APPENDIX 17

DESIGN REPORT [TONKIN AND TAYLOR]

REPORT

Awanui River Flood Protection Scheme

Preliminary Design Report

Prepared for
Northland Regional Council
Prepared by
Tonkin & Taylor Ltd
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1 Introduction

Northland Regional Council (NRC) appointed Tonkin & Taylor Limited (T+T) to review the preliminary design of the Awanui River flood protection scheme in June 2018. The original preliminary design was provided by T+T to NRC in 2014 as part of flood modelling and mitigation investigations for Council. The review of the preliminary design was to be conducted with new LiDAR survey data available and following the Rangitaiki River Scheme Review into April 2017 Edgecumbe flood event.

The Awanui River and its tributaries drain the northern side of the Mangamuka Range and flow northwards through Kaitāia and across the Awanui flats to discharge to the Rangaunu Harbour at Unahi. Kaitāia sits on the floodplain at the point where the Awanui River is confined before spilling out on to an alluvial fan and the Awanui flats. There are three principal tributaries in the hills upstream of Kaitāia: Te Puhi Stream, Victoria River and Takahue River. The Tarawhataroa Stream flows through the western parts of Kaitāia, and into the Awanui River further downstream.

Extensive drainage and flood control works have been constructed to lessen the flood risk on the Awanui flats. Initial works in the early 1900s focused on bringing land near the harbour into production by preventing tidal flooding. Stopbanks and floodgates gradually extended upstream to provide flood management for more productive floodplain land. Downstream of Kaitaia, the Whangatane Spillway was constructed in 1928 to divert high flood flows more directly to the harbour. Following a significant flood in 1958, which flowed through urban Kaitāia, there was a comprehensive upgrade to the scheme with stopbanks constructed around Kaitāia and increased capacity in the Whangatane Spillway.

Initially the Awanui River Flood Management scheme was managed by District Councils. Since 2005 NRC has been responsible for operation of the Scheme, and has planned the following improvements:

- Modifications to stabilise stopbanks and increase capacity
- Reducing flood overflow to Tarawhataroa Stream
- An emergency spillway beneath the slow-moving Bell's Hill slip
- Annual maintenance works to protect public safety.

This T+T review was conducted in two stages:

- Stage 1: Baseline model build to understand the distribution of 100 year ARI flood flows between the Awanui River, Tarawhataroa and Whangatane Spillway
- Stage 2: Design and analysis to update the Awanui Preliminary Design and provide a digital terrain model of the scheme design to NRC.

This report presents the outcome of the review in the following sections:

- Section 2 Hydraulic model build
- Section 3 Preliminary design
- Section 4 Bridge waterways

The detailed scope of this review is as set out in the T+T Letter of Engagement to NRC dated 20 June 2018.

2 Hydraulic model build

2.1 Model overview

Hydraulic modelling has been undertaken using MIKE Powered by DHI software. The model is comprised of a one-dimensional (1D) model of the river channels using MIKE 11 linked to a two-dimensional (2D) model of the flood plain using MIKE 21.

The model extents include the reaches of the Awanui River, Whangatane Spillway and the Tarawhataroa Stream relevant to the proposed works. **Figure 2.1** shows an overview of the model extents. The key areas of interest are:

- The area south of Kaitaia where the Awanui River spills across State Highway 1 (SH1) into the Tarawhataroa Stream
- The reach of the Awanui River through Kaitaia, from the SH1 spill through to the Waikuruki Bridge (also known as the North Road Bridge)
- The Awanui River choke, where the overflow weir into the Whangatane Spillway is located
- The length of the Whangatane Spillway from the Awanui River through to the confluence with the Mangatete River.

The full length of the Awanui River downstream of Kaitaia has been included in the model extent. However, no attempts have been made to calibrate or ensure the accuracy of the model outside of the areas of interest between the SH1 spill and Waikuruki Bridge.

Three different scenarios have been modelled:

1. The Calibration scenario represents the river topography at the time of three key flood events between 2007 and 2011. A smaller 2D model extent was used to enable faster run times
2. The Baseline scenario captures changes to the river topography between 2011 and 2019. The 2D model was extended to enable better representation of out of bank flooding along the Whangatane Spillway
3. The Design scenario incorporates the proposed scheme design into the baseline scenario, including the proposed spillways, stopbanks and benching works.

Table 2-1 summarises the key model parameters used for all three scenarios.

A detailed set of maps showing the hydraulic model extent and river chainages is provided in Appendix A1.

The One Tree Point (OTP) 1964 vertical datum has been used for the hydraulic modelling and throughout the assessment.

Table 2-1: Summary of hydraulic model parameters

Software version	MIKE FLOOD Version 2016 Service Pack 3
Grid size	4 m by 4 m
Time step	0.5 seconds
2D eddy viscosity	0.64

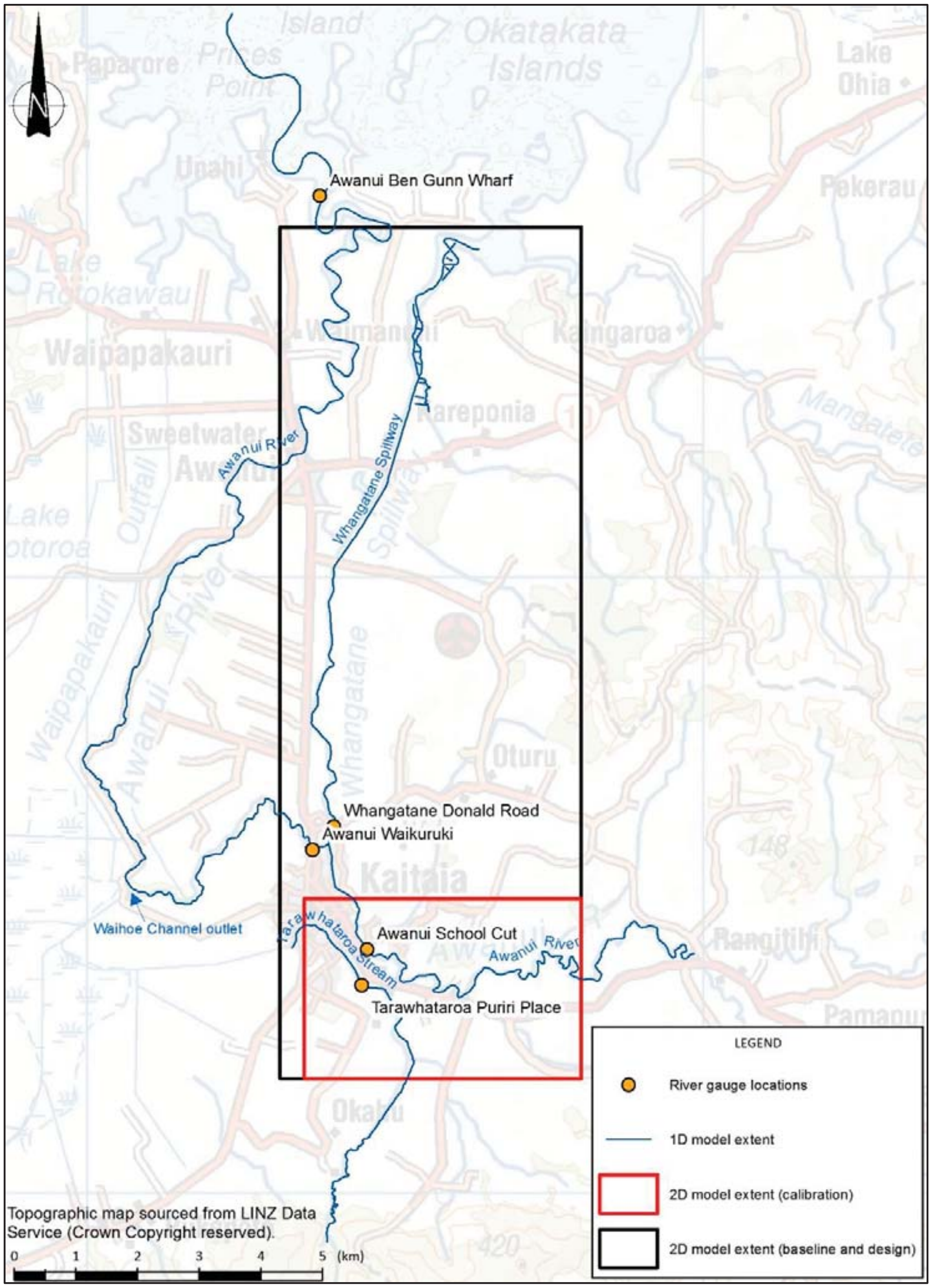

Figure 2.1: Model overview

2.2 Calibration model

2.2.1 Model overview

Data for three historic flood events were provided by NRC for calibrating the hydraulic model. The three events were:

1 January 2011
2 February 2007
3 July 2007.

The particular focus of the calibration was to model accurately the flow splits at the SH1 overflow south and upstream of Kaitaia, and the flow split at the Whangatane Spillway intake further downstream.

The 2D domain has been limited to just the area around the SH1 overflow to reduce the run time of the calibration model.

2.2.2 Model inputs provided by DHI

Concurrently with the hydraulic model build by T+T, DHI has been developing a full hydrological and hydraulic model for the entire Awanui River catchment. DHI provided some of its initial network and cross section model files to use as a base for the development of the T+T model.

The following components were included in the provided files:

1. River branches within the network file. Branch locations were defined by DHI to match the 2018 LiDAR Digital Elevation Model (DEM) provided by NRC
2. Cross sections for these branches:
   - The 2018 LiDAR data do not include data below the water level in the river channels. Therefore DHI combined the existing survey data for the channels (generally from NRC between 2004 and 2015) and interpolated along the branches to create a DEM for the low flow channel. Cross sections were then extracted from the combined DEM, comprising the survey DEM below the water level and the LiDAR DEM above the water level
   - Earthworks have been constructed at the Whangatane Spillway intake and the Awanui River choke since the calibration events and prior to the 2018 LiDAR survey, therefore cross sections in these locations were extracted from pre works survey data provided by NRC
3. Hydraulic structures within the network file, including bridges, weirs and culverts.

River branches and structures outside of the area relevant to the proposed works were cropped out of the T+T model.

2.2.3 Topography modifications

The topography in the 1D domain provided by DHI has been modified in four locations:

1. At the downstream end of the Whangatane Spillway. The model was glass-walling in this location due to the 1D-only representation. The 1D model was therefore extended to include flooded areas outside of the main channel by adding additional branches and extending existing cross sections. Data for new and extended cross sections were extracted from the 2018 LiDAR DEM. Some glass-walling does still occur in the calibration events in other sections of the Spillway which could not effectively be represented in the 1D model. The accuracy of the calibration downstream of these locations is limited
2. Between chainages 10510 and 10720 on the Awanui River. Cross sections were extended based on the 2018 DEM to prevent glass-walling
3. Between chainages 11620 and 11800 on the Awanui River. Cross sections were extended based on the 2018 DEM to prevent glass-walling
4. At the locations of the natural sandstone wedges on the Whangatane Spillway (chainages 9420, 9450 and 9570 to 9630). Cross sections were updated to include these wedges from survey data provided by NRC.

The model topography across the 2D domain is based on the 1 m resolution DEM provided by NRC from 2018 LiDAR. One modification was made to the 2D grid elevations just south of SH1, where the channel invert level was interpolated through a track crossing to maintain connectivity.

2.2.4 Boundary conditions

Inflow boundaries have been applied at the upstream model extents on the Awanui River and Tarawhataroa Stream using flows provided by NRC based on hydrological analysis of the School Cut and Puriri Place river gauges. The magnitude and timing of the inflows were adjusted as part of the calibration process (refer **Section 2.2.8**).

Additional inflows to the Awanui River and Whangatane Spillway downstream of the School Cut gauge have not been considered as part of this model but will likely be included within the full catchment model produced by DHI.

Tidal boundaries have been applied at the downstream ends of the Whangatane Spillway and the Awanui River using data provided by NRC from the Ben Gunn gauge.

The downstream boundary on the Tarawhataroa has been defined using a flow stage (QH) relationship based on the conveyance in the final cross section. A sensitivity test has shown that this boundary is sufficiently downstream of the area of interest that water levels are not sensitive to the boundary condition. A free outflow has been added to the boundary of the 2D domain adjacent to where the Tarawhataroa Stream exits the 2D model to prevent water from ponding at the boundary and impacting on water levels in the Awanui River.

Results sensitivity has also been modelled to confirm that the exclusion of the Waihoe channel from the model does not have a significant effect on the model results in the Awanui River within the area of interest.

2.2.5 Roughness

1D roughness parameters have been determined through calibration of the hydraulic model, refer **Section 2.2.8** below. The calibrated roughness values are generally significantly higher than expected, this likely reflects additional energy losses associated with the sinuosity of the channel, and potentially unknown vegetation and/or debris blockages at the time of the calibration events.

2D roughness parameters have been defined based on land use using the Land Cover Database (LCDB) v4.1 and building footprints and road parcels downloaded from LINZ Data Service. The road parcels were adjusted in some locations to match aerial photos. **Table 2-2** lists the Manning's M roughness values assigned to each land use, values have been chosen based on guidance from the Auckland Council Stormwater Modelling Specifications (2011). The 2D roughness values were modified in the area between SH1 and the Tarawhataroa Stream as part of the calibration process.

Table 2-2: 2D roughness values

Land use	Manning's M
Building footprints	2.9
Forest	6.7
Herbaceous vegetation	9.1
Built-up area	10.0
Gorse, manuka, kanuka, mangroves	18.9
Grass areas	20.0
Open water	23.3

Land use	Manning's M
Surface mine or dump	37.0
Urban parkland	47.6
Road parcels	50.0

2.2.6 Hydraulic structures

Nine weirs and 15 bridges are included within the model. The locations are shown on the maps in Appendix A1. Structures have been represented in the model as they were in the DHI model.

2.2.7 1D/2D linking

Lateral links have been used to connect the 1D and 2D domains along the top of the stopbanks. A weir structure type was used for the lateral links, and levels were sourced from the Mike 21 elevation grid. The default depth tolerance (0.1 m), weir coefficient (1.838) and friction (n = 0.05) were maintained. Lateral links have not been extended along the full 1D extent within the 2D domain but have been located where necessary to model the SH1 overflow accurately.

2.2.8 Calibration

The following calibration data were provided by NRC for the three historic flood events:

- Time series data for water levels and flows at three gauges:
  - Awanui at School Cut
  - Tarawhataroa at Puriri Place
  - Whangatane Spillway at Donald Road
- Time series data for water levels at the Awanui at Ben Gunn tidal gauge
- Surveyed observed flood levels at points along the river channels
- Gauge rating data for six gauges within the area of interest.

For each event a number of calibration runs were modelled, testing the effects of variations to the following parameters:

- 1D Manning's n roughness values
- Timing and magnitude of the inflows
- 2D Manning's M roughness values across the area of the SH1 overflow
- Additional energy losses at the choke and modified weir losses at the Whangatane Spillway intake
- Lateral link parameters.

Of the parameters listed, only the first three were found to be effective at matching the model results to the recorded data.

For the January 2011 and February 2007 events a suitable calibration was achieved using the same set of parameters:

- 1D Manning's n roughness values for each reach are listed in Table 2.3
- Inflows on the Awanui River were increased by 8 % from those provided by NRC
- The 2D Manning's M roughness value was increased from 20 to 40 for a grassed area between SH1 and the Tarawhataroa Stream.

For the larger July 2007 event, different parameters were required to match observed flood data. This could potentially be due to differences in vegetation, debris blockages or channel morphology in this event:

- Lower Manning's n roughness values were applied in the upper reaches of the Awanui River, as shown in **Table 2.3**
- The 2D Manning's M roughness value was decreased from 20 to 14.3 for all the 2D grass areas.

A comparison of the gauge and modelled flows at the Awanui School Cut gauge for the three events using the final parameters is provided in **Figure 2.2**. Additional results for all three gauges are provided in Appendix A2.

Roughness values for the Baseline scenario model were chosen based on the results of the calibration and following discussion with NRC. The parameters from the July 2007 calibration were considered to be the most applicable due to the magnitude of the event compared to the design events. A small variation was applied to 1D roughness values along one section of the Tarawhataroa Stream where it was considered that the calibration could have been improved. The adopted roughness values are listed in **Table 2.3**.

Table 2.3: 1D roughness values for Calibration and Baseline scenarios

Branch	Chainage	Nearest landmark	1D Manning's n roughness values			
			Jan 2011	Feb 2007	Jul 2007	Baseline
Tarawhataroa	0		0.055	0.055	0.055	0.055
Tarawhataroa	5000		0.055	0.055	0.055	0.055
Tarawhataroa	5500		0.055	0.055	0.055	0.060
Tarawhataroa	6500		0.055	0.055	0.055	0.060
Tarawhataroa	6800	Bank Street	0.035	0.035	0.035	0.035
Tarawhataroa	7000		0.035	0.035	0.035	0.035
Tarawhataroa	7200		0.055	0.055	0.055	0.055
Tarawhataroa	7870		0.055	0.055	0.055	0.055
Whangatane Spillway	0		0.048	0.048	0.048	0.048
Whangatane Spillway	5200	Quarry Road	0.038	0.038	0.038	0.038
Whangatane Spillway	7050	SH10	0.038	0.038	0.038	0.038
Whangatane Spillway	11880	Mangatete River	0.038	0.038	0.038	0.038
Awanui	0		0.098	0.098	0.070	0.070
Awanui	7000		0.098	0.098	0.070	0.070
Awanui	7930		0.098	0.098	0.090	0.090
Awanui	9000		0.090	0.090	0.090	0.090
Awanui	10061	Te Ahu	0.045	0.045	0.045	0.045
Awanui	11800	Allen Bell Park	0.045	0.045	0.045	0.045
Awanui	12025	Whangatane Spillway	0.085	0.085	0.085	0.085
Awanui	15000		0.085	0.085	0.085	0.085
Awanui	18000		0.050	0.050	0.050	0.050

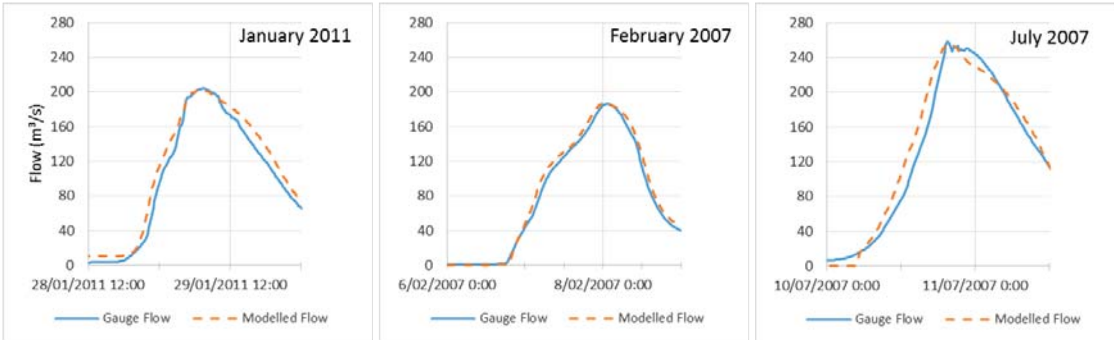

Figure 2.2: Comparison of gauge and modelled flows at the School Cut gauge for the calibration events

2.3 Baseline model

2.3.1 Model overview

The Baseline model was updated from the calibration scenario to reflect changes in topography since the calibration events, these are detailed in **Section 2.3.2**.

Additionally, the 2D domain was extended to provide a better representation of out of bank flooding along the full length of the Whangatane Spillway. Lateral links were extended to match the extended 2D domain, and additional free flow boundaries were included where necessary to prevent ponding on model boundaries. No overflows or connectivity to the floodplain has been considered for the Awanui River downstream of chainage 13390 as this is outside the area of interest for the proposed scheme upgrade works.

The model was run for the 100 year and 20 year Annual Recurrence Interval (ARI) events using inflows for the Awanui River and the Tarawhataroa Stream provided by NRC. The downstream tidal boundary was also provided by NRC based on a present day 2 year ARI storm surge sea level. The timing of the peak of the tide was aligned to match the peak flow.

2.3.2 Topography updates

Topography within the 1D river channel and the 2D grid was updated in the following locations to reflect earthworks undertaken along the Awanui River and Whangatane Spillway since the calibration events and since the 2018 LiDAR was flown:

1 2016/2017 earthworks at the Awanui River choke (chainages 11890 to 12310) and entrance to the Whangatane Spillway (chainages 0 to 275). Levels for these cross sections were updated using the DEM provided by DHI which combined the 2018 LiDAR with the interpolated low flow channels
2 2019 Bells Hill earthworks (Awanui River, chainages 10510 to 11110). 1D cross sections and 2D grid levels were updated based on as-built survey provided by NRC. Cross sections were extended to reach the top of the new stopbanks
3 2019 Te Ahu benching works (Awanui River, chainages 9880 to 10090). 1D cross sections and 2D grid levels were updated based on as-built survey provided by NRC.

Additional works have also recently been constructed along downstream sections of the Awanui River, outside of the area of interest for this model. These changes have not been incorporated.

The earthworks extents are shown in **Figure 2.3**.

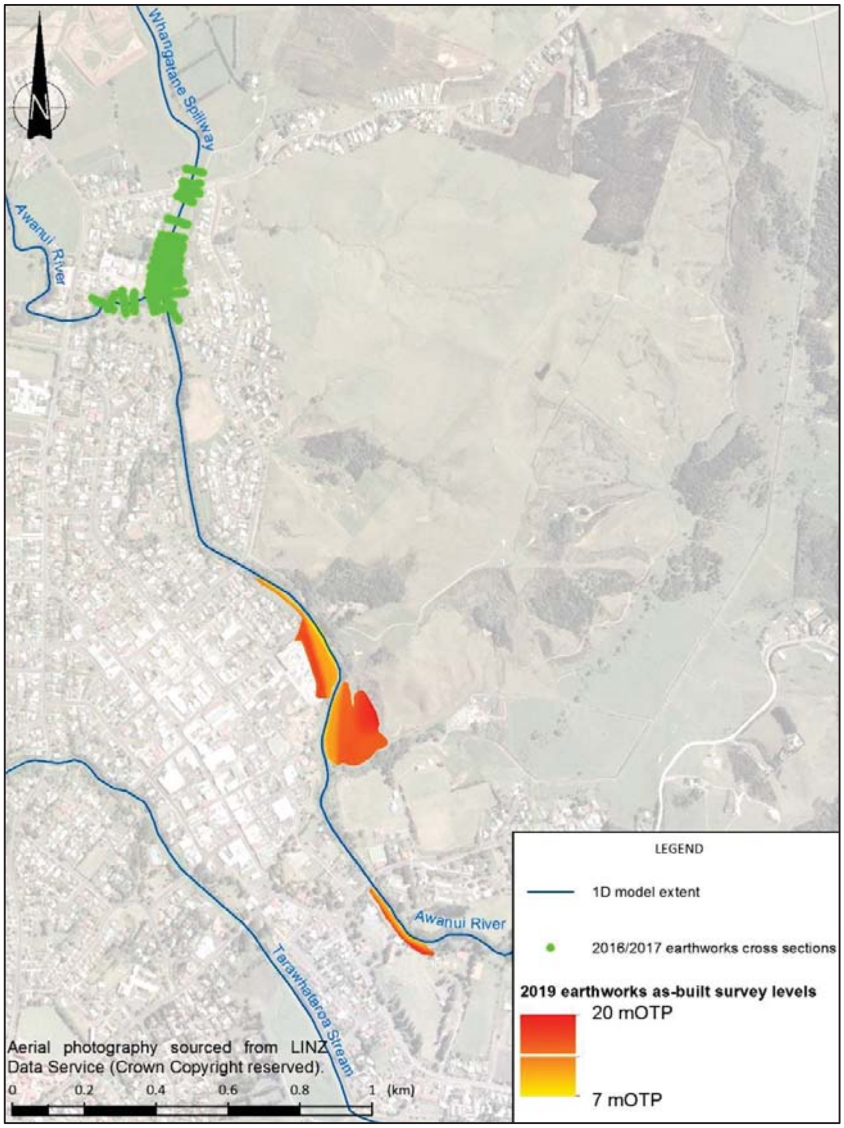

Figure 2.3: Areas of earthworks updates for Baseline scenario

In the model the weir at the entrance to the Whangatane Spillway became unstable when updated to match the modified topography. This was removed from the model and test runs indicated that there was negligible effect on the model results from the removal of this weir.

One additional modification to the topography was made due to the extension of the 2D domain. The level of the Oinu floodgate, which discharges into the Whangatane Spillway, was not captured by the 2018 DEM therefore grid levels were raised to 4.7 mOTP to represent this floodgate better.

2.4 Design model

2.4.1 Model overview

A combined design surface for the proposed scheme was provided by NRC on 4 December 2019. The proposed works include benching, spillways and stopbanks along the Awanui River and the Whangatane Spillway and are shown in **Figure 2.4**.

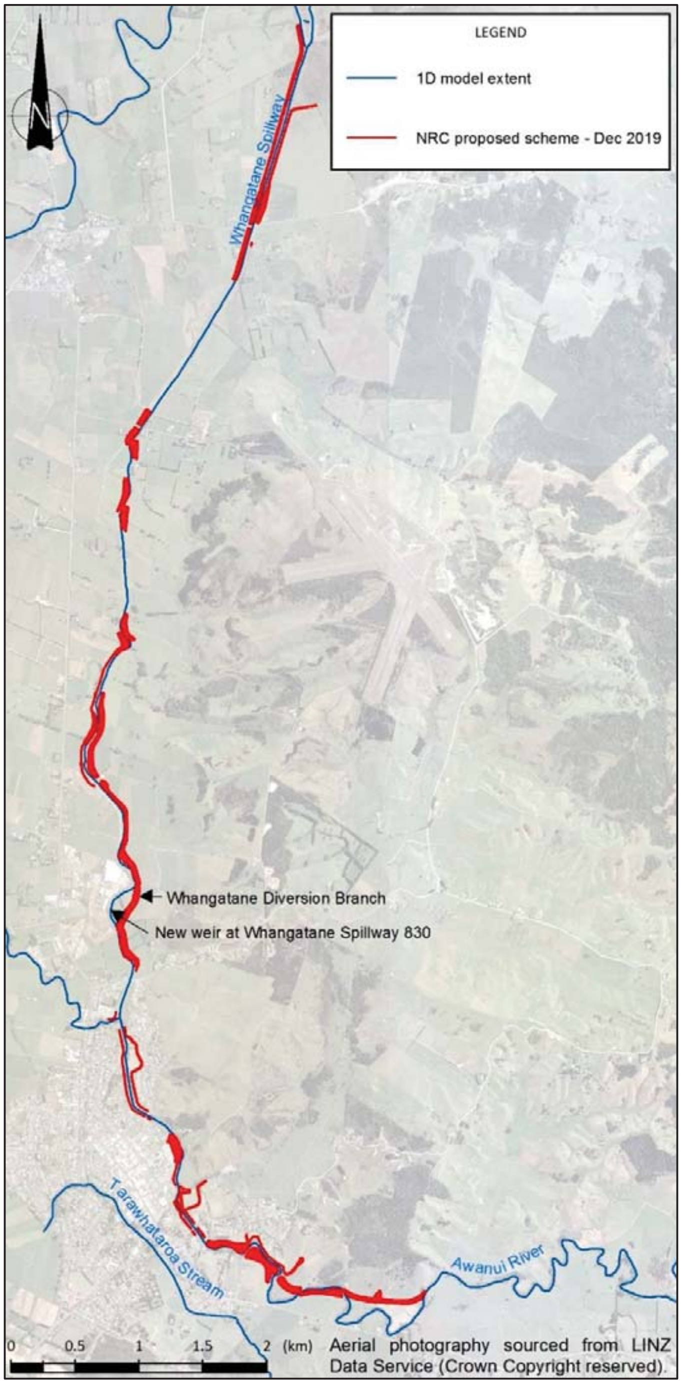

Figure 2.4: Proposed scheme footprint

2.4.2 Topography updates

Updated network and cross section files were provided by DHI on 4 December 2019 incorporating the design scheme provided by NRC. The updated cross sections were merged into the T+T Baseline model, along with the addition of the Whangatane Diversion Branch and the weir at Whangatane Spillway chainage 830.

The design surface was incorporated into the Baseline 2D model topography, and the 1D/2D lateral links were updated where required to match the updated stopbank locations.

2.4.3 Sandstone wedge removal

NRC also requested a variation on the design scenario to assess the impacts of removing the natural sandstone wedges (rock outcrop) within the channel towards the downstream end of the Whangatane Spillway, as shown in **Figure 2.5**. These were removed by interpolating bed levels between cross sections 9420, 9450, 9570, 9630.

The results demonstrate that there was only an insignificant, localised impact on water levels due to the removal of these wedges, as shown in **Figure 2.6**.

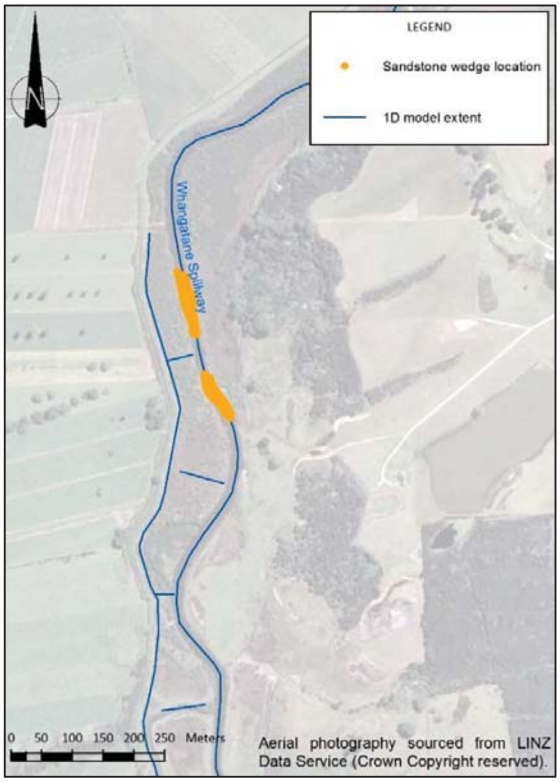

Figure 2.5: Sandstone wedge locations

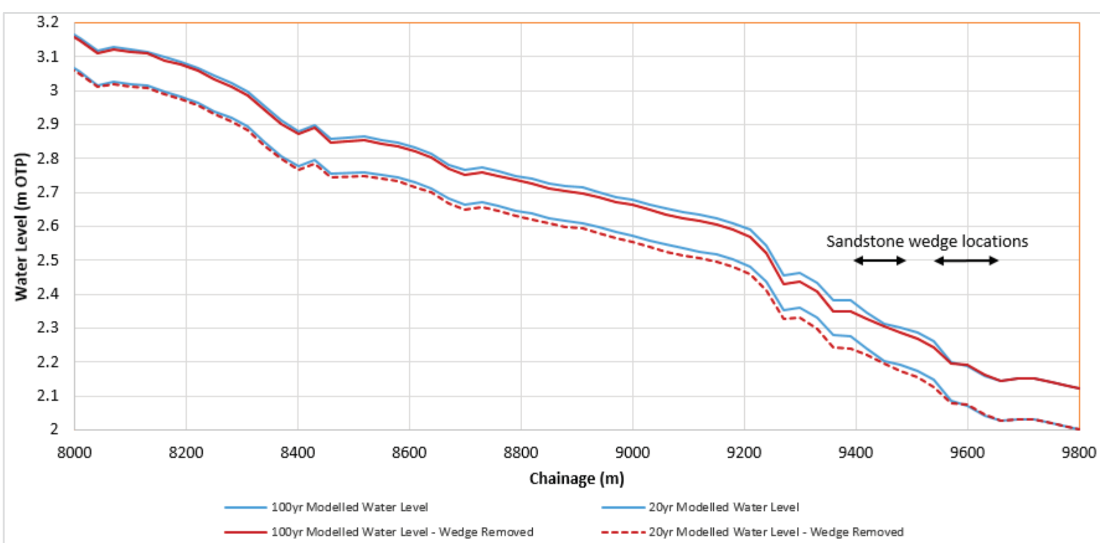

Figure 2.6: Longsection comparison along Whangatane Spillway with and without sandstone wedges

2.5 Model results

Maps showing the comparison between baseline and design scenario flood extents are provided in **Appendix A**.

Table 2.4 and **Table 2.5** summarise peak modelled flows, water levels and velocities at a number of key locations in the 20 year and 100 year ARI events for the Baseline and Design scenarios. **Table 2.6** provides the key thresholds when the spillways are activated for these scenarios.

Table 2.4: Model results for key locations in the 20 year ARI event

Location (with model branch and chainage)	Baseline scenario			Design scenario		
	Flow (m^3/s)	Water level (mOTP)	Velocity (m/s)	Flow (m^3/s)	Water level (mOTP)	Velocity (m/s)
Awanui at School Cut (Awanui 10030)	250	15.45	1.5	301	15.87	1.7
Church Road Bridge (Awanui 10130)	250	15.34	1.9	301	15.76	2.0
Allen Bell Drive Bridge (Awanui 11184)	249	14.43	1.6	301	14.78	1.9
SH1 Waikuruki/North Road Bridge (Awanui 12300)	75	12.91	0.8	85	13.10	0.8
Donald Road Bridge (Whangatane Spillway 276)	173	12.44	1.6	215	12.28	2.1
Quarry Road Bridge (Whangatane Spillway 5194)	167	5.05	1.2	202	5.39	1.3
SH10 Bridge (Whangatane Spillway 7022)	160	3.37	1.4	186	3.95	1.5
SH1 overflow	98	-	-	43	-	-
Puriri Place Bridge (Tarawha 6006)	109	15.49	1.5	53	14.31	1.1

Table 2.5: Model results for key locations in the 100 year ARI event

Location (with model branch and chainage)	Baseline scenario			Design scenario		
	Flow (m^3/s)	Water level (mOTP)	Velocity (m/s)	Flow (m^3/s)	Water level (mOTP)	Velocity (m/s)
Awanui at School Cut (Awanui 10030)	284	15.83	1.6	356	16.39	1.8
Church Road Bridge (Awanui 10130)	284	15.71	1.9	356	16.27	2.2
Allen Bell Drive Bridge (Awanui 11184)	281	14.74	1.7	354	15.31	2.2
SH1 Waikuruki/North Road Bridge (Awanui 12300)	83	13.07	0.8	100	13.33	0.9

Location (with model branch and chainage)	Baseline scenario			Design scenario		
	Flow (m³/s)	Water level (mOTP)	Velocity (m/s)	Flow (m³/s)	Water level (mOTP)	Velocity (m/s)
Donald Road Bridge (Whangatane Spillway 276)	192	12.59	1.8	247	12.66	2.3
Quarry Road Bridge (Whangatane Spillway 5194)	177	5.14	1.2	212	5.48	1.4
SH10 Bridge (Whangatane Spillway 7022)	167	3.39	1.4	191	4.04	1.6
SH1 overflow	204	-	-	133	-	-
Puriri Place Bridge (Tarawha 6006)	223	16.65	1.5	150	15.96	1.5

Table 2.6: Key thresholds when spillways are activated

Spill location	Baseline scenario – 100 year ARI		Design scenario – 100 year ARI	
	Level at School Cut (mOTP)	Flow at School Cut (m³/s)	Level at School Cut (mOTP)	Flow at School Cut (m³/s)
Spill over SH1 first occurs	14.67	193	14.92*	231
Whangatane Spillway starts to operate	9.86	14	9.88	14
Spillways upstream of Kaitaia start to operate	Spillway 1 (refer Figure 2.7)		13.47	126
	Spillway 4 (refer Figure 2.7)		12.43	78
	Spillway 5 (refer Figure 2.7)		11.95	61
	Spillway 6 (refer Figure 2.7)		14.27	180

*Other modelling for NRC shows lower flood levels at School Cut when the spill over SH1 first occurs. It is recommended that the difference in model results is investigated and reconciled.

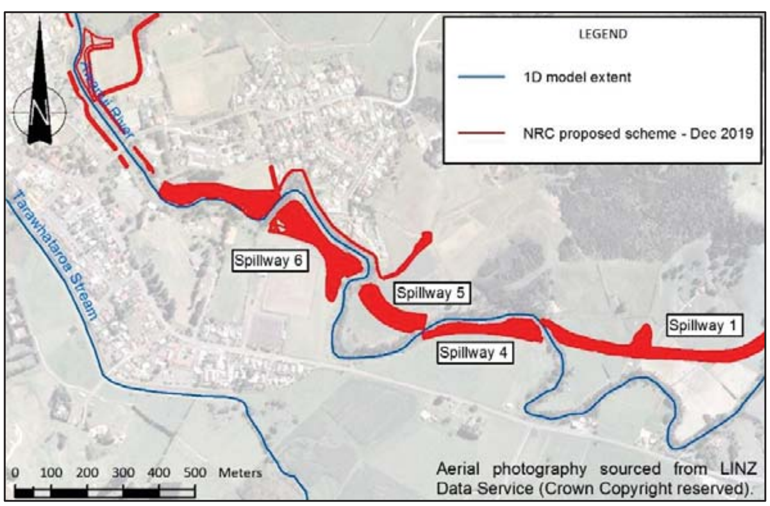

Figure 2.7: Proposed new spillways south of Kaitaia

2.6 Model log

Table 2.7 presents the simulation names for the final version of each model scenario.

Table 2.7: Model simulation names

Simulation name	Scenario
Awanui_Jan2011_071	Calibration scenario for the January 2011 event
Awanui_Feb2007_072	Calibration scenario for the February 2007 event
Awanui_July2007_085	Calibration scenario for the July 2007 event
Awanui_BLN_020yr_026	Baseline/existing scenario including recent works at Te Ahu and Bells Hill, 20 year ARI
Awanui_BLN_100yr_028	Baseline/existing scenario including recent works at Te Ahu and Bells Hill, 100 year ARI
Awanui_DSN_020yr_027	Design scenario provided by NRC December 2019, 20 year ARI
Awanui_DSN_100yr_029	Design scenario provided by NRC December 2019, 100 year ARI
Awanui_DSN_020yr_032	Design scenario provided by NRC December 2019, with sandstone wedge removal, 20 year ARI
Awanui_DSN_100yr_033	Design scenario provided by NRC December 2019, with sandstone wedge removal, 100 year ARI

3 Preliminary design

3.1 Overall objective

We understand the overall objectives of the scheme design are to:

- Provide adequate flood protection along the Awanui River from State Highway 1 overflow to the junction with Whangatane spillway channel and also along the Whangatane spillway channel
- Reduce flow across the SH1 into the Tarawhataroa Stream during the 20 year and 100 year design flood events
- Minimise private land acquisition as a result of scheme upgrade as far as reasonably practical
- Increase flood flow into the Whangatane spillway channel during large flood events (20 year or larger) by improving hydraulic efficiency of the channel.

The scope of the scheme design does not cover:

- Flood protection improvement works along and downstream of the Tarawhataroa Stream
- Flood protection improvement works along the Awanui River from North Road bridge to the estuary
- Improvement works on the true left bank of the Awanui River, opposite Bells Hill, which have already been completed
- Improvement works required to bridges and road crossings.

The Rangitaiki River Scheme Review Report supplied to Bay of Plenty Regional Council in September 2017, investigated flooding issues following the failure of the stopbank through Edgecumbe. The report made recommendations regarding:

- The legal and planning framework for flood hazard management
- The College Road floodwall
- Operation of Matahina Dam
- Reid's Floodway
- Evacuation planning
- Long-term strategy and design philosophies
- Community engagement.

Many of these were specific to the Rangitaiki Scheme and its operation. However, general recommendations as they relate to flood scheme design include:

- Widening the channel and "making room" for the river
- Diverting flood water away from high density residential or developed areas
- Having designated low points on the stopbanks for flows that exceed the design capacity of the scheme, i.e. a concept of controlled, compartment flooding.

Learnings taken from the Rangitaiki River Scheme Review report were applied to the preliminary design for the Awanui flood protection scheme where applicable. One example was the proposed creation of "benches" and "spillways" along the course of the floodway, to increase cross sectional area and improve hydraulic capacity of the channel.

Less stopbanking works are generally proposed along the true right bank of the Whangatane spillway channel, as the land on the true right bank requires less protection. Thus, in an extreme flood event it is expected that the land on the true right bank will be flooded sooner reducing flood loading on other parts of the flood protection scheme.

3.2 Design criteria

As discussed, and confirmed with NRC, the following design criteria were adopted for the Awanui River flood protection scheme:

- Freeboard criteria for flood protection:
  - Minimum 500 mm freeboard above estimated 100 year peak flood levels in urban areas
  - Minimum 300 mm freeboard above estimated 20 year peak flood levels in rural areas
  - Boundaries that separate urban and rural areas are shown in **Figure 3.1** below
- Where there is sufficient space for an earth stopbank, the stopbank is to have 3 m wide crest and 1V:2H batter slopes. The 3 m wide crest provides vehicle access if necessary as well as access for maintenance
- Where there is insufficient space for an earth stopbank, timber flood walls are proposed
- The reserve area adjacent to the A&P Showgrounds on the true right bank of the river can be allowed to flood and therefore does not need flood protection. The adjacent properties will still be protected
- An option that allows Remembrance Park to flood was previously selected as the preferred option by NRC and therefore the park does not need flood protection
- Where possible, benches are proposed upstream and/or downstream of bridges along the channel to increase channel capacity and reduce water level under the bridges during flood events
- The land on the true right bank of the Whangatane Spillway channel requires a lower degree of flood protection compared to the land on the true left bank.

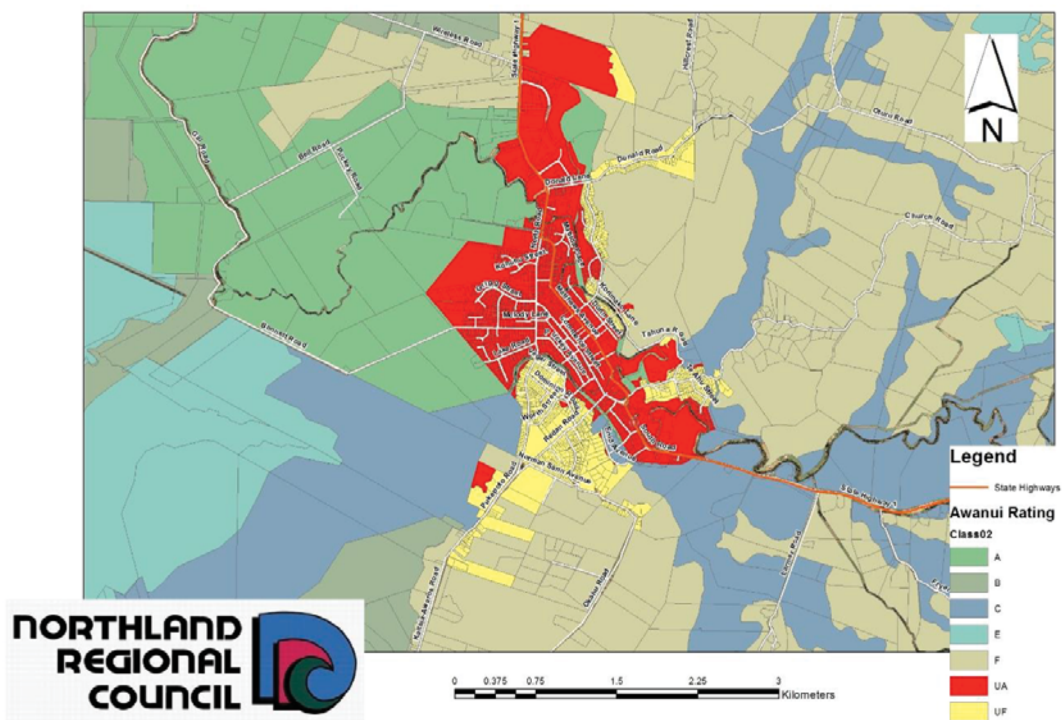

Figure 3.1: NRC map showing urban area in red

3.3 Scheme design

The scheme design aims to improve channel capacity of the flood protection scheme by carrying out the following works along the Awanui River through Kaitaia as well as along the Whangatane spillway channel:

- Adding spillways to increase channel capacity and straighten the channel
- Channel widening by lowering the river side berm and creating benches
- Raising stopbanks.

The proposed upgrade works for the flood protection scheme are shown on the digital terrain model issued separately to NRC in electronic format.

3.3.1 Awanui River through urban Kaitaia

There are four new spillways proposed along the Awanui River in the vicinity of the existing SH1 overflow. These spillways are shallow overflow channels which will normally be dry and are only activated during flood events. The spillways increase the channel capacity of the Awanui River by providing additional cross section area and reducing hydraulic head losses by straightening the channel. These in turn reduce the flow over SH1 during the 20 year and 100 year design flood events.

Throughout urban Kaitaia channel widening is also proposed by lowering the berm area on the river side of the existing stopbank, thereby creating “benches” along the channel. Benches are proposed at the following locations:

- On the true left bank, opposite Rongopai Place
- The reserve on the true right bank next to the A&P Showground
- Remembrance Park on the true right bank.

Improvement works on the true left bank of the river opposite Bells Hill were designed separately and do not form part of this project but have been included in the model.

Space limitation is the biggest constraint for upgrading the flood protection scheme throughout urban Kaitaia, similar to other flood schemes in urban areas in New Zealand. Flood walls are therefore proposed in locations where space limitation is an issue.

No further improvement works are proposed along Matthews Park as well as at Whangatane Spillway intake, as works have already been constructed by NRC at this location.

3.3.2 Whangatane Spillway channel

Significant benching works are proposed along the Whangatane Spillway channel to improve hydraulic efficiency, particularly along the true right bank of the channel. Observations from a site visit in 2019 by NRC staff helped inform the locations of these benches.

Where possible, benches have been proposed upstream and downstream of bridges with the aim to reduce flood level and flow velocity at the bridge locations.

A spillway is also proposed on the true right bank adjacent to the Juken Mill, just North of Kaitaia, to increase hydraulic capacity of the channel in this location.

The benches have a nominal slope of 2.5% towards the river. The benches are created so that the top levels of the benches are approximately 1.5 m above the channel invert. The intention is to maximise the hydraulic capacity of the channel while maintaining the flow in the stream channel during smaller flood events.

4 Bridge waterways

The hydraulics of the bridge waterways have been assessed to determine flood freeboard and provide preliminary design details for scour protection. The following bridges have been assessed:

- the Whangatane Spillway bridges
  - the Donald Road Bridge (model chainage 275.5)
  - the Quarry Road Bridge (model chainage 5194)
  - State Highway 10 (SH10) Bridge (model chainage 7022)
- the three bridges on the Awanui River
  - Church Road Bridge (model chainage 10130)
  - Allen Bell Drive Bridge (model chainage 11184)
  - SH1 Waikuruki/ North Road Bridge (model chainage 12300).

4.1 Hydraulics

The Bridge Manual recommends minimum 0.6 m clearance from the design flood stage to the underside of the bridge superstructure (minimum 1.2 m clearance where there is the possibility that large trees may be carried down the waterway).

4.1.1 Whangatane Spillway bridges

The following data is derived from flood model as described in **Section 2**.

Table 4.1: Summary of discharges, flood levels and velocities at Whangatane Spillway bridge crossings

Models/ flood events	Donald Road Bridge (Whangatane Spillway 275.5)			Quarry Road Bridge (Whangatane Spillway 5194)			SH10 Bridge (Whangatane Spillway 7022)		
	Flow (m^3/s)	Level (mOTP)	Velocity (m/s)	Flow (m^3/s)	Level (mOTP)	Velocity (m/s)	Flow (m^3/s)	Level (mOTP)	Velocity (m/s)
20Yr-Baseline	173	12.44	1.6	167	5.05	1.2	160	3.37	1.4
100Yr-Baseline	192	12.59	1.8	177	5.14	1.2	167	3.39	1.4
20Yr- Scheme Design	215	12.28	2.1	202	5.39	1.3	186	3.95	1.5
100Yr-Scheme Design	247	12.66	2.3	212	5.48	1.4	191	4.04	1.6

Table 4.2: Summary of available freeboard at Whangatane Spillway bridge crossings (negative values indicate water level above the bridge soffit)

Models/ flood events	Donald Road Bridge (Whangatane Spillway 275.5)			Quarry Road Bridge Whangatane Spillway 5194)			SH10 Bridge (Whangatane Spillway 7022)		
	Soffit (mOTP)	Deck (mOTP)	Freeboard (m)	Soffit (mOTP)	Deck (mOTP)	Freeboard (m)	Soffit (mOTP)	Deck (mOTP)	Freeboard (m)
20Yr- Baseline	12.30	13.26	-0.14	5.28	5.72	0.24	3.69	4.81	0.32
100Yr- Baseline	12.30	13.26	-0.29	5.28	5.72	0.15	3.69	4.81	0.29
20Yr- Scheme Design	12.30	13.26	0.02	5.28	5.72	-0.11	3.69	4.81	-0.27
100Yr- Scheme Design	12.30	13.26	-0.35	5.28	5.72	-0.20	3.69	4.81	-0.36

The model results show that there is not adequate freeboard at any of the Whangatane Spillway bridges to meet Bridge Manual recommended minimum clearance.

4.1.2 Awanui River bridges

The following data are derived from flood model as described in **Section 2**.

Table 4.3: Summary of discharges, flood levels and velocities at Awanui River bridge crossings

Models/ flood events	Church Road Bridge (Awanui 10130)			Allen Bell Drive Bridge (Awanui 11184)			SH1 Waikuruki Bridge (Awanui 12300)		
	Flow (m^3/s)	Level (mOTP)	Velocity (m/s)	Flow (m^3/s)	Level (mOTP)	Velocity (m/s)	Flow (m^3/s)	Level (mOTP)	Velocity (m/s)
20Yr- Baseline	250	15.34	1.9	249	14.43	1.6	75	12.91	0.8
100Yr- Baseline	284	15.71	1.9	281	14.74	1.7	83	13.07	0.8
20Yr- Scheme Design	301	15.76	2.0	301	14.78	1.9	85	13.10	0.8
100Yr- Scheme Design	356	16.27	2.2	354	15.31	2.2	100	13.33	0.9

Table 4.4: Summary of available freeboard at Awanui River bridge crossings (negative values indicate WL above the bridge soffit level)

Models/ flood events	Church Road Bridge (Awanui 10130)			Allen Bell Drive Bridge (Awanui 11184)			SH1 Waikuruki Bridge (Awanui 12300)		
	Soffit (mOTP)	Deck (mOTP)	Freeboard (m)	Soffit (mOTP)	Deck (mOTP)	Freeboard (m)	Soffit (mOTP)	Deck (mOTP)	Freeboard (m)
20Yr-Baseline	16.43	17.54	1.09	14.36	15.74	-0.07	13.29	14.08	0.38
100Yr-Baseline	16.43	17.54	0.72	14.36	15.74	-0.38	13.29	14.08	0.22
20Yr-Scheme Design	16.43	17.54	0.67	14.36	15.74	-0.42	13.29	14.08	0.19
100Yr-Scheme Design	16.43	17.54	0.16	14.36	15.74	-0.95	13.29	14.08	-0.04

The model results also show that none of the bridges over the Awanui River meet Bridge Manual recommended minimum clearance.

4.2 Preliminary bridge scour assessment

To assess the potential scour effects of the proposed scheme upgrade on bridges, a preliminary desktop scour assessment was carried out.

The methodology adopted for the scour assessment is in line with the New Zealand Transport Agency Bridge Manual 3rd Edition (2018) (referred to as *Bridge Manual* henceforth) which refers to the monograph Bridge Scour (Melville and Coleman, 2000) (referred to as *Bridge Scour* henceforth).

In line with the scope as desktop based preliminary scour assessment, long term geomorphic trends were not considered. This assessment considers only short-term general scour, contraction scour and local scour which are likely to be the primary scour hazards for these bridges.

The assessment was based on input parameters from the hydraulic modelling and the following information provided by NRC:

- Site photos (unknown times and dates)
- 2007 flood video footage
- Bridge sketches.

It is recommended that the conclusions of this assessment are confirmed by an on-site inspection and a geotechnical assessment of embankment stability at each bridge site, and a specific assessment of bed morphological trends.

This assessment excludes analysis of bridge structural and geotechnical stability under design flood and scour loads. A bridge structural engineering assessment is required to draw conclusions on bridge stability, particularly due to scour around piers in flood conditions.

The assessment provides key findings and recommendations for providing the scour protection (refer to **Figure 4.1** below for a typical bridge scoured section).

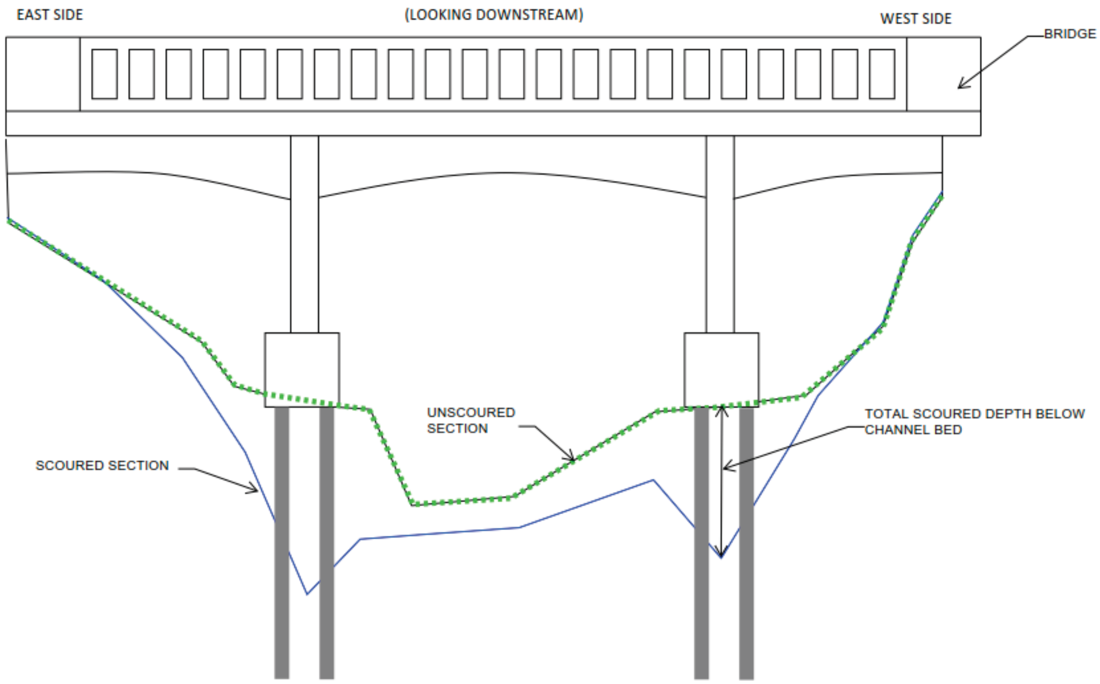

Figure 4.1: Typical bridge scoured section

4.2.1 Approach

Following the guidance in the *Bridge Manual* and *Bridge Scour*, total scour depths are calculated by calculating the general and contraction scour depths, adjusting the bridge hydraulics for the enlarged scoured section, and then calculating local scour. The total scour depth is the sum of the general, contraction and local scour effects. The effects of debris loading where relevant are calculated by assuming debris raft dimensions in line with *Bridge Manual* recommendations, and calculating an equivalent pier width, and adjusting the pier scour calculations using this equivalent width. Refer to **Figure 6.1** and **Figure 6.2** in **Appendix B** for further details.

For this assessment, short term general scour and contraction scour has been calculated using the New Zealand Railways (Holmes, 1974) method as reproduced in *Bridge Scour* (refer to **Figure 6.3** in **Appendix B** for further details). This method was selected as bed material distribution data were not available for the assessment. We have applied a 20 % increase to the calculated scour depth, given there is no safety factor included in this method. This method can in some cases produce conservative scour depths and should be reviewed in the detailed design stage. *Bridge Scour* recommends considering a range of methods to determine general scour, however, detailed data as recommended in **Section 4.2** are required for these methods. On site observations, and measurements would aid engineering judgement and would provide a basis for a robust assessment of a general and contraction scour.

Local pier scour was calculated using the Melville (1997) method recommended in *Bridge Scour* (refer to **Appendix B** Section B4 for further details). The sediment size factor used in the Melville method uses bed particle size as an input. No particle size distribution data were provided for this assessment. To test the sensitivity of the sediment size factor to particle sizes we assumed that the bridges channel bed material is fine-grained, Mangakahia clay loam with a mean particle size (d_{50}) of 0.06 mm (based on the NRC soil maps). This small particle sizes results in a sediment size factor of 1 (the maximum and most conservative value) therefore it was concluded that the calculated local scour depths are not sensitive to this factor.

The effect of debris on bridge piers under Church Road and Allen Bell Drive has not been assessed due to piers being anchored in the embankments (see **Figure 4.5** and **Figure 4.6**). It was assumed that any debris load in the flow would pass in the main channel without significant accumulation around the piers.

4.2.2 Whangatane Spillway bridges

We have reviewed the data provided by NRC, i.e. the site photographs and the 2007 flood footage to assess bridge environment. We have concluded that the embankments appeared to be well vegetated and in good condition, however this assessment must not be a basis for a stability assessment without a geotechnical investigation as recommended in **Section 4.2**.

The summary of the total scoured depths below the channel bed around pier locations for the Spillway bridges is shown in **Table 4.6**.

4.2.2.1 Donald Road Bridge

The bridge superstructure is supported on three piers, each consisting of four circular wooden piers. The pier diameters have not been surveyed, and were assumed to be 0.5 m.

Figure 4.2: Donald Lane Bridge

4.2.2.2 Quarry Road Bridge

The bridge superstructure is supported on three downwards tapering concrete piers with slab footings. The pier dimensions have not been surveyed and were assumed to have an average pier width to be 0.5 m.

Figure 4.3: Quarry Road Bridge

4.2.2.3 State Highway 10 Bridge

The bridge superstructure is supported on two rectangular concrete piers with slab footings. The pier dimensions have been surveyed and are 0.3 m wide with a 1.2 m wide slab footing.

The Holmes method indicates that no short-term general scour and contraction scour would occur. The Holmes method does not include the impact of bed material size and due to lack of the channel bed material data, we were unable to consider other methods to compare the results. It is likely that some short-term general scour and contraction scour would occur during a flood event. We recommend further site investigations as specified in **Section 4.2** are undertaken to provide additional data inputs and confirm our assumptions.

Figure 4.4: State Highway 10 Bridge

4.2.3 Awanui River Bridges

We have reviewed the data provided by NRC which included site photographs and video footage of the 2007 flood to assess the bridge environment. The photographs and footage show that the embankments appeared to be well vegetated and in good condition. However, these observations do not comprise a stability assessment, and we recommend a geotechnical investigation as indicated in **Section 4.2**.

The summary of the total scoured depths below the channel bed around pier locations for the Awanui River bridges is shown in **Table 4.5**.

4.2.3.1 Church Road Bridge and Allen Bell Drive Bridge

Each bridge superstructure is supported on two angled concrete piers. The piers appear to be set back and anchored in the embankment with the bottom of the footing above the normal water level (**Figure 4.5** and **Figure 4.6**). For this reason and that the approach velocities are low (2.2 m/s in the 100 year ARI flood) we have assumed that the bridge is not at risk from local pier scour.

We have used the New Zealand Railways (Holmes, 1974) method to calculate the short term general scour and contraction scour depths, as described in **Section 4.2.2**.

For bridge cross sections refer to **Figure 4.5** and **Figure 4.6** below.

Figure 4.5: Church Road Bridge

Figure 4.6: Allen Bell Drive Bridge

4.2.3.2 SH1 Waikuruki/ North Road Bridge

The bridge superstructure is supported on two rectangular concrete piers with slab footings. The pier dimensions have not been surveyed, we assumed a 0.3 m wide pier with a 0.7 m wide slab footing for the calculations.

The Holmes method indicates that no short-term general scour and contraction scour would occur for either 100 year ARI or 20 year ARI flood event. Considering that the method does not include any bed material effects and due to lack of the channel bed material data we are unable to consider other methods to compare the results. It would be prudent though to assume that some general and contraction scour occurs during a flood even, as based off **Figure 4.7** it appears there is some evidence of bed lowering as the piles of one of the piers are exposed. We recommend further site investigations as specified in **Section 4.2** to confirm our assumptions.

Figure 4.7: SH1 Waikuruki/North Road Bridge

4.2.4 Estimated scour depths

Table 4.5 and **Table 4.6** show calculated scour depths for the Awanui River and Whangatane Spillway bridges, respectively.

Table 4.5: Summary of scoured depths below channel bed around a pier location (for the Awanui River in Baseline scenario and Scheme Design)

Scour types	ARI	Awanui River-Baseline			Awanui River - Scheme Design		
		Church Road Bridge (m)	Allen Bell Drive Bridge (m)	SH1 Waikuruki Bridge (m)	Church Road Bridge (m)	Allen Bell Drive Bridge (m)	SH1 Waikuruki Bridge (m)
Short term general and contraction scour	20 years	0.52	0	0	0.43	0	0
	100 years	0.28	0	0	0.10	0.42	0
Local scour/pier scour	20 years	-	-	3.39	-	-	3.29
	100 years	-	-	3.30	-	-	3.22

Scour types	ARI	Awanui River-Baseline			Awanui River - Scheme Design		
		Church Road Bridge (m)	Allen Bell Drive Bridge (m)	SH1 Waikuruki Bridge (m)	Church Road Bridge (m)	Allen Bell Drive Bridge (m)	SH1 Waikuruki Bridge (m)
Total scour depth (Tds) *	**20 years**	**0.52**	**0**	**3.39**	**0.43**	**0**	**3.29**
	100 years	**0.28**	**0**	**3.30**	**0.10**	**0.42**	**3.22**

*Refer discussions in paragraphs below

Table 4.6: Summary of scoured depths below channel bed around a pier location (for the Whangatane Spillway in Baseline scenario and Scheme Design)

Scour types	ARI	Whangatane Spillway - Baseline			Whangatane Spillway - Scheme Design		
		Donald Road Bridge (m)	Quarry Road Bridge (m)	SH10 Bridge (m)	Donald Road Bridge (m)	Quarry Road Bridge (m)	SH10 Bridge (m)
Short term general and contraction scour	20 years	1.66	0.99	0	2.36	1.59	0
	100 years	2.20	1.08	0	3.40	1.84	0
Local scour/pier scour	20 years	2.62	3.38	2.86	2.44	3.11	2.77
	100 years	2.48	3.32	2.84	2.24	3.03	2.77
Total scour depth (Tds) *	**20 years**	**4.28**	**4.37**	**2.86**	**4.80**	**4.70**	**2.77**
	100 years	**4.68**	**4.39**	**2.84**	**5.64**	**4.87**	**2.77**

*Refer discussions in paragraphs below

For the SH10 Bridge the calculated scour depths shown in **Table 4.6** indicate that the total scoured depth in the Scheme Design is similar for both the 20 year ARI and 100 year ARI events. The likely reasons for this are the small differences in the predicted flood levels, flow rates and flow velocities for both the 20 year ARI and 100 year ARI flood events. We recommend considering other methods for estimating total scoured depth once detailed data as specified in **Section 4.2** is available.

The Holmes method indicates that no short-term general scour and contraction scour occur for either 20 year ARI or 100 year ARI flood event for the Allen Bell Drive, SH1 Waikuruki/ North Road Bridge and SH10 Bridges. Considering that the method does not include any bed material effects and due to lack of the channel bed material data we are unable to consider other methods to compare the results. It would be prudent though to assume that some general and contraction scour occurs during a flood event. We recommend further site investigations as specified in **Section 4.2.5** to confirm our assumptions.

For the Church Road Bridge calculated scour depths (shown in **Table 4.5**), the Holmes equation (refer to **Appendix B** Section B3 for the equation formulae) used to calculate the general and contraction scour depth produced a lower estimate of scour for the 100 year ARI event than the 20 year ARI event. This result can be produced by this equation for cross sections where flow width does not increase significantly with flow depth. These depths should be taken as a rough indication and is close to what we would consider the margin of error for this method. If additional information at the site was available, such as bed material particle size distribution, this depth could be checked with alternative methods.

Similarly, the pier scour formula from Bridge Scour used is dependent on an equivalent pier width calculated using the NZTA Bridge Manual debris raft formula. This is sensitive to flow depth and can produce higher equivalent pier widths for lower flows than higher flows. This can result in higher predicted pier scour depths for lower flow rates. Similar to general scour depths, pier scour depths can be revisited (and potentially reduced) if additional information such as bed material particle size distribution is known.

4.2.5 Recommendations

We recommend the following additional scour design checks:

- Field inspection to assess channel stability, bed material type and susceptibility to erosion, bank vegetation type and potential for debris accumulation around the piers
- Geotechnical investigations at river embankments to check for bank slope stability and susceptibility to erosion
  - If bank slope stability is critical, toe buttressing could be provided which may also require rock riprap
- Geomorphic trend analysis to assess the potential for long or short term bed degradation
- Bridge pier and foundation survey
- Structural assessment of the existing bridge piers to confirm general condition and depth to assess their ability to withstand scour

Subject to a more detailed analysis including site investigations, and a bridge stability analysis, piers may be protected from scour by the installation of riprap aprons around the piers. We have calculated a preliminary riprap size (**Table 4.7**) using the Lauchlan (1999) method as reproduced in *Bridge Scour* for the bridges which have piers in the main channel (SH10, SH1 Waikuriki, Quarry Road, and Donald Road). This rock should be placed in an apron around the bridge piers as shown in **Figure 4-8** below.

Table 4.7: Preliminary riprap median size for 100 year ARI event

Bridge	d50 (mm)*
SH1 Waikuruki/North Road Bridge	120
Donald Road Bridge	340
Quarry Road Bridge	190
SH10 Bridge	230

* Based on a rock density of SG = 2.65, and safety factor = 1.1

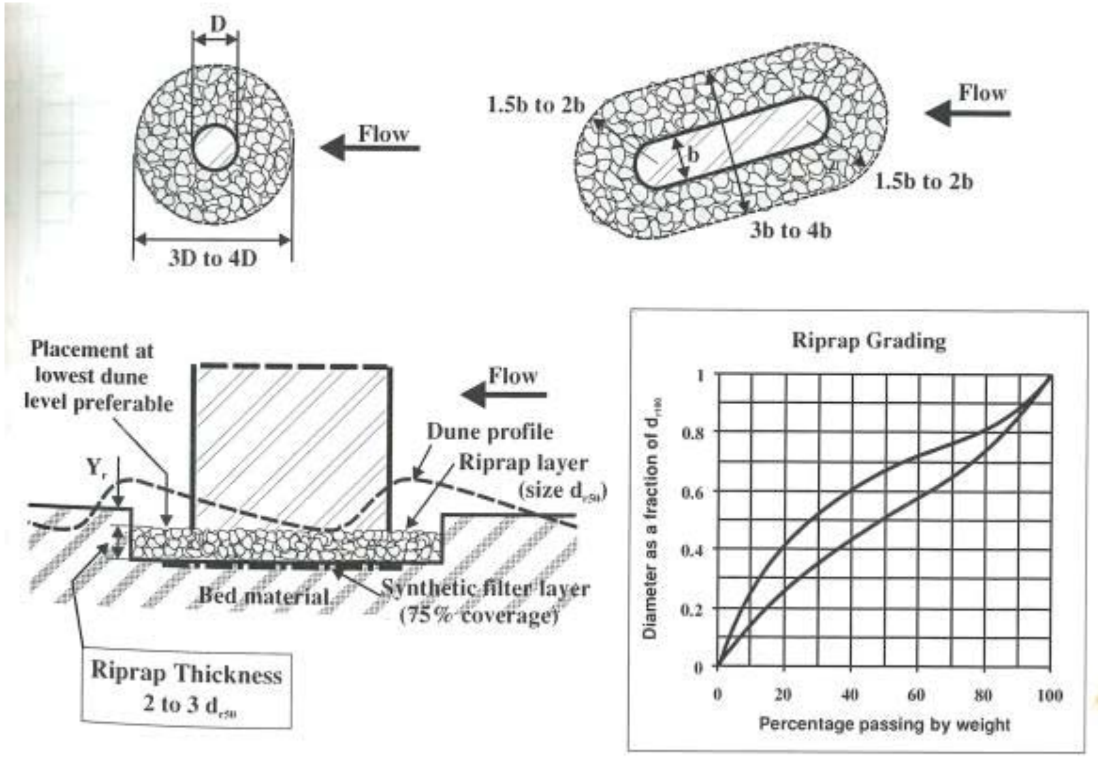

Figure 4-8: Riprap placement details (Reproduced from Figure 9.33 in Bridge Scour)

5 References

Auckland Council (2011). Stormwater Flood Modelling Specifications, November 2011.

Melville, B. W. and Coleman, S. E. (2000). *Bridge Scour*. Water Resources Publications.

NZ Transport Agency Bridge Manual SP/M/022, Third edition, Amendment 3 (2018)

6 Applicability

This report has been prepared for the exclusive use of our client Northland Regional 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.

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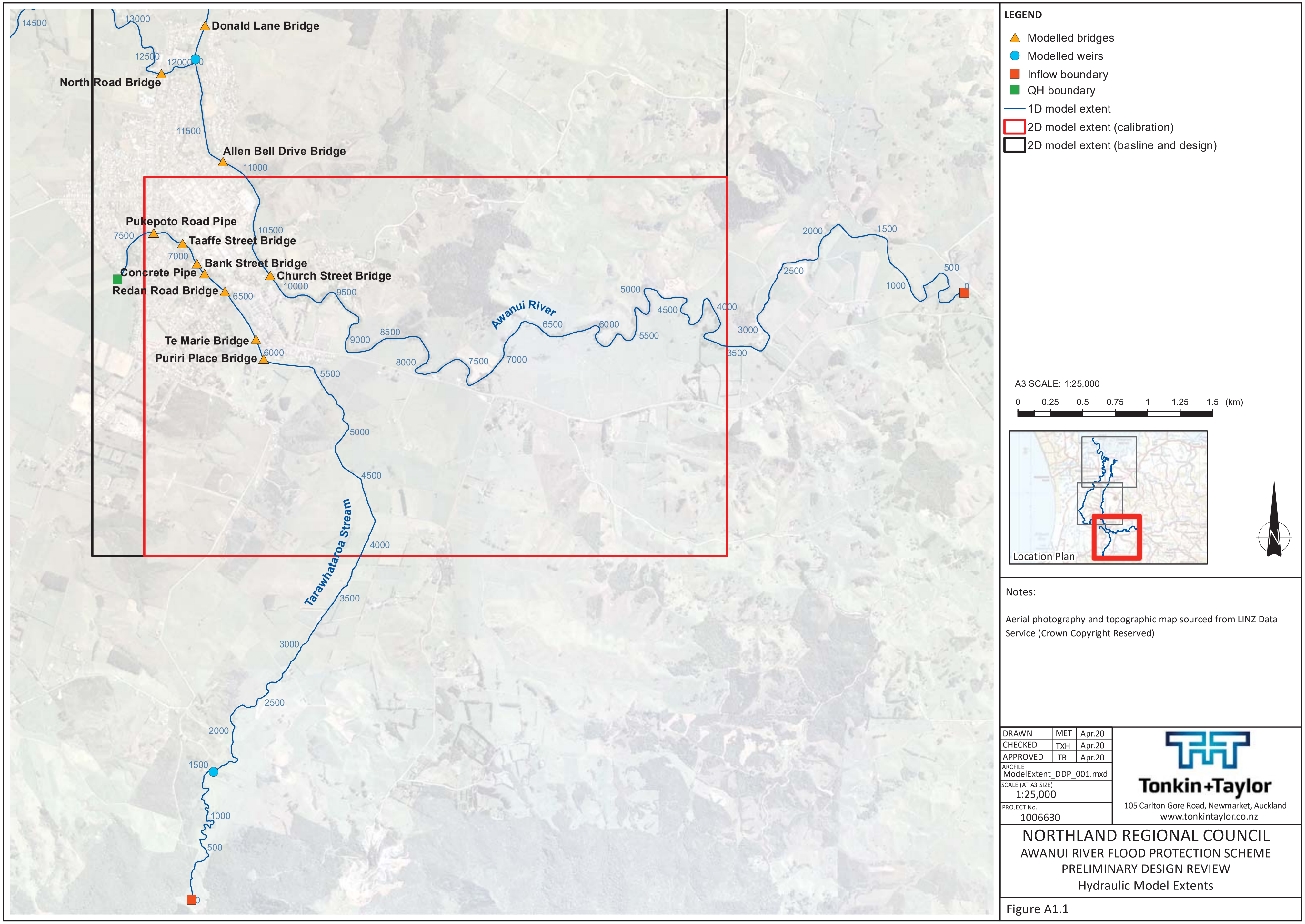
Appendix A: Hydraulic model figures

- **A1: Hydraulic model extents**
- **A2: Calibrations event gauge flows and water levels**
- **A3: Calibration event longsections**
- **A4: Baseline and design scenario flood extents – 20 year**
- **A5: Baseline and design scenario flood extents – 100 year**

A1 Hydraulic model extents



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Aerial photography and topographic map sourced from LINZ Data Service (Crown Copyright Reserved)

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APPROVED	TB	Apr.20
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SCALE (AT A3 SIZE) 1:25,000		
PROJECT No. 1006630		

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PRELIMINARY DESIGN REVIEW
Hydraulic Model Extents

Figure A1.1

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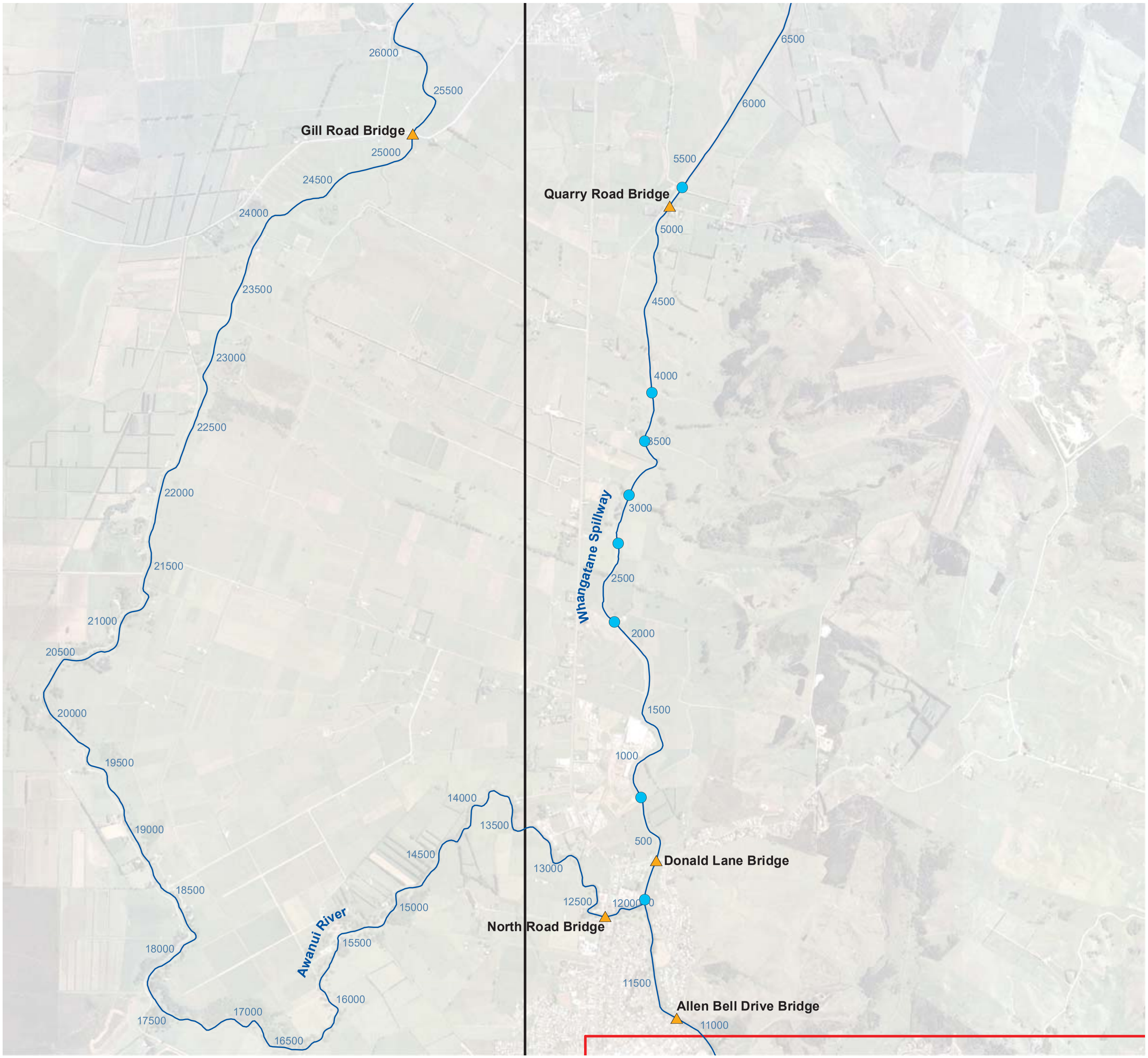

LEGEND

- Modelled bridges
- Modelled weirs
- 1D model extent
- 2D model extent (calibration)
- 2D model extent (basline and design)

A3 SCALE: 1:25,000

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Notes:

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Hydraulic Model Extents

Figure A1.2

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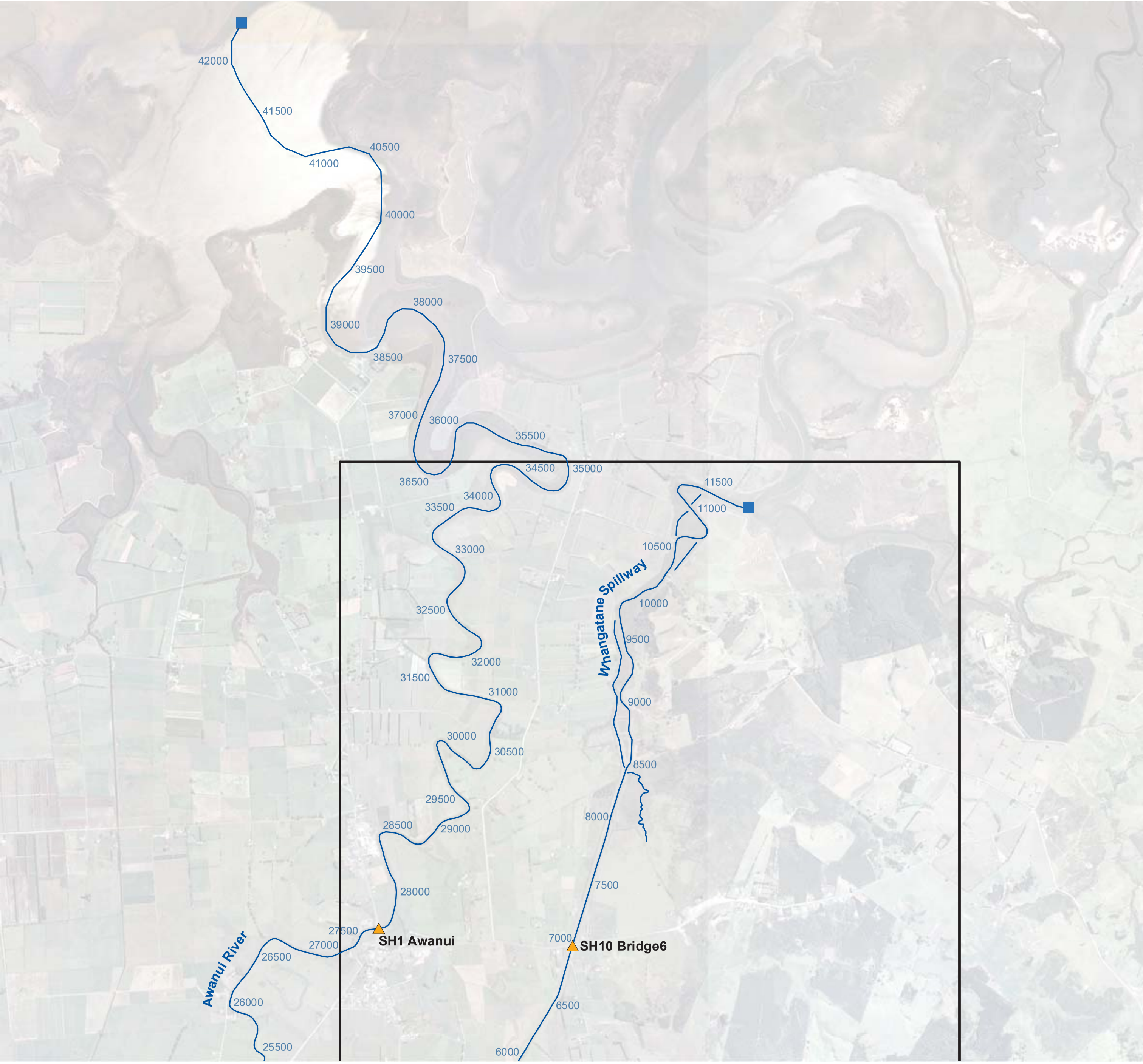

LEGEND

- Modelled bridges
- Tidal boundary
- 1D model extent
- 2D model extent (basline and design)

A3 SCALE: 1:30,000

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Location Plan

Notes:

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APPROVED	TB	Apr.20
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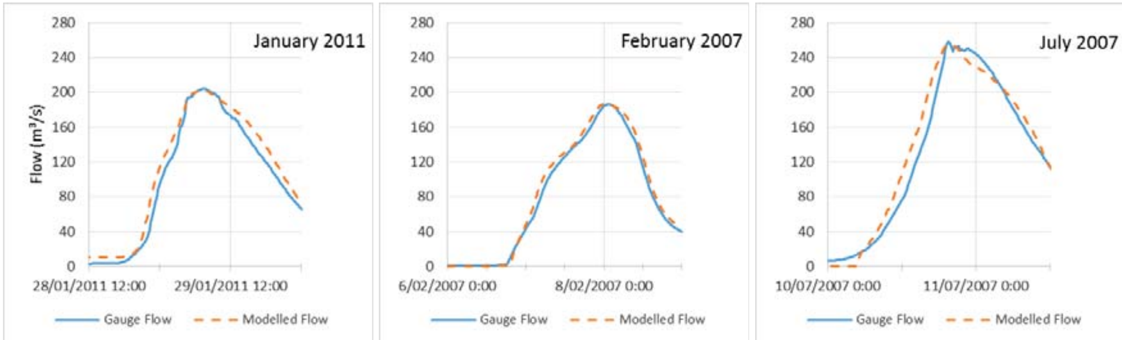
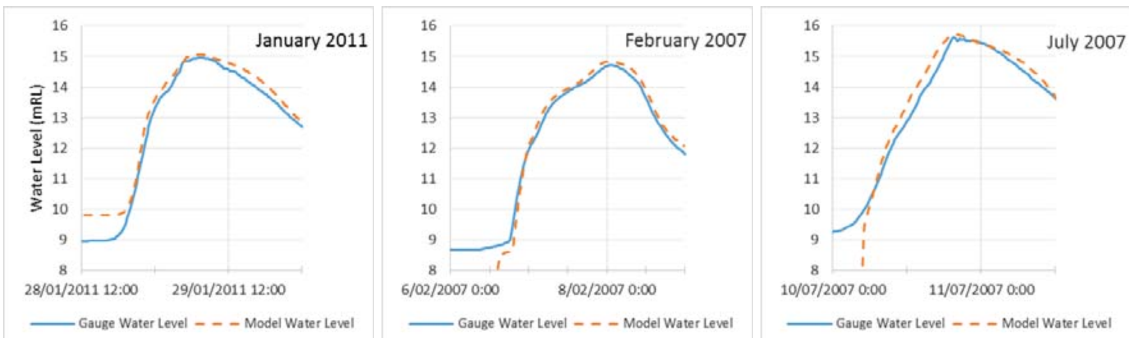
Figure A1.3

A2 Calibration event gauge flows and water levels

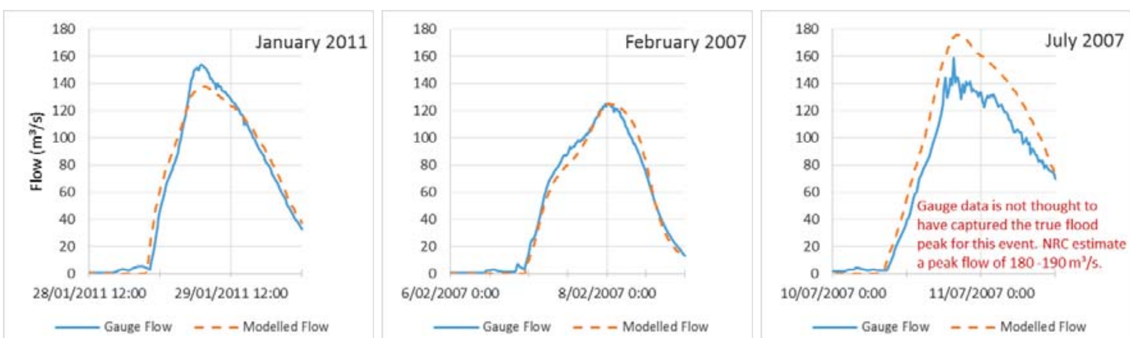
Awanui at School Cut flow comparison



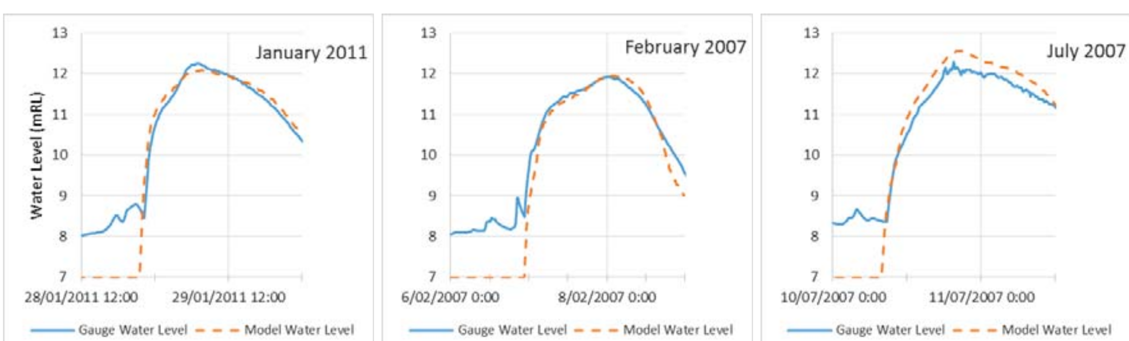
Awanui at School Cut water level comparison



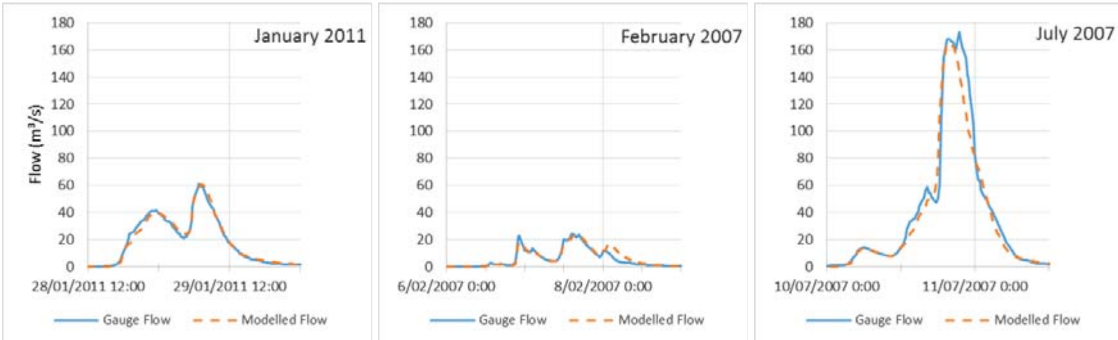
Whangatane Spillway at Donald Road flow comparison



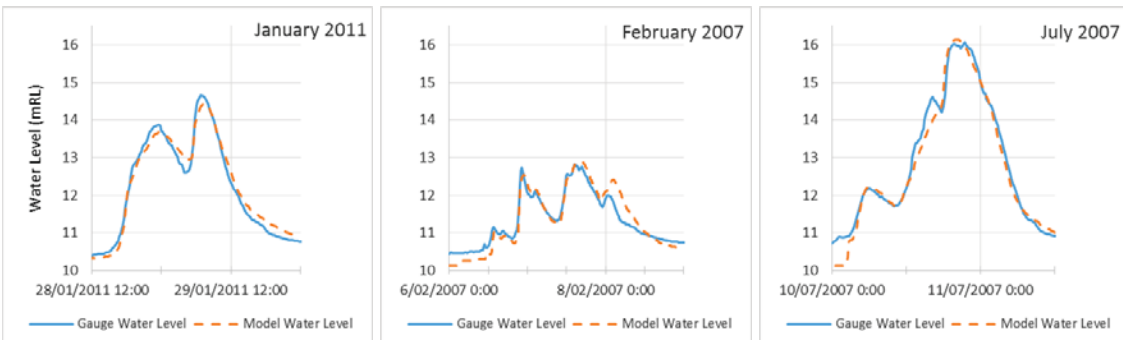
Whangatane Spillway at Donald Road water level comparison



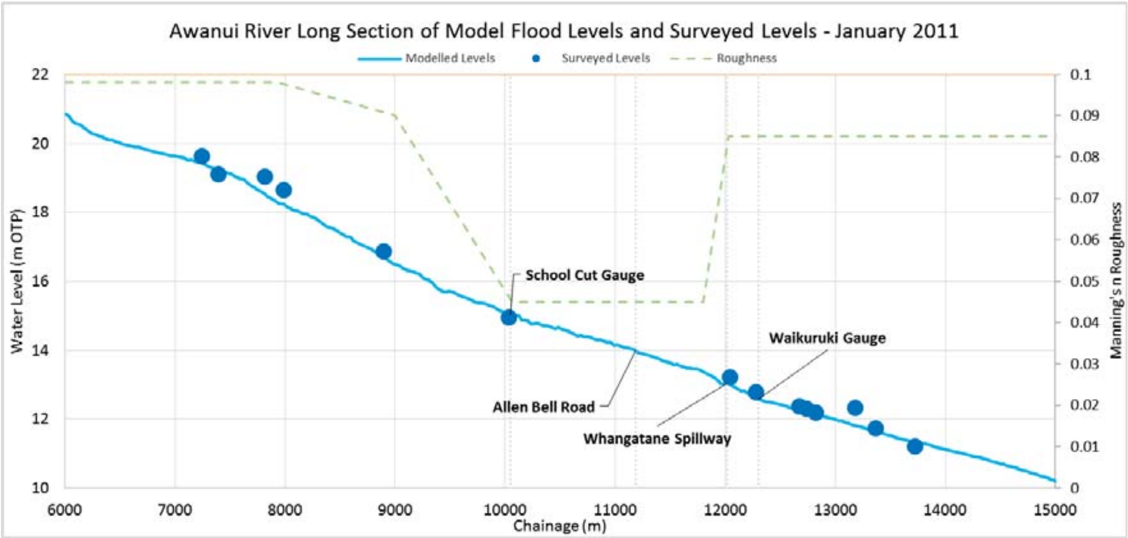
Tarawhataroa at Puriri Place flow comparison

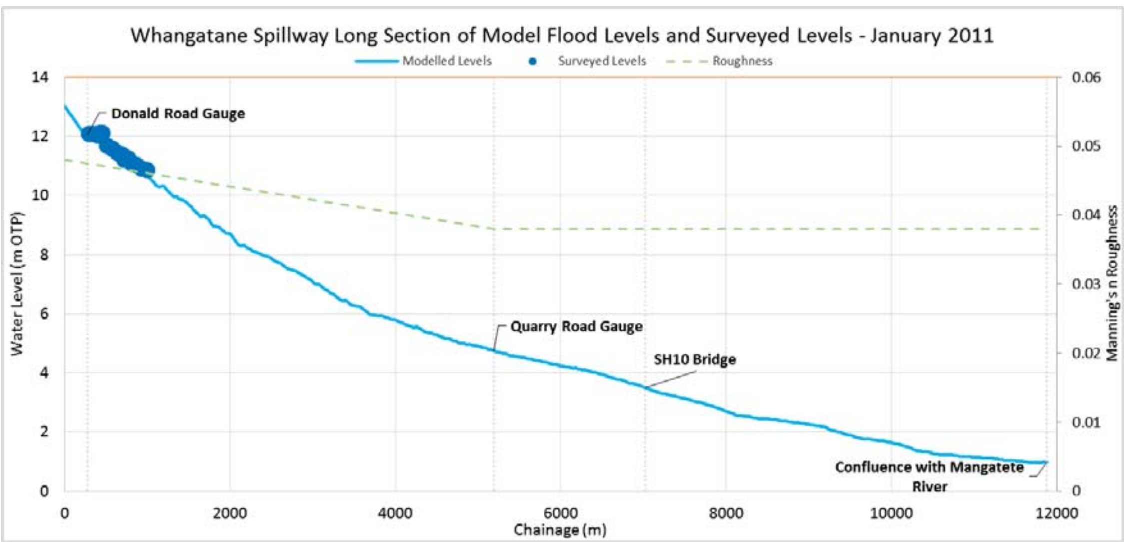


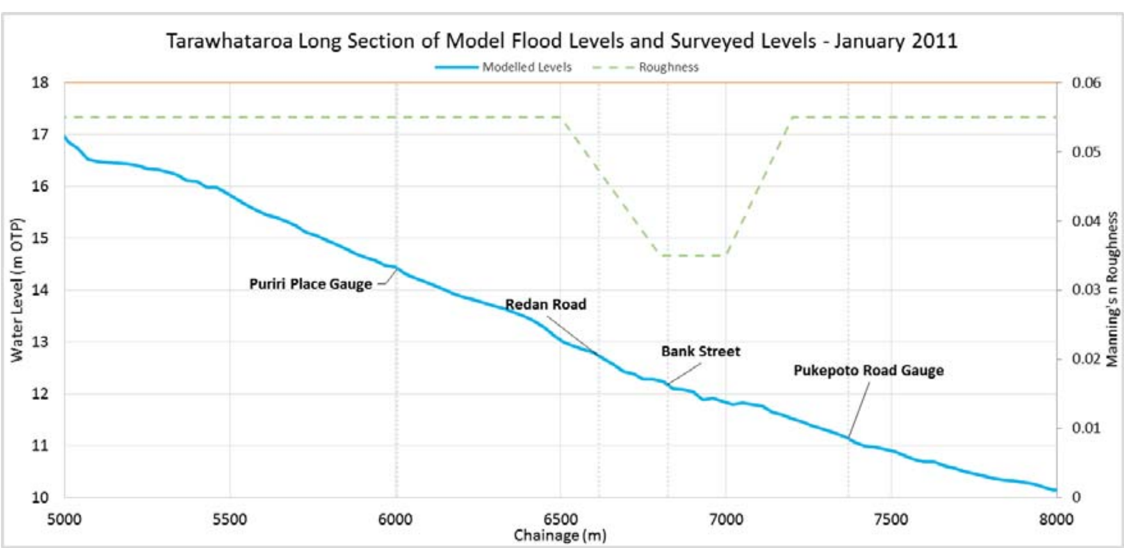
Tarawhataroa at Puriri Place water level comparison

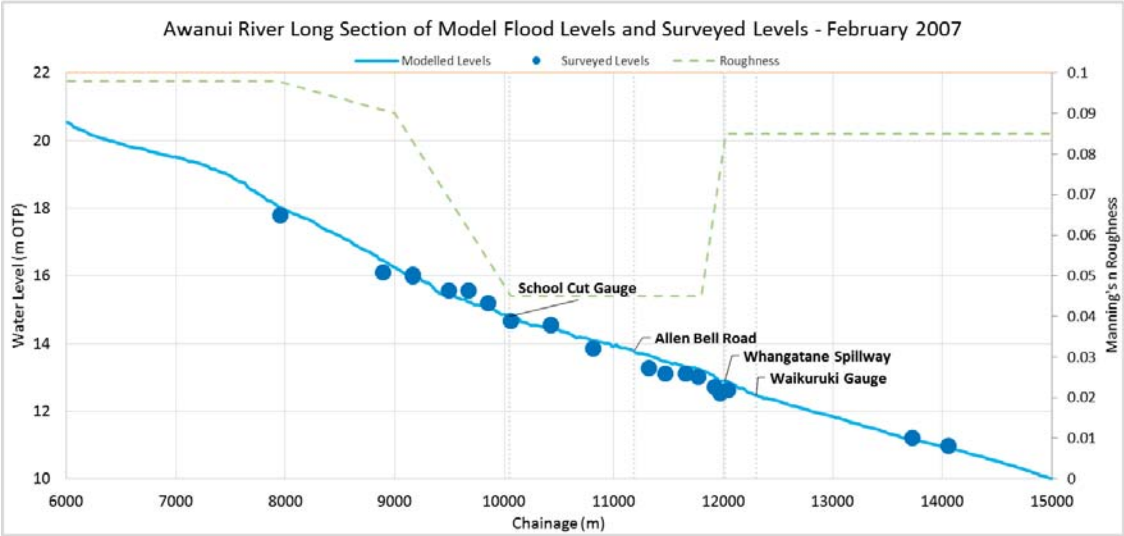


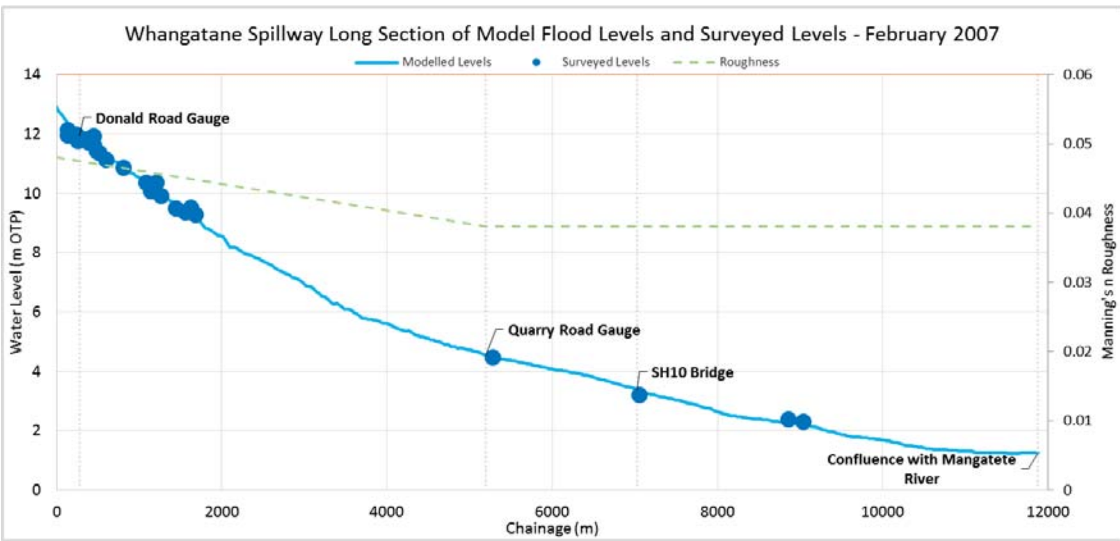
A3 Calibration event longsections








Awanui River Long Section of Model Flood Levels and Surveyed Levels - February 2007
Modelled Levels
Surveyed Levels
Roughness
Water Level (m OTP)
Manning's n Roughness
Chainage (m)
School Cut Gauge
Allen Bell Road
Whangatane Spillway
Waikuruki Gauge


Whangatane Spillway Long Section of Model Flood Levels and Surveyed Levels - February 2007
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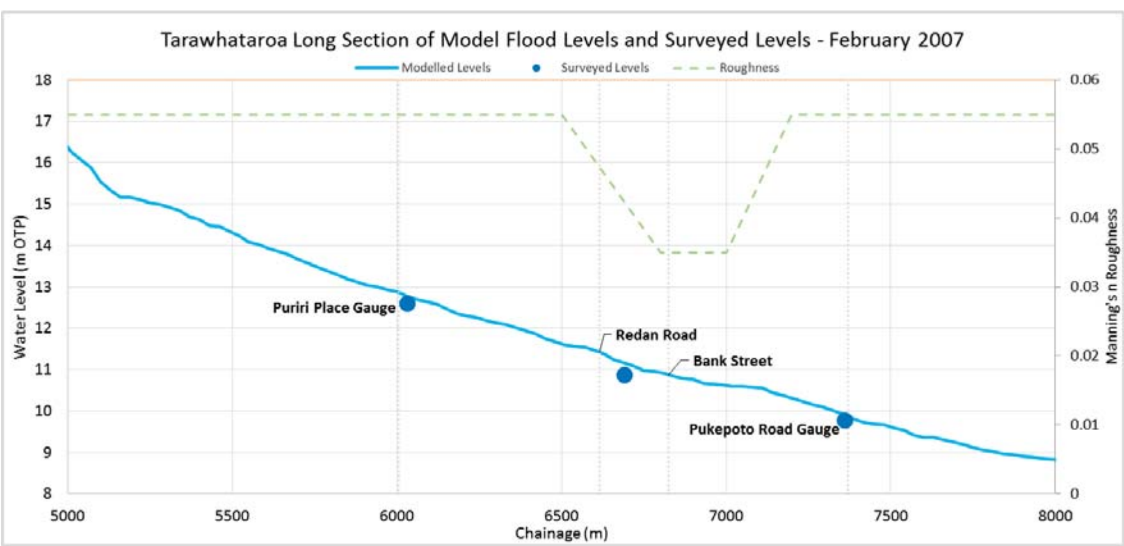
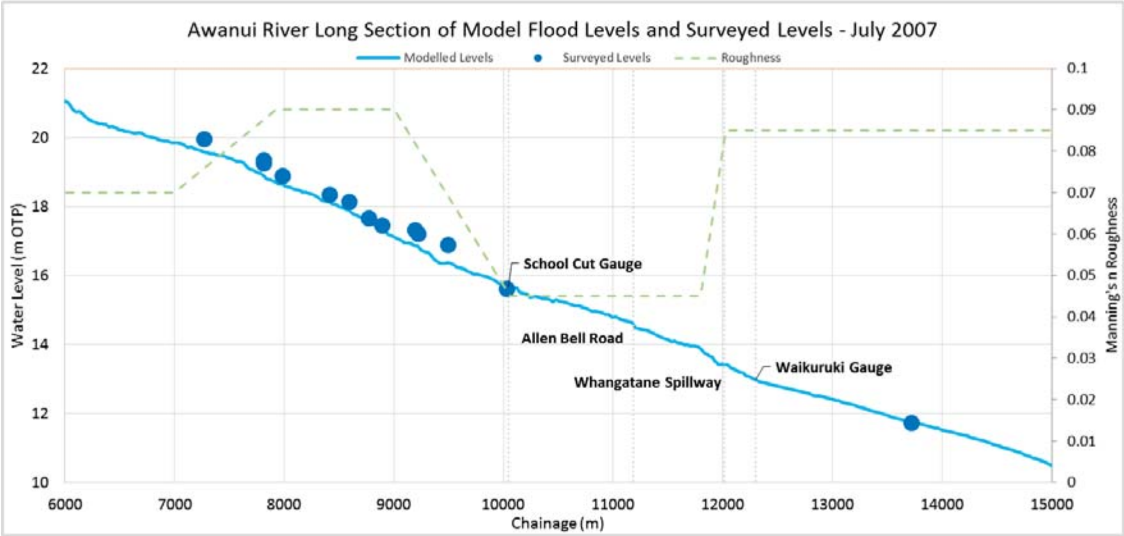
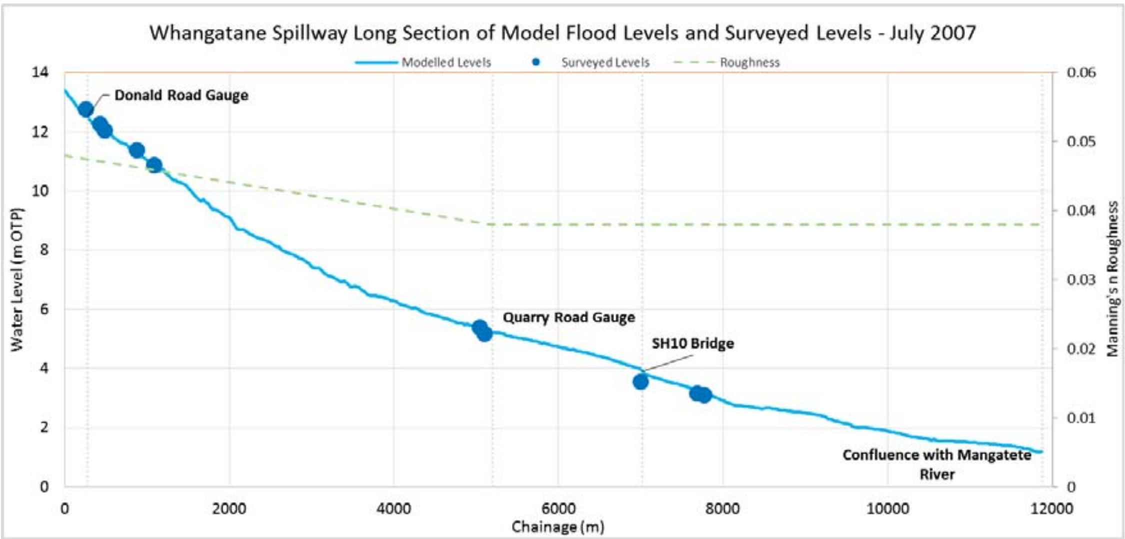
Surveyed Levels
Roughness
Water Level (m OTP)
Manning's n Roughness
Chainage (m)
Donald Road Gauge
Quarry Road Gauge
SH10 Bridge
Confluence with Mangatete River


Tarawhataroa Long Section of Model Flood Levels and Surveyed Levels - February 2007
Modelled Levels
Surveyed Levels
Roughness
Water Level (m OTP)
Manning's n Roughness
Chainage (m)
Puriri Place Gauge
Redan Road
Bank Street
Pukepoto Road Gauge


Awanui River Long Section of Model Flood Levels and Surveyed Levels - July 2007
Modelled Levels
Surveyed Levels
Roughness
Water Level (m OTP)
Manning's n Roughness
Chainage (m)
School Cut Gauge
Allen Bell Road
Whangatane Spillway
Waikuruki Gauge


Whangatane Spillway Long Section of Model Flood Levels and Surveyed Levels - July 2007
Modelled Levels
Surveyed Levels
Roughness
Water Level (m OTP)
Manning's n Roughness
Chainage (m)
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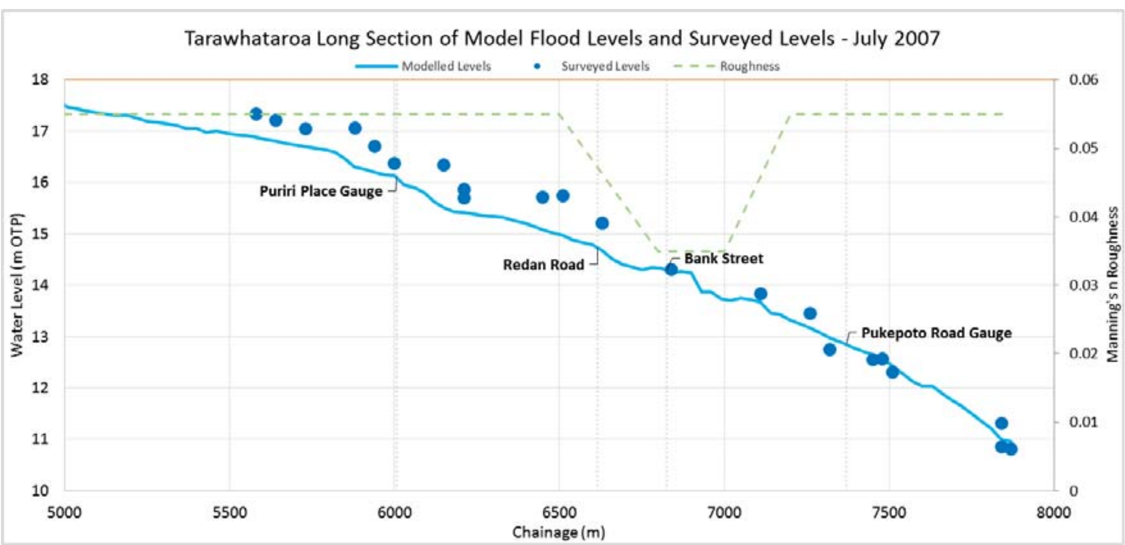
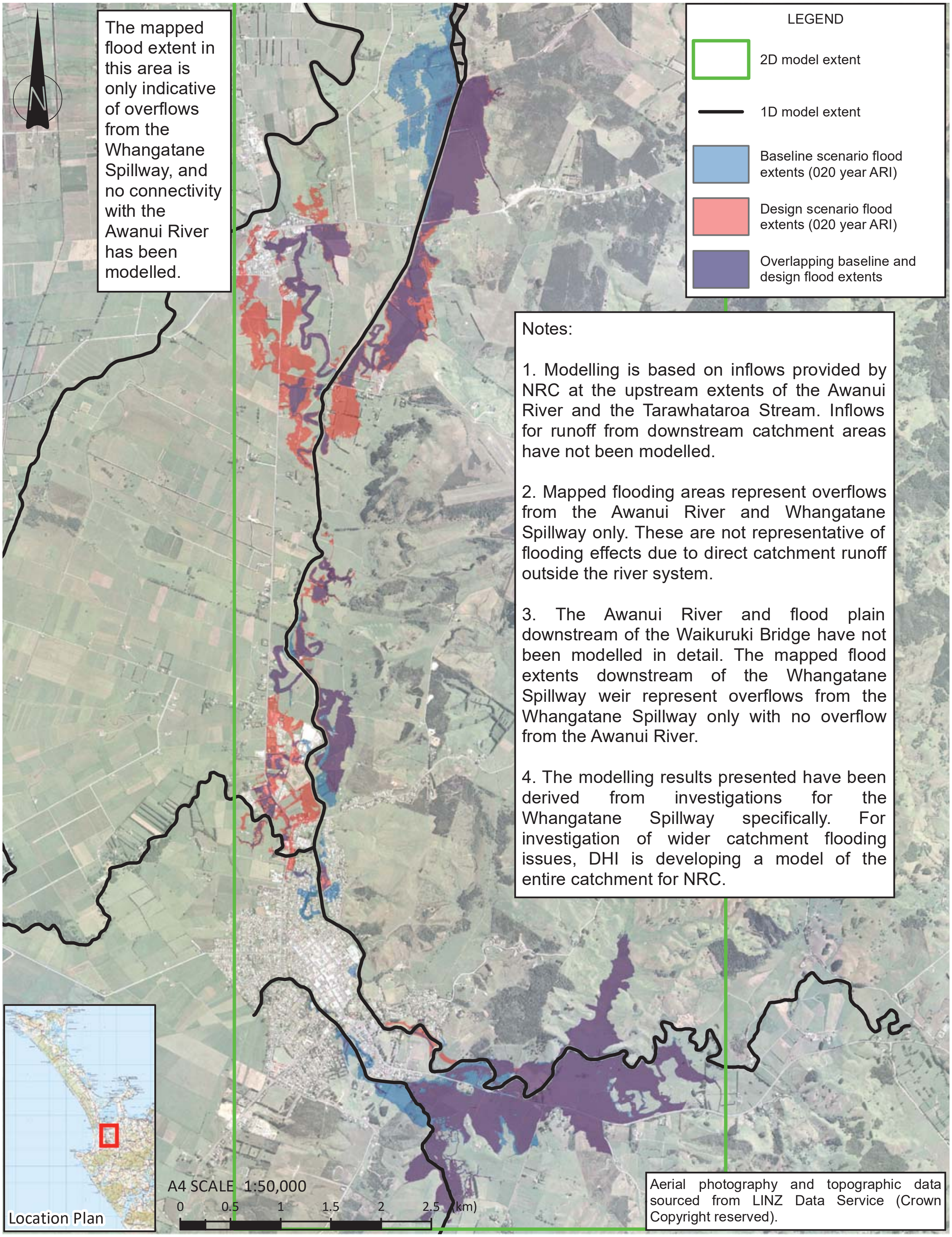
SH10 Bridge
Confluence with Mangatete River


Tarawhataroa Long Section of Model Flood Levels and Surveyed Levels - July 2007
Modelled Levels
Surveyed Levels
Roughness
Water Level (m OTP)
Manning's n Roughness
Chainage (m)
Puriri Place Gauge
Redan Road
Bank Street
Pukepoto Road Gauge

A4 Baseline and design scenario flood extents – 20 year



Notes:

1. Modelling is based on inflows provided by NRC at the upstream extents of the Awanui River and the Tarawhataroa Stream. Inflows for runoff from downstream catchment areas have not been modelled.

2. Mapped flooding areas represent overflows from the Awanui River and Whangatane Spillway only. These are not representative of flooding effects due to direct catchment runoff outside the river system.

3. The Awanui River and flood plain downstream of the Waikuruki Bridge have not been modelled in detail. The mapped flood extents downstream of the Whangatane Spillway weir represent overflows from the Whangatane Spillway only with no overflow from the Awanui River.

4. The modelling results presented have been derived from investigations for the Whangatane Spillway specifically. For investigation of wider catchment flooding issues, DHI is developing a model of the entire catchment for NRC.

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DRAWN	MET	Apr.20
CHECKED	TXH	Apr.20
APPROVED	TB	Apr.20
ARCFILE Awanui_20yrExtents_01.mxd		
SCALE (AT A4 SIZE) 1:50,000		
PROJECT No. 1006630		

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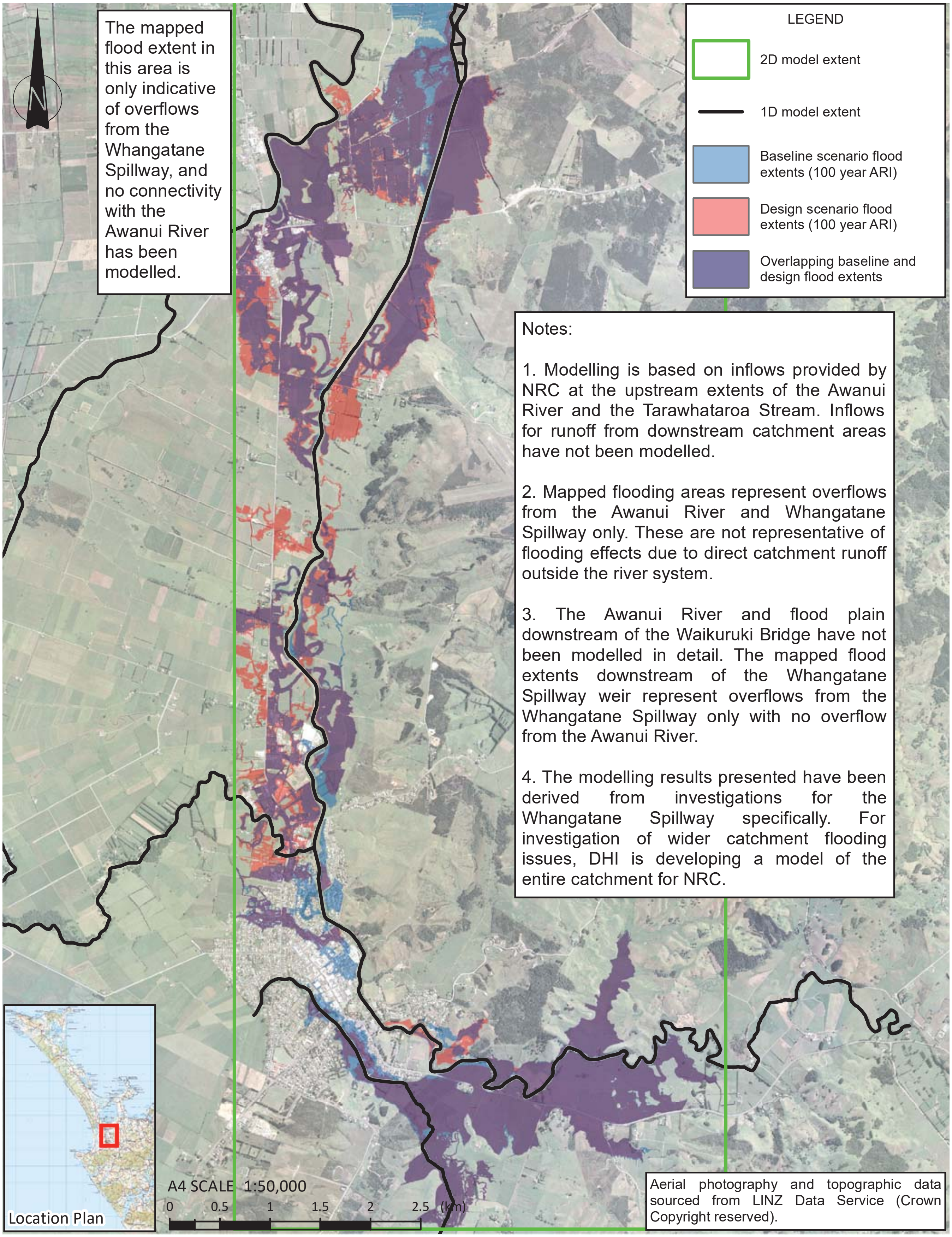
AWANUI RIVER FLOOD PROTECTION SCHEME
PRELIMINARY DESIGN REVIEW
Baseline and Design Scenario Flood Extents - 20 year

FIGURE No. Figure A4

Rev. 0

A5 Baseline and design scenario flood extents – 100 year



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DRAWN	MET	Apr.20
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ARCFILE
Awanui_100yrExtents_01.mxd

SCALE (AT A4 SIZE)
1:50,000

PROJECT No.
1006630

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PRELIMINARY DESIGN REVIEW
Baseline and Design Scenario Flood Extents - 100 year

FIGURE No. Figure A5

Rev. 0

Appendix B: Bridge scoured depth calculation methods and calculation sheets

B1 Equivalent pier width formulae as in *NZ Transport Agency Bridge Manual 3rd Edition (2018)*

2.3.5 Scour

The estimation of scour should be based on *Bridge scour*[21]. This publication replaces section 6 of *Waterway design*[12].

The pier scour depth induced by debris rafts such as described in 3.4.8(c) and as shown in figure 2.2 shall be estimated using an equivalent pier width a_d^* from the equations:

$$a_d^* = \frac{K_{d1}(TW)\left(^L/_y\right)^{K_{d2}} + (y - K_{d1}T)a}{y} \quad \text{for} \quad ^L/_y > 1.0$$

$$a_d^* = \frac{K_{d1}(TW) + (y - K_{d1}T)a}{y} \quad \text{for} \quad ^L/_y \leq 1.0$$

Where:

- K_{d1} = 0.79 for rectangular debris, 0.21 for triangular debris.
- K_{d2} = -0.79 for rectangular debris, -0.17 for triangular debris.
- L = length of debris upstream from pier face (m). L shall be taken as lying within the range $0.4W < L < 1.3W$.
- y = depth of approach flow (m).
- T = thickness of debris normal to flow (m), which shall be taken as the maximum rootball diameter of a tree likely to be transported by the river, (typically up to ~2m), or half the depth of the upstream flow, whichever is the greater, but not greater than 3.0m.
- W = width of debris normal to flow (m), equal to the average of the span lengths either side of the pier, but not greater than the length of the largest tree likely to be transported by the river, or greater than 15m.
- a = pier width (without debris) normal to flow (m).

Figure 6.1: Equivalent pier width (Reproduced from Section 2.3.5 Scour in Bridge Manual)

B2 Debris raft loading as in *NZ Transport Agency Bridge Manual 3rd Edition (2018)*

Figure 2.2: Debris raft for pier scour assessment

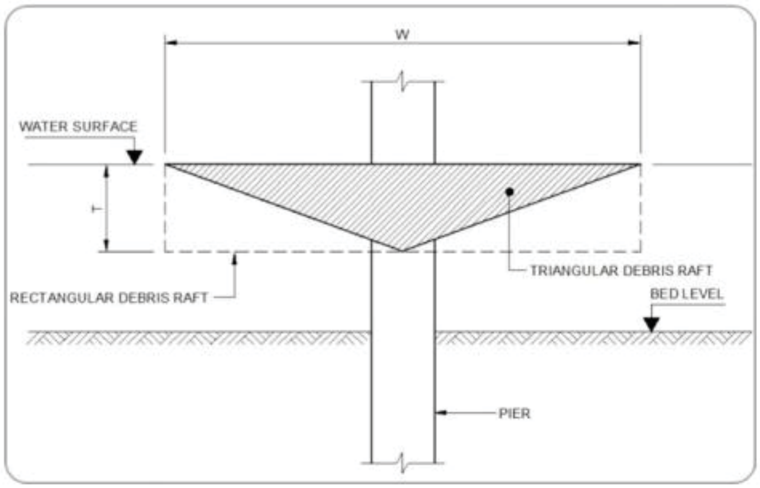


Figure 6.2: Debris raft loading (Reproduced from Section 2.3.5 Scour in Bridge Manual)

B3 The New Zealand Railways Formulation of Holmes (1974) as in *Bridge Scour (Melville and Coleman, 2000)*

The New Zealand Railways Formulation of Holmes (1974)

The empirical New Zealand Railways method was developed by Holmes (1974) to estimate the total scour at a bridge site. General scour is not predicted as an independent quantity. Instead, the scoured flow depth, y_s, for the combination of general scour and contraction scour is predicted as the greater of

$$y_s = y \quad \text{or} \quad y_s = \frac{y_r V_1 K}{\sqrt{(A/W)}} \qquad \textbf{(4.13)}$$

where

$$V_1 = C\left(\frac{Q}{A}\right)\left(\frac{y}{A/W}\right)^{2/3} \quad \text{and} \quad K = \sqrt{\frac{W}{4.83Q^{0.5}}} \leq 1$$

and y (m) is the unscoured flow depth; y_r (m) is the water level rise from low water to flood stage; A (m^2) is flow area for the unscoured profile; W (m) is waterway width allowing for berm flow, taken as $1.25W_{80}$; W_{80} is the width of the waterway, including the main channel, which carries 80% of the total flow; V_1 (m/s) is approach velocity; the coefficient C is 1.2 where converging flows are encountered, such as in braided streams, and 1.0 in other cases; Q (m^3/s) is flow rate; and K is a factor dependent on waterway width and the Lacey regime width. Knowing the water surface level corresponding to Q, the scoured bed level can then be determined.

Figure 6.3: The New Zealand Railways Formulation of Holmes (1974) (Reproduced from Section 4.3.3 in *Bridge Scour)*

B4 The Melville method (1997) to estimate local scour as in *Bridge Scour (Melville and Coleman, 2000)*

B4.1 Procedure

1. NONUNIFORM PIERS:	For nonuniform piers, calculate b_e from (6.8) For debris effects, evaluate b_e from (6.22)
2. DEPTH-SIZE:	Find K_{yb} (m) from (6.4) Use b_e for nonuniform piers and debris
3. FLOW INTENSITY:	Find K_I from (6.6)
4. SEDIMENT SIZE:	Find K_d from (6.7) Use d_{50} size for uniform sediments Use d_{50a} size for nonuniform sediments Use b_e for nonuniform piers and debris
5. FOUNDATION SHAPE:	Find K_s from Table 7.3 ($K_s = 1$ for $\theta > 5°$)
6. FLOW ALIGNMENT:	Find K_θ from (6.10)
7. EQUILIBRIUM TIME:	Find t_e from (6.14)
8. TIME (DURATION OF SCOURING):	Find K_t from (6.13)
9. LOCAL SCOUR DEPTH:	Find d_s from:

$$\boxed{d_s = K_{yb} K_I K_d K_s K_\theta K_t}$$

Figure 7.13. Procedure for estimation of local scour depth at bridge piers. Refer Table 7.3, Figure 7.10 and Figure 7.12.

B4.2 Factors to estimate local scour depth at bridge piers as in *Bridge Scour (Melville and Coleman, 2000)*

Table 7.3. Summary of factors for estimation of local scour depth at bridge piers.

Factor	Symbol	Method of Estimation		Ref.
Depth-size factor	K_{yb}	$K_{yb} = 2.4b$	$\frac{b}{y} < 0.7$	(6.4)
		$K_{yb} = 2\sqrt{yb}$	$0.7 < \frac{b}{y} < 5$	
		$K_{yb} = 4.5y$	$\frac{b}{y} > 5$	
Flow intensity factor	K_I	$K_I = \frac{V-(V_a-V_c)}{V_c}$	$\frac{V-(V_a-V_c)}{V_c} < 1$	(6.6)
		$K_I = 1$	$\frac{V-(V_a-V_c)}{V_c} \geq 1$	
Sediment size factor	K_d	$K_d = 0.57\log\left(2.24\frac{b}{d_{50}}\right)$	$\frac{b}{d_{50}} \leq 25$	(6.7)
		$K_d = 1.0$	$\frac{b}{d_{50}} > 25$	
Shape factor	K_s	Shape	K_s	Table 6.3
		Circular	1.0	
		Round Nosed	1.0	
		Square Nosed	1.1	
		Sharp Nosed	0.9	
		Skewed piers	1.0	
Equivalent size for nonuniform piers	b_e	$b_e = b$	Case I	(6.8)
		$b_e = b\left(\frac{y+Y}{y+b^*}\right) + b^*\left(\frac{b^*-Y}{b^*+y}\right)$ $Y \leq b^*,\ -Y \leq y$	Case II Case III	
		$b_e = b^*$	Case IV	

Table 7.3. (cont.) Summary of factors for estimation of local scour depth at bridge piers.

Factor	Symbol	Method of Estimation					Ref.
Multiplying factors for pile groups	K_sK_θ	Type	S_p/D_p	K_sK_θ: $\theta<5°$	$\theta=5°\to45°$	$\theta=90°$	Table 6.4
		Single Row	2	1.12	1.40	1.20	
			4	1.12	1.20	1.10	
			6	1.07	1.16	1.08	
			8	1.04	1.12	1.02	
			10	1.00	1.00	1.00	
		Double Row	2	1.50	1.80	-	
			4	1.35	1.50	-	
Alignment factor	K_θ	$K_\theta = \left(\frac{l}{b}\sin\theta + \cos\theta\right)^{0.65}$	non-circular piers				(6.10)
		$K_\theta = 1.0$	circular piers				
Time factor	K_t	$K_t = \exp\left\{-0.03\left\lvert\frac{V_c}{V}\ln\left(\frac{t}{t_e}\right)\right\rvert^{1.6}\right\}$	$\frac{V}{V_c} \leq 1$				(6.13)
		$K_t = 1.0$	$\frac{V}{V_c} > 1$				
Equilibrium time	t_e	$t_e(days) = 48.26\frac{b}{V}\left(\frac{V}{V_c} - 0.4\right)$	$\frac{y}{b} > 6, \frac{V}{V_c} > 0.4$				(6.14)
		$t_e(days) = 30.89\frac{b}{V}\left(\frac{V}{V_c} - 0.4\right)\left(\frac{y}{b}\right)^{0.25}$	$\frac{y}{b} \leq 6, \frac{V}{V_c} > 0.4$				
Equivalent size of a pier with debris	b_e	$b_e = \frac{0.52T_d b_d + (y - 0.52T_d)b}{y}$					(6.22)

Figure 6.4: The Melville method (1997) to calculate local scour depth (Reproduced from Section 7.6 in *Bridge Scour*)

B4.3 Local scour depth estimations as in *Bridge Scour (Melville and Coleman, 2000)*

THRESHOLD VELOCITY, V_c:

1. **Find u_{*c} for d_{50} size from Shields diagram or (for quartz sand in water at 20°C)**

$$u_{*c} = 0.0115 + 0.0125 d_{50}^{1.4} \qquad 0.1\text{mm} < d_{50} < 1\text{mm}$$

$$u_{*c} = 0.0305 d_{50}^{0.5} - 0.0065 d_{50}^{-1} \qquad 1\text{mm} < d_{50} < 100\text{mm}$$

2. **Find V_c from logarithmic velocity distribution (for fully turbulent flow)**

$$\frac{V_c}{u_{*c}} = 5.75 \log\left(5.53 \frac{y}{d_{50}}\right)$$

ARMOUR PEAK VELOCITY, V_a (σ_g>1.3 only):

3. **Find d_{50a} from**

$$d_{50a} = \frac{d_{max}}{1.8}$$

4. **Find u_{*ca} for d_{50a} size from Shields diagram or (for quartz sand in water at 20°C)**

$$u_{*ca} = 0.0115 + 0.0125 d_{50a}^{1.4} \qquad 0.1\text{mm} < d_{50a} < 1\text{mm}$$

$$u_{*ca} = 0.0305 d_{50a}^{0.5} - 0.0065 d_{50a}^{-1} \qquad 1\text{mm} < d_{50a} < 100\text{mm}$$

5. **Find V_{ca} from**

$$\frac{V_{ca}}{u_{*ca}} = 5.75 \log\left(5.53 \frac{y}{d_{50a}}\right)$$

6. **Calculate**

$$V_a = 0.8 V_{ca}$$

VELOCITY PARAMETER:

7. **Evaluate**

$$\frac{\left[V - (V_a - V_c)\right]}{V_c} \qquad \left(= \frac{V}{V_c} \quad \text{for } \sigma_g < 1.3\right)$$

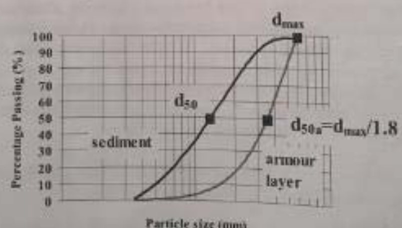


Figure 6.6. Method to determine the armour velocity V_a.

6.3. SCOUR DEPTH ESTIMATION

The design approach is to consider bridge piers and abutments together and develop a design method that can be applied in both cases. The design method rests on the following relation for the depth of local scour:

$$d_s = K_{yB} K_I K_d K_s K_\theta K_G K_t \tag{6.3}$$

where the K's are empirical expressions accounting for the various influences on scour depth: K_{yB} = depth-size = K_{yb} for piers and K_{yL} for abutments; K_I = flow intensity; K_d = sediment size; K_s = pier or abutment shape; K_θ = pier or abutment alignment; K_G = channel geometry; and K_t = time. K_I is formulated to include sediment gradation effects as well as flow velocity effects. K_{yB} = f(y, B) and d_s have the dimension of length, while the other K's are dimensionless. Equation 6.3 applies to local scour only; that is, it is independent of contraction effects, as represented by the ratio of the widths of the bridge opening and channel. Contraction effects are considered in Chapter 5.

The terms in (6.3) are considered individually in the following sections.

6.3.1. Flow Depth - Foundation Size (Depth - Size) Factor, K_{yB}

The local scour classification given in Table 6.1 is derived from a plot of all reliable pier and abutment local scour depth data that are unaffected by flow intensity, sediment size, sediment gradation, foundation shape and alignment, channel geometry and time, because the following conditions apply to all the data:

- V/V_c=1, that is, the threshold condition (refer Section 6.2.1);
- B/d_{50}>50, that is, coarse sediment (refer Section 6.2.3);
- σ_g<1.3, that is, uniform sediment (refer Section 6.2.4);
- circular piers and vertical-wall abutments, that is, the standard foundation shapes (refer Section 6.2.5);
- θ=0° (90°), that is, aligned piers (abutments) (refer Section 6.2.6);
- rectangular channels (refer Section 6.2.7); and
- equilibrium scour (refer Section 6.2.8).

Hence these data demonstrate the influence of $K_{yB} = f(y, B)$ on local scour depth. The data plots are given in Figure 6.26; which includes data by Chabert and Engeldinger (1956), Laursen and Toch (1956), Hancu (1971), Bonasoundas (1973), Basak (1975), Jain and Fischer (1979), Chee (1982), Chiew (1984), and Ettema (1980), for piers; and Gill (1972), Wong (1982), Tey (1984), Kwan (1984, 1988), Kandasamy (1989), and Dongol (1994), for abutments.

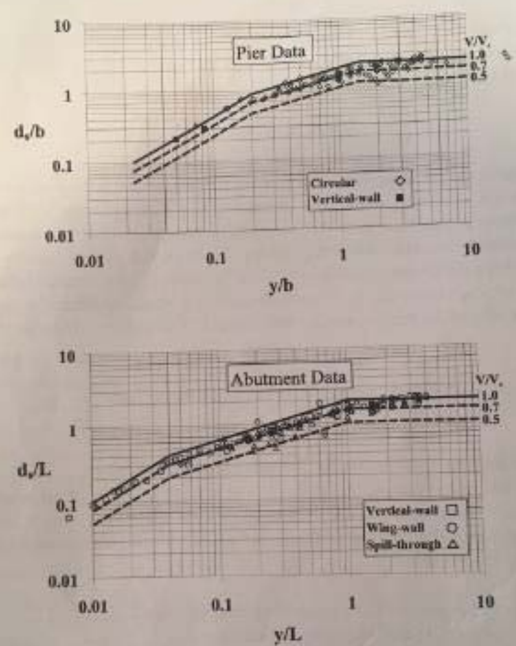


Figure 6.26. The influence of flow shallowness on local scour depth.

The solid lines in Figure 6.26 are envelopes to the data and apply, from left to right respectively, to wide (long), intermediate width (length) and narrow (short) piers (abutments) at threshold conditions. For clear-water scour at reduced flow velocities, lesser scour depths are developed. The dashed lines in Figure 6.26 are indicative of scour depths for different values of V/V_c and it

is apparent that scour depths can be reduced considerably from the maxima at the threshold condition. The lines have been plotted assuming power relations between clear-water scour depth and flow velocity. If other scour depth influences (especially sediment gradation, abutment shape and channel geometry) were present, actual scour depths would be further reduced from the maxima defined by the lines in Figure 6.26.

The equations of the upper-limit lines define the depth-size factors for piers,

$$K_{yb} = 2.4b \qquad \frac{b}{y} < 0.7 \tag{6.4a}$$

$$K_{yb} = 2\sqrt{yb} \qquad 0.7 < \frac{b}{y} < 5 \tag{6.4b}$$

$$K_{yb} = 4.5y \qquad \frac{b}{y} > 5 \tag{6.4c}$$

and for abutments,

$$K_{yL} = 2L \qquad \frac{L}{y} < 1 \tag{6.5a}$$

$$K_{yL} = 2\sqrt{yL} \qquad 1 < \frac{L}{y} < 25 \tag{6.5b}$$

$$K_{yL} = 10y \qquad \frac{L}{y} > 25 \tag{6.5c}$$

6.3.2. Flow Intensity Factor, K_I

K_I represents the effects of flow intensity on local scour depth. It is defined, for each set of data, as the scour depth at a particular flow intensity divided by the maximum scour depth for the data set, where V is systematically varied for each data set and all other dependent parameters are held constant. The scour maxima used occur at the threshold peak for uniform sediments and the live-bed peak for nonuniform sediments.

Figure 6.27 (uniform sediments) and Figure 6.28 (nonuniform sediments) are plots of laboratory data from many sources for local scour at piers and abutments in terms of K_I. The nonuniform sediment data are plotted in terms of a transformed velocity parameter, as shown. The transformed velocity parameter aligns the armour peaks (that is $V=V_a$) for nonuniform sediments with varying σ_g with the threshold peak for uniform sediments. For uniform sediments, $V_a = V_c$ and $[V-(V_a-V_c)]/V_c = V/V_c$. The transformed velocity parameter incorporating V_c largely accounts for the effects of sediment nonuniformity as well as those of flow velocity, although the smaller values of scour depth at $[V-(V_a-V_c)]/V_c=1$, as σ_g increases, remain. Thus, the effects of

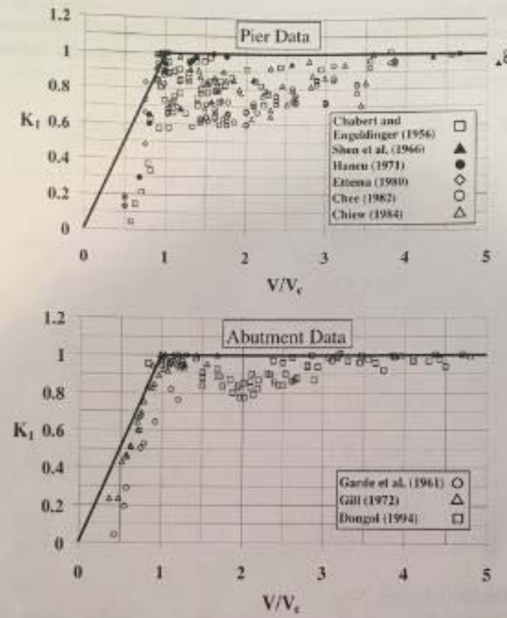


Figure 6.27. Influence of flow intensity on local scour depth in uniform sediment.

sediment nonuniformity are mostly accounted for in the flow intensity factor. It is apparent that all of the data are enveloped by a value of K_I increasing linearly from zero to unity at the threshold condition and thereafter remaining unchanged, that is

mean flow velocity. page 154.

$$K_I = \frac{V-(V_a-V_c)}{V_c} \qquad \frac{V-(V_a-V_c)}{V_c} < 1 \tag{6.6a}$$

$$K_I = 1 \qquad \frac{V-(V_a-V_c)}{V_c} \geq 1 \tag{6.6b}$$

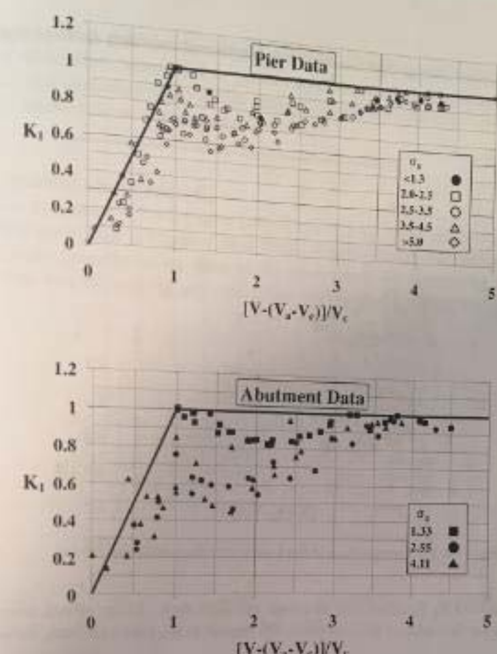


Figure 6.28. Influence of flow intensity on local scour depth in nonuniform sediment.

For estimation of live-bed scour depth at foundations in sandy materials, the K_I function shown is appropriate for design purposes, because very high flow velocities can occur, that is, the live-bed peak (which approaches the threshold peak) can be reached. However, in graded gravel beds, the live-bed scour depths could be substantially less than predicted using (6.6), because $[V-(V_a-V_c)]/V_c$ may not exceed 2.5 to 3.0, that is the live-bed peak may not be, and in many cases is unlikely to be, reached.

Figure 6.29 is a comparison of U.S. field data (Section 1.5.3) with the laboratory-based envelope curves for K_I. Because many of these data were collected at sites where the bed material is nonuniform, the transformed velocity parameter is used in Figure 6.29. The field scour depths are normalised using the projected pier width, b_p, to compensate for pier skewness effects inherent in the data. The armour peak velocity was determined assuming d_{84} to be representative of the maximum grain size in the bed material.

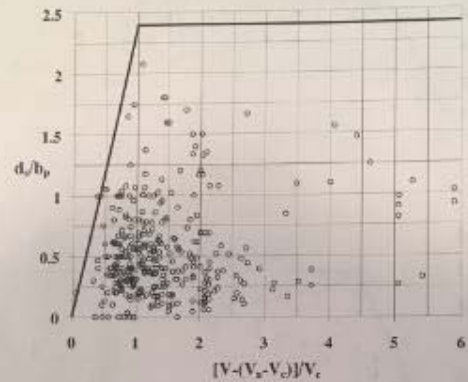


Figure 6.29. Comparison of field data with flow intensity curves.

The laboratory-derived K_I function also envelops the field data. Other effects inherent in the field data account for the reduced scour depths with respect to the envelope lines, including:

- the difficulties inherent in field measurement of scour depth, as discussed in Section 1.5.3;
- time, where the field data were measured during floods that would not necessarily persist long enough for equilibrium scour depths to be developed, especially for lower flow velocities;
- unsteady flow, flows in the field being unsteady, whereas the laboratory data were all measured for steady flow conditions;
- relatively shallow flow depth, as discussed in Section 6.2.2;
- relatively coarse sediment size, as discussed in Section 6.2.3;
- sediment gradation, the transformed velocity parameter accounting only partly for sediment gradation effects;
- pier shape, the data including measurements for a variety of pier shapes and types, although these differences are not distinguished in the plot; and
- pier skewness, recognising that b_p does not wholly account for skewness effects.

Figure 6.29 shows that the laboratory-derived relations for K_I generally lead to conservative estimates of scour depth when used to predict actual scour depths in field conditions. The implication is that laboratory-based design relations should be used with care for estimation of "most likely" depths of scour for given field conditions.

6.3.3. Sediment Size Factor, K_d

The pier data by Ettema (1980), Chiew (1984) and Baker (1986) and the abutment data by Dongol (1994) are plotted in Figure 6.30 in terms of the sediment size multiplying factor, K_d, which is defined generally as the ratio of the scour depth for a particular B/d_{50} to that for $B/d_{50} \geq 250$. The data for uniform and nonuniform sediments are plotted separately. The plots

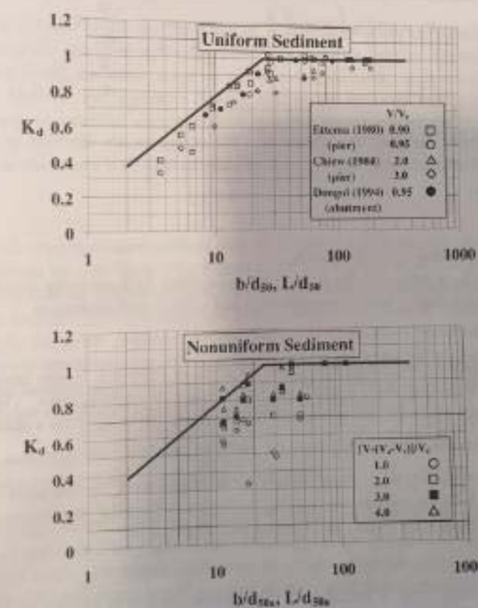


Figure 6.30. Influence of sediment coarseness on local scour depth.

show that the influence of relative sediment size on scour depth is the same for both piers and abutments, although few data are shown for abutments. Because the condition $L/d_{50}<50$ is unlikely in practice, it is considered that the few abutment data shown in Figure 6.30 are adequate for definition of K_d for abutments.

Nonuniform sediments are characterised by channel bed armouring as discussed earlier. The nonuniform sediment data in Figure 6.30 are plotted for different values of the velocity parameter $[V-(V_a-V_c)]/V_c = 1.0, 2.0, 3.0$ and 4.0. The data are plotted in terms of b/d_{50a} or L/d_{50a} because the median size of the armour layer is considered to be the characteristic sediment size.

The envelope curves in Figure 6.30 define the sediment size factor for design purposes and have the equations

$$K_d = 0.57\log\left(2.24\frac{B}{d_{50}}\right) \qquad \frac{B}{d_{50}} \le 25 \qquad \textbf{(6.7a)}$$

$$K_d = 1.0 \qquad \frac{B}{d_{50}} > 25 \qquad \textbf{(6.7b)}$$

where B≡b for piers and B≡L for abutments. The equations are identical to those given in Melville and Sutherland (1988). For nonuniform sediments, (6.7) is expressed in terms of B/d_{50a} rather than B/d_{50}.

6.3.4. Foundation Shape Factor, K_s

The shape factor K_s is defined as the ratio of the scour depth for a particular foundation shape to that for the standard shapes, namely circular piers and vertical-wall abutments.

Uniform piers

Recommended shape factors for uniform piers are given in Table 6.3. These factors, taken from Melville (1997), show that shape is relatively insignificant for uniform piers. The shape factors should only be used where the pier is aligned with the flow, that is, $K_s=1$ for a skewed pier.

Tapered piers

Based on data by Chiew (1984), shape factors for oblong-shaped, tapered piers with $\alpha = 22.5°$ are $K_s = 1.20$ and 0.76 for downwards-tapering and upwards-tapering piers respectively. α is the slope angle of the taper with respect to a vertical line, Figure 6.14.

Nonuniform piers

The four cases of local scour at nonuniform piers, where the pier is founded on a wider element (caissons, slab footings and pile caps), are shown in Figure 6.15. For ***Case I***, the local scour is

Table 6.3. Shape factors for uniform piers and shorter abutments.

Foundation Type	Shape	K_s
Uniform Pier	Circular	1.0
	Round Nosed	1.0
	Square Nosed	1.1
	Sharp Nosed	0.9
Abutment	Vertical-wall	1.0
	Wing-wall	0.75
	Spill-through 0.5:1 (H:V)	0.6
	Spill-through 1:1	0.5
	Spill-through 1.5:1	0.45

estimated using the pier width b. For ***Case II***, a procedure given by Melville and Raudkivi (1996) to estimate the size of an equivalent uniform pier can be applied. The equivalent uniform pier induces (at least) the same scour as the nonuniform pier. The procedure is therefore conservative. Melville and Raudkivi (1996), who measured scour depths at a circular pier founded on a larger concentric, circular caisson, give the following relation:

$$b_e = b\left(\frac{y+Y}{y+b^*}\right) + b^*\left(\frac{b^*-Y}{b^*+y}\right) \qquad \textbf{(6.8)}$$

where b_e = width of an equivalent uniform pier; b^* = caisson width; and the equation is restricted to the range defined by $Y \le b^*$ and $-Y \le y$, Figure 6.15. The relation for b_e can be used for Case II nonuniform piers that are geometrically similar to the caisson foundation shown in Figure 6.15, including piers founded on slab footings and piled foundations, unless the footing or the pile cap is undermined by the scour. Equation 6.8 also applies to ***Case III*** caisson foundations and may be used to give conservative scour estimates for Case III piled foundations. For ***Case IV*** caisson foundations, the local scour is estimated using the caisson width b^*. This approach would also give a conservative estimate of Case IV local scour at a piled foundation.

Shape factors, based on data by Hannah (1978), for piled pier foundations where the pile cap is clear of the water surface (Case V) are given in Table 6.4.

The pile-group shape factor values are shown in Table 6.4 for a single row and a double row of piles in terms of approach flow angle, θ, pile diameter, D, and pile spacing (measured centre-to-centre), S_p. The single-row values apply also to a pier comprising a row of cylinders. The values shown include pier alignment effects and shape effects, that is, they represent K_sK_θ.

Table 6.4. Multiplying factors (K_sK_θ) for pile groups (Case V).

Type	S_p/D_p	K_sK_θ		
		θ< 5°	θ= 5°→45°	θ= 90°
Single row	2	1.12	1.40	1.20
	4	1.12	1.20	1.10
	6	1.07	1.16	1.08
	8	1.04	1.12	1.02
	10	1.00	1.00	1.00
Double row	2	1.50	1.80	-
	4	1.35	1.50	-

Abutments

Recommended shape factors for shorter abutments are given in Table 6.3. For longer abutments, shape effects are less significant as discussed in Section 6.2.5. The adjusted shape factor K_s^* for longer abutments is

$$K_s^* = K_s \qquad \frac{L}{y} \le 10 \tag{6.9a}$$

$$K_s^* = K_s + 0.667(1-K_s)\left(0.1\frac{L}{y}-1\right) \qquad 10 < \frac{L}{y} < 25 \tag{6.9b}$$

$$K_s^* = 1.0 \qquad \frac{L}{y} \ge 25 \tag{6.9c}$$

where K_s is the shape factor for a shorter abutment, given in Table 6.3.

6.3.5. Foundation Alignment Factor, K_θ

The alignment factor K_θ is defined as the ratio of the local scour depth at a skewed bridge foundation to that at an aligned foundation. Bridge piers are aligned if $\theta = 0°$ (Figure 6.17), while abutments are considered to be aligned where $\theta = 90°$ (Figure 6.18). The chart of multiplying factors, K_θ, given in Figure 6.17, is recommended for ***non-cylindrical piers***. The following equation is a good approximation to the curves in Figure 6.17:

$$K_\theta = \left(\frac{b_p}{b}\right)^{0.65} = \left(\frac{l}{b}\sin\theta + \cos\theta\right)^{0.65} \tag{6.10}$$

in which $b_p = l\sin\theta + b\cos\theta$ = projected width of a rectangular pier of length l and breadth b. For ***circular piers***, $K_\theta = 1.0$.

Recommended alignment factors for longer ***abutments*** are given in Table 6.5.

Table 6.5. Alignment factor K_θ for abutments.

θ(°)	30	45	60	90	120	135	150
K_θ	0.90	0.95	0.98	1.0	1.05	1.07	1.08

The values in Table 6.5 are derived from the envelope curve in Figure II.13, Appendix II. For shorter abutments, alignment effects are less significant as discussed in Section 6.2.6. The adjusted alignment factor K_θ^* for shorter abutments is

$$K_\theta^* = K_\theta \qquad \frac{L}{y} \ge 3 \tag{6.11a}$$

$$K_\theta^* = K_\theta + (1-K_\theta)\left(1.5-0.5\frac{L}{y}\right) \qquad 1 < \frac{L}{y} < 3 \tag{6.11b}$$

$$K_\theta^* = 1 \qquad \frac{L}{y} \le 1 \tag{6.11c}$$

where K_θ is from Table 6.5.

6.3.6. Approach Channel Geometry Factor, K_G

The approach channel geometry factor K_G is the ratio of the local scour depth at a bridge foundation to that at the same foundation sited in the equivalent rectangular channel. The local scour at bridge ***piers*** is considered not to be affected by approach channel geometry as long as appropriate values of y and V are used to estimate the scour depth. If values of y and V are selected to be representative of the flow approaching the particular pier, K_G=1.0.

For bridge ***abutments*** in rectangular channels (Case A of Figure 6.21), K_G=1.0 by definition. For abutments in compound channels, K_G depends on the position of the abutment in the compound channel (Figure 6.21). At Case B abutments, the following equation is recommended:

$$K_G = \sqrt{1-\left(\frac{L^*}{L}\right)\left[1-\left(\frac{y^*}{y}\right)^{5/3}\left(\frac{n}{n^*}\right)\right]} \tag{6.12}$$

where L and L* = total projected length of the abutment (including the bridge approach) and projected length of the abutment (including the bridge approach) spanning the flood channel, respectively; y and y* = flow depths in the main and flood channels, respectively; and n and n* = Manning roughness coefficients for the main and flood channels, respectively. The equation is derived from a simple theoretical analysis based on the ratio of flows deflected by the abutment, including the bridge approach, in a compound channel to such flows in the corresponding rectangular channel. The equation, which is plotted in Figure 6.31 for ranges of values of the ratios (L^*/L), (y/y^*) and (n/n^*), fits Melville's (1995) data well, as shown in Figure II.14, Appendix II. Figure 6.31 shows that scour depths at abutments situated near the edge of the main channel of a compound channel ($L^*/L \rightarrow 1$) having shallow flow depth ($y/y^* \rightarrow \infty$) and high roughness ($n/n^* \rightarrow 0$) on the flood plain, can be reduced to 10% of the scour depth in the equivalent rectangular channel. As discussed in Section 6.2.7, Case C (Figure 6.21) can be considered to be a special condition of Case A if the flow in the main channel is ignored; thus K_G=1.0. For Case D abutments where the abutment is sited at about the edge of the main channel, K_G can be estimated from (6.12) with L^*/L=1.0. No specific information is available to aid estimation of K_G for other Case D abutments; such situations could be treated by interpolating conservatively between scour depth estimates for longer (Case B) and shorter (Case C) abutments sited in the same channel.

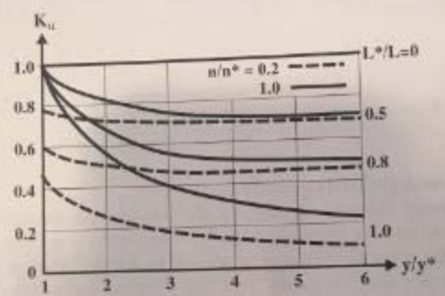
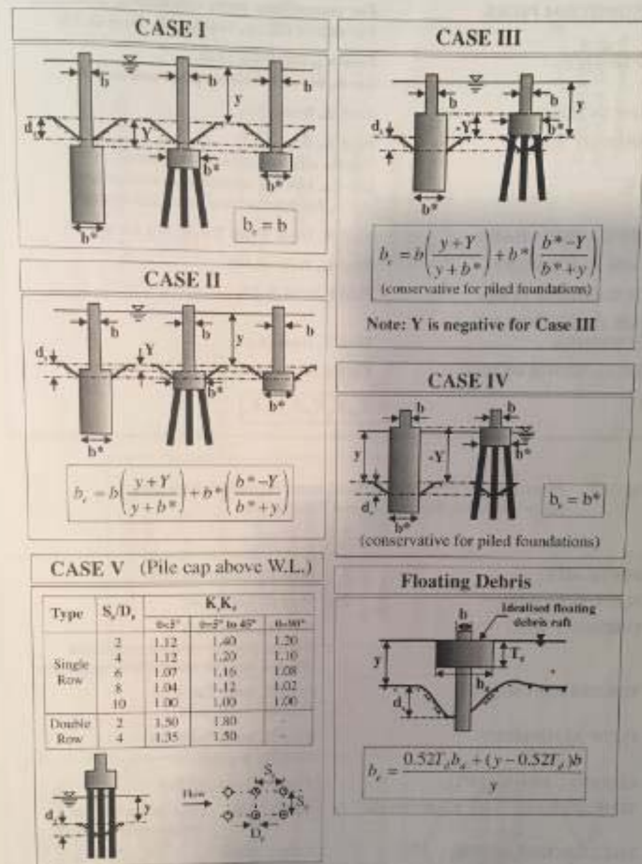


Figure 6.31. Influence of channel geometry on local scour depth at abutments.

6.3.7. Time Factor, K_t

The time factor is defined as the ratio of local scour depth d_s at a particular time t to the equilibrium scour depth d_{se} which occurs at time t_e. The value of K_t at a site depends on whether conditions are clear-water or live-bed. Under live-bed conditions, the equilibrium depth of local scour is attained rapidly and K_t = 1.0 can be assumed.

For local scour at circular bridge ***piers*** under clear-water conditions, K_t is given by the following function, which is plotted in Figure 6.23:



Type	S_s/D_s	$K_s K_\theta$		
		θ<5°	θ=5° to 45°	θ=90°
Single Row	2	1.12	1.40	1.20
	4	1.12	1.20	1.10
	6	1.07	1.16	1.08
	8	1.04	1.12	1.02
	10	1.00	1.00	1.00
Double Row	2	1.50	1.80	-
	4	1.35	1.50	-

Figure 7.12. Method for estimation of local scour depth at nonuniform piers.

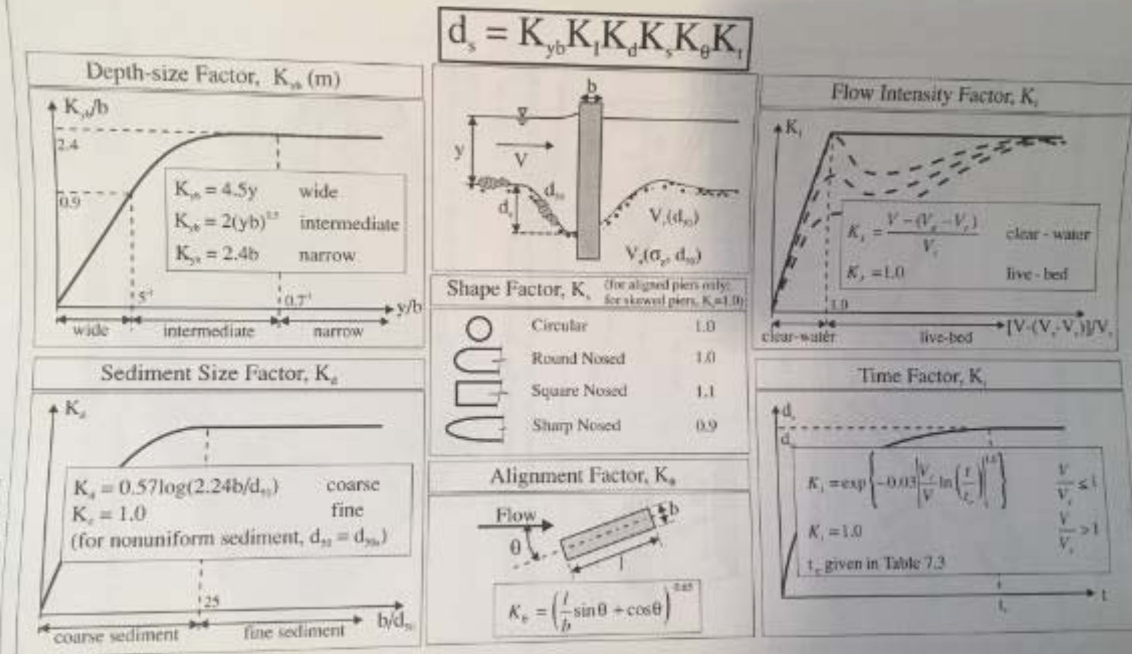


Figure 7.10 Method for estimation of local scour depth at piers.

B4.4 Bridge scoured depth calculations. Scheme Design

Scheme Design
Church Road Bridge (Awanui River)

The New Zealand Railway Formulation of Holmes (1974) as in *Bridge Scour* by Melville and Coleman, 2000

The method combines general scour and contraction scour

New Zealand Railways (Holmes, 1974)[8]	$K = \sqrt{\frac{W}{4.83Q^{0.5}}} \leq 1$ $V_1 = C\left(\frac{Q}{A}\right)\left(\frac{y}{A/W}\right)^{2/3}$ Max. of $y_s = y$ or $y_s = \frac{y_r V_1 K}{\sqrt{(A/W)}}$

Hydraulic Calculations

20yr	100yr		
148.60	165.10	A (m2)	flow area of the unscoured profile
22.00	22.00	m	flood channel width (assumed)(from abutment to abutment)
34.80	34.80	m	bridge (main)channel width
4.27	4.74	y (m)	hydraulic depth
7.83	8.34	m	approach flow depth
2.03	2.16	V (m/s)	mean channel velocity
0.01	0.01	uc	(from Figure 6.6 in *Bridge Scour*)
14.92	15.18	Vc/uc	(from Figure 6.6 in *Bridge Scour*)
0.17	0.18	Vc (m/s)	competent velocity
0.50	0.50	(m)	unscoured flow depth/low flow ASSUMED
0.06	0.06	d50	sediment size for which 50%of the sediment is finer Mangakahia mottled clay loam

Scoured Depth Calculations

20yr	100yr		
301.00	356.20	Q (m3/s)	flow rate
34.80	34.80	W (m)	waterway width allowing for bem flow, taken as W80*1.25. W80 is waterway width, including the main channel which carries 80% of the flow; W80*1.25 - in this case similar to bridge opening width
0.64	0.62	K	factor dependent on waterway width and the Lacey regime width
1.00	1.00	C	coefficient; 1.2 if converging flows are encountered; 1.0 in other cases
2.03	2.16	V1 (m/s)	approach velocity
7.33	7.84	yr (m)	water level rise from low water to flood stage
4.63	4.80	ys (m)	scoured flow depth below flood level
4.63	4.80	MAX ys	maximum scoured flow depth below flood level
0.36	0.05	Tds (m)	Total scoured depth below bed level (= MAX ys-y)
0.43	**0.1**	**Tds (m)**	**Total scoured depth below bed level,incl 20% allowance for no safety factor inclusion in the method**

Scheme Design
Allen Bell Drive Bridge (Awanui River)

The New Zealand Railway Formulation of Holmes (1974) as in *Bridge Scour* by Melville and Coleman, 2000

The method combines general scour and contraction scour

New Zealand Railways (Holmes, 1974)[8]	$K = \sqrt{\frac{W}{4.83Q^{0.5}}} \le 1$ $V_1 = C\left(\frac{Q}{A}\right)\left(\frac{y}{A/W}\right)^{2/3}$ Max. of $y_s = y$ or $y_s = \frac{y_r V_1 K}{\sqrt{(A/W)}}$

Hydraulic Calculations

20yr	100yr		
161.00	161.00	A (m2)	flow area of the unscoured profile
23.20	23.20	m	flood channel width (assumed)(abutment to abutment)
34.00	34.00	m	bridge (main)channel width
4.74	4.74	y (m)	hydraulic depth
8.19	8.72	m	approach flow depth
1.87	2.20	V (m/s)	mean channel velocity
0.01	0.01	uc	(from Figure 6.6 in *Bridge Scour*)
15.18	15.18	Vc/uc	(from Figure 6.6 in *Bridge Scour*)
0.18	0.18	Vc (m/s)	competent velocity
0.50	0.50	m	unscoured flow depth/low flow ASSUMED
0.06	0.06	d50	sediment size for which 50%of the sediment is finer Mangakahia mottled clay loam

Scoured Depth Calculations

20yr	100yr		
300.80	354.20	Q (m3/s)	flow rate
34.00	34.00	W (m)	waterway width allowing for bem flow, taken as W80*1.25. W80 is waterway width, including the main channel which carries 80% of the flow; W80*1.25 - in this case similar to bridge opening width
0.64	0.61	K	factor dependent on waterway width and the Lacey regime width
1.00	1.00	C	coefficient; 1.2 if converging flows are encountered; 1.0 in other cases
1.87	2.20	V1 (m/s)	approach velocity
7.69	8.22	yr (m)	water level rise from low water to flood stage
4.21	5.08	ys (m)	scoured flow depth below flood level
4.74	5.08	MAX ys	maximum scoured flow depth below flood level
0.00	0.35	Tds (m)	Total scoured depth below bed level (= MAX ys-y)
0.00	**0.42**	**Tds (m)**	**Total scoured depth below bed level,incl 20% allowance for no safety factor inclusion in the method**

Scheme Design
SH1 Waikuruki Bridge (North Road, Awanui River)

The New Zealand Railway Formulation of Holmes (1974) as in *Bridge Scour* by Melville and Coleman, 2000
The method combines general scour and contraction scour

New Zealand Railways (Holmes, 1974)[8]	$K = \sqrt{\frac{W}{4.83Q^{0.5}}} \le 1$ $V_1 = C\left(\frac{Q}{A}\right)\left(\frac{y}{A/W}\right)^{2/3}$ Max. of $y_s = y$ or $y_s = \frac{y_r V_1 K}{\sqrt{(A/W)}}$

Hydraulic Calculations

20yr	100yr		
104.60	108.40	A (m2)	flow area of the unscoured profile
28.50	29.50	m	flood channel width (assumed)
30.30	30.30	m	bridge (main)channel width
3.45	3.58	y (m)	hydraulic depth
6.57	6.80	m	approach flow depth
0.81	0.92	V (m/s)	mean channel velocity
0.01	0.01	uc	(from Figure 6.6 in *Bridge Scour*)
22.88	22.97	Vc/uc	(from Figure 6.6 in *Bridge Scour*)
0.26	0.26	Vc (m/s)	competent velocity
0.50	0.50	m	unscoured flow depth/low flow ASSUMED
0.002	0.002	d50	sediment size for which 50%of the sediment is finer Mangakahia silt loam and clay loam

Scoured Depth Calculations

20yr	100yr		
85	99.7	Q (m3/s)	flow rate
30.30	30.30	W (m)	waterway width allowing for bem flow, taken as W80*1.25. W80 is waterway width, including the main channel which carries 80% of the flow; W80*1.25 - in this case similar to bridge opening width
0.82	0.79	K	factor dependent on waterway width and the Lacey regime width
1	1	C	coefficient; 1.2 if converging flows are encountered; 1.0 in other cases
0.81	0.92	V1 (m/s)	approach velocity
6.07	6.30	yr (m)	water level rise from low water to flood stage
2.19	2.43	ys (m)	scoured flow depth below flood level
3.45	3.58	MAX ys	maximum scoured flow depth below flood level
0.00	0.00	yds	scoured depth below bed level (= MAX ys-y)
0.00	**0.00**	**yds (m)**	**scoured depth below bed level,incl 20% allowance for no safety factor inclusion in the method**

Scheme Design
SH1 Waikuruki Bridge (North Road, Awanui River)

Equivalent pier width calculation as in NZ Transport Agency *Bridge Manual* 3rd Edition, 2018

$$a_d^* = \frac{K_{d1}(TW)\left(L/y\right)^{K_{d2}} + (y - K_{d1}T)a}{y} \quad \text{for} \quad L/y > 1.0$$

$$a_d^* = \frac{K_{d1}(TW) + (y - K_{d1}T)a}{y} \quad \text{for} \quad L/y \leq 1.0$$

Rectangular debris raft

20yr	100yr	(m)	
3.00	3.00	W	width of debris normal to flow (average of the span widths, but not greater than length of largest tree likely to be transported
1.20	1.20	T	thickness of debris normal to flow (up to ~2m not greater than 3)
1.20	1.20	L	length of debris upstream from pier face. L to lie within range 0.4W to 1.3W
3.45	3.58	y	depth of approach flow (refer to general and contraction scour calculations)
0.35	0.34	L/y	
0.79	0.79	Kd1	for rectrangular debris
-0.79	-0.79	Kd2	for rectrangular debris
0.58	0.58	a	pier width (refer to local scoured depth estimation)
1.25	1.22	a*d	equivalent pier width

Triangular debris raft

20yr	100yr	(m)	
3.00	3.00	W	width of debris normal to flow (average of the span widths, but not greater than length of largest tree likely to be transported
1.20	1.20	T	thickness of debris normal to flow (up to ~2m not greater than 3)
1.20	1.20	L	length of debris upstream from pier face. L to lie within range 0.4W to 1.3W
3.45	3.58	y	depth of approach flow (refer to general and contraction scour calculations)
0.35	0.34	L/y	
0.21	0.21	Kd1	for triangular debris
-0.17	-0.17	Kd2	for triangular debris
0.58	0.58	a	pier width (refer to local scoured depth estimation)
0.76	0.75	a*d	equivalent pier width

1.25	**1.22**	**MAX a*d =be***	**equivalent size of the pier that induces about the same scour depth as the actual pier with accumulated debris**

Scheme Design

SH1 Waikuruki Bridge (North Road, Awanui River)

Local scoured depth estimation at pier as in *Bridge Scour* by Melville and Coleman, 2000

Melville Method (1997)

$$\boxed{d_s = K_{yb} K_I K_d K_s K_\theta K_t}$$

Hydraulics after general scour

20yr	100yr			
3.45	3.58	yms=MAX ys (m)	flow depth from flood level to the mean scoured bed level (refer to general and contraction scour calculations)	
0.002	0.002	d50	sediment size for which 50%of the sediment is finer; Mangakahia silt loam and clay loam	
0.50	0.50	b (m)	pier width	ASSUMED
9.10	9.10	l (m)	approx pier length	ASSUMED
-1.00	-1.00	Y` (m)	exposed foundation before scour	ASSUMED
0.70	0.70	b* (m)	foundation width	ASSUMED
0.58	0.58	be (m)	equivalent size for non uniform piers , slab footing Case III (refer to Figure 7.12 in *Bridge Scour*)	
1.25	1.22	be* (m)	equivalent pier width calculated to *Bridge Manual*	
0.01	0.01	uc	(from Figure 6.6 in *Bridge Scour*)	
22.88	22.97	Vc/uc	(from Figure 6.6 in *Bridge Scour*)	
0.81	0.92	V (m/s)	mean channel velocity	
0.26	0.26	Vc (m/s)	competent velocity	
3.09	3.48	Velocity parameter	V/Vc<1 for clear-water scour conditions; V/Vc >1 for live-bed scour conditions	

Local scoured depth estimation at piers as in *Bridge Scour (Section 6.3)*

20yr	100yr			
0.36	0.34	be*/yms	<0.7	
2.99	2.93	Kyb	depth-size factor	
1.00	1.00	Kl	flow intensity for live bed conditons	
1.00	1.00	Kd	sediment size factor (assumed)	
0.00	0.00	theta	(approx alignment); approach flow in line with piles	
1.10	1.10	Ks	pier or abutment shape	ASSUMED
1.00	1.00	Ktheta	pier or abutment alignment	
1.00	1.00	Kt	time factor (under live bed conditions)	
3.29	**3.22**	**ds (m)**	**local scoured depth below bed level**	

3.29	**3.22**	**Tds=yds+ds (m)**	**Total scoured depth below bed level**

Scheme Design
Donald Road Bridge (Whangatane spillway)

The New Zealand Railway Formulation of Holmes (1974) as in *Bridge Scour* by Melville and Coleman, 2000
The method combines general scour and contraction scour

New Zealand Railways (Holmes, 1974)[8]	$K = \sqrt{\dfrac{W}{4.83Q^{0.5}}} \leq 1$ $V_1 = C\left(\dfrac{Q}{A}\right)\left(\dfrac{y}{A/W}\right)^{2/3}$ Max. of $y_s = y$ or $y_s = \dfrac{y_r V_1 K}{\sqrt{(A/W)}}$

Hydraulic Calculations

20yr	100yr		
104.90	105.20	A (m2)	flow area of the unscoured profile
34.00	35.00	m	flood channel width (assumed)
40.16	40.16	m	bridge (main)channel width
2.61	2.62	y (m)	hydraulic depth
5.29	5.67	m	approach flow depth
2.05	2.35	V (m/s)	mean channel velocity
0.01	0.01	uc	(from Figure 6.6 in *Bridge Scour*)
13.69	13.70	Vc/uc	(from Figure 6.6 in *Bridge Scour*)
0.16	0.16	Vc (m/s)	competent velocity
0.50	0.50	m	unscoured flow depth/low flow ASSUMED
0.06	0.06	d50	sediment size for which 50%of the sediment is finer Mangakahia mottled clay loam

Scoured Depth Calculations

20yr	100yr		
215.30	247.00	Q (m3/s)	flow rate
40.16	40.16	W (m)	waterway width allowing for bem flow, taken as W80*1.25. W80 is waterway width, including the main channel which carries 80% of the flow; W80*1.25 - in this case similar to bridge opening width
0.75	0.73	K	factor dependent on waterway width and the Lacey regime width
1.00	1.00	C	coefficient; 1.2 if converging flows are encountered; 1.0 in other cases
2.05	2.35	V1 (m/s)	approach velocity
4.79	5.17	yr (m)	water level rise from low water to flood stage
4.58	5.46	ys (m)	scoured flow depth below flood level
4.58	5.46	MAX ys	maximum scoured flow depth below flood level
1.97	2.84	yds	scoured depth below bed level (= MAX ys-y)
2.36	**3.40**	**yds (m)**	**scoured depth below bed level,incl 20% allowance for no safety factor inclusion in the method**

Scheme Design
Donald Road Bridge (Whangatane spillway)

Equivalent pier width calculation as in NZ Transport Agency *Bridge Manual* 3rd Edition, 2018

$$a_d^* = \frac{K_{d1}(TW)(L/y)^{K_{d2}} + (y - K_{d1}T)a}{y} \quad \text{for} \quad L/y > 1.0$$

$$a_d^* = \frac{K_{d1}(TW) + (y - K_{d1}T)a}{y} \quad \text{for} \quad L/y \leq 1.0$$

Rectangular debris raft

20yr	100yr		
		(m)	
3.00	3.00	W	width of debris normal to flow (average of the span widths, but not greater than length of largest tree likely to be transported
1.20	1.20	T	thickness of debris normal to flow (up to ~2m not greater than 3)
1.20	1.20	L	length of debris upstream from pier face. L to lie within range 0.4W to 1.3W
4.58	5.46	y	depth of approach flow (refer to general and contraction scour calculations)
0.26	0.22	L/y	
0.79	0.79	Kd1	for rectrangular debris
-0.79	-0.79	Kd2	for rectrangular debris
0.50	0.50	a	pier width (refer to local scoured depth estimation)
1.02	0.93	a*d	equivalent pier width

Triangular debris raft

20yr	100yr		
		(m)	
3.00	3.00	W	width of debris normal to flow (average of the span widths, but not greater than length of largest tree likely to be transported
1.20	1.20	T	thickness of debris normal to flow (up to ~2m not greater than 3)
1.20	1.20	L	length of debris upstream from pier face. L to lie within range 0.4W to
4.58	5.46	y	depth of approach flow (refer to general and contraction scour calculations)
0.26	0.22	L/y	
0.21	0.21	Kd1	for triangular debris
-0.17	-0.17	Kd2	for triangular debris
0.50	0.50	a	pier width (refer to local scoured depth estimation)
0.64	0.62	a*d	equivalent pier width
1.02	**0.93**	**MAX a*d =be***	**equivalent size of the pier that induces about the same scour depth as the actual pier with accumulated debris**

Scheme Design
Donald Road Bridge (Whangatane spillway)

Local scoured depth estimation at pier as in *Bridge Scour* by Melville and Coleman, 2000
Melville Method (1997)

$$\boxed{d_s = K_{yb}K_I K_d K_s K_\theta K_t}$$

Hydraulics after general scour

20yr	100yr		
4.58	5.46	yms=MAX ys (m)	flow depth from flood level to the mean scoured bed level (refer to general and contraction scour calculations)
0.06	0.06	d50 (m)	sediment size for which 50%of the sediment is finer; Mangakahia clay loam
0.50	0.50	b=be (m)	pier width ASSUMED
1.02	0.93	be* (m)	equivalent pier width calculated to *Bridge Manual*
0.01	0.01	uc	(from Figure 6.6 in *Bridge Scour*)
15.10	15.53	Vc/uc	(from Figure 6.6 in *Bridge Scour*)
1.17	1.13	V (m/s)	mean channel velocity
0.18	0.18	Vc (m/s)	competent velocity
6.60	6.18	Velocity parameter	V/Vc<1 for clear-water scour conditions; V/Vc >1 for live-bed scour conditions

Local scoured depth estimation at piers as in *Bridge Scour (Section 6.3)*

20yr	100yr		
0.22	0.17	be*/yms	<0.7
2.44	2.24	Kyb	depth-size factor
1	1	Kl	flow intensity for live bed conditons
1.00	1.00	Kd	sediment size factor (assumed)
0	0	theta	(approx alignment); approach flow in line with piles
1	1	Ks*Ktheta	For a pile group (Sp=pipe spacing, Dp=pipe dia; Sp/D; Table 6.4 in Bridge Scour for a single row of piles;

Sp	8.5	ASSUMED
Dp	0.50	
Sp/Dp	17	

1	1	Kt	time factor (under live bed conditions)
2.44	**2.24**	**ds (m)**	**local scoured depth below bed level**
4.80	**5.645**	**Tds=yds+ds (m)**	**Total scoured depth below bed level**

Scheme Design
Quarry Road Bridge (Whangatane spillway)

The New Zealand Railway Formulation of Holmes (1974) as in *Bridge Scour* by Melville and Coleman, 2000

The method combines general scour and contraction scour

New Zealand Railways (Holmes, 1974)[8]	$K = \sqrt{\dfrac{W}{4.83Q^{0.5}}} \leq 1$ $V_1 = C\left(\dfrac{Q}{A}\right)\left(\dfrac{y}{A/W}\right)^{2/3}$ Max. of $y_s = y$ or $y_s = \dfrac{y_r V_1 K}{\sqrt{(A/W)}}$

Hydraulic Calculations

20yr	100yr		
149.70	149.80	A (m2)	flow area of the unscoured profile
52.00	52.00	m	flood channel width (assumed)
52.00	52.00	m	bridge (main)channel width
2.88	2.88	y (m)	hydraulic depth
6.58	6.66	m	approach flow depth
1.35	1.41	V (m/s)	mean channel velocity
0.01	0.01	uc	(from Figure 6.6 in *Bridge Scour*)
13.94	13.94	Vc/uc	(from Figure 6.6 in *Bridge Scour*)
0.16	0.16	Vc (m/s)	competent velocity
0.50	0.50	m	unscoured flow depth/low flow ASSUMED
0.06	0.06	d50	sediment size for which 50%of the sediment is finer Mangakahia mottled clay loam

Scoured Depth Calculations

20yr	100yr		
201.50	211.80	Q (m3/s)	flow rate
52.00	52.00	W (m)	waterway width allowing for bem flow, taken as W80*1.25. W80 is waterway width, including the main channel which carries 80% of the flow; W80*1.25 - in this case similar to bridge opening width
0.87	0.86	K	factor dependent on waterway width and the Lacey regime width
1.00	1.00	C	coefficient; 1.2 if converging flows are encountered; 1.0 in other cases
1.35	1.41	V1 (m/s)	approach velocity
6.08	6.16	yr (m)	water level rise from low water to flood stage
4.20	4.41	ys (m)	scoured flow depth below flood level
4.20	4.41	MAX ys	maximum scoured flow depth below flood level
1.32	1.53	yds	scoured depth below bed level (= MAX ys-y)
1.59	**1.84**	**yds (m)**	**scoured depth below bed level,incl 20% allowance for no safety factor inclusion in the method**

Scheme Design
Quarry Road Bridge (Whangatane spillway)

Equivalent pier width calculation as in NZ Transport Agency *Bridge Manual* 3rd Edition, 2018

$$a_d^* = \frac{K_{d1}(TW)(L/y)^{K_{d2}} + (y - K_{d1}T)a}{y} \quad \text{for} \quad L/y > 1.0$$

$$a_d^* = \frac{K_{d1}(TW) + (y - K_{d1}T)a}{y} \quad \text{for} \quad L/y \leq 1.0$$

Rectangular debris raft

20yr	100yr	(m)	
3.00	3.00	W	width of debris normal to flow (average of the span widths, but not greater than length of largest tree likely to be transported
1.20	1.20	T	thickness of debris normal to flow (up to ~2m not greater than 3)
1.20	1.20	L	length of debris upstream from pier face. L to lie within range 0.4W to 1.3W
4.20	4.41	y	depth of approach flow (refer to general and contraction scour calculations)
0.29	0.27	L/y	
0.79	0.79	Kd1	for rectrangular debris
-0.79	-0.79	Kd2	for rectrangular debris
0.65	0.64	a	pier width (refer to local scoured depth estimation)
1.18	1.15	a*d	equivalent pier width

Triangular debris raft

20yr	100yr	(m)	
3.00	3.00	W	width of debris normal to flow (average of the span widths, but not greater than length of largest tree likely to be transported
1.20	1.20	T	thickness of debris normal to flow (up to ~2m not greater than 3)
1.20	1.20	L	length of debris upstream from pier face. L to lie within range 0.4W to 1.3W
4.20	4.41	y	depth of approach flow (refer to general and contraction scour calculations)
0.29	0.27	L/y	
0.21	0.21	Kd1	for triangular debris
-0.17	-0.17	Kd2	for triangular debris
0.65	0.64	a	pier width (refer to local scoured depth estimation)
0.79	0.78	a*d	equivalent pier width
1.18	**1.15**	**MAX a*d =be***	**equivalent size of the pier that induces about the same scour depth as the actual pier with accumulated debris**

Scheme Design
Quarry Road Bridge (Whangatane spillway)

Local scoured depth estimation at pier as in *Bridge Scour* by Melville and Coleman, 2000
Melville Method (1997)

$$\boxed{d_s = K_{yb}K_I K_d K_s K_\theta K_t}$$

Hydraulics after general scour

20yr	100yr			
4.20	4.41	yms=MAX ys (m)	flow depth from flood level to the mean scoured bed level (refer to general and contraction scour calculations)	
0.06	0.06	d50 (m)	sediment size for which 50%of the sediment is finer; Mangakahia clay loam	
0.5	0.5	b (m)	pier width	ASSUMED
8.20	8.20	l (m)	approx pier length	ASSUMED
-1.00	-1.00	Y` (m)	exposed foundation before scour	ASSUMED
0.90	0.90	b* (m)	foundation width	ASSUMED
0.65	0.64	be (m)	equivalent size for non uniform piers; non uniform, downward tapering Case III (refer to Figure 7.12 in *Bridge Scour*)	
1.18	1.15	be* (m)	equivalent pier width calculated to *Bridge Manual*	
0.01	0.01	uc	(from Figure 6.6 in *Bridge Scour*)	
14.88	15.00	Vc/uc	(from Figure 6.6 in *Bridge Scour*)	
0.92	0.92	V (m/s)	mean channel velocity	
0.17	0.18	Vc (m/s)	competent velocity	
5.28	5.24	Velocity parameter	V/Vc<1 for clear-water scour conditions; V/Vc >1 for live-bed scour conditions	

Local scoured depth estimation at piers as in *Bridge Scour (Section 6.3)*

20yr	100yr			
0.28	0.26	be*/yms	<0.7	
2.83	2.76	Kyb	depth-size factor	
1.00	1.00	KI	flow intensity for live bed conditons	
1.00	1.00	Kd	sediment size factor (assumed)	
0.00	0.00	theta	(approx alignment); approach flow in line with piles	
1.10	1.10	Ks	pier or abutment shape	ASSUMED
1.00	1.00	Ktheta	pier or abutment alignment	
1.00	1.00	Kt	time factor (under live bed conditions)	
3.11	**3.03**	**ds (m)**	**local scoured depth below bed level**	

4.70	**4.87**	**Tds=yds+ds (m)**	**Total scoured depth below bed level**

Scheme Design
SH10 Bridge (Whangatane spillway)

The New Zealand Railway Formulation of Holmes (1974) as in *Bridge Scour* by Melville and Coleman, 2000

The method combines general scour and contraction scour

New Zealand Railways (Holmes, 1974)[8]	$K = \sqrt{\dfrac{W}{4.83Q^{0.5}}} \le 1$ $V_1 = C\left(\dfrac{Q}{A}\right)\left(\dfrac{y}{A/W}\right)^{2/3}$ Max. of $y_s = y$ or $y_s = \dfrac{y_r V_1 K}{\sqrt{(A/W)}}$

Hydraulic Calculations

20yr	100yr		
123.20	123.20	A (m2)	flow area of the unscoured profile
38.20	38.20	m	flood channel width (assumed)
38.20	38.20	m	bridge (main)channel width
3.23	3.23	y (m)	hydraulic depth
5.30	5.39	m	approach flow depth
1.51	1.55	V (m/s)	mean channel velocity
0.01	0.01	uc	(from Figure 6.6 in *Bridge Scour*)
14.22	14.22	Vc/uc	(from Figure 6.6 in *Bridge Scour*)
0.17	0.17	Vc (m/s)	competent velocity
0.50	0.50	m	unscoured flow depth/low flow ASSUMED
0.06	0.06	d50	sediment size for which 50%of the sediment is finer Mangakahia clay loam

Scoured Depth Calculations

20yr	100yr		
186.00	190.60	Q (m3/s)	flow rate
38.20	38.20	W (m)	waterway width allowing for bem flow, taken as W80*1.25. W80 is waterway width, including the main channel which carries 80% of the flow; W80*1.25 - in this case similar to bridge opening width
0.76	0.76	K	factor dependent on waterway width and the Lacey regime width
1.00	1.00	C	coefficient; 1.2 if converging flows are encountered; 1.0 in other cases
1.51	1.55	V1 (m/s)	approach velocity
4.80	4.89	yr (m)	water level rise from low water to flood stage
3.07	3.19	ys (m)	scoured flow depth below flood level
3.23	3.23	MAX ys	maximum scoured flow depth below flood level
0.00	0.00	yds	scoured depth below bed level (= MAX ys-y)
0.00	**0.00**	**yds (m)**	**scoured depth below bed level,incl 20% allowance for no safety factor inclusion in the method**

Scheme Design
SH10 Bridge (Whangatane spillway)

Equivalent pier width calculation as in NZ Transport Agency *Bridge Manual* 3rd Edition, 2018

$$a_d^* = \frac{K_{d1}(TW)(L/y)^{K_{d2}} + (y - K_{d1}T)a}{y} \quad \text{for} \quad L/y > 1.0$$

$$a_d^* = \frac{K_{d1}(TW) + (y - K_{d1}T)a}{y} \quad \text{for} \quad L/y \leq 1.0$$

Rectangular debris raft

20yr	100yr	(m)	
3.00	3.00	W	width of debris normal to flow (average of the span widths, but not greater than length of largest tree likely to be transported
1.20	1.20	T	thickness of debris normal to flow (up to ~2m not greater than 3)
1.20	1.20	L	length of debris upstream from pier face.L to lie within range 0.4W to 1.3W
3.23	3.23	y	depth of approach flow (refer to general and contraction scour calculations)
0.37	0.37	L/y	
0.79	0.79	Kd1	for rectrangular debris
-0.79	-0.79	Kd2	for rectrangular debris
0.24	0.24	a	pier width (refer to local scoured depth estimation)
1.05	1.05	a*d	equivalent pier width

Triangular debris raft

20yr	100yr	(m)	
3.00	3.00	W	width of debris normal to flow (average of the span widths, but not greater than length of largest tree likely to be transported
1.20	1.20	T	thickness of debris normal to flow (up to ~2m not greater than 3)
1.20	1.20	L	length of debris upstream from pier face.L to lie within range 0.4W to 1.3W
3.23	3.23	y	depth of approach flow (refer to general and contraction scour calculations)
0.37	0.37	L/y	
0.21	0.21	Kd1	for triangular debris
-0.17	-0.17	Kd2	for triangular debris
0.24	0.24	a	pier width (refer to local scoured depth estimation)
0.45	0.45	a*d	equivalent pier width
1.05	**1.05**	**MAX a*d =be***	**equivalent size of the pier that induces about the same scour depth as the actual pier with accumulated debris**

Scheme Design
SH10 Bridge (Whangatane spillway)

Local scoured depth estimation at pier as in *Bridge Scour* by Melville and Coleman, 2000
Melville Method (1997)

$$d_s = K_{yb} K_I K_d K_s K_\theta K_t$$

Hydraulics after general scour

20yr	100yr			
3.23	3.23	yms=MAX ys (m)	flow depth from flood level to the mean scoured bed level (refer to general and contraction scour calculations)	
0.06	0.06	d50 (m)	sediment size for which 50%of the sediment is finer; Mangakahia clay loam	
0.30	0.30	b (m)	pier width	SURVEYED
9.30	9.30	l (m)	approx pier length	ASSUMED
1.50	1.50	Y` (m)	exposed foundation before scour	ASSUMED
1.20	1.20	b* (m)	foundation width	SURVEYED
0.24	0.24	be (m)	equivalent size for non uniform piers, slab footing Case II (refer to Figure 7.12 in *Bridge Scour*)	
1.05	1.05	be* (m)	equivalent pier width calculated to *Bridge Manual*	
0.01	0.01	uc	(from Figure 6.6 in *Bridge Scour*)	
14.22	14.22	Vc/uc	(from Figure 6.6 in *Bridge Scour*)	
1.51	1.55	V (m/s)	mean channel velocity	
0.17	0.17	Vc (m/s)	competent velocity	
9.04	9.26	Velocity parameter	V/Vc<1 for clear-water scour conditions; V/Vc >1 for live-bed scour conditions	

Local scoured depth estimation at piers as in *Bridge Scour (Section 6.3)*

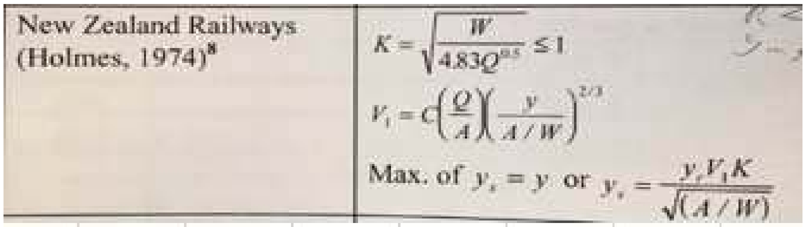
20yr	100yr			
0.33	0.33	be*/yms	<0.7	
2.52	2.52	Kyb	depth-size factor	
1.00	1.00	Kl	flow intensity for live bed conditons	
1.00	1.00	Kd	sediment size factor (assumed)	
0.00	0.00	theta	(approx alignment); approach flow in line with piles	
1.10	1.10	Ks	pier or abutment shape	ASSUMED
1.00	1.00	Ktheta	pier or abutment alignment	ASSUMED
1.00	1.00	Kt	time factor (under live bed conditions)	
2.77	**2.77**	**ds (m)**	**local scoured depth below bed level**	
2.77	**2.77**	**Tds=yds+ds (m)**	**Total scoured depth below bed level**	

B4.5 Bridge scoured depth calculations. Baseline scenario

Baseline Sceanrio
Church Road Bridge (Awanui River)

The New Zealand Railway Formulation of Holmes (1974) as in *Bridge Scour* by Melville and Coleman, 2000

The method combines general scour and contraction scour



New Zealand Railways (Holmes, 1974)[8]	$K = \sqrt{\frac{W}{4.83Q^{0.5}}} \leq 1$ $V_1 = C\left(\frac{Q}{A}\right)\left(\frac{y}{A/W}\right)^{2/3}$ Max. of $y_s = y$ or $y_s = \frac{y_r V_1 K}{\sqrt{(A/W)}}$

Hydraulic Calculations

20yr	100yr		
135.90	147.30	A (m2)	flow area of the unscoured profile
22.00	22.00	m	flood channel width (assumed)(from abutment to abutment)
34.80	34.80	m	bridge (main)channel width
3.91	4.23	y (m)	hydraulic depth
7.41	7.78	m	approach flow depth
1.84	1.93	V (m/s)	mean channel velocity
0.01	0.01	uc	(from Figure 6.6 in B*ridge Scour*)
14.70	14.90	Vc/uc	(from Figure 6.6 in B*ridge Scour*)
0.17	0.17	Vc (m/s)	competent velocity
0.50	0.50	m	unscoured flow depth/low flow ASSUMED
0.06	0.06	d50	sediment size for which 50%of the sediment is finer Mangakahia mottled clay loam

Scoured Depth Calculations

20yr	100yr		
249.80	284.30	Q (m3/s)	flow rate
34.80	34.80	W (m)	waterway width allowing for bem flow, taken as W80*1.25. W80 is waterway width, including the main channel which carries 80% of the flow; W80*1.25 - in this case similar to bridge opening width
0.68	0.65	K	factor dependent on waterway width and the Lacey regime width
1.00	1.00	C	coefficient; 1.2 if converging flows are encountered; 1.0 in other cases
1.84	1.93	V1 (m/s)	approach velocity
6.91	7.28	yr (m)	water level rise from low water to flood stage
4.34	4.46	ys (m)	scoured flow depth below flood level
4.34	4.46	MAX ys	maximum scoured flow depth below flood level
0.43	0.23	Tds	Total scoured depth below bed level (= MAX ys-y)
0.52	**0.28**	**Tds (m)**	**Total scoured depth below bed level,incl 20% allowance for no safety factor inclusion in the method**

Baseline Scenario
Allen Bell Drive Bridge (Awanui River)

The New Zealand Railway Formulation of Holmes (1974) as in *Bridge Scour* by Melville and Coleman, 2000

The method combines general scour and contraction scour

New Zealand Railways (Holmes, 1974)[8]	$K = \sqrt{\frac{W}{4.83Q^{0.5}}} \leq 1$ $V_1 = C\left(\frac{Q}{A}\right)\left(\frac{y}{A/W}\right)^{2/3}$ Max. of $y_s = y$ or $y_s = \frac{y_r V_1 K}{\sqrt{(A/W)}}$

Hydraulic Calculations

20yr	100yr		
160.70	161.00	A (m2)	flow area of the unscoured profile
23.20	23.20	m	flood channel width (assumed)(abutment to abutment)
34.00	34.00	m	bridge (main)channel width
4.73	4.74	y (m)	hydraulic depth
7.84	8.15	m	approach flow depth
1.55	1.75	V (m/s)	mean channel velocity
0.01	0.01	uc	(from Figure 6.6 in *Bridge Scour*)
15.17	15.18	Vc/uc	(from Figure 6.6 in *Bridge Scour*)
0.18	0.18	Vc (m/s)	competent velocity
0.50	0.50	m	unscoured flow depth/low flow
0.06	0.06	d50	sediment size for which 50%of the sediment is finer Mangakahia mottled clay loam

Scoured Depth Calculations

20yr	100yr		
248.90	281.50	Q (m3/s)	flow rate
34.00	34.00	W (m)	waterway width allowing for bem flow, taken as W80*1.25. W80 is waterway width, including the main channel which carries 80% of the flow; W80*1.25 - in this case similar to bridge opening width
0.67	0.65	K	factor dependent on waterway width and the Lacey regime width
1.00	1.00	C	coefficient; 1.2 if converging flows are encountered; 1.0 in other cases
1.55	1.75	V1 (m/s)	approach velocity
7.34	7.65	yr (m)	water level rise from low water to flood stage
3.49	3.98	ys (m)	scoured flow depth below flood level
4.73	4.74	MAX ys	maximum scoured flow depth below flood level
0.00	0.00	Tds	Total scoured depth below bed level (= MAX ys-y)
0.00	**0.00**	**Tds (m)**	**Total scoured depth below bed level,incl 20% allowance for no safety factor inclusion in the method**

Baseline Sceanrio
SH1 Waikuruki Bridge (North Road, Awanui River)

The New Zealand Railway Formulation of Holmes (1974) as in *Bridge Scour* by Melville and Coleman, 2000
The method combines general scour and contraction scour

New Zealand Railways (Holmes, 1974)[8]	$K = \sqrt{\dfrac{W}{4.83Q^{0.5}}} \leq 1$ $V_1 = C\left(\dfrac{Q}{A}\right)\left(\dfrac{y}{A/W}\right)^{2/3}$ Max. of $y_s = y$ or $y_s = \dfrac{y_r V_1 K}{\sqrt{(A/W)}}$

Hydraulic Calculations

20yr	100yr		
99.40	103.90	A (m2)	flow area of the unscoured profile
28.50	29.50	m	flood channel width (assumed)
30.33	30.33	m	bridge (main)channel width
3.28	3.43	y (m)	hydraulic depth
6.38	6.54	m	approach flow depth
0.76	0.80	V (m/s)	mean channel velocity
0.01	0.01	uc	(from Figure 6.6 in *Bridge Scour*)
22.75	22.86	Vc/uc	(from Figure 6.6 in *Bridge Scour*)
0.26	0.26	Vc (m/s)	competent velocity
0.50	0.50	m	unscoured flow depth/low flow ASSUMED
0.002	0.002	d50	sediment size for which 50%of the sediment is finer Mangakahia silt loam and clay loam

Scoured Depth Calculations

20yr	100yr		
75.2	83.4	Q (m3/s)	flow rate
30.33	30.33	W (m)	waterway width allowing for bem flow, taken as W80*1.25. W80 is waterway width, including the main channel which carries 80% of the flow; W80*1.25 - in this case similar to bridge opening width
0.85	0.83	K	factor dependent on waterway width and the Lacey regime width
1	1	C	coefficient; 1.2 if converging flows are encountered; 1.0 in other cases
0.76	0.80	V1 (m/s)	approach velocity
5.88	6.04	yr (m)	water level rise from low water to flood stage
2.09	2.17	ys (m)	scoured flow depth below flood level
3.28	3.43	MAX ys	maximum scoured flow depth below flood level
0.00	0.00	yds	scoured depth below bed level (= MAX ys-y)
0.00	**0.00**	**yds (m)**	**scoured depth below bed level,incl 20% allowance for no safety factor inclusion in the method**

Baseline Scenario
SH1 Waikuruki Bridge (North Road, Awanui River)

Equivalent pier width calculation as in NZ Transport Agency *Bridge Manual* 3rd Edition, 2018

$$a_d^* = \frac{K_{d1}(TW)(L/y)^{K_{d2}} + (y - K_{d1}T)a}{y} \quad \text{for} \quad L/y > 1.0$$

$$a_d^* = \frac{K_{d1}(TW) + (y - K_{d1}T)a}{y} \quad \text{for} \quad L/y \le 1.0$$

Rectangular debris raft

20yr	100yr	(m)	
3.00	3.00	W	width of debris normal to flow (average of the span widths, but not greater than length of largest tree likely to be transported
1.20	1.20	T	thickness of debris normal to flow (up to ~2m not greater than 3)
1.20	1.20	L	length of debris upstream from pier face.L to lie within range 0.4W to 1.3W
3.28	3.43	y	depth of approach flow (refer to general and contraction scour calculations)
0.37	0.35	L/y	
0.79	0.79	Kd1	for rectrangular debris
-0.79	-0.79	Kd2	for rectrangular debris
0.59	0.58	a	pier width (refer to local scoured depth estimation)
1.28	1.25	a*d	equivalent pier width

Triangular debris raft

20yr	100yr	(m)	
3.00	3.00	W	width of debris normal to flow (average of the span widths, but not greater than length of largest tree likely to be transported
1.20	1.20	T	thickness of debris normal to flow (up to ~2m not greater than 3)
1.20	1.20	L	length of debris upstream from pier face.L to lie within range 0.4W to 1.3W
3.28	3.43	y	depth of approach flow (refer to general and contraction scour calculations)
0.37	0.35	L/y	
0.21	0.21	Kd1	for triangular debris
-0.17	-0.17	Kd2	for triangular debris
0.59	0.58	a	pier width (refer to local scoured depth estimation)
0.77	0.76	a*d	equivalent pier width
1.28	**1.25**	**MAX a*d =be***	**equivalent size of the pier that induces about the same scour depth as the actual pier with accumulated debris**

Baseline Scenario

SH1 Waikuruki Bridge (North Road, Awanui River)

Local scoured depth estimation at pier as in *Bridge Scour* by Melville and Coleman, 2000

Melville Method (1997)

$$\boxed{d_s = K_{yb}K_I K_d K_s K_\theta K_t}$$

Hydraulics after general scour

20yr	100yr			
3.28	3.43	yms=MAX ys (m)	flow depth from flood level to the mean scoured bed level (refer to general and contraction scour calculations)	
0.002	0.002	d50	sediment size for which 50%of the sediment is finer; Mangakahia silt loam and clay loam	
0.50	0.50	b (m)	pier width	ASSUMED
9.10	9.10	l (m)	approx pier length	ASSUMED
-1.00	-1.00	Y` (m)	exposed foundation before scour	ASSUMED
0.70	0.70	b* (m)	foundation width	ASSUMED
0.59	0.58	be (m)	equivalent size for non uniform piers , slab footing Case III (refer to Figure 7.12 in *Bridge Scour)*	
1.28	1.25	be* (m)	equivalent pier width calculated to *Bridge Manual*	
0.01	0.01	uc	(from Figure 6.6 in *Bridge Scour*)	
22.75	22.86	Vc/uc	(from Figure 6.6 in *Bridge Scour*)	
0.76	0.80	V (m/s)	mean channel velocity	
0.26	0.26	Vc (m/s)	competent velocity	
2.89	3.05	Velocity parameter	V/Vc<1 for clear-water scour conditions; V/Vc >1 for live-bed scour conditions	

Local scoured depth estimation at piers as in *Bridge Scour (Section 6.3)*

20yr	100yr			
0.39	0.37	be*/yms	<0.7	
3.08	3.00	Kyb	depth-size factor	
1.00	1.00	Kl	flow intensity for live bed conditons	
1.00	1.00	Kd	sediment size factor (assumed)	
0.00	0.00	theta	(approx alignment); approach flow in line with piles	
1.10	1.10	Ks	pier or abutment shape	ASSUMED
1.00	1.00	Ktheta	pier or abutment alignment	
1.00	1.00	Kt	time factor (under live bed conditions)	
3.39	**3.30**	**ds (m)**	**local scoured depth below bed level**	

3.39	**3.30**	**Tds=yds+ds (m)**	**Total scoured depth below bed level**

Baseline Scenario
Donald Road Bridge (Whangatane spillway)

The New Zealand Railway Formulation of Holmes (1974) as in *Bridge Scour* by Melville and Coleman, 2000

The method combines general scour and contraction scour

New Zealand Railways (Holmes, 1974)[8]	$K = \sqrt{\frac{W}{4.83Q^{0.5}}} \leq 1$ $V_1 = C\left(\frac{Q}{A}\right)\left(\frac{y}{A/W}\right)^{2/3}$ Max. of $y_s = y$ or $y_s = \frac{y_r V_1 K}{\sqrt{(A/W)}}$

Hydraulic Calculations

20yr	100yr		
105.20	105.20	A (m2)	flow area of the unscoured profile
35.00	35.00	m	flood channel width (assumed)
40.16	40.16	m	bridge (main)channel width
2.62	2.62	y (m)	hydraulic depth
5.45	5.60	m	approach flow depth
1.64	1.83	V (m/s)	mean channel velocity
0.01	0.01	uc	(from Figure 6.6 in *Bridge Scour*)
13.70	13.70	Vc/uc	(from Figure 6.6 in *Bridge Scour*)
0.16	0.16	Vc (m/s)	competent velocity
0.50	0.50	m	unscoured flow depth/low flow ASSUMED
0.06	0.06	d50	sediment size for which 50%of the sediment is finer Mangakahia mottled clay loam

Scoured Depth Calculations

20yr	100yr		
173.00	192.00	Q (m3/s)	flow rate
40.16	40.16	W (m)	waterway width allowing for bem flow, taken as W80*1.25. W80 is waterway width, including the main channel which carries 80% of the flow; W80*1.25 - in this case similar to bridge opening width
0.80	0.77	K	factor dependent on waterway width and the Lacey regime width
1.00	1.00	C	coefficient; 1.2 if converging flows are encountered; 1.0 in other cases
1.64	1.83	V1 (m/s)	approach velocity
4.95	5.10	yr (m)	water level rise from low water to flood stage
4.00	4.45	ys (m)	scoured flow depth below flood level
4.00	4.45	MAX ys	maximum scoured flow depth below flood level
1.38	1.84	yds	scoured depth below bed level (= MAX ys-y)
1.66	**2.20**	**yds (m)**	**scoured depth below bed level,incl 20% allowance for no safety factor inclusion in the method**

Baseline Scenario
Donald Road Bridge (Whangatane spillway)

Equivalent pier width calculation as in NZ Transport Agency *Bridge Manual* 3rd Edition, 2018

$$a_d^* = \frac{K_{d1}(TW)(L/y)^{K_{d2}} + (y - K_{d1}T)a}{y} \quad \text{for} \quad L/y > 1.0$$

$$a_d^* = \frac{K_{d1}(TW) + (y - K_{d1}T)a}{y} \quad \text{for} \quad L/y \leq 1.0$$

Rectangular debris raft

20yr	100yr		
		(m)	
3.00	3.00	W	width of debris normal to flow (average of the span widths, but not greater than length of largest tree likely to be transported
1.20	1.20	T	thickness of debris normal to flow (up to ~2m not greater than 3)
1.20	1.20	L	length of debris upstream from pier face.L to lie within range 0.4W to 1.3W
4.00	4.45	y	depth of approach flow (refer to general and contraction scour calculations)
0.30	0.27	L/y	
0.79	0.79	Kd1	for rectrangular debris
-0.79	-0.79	Kd2	for rectrangular debris
0.50	0.50	a	pier width ASSUMED
1.09	1.03	a*d	equivalent pier width

Triangular debris raft

20yr	100yr		
		(m)	
3.00	3.00	W	width of debris normal to flow (average of the span widths, but not greater than length of largest tree likely to be transported
1.20	1.20	T	thickness of debris normal to flow (up to ~2m not greater than 3)
1.20	1.20	L	length of debris upstream from pier face.L to lie within range 0.4W to
4.00	4.45	y	depth of approach flow (refer to general and contraction scour calculations)
0.30	0.27	L/y	
0.21	0.21	Kd1	for triangular debris
-0.17	-0.17	Kd2	for triangular debris
0.50	0.50	a	pier width ASSUMED
0.66	0.64	a*d	equivalent pier width
1.09	**1.03**	**MAX a*d =be***	**equivalent size of the pier that induces about the same scour depth as the actual pier with accumulated debris**

Baseline Scenario
Donald Road Bridge (Whangatane spillway)

Local scoured depth estimation at pier as in *Bridge Scour* by Melville and Coleman, 2000
Melville Method (1997)

$$\boxed{d_s = K_{yb} K_I K_d K_s K_\theta K_t}$$

Hydraulics after general scour

20yr	100yr		
4.00	4.45	yms=MAX ys (m)	flow depth from flood level to the mean scoured bed level (refer to general and contraction scour calculations)
0.06	0.06	d50 (m)	sediment size for which 50%of the sediment is finer; Mangakahia clay loam
0.50	0.50	b=be (m)	pier width ASSUMED
1.09	1.03	be* (m)	equivalent pier width calculated to *Bridge Manual*
0.01	0.01	uc	(from Figure 6.6 in *Bridge Scour*)
14.76	15.03	Vc/uc	(from Figure 6.6 in *Bridge Scour*)
1.08	1.07	V (m/s)	mean channel velocity
0.17	0.18	Vc (m/s)	competent velocity
6.22	6.08	Velocity parameter	V/Vc<1 for clear-water scour conditions; V/Vc >1 for live-bed scour conditions

Local scoured depth estimation at piers as in *Bridge Scour (Section 6.3)*

20yr	100yr		
0.27	0.23	be*/yms	<0.7
2.62	2.48	Kyb	depth-size factor
1	1	Kl	flow intensity for live bed conditons
1	1	Kd	sediment size factor (assumed)
0	0	theta	(approx alignment); approach flow in line with piles
1	1	Ks*Ktheta	For a pile group (Sp=pipe spacing, Dp=pipe dia; Sp/D; Table 6.4 in *Bridge Scour* for a single row of piles;

Sp	8.5	ASSUMED
Dp	0.50	
Sp/Dp	17	

1	1	Kt	time factor (under live bed conditions)
2.62	**2.48**	**ds (m)**	**local scoured depth below bed level**
4.28	**4.68**	**Tds=yds+ds (m)**	**Total scoured depth below bed level**

Baseline Scenario
Quarry Road Bridge (Whangatane spillway)

The New Zealand Railway Formulation of Holmes (1974) as in *Bridge Scour* by Melville and Coleman, 2000
The method combines general scour and contraction scour

New Zealand Railways (Holmes, 1974)[8]	$K = \sqrt{\frac{W}{4.83Q^{0.5}}} \le 1$ $V_1 = C\left(\frac{Q}{A}\right)\left(\frac{y}{A/W}\right)^{2/3}$ Max. of $y_s = y$ or $y_s = \frac{y_r V_1 K}{\sqrt{(A/W)}}$

Hydraulic Calculations

20yr	100yr		
144.00	146.70	A (m2)	flow area of the unscoured profile
52.00	52.00	m	flood channel width (assumed)
51.63	51.63	m	bridge (main)channel width
2.79	2.84	y (m)	hydraulic depth
6.23	6.32	m	approach flow depth
1.16	1.21	V (m/s)	mean channel velocity
0.01	0.01	uc	(from Figure 6.6 in *Bridge Scour*)
13.86	13.90	Vc/uc	(from Figure 6.6 in *Bridge Scour*)
0.16	0.16	Vc (m/s)	competent velocity
0.50	0.50	m	unscoured flow depth/low flow ASSUMED
0.06	0.06	d50	sediment size for which 50%of the sediment is finer Mangakahia mottled clay loam

Scoured Depth Calculations

20yr	100yr		
166.90	177.30	Q (m3/s)	flow rate
51.63	51.63	W (m)	waterway width allowing for bem flow, taken as W80*1.25. W80 is waterway width, including the main channel which carries 80% of the flow; W80*1.25 - in this case similar to bridge opening width
0.91	0.90	K	factor dependent on waterway width and the Lacey regime width
1.00	1.00	C	coefficient; 1.2 if converging flows are encountered; 1.0 in other cases
1.16	1.21	V1 (m/s)	approach velocity
5.73	5.82	yr (m)	water level rise from low water to flood stage
3.62	3.74	ys (m)	scoured flow depth below flood level
3.62	3.74	MAX ys	maximum scoured flow depth below flood level
0.83	0.90	yds	scoured depth below bed level (= MAX ys-y)
0.99	**1.08**	**yds (m)**	**scoured depth below bed level,incl 20% allowance for no safety factor inclusion in the method**

Baseline Scenario
Quarry Road Bridge (Whangatane spillway)

Equivalent pier width calculation as in NZ Transport Agency *Bridge Manual* 3rd Edition, 2018

$$a_d^* = \frac{K_{d1}(TW)(L/y)^{K_{d2}} + (y - K_{d1}T)a}{y} \quad \text{for} \quad L/y > 1.0$$

$$a_d^* = \frac{K_{d1}(TW) + (y - K_{d1}T)a}{y} \quad \text{for} \quad L/y \leq 1.0$$

Rectangular debris raft

20yr	100yr		
		(m)	
3.00	3.00	W	width of debris normal to flow (average of the span widths, but not greater than length of largest tree likely to be transported
1.20	1.20	T	thickness of debris normal to flow (up to ~2m not greater than 3)
1.20	1.20	L	length of debris upstream from pier face.L to lie within range 0.4W to 1.3W
3.62	3.74	y	depth of approach flow (refer to general and contraction scour calculations)
0.33	0.32	L/y	
0.79	0.79	Kd1	for rectrangular debris
-0.79	-0.79	Kd2	for rectrangular debris
0.67	0.66	a	pier width (refer to local scoured depth estimation)
1.28	1.26	a*d	equivalent pier width

Triangular debris raft

20yr	100yr		
		(m)	
3.00	3.00	W	width of debris normal to flow (average of the span widths, but not greater than length of largest tree likely to be transported
1.20	1.20	T	thickness of debris normal to flow (up to ~2m not greater than 3)
1.20	1.20	L	length of debris upstream from pier face.L to lie within range 0.4W to 1.3W
3.62	3.74	y	depth of approach flow (refer to general and contraction scour calculations)
0.33	0.32	L/y	
0.21	0.21	Kd1	for triangular debris
-0.17	-0.17	Kd2	for triangular debris
0.67	0.66	a	pier width (refer to local scoured depth estimation)
0.83	0.82	a*d	equivalent pier width
1.28	**1.26**	**MAX a*d=be***	**equivalent size of the pier that induces about the same scour depth as the actual pier with accumulated debris**

Baseline Scenario
Quarry Road Bridge (Whangatane spillway)

Local scoured depth estimation at pier as in *Bridge Scour* by Melville and Coleman, 2000
Melville Method (1997)

$$d_s = K_{yb} K_I K_d K_s K_\theta K_t$$

Hydraulics after general scour

20yr	100yr			
3.62	3.74	yms=MAX ys (m)	flow depth from flood level to the mean scoured bed level (refer to general and contraction scour calculations)	
0.06	0.06	d50 (m)	sediment size for which 50%of the sediment is finer; Mangakahia clay loam	
0.5	0.5	b (m)	pier width	ASSUMED
8.20	8.20	l (m)	approx pier length	ASSUMED
-1.00	-1.00	Y` (m)	exposed foundation before scour	ASSUMED
0.90	0.90	b* (m)	foundation width	ASSUMED
0.67	0.66	be (m)	equivalent size for non uniform piers; non uniform, downward tapering Case III (refer to Figure 7.12 in *Bridge Scour*)	
1.28	1.26	be* (m)	equivalent pier width calculated to *Bridge Manual*	
0.01	0.01	uc	(from Figure 6.6 in *Bridge Scour*)	
14.51	14.59	Vc/uc	(from Figure 6.6 in *Bridge Scour*)	
0.89	0.92	V (m/s)	mean channel velocity	
0.17	0.17	Vc (m/s)	competent velocity	
5.25	5.36	Velocity parameter	V/Vc<1 for clear-water scour conditions; V/Vc >1 for live-bed scour conditions	

Local scoured depth estimation at piers as in *Bridge Scour (Section 6.3)*

20yr	100yr			
0.35	0.34	be*/yms	<0.7	
3.07	3.01	Kyb	depth-size factor	
1.00	1.00	Kl	flow intensity for live bed conditons	
1.00	1.00	Kd	sediment size factor (assumed)	
0.00	0.00	theta	(approx alignment); approach flow in line with piles	
1.10	1.10	Ks	pier or abutment shape	ASSUMED
1.00	1.00	Ktheta	pier or abutment alignment	
1.00	1.00	Kt	time factor (under live bed conditions)	
3.38	**3.32**	**ds (m)**	**local scoured depth below bed level**	

4.37	**4.39**	**Tds=yds+ds (m)**	**Total scoured depth below bed level**

Baseline Scenario
SH10 Bridge (Whangatane spillway)

The New Zealand Railway Formulation of Holmes (1974) as in *Bridge Scour* by Melville and Coleman, 2000
The method combines general scour and contraction scour

New Zealand Railways (Holmes, 1974)[8]	$K = \sqrt{\frac{W}{4.83Q^{0.5}}} \le 1$ $V_1 = C\left(\frac{Q}{A}\right)\left(\frac{y}{A/W}\right)^{2/3}$ Max. of $y_s = y$ or $y_s = \frac{y_r V_1 K}{\sqrt{(A/W)}}$

Hydraulic Calculations

20yr	100yr		
118.20	119.50	A (m2)	flow area of the unscoured profile
38.20	38.20	m	flood channel width (assumed)
38.21	38.21	m	bridge (main)channel width
3.09	3.13	y (m)	hydraulic depth
4.71	4.74	m	approach flow depth
1.35	1.39	V (m/s)	mean channel velocity
0.01	0.01	uc	(from Figure 6.6 in *Bridge Scour*)
14.12	14.14	Vc/uc	(from Figure 6.6 in *Bridge Scour*)
0.17	0.17	Vc (m/s)	competent velocity
0.50	0.50	m	unscoured flow depth/low flow ASSUMED
0.06	0.06	d50	sediment size for which 50%of the sediment is finer Mangakahia clay loam

Scoured Depth Calculations

20yr	100yr		
159.90	166.50	Q (m3/s)	flow rate
38.21	38.21	W (m)	waterway width allowing for bem flow, taken as W80*1.25. W80 is waterway width, including the main channel which carries 80% of the flow; W80*1.25 - in this case similar to bridge opening width
0.79	0.78	K	factor dependent on waterway width and the Lacey regime width
1.00	1.00	C	coefficient; 1.2 if converging flows are encountered; 1.0 in other cases
1.35	1.39	V1 (m/s)	approach velocity
4.21	4.24	yr (m)	water level rise from low water to flood stage
2.56	2.62	ys (m)	scoured flow depth below flood level
3.09	3.13	MAX ys	maximum scoured flow depth below flood level
0.00	0.00	yds	scoured depth below bed level (= MAX ys-y)
0.00	**0.00**	**yds (m)**	**scoured depth below bed level,incl 20% allowance for no safety factor inclusion in the method**

Baseline Sceanrio
SH10 Bridge (Whangatane spillway)

Equivalent pier width calculation as in NZ Transport Agency *Bridge Manual* 3rd Edition, 2018

$$a_d^* = \frac{K_{d1}(TW)(L/y)^{K_{d2}} + (y - K_{d1}T)a}{y} \quad \text{for} \quad L/y > 1.0$$

$$a_d^* = \frac{K_{d1}(TW) + (y - K_{d1}T)a}{y} \quad \text{for} \quad L/y \leq 1.0$$

Rectangular debris raft

20yr	100yr	(m)	
3.00	3.00	W	width of debris normal to flow (average of the span widths, but not greater than length of largest tree likely to be transported
1.20	1.20	T	thickness of debris normal to flow (up to ~2m not greater than 3)
1.20	1.20	L	length of debris upstream from pier face.L to lie within range 0.4W to 1.3W
3.09	3.13	y	depth of approach flow (refer to general and contraction scour calculations)
0.39	0.38	L/y	
0.79	0.79	Kd1	for rectrangular debris
-0.79	-0.79	Kd2	for rectrangular debris
0.24	0.24	a	pier width (refer to local scoured depth estimation)
1.08	1.07	a*d	equivalent pier width

Triangular debris raft

20yr	100yr	(m)	
3.00	3.00	W	width of debris normal to flow (average of the span widths, but not greater than length of largest tree likely to be transported
1.20	1.20	T	thickness of debris normal to flow (up to ~2m not greater than 3)
1.20	1.20	L	length of debris upstream from pier face.L to lie within range 0.4W to 1.3W
3.09	3.13	y	depth of approach flow (refer to general and contraction scour calculations)
0.39	0.38	L/y	
0.21	0.21	Kd1	for triangular debris
-0.17	-0.17	Kd2	for triangular debris
0.24	0.24	a	pier width (refer to local scoured depth estimation)
0.46	0.46	a*d	equivalent pier width
1.08	**1.07**	**MAX a*d =be***	**equivalent size of the pier that induces about the same scour depth as the actual pier with accumulated debris**

Baseline Scenario
SH10 Bridge (Whangatane spillway)

Local scoured depth estimation at pier as in *Bridge Scour* by Melville and Coleman, 2000
Melville Method (1997)

$$\boxed{d_s = K_{yb}K_I K_d K_s K_\theta K_t}$$

Hydraulics after general scour

20yr	100yr			
3.09	3.13	yms=MAX ys (m)	flow depth from flood level to the mean scoured bed level (refer to general and contraction scour calculations)	
0.06	0.06	d50 (m)	sediment size for which 50%of the sediment is finer; Mangakahia clay loam	
0.30	0.30	b (m)	pier width	SURVEYED
9.30	9.30	l (m)	approx pier length	ASSUMED
1.50	1.50	Y` (m)	exposed foundation before scour	ASSUMED
1.20	1.20	b* (m)	foundation width	SURVEYED
0.24	0.24	be (m)	equivalent size for non uniform piers, slab footing Case II (refer to Figure 7.12 in *Bridge Scour*)	
1.08	1.07	be* (m)	equivalent pier width calculated to *Bridge Manual*	
0.01	0.01	uc	(from Figure 6.6 in *Bridge Scour*)	
14.12	14.14	Vc/uc	(from Figure 6.6 in *Bridge Scour*)	
1.35	1.39	V (m/s)	mean channel velocity	
0.17	0.17	Vc (m/s)	competent velocity	
8.16	8.39	Velocity parameter	V/Vc<1 for clear-water scour conditions; V/Vc >1 for live-bed scour conditions	

Local scour estimation at piers to Section 6.3 in Bridge Scour

20yr	100yr			
0.35	0.34	be*/yms	<0.7	
2.60	2.58	Kyb	depth-size factor	
1.00	1.00	Kl	flow intensity for live bed conditons	
1.00	1.00	Kd	sediment size factor (assumed)	
0.00	0.00	theta	(approx alignment); approach flow in line with piles	
1.10	1.10	Ks	pier or abutment shape	ASSUMED
1.00	1.00	Ktheta	pier or abutment alignment	ASSUMED
1.00	1.00	Kt	time factor (under live bed conditions)	
2.86	**2.84**	**ds (m)**	**local scoured depth below bed level**	

2.86	**2.84**	**Tds=yds+ds (m)**	**Total scoured depth below bed level**

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