flood effects to properties on Artillery Drive and Old Wairoa Road, as shown in Figure Appendix C.4.

Option scenario 2 shows that raising the spillway to 15.68 m RL does not have a negative flood effect as the peak water level increase in the wetland is < 0.01 m. With option scenario 2b (includes increased roughness in the ADST) the peak water level in the wetland of 15.48 is below the 15.68 m RL. Appendix Figures C.5 to C.7 show that option scenario 2 does not increase flood levels outside of McLennan Park, even with increased roughness applied to the ADST.

Table 3.1: Summary of modelled results

Event (ARI MPD CC)	100 year									10 year
Scenario	Baseline	Baseline Sensitivity				Option Scenario				Baseline
		1	2	3	4	1	1b	2	2b	
Peak water level in upper McLennan wetland (m RL)	15.17	15.16	15.36	15.17	15.17	15.06	15.36	15.17	15.48	14.36
Freeboard to current spillway level (15.07 m RL)	-0.10	-0.09	-0.29	-0.10	-0.10	0.01	-0.29	-0.10	-0.41	0.71
Peak flow Artillery Drive Stormwater Tunnel (m^3/s)	26.26	26.26	22.30	26.26	26.26	26.25	22.30	26.26	22.41	26.00
Peak flow over spillway (m^3/s)	0.48	0.32	6.04	0.50	0.50	0.00	0.00	0.00	0.00	0.00
Duration for water level above existing spillway level* (hours:minutes)	0:52	0:47	1:44	0:53	0:52	0:00	1:54	0:53	2:14	0:00
**Duration reported as time water level exceeds 15.07 m RL (lowest crest level of existing spillway). In the option scenarios, where the water level exceeds 15.07 m RL there is no flow over the spillway as it has been raised.*										

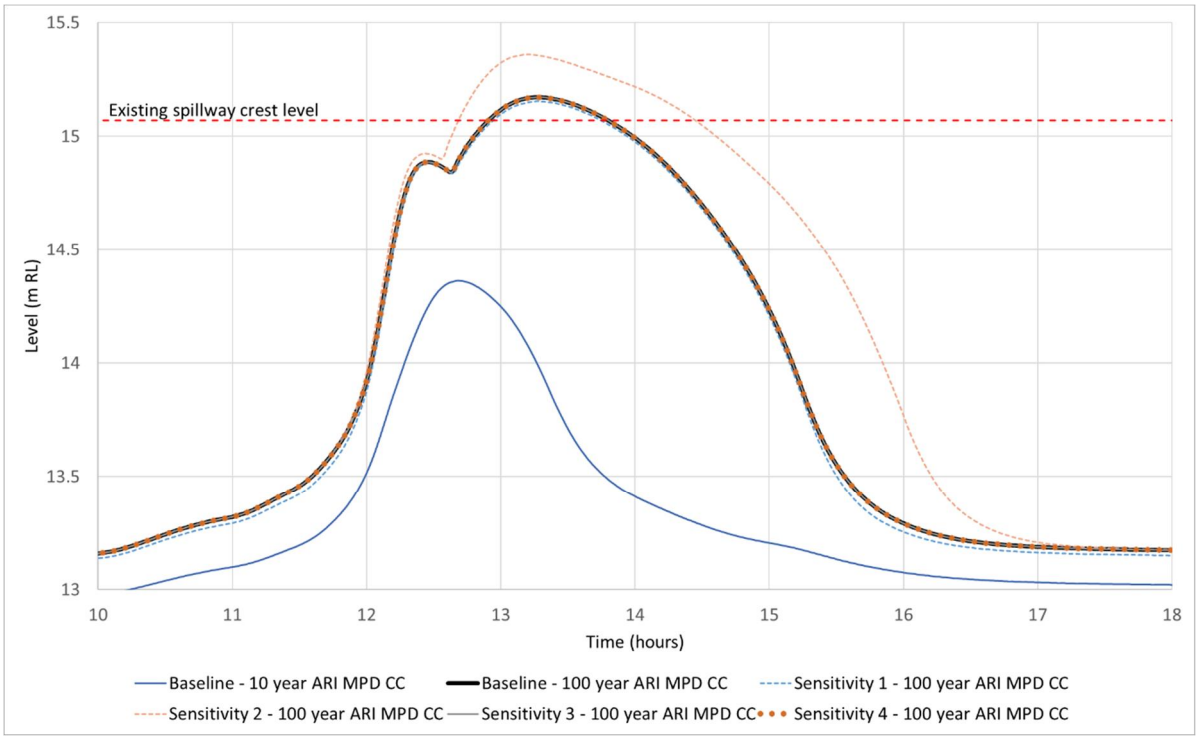


Figure 3.1: Water level in upper McLennan wetland. Baseline and sensitivity scenarios.

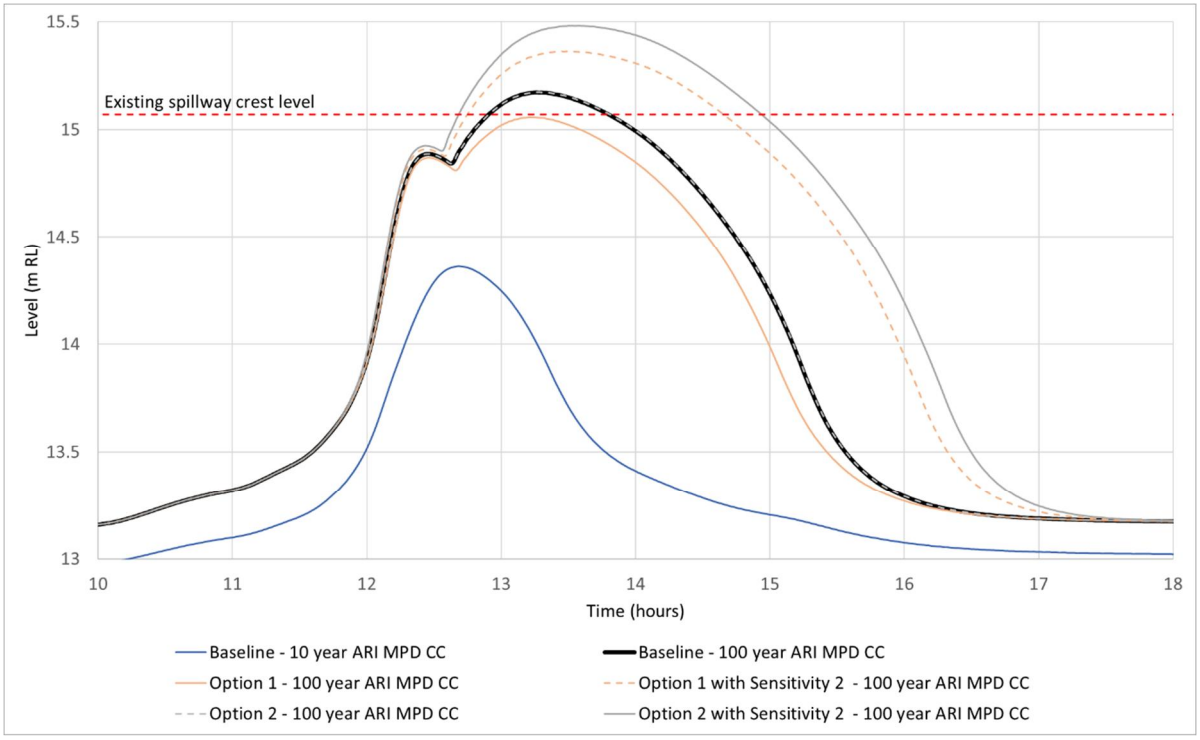


Figure 3.2: Water level in upper McLennan wetland. Option scenarios.

4 Conclusions and recommendations

A flood model has been built using the latest available survey and design data to represent an appropriate flood hydrograph and hydraulic detail at McLennan wetland to assess flood effects associated with potential raising of the spillway.

Results from the study show:

- In the 100 year ARI MPD CC rainfall event the peak water level in the upper wetland is 15.17 m RL which is 100 mm above the lowest crest level of the existing spillway. Potential deterioration of the ADST pipe wall overtime means that hydraulic roughness values could increase this peak level to 15.36 m RL.
- Raising the spillway and embankment above 15.68 m RL has negative flood effects as an overland flowpath into the wetland at Maadi Place becomes obstructed preventing flood flows from entering the wetland and causes flooding in areas not flooded previously.
- Raising the spillway and embankment up to a level of 15.68 m RL does not result in increased flood levels (greater than 0.05 m) outside of McLennan Park, even with an increased roughness applied to the ADST.

Recommendations from this study are:

- The spillway is raised to a minimum level of 15.48 m RL. The spillway could be raised to a lower level of 15.17 m RL provided that the existing pipe wall roughness of the ADST is retained through regular inspection and maintenance. The operations and maintenance manual for the ADST[13] does not currently specify a maintenance plan for this.
- Raising the spillway above 15.68 m RL is not recommended as this will cause backwater effects and/or obstruction to the flowpath into the wetland at Maadi Place. Alternatively, the

[13] Artillery Drive Stormwater Tunnel: Operations and Maintenance Manual. Auckland Council Healthy Waters Design Office. Final Version 1.0. 19/07/2019.

overland flowpath could be diverted away from the wetland and managed with upgrade works to the stormwater network or flowpaths to the east of the wetland. This scenario has not been assessed.

- The required freeboard and any modifications to the wetland structures should be in accordance with the latest New Zealand Society on Large Dams (NZSOLD) and other relevant guidelines.
- This study is a hydraulic / flood assessment only and other potential effects associated with raising the spillway (structural, aesthetic, public access impacts for example) have not been considered.
- Computational fluid dynamic (CFD) or physical modelling of the ADST and associated inlet structures would verify the accuracy of the estimated capacity of the structures. In particular at the stages where complex hydraulics occur at the bellmouth (spilling over the four separator blocks in the structure) and when the inlet structure transitions from weir to orifice control.
- The majority of overland flowpaths in the flood model use 2016 LiDAR (unless otherwise specified) and these ground levels are subject to change as development in the catchment occurs. It is recommended that any proposed modification to overland flowpaths in the catchment are assessed (or implemented into the flood model) to ensure the impact on the timing and shape of the flood hydrograph at McLennan wetland is realised. Alternatively, a future terrain model scenario can be developed to represent development ground levels and any resulting impacts on flood hydrograph timing and shape.

5 Applicability

This report has been prepared for the exclusive use of our client Auckland Council, with respect to the particular brief given to us and it may not be relied upon in other contexts or for any other purpose, or by any person other than our client, without our prior written agreement.

Tonkin & Taylor Ltd

Report prepared by:

.....

James Mogridge

Water engineer and modeller

Report reviewed by:

.....

Sarah Basheer

Project Manager

Authorised for Tonkin & Taylor Ltd by:

.....

Tim Fisher

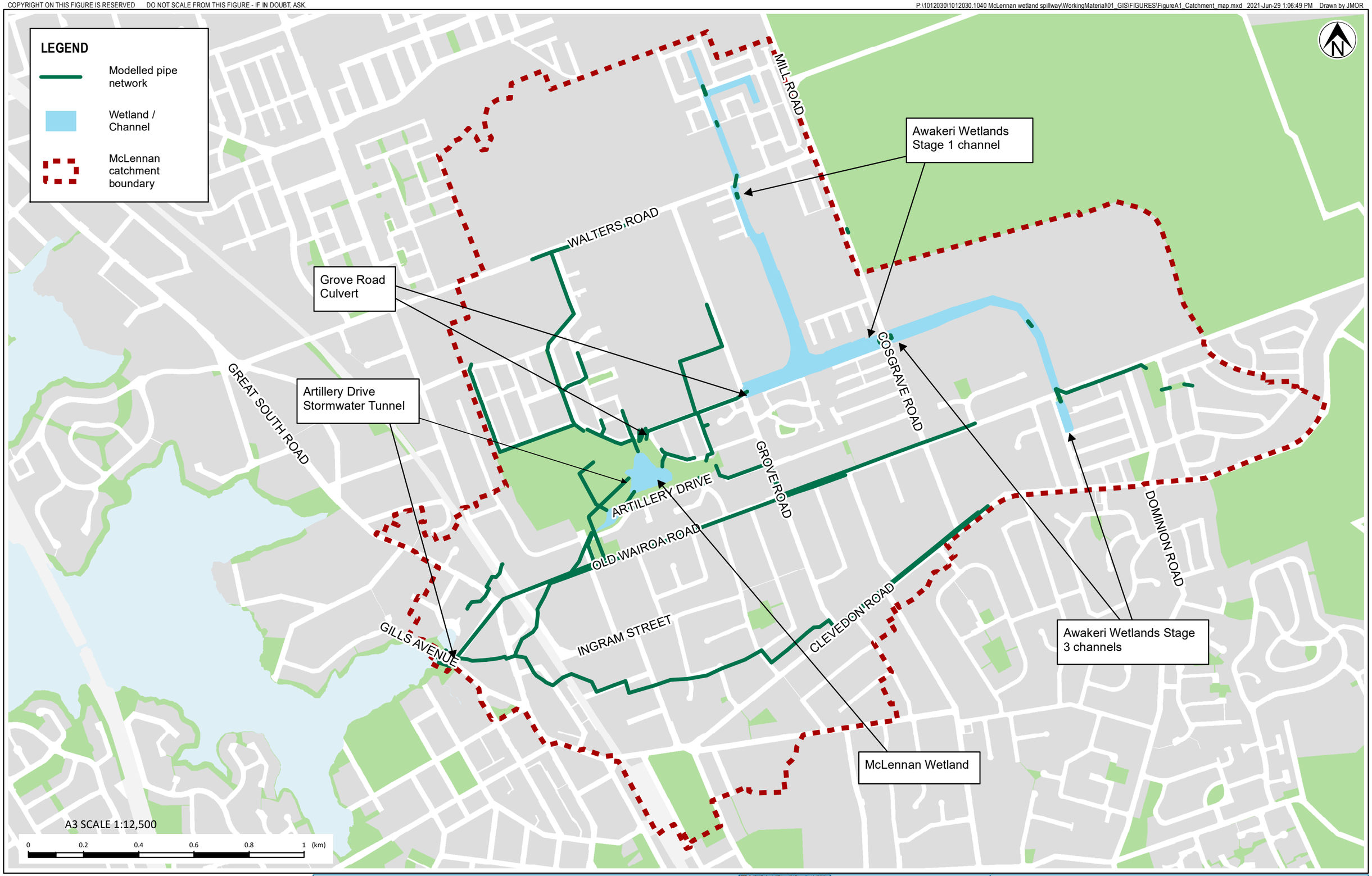
Project Director

JMOR
p:\1012030\1012030.1040 mclennan wetland spillway\issueddocuments\2021-06-30 final report\1012030.1040-rpt-mclennan_wetland_spillway_options-2021-06-30.docx

Appendix A: Catchment background

Figure Appendix A.1: Catchment map MPD

P:\1012030\1012030.1040 McLennan wetland spillway\WorkingMaterial\01_GIS\FIGURES\FigureA1_Catchment_map.mxd 2021-Jun-29 1:06:49 PM Drawn by JMOR



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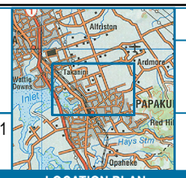
NOTES:

Basemap: Land use map created using Auckland Unitary Plan

This figure has been prepared for Auckland Council with respect to the particular brief given to Tonkin + Taylor for the study. The supporting information, methodology and assumptions adopted in the development of the figures can be found in the McLennan wetland spillway options report (2021). Auckland Council and Tonkin + Taylor do not accept responsibility for any loss or damage resulting from the use of the information and any person relying on the information does so at their own risk.

REV	DESCRIPTION	GIS	CHK	DATE
0	First version	JMOR	SGB	25/05/21



PROJECT No.	1012030.1040		
DESIGNED	JMOR	JUN.21	
DRAWN	JMOR	JUN.21	
CHECKED	SGB	JUN.21	
APPROVED		DATE	

CLIENT **AUCKLAND COUNCIL**

PROJECT **MCLENNAN WETLAND SPILLWAY OPTIONS MODELLING**

TITLE CATCHMENT MAP (MPD)

SCALE (A3) 1:12,500 FIG No. FIGURE A.1 REV 0

Appendix B: Flood Model Review

- Auckland council model review documentation

Auckland Council Model Review

Section 1 - Model Metadata

General Model Info	
Main Consolidated SW Catchment:	**Pahurehure Inlet**
Council Project Manager	Carmel O'Sullivan / Danny Curtis
Other SW Catchment within Model Extent:	-
Other relevant SW Catchment for model inputs:	-
Model Name:	McLennan spillway options - baseline MPD model
Model Horizon ID:	
Model Software, AND Version:	DHI 2017 (Mike Urban, Mike 21, Mike 11)
Type of Model:	Framework Model (FWM)
Model Created By	James Mogridge (Tonkin and Taylor)
Is this model an update based on a previous model?	No
Is the model built as per the SW Modelling Specs?	NO
Model Description:	Model has been schematised to represent an appropriate flood hydrograph at McLennan Wetland with an appropriate level of detail around the wetland to assess flood effects following potential spillway raising options. Refer to the model purpose and objectives below. The model therefore only includes the primary pipe trunklines (generally these are pipes greater than or equal to 900mm in diameter), key connectivity pipes and pipes that may affect hydraulics at the McLennan wetland. The model has been built as per the SW modelling specs where appropriate, noting that some elements of the spec are not applicable to this model (including all pipes >=300mm, maximum sub-catchment size etc.) The flood model topography, assets and hydrology represent a 'future base scenario' MPD, incorporating the design of the upstream works (where as-built or design topo is known/available) related to conveyance channels (Awakeri wetlands and Kauri Flats conveyance channels/wetlands) and future land use as per the Unitary Plan.
Model Purpose / Objectives:	Options models to identify an optimum spillway level to enable the Artillery Tunnel to perform as per design. Model to identify resulting flood effects and hydraulics from a raised spillway level. This project is required to facilitate the continued upstream development of the Takanini area in line with the Healthy Waters preferred stormwater management approach. Currently the McLennan wetland spillway is set too low to allow for the effective operation of the constructed Artillery Drive Tunnel. As a response there is increased flood risk to properties downstream of the wetland during a high return period event.
Limitations specific to this model:	There are areas of development which post-date the 2016 LiDAR topography on the floodplain. Where required and agreed these have been rectified with topography created through interpolation of the manhole lid levels within the new developments.
Is this model fit for producing floodplain for publication?	NO
If answered "NO" for the above question, why not?	refer to model and project purpose. Model is not detailed in upper catchment as this is not required for purpose of model.
Model Files and Documentation	
File directory for model deliverables (MUST COMPLETE): (All model deliverables are to be stored at respective catchment folder(s) under "U:\COO\IES \StormWaterModels\00 Model DELIVERABLES\...")	U:\COO\IES\StormWaterModels\00 Model DELIVERABLES\Manukau Harbour\Pahurehure Inlet\McLennan Wetland Model 2021

Auckland Council Model Review

Section 1 - Model Metadata

Is model report supplied (must have, but can be draft):	NO
Is model extent polygon supplied (must have):	YES
Is model schematisation map supplied (must have):	YES
Is model data flag file supplied:	NO
Are model results supplied:	YES
List out all scenarios modelled (design storm events, validation events, sensitivity analysis runs, etc.)	100yr MPD CC
List relevant input/calculation files supplied:	Hydrology spreadsheet Artillery Tunnel head losses QH calculation spreadsheet
Is WaterRIDE file supplied (only at FINAL delivery):	NO
Model Metadata	
Any DEM modifications? If yes, describe in more detail.	The following DEM modifications (to the 2016 LiDAR DEM) are included. The extent of these modifications are shown in the model schematisation map package provided. **Awakeri Wetlands stage 1:** As-built survey data 2D surface (Surveyworx 2020) **Awakeri Wetlands stage 2 and 3:** 2D surface from Awakeri HEC-RAS model (Awakeri_HECRAS_Rev3) **Grove Road outlet:** 2D surface around Grove Road Culvert outlet (McLennan wetland) derived from drawing provided by AC (*117177-9-1-C GROVE ROAD OUTLET AREA ASBUILT PLAN.dwg*). **McLennan wetland -** 2017 survey contours (SW POND SURVEY AC-HWD-PIN_4417) **Kauri Flats channels:** 2D surface of channels/wetlands created from topography in drawings provided by AC (*117107 - 820-STG5_Stormwater_Rev G.dwg and 117107-101-1-J Asbuilt Plan - Stormwater.dwg*). **Artistry lane and Swamp Kauri developments:** There were significant differences observed at these developments between manhole lid levels and the 2016 LiDAR (the 2016 LiDAR appears to have been captured during earthworks of the development). A 2D surface has been created through interpolation of the manhole lid levels as these are more representative of the developed ground level. There are likely to be uncertainties in the overland flowpaths through these areas and it is recommended that the model is updated with surveyed ground levels/new LiDAR when available. **2d mesh modifications shapefile:** shapefile shows location and elevation of localised modifications to the mesh. These are primarily minor ground level changes at culvert inlets/outlets (to match mesh with invert levels) and to remove blockages caused by footbridges in the Awakeri Wetlands 2D surface.
Mesh Type	Flexible Triangular Mesh
Mesh Size	Maximum element areas have been defined as follows: 1m2 around smaller stormwater channels/roadside drains, 2m2 in and around the Awakeri wetland channels and McLennan wetlands. 4m2 top 6m2 on floodplain. 10m2 -20m2 in areas outside catchment/areas of interest
Soakage representation	Soakage is present in catchment but thought not to perform well in winter months especially during high magnitude flood events. It agreed during model schematisation workshop with AC (15/01/2021) to not be included as effects on flooding likely to be minimal. soakage in the catchment is primarily for peat recharge.
Pipe network modelled (e.g. all pipes >=300mm, etc.)	primary pipe trunklines (generally these are pipes greater than or equal to 900mm in diameter), key connectivity pipes and pipes that may affect hydraulics at the McLennan wetland.

Auckland Council Model Review

Section 1 - Model Metadata

Key structures modelled? Describe type and number	Artillery Drive Tunnel - The previous Q-H relationship (HAT 2.34) used in the "Assessment of McLennan Upper Wetland and Artillery Drive Tunnel Design Performance, AC 2016) has been updated. This Q-H relationship was derived using spreadsheet calculations and included all hydraulic losses such as the bell-mouth inlet, the outlet and various horizontal /vertical bends. The calculations were based on Jacobs design of the structure. A new QH has been developed due to the following considerations…. • The As-built drawings in the Artillery Drive Stormwater Tunnel Operations and Maintenance manual show that a 2500 mm internal diameter tunnel has been installed. The detailed design report, drawings, and calculations showed a 2470 mm internal diameter. The as built tunnel therefore has an increased capacity compared with design. • The As-built drawings in the Artillery Drive Stormwater Tunnel Operations and Maintenance manual (and recent photos of the McLennan wetland) show that a low flow 1050 mm scruffy dome has been installed at 11.7 mRL instead of a low flow slot in the main structure from intended design. • The As-built drawings in the Artillery Drive Stormwater Tunnel Operations and Maintenance manual (and recent photos of the McLennan wetland) show that 4x250mm wide raised separator blocks are included on the bellmouth weir structure (assumed to be for structural reasons). These separators reduce the effective weir length of the structure at 12.7 mRL. The hydraulics are complex when the waterlevel exceeds the top of the separators (13.0 mRL) as multiple weirs structures of different type, crest level, and orientation will be active. In the baseline QH curve the separators are included at all elevations (where weir control is dominant). A sensitivity analysis has therefore been completed on the QH curve where these separator structures are fully removed (full diameter of bellmouth at 12.7 mRL is used in weir equation) to understand the uncertainty of the complex hydraulics described above.
	As a QH relationship has been used, Losses have therefore not been included at the tunnel shafts/bends within the model, to avoid double counting the losses (incorporated into the Q-H relationship). Grove Road Culvert - modelled in Mike11. Energy losses have been modelled as follows: Inlet - 0.26 (rectangular culvert, flared wingwalls /top edge bevelled /single barrel) Total bend loss of 0.3 (12 degree bend 0.05 and 60 degree bend 0.25) M21 Dike structures (weirs/spillways) - M21 Dike structures have been used to define the crest level of the McLennan wetland spillway (2017 survey data). The north and south spillway crests have a coefficient of 1.6 (grass embankments) and the broad crested rock armoured spillway has a coefficient of 1.2. The weir structure just upstream of the Grove Road Culvert inlet fish ladder has also been incorporated as a M21 dike structure with a coefficient of 1.6. The fish ladder weirs and the low flow weirs through the Awakeri wetlands are not modelled (other than being within the 2d mesh DEM) as they are deemed to have a negligible impact in high magnitude flood events. sensitivity analysis is proposed for the Q-H relationship at artillery drive tunnel (as shown in calculation spreadsheet) and the spillway coefficient use
Open channel / stream representation description	Open channels are represented in 2D.
MPD representation (Unitary Plan, District Plan, etc.)	Unitary Plan. MPD impervious as per modelling recommendations in AC memo *(AUP Imperviousness for Hydraulic Modelling 2019-09-04)*
Climate change allowances	2.1 degree Celsius
Tide Boundary Level (current and future)	Artillery Drive Tunnel Design highest astronomical tide level of 2.34mRL. Boundary level requested by AC.
Simulation Duration (24hrs, etc.)	24 hours
Simulation Timesteps	M21FM timestep of 0.25 (0.05-0.25 solution technique)
Model Run Time (How long did it take to run)	The model takes approximately 8 hours to simulate 24 hours on a standard GPU machine

Auckland Council Model Review

Section 2 Review Summary

Review Summary	
Reviewed By (Person/Organisation):	Jahangir Islam, Auckland Council
Type of Review (Standard Review or Partial Review)	Partial review on specifics (describe scope below)
Review Scope Description:	Review of model built for development purposes only, i.e. not a catchment wide model
Summarise Key Findings of the Review:	1. Initial conditions at McLennan upper and lower ponds are not appropriate. 2. Some invert levels and pipe diameter are different from the pond survey data. 3. Artillery Drive Tunnel inlets are not modelled according to as-built plans. 4. Box culverts under Battalion Drive need to be included in the model. **2nd Review: All modelling issues are fixed.**

Document Control

Model Revision	Delivery Date	Review Version	Review Date	Review Completed By, Company
1st version	2021	1st review	1/04/2021	Jahangir Islam, AC
2nd version	23/04/2021	2nd review	30/04/2021	Jahangir Islam, AC

Overview of Review Findings

Traffic Light Rating Scores (0 - no issue, 3 - major issue)

0 - No issue found

1 - Minor issue or non-standard approach, but unlikely to significantly impact on objectives of the study

2 - Some concerns, likely to have an impact on model results

3 - Concerns that may have a significant impact on model results and not meeting the study objectives

Review Section	Traffic Light	Comments
A - Overview		
A:1 Deliverables	0	
A:2 Previous Review Comments	0	
A:3 Model Speed and Stability	0	
B - Detailed Model Review		
B:1 Model Boundary Conditions	2	Initial conditions issue at McLennan ponds
B:2 Model Catchments	0	
B:3 Pipe Networks	3	Pipe diameter and invert levels issue
B:4 Channel / Stream Networks	0	
B:5 Hydraulic Structures and Control Elements	3	Artillery Drive Tunnel inlets modelling issue
B:6 Other Asset Features	0	
B:7 1D Overland Flow Paths	0	
B:8 2D Model Components	3	Box culverts under Battalion Drive missing
C - Model Results Review		
C:1 Model Results Check	0	
C:2 Model Validation	0	
D - Additional Checks		
D:1 Additional Check Items	0	

Auckland Council Model Review

Section 3 Review Details

Instruction Notes:

1. About FIGURES *-- Please note figures should be clearly* ***labelled*** *and included the FIGURES tab and* ***referenced*** *in the review comments.*

2. Traffic Light Rating Scores *(0 - no issue, 3 - major issue)*

0 - No issue found

1 - Minor issue or non-standard approach, but unlikely to significantly impact on objectives of the study

2 - Some concerns, likely to have an impact on model results

3 - Concerns that may have a significant impact on model results and not meeting the study objectives

A - General Information Review

A:1 - Deliverables

Item	Description	Rating Score	Modeller's Initial Notes	Reviewer's Comments	Modellers Response	Reviewer's Comments (2nd)
A:1.1	Is tab "Section 1 - Model Metadata" filled in and does it provide an accurate summary of the supplied model data.	0	Tab 1 completed. Arcmap MPK of model schematisation provided. The GIS layers in this MPK include comments within attribute tables detailing asset data sources and any assumptions.	Yes	-	
A:1.2	Have all agreed deliverables been provided – Reporting, Model Database, Survey etc.	0	Options and assessment and reporting programmed for completion after review of baseline model.	Yes	-	
A:1.3	Is the model delivered in the required software version?	0	Model has been built and run with DHI 2017	Yes	-	
A:1.4	Are all associated model input files supplied in specified format, i.e. as part of the icmt file or in folders with appropriate naming conversion if using other software.	0	model files provided	Yes	-	
A:1.5	Are all required modelled scenarios included in the deliverable? Does the model database include result files for all the scenarios?	0	100 year MPD baseline scenarios provided for initial review. Options assessment programmed for completion after review of baseline model.	Yes	-	

A:2 - Previous Review Comments

Item	Description	Rating Score	Modeller's Initial Notes	Reviewer's Comments	Modellers Response	Reviewer's Comments (2nd)
A:2.1	Confirm that all previous review comments have been incorporated or resolved, if any (such as MEDAR recommendations, etc.). List any that have not, and comment on impact to model usability.	0	No previous review. Notes/agreed actions from model schematisation workshop on 15/01/2021 attached with model. This includes instruction from AC on soil type to adopt for the catchment pervious areas.	N/A	-	
A:2.2	Assess model against any other review recommendations produced during the model development. If there was no formal process for resolving the reviewers comments, then each item should be listed below and a comment made as to whether or not the issue has been resolved, and if it has significant impacts.	0	see comment above	N/A	-	
A:2.3	Identify and document any agreed divergence from spec and adopted model build process	0	no divergence from model approach outlined in project scope	None	-	

A:3 - Model Speed and Stability

Item	Description	Rating Score	Modeller's Initial Notes	Reviewer's Comments	Modellers Response	Reviewer's Comments (2nd)
A:3.1	Check model simulation period and time steps, including result time steps.	0	24 hour simulation. 0.25 second timestep. Mike urban results - 1 min output interval Mike 21 2D results - 5 minute output interval	OK	-	
A:3.2	Comment on run time expected in terms of the catchment size and complexity.	0	The model takes approximately 8 hours to simulate 24 hours on a standard GPU machine	OK	-	
A:3.3	Check model validation errors and warning messages.	0	MU warnings include manhole sizes (smaller than connected links), short pipe lengths (minimum 10m pipe length applied) and negative pipe grades (see B.3.11)	OK	-	
A:3.4	Assess model stability i.e. identify time step critical locations. Any apparent issues in model results caused by model instabilities? Is peak impacted by instabilities?	0	some instabilities at pipe 3000059640 (MU) and AWACUL1&2 (M11) but these do not affect hydrograph peak. The Walters Road Culvert (WALCUL) and culvert immediately downstream (AWACUL3&4) have not been coupled as numerous stability issues were encountered at these locations during the model build despite a range of tests (coupling parameters, MU and M11 representation, invert levels). The culvert openings are currently modelled in 2D only (topographical opening in the mesh). The upstream water level does not reach the soffit level of the culverts and these culverts are sufficiently far upstream of McLennan wetland to have a minimal impact on results.	OK	-	

Auckland Council Model Review

Section 3 Review Details

Item	Description	Rating Score	Modeller's Initial Notes	Reviewer's Comments	Modellers Response	Reviewer's Comments (2nd)
A:3.5	Review mass balance (<1%, if more than 1%, find out why & whether improvements should be made, discuss with AC if mass balance error cannot be reduced)	0	Mass balance calculated at 0.4%	OK	-	

Review Hold Point – if there is any corrective action required as a result of the above – the review is to be halted until the issue is resolved to the satisfaction of the appointed reviewer and Auckland Council

B - Detailed Model Review

B:1 - Model Boundary Conditions

Item	Description	Rating Score	Modeller's Initial Notes	Reviewer's Comments	Modellers Response	Reviewer's Comments (2nd)
B:1.1	Confirm rainfall values and profiles used are appropriate, and that modelled values are equivalent to what is included in the associated reporting.	0	TP108/SW code of practice rainfall profiles and climate change applied. 24 hour rainfall depths extracted at McLennan wetland upstream catchment centroid (1773870, 5897860) - 10 year ARI 140mm, 100 year ARI 222mm.	OK	-	
B:1.2	Assess downstream water levels with reference to coastal marine boundary or other software	0	Artillery Drive Tunnel Design highest astronomical tide level of 2.34mRL. Boundary level requested by AC. Note: NIWA MHWS10%ile +1m SLR is 3.13m RL but the Artillery tunnel was designed with a HAT tidal condition	OK	-	
B:1.3	Describe and review any inflow boundary conditions	0	Hydrological inflows modelled in Mike Urban and loaded to pipe network or M21 following AC modelling spec approach. In Time of concentration calcs, slopes less than 0.005 (0.5%) have been changed to 0.005. to prevent long lag times	OK	-	
B:1.4	Check how model initial conditions are applied for both 1D and 2D. The use of model features such as initial condition zone for tidal areas and ponds, etc.	2	Initial conditions applied in 2D model at following locations: elevations below 2.34mRL (downstream boundary water level) - IWL set at 2.34mRL McLennan wetland upper wetland - IWL of 11.5mRL - reported permanent water level in artillery tunnel detailed design report McLennan wetland lower wetland - IWL of 8.4mRL - surveyed water level in McLennan wetland 2006 as built drawings	The initial conditions used in the model at McLennan wetland upper and lower ponds are not appropriate, should be based on the normal water level shown in the 2017 McLennan Reserve pond survey plans.	The initial conditions at the McLennan Wetlands have updated using 2017 survey plans.	Model updated.
B:1.5	Check time varying inputs and make sure their start and finish time aligns with simulation setting.	0	checked	OK	-	
B:1.6	How is climate change applied? Check rainfall and tide boundary	0	TP108/SW code of practice rainfall profiles and climate change applied. no climate change applied to downstream boundary	OK	-	

B:2 - Model Catchments

Item	Description	Rating Score	Modeller's Initial Notes	Reviewer's Comments	Modellers Response	Reviewer's Comments (2nd)
B:2.1	Review modelled catchment extent. Confirm that it follows contours, and incorporates or excludes any additional primary network which is not consistent with the contours. Any flow transfers across catchment boundaries?	0	Glasswalling occurs along the northern boundary of the model. This is as per schematisation agreement (assume Bruce pullman park subcatchments flow towards the wetland)/administrative catchment of McLennan wetland whereby future developments may contour flowpaths to flow towards the wetland (despite the 2016 LiDAR suggesting that overland flows currently go north - away from the wetland)	OK	-	
B:2.2	Subcatchment extents and sizes. Comment on methodology used for subcatchments delineation – is it appropriate, are there any limitations? Comment on subcatchment size. Any impact on model usefulness.	0	Subcatchments have been delineated using: 2016 LiDAR Existing stormwater network Anticipated future stormwater upgrades (discharge to Awakeri Wetlands), Unitary Plan and previous reporting/scheme catchment for Artillery Tunnel. 3 sub-catchments were added following the model schematisation workshop (rural1, rural2 and rural3) due to uncertainty in direction of the 2016 lidar overland flowpaths. These catchments have been loaded to the 2D model grid to ensure any flows that do enter the McLennan catchment from these areas are captured.	OK	-	
B:2.3	Spot check subcatchment loading nodes are assigned properly.	0	Hydrological inflows modelled in Mike Urban and loaded to pipe network or M21 following AC modelling spec approach	OK	-	

Auckland Council Model Review

Section 3 Review Details

Item	Description	Rating Score	Modeller's Initial Notes	Reviewer's Comments	Modellers Response	Reviewer's Comments (2nd)
B:2.4	Check hydrological method used	0	UHM - SCS dimensionless hydrograph approach, SCS generalised loss method	OK	-	
B:2.5	Identify the curve numbers used in the model. Compare to Auckland Council Soil Maps to confirm appropriate use of curve number for pervious land use.	0	CN 98 for impervious. Pervious CN values assigned as urban good condition grass cover soil types D and C, as per AC instruction relating to the high soil moisture content and peat soils. Shapefile provided by AC (see arcmap MPK) of where to apply soil group D (CN 80), with soil group C (CN 74) to be applied elsewhere. A weighted pervious CN has been applied in the subcatchments that cover/overlap both the soil group D and C extents	OK	-	
B:2.6	Check impervious coverage and compare numbers extracted from model with reported figures. Spot check ED imperviousness using existing impervious layers and aerial photographs – include a screen dump of any issues identified. Review approach for defining MPD.	0	MPD impervious as per modelling recommendations in AC memo (AUP Imperviousness for Hydraulic Modelling 2019-09-04). No ED scenario.	OK	-	
B:2.7	Spot check and document time of concentration for catchments, comparing to TP108 graphical calculations.	0	TP108 graphical method used to derive subcatchment TOC/lag times. Minimum subcatchment slope of 0.5% applied (e.g. 0.5% used if subcatchment slope less than 0.5%) to prevent unrealistically long lag times	OK	-	
B:2.8	Check initial abstraction (Ia) ranges in existing / future scenarios.	0	Initial abstraction of 5mm applied in pervious and 0mm applied in impervious areas. Approach agreed during model schematisation workshop 15/01/21	OK	-	
B:2.9	Check catchment length, slope and Tc are correctly assigned.	3	TP108 graphical method used to derive subcatchment TOC/lag times. Minimum subcatchment slope of 0.5% applied (e.g. 0.5% used if subcatchment slope less than 0.5%) to prevent unrealistically long lag times	The lag times used in the model are not appropriate. Subcatchment lengths should be estimated as the furthest upstream point to the loading nodes. The channelisation factor should be 0.6 for both impervious and pervious areas if subcatchment drained by piped network systems and 0.8 if drained by engineered grass channels.	Flowpath lengths and slopes have been updated to loading node points. A channelisation factor of 0.6 has been applied to all pervious and impervious catchments as agreed at model review meeting. The updated lag times have been added to the model and new hydrology generated.	Model updated.

B:3 - Pipe Networks

Item	Description	Rating Score	Modeller's Initial Notes	Reviewer's Comments	Modellers Response	Reviewer's Comments (2nd)
B:3.1	Confirm all critical network and structures are included in model (trunk network, known flooding points, key structures, etc.)	0	all key structures that affect flow/level at McLennan wetland are included	Yes	-	
B:3.2	Check if the model extent is suitable for generating floodplains, i.e. does it extend far enough upstream and include all flood prone areas.	0	model not for floodplain mapping	Model extent is appropriate.	-	
B:3.3	Check asset naming convention. Can model ID be linked to assets in the GIS	0	SAPID's have been used on all assets where available.	OK	-	
B:3.4	Confirm node/manhole data source flagging and if it is documented for attributes such as lid level, invert level, shaft area, flood type, etc.	0	Lid, invert and diameter source flags have been added to the Mike Urban model	OK	-	
B:3.5	Confirm pipe asset data source flagging and if it is documented for attributes like shape, diameter / width/ height, material, upstream and downstream inverts, etc.	0	pipe diameter and invert data flags have been added to the Mike Urban model	OK	-	
B:3.6	Spot check data entry of asset inspection/survey records for 5 locations	3	checked	Invert levels of the lower pond outlet pipes and some other incoming pipes to the pond do not match with the 2017 McLennan Reserve pond survey data.	Pipe inverts around the McLennan Wetland have been updated using the 2017 survey data	Model updated.
B:3.7	Spot check node attributes (diameter, shaft area, invert level and lid level) match asset data or are interpolated appropriately.	0	checked	OK	-	

Auckland Council Model Review

Section 3 Review Details

Item	Description	Rating Score	Modeller's Initial Notes	Reviewer's Comments	Modellers Response	Reviewer's Comments (2nd)
B:3.8	Compare node lid levels to LiDAR	0	Artistry lane and Swamp Kauri developments: There were significant differences observed at these developments between manhole lid levels and the 2016 LiDAR (the 2016 LiDAR appears to have been captured during earthworks of the development). A 2D surface has been created through interpolation of the manhole lid levels as these are more representative of the developed ground level. There are likely to be uncertainties in the overland flowpaths through these areas and it is recommended that the model is updated with surveyed ground levels/new LiDAR when available.	OK	-	
B:3.9	Check cover types are appropriate i.e. sealed, stored, 2D, etc.	0	Normal manholes: set to 'normal' Loading nodes: set to 'sealed' Dummy loading nodes: set to 'normal' (so can spill to M21) Assumed manholes/connection nodes added at pipe ends (missing asset data): set to 'sealed'. Shafts and bends in Artillery Tunnel and Grove Culvert set to sealed.	OK	-	
B:3.10	Check pipe attributes (diameter, shape, length, material, invert levels) match asset data or are interpolated sensibly	3	checked	The diameter of the culvert connecting upper pond to lower pond should be 1350mm (not 200mm used in the model). The invert levels should also need to be updated based on the 2017 McLennan Reserve pond survey data.	Following discussion and instruction from AC in model review meeting the 1350mm pipe has been fitted with a 200mm orifice plate.	Model updated as per discussion in model review meeting.
B:3.11	Check pipe long section and gradient for steep, zero and negative grades.	0	Following pipes have negative grades, these have currently been left as negative as inverts are based on AC asset data with no further info available. 2001050139 2001072375 2001095319 AC invert data shows increase from 6.04 to 6.08 - left as negative grade as no further info available. 2001072529 AC invert data shows increase from 6.87 to 6.98 - pipe amalgamated with downstream pipe to remove short pipe length and negative grade (improved model stability). 2001094097 PIPE635211 Unknown pipe direction - AC data invert levels of 20.05 at southern end and 20.51 at northern end. left as negative grade as no further info available and pipe could be flowing in north to south direction. 2001054451 AC invert data shows increase from 20.95 to 21.07 - left as negative grade as no further info available. 2000073244 AC data in nodes and pipes shows negative grade - pipe amalgamated with downstream pipe to remove short pipe length and negative grade (improved model stability).	OK	-	
B:3.12	Check if continuation pipe is matched using soffit levels	0	checks made	OK	-	
B:3.13	Ground cover. Identify pipes that have insufficient cover – less than 300mm.	0	Pipe 3000023255 sits above ground level but this is an outlet from a stormwater pond in the upstream catchment. All others have >300mm ground cover.	OK	-	
B:3.14	Identify any network which has decreasing diameters in a down-stream direction.	0	Pipes downstream of the following nodes are recorded as reducing in diameter - these diameters are as per the asset data provided and shown on Geomaps: 2001077219, 2001070994, 2001064948, CONNECT1, LATERAL12, LATERAL14, CONNECT12	OK	-	
B:3.15	Check pipe lengths less than 10m, and if any actions required.	0	A minimum pipe length of 10m has been applied for improved model stability	OK	-	

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Item	Description	Rating Score	Modeller's Initial Notes	Reviewer's Comments	Modellers Response	Reviewer's Comments (2nd)
B:3.16	Check pipe roughness assumptions appropriate for material and condition	0	All pipes and culverts assigned as concrete normal (n 0.013) apart from: 3 existing pipe assets with unknown material type - assumed to be concrete normal (n 0.013) 2 pipes material "Brick". Roughness assigned as concrete rough (n 0.015) at SW_LINE12 a roughness value of 0.011 has been applied due to lining, following confirmation with AC. At SW_LINE11 (arched armco pipe) the manning's roughness is currently set to concrete. The dimensions and roughness of this arched pipe require confirmation with AC. Unlikely to have an impact on this assessment but should be incorporated in any future model updates.	OK	-	
B:3.17	Check manhole head losses in the model.	0	Manholes - Km 0.3 mean energy approach Inlets to pipes and culverts - Total HLC 0.5 Outlets from pipes/culverts - Total HLC 1.0 For connection nodes added (no asset data) Km 0.3 mean energy approach has been applied at pipe junctions (3 or more pipes) and 'no cross section changes' applied at pipe joins (2 pipes).	OK	-	
B:3.18	Check entry and exit losses of pipes and any minor losses caused by bends, side connections or joint defects, etc.	0	Standard culvert inlet (0.5) and outlet (1.0) HLC's have been applied apart from the Grove Road Culvert. The Grove Road culvert energy losses have been modelled as follows: Inlet - 0.26 (rectangular culvert, flared wingwalls /top edge bevelled /single barrel) Total bend loss of 0.3 (12 degree bend 0.05 and 60 degree bend 0.25)	OK	-	
B:3.19	Check natural depression areas or dry pond are modelled with proper outlet configuration i.e. it drains properly after flooding.	0	Drainage from sports field adjacent to McLennan wetland to be added to model (200mm pipe to lower wetland). Ponding on upstream side of railway near Ingram Street - no obvious outlets from this area other than pipe network already modelled. This is also outside of study area of interest	OK	Drainage from the sports field adjacent to McLennan wetland has been added to model. Using 2017 survey data and a 200mm pipe to the lower wetland shown in AC_Data.	OK
B:3.20	How is storage compensation applied to any trimmed network.	0	no specific compensation has been applied. The extent of upstream pipe networks in currently undeveloped areas is unknown. The low flow/permanent water level in the Awakeri wetland channels is not included and this compensates the trimmed network storage to an extent.	OK		

B:4 - Channel/Stream Networks

Item	Description	Rating Score	Modeller's Initial Notes	Reviewer's Comments	Modellers Response	Reviewer's Comments (2nd)
B:4.1	Are channels modelled appropriately? (in 2D or as 1D river reaches)	0	Awakeri wetlands/conveyance channels and Kauri Flats channel topography represented in 2D. Roadside drains LiDAR 2016. All open channels are modelled with 1m2 resolution (highest resolution used in the 2D mesh)	OK	-	
B:4.2	In case of burning surveyed cross-sections in 2D, spot check cross-sections from 2D bathymetry compared to the surveyed cross-sections.	0	checks made	OK	-	
B:4.3	Spot check modelled cross-sections and banklines with LiDAR	0	Awakeri wetland channels do not tie in with LiDAR at numerous locations (due to recent development), but the flow remains in bank in the 100 year MPD	OK	-	
B:4.4	Is location and spacing between cross sections appropriate? (e.g. maximum dx in MIKE11)	0	n/a - no 1D channel model	N/A	-	
B:4.5	Spot check of modelled cross-sections whether it includes low flow channel.	0	n/a - no 1D channel model	N/A	-	
B:4.6	Spot check data entry of survey records for 5 locations	0	No surveyed cross sections. 2D surfaces from TIN's/dwg's or existing HEC-RAS 2D surface.	N/A	-	
B:4.7	Identify any topography which may cause instabilities – such as flat sections.	0	majority of catchment is of very flat topography including through the culvert structures of the Awakeri Wetlands	OK	-	
B:4.8	Review the use of “channel markers” or “new panels”.	0	n/a - no 1D channel model	N/A	-	

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Item	Description	Rating Score	Modeller's Initial Notes	Reviewer's Comments	Modellers Response	Reviewer's Comments (2nd)
B:4.9	Identify if cross sections are drawn properly: - check length and extents sufficient to cover flood flows - any sections which are not perpendicular to the direction of flow. - are sections straight lines? Comment on the impact to the conveyance, and to the model results.	0	n/a - no 1D channel model	N/A	-	
B:4.10	Check locations where flooding extends from the channel to the 2D mesh – comment on merging of 1D/2D representation.	0	n/a - no 1D channel model	N/A	-	
B:4.11	Comment on application of roughness values.	0	a roughness value of n 0.04 has been applied to the Awakeri wetlands/channels. This is to account for vegetation, scour protection, logs etc within the channels. Bend/losses around structures in the the wetlands is accounted for by using 2D modelling approach.	OK	-	
B:4.12	Identify any double counting of volumes, in overland flow paths basins other cross sections	0	n/a - no 1D channel model or basins	N/A	-	
B:4.13	Check gradient for steep, zero and negative grades.	0	n/a - no 1D channel model	N/A	-	
B:4.14	Confirm no double counting of flood storage volumes, at locations such as basins or connection nodes at the ends of channels, , etc.	0	n/a - no 1D channel model or basins	N/A	-	

B:5 - Hydraulic Structures and Control Elements

Item	Description	Rating Score	Modeller's Initial Notes	Reviewer's Comments	Modellers Response	Reviewer's Comments (2nd)
B:5.1	Are inlets represented correctly? Do they align with surrounding terrain and have correct inlet control/headloss parameters?	0	Standard culvert inlet (0.5) and outlet (1.0) HLC's have been applied apart from the Grove Road Culvert.	OK	-	
B:5.2	Check outlet and/or outfall representations. Do they align with surrounding terrain or connect appropriately with downstream features?	0	minor changes to the 2D mesh have been made to ensure outlet levels match the 2D topography.	OK	-	
B:5.3	Check representation of culverts. Shape, number of barrows, inlet/outlet losses, roughness, gradient, etc.	0	The Grove Road culvert energy losses have been modelled as follows: Inlet - 0.26 (rectangular culvert, flared wingwalls /top edge bevelled /single barrel) Total bend loss of 0.3 (12 degree bend 0.05 and 60 degree bend 0.25)	OK	-	
B:5.4	Review bridges representation: - cross sections - contraction and expansion losses - bridge deck, profile and coefficients - bridge skew - bridge opening, gradient, inlet and outlet losses - bridge piers or other obstructions	0	No bridges modelled - footbridges in Awakeri Wetland assumed to have minimal impact on flows at McLennan wetland (i.e. considered negligible with regards to the purpose of this project).	N/A	-	
B:5.5	Check representation of storages, depressions, dams or constructed ponds: - stage storage relationship - any controls - inlets and outlets - initial or permanent water levels - overtopping arrangements (single level or irregular shape; weir coefficients; 2D mesh / breaklines);	3	Artillery Drive Tunnel - The previous Q-H relationship (HAT 2.34) used in the "Assessment of McLennan Upper Wetland and Artillery Drive Tunnel Design Performance, AC 2016) has been used. This Q-H relationship was derived using spreadsheet calculations and included all hydraulic losses such as the bell-mouth inlet, the outlet and various horizontal /vertical bends. The calculations are based on Jacobs design of the structure. Losses have therefore not been included at the tunnel shafts/bends within the model, to avoid double counting the losses (incorporated into the Q-H relationship).	The inlets of the Artillery Drive Tunnel (two scruffy domes) should be modelled based on as-built plans - see Figure 1.	A new QH relationship has been developed based on the as-builts. Refer to description in metadata tab and provided spreadsheet. The Inlet structure is modelled as an outlet for stability, with QH control from spreadsheet in pipe. ILOL54728 is used as the control node for QH as MU does not allow outlet nodes for control. ILOL54728 has consistent WL with M21 across the wetland. It was found that modelling the Artillery inlet as a manhole did not cause the QH to work as intended (water level in dam used as H) due to a drop in WL within the manhole structure.	Model updated as per discussion in model review meeting.

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Item	Description	Rating Score	Modeller's Initial Notes	Reviewer's Comments	Modellers Response	Reviewer's Comments (2nd)
B:5.6	Check pump configurations. On/off levels, pump type, pump curve, pump controls, etc.	0	no pumps in the catchment/modelled	N/A	-	

B:6 - Other Asset Features

Item	Description	Rating Score	Modeller's Initial Notes	Reviewer's Comments	Modellers Response	Reviewer's Comments (2nd)
B:6.1	Soakage modelling methods and representation in the model.	0	Soakage is present in catchment but thought not to perform well in winter months especially during high magnitude flood events. It agreed during model schematisation workshop with AC (15/01/2021) to not be included as effects on flooding likely to be minimal. soakage in the catchment is primarily for peat recharge.	N/A	-	
B:6.2	How is the soakage outlet capacity modelled. The assumptions, e.g. ARIs, etc.	0	soakage not included (refer to above comment)	N/A	-	
B:6.3	Review the use of weir units in the model. Comment on the weir representation and coefficients used	0	M21 Dike structures (weirs/spillways) - M21 Dike structures have been used to define the crest level of the McLennan wetland spillway (2017 survey data). The north and south spillway crests have a coefficient of 1.6 (grass embankments) and the broad crested rock armoured spillway has a coefficient of 1.2. The weir structure just upstream of the Grove Road Culvert inlet fish ladder has also been incorporated as a M21 dike structure with a coefficient of 1.6. The fish ladder weirs and the low flow weirs through the Awakeri wetlands are not modelled (other than being within the 2d mesh DEM) as they are deemed to have a negligible impact in high magnitude flood events.	OK	-	
B:6.4	Review the use of orifice units in the model, comment on the associated coefficients applied.	0	no orifice units used in model	N/A	-	
B:6.5	Check representation of tunnels/underpasses	0	Artillery Drive Tunnel - The previous Q-H relationship (HAT 2.34) used in the "Assessment of McLennan Upper Wetland and Artillery Drive Tunnel Design Performance, AC 2016) has been used. This Q-H relationship was derived using spreadsheet calculations and included all hydraulic losses such as the bell-mouth inlet, the outlet and various horizontal /vertical bends. The calculations are based on Jacobs design of the structure. Losses have therefore not been included at the tunnel shafts/bends within the model, to avoid double counting the losses (incorporated into the Q-H relationship).	OK	-	

B:7 - 1D Overland Flow Paths

Item	Description	Rating Score	Modeller's Initial Notes	Reviewer's Comments	Modellers Response	
B:7.1	Modelled overland flow paths locations and downstream connectivity.	0	n/a - no overland 1D model	N/A	-	
B:7.2	Comment on application of roughness values applied to 1D overland flow paths.	0	n/a - no overland 1D model	N/A	-	
B:7.3	Review section shape for 1D overland flow paths	0	n/a - no overland 1D model	N/A	-	
B:7.4	Check OLFP gradient and levels	0	n/a - no overland 1D model	N/A	-	

B:8 - 2D Model Components

Item	Description	Rating Score	Modeller's Initial Notes	Reviewer's Comments	Modellers Response	Reviewer's Comments (2nd)
B:8.1	Review 2D extent and mesh sizes (any terrain sensitive meshing, and no extremely large or small meshes) Are mesh sizes appropriate at inlets and outlets.	0	1m2 in and around channels and wetlands. 4m2 on floodplain. 10m2 in areas outside catchment/areas of interest	OK		
B:8.2	How have building footprints been represented	0	No changes to 2016 LiDAR DEM at buildings. 2D roughness of n 0.35 applied at existing building footprints	OK		
B:8.3	Review DEM and identify if any errors in DEM, e.g. around buildings	0	Model DEM at swamp Kauri development (see FIGURES tab)	OK		
B:8.4	Check representation of any key obstructions	3	culvert/pipe asset data required at Battalion Drive see FIGURES tab)	Box culverts under Battalion Drive need to be included in the model. AC Project Manager will provide the available data.	The structure has been added using HNZ as-built provided (DWG 125173-AB3B-420)	Model updated.
B:8.5	Check roughness zones and values	0	Roughness shapefile with land use attributes included in ArcMap MPK. Values were defined using Unitary plan, roads, building footprints and wetland extents used to define values. Concrete paths and vegetation around McLennan wetland digitised manually using aerials	OK	-	

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Item	Description	Rating Score	Modeller's Initial Notes	Reviewer's Comments	Modellers Response	Reviewer's Comments (2nd)
B:8.6	Review and check double counting between 1D and 2D model components. For example 2D cells not blocked out where flow is represented in 1D.	0	no 1D channels	N/A	-	
B:8.7	Check 1D/2D interface and coupling method is appropriate. Check appropriate 1D/2D connections are applied at 2D nodes, inline banks, river reach banks, etc. E.g. appropriate Qmax at 2D manhole, RESERVOIRHEIGHT= 100m, M21_AS_GROUNDLEVEL=0 in dhiapp.in file	0	RESERVOIRHEIGHT= 100m, M21_AS_GROUNDLEVEL=0 in dhiapp.in file	OK	-	

C - Model Results Review

C:1 - Model Results Check

Item	Description	Rating Score	Modeller's Initial Notes	Reviewer's Comments	Modellers Response	Reviewer's Comments (2nd)
C:1.1	Have all events been simulated and results provided?	0	100 year MPD provided for initial review before options are modelled	100yr ARI MPD CC	-	
C:1.2	All correct input data assigned to the run file for each simulation? and check simulation start and stop times.	0	checked	OK	-	
C:1.3	Check if flow, level and velocity are within reasonable range for pipes. - Identify Pipes with velocities >6m/s; - Check if inlet control should be included.	0	checks made - no pipes with velocity over 6m/s	OK	-	
C:1.4	Check if flow, level and velocities are within reasonable range for overland flow paths, open channels and floodplain	0	checks made	OK	-	
C:1.5	Is there any depression area or ponding not drained at the end of simulation? Check outlet configuration for depression.	0	Drainage from sports field adjacent to McLennan wetland to be added to model (200mm pipe to lower wetland). Ponding on upstream side of railway near Ingram Street - no obvious outlets from this area other than pipe network already modelled. This is also outside of study area of interest	OK	Drainage from the sports field adjacent to McLennan wetland has been added to model. Using 2017 survey data and a 200mm pipe to the lower wetland shown in AC_Data.	OK
C:1.6	Are predicted losses at manhole and pipe connections within reasonable range and as expected?	0	checked	OK	-	
C:1.7	Are predicted losses at inlet and outlet within reasonable range and as expected?	0	checked	OK	-	
C:1.8	Culvert Performance: - Is culvert operating as expected? Head losses within reasonable range. - Is flow limiting observed for 1D/2D connection at inlet/outlet? - Spot Check with HY8 and manuals calcs at least 2 locations, more maybe required if model includes large number of culverts.	0	Grove road culvert performing as expected. some instabilities at AWACUL1&2 (M11) but these do not affect hydrograph peak. The Walters Road Culvert (WALCUL) and culvert immediately downstream (AWACUL3&4) have not been coupled as numerous stability issues were encountered at these locations during the model build despite a range of tests (coupling parameters, MU and M11 representation, invert levels). The culvert openings are currently modelled in 2D only (topographical opening in the mesh). The upstream water level does not reach the soffit level of the culverts and these culverts are sufficiently far upstream of McLennan wetland to have a minimal impact on results.	OK	-	
C:1.9	Bridge Performance: - Is bridge operating as expected? - Are contraction and expansion losses within reasonable range.	0	n/a - no bridges in model	N/A	-	
C:1.10	Check if 1D / 2D flow transfers as expected. Any location with significant instabilities	0		OK	-	
C:1.11	Check if pump operation as expected	0	n/a - no pumps modelled	N/A	-	

C:2 - Model Validation

Item	Description	Rating Score	Modeller's Initial Notes	Reviewer's Comments	Modellers Response	Reviewer's Comments (2nd)

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Item	Description	Rating Score	Modeller's Initial Notes	Reviewer's Comments	Modellers Response	Reviewer's Comments (2nd)
C:2.1	Compare TP108 graphical and modelled peak flows at a range of key locations, comment on any significant differences, and the impact on model predicted flows.	0	UHM and graphical method compared. No major differences in peak flow observed. Largest difference occurred at the impervious area of subcatchment AWA_SC3_Imp. The UHM method gives a flow of 2.91 whereas the tp108 graphical method gives a flow of 2.83 (UHM flow increased by 3%)	OK	rechecked with updated hydrology and no major differences in peak flow observed	
C:2.2	Check if overall flood extent sensible. Compare new flood extent with any previous floodplains.	0	flood extents in area of interest deemed sensible	Overall flood extent are reasonable.	-	
C:2.3	Validation against RFS records, anecdotal evidence?	0	validation not part of scope. Model is MPD future base scenario terrain, channels and land use so RFS records not applicable in area of interest	N/A	-	
C:2.4	Validation against gauged data or flood surveys?	0	as per comment above	N/A	-	

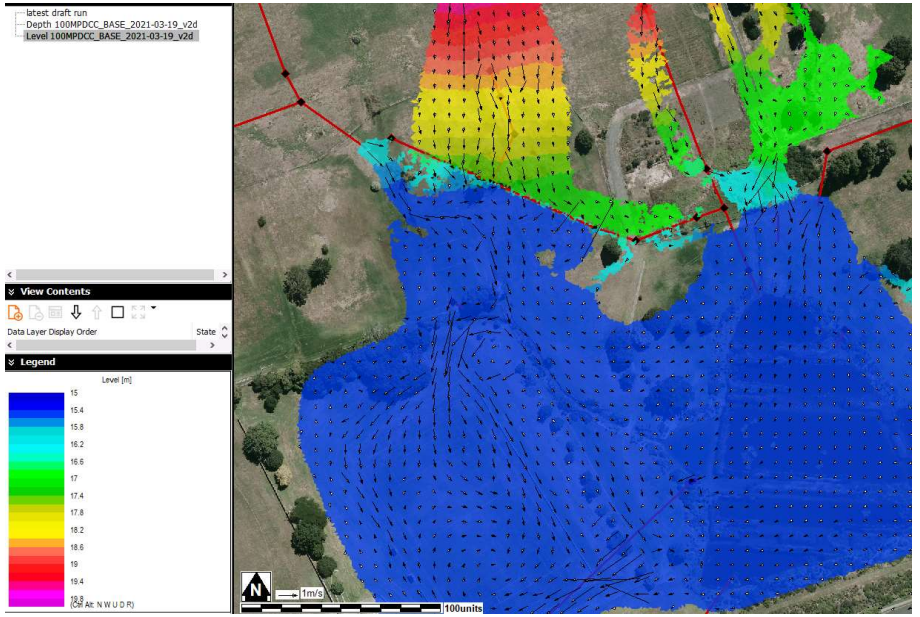
D - Additional Checks

D:1 - Additional Check Items

Item	Description	Rating Score	Modeller's Initial Notes	Reviewer's Comments	Modellers Response	Reviewer's Comments (2nd)
D:1.1	Does the model report provides adequate documentation on: - project objectives and purpose; - data analysis and model schematisation; - modelling methodology for key model components - assumptions and limitations.	0	report and options assessment programmed for after baseline model review	In future stage	-	
D:1.2	If applicable, are options represented adequately with appropriate levels of details? Comment on confidence level based on both model setup and model results.	0	report and options assessment programmed for after baseline model review	In future stage	-	
D:1.3	Should any aspects of the model be refined or redone in order to further investigate flooding effects?	0	model topography should be updated with new terrain of developments if smaller pipe networks are modelled	OK	-	
D:1.4	Which scenarios are modelled? Comment on the adequacy of scenarios modelled for achieving the project objectives	0	100 year MPD provided for initial review before options are modelled	100yr ARI MPD CC	-	
D:1.5	Any other assumptions used in the model that may have an impact on the overall model performance and meeting project objectives?	0	Model DEM at swamp Kauri development (see FIGURES tab) and culvert/pipe asset data required at Battalion Drive see FIGURES tab)	AC Project Manager will provide the available data.	The Battalion drive culvert has been added but the DEM at the developments has not been provided. The mesh has been updated at the edge of the developments to interpolate/smooth edges and remove the vertical drops previously experienced.	OK
D:1.6	Describe any additional checks or issues to raise	0	More recent survey (2017) of the McLennan wetland was recently provided in PDF format. If the survey is available in dwg format then the contours should be updated in the model (if required)	AC Project Manager will provide the available data.	The 2D mesh has been updated with 2017 survey data at McLennan wetland.	OK

Artistry lane and Swamp Kauri developments:
There were significant differences observed at these developments between manhole lid levels and the 2016 LiDAR (the 2016 LiDAR appears to have been captured during earthworks of the development). A 2D surface has been created through interpolation of the manhole lid levels as these are more representative of the developed ground level. There are likely to be uncertainties in the overland flowpaths through these areas and it is recommended that the model is updated with surveyed ground levels/new LiDAR when available.
The model results show a sudden drop in terrain and water level at the Swamp Kauri development (area 1 in figures below) where the terrain created from the Lid levels does not tie in well with the LiDAR. The exact slope to the developed ground level is unknown.

UPDATE:
The steep drops between the development DEM's and the LiDAR/2017 survey contours have been smoothed through interpolation between the datasets.



asset data required for culvert/pipe discharging to wetland from Military camp

UPDATE: Data received and structures have been added to the model

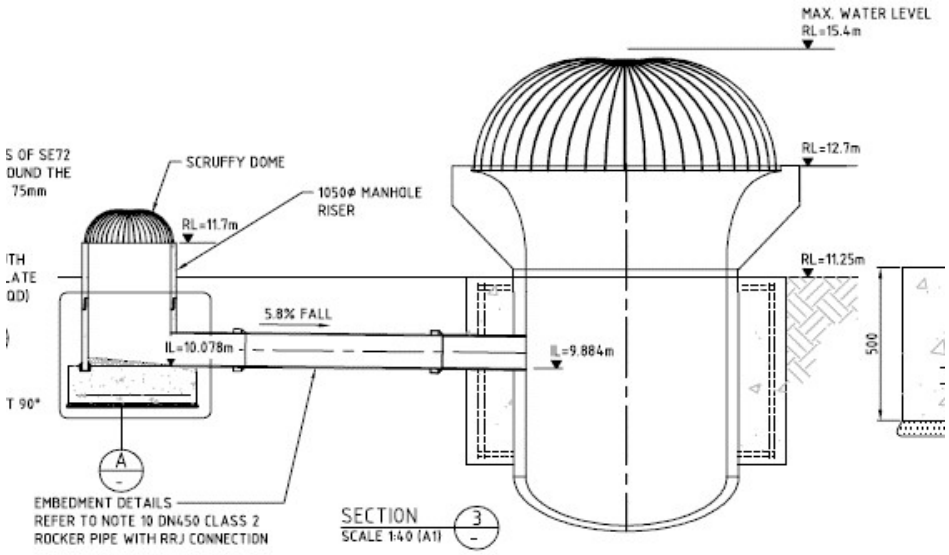

MAX. WATER LEVEL
RL=15.4m
S OF SE72
DUND THE
75mm
SCRUFFY DOME
1050ø MANHOLE
RISER
RL=11.7m
RL=12.7m
RL=11.25m
TH
ATE
QD)
5.8% FALL
IL=10.078m
IL=9.884m
500
T 90°
A
EMBEDMENT DETAILS
REFER TO NOTE 10 DN450 CLASS 2
ROCKER PIPE WITH RRJ CONNECTION
SECTION
SCALE 1:40 (A1)
3

Figure 1


Scruffy Dome

Appendix C: Flood extent figures