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Tauranga Taupo - service level review report

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REPORT

Tonkin+Taylor

Tauranga Taupo

Service Level Review report

Prepared for

Waikato Regional Council

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Exceptional thinking together

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Executive summary

The Waikato Regional Council manages the Tauranga Taupo scheme assets which include approximately 3600 m of stopbanks and spillways constructed in stages between 2003 and 2007.

The scheme provides flood alleviation to the communities of Oruatua and Te Rangiita situated on the true left and right banks and the State Highway assets including the bridge over the river.

The following parameters form the basis of the service level review:

- The 2016 LiDAR and cross section survey
- The 2015 assets crest level survey
- A design 2% Annual Exceedance Probability discharge event
- A design tailwater water level of 357 mRL at the lake
- No bridge blockage
- The 2017 model results

Based on the above parameters, key review findings include the following:

- The predicted water level during the design flood at the Heuheu Parade and Eastern Stopbanks are below original design flood levels (therefore the scheme is providing the intended level of service in these locations).
- The predicted water level is exceeding the Western Stopbank crest level in several locations, the most significant of which is around Chainage 400.
- The ground level at the Maniapoto Spillway is higher than design ground level causing less water to be diverted at the Kiko Spillway than allowed for in the design.
- The water level is below design flood level at the Kiko Spillway causing less water to be diverted through Kiko Spillway and more water being conveyed through the channel downstream.
- The predicted water level is below design flood level at the Quarry Closure Spillway and water is predicted to outflank the Quarry Closure Bank both at its upstream and downstream ends.
- Better survey data is required to gain confidence in the results of this model:
  - In heavily vegetated areas such as the area between the Western Stopbank and the channel and around the upstream and downstream ends of the Quarry Closure Bank.
  - At Maniapoto's Bend to determine the condition of the Bed Control Structure (also known as Grade Control Structure) and erosion protection at the left bank.

1 Introduction

1.1 Purpose

The purpose of this report is to provide a service level review for the Tauranga Taupo Flood Protection Scheme. The scheme provides flood control to the communities and assets situated within the floodplain at Oruatua and Te Rangiita. As the Tauranga Taupo is a dynamic river with a highly mobile sediment load, service level reviews are to be scheduled every 5 years to ensure the scheme is providing the agreed level of protection to the community. This is the first service level review for the scheme.

1.2 Background

The Tauranga Taupo catchment is one of the main catchments contributing to Lake Taupo. The river mouth is located approximately 10.5 km north-east of Turangi. The catchment area is estimated at 230 km^2 draining from the steep terrain of the Kaimanawa Range, mainly through forestry and farmland. The river crosses under the State Highway 1 (SH1) Bridge at Oruatua before entering Lake Taupo at the south-eastern shore.

The steep catchment, geology, and gravel bed contribute to the dynamic nature of the Tauranga Taupo River, where the river can rise rapidly to high flows under intense rainfall. Historically, flood events have resulted in flooding and overland flows with associated issues including the diversion of the river through the Te Rangiita Quarry in the 2001 flood event as well as damage to farmland with erosion and deposition of sediment and debris in many other events.

The Tauranga Taupo scheme assets include approximately 3600 m of stopbanks and spillways constructed in 2006 and 2007. The scheme provides flood alleviation to the communities of Oruatua and Te Rangiita situated on the true left and right banks and the State Highway assets including the bridge over the river.

1.3 Catchment description

The Tauranga Taupo River drains from the Kaimanawa Range and discharges to the south-eastern shore of Lake Taupo with a catchment area estimated at 230 km^2 (Figure 1-1). The catchment is the second largest of the eastern Lake Taupo catchments after the Tongariro River.

The majority of the catchment is in steep mountainous terrain within the Kaimanawa Forest Park. Elevations in the catchment fall over 1200 m from 1570 m at the highest elevation to 357 m at Lake Taupo. The river is approximately 45 km long from the headwaters down to Lake Taupo. The ground surface layers of the catchment include volcanic ash, pumice and porous gravel and sand. The nature of these soils makes the catchment prone to erosion and slips.

Vegetation in the upper catchment is predominantly native cover (67.5%), with production forestry dominant in the lower catchment (30.3%), and the remainder in pasture, scrub, quarry, urban and roading. The Te Rangiita Quarry fulfils a valuable role in the management of peak flows in the Tauranga Taupo River by providing a limited storage area for flows over the Quarry Closure Bank Spillway.

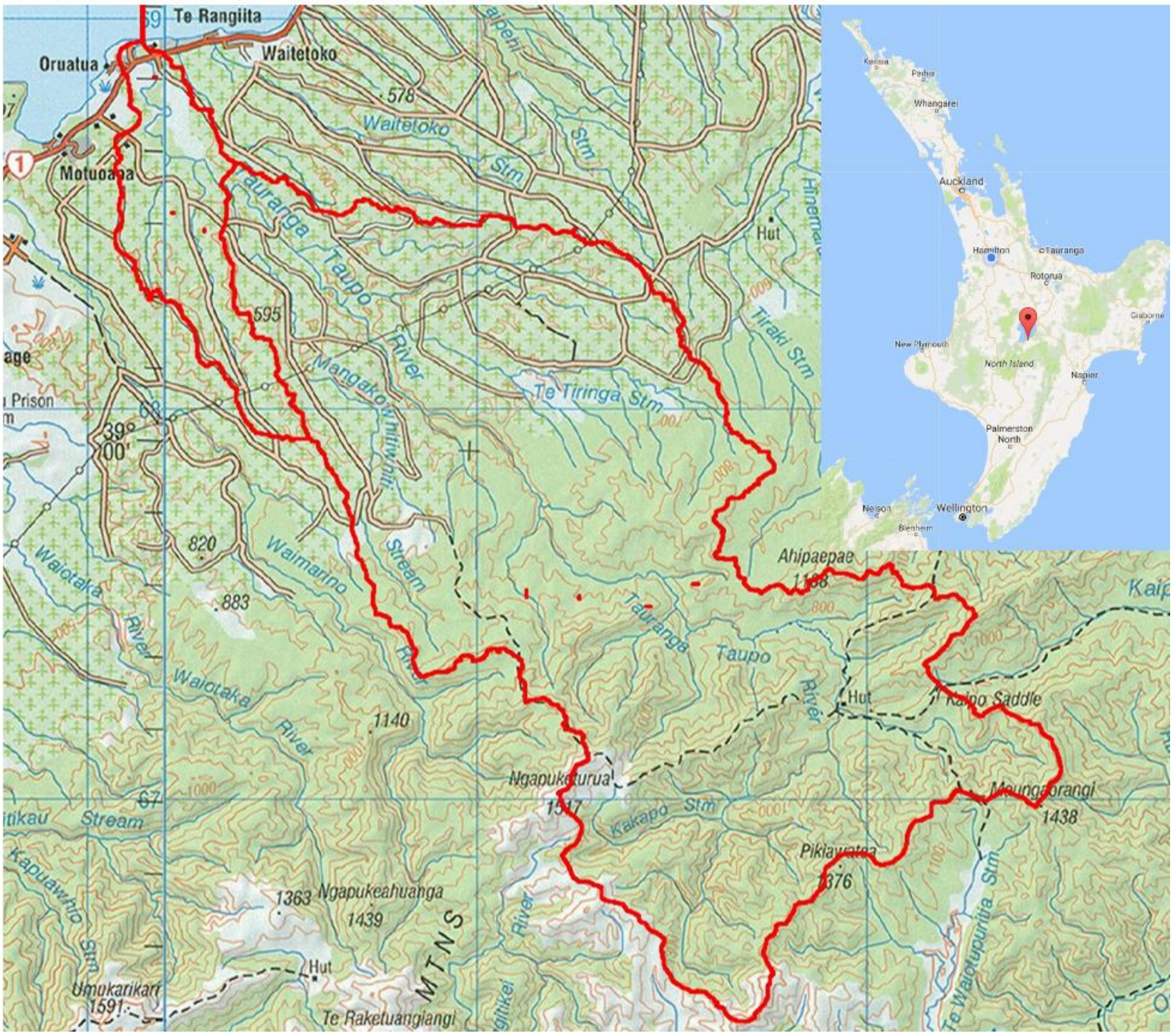

Figure 1-1: Tauranga Taupo location plan and catchment area

1.4 Flood protection scheme and existing design

The Tauranga Taupo Flood Control Scheme construction was completed in 2007. The scheme assets are described in the subsection below and As Built design drawings can be found in Appendix G.

1.4.1 Scheme assets

The scheme assets shown in Figure 1-2 include approximately 3600 m of stopbanks and spillways, providing flood alleviation to the communities of Oruatua (left bank) and Te Rangiita (right bank) as well as the State Highway assets e.g. the road south of the SH1 Bridge.

1.4.2 Design hydrology

The hydrological analysis undertaken in 2002 for design purposes was based on the Te Kono gauge (refer to Figure 2-1 for gauge location) with records starting in 1976. The analysis is summarised in Table 1.1 below.

Subsequent analysis has been undertaken (e.g. Opus 2010), however, as far as the writer is aware the design hydrology for the scheme to date has not been updated prior to this review.

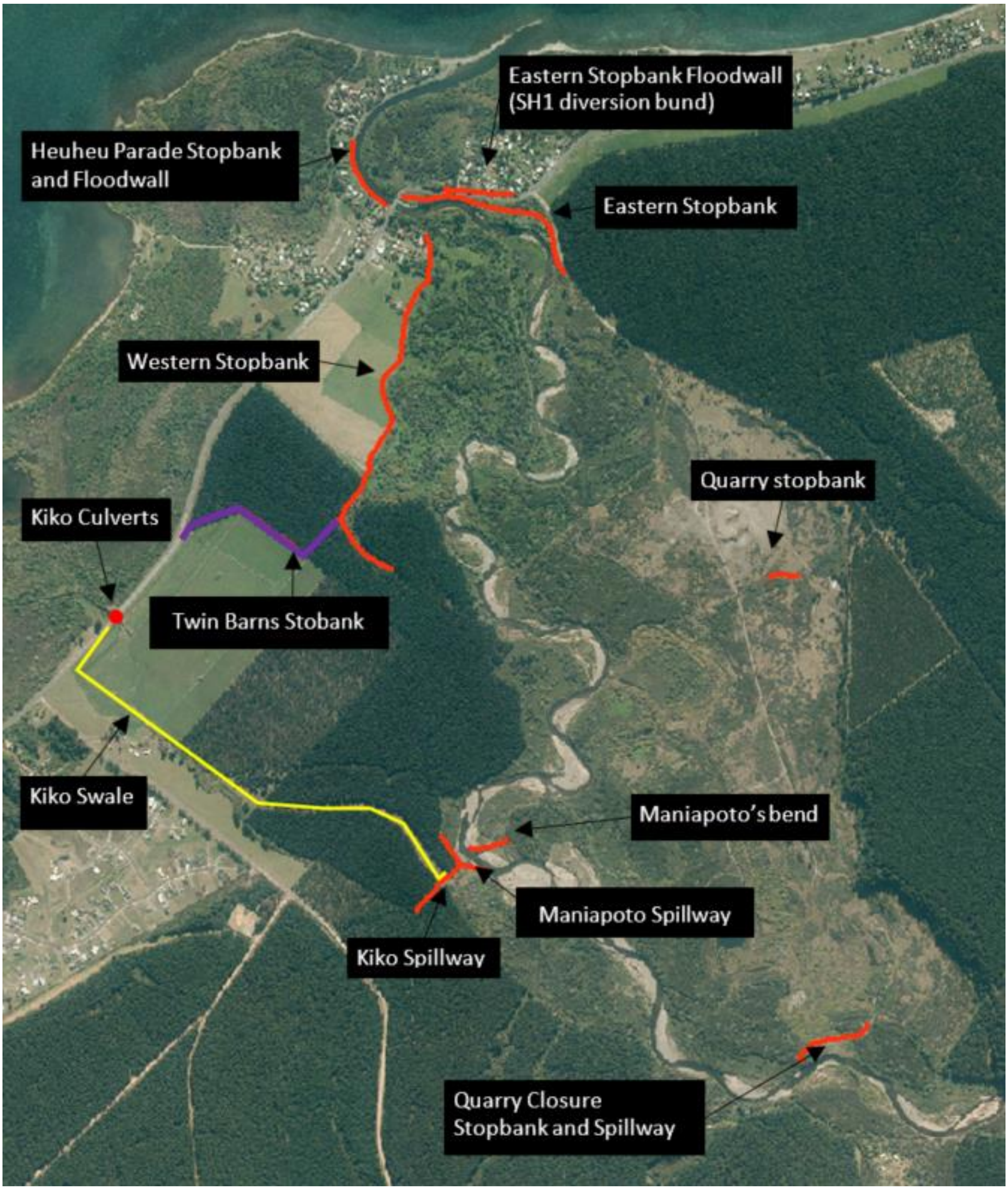

Figure 1-2: Tauranga Taupo Scheme Assets location

Table 1.1: Tauranga-Taupo River return period discharge flood estimates

Annual exceedance probability (AEP %)	Estimated peak flow at Te Kono - m^3/s	Upper bound bstimate (1 standard deviation) - m^3/s	Lower bound estimate (1 standard deviation) - m^3/s
Mean annual flood (MAF)	150	161	140
20	197	212	182
10	235	255	214
5	271	297	245
2.9 (35 year ARI)	299	330	269
2	318	351	284
1	353	392	313
0.5	388	433	342

1.4.3 Design level of service

The scheme has been designed to two design standards, these are:

- The 2% AEP flood standard in terms of protection to the urban areas, and
- The 5% AEP flood standard for the State Highway and those rural areas not in the quarry or vegetated section of the floodplain.

In general, the 400 mm freeboard provided on stopbanks protecting urban areas is expected to contain around the 1% AEP flood. Rural areas, other than the regularly flooded river reserves, should begin to experience overland flows with floods in excess of the 5% AEP approximate flood i.e. the quarry area and Kiko plantation swale area. The following table sets out the design standards of the different elements of the scheme:

Table 1.2: Tauranga Taupo Scheme asset design standard

Scheme Eelement	**Design standard/ flows (% AEP)**	**Overtopping flows in 2% AEP (50-year) flood**	**Design (mRL) – from design drawings**
Right Bank			
Quarry Closure Bank	2% (318 m^3/s)	No Overtopping	368.7
Quarry Closure Bank Spillway	<10% (235 m^3/s)	1.5 m^3/s to 30 m^3/s	367.7
Quarry attenuation	2%	30 m^3/s	
Eastern Stopbank	2% (209 m^3/s)	No overtopping	359.5 - 360.4
Eastern Stopbank Spillway	>2% (209 m^3/s)	No overtopping	359.1 (DS Secondary Spillway) 359.15 – 359.4 (US Primary Spillway)
SH1 Diversion Bund	Between 2% and 0.5%		359.32
Left Bank			
Maniapoto's Bend	Ensure stability and ensure no river diversion into Kiko Swale		
Maniapoto's Bend Bed Control and Rediversion Bank	Ensure design cross section is stable and flows distributed between main channel and Kiko Spillway		
Kiko Spillway	< Annual (120 m^3/s)	23 m^3/s to 82 m^3/s	363.1 – 363.26
Kiko Spillway to Twin Barn	5% (175 m^3/s)		361.0 -361.1
Western Stopbank	2%	No overtopping	360.2 – 361.1
Heuheu Parade Stopbank and Floodwall	2%	No overtopping	358.90 – 359.41 floodwall and 358.90 – 358.6 for stopbank

The scheme elements, design standard/flows and the overtopping flows in 2% AEP (50-year) flood were taken from the Tauranga Taupo Catchment Management Plan while the design levels were taken from the design drawings.

Consideration has also been given to managing the effects of floods in excess of the design standards. Some damage is to be expected to non-critical components, but key elements, such as the Quarry Closure Bank and Kiko and Maniapoto Bend Spillways and the urban stopbanks, are designed not to fail catastrophically in extreme flood conditions.

2 2017 Assessment of Hydrology and hydraulics

2.1 Previous service level reviews and outcome

Since completion in 2008 there has been no previous service level review undertaken on the Tauranga Taupo Flood Control Scheme. These are now scheduled to be undertaken at 5 year intervals.

2.2 Hydrology

2.2.1 Gauges

The Tauranga Taupo catchment area is approximately 230 km². A water level gauge at Te Kono in the lower reaches allows for the estimation of flows, and with a catchment of 197 km² covers 90% of the total catchment area (Figure 2-1). The remaining ungauged tributary inflows are in the lower catchment, are relatively small, and will have a faster time of concentration than the remainder of the catchment. The Te Kono gauge provides 42 years of continuous flow data from February 1976, providing a suitable record for flood frequency estimation.

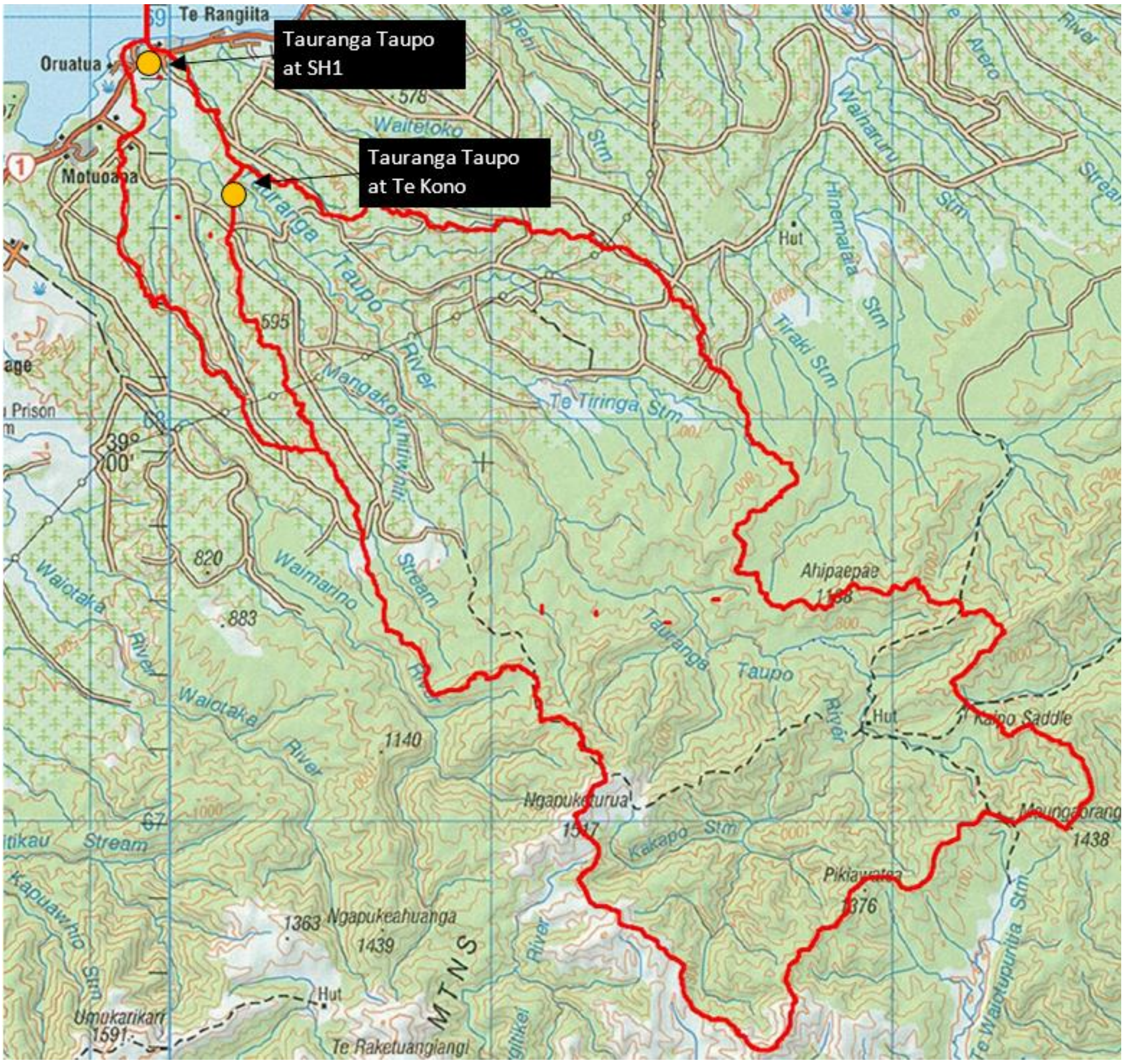

Figure 2-1: Tauranga Taupo Gauges

Hydrological analysis was undertaken as part of this review in order to estimate the following two main parameters and compare to those assumed during design of the scheme:

- Peak flows and hydrographs for events up to the 2% AEP, and
- Peak flows and hydrographs for the 1% AEP and 1% AEP with climate change events.

The findings are presented in the memo in Appendix A and summarised in the section below.

2.2.2 Peak flows and hydrographs for events up to the 2% AEP

For this analysis, the record at Te Kono was used. The record shows typical annual flow variation, characterised by long periods of low flow interspersed with short duration high magnitude events. Five larger events occurred in the short period between 1998 and 2004, with a more recent event in 2015. Flood frequency analysis was carried out and the revised peak discharges for design events are presented in Table 2.1 below.

Table 2.1: Summary of flood frequency analysis

Annual Exceedance Probability (AEP, %)	Discharge (m^3/s)
50	140
20	195
10	231
5	264
2	305
1	335

To estimate the design hydrographs, the observed hydrographs were normalised, by dividing flows by the hydrograph peak and then averaging the normalised hydrographs. Following further inspection (Figure 2-2), it is apparent that the 1990 hydrograph is not a typical single event hydrograph and could be excluded from calculation of the design hydrograph. Normalised hydrographs for the six events (analysis taken by Waikato Regional Council (WRC)) as well as the Tonkin & Taylor Ltd (T+T) average (which excludes the 1990 event) are shown in Figure 2-2.

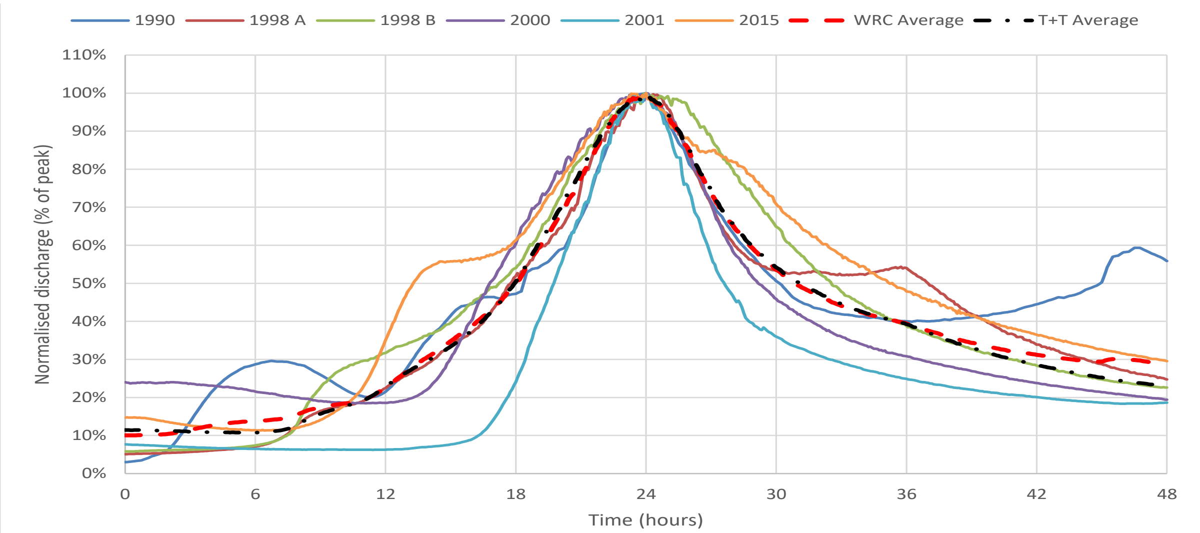

Figure 2-2: Tauranga Taupo River at Te Kono: Normalised hydrographs

The design hydrograph used for hydraulic modelling was based on the five events (i.e. excluding the 1990 event).

2.2.3 Peak flows and hydrographs for the 1% AEP and 1% AEP with climate change

Storm rainfall was estimated based on the HIRDS V3 database for present day and projected climate change to 2090 (2.1°C increase in temperature) for a location near the centroid of the catchment.

Design hydrographs for a 1% AEP design storm and 1% AEP with climate change were generated using a HEC-HMS model. Initial model parameters were estimated based on land use, soil permeability, catchment slope and longest water course. These parameters were adjusted to improve the comparison between the simulated hydrograph and the design hydrograph generated as the average of five of the largest historic hydrographs (1990 was excluded) and scaled to the 1% AEP peak.

Table 2.2: Peak flows for 1% AEP and 1%+ Climate Change

Design event (AEP)	Peak Flow (m^3/s)
1%	335
1% + climate change	429

The calibrated model was used to generate the 1% AEP hydrographs for the full catchment for present and projected 2090 storm rainfall. These hydrographs are shown in Figure 2-3.

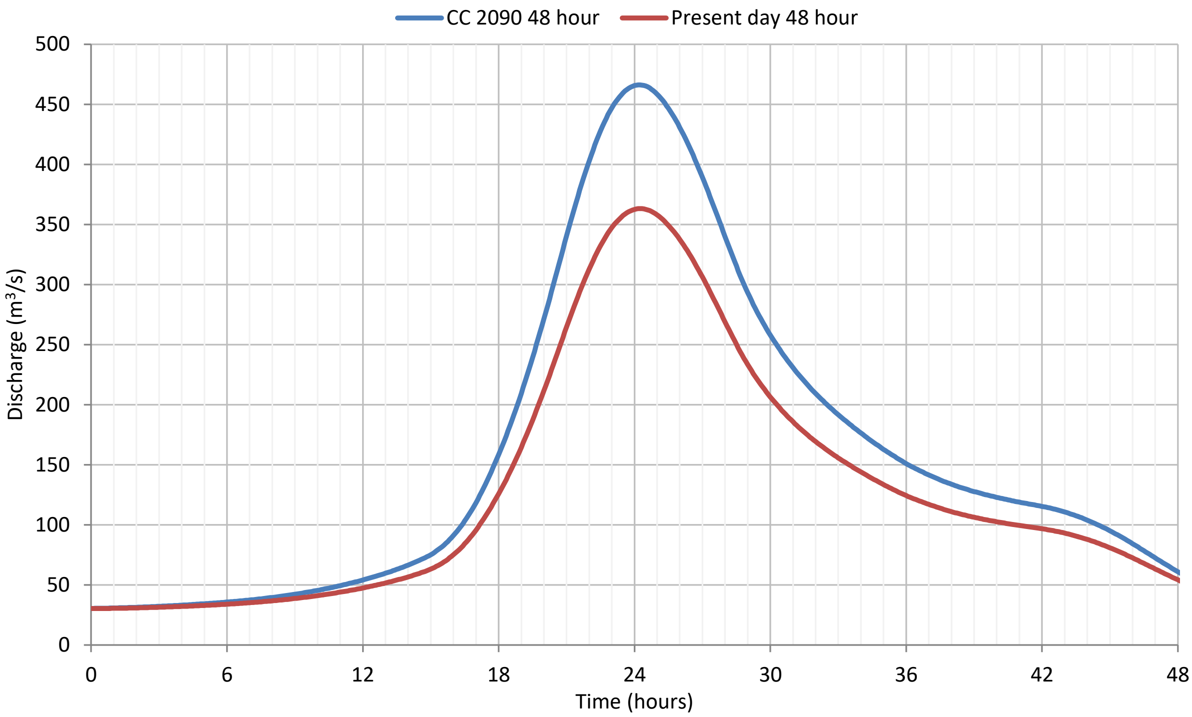

Figure 2-3: Tauranga Taupo Catchment: 48 hour design hydrographs (peak for entire catchment)

2.2.4 Comparison of the design and 2017 analysis

A comparison of the peak flows from the original design and most recent analysis is presented in Table 2.3 below.

Table 2.3: Flood frequency analysis table (2017 vs existing design)

	Design			2017 review
Percent Annual Exceedance Probability (AEP, %)	**Estimated Peak Flow (m^3/s)**	**Upper Bound Estimate (1 standard deviation) (m^3/s)**	**Lower Bound Estimate (1 standard deviation) (m^3/s)**	**Estimated discharge at Te Kono (m^3/s)**
(Mean Annual Flood)	150	161	140	140
20	197	212	182	195
10	235	255	214	231
5	271	297	245	264
2.9 (in 35 year)	299	330	269	
2	318	351	284	305
1	353	392	313	335
0.5	388	433	342	
1 + climate change				429

When comparing the original design and 2017 hydrology, it is apparent that the revised hydrological analysis yields similar results to the original and certainly remains to be within one standard deviation of the design analysis (despite adding 13 years to the record). The revised 2017 peak flows and hydrographs were used for the hydraulic modelling undertaken as a part of this service level review.

2.2.5 Recent floods

The Tauranga Taupo River Catchment Management Plan states the following with regards to the recent flood history of the river:

"Major flood events in the Tauranga Taupo River catchment in 1958 and 1964 prompted investigations as to the provision of flood protection for the communities of Oruatua and Te Rangiita. Several options and schemes were determined by the then Waikato Valley Authority (WVA) in 1966 and 1975 but were not adopted due to funding constraints upon the WVA and the Taupo County Council.

A proposed scheme was adopted 1981 and partially implemented. Works included the construction of stopbanks in the vicinity of Kiko farm, formalisation of the natural overflow at Maniapoto's Bend (Kiko Overflow) to allow peak flows to bypass the settlements of Te Rangiita and Oruatua and the construction of the Kiko Culverts under State Highway 1.

Floods in July 1998, October 2000, December 2001, May 2003 and February 2004 have drawn attention to the risks to both rural and urban land from flooding. The December 2001 flood broke through into the excavated quarry area and formed a new course for the river. This breakout effectively de-watered a substantial length of the natural course of the river, changed the balance of flooding and flood risk and substantially affected the trout fishery. In December 2003 the river was put back into its original course with the construction of a closure embankment at the breakout point into the quarry. Significant works were undertaken to improve the hydraulic efficiency of the natural spill area at Maniapoto's Bend, a grassed spillway channel was formed and the capacity of culverts under SH1 were increased.

The February 2004 flood resulted in some changes in the channel and activation of a number of erosion sites. There were no major changes in channel alignment during the flood event, however it seems that a significant volume of sediment and debris was mobilised from within the river channel."

The April 2015 flood is the most recent flood which came close to the SH1 bridge soffit, and a flood mark survey was carried out after the flood on most of the scheme assets to record peak water levels. This event was used as a calibration event when the 2017 modelling was undertaken to ensure the model was fairly representing the behaviour of the river during high rainfall runoff events.

2.3 Hydraulic modelling

2.3.1 Model type

The coupled 1D/2D hydraulic model uses MIKE 11 and MIKE 21 software by DHI. The model was mainly developed by WRC, and reviewed and refined by T+T.

2.3.2 Survey data

The latest full cross-section survey of the lower Tauranga Taupo River and LiDAR survey were undertaken in 2016. The cross-sections and LiDAR surveys were used in the model to represent both the terrain/ground levels, and bed and banks.

The MIKE11 model was developed using the cross-sections described above. A comparison of both surveys showed a good correlation except where significant water depth lies in the channel. Here the LiDAR data has subsequently been used to extend some cross-sections in the MIKE11 model as necessary (by extracting cross sections from LiDAR and using the cross section survey data to estimate the channel thalweg in LiDAR extracted cross sections).

The MIKE21 model uses a 5 m topographic grid derived from the LIDAR data. The stopbanks were surveyed in 2015 and were "burned into the LiDAR" to represent the stopbank levels.

2.3.3 Datum

The model vertical datum is Moturiki Datum, and horizontal New Zealand Transverse Mercator (NZTM) Datum.

2.3.4 Model assumptions

It is important that the limitations of this hydraulic model and its results are understood when using this information. These limitations are outlined as follows:

- T+T and WRC, while providing the information in good faith, accepts no responsibility for any loss, damage, injury, or loss in value of any person, property, service or otherwise resulting from river flood hazards or knowledge of river flood hazards in the Waikato Region.
- This hydraulic model is based on the existing (as per the 2015 and 2016 surveys) condition of the Tauranga Taupo River catchment. Any significant change to this condition will affect the river and catchment characteristics that affect the Te Rangiita and Oruatua community. For example, land use changes, deforestation, the intensification of development, riverbed changes during a flood event. Where significant changes do occur, this hydraulic model should be reviewed.
- This hydraulic model is based on the channel/floodplain geometry and stopbanks as surveyed in 2015 and 2016. It is acknowledged that some changes may have subsequently occurred since this time. The surveys also exclude existing and future obstructions such as fences, trees,

and buildings. These obstructions may cause localised changes to the flood extent, depth and/or speed.

- Due to the nature of the floodplain (heavily vegetated) particularly within the quarry and near the Western Stopbank, it should be expected that some parts of the terrain are interpolated with high uncertainty. This was confirmed on site in two particular areas, being the right bank of the Quarry Closure Bank and the area adjacent to the Western Stopbank.
- This hydraulic model is only relevant to flood events in the Tauranga Taupo River. Other sources of flooding (e.g. localised ponding, stormwater infrastructure, flooding caused by high lake levels and other smaller waterways) are not included in this assessment.
- This hydraulic model is based on clear and free flowing water. It does not explicitly include the movement of gravel, sediment and debris. It also does not include instances where large debris blockages and flows occur.
- The hydraulic model results are based on the hydrological assessment in Section 2.2 above. Any changes to the hydrology (peak discharge, volume of water discharged or duration) is likely to result in different water levels in the vicinity of the Flood Control Scheme assets.
- The 1D model (representing the channel) assumes a straight channel and therefore velocities within the model are not represented at bends in the 1D channel. In addition, the velocity is not transferred to the 2D model through the lateral links used to transfer water from the 1D to the 2D model and vice versa.
- T+T received the model from WRC to review and has made simple amendments to the model (roughness value, couple links flooding and drying parameters). The model structure (i.e. LiDAR, area blocked out within the LiDAR, areas where the channel is identified in the 1D model) were not modified from the model received from WRC.

In addition to the above and as stated in the 2002 AEE:

"*It is worth nothing that there is a degree of uncertainty in the actual water levels during any particular flood, likely in the order of one to three hundred millimetres in a major flood, depending on the actual hydraulic conditions of the flood, including debris, load and any bed aggradation or scour. Variations from the predicted water level upstream of the highway bridge could be substantially greater, particularly if a debris raft built up upstream of the bridge piers to the extent that the flood level exceeds the bridge soffit level. Unsteady surging flow could occur under the bridge under these circumstances, accompanied by substantial fluctuations in the flood level just upstream of the bridge. These effects translate into a degree of uncertainty in the overflow at Te Rangiita because of the amount of spill flow affected by the flood level at the bridge.*

- *Accept and allow for the fact that there is an inherent degree of uncertainty in the interpretation and hydraulic modelling of the floodplain behaviour.*
- *Accept also the fact that the river is very dynamic in morphological terms, and actual hydraulic conditions during floods could be significantly different to the modelled conditions."*

2.3.5 Boundary conditions

The boundary conditions used in the model consisted of the upper boundary (discharge hydrographs based on the hydrographs described in Section 2.2.2 above), and lower boundary (constant lake level based on data supplied by WRC. Table 2.4 below shows the boundary conditions used for each of the simulated events.

Table 2.4: Model boundary conditions

Design event (percent AEP)	Upstream Boundary (m^3/s)	Downstream Boundary - lake level (mRL)
50	140.0	357.0
10	231.0	357.0
5	264.0	357.0
2	305.0	357.0
1	334.5	357.0
1 + Climate change	428.7	357.79[1]

2.3.6 Roughness

The roughness was represented by a roughness map for the Mike21 (2D model) mainly based on Van Te Chow Manning's roughness numbers, some values were slightly modified following the calibration event to ensure the model produces reasonable outputs.

The channel roughness in the Mike11 (1D) model as well as the 2D model has been set to a constant n= 0.025. This is at the lower bound of the recommended theoretical value of 0.025 – 0.06 for gravel bed rivers (particularly in 2D modelling). However, upon literature review (presented in Table 2.5) the values presented do align with the values presented in the Conveyance Estimate System (CES)[2].

Table 2.5: Literature Review for channel roughness values

Literature	Channel	Lower	Normal	Upper
		Mannings n		
CES	Channel: Gravel bed rivers (Fine gravel 2-7 mm)	0.017	**0.022**	0.025
CES	Channel: Gravel bed rivers (gravel 7-20 mm)	0.02	**0.025**	0.028
Chow 1959	Channel: bottom: gravels, cobbles, and few boulders	0.030	**0.040**	0.050
Chow 1980	Channel: Major Stream – regular section with no boulders or brush	0.025	-	0.06

The values used in the model are presented in Figure 2-4 and Table 2.6 below.

[1] Based on the Opus 2010 report and accepted by WRC

[2] Conveyance Estimation System roughness values, plus lower and upper values, are assigned via a set of databases which contain roughness data from various sources of literature compiled by the Scottish Government Environment Agency.

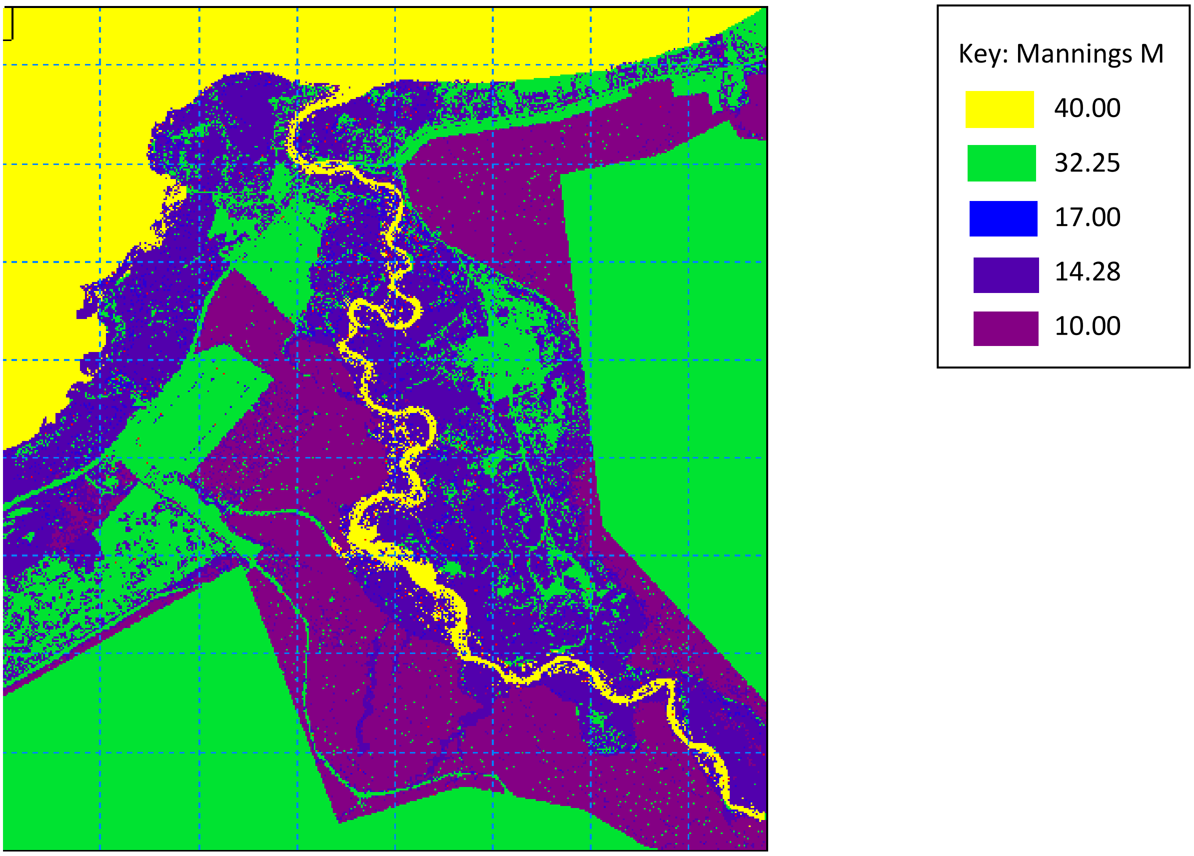

Figure 2-4: Model roughness map

Table 2.6: Model roughness table

Description (Chow 1980)	Mannings M	Mannings n
Channel (1D and 2D)	40	0.025
Grass (floodplain some quarry areas mainly in clear areas)	32.25	0.03
Light brush and trees (mainly within the quarry left bank near the Western Stopbank)	17	0.06
Medium to dense brush (as above)	14.28	0.07
Bush (mainly outside of the floodplain)	10	0.10

2.3.7 Structures

2.3.7.1 Bridges

The MIKE11 model includes the SH1 Bridge. Surveyed cross-sections at the bridge have been used to represent the channel immediately upstream and downstream of the structure. The bridge openings were based on survey, best estimates and include soffit and piers represented using the bridge function. Overflows are included at bridge deck level using the weir function. Debris blockage of the waterway or the effect of debris upstream of the bridge has not been included in the assessment but as a part of a sensitivity analyses. Bridge piers were represented by a blockage within the channel equating to approximately 5% of the waterway area.

2.3.7.2 Culverts

In addition to the bridge, the four Kiko Culverts were also included. These are twin 1200 mm diameter culverts and twin 1600 mm diameter culverts under SH1 conveying the flow from the Kiko Swale to the lake.

2.4 Calibration event

The June 2015 event was used as the calibration event for the model. Data from Te Kono gauge and the lake level were used as the upstream and downstream boundary conditions while the results were compared with the water level survey undertaken in the vicinity of all assets after the event. Table 2.7 below shows the comparison of the modelling results with the surveyed water levels.

Table 2.7: Comparison of surveyed and modelled water levels

Asset	Surveyed Water Level (mRL)	Modelled Water Level (mRL)	Difference (Surveyed – Modelled, m)
Kiko Spillway	363.52 – 363.64	363.51 - 363.65	0.01 (-0.01)
Western Stopbank (upstream section)	360.53 – 359.2	360.9 - 358.82	(-0.37) – 0.38
Eastern Stopbank	358.97 – 358.44	358.91 – 358.66	0.06 (-0.22)
Heuheu Parade	358.52 – 357.96	358.32 – 358.03	0.2 (-0.07)
SH1 Bridge	Observed – 358.5	358.56	(-0.06)

The model shows reasonable correlation with the surveyed water levels in most areas. The largest difference in the recorded and modelled water levels are at the Western Stopbank. Inspection of the area during the site visit showed that the area is heavily vegetated and it is highly likely that the ground levels are not represented well in this area of the model based on LiDAR data. To gain more reliable results in these areas a topographical survey could be conducted, however this would require vegetation clearance in order for these areas to be accessible. This would also be true in other heavily vegetated parts of the floodplain including the downstream part of the Quarry Closure Bank and areas within the quarry.

The Tauranga Taupo River system is sensitive to both the peak flow and volume as it relies on some parts of the system to store the flood flows prior to discharging through the main channel or Kiko Spillway. It is noted that although the peak discharge for this event is comparable to the 5% AEP, the volume is more comparable to the 2% AEP, therefore the higher river levels observed are to be expected.

2.5 Design events

The following design events were simulated using the 1D/2D coupled models:

- 50% AEP storm
- 10% AEP storm
- 5% AEP storm
- 2% AEP storm
- 1%AEP storm
- 1% AEP storm plus allowance for climate change

Flood extents from these events are attached in Appendix D and results from those events are presented in Section 3 below.

3 Assessment of level of service

3.1 Performance grades

To enable measurement of stopbank performance against targets, stopbanks (and spillways) are graded on a 1 to 5 scale. The performance grade is based on the available freeboard relative to the design freeboard (where the available freeboard is the difference between the surveyed actual stopbank crest level and the design flood level). The five performance grades for both stopbanks and spillways are set out in Table 3.1 and Figure 3-1. The Zone Management Plan states that service levels will be achieved by maintaining stopbanks to achieve performance grade 4 or better.

Current performance has been assessed against the design performance standard based on most recent stopbank crest level survey data.

Table 3.1: WRC Stopbank and spillway performance grades

Performance Grade	Stopbanks	Spillways
	P= (Actual Freeboard/Design Freeboard) x 100%	**A*[1] = Actual Freeboard**
1	P > = 100%	|A|<= 0.025 m
2	100% > P >= 50%	0.025<|A| <= 0.05
3	50% > P >= 25%	0.050<|A| <= 0.075
4	25% > P >= 0%	0.075 <|A| <= 0.1
5	P < 0%	0.1<|A|

*[1] Actual freeboard is the difference between the stopbank or spillway level and the design event level. For stopbanks the freeboard should be 0.4 m and for spillways the freeboard should be 0.

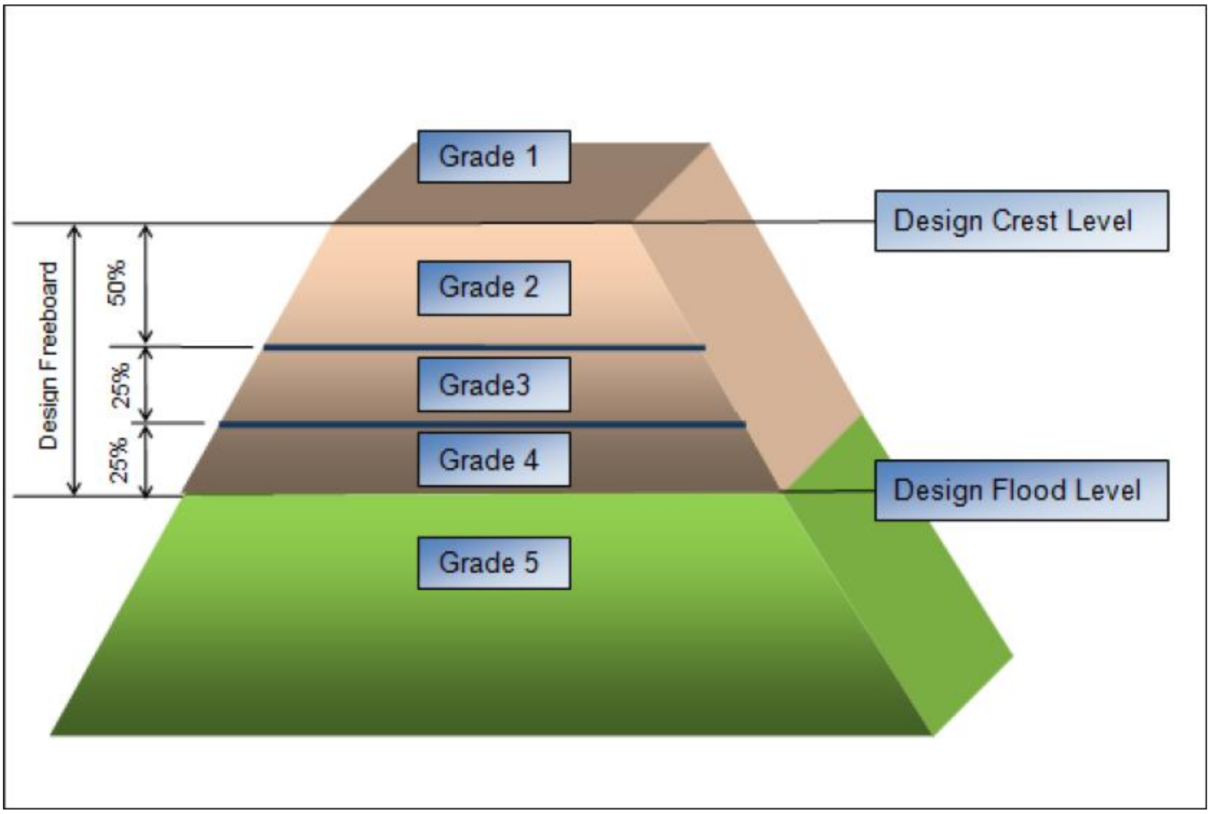

Figure 3-1: WRC Diagrammatic representation of stopbank performance grades

3.2 Stopbank performance based on water level

The results of the model are discussed in the following subsections for each asset and is then followed by an overall scheme performance discussion.

3.2.1 Heuheu Parade Stopbank

Heuheu Parade Stopbank is a timber floodwall, mainly providing freeboard to protect dwellings to the left side of the river on Heuheu Parade. Figure 3-2 shows the location of the stopbank while Figure 3-3 shows a long section along the stopbank with the different AEP flood events.

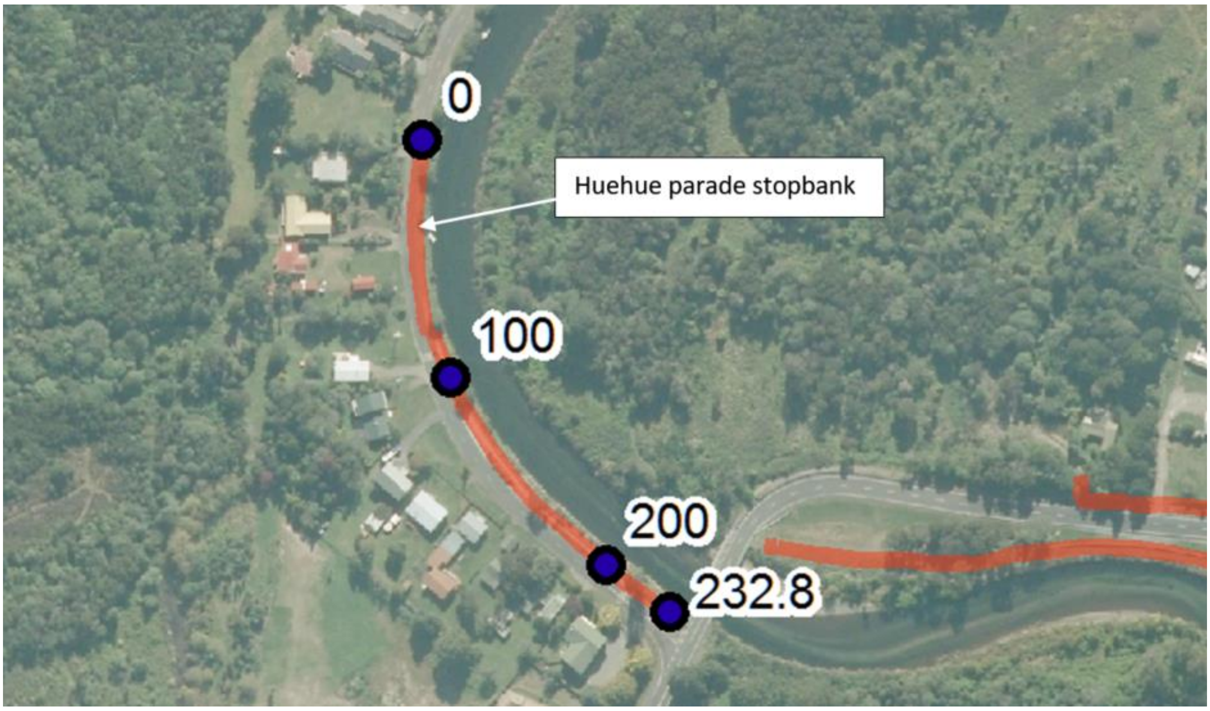

Figure 3-2: Heuheu Parade Stopbank - location map

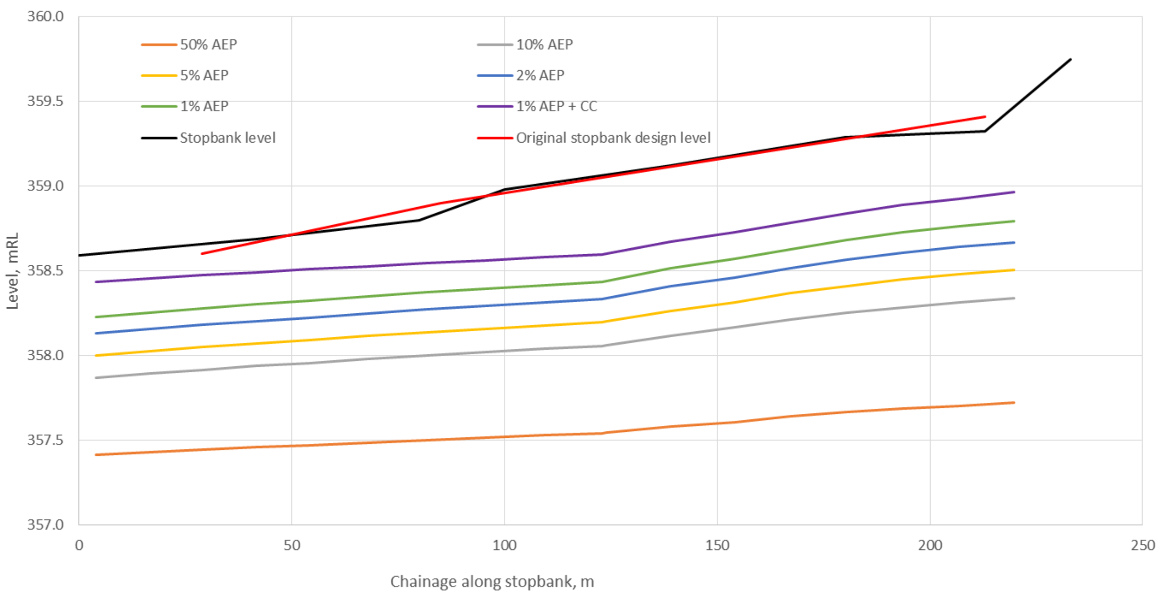

Figure 3-3: Heuheu Parade Stopbank system performance based on a range of events

The stopbank is designed for the 2% AEP event with 400 mm freeboard. The table below shows that the freeboard is maintained throughout the stopbank and the performance grade for the entire stopbank is 1.

Table 3.2: Heuheu Parade Stopbank performance grade

Chainage (SB only, m)	2% AEP flood level (mRL)	SB height (mRL)	Actual Freeboard (m)	P (Actual Freeboard/Design freeboard)	Performance Grade
0	358.12	358.59	0.47	117%	1
41.5	358.20	358.69	0.49	122%	1
80	358.27	358.80	0.53	133%	1
100	358.30	358.98	0.68	170%	1
139	358.41	359.12	0.71	178%	1
180	358.56	359.29	0.72	181%	1
213	358.65	359.32	0.67	168%	1
233	358.70	359.75	1.05	262%	1

It should be noted that although water level is not expected to overtop the stopbank at this location, in events larger than the 1% AEP event, the model shows water levels exceeding the left side of the bridge (refer to Figure 3-4 below), overtopping the road (SH1) and causing flooding to the houses protected by this section of floodwall.

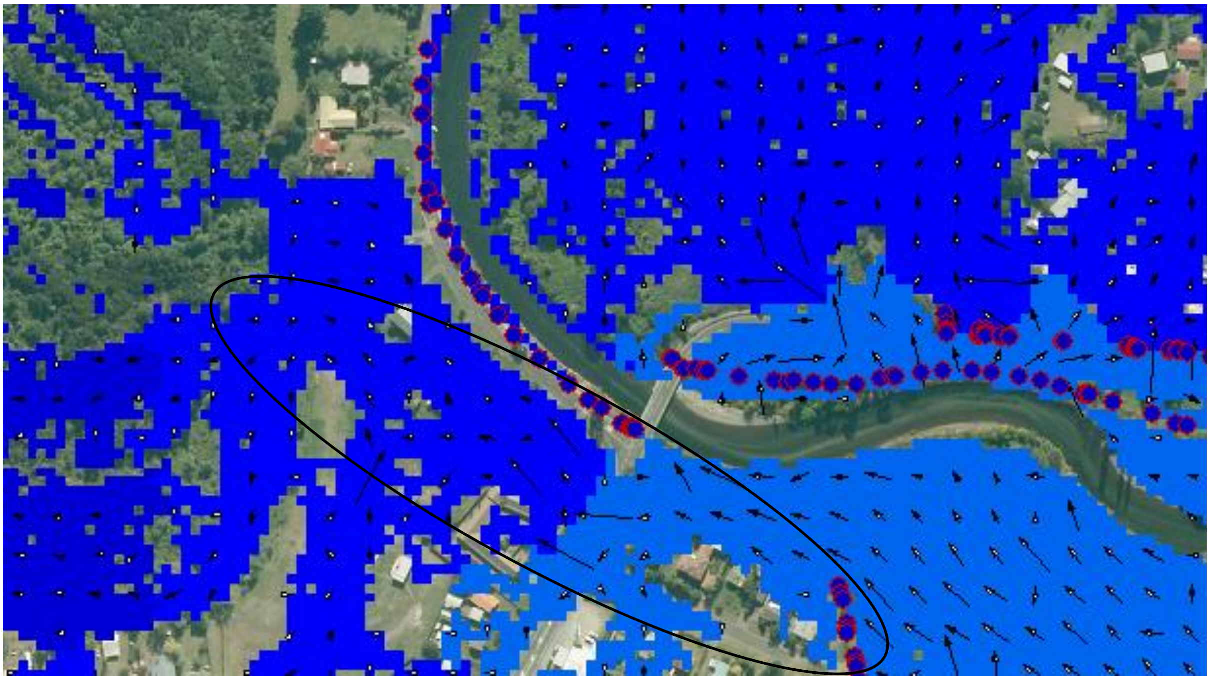

Figure 3-4: Heuheu Parade flooding in 1%AEPplus climate change event

3.2.2 Eastern Stopbank

The Eastern Stopbank is a combination of earth embankment and timber floodwall. It consists of sections separated by two spillways. The stopbank provides flood control to the properties to the eastern side of the river on SH1 as well as to Te Rangiita. Figure 3-5 shows the location of the stopbank while Figure 3-6 shows a long section along the stopbank with the different AEP flood events.

The Eastern Stopbank Floodwall (SH1 Diversion Bund) is also shown in Figure 3-5 this floodwall was designed to provide freeboard for the properties along SH1 in the 2% AEP event, and to provide an overtopping mechanism should a significant degree of blockage occur at the bridge.

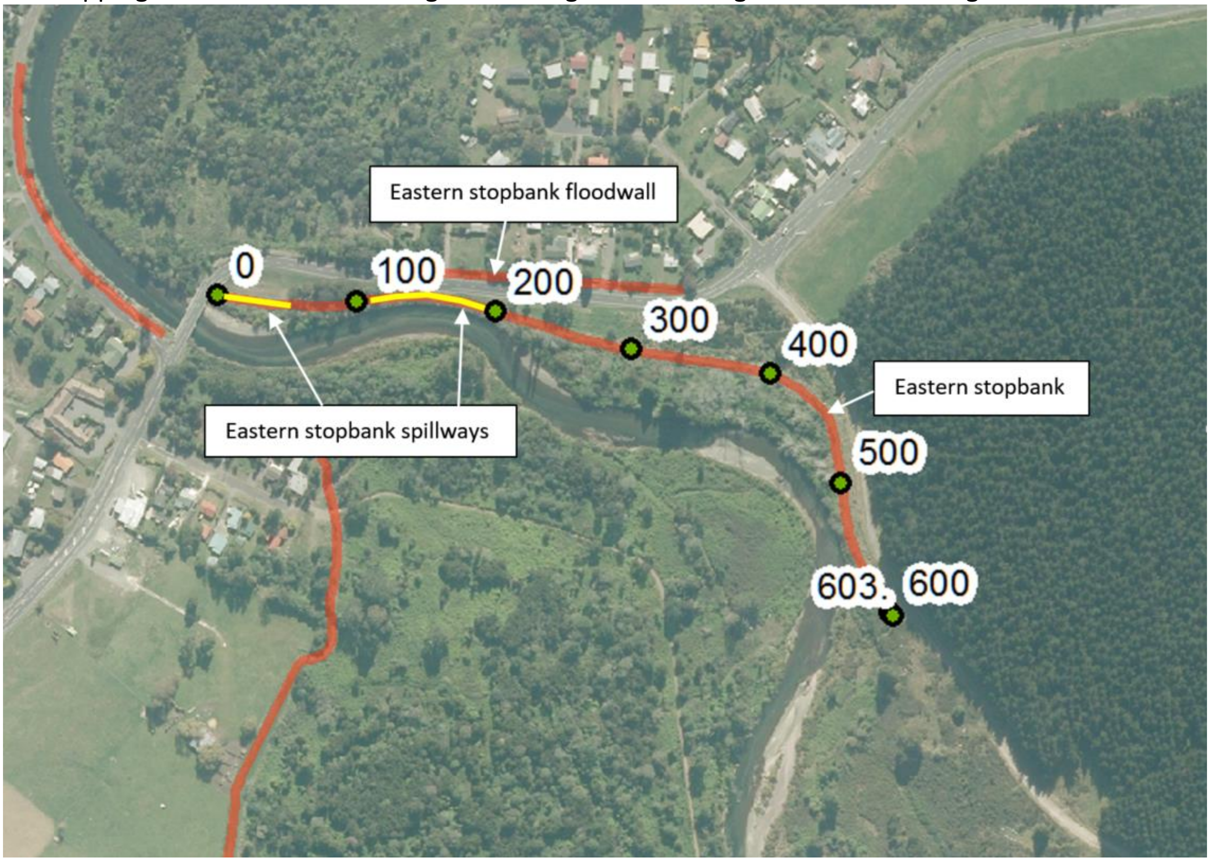

Figure 3-5: Eastern Stopbank location map

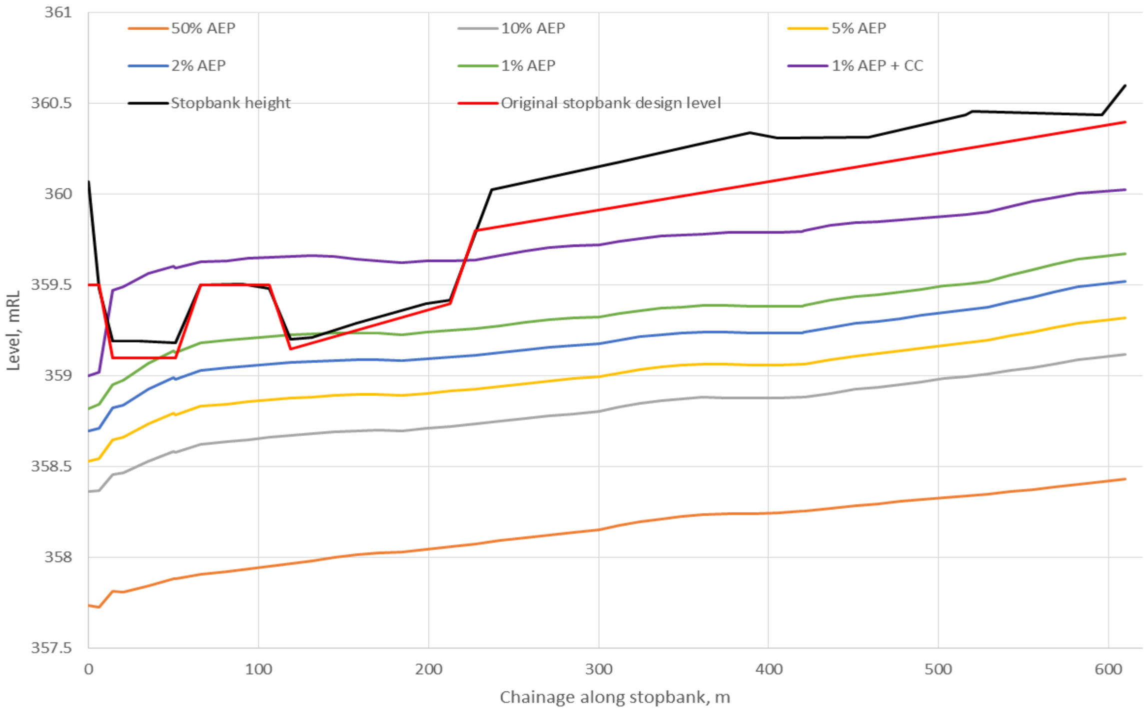

Figure 3-6: Eastern Stopbank system performance based on a range of events

The Eastern Stopbank spillway is set at the bridge soffit level of 359.1 mRL and is designed to provide safe conveyance for the over design event, as well as when there is significant blockage at the bridge. The stopbank is designed for the 2% AEP event with 400 mm freeboard. Table 3.3 below shows performance grade for the stopbank. The majority of the stopbank is performance grade 1, however as the water level in the design event is less than 100 mm below the spillway crest level, the spillway grade is 5 (it is not operating as designed in the design event).

The Eastern Stopbank floodwall is overtopped in the 1% AEP with climate change (as can be seen in Appendix D) partially due to the high lake levels and the high flow event. It may also overtop if there is significant blockage at the bridge during a flood event.

Table 3.3: Eastern Stopbank performance grade

Chainage (SB only, m)	2% AEP flood level (mRL)	SB height (mRL)	Actual Freeboard (m)	P (Actual Freeboard/Design freeboard) %	Performance Grade
0	358.70	360.07	1.38	344%	1
6	358.71	359.48	0.77	192%	1
14*[1]	358.82	359.19	0.37	NA	5
30*[1]	358.90	359.19	0.29	NA	5
51*[1]	358.98	359.18	0.20	NA	5
66	359.03	359.50	0.47	118%	1
90	359.05	359.51	0.46	114%	1
93.5	359.06	359.50	0.44	111%	1
106	359.07	359.48	0.41	104%	1
118.5*[1]	359.07	359.20	0.13	NA	5
131*[1]	359.08	359.21	0.13	NA	5
157.5*[1]	359.09	359.29	0.20	NA	5
198.43*[1]	359.09	359.40	0.31	NA	5
212.86*[1]	359.10	359.42	0.32	NA	5
236.85	359.12	360.03	0.90	225%	1
312.2	359.20	360.18	0.98	246%	1
389	359.24	360.34	1.10	276%	1
405.25	359.24	360.31	1.07	269%	1
459	359.30	360.31	1.02	254%	1
516	359.36	360.44	1.08	269%	1
520	359.37	360.46	1.09	272%	1
596	359.51	360.44	0.93	233%	1
603	359.52	360.60	1.08	270%	1
*[1] Shaded cells indicate spillway section					

3.2.3 Western Stopbank

The Western Stopbank is an earth embankment, and the longest stopbank of the scheme at approximately 1150 m. The stopbank provides flood control to the properties to the western side of the river and SH1 as well as to Oruatua. Figure 3-7 shows the location of the stopbank while Figure 3-8 shows a long section along the stopbank with the different AEP flood events.

Figure 3-7: Western Stopbank location map

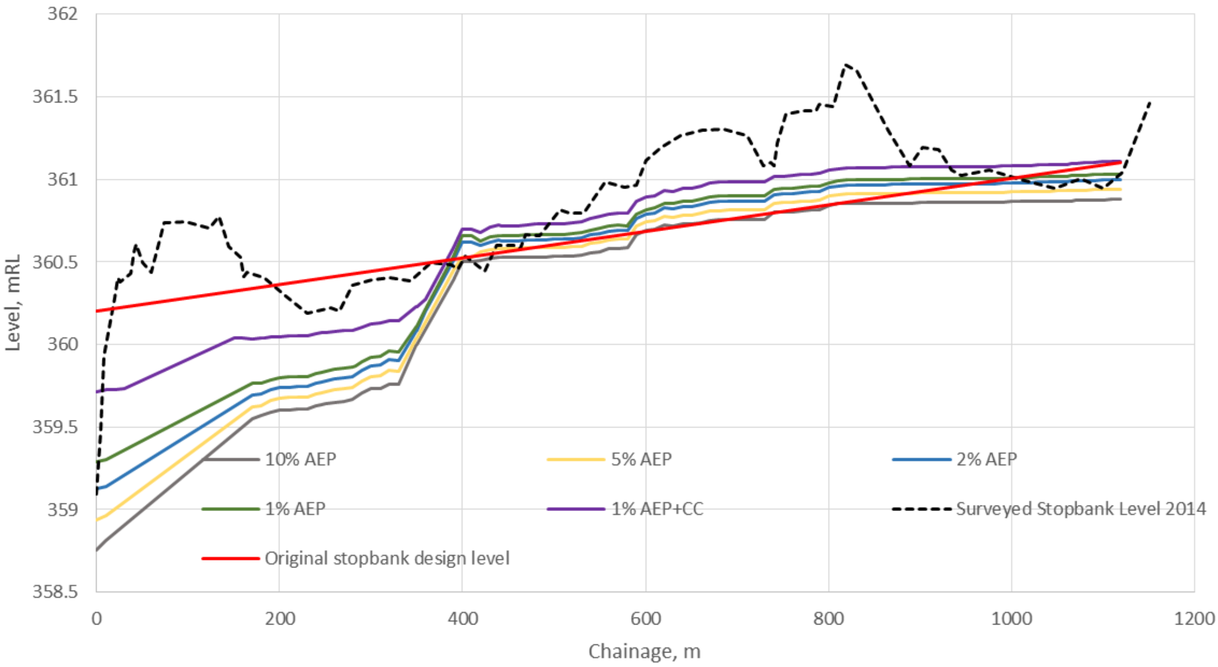

Figure 3-8: Western Stopbank system performance based on a range of events

The model results show a step between chainages 300 and 400. The results presented in Figure 3-8 are based on the 2D model results (taken from the area immediately adjacent to the stopbank). The terrain was examined in this area and no clear anomalies were observed, however, the LiDAR shows some high ground in the area, refer to Appendix I for terrain near the Western Stopbank based on LiDAR. Additional ground survey is recommended (refer to Section 7 of this report) to confirm the ground levels in this area particularly given the heavy vegetation, this is discussed further below.

The model shows overtopping of the stopbank near Chainage 400 (marked by the black circle on Figure 3-7 above). Table 3.4 below shows the flow and volume associated with this overtopping.

It should be noted that in 2005 a T+T letter to WRC dated 01/09/2005 raised issues regarding higher water level observed adjacent to the Western Stopbank and within the Oruatua Reserve area during flood events.

Table 3.4: Discharge over the Western Stopbank

Event	Discharge (m^3/s)	Volume (m^3)
10%	0.1	5,400
5%	0.2	8,600
2%	0.6	16,700
1%	4.4	30,700
1% + Climate change	1.2	94,000

The Western Stopbank is designed for the 2% AEP event with 400 mm freeboard. The table below shows that the freeboard is reduced in many sections of the stopbank, therefore the performance grade of the stopbank ranges from grade 1 to grade 5. There is high uncertainty in the model in this area as discussed in Section 2.4 due to the dense vegetation on the floodplain potentially causing ground levels to be higher than they are in reality and therefore affecting the results of the hydraulic model.

In addition, the channel cross section survey shows that the channel thalweg in this area of the river remains around 356 mRL and drops away rapidly to RL 354 m downstream. Given these unusual results (for cross section 9, 10, 11), we recommend further ground survey to confirm the validity of the 2016 cross section survey, and the LIDAR data in this area.

Survey along the Western Stopbank crest also confirms that some parts of the stopbank are below the original stopbank design level.

Table 3.5: Western Stopbank performance grade

Chainage (SB only, m)	2% AEP flood level (mRL)	SB height (mRL)	Actual Freeboard (m)	P (Actul Freeboard/Design freeboard)	Performance Grade
0	359.13	359.09	-0.03	-9%	5
5	359.13	359.47	0.34	84%	2
8	359.14	359.93	0.79	198%	1
97	359.44	360.74	1.30	325%	1
207	359.74	360.29	0.55	137%	1
300	359.87	360.39	0.52	129%	1
343	360.02	360.39	0.37	92%	2
388	360.49	360.48	-0.01	-2%	5
463	360.63	360.58	-0.05	-13%	5
484	360.64	360.66	0.02	6%	4
517	360.64	360.80	0.16	39%	3
556	360.68	360.99	0.31	77%	2
617	360.82	361.20	0.38	95%	2
662	360.85	361.30	0.45	113%	1
711	360.87	361.27	0.40	100%	2
740	360.91	361.08	0.17	43%	3
744	360.91	361.22	0.31	77%	2
753	360.91	361.40	0.48	121%	1
831	360.97	361.66	0.69	173%	1
865	360.97	361.30	0.33	83%	2
889	360.97	361.08	0.11	28%	3
903	360.97	361.20	0.22	56%	2
935	360.97	361.06	0.08	21%	4
1011	360.98	361.00	0.02	5%	4
1048	360.99	360.95	-0.04	-10%	5
1076	360.99	361.01	0.02	4%	4
1101	361.00	360.94	-0.05	-13%	5
1121	361.00	361.04	0.05	11%	4
1151	361.00	361.46	0.46	116%	1

3.2.4 Maniapoto’s Bend Bed Control Structure and Kiko and Manaipoto Spillways

The Bed Control Structure at Maniapoto’s bend is designed to be 40 m wide (bottom of the left bank to bottom of the right bank), 6 m long (along the channel) and sloped with the highest level at the right side at 361.92 mRL and lowest level at the left side at 361.63 mRL. The structure was not surveyed and therefore, is not represented in the model. The channel at this location is represented in the model as follows:

- Cross section 22, located approximately 30 m upstream of the Bed Control Structure has been surveyed. Figure 3-9 shows a comparison between this cross section and the Bed Control Structure design as well as the downstream interpolated cross section
- The survey at cross section 22 shows levels similar to the Bed Control Structure (although wider at 65 m). Also, there is an area with a lower bed level on the left side of the channel (refer to below to show cross sections relevant in the area and Appendix E for cross section survey locations).
- An interpolated cross section 70 m downstream (halfway between cross sections 21 and 22) shows a lower bed level and is approximately 45 m wide.

The Bed Control Structure should be surveyed. It may then be compared to its design parameters to confirm whether the left bank revetment and end of the Bed Control Structure has failed. If so, reconstruction back to design levels will be required, and the model should be adjusted to the design levels/width to gain a more accurate representation of how the system will perform in this location when the structure is restored to design levels.

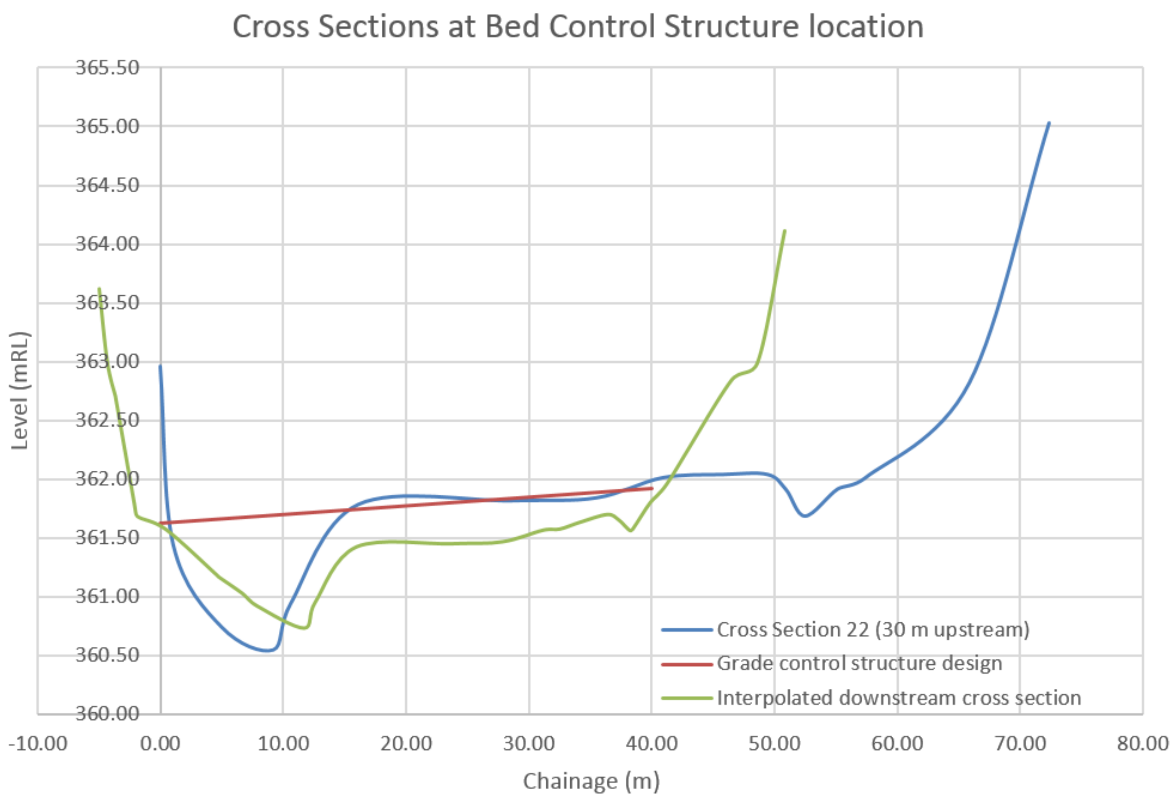

Figure 3-9: Cross section 22 and Bed Control Structure design comparison

Maniapoto and Kiko Spillways on the left bank at Maniapoto's Bend, immediately upstream of the Bed Control Structure are the main spillways designed to convey approximately 25-30% of the peak discharge in the design flood from the main channel to the lake via the Kiko Swale. The left bank at Maniapoto's Bend also includes erosion protection (rock rip rap) upstream of the spillway (80m) and downstream of the spillway (100m). Figure 3-10 shows the location of the spillways, the Bed Control Structure as well as the erosion protection structures.

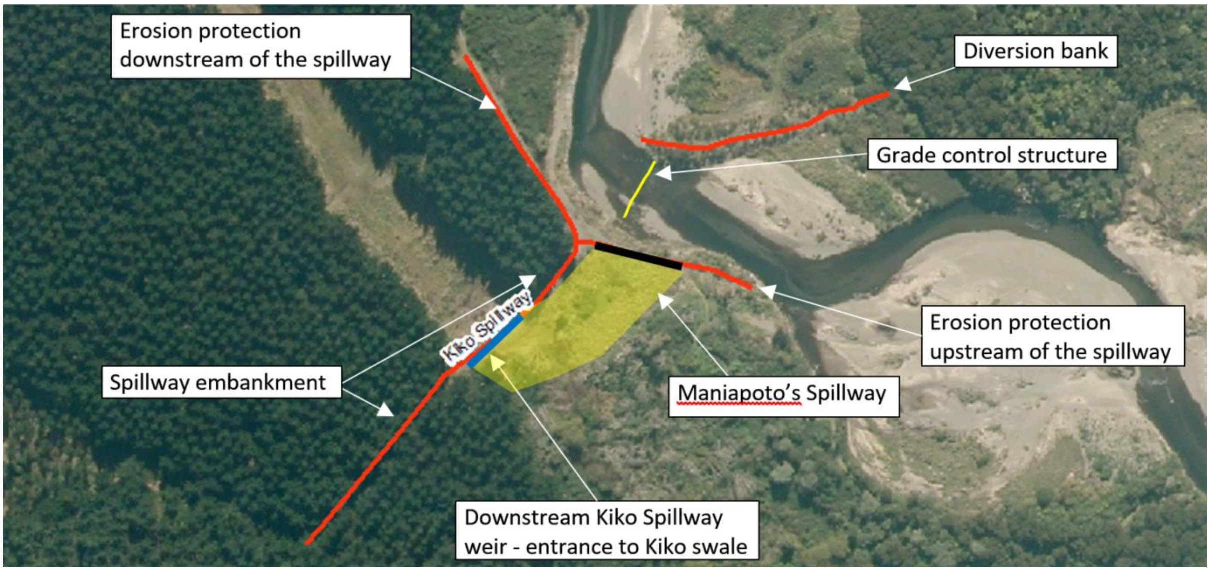

Figure 3-10: Kiko and Maniapoto Spillway location map

Kiko and Maniapoto Spillways are designed to provide a safe conveyance starting from events less than the annual event. The river should overtop the left bank spillway (black line), flowing through the yellow shaded area, and then crossing the weir at the entrance to the Kiko Swale (blue line). Figure 3-11 shows the 2016 LIDAR ground levels (terrain) in the vicinity of the left bank Kiko Spillway entrance. The figure shows that levels in this area are variable and up to 700 mm higher than the downstream spillway weir at the entrance to the swale which has a general ground level of 363.2 mRL. This requires truthing with ground survey.

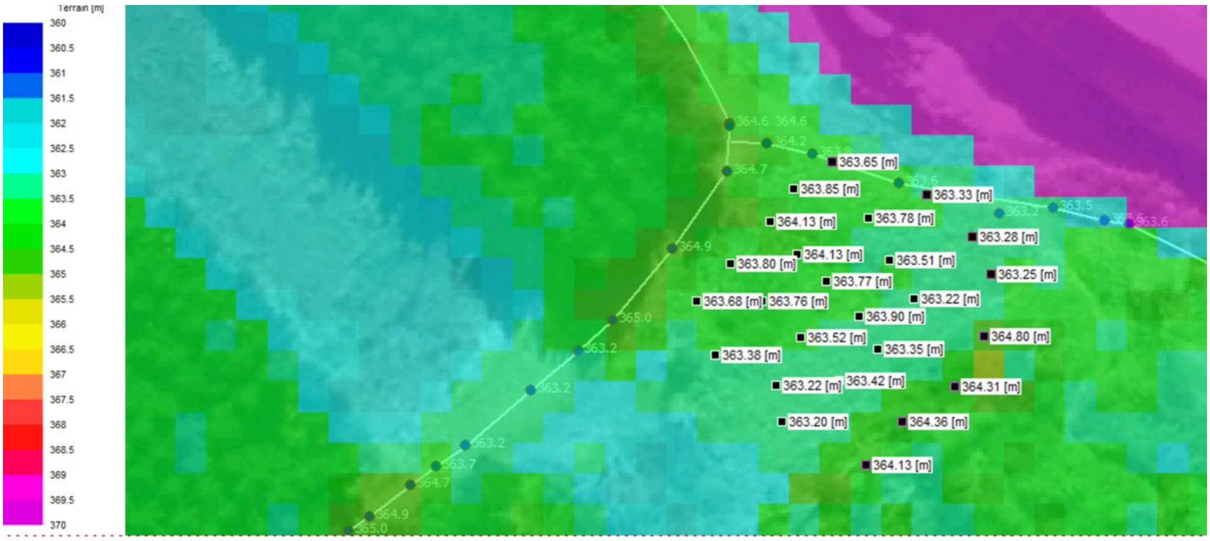

Figure 3-11: Ground level (terrain from LIDAR) in the vicinity of Maniapoto's Spillway entrance

Figure 3-12 below shows the modelled events overtopping the Manaiapoto Spillway entrance. However, the volume of water overtopping the entrance is not enough to overtop the downstream Kiko Spillway. This can be seen in the flood extent map for the 50% AEP in Appendix D.

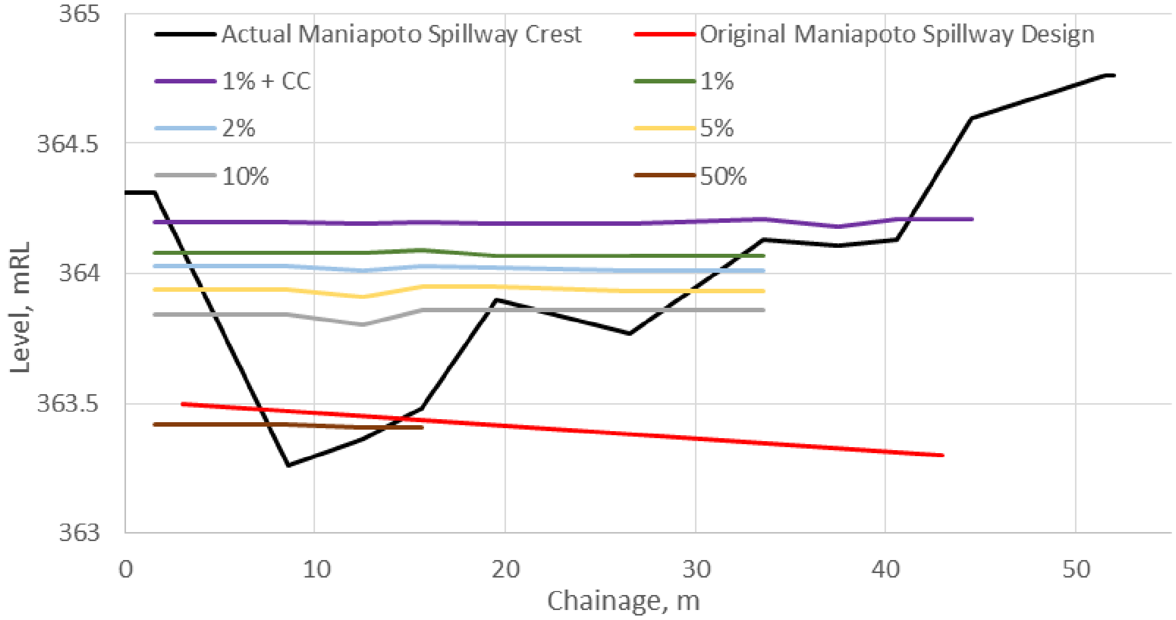

Figure 3-12: Cross section across Manaiapoto Spillway entrance performance based on a range of events – looking at the spillway from the river, section drawn with Chainage zero at upstream end

Figure 3-13 shows the model results for the different flood levels at downstream Kiko Spillway weir (over marked blue cross section in Figure 3-10, entrance to the Kiko Swale) during different events. It is noted that the spillway does not operate in the model as frequently as designed, this is discussed below.

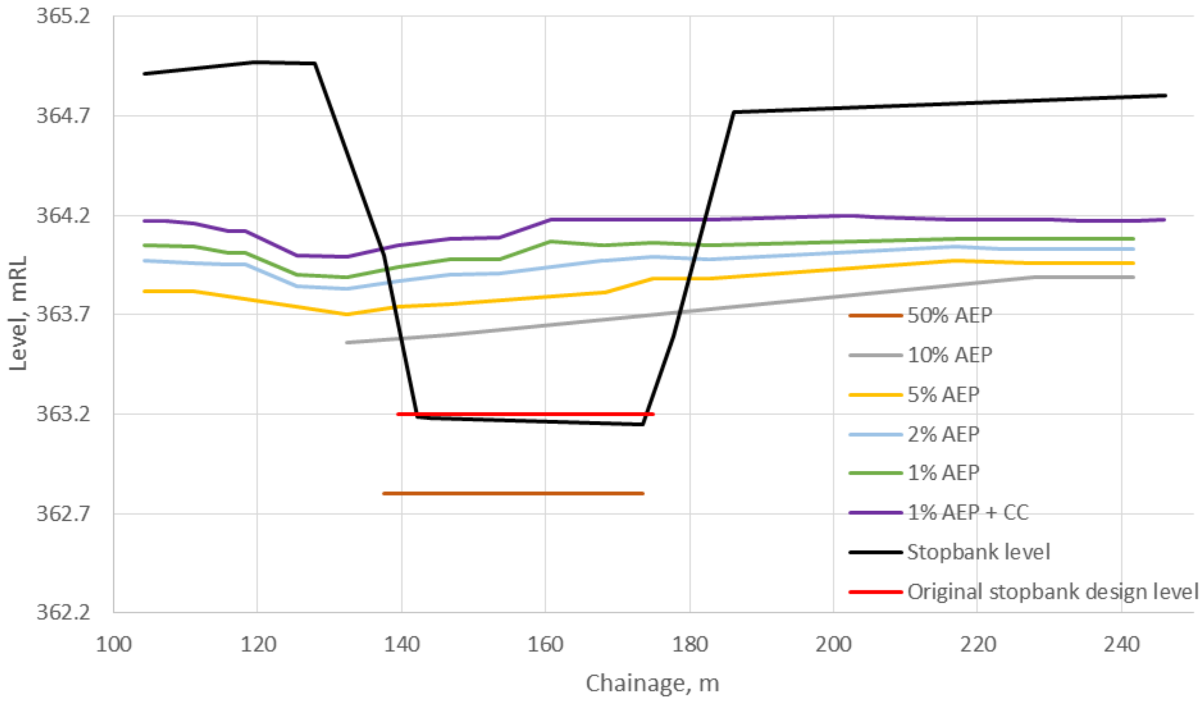

Figure 3-13: Downstream Kiko Spillway weir performance based on a range of events – downstream to upstream (true right bank to true left bank)

The results of the model in this area show that the 50% AEP would overtop the downstream Kiko Spillway weir if enough volume could overtop the Maniapoto Spillway entrance. Similarly with other events, they indicate that the downstream Kiko Spillway weir is more likely to overtop and convey a higher discharge if the Maniapoto Spillway is lower.

The modelled calibration event showed the water levels at the Kiko Spillway are within 10 mm of the measured water level, confirming that the actual water level is represented well in the model in this location.

Table 3.6 below shows water level is not high enough for the downstream Kiko Spillway weir (due to the high left bank spillway entrance) to overtop during the 50% AEP and therefore the performance grade of 5 for the spillway. This is causing more water to be conveyed down the channel to the river mouth in the 50% AEP event. Other comments regarding the performance of the downstream Kiko Spillway weir are provided in Section 3.4 below.

Table 3.6: Downstream Kiko Spillway weir performance grade

Chainage (SB only, m)	2% AEP flood level (mRL)	50% AEP flood level (mRL)	SB height (mRL)	Actual Freeboard (m)	P (Actual Freeboard/Design freeboard)	Performance Grade
104.29	363.97		364.91	0.94	235%	1
119.53	363.93		364.97	1.04	260%	1
127.93	363.84		364.96	1.12	281%	1
137.63	363.86		364.00	0.14	35%	1
142.13*[1]		362.80	363.19	0.39	NA	5
143.93*[1]		362.80	363.18	0.38	NA	5
173.43*[1]		362.80	363.15	0.35	NA	5
177.83*[1]		362.80	363.59	0.79	NA	5
186.13	363.99		364.72	0.73	183%	1
246.13	364.03		364.80	0.77	193%	1
*[1] Shaded cells indicate spillway section						

3.2.5 Kiko Culverts

The Kiko Culverts are located under SH1. The performance measure of these culverts is the duration which the SH1 road and the paddock upstream of the culvert remain under water. The design requirement is that water drains from the area upstream of the culvert within approximately 1 day during the 2% AEP. The discharge over the road is summarised in Table 3.7. The duration of ponding over the area upstream of the culverts is estimated by the model at approximately 27 hours during the 2% AEP.

Figure 3-14: Kiko Culverts

Table 3.7: Discharge and flooding duration over SH1 at Kiko Culvert

Event	Discharge over the road (m^3/s)	Duration of discharge over the road (hours)
2% AEP	14	5
1% AEP	25	7.5
1% AEP + CC	50	11

3.2.6 Quarry Closure

The Quarry Closure stopbank is an earth embankment in two sections, upstream and downstream, separated by the Quarry Closure Spillway. The stopbank was constructed in order to re-divert the river back to its original course following the 2001 event when the river diverted into the quarry. The spillway is designed to overtop during the 10% AEP event to provide controlled spill and flood storage within the quarry. Figure 3-15 shows the location of the stopbank while Figure 3-16 shows a long section along the stopbank with the different AEP flood events.

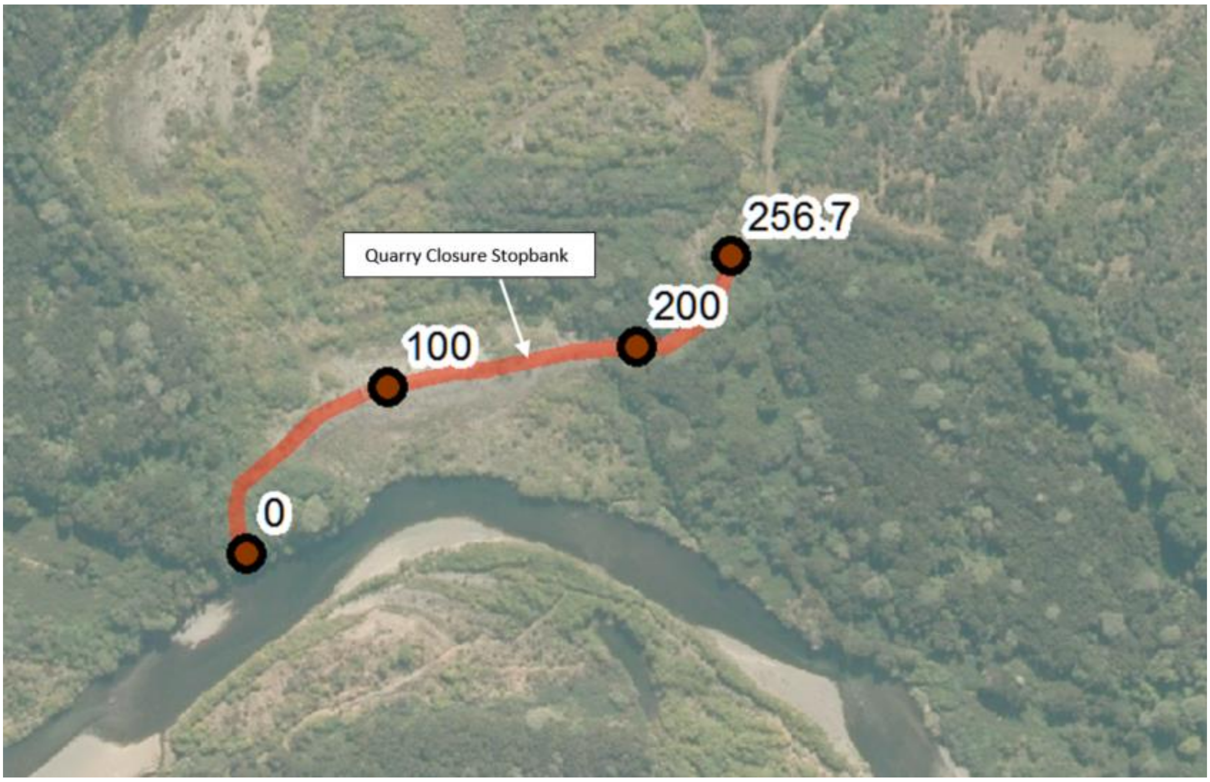

Figure 3-15: Quarry Closure Stopbank location map

Table 3.8 below shows that the freeboard is reduced in both the upstream and downstream sections of the stopbank and the performance grade of the stopbank ranges from grade 1 to grade 2. In addition, the water is more than 100 mm below spillway crest height during the design overtopping event (10% AEP) so the spillway performance is grade 5 due to it not operating during its design event. The model shows that the spillway only starts to operate in the 2% AEP with only 1 m³/s (this is discussed further in Section 3.3 of this report).

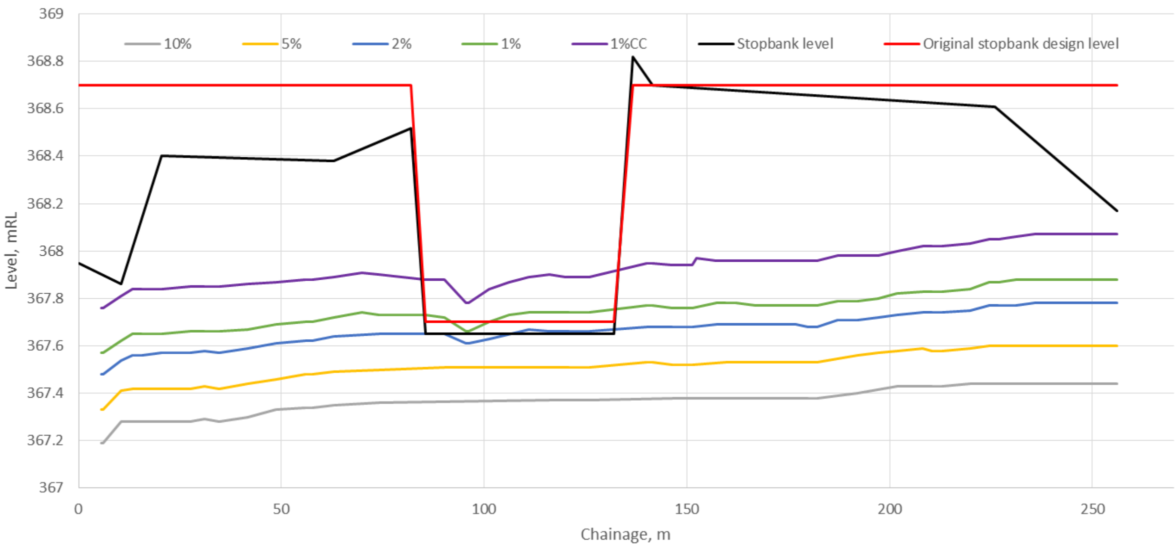

Figure 3-16: Quarry Closure stopbank system performance based on a range of events

It should again be noted that there is high uncertainty in the model in this area. As discussed in Section 2.4, this is due to the dense vegetation on the floodplain, potentially causing ground levels to be recorded by LiDAR at a level higher (or lower if data is interpolated between two low points with no data available in densely vegetated areas) than they are in reality, thereby affecting the results of the hydraulic model. However, from the stopbank survey results it also appears that some parts of the stopbank and spillway are below the original stopbank design level.

Table 3.8: Quarry Closure Stopbank performance grade

Chainage (SB only, m)	2% AEP flood level for stopbanks, (mRL)	10% AEP flood level for spillways (mRL)	SB height (mRL)	Actual Freeboard (m)	P (Actual Freeboard/Design freeboard)	Performance Grade
0	367.48		367.95	0.47	117%	1
10.5	367.54		367.86	0.32	80%	2
20.44	367.57		368.40	0.83	207%	1
62.93	367.64		368.38	0.74	185%	1
82	367.65		368.52	0.87	217%	1
85.51	367.65	367.67	367.65		NA	5
132	367.67	367.36	367.65		NA	5
136.64	367.68		368.82	1.14	286%	1
141.54	367.68		368.70	1.02	255%	1
226	367.77		368.61	0.84	210%	1
256.7	367.78		368.17	0.39	98%	2
*[1] Shaded cells indicate spillway section						

During the site inspection the upstream and downstream ends of the stopbank were inspected and found to be heavily vegetated confirming possible uncertainty in LiDAR.

3.3 Overall scheme performance based on discharge

The Tauranga Taupo scheme design depends on the balance of discharge upstream and downstream of Maniapoto's Bend, as well as through the quarry on the right bank, the relic channel on the left bank (downstream of Maniapoto Bend), and an area adjacent to the Western Stopbank and the channel. The performance of the scheme relies on the intricate balance and timing of water entering and exiting the different floodplain areas to reduce the risk downstream at Oruatua and Te Rangiita. Figure 3-17 shows how the system is expected to perform, while Table 3.9 shows the design discharge compared to the discharge from the 2017 model for the different design events.

When balance in not achieved in one part of the scheme there are consequences on the remaining parts. The results of the hydraulic analysis show that, in general, there is more water conveyed within the channel and less water being conveyed through the floodplain. The effects of this may be detrimental for the scheme performance, however the model shows that the bridge will only overtop in events larger than the 1% AEP. We note that water levels at the Kiko Spillway weir and the bridge were within 10 mm and 60 mm respectively in the calibration event, suggesting that the water level is represented fairly well in the model in these areas.

Table 3.9: Discharge comparison (design vs 2017)

Flow at Key Locations (m3/s)	Annual Exceedance Probability						
	Mean Annual Flood (MAF)	**10%**	**5%**	**2%**	**1%**	**0.5%**	**1% + CC**
Te Kono Guage	150	235	271	318	353	388	
Te Kono Guage (2017)	140	231	264	305	~335		~429
Overflow into Quarry via spillway	0	1.5	11	30	45	62	
Overflow into Quarry via spillway (2017)[3]		0	0	~1	4		~14
Discharge into the quarry downstream of the Quarry Closure Bank (m³/s) -(2017)		0.3	~2	5	~8		16
Discharge into the quarry upstream of the Quarry Closure Bank (m³/s)- (2017)			0	~0.2	~1		10
Total discharge into the quarry (m³/s) – including side spill- (2017)		0	2.2	10	20		60
Kiko Overflow (spillway and side inflows)	23	60	72	82	91	99	
Kiko Overflow (spillway only) - (2017)	0	~7	~13	~20	~25		~23
Kiko Overflow (spillway and side inflows) - (2017)	0	9	~15	~26	~33		~51
Kiko overflow Left Bank - 2017	0.1	4.0	5.3	5.8	6.1		6.4
Right Bank Overflow – across SH1	0	0	0	4	12	20	
Right Bank Overflow – across SH1 (2017)	0	0	0	0	0		40
Under SH1	125	164	175	205	235	246	
Under SH1 (2017)	136	218	242	247	280		296
Note: Not listed are right bank natural overflows both into the quarry and down the secondary channel. Shaded cells indicate results from the 2017 service level review model							

Also of note is the importance of the expected volume of each design storm event, as the system depends on storage within the quarry and other parts of the floodplain and if the stored volume exceeds the design volume, higher water levels should be expected (as observed in the 2015 event used for calibration).

[3] For location of the overflow refer to Appendix C

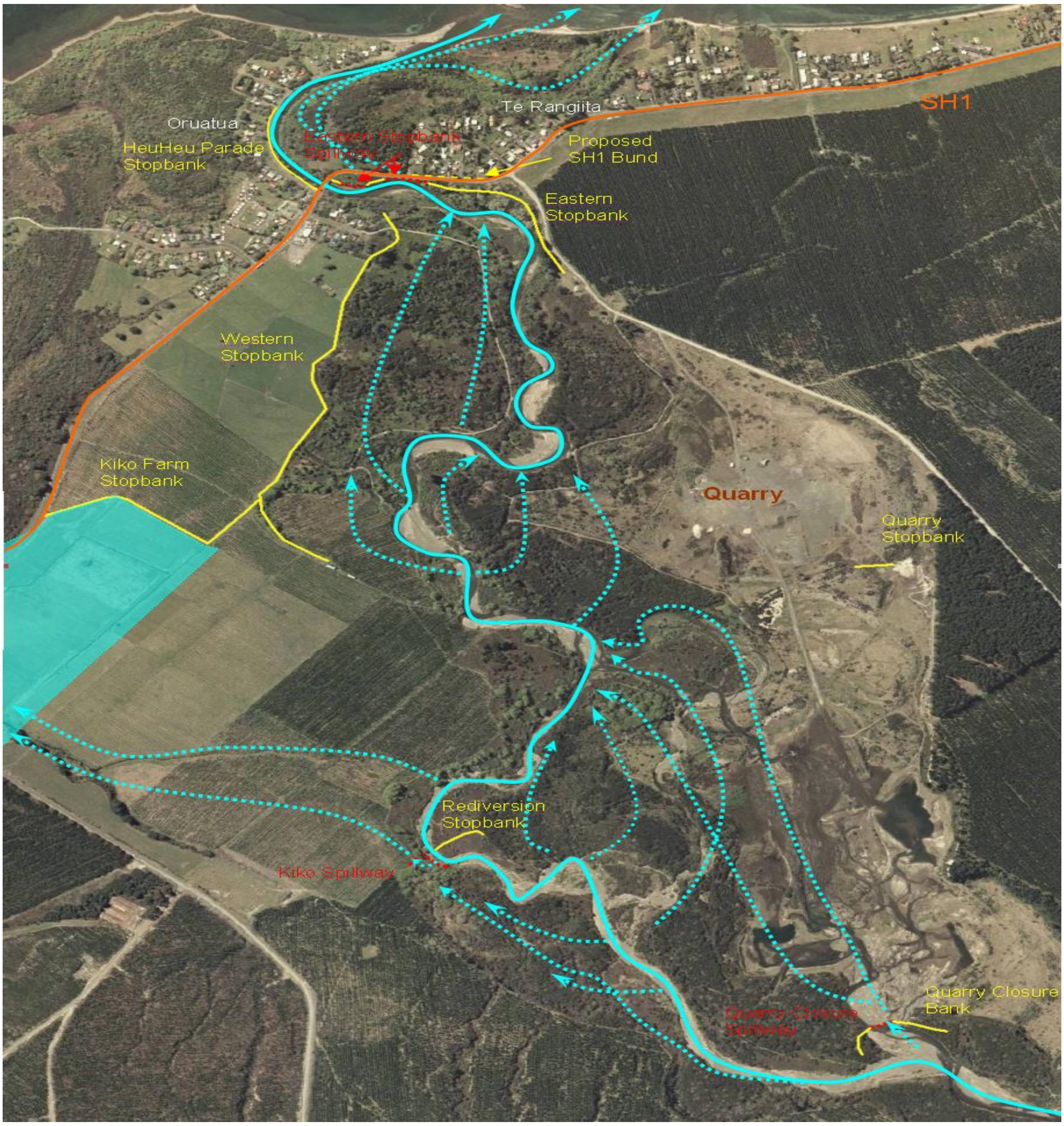

Figure 3-17: Scheme design plan

The results of the model summarised in Table 3.9 show that during the design event the following can be observed:

- At the Quarry Closure StopBank
  - The Quarry Closure Spillway conveys 1 m³/s in the model compared with the design flow of 30 m³/s during the 2% AEP event, and does not operate at all during the 10% AEP event in the model (the design event it is meant to start operating at). If considering all flow across the quarry during this event in the model this becomes 10 m³/s.
  - The model also shows water entering the quarry at the downstream end of the stopbank during the 10% AEP event (note the uncertainty associated with LiDAR in this area is likely to influence the results in the vicinity).
  - The model shows that the water outflanks the Quarry Closure Stopbank at the upstream end in events equal to and larger than the 1% AEP. This is not consistent with

observations by Taupo staff where water has been observed to pass around the upstream side of the Quarry Closure Stopbank and into the quarry in events such as the 2015 event which was similar to a 5% AEP in peak discharge.

- The area upstream of the Quarry Closure Stopbank is densely vegetated, which could indicate another scenario where vegetation is causing an over estimation of the ground levels.

- At Maniapoto's Bend
  - In the model, for the 2% AEP design event, the Kiko Spillway weir conveys 20 m^3/s, or when all overflows are taken into account, the total discharge is shown to be 26 m^3/s. This is significantly lower than the design discharge of 82 m^3/s during the 2% AEP.
  - We note, however, that the water level observed during the 2015 calibration event and the modelled water level across the spillway were within 0.01 m. This suggests reasonable confidence regarding the potential for flow over the downstream Kiko Spillway and in this area of the model. However, it also confirms that the combined spillway does not operate as intended in the design, this is most likely due to the higher than design ground levels at the Maniapoto Spillway and the area between the two spillways.
- Model results show that the SH1 Bridge is likely to convey larger flows prior to overtopping (providing there is no blockage / constriction caused by debris build up on the bridge piers). It should be noted that water levels at this location are highly influenced by tailwater level (Lake Taupo level), as is the bridge conveyance at this location.

3.4 Velocity

Velocity is not stated as a design parameter for any part of the scheme, however, it is used as an indicator to highlight potential areas at risk of a break out or severe erosion. The scheme design standard (Table 1.2) indicates that at Maniapoto's Bend, stability needs to be maintained in order to ensure that the river will not divert into Kiko Spillway, and that the design cross section in the river is stable and flows are distributed between the main channel and Kiko Spillway.

Velocity in the vicinity of Maniapoto's Bend is important as it is an indicator for areas that are likely to experience high erosion and therefore most likely to be where the river breaks out. Figure 3-18 shows the areas on the right bank and left bank where velocities were taken from the model (for maps showing velocity values refer to Appendix B). These are summarised in Table 3.10.

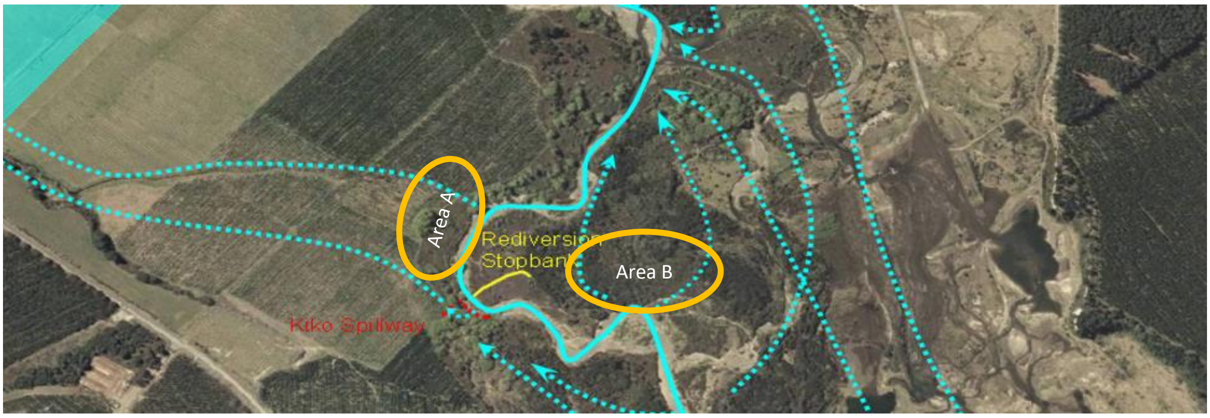

Figure 3-18: Maniapoto's Bend overflow area of interest for velocity

Table 3.10: Velocities on the right bank and left bank overflows at Maniapoto's Bend

AEP event (%)	Velocity at Area A (m/s)	Velocity at Area B (m/s)
Annual event	0	0
10%	Up to 1.69	Up to 0.43
5%	Up to 1.99	Up to 0.5
2%	Up to 2.23	Up to 0.56
1%	Up to 2.34	Up to 0.61
1% + CC	Up to 2.52	Up to 0.7

The erosion protection on the left bank around Maniapoto's Bend extends approximately 80 m upstream of Maniapoto Spillway and 100 m downstream of the spillway. The area showing highest velocities is immediately downstream of the erosion protection works.

4 Analysis of channel survey data

Channel cross-sections in the area covered by the model and scheme assets were surveyed in 2008 and 2016. Cross sections sharing roughly the same alignment were compared for changes in cross section area and minimum invert levels (channel thalweg). Appendix E shows the cross section locations while Appendix F shows the cross section from the 2008 and 2016 surveys.

The majority of the cross sections have been re-aligned as the river channel has moved laterally causing river re-alignment.

The analysis to determine the changes in cross sections allows a general assessment of degradation and aggradation applicable to these cross sections. This analysis may be broadly applied to the area between cross sections with caution. It should be noted that no certainty is provided regarding interpolation between those cross sections and only a full channel survey covering the entire length of the channel can be used for an accurate estimate of the channel aggradation and degradation given two sets of data.

The data analyses indicate the following changes in cross section area between 2008 and 2016:

- Section between cross section 1 and 9 are not fully surveyed (i.e. their extents are different to those in 2008) therefore any comparison is only applicable to the area of overlap between the two surveys. In general channel thalweg is decreasing in this area.
- Cross section 10 shows both an increase in channel thalweg and a decrease in channel capacity suggesting aggradation in this section.
- Sections between cross section 11 and 34 are showing signs of degradation. This is observed by the general lowering of the channel thalweg (ranging between -0.25 and -1.56 m for these cross sections) as well as the increase in cross section area when comparing the two sets of surveys. Considering this, the results of the model are consistent with this observation, showing that more water is conveyed in the channel and the water levels are not high enough for sufficient water to spill into the quarry and the Kiko Spillway.

Table 4.1 shows the channel cross section area and change in area between 2008 and 2016 (2016-2008). Note the area is calculated and compared for a common level at each cross section, and comparing areas along the reach is not recommended.

Typically a degrading channel has little effect in terms of flood mitigation (strictly speaking and disregarding effects of erosion) as it presents a larger cross section area to convey the flood flows.

However, as the Tauranga Taupo system requires balance through using the capacity available both in the floodplain and the channel (as well as conveying flow through Kiko Spillway), monitoring should be undertaken at regular time intervals to assess both the capacity of the channel and the role it plays in the overall performance of the scheme.

Table 4.1: Comparison in cross section area between 2008 and 2016 data

Cross section	Location	Realigned Channel (C) and/or cross section(XS)	XS area 2008	XS area 2016	Change ratio*1	Cross section area trend	Channel thalweg *2 change (m)	Channel thalweg trend	Comment
1	River mouth	No	76.0	74.7	-2%	Decreasing	-0.11	Decreasing	
2		No	63.2	64.5	2%	Increasing	-0.06	Decreasing	2016 XS not complete to ToB*3
3		No	63.3	58.2	-9%	Decreasing	-0.02	Decreasing	2016 XS not complete to ToB
4	Heuheu Parade SB	No	70.2	65.4	-7%	Decreasing	0.03	Increasing	2016 XS not complete to ToB
5	DS of SH1 Bridge	No	54.7	66.8	18%	Increasing	-0.50	Decreasing	2016 XS not complete to ToB
6	US of SH1 Bridge	Yes (XS)	65.5	58.5	-12%	Decreasing	-0.68	Decreasing	
7	Eastern SB	Yes (XS)	65.2	59.1	-10%	Decreasing	0.03	Increasing	2016 XS not complete to ToB
8	Eastern SB	Yes (XS)	67.7	59.7	-13%	Decreasing	-0.45	Decreasing	2016 XS not complete to ToB
9	Western SB	No	64.0	63.5	-1%	Decreasing	-0.11	Decreasing	2016 XS not complete to ToB
10	Western SB	Yes (XS)	71.6	67.0	-7%	Decreasing	0.89	Increasing	Aggradation
11	Western SB	Yes (XS)	55.8	70.6	21%	Increasing	-1.17	Decreasing	Degradation and erosion of the LB *4
12	Western SB	Yes (XS)	53.1	51.6	-3%	Decreasing	-0.90	Decreasing	Degradation and erosion of the LB
13	Western SB	Yes (XS)	52.2	72.1	28%	Increasing	-0.59	Decreasing	Degradation and erosion of the RB*5
15		Yes (XS, C)	59.9	69.0	13%	Increasing	-1.37	Decreasing	Degradation
16		Yes (XS, C)	68.3	79.9	15%	Increasing	-0.41	Decreasing	Degradation
17	DS of Crescent loop	Yes (XS)	39.8	65.7	39%	Increasing	-0.52	Decreasing	Degradation and erosion of the RB
18	US of Crescent loop	Yes (XS, C)	57.5	98.1	41%	Increasing	-1.56	Decreasing	Degradation, erosion of the LB

Cross section	Location	Realigned Channel (C) and/or cross section(XS)	XS area 2008	XS area 2016	Change ratio*1	Cross section area trend	Channel thalweg *2 change (m)	Channel thalweg trend	Comment
19		Yes (XS, C)	45.6	75.3	39%	Increasing	-0.38	Decreasing	Degradation, erosion of the LB
20	Maniapoto's Bend	Yes (XS, C)	55.5	72.4	23%	Increasing	-0.25	Decreasing	Degradation, erosion of the LB
21	Maniapoto's Bend	Yes (XS, C)	48.9	83.1	41%	Increasing	-0.26	Decreasing	Degradation erosion of the LB
26		Yes (XS, C)	27.3	82.6	67%	Increasing	-0.16	Decreasing	Degradation, erosion of the RB
27		Yes (XS, C)	33.7	67.9	50%	Increasing	-0.78	Decreasing	Degradation, erosion of the RB
28		Yes (XS, C)	39.9	64.2	38%	Increasing	-0.88	Decreasing	Degradation, erosion of the LB
29	DS of Quarry Closure Bank	Yes (XS, C)	37.9	54.8	31%	Increasing	-0.73	Decreasing	Degradation, erosion of the RB
30		Yes (XS)	46.4	70.8	34%	Increasing	-0.64	Decreasing	Degradation
31	Quarry Closure Bank spillway	Yes (XS)	38.7	62.9	38%	Increasing	-0.82	Decreasing	Degradation
32		No	56.8	92.1	38%	Increasing	-0.40	Decreasing	Degradation
33		No	28.0	57.9	52%	Increasing	-0.28	Decreasing	Degradation, and erosion of LB & RB
34		Yes (XS)	47.0	75.3	38%	Increasing	-0.68	Decreasing	Degradation

*1 Positive % indicated increase in channel capacity from 2008 to 2016, *2 Negative indicates 2017 thalweg is lower than 2008, thalweg, * 3 ToB = Top of Banks,*4 LB = Left bank, *5 RB = Right Bank

5 Aerial photography and site visit observation

Oblique photographs (taken on the day of the site visit on 8/9 Jan 2018 and shown in Appendix H) as well as channel alignment observations (supplied by WRC – refer to Appendix E) were inspected for any information which may have been missed from the site visit, model information and cross section information. The advantage these provide is that they help piece together areas where ground observation is too difficult due to access issues (dense vegetation etc.). The following observations can be made:

5.1 Quarry Closure Bank

Area 1: An area approximately 500 m upstream of the Quarry Closure Bank (in the vicinity of XS 33) has an island which has been forming since 2005 as can be seen the 2005, 2013 and 2014 Aerial photographs in Appendix H The latest aerial photograph (taken on the day of the site visit also in Appendix H shows that vegetation has established on the island. This in turn is likely to accumulate more debris on the island, likely to result in elevation of water levels in the vicinity.

It should be noted that the model includes the two channels on each side of the island, in the 1D model, but also the right hand channel is included in the 2D model. This is effectively increases the water conveyance within the modelled channel. Therefore, it is likely resulting in lower water levels than those observed by field staff (who have reported that the river is outflanking the upstream section of the Quarry Closure Bank).

The 1% AEP plus climate change flow vectors (Appendix H) show that water outflanking the upstream section of the Quarry Closure Bank is indeed being conveyed from the area upstream, and not back flowing from immediately opposite the Closure Bank).

As more water is conveyed into the quarry upstream of the Closure Bank it is likely that as the river reaches the Quarry Closure Spillway, water levels are lower, this in combination with the degrading channel is likely to affect the operation of the Quarry Closure Spillway as has been observed by field staff.

Area 2: This area is on the right bank immediately downstream of Area 1. The aerial photos show that gravel has been building on the area between 2005 and 2014 and vegetation can be seen established in the 2017 aerial photograph. This is likely to increase water level in the vicinity of Area 2, again increasing the water levels upstream of the Quarry Closure Bank.

Area 3: This area is on the left bank, immediately opposite the Quarry Closure Bank (in the vicinity of cross section 31). The gravel shoal has established since 2005 and shows vegetation in all subsequent photos. Water levels are also likely to be affected by this, however, cross section 31 shows that the gravel shoal has been lowered by approximately 600 mm between 2008 and 2016. This in addition to the higher water levels causing water to outflank the Quarry Closure Bank and a lower bed level (approximately 800 mm lower) is likely to be the cause of the lower water levels in the vicinity of the Quarry Closure Spillway.

5.2 Maniapoto's Bend

Between 2008 and 2010 the area upstream of Maniapoto's Bend was straightened in order to ease the pressure on the area to the right of the Rediversion Bank where the river was at risk of a breakout (therefore completely bypassing Kiko Spillway). The river is slowly regaining its meandering pattern in this area, however for the moment the low flow channel has remained closer to the diversion bund. During the site visit and walk over of the area downstream of the rock control weir, low level areas were noted where it is most likely that the river overtops the left bank.

The erosion protection on the left bank at Maniapoto's Bend was not visible at its upstream nor at its downstream ends. However, both the 2017 site visit aerial photograph and the 2012 aerial photograph (Appendix H and Appendix E) show that the rock riprap downstream of the Bed Control Structure is likely to be still in place. Survey should be carried out to confirm the presence of the rock riprap upstream of the Bed Control Structure.

There may be a risk of the river changing course and outflanking the upstream end of the erosion protection and increasing the likelihood of diverting through Kiko Spillway. Similarly, downstream of the erosion protection works at Maniapoto's Bend (refer to Section 3.4 discussing velocities at Maniapoto's bend) there is a risk of the river breaking through the left bank and diverting through the pine forest toward the Kiko Culverts, across SH1 and to the lake.

5.3 Western Stopbank

5.3.1 Stopbank

During the site visit, the entire length of the Western Stopbank was inspected. Many fallen trees were noted across the stopbank (fallen onto its crest) in the section between the upstream end of the Western Stopbank and the Twin Barns Stopbank.

A rabbit hole was noted between Chainage 600 and 750, and dense vegetation was confirmed between the stopbank and the river channel.

5.3.2 Channel

Aerial photographs taken on the same day as the site visit show some debris in the channel throughout the reach. While cross section locations (Appendix E) show that the channel has been increasing its meandering pattern in this reach. Increasing meandering patterns indicate a lower slope which is observed in this area between cross section 9 and 11, however it should be noted that this is followed by a steeper section immediately downstream as discussed earlier in Section 3.2.3

5.4 Eastern Stopbank and Heuheu Parade

No significant features were noted during the site visit, the erosion protection adjacent to Heuheu Parade was still visible. However there were gaps forming between the wooden planks of the timber floodwall as the timber has dried.

6 Summary/Conclusions

A summary of the findings of from this service level review are presented below.

6.1 Hydrology

Design peak and hydrograph: The Tauranga Taupo hydrology was reviewed using data from 2004 to 2017. The 2017 estimate for the design events at Te Kono are as summarised in

Table 6.1 below, and are similar to estimates made for the design.

Design Volume: In addition to the design peaks, it was also established that the system is highly sensitive to the volume of rainfall and therefore runoff experienced by the catchment. Particularly as some parts of the floodplain (such as the quarry) are meant to attenuate the flood peaks. These areas can only attenuate a finite volume of flood waters, and an event with rainfall runoff larger than the design volume for a certain AEP should be considered an over design event. The 2017 review shows that the design discharge at Te Kono is slightly lower than the original design discharge for this scheme.

Table 6.1: 2017 Hydrological record review

Percent Annual Exceedance Probability (%AEP)	Estimated discharge at Te Kono (m^3/s) 2017 review
Mean Annual Flood (MAF)	140
20	195
10	231
5	264
2	305
1	335
1 + climate change	429

6.2 Scheme performance and channel hydraulics

1. General comments – Water levels: The results of the 2017 service level review model show that modelled water levels are:
    a. Lower than **design** water levels in most areas, particularly in areas such as the Quarry Closure Bank spillway, and the Eastern Stopbank.
    b. Higher than **design** water levels adjacent to the Western Stopbank. This may be due to the uncertainty in LiDAR survey in those locations
    c. Within 0.01 m of **observed** water levels at Kiko Spillway.
2. General comments – Discharge: The following comments are made with regards to the performance of the scheme assets in relation to discharge. The results of the 2017 service level review model shows that generally speaking, the modelled discharge:
    a. In the channel is larger than **design** discharge.
    b. In the floodplain (including Kiko Spillway and the quarry) is less than **design** discharge.
    c. Under the SH1 Bridge is larger than **design** discharge, however, this is highly sensitive to bridge debris blockage and bed levels upstream of the bridge, channel roughness and lake level.
3. General comments – Channel cross sections: The channel cross sections have shown signs of degradation for the majority of the surveyed cross sections, with the only exceptions being at the river mouth and at cross section 10.

4 Heuheu Parade

a Timber floodwall levels: Figure 3-3 shows floodwall levels slightly lower than design levels in two spots at the upstream and downstream end of the floodwall. It should be noted that during the site visit the floodwall visibly appeared to have a constant grade and no sudden change in height was observed suggesting that the survey data at each end may not be accurate.

b Water levels: Modelled water levels are lower than the design water levels in this area. Model results indicate a freeboard of 0.47 m to 1.05 m during the 2% AEP event. Note the lake level used during this event is RL357 m.

c Bed level and channel cross sections: The 2016 cross section survey does not extend as far as the 2008 cross section survey in this area. Therefore a direct comparison cannot be made. However the general observation is that the bed level is very slightly decreasing in the area downstream of the SH1 Bridge. Immediately adjacent to Heuheu Parade Stopbank a slight increase of 0.03 m is recorded (cross section 4).

d Other observations – Timber flood wall condition: during the site visit it was noted that the timber planks forming the floodwall have dried and shrunk resulting in open joints where water can pass between the planks therefore not forming a water tight structure. However these open joints are generally above the predicted flood level in the freeboard range.

5 Eastern Stopbank

a Eastern Stopbank Levels: Figure 3-6 shows that the Eastern Stopbank level is higher than the design level.

b Water levels and spillway operation: Modelled water levels are lower than design water levels including adjacent to the spillways. This results in larger flows to be conveyed in the channel and under the bridge, although these results do not allow for bridge blockage, and are dependent on the design lake level of 357 mRL.

c Bed level and channel cross sections: The 2016 cross section survey does not extend as far as the 2008 cross section survey in this area. Therefore a direct comparison cannot be made. However the general observation is that the bed level is generally decreasing in the 2016 survey.

d Other observations – Timber floodwall condition: during the site visit it was noted that the Timber planks forming the floodwall have shrunk resulting in open joints where water can pass between the planks therefore not forming a water tight structure.

6 Western Stopbank

a Western Stopbank levels: The stopbank survey data provided suggest the Western Stopbank level is lower than the design level in some areas.

b Water levels: The model shows water levels higher than the design water levels for a limited section of the Western Stopbank. The stopbank overtops around Chainage 400. Field staff confirm that the water level adjacent to the Western Stopbank was higher than expected during the 2015 event confirming model results.

c Bed levels and channel cross section: Cross section 10 shows an increase in the bed level and decrease in cross section area showing aggradation in the channel.

d Vegetation: Dense vegetation adjacent to the Western Stopbank was noted during the site visit which may have caused the ground levels in the LiDAR data being higher than actual ground levels, thereby giving higher modelled water levels in this area.

e Channel Debris: Debris was observed in the channel in the area immediately downstream of Maniapoto's Bend. Debris can cause bridge blockage if it is conveyed downstream, therefore affecting bridge capacity, alternatively it can also encourage other debris and gravel build up reducing channel capacity in the area.

f Other issues: in addition to the above, the following was noted during the site visit:

i Some fallen trees were observed on the Western Stopbank in the area between the upstream section of the stopbank and the Twin Barns Stopbank

ii A rabbit hole was found between chainages 600 and 750.

7 Maniapoto's Bend – Kiko Spillways

a Maniapoto Spillway levels: Figure 3-11 shows ground levels in the Maniapoto Spillway up to 700 mm higher than design levels. This essentially limits the flow through this spillway and into the downstream Kiko Spillway.

b Maniapoto Spillway operation: The model shows the spillway operating during the 50% AEP, however, the volume of flow is relatively small and not enough to cause the downstream spillway to operate to design levels.

c Kiko Spillway weir levels: Figure 3-13 shows that the Kiko Spillway is slightly lower than the design level.

d Kiko Spillway weir operations: The model in this area shows that there is significantly less water going over the Kiko Spillway than designed (20 m^3/s compared with the designed 82 m^3/s). The results of the modelling from the calibration event showed modelled water levels are within 10 mm of those surveyed by field staff for the event. This gives confidence in the model results in this vicinity.

e Discharge in the main channel: Due to less modelled water going through the Spillways, the model shows that more water is conveyed downstream through the main channel in all events. The model predicts overtopping of the left bank at the downstream end of Maniapoto's Bend.

f Bed levels and channel cross sections: Comparing the 2008 and 2016 cross section surveys indicates the bed level is generally decreasing and cross section area increasing indicating a degrading channel in the location of these cross sections (XS 20, 26, 27). Note there are no cross sections near Kiko Spillway from the 2008 survey and therefore no comparison has been made in this vicinity between 2008 and 2016. In addition, no survey has been undertaken downstream of the Bed Control Structure in the area where overtopping is occurring. A cross section survey would confirm both bed and bank levels at this location

g Bed Control Structure: The Bed Control Structure was not surveyed and as a result was not included in the model, a cross section 30 m upstream of the structure was surveyed and included in the model, the cross section shows similar levels, however, a lower level is observed on the left side of the channel.

h Channel debris: Debris was observed in the channel in the area immediately upstream of Maniapoto's bend. Debris can cause bridge blockage if it is conveyed downstream, therefore affecting bridge capacity, alternatively it can also encourage other debris and gravel build up reducing channel capacity in the area.

8 Maniapoto's Bend – erosion protection

a Upstream erosion protection: The left bank rock revetment has either failed (slumped into river or washed downstream) or been covered by sediment and vegetation. This needs to be confirmed through survey and comparison with as-built drawings.

b Risk of upstream erosion and river diversion through Kiko Spillway: The main river channel remains on the right hand side, easing pressure on the left bank. However should the river return to the left bank where it was following the 2008-2010 floods (prior to straightening) there will be a higher risk of the river diverting the majority (or perhaps all) of its discharge through the Kiko Spillway.

c Downstream erosion protection: Aerial photographs taken during the site visit indicate that the original rock rip-rap erosion protection remains in place although is covered in vegetation.

d Risk of downstream erosion and river diversion: Modelling results (supported by site observations made by Taupo staff) show the left bank downstream of the erosion protection at Maniapoto's Bend is being overtopped. Velocities in the area of overtopping range from 1.69 m/s in the 10% AEP event to 2.52 m/s in the 1% AEP with climate change event.

9 Quarry Closure Bank

a Outflanking of the Quarry Closure Bank on upstream side: This has been observed by the Taupo field staff. Model results only show water outflanking the Quarry Closure Bank for events larger than 2% AEP.

b Quarry Closure Bank levels: Survey data (refer to Figure 3-16) suggests that the stopbank levels (upstream and downstream of the spillway) are lower than the design levels.

c Quarry Closure Spillway level: Survey data suggests that the spillway level is slightly lower than the design spillway level.

d Water levels across Quarry Closure Bank Spillway: The model results show that the water levels in the vicinity of the spillway are lower than the design water levels. The spillway was designed to overtop during the 10% AEP event. The model shows that the spillway does not operate until the 2% AEP and with less than one thirtieth of the flow (design flow 30 m^3/s, actual flow 1 m^3/s). Field staff confirm that they have not observed the spillway operate in any recent events including the 2015 event which had a peak higher than the 5% AEP and a volume comparable with the 2% AEP.

e Bed levels and channel cross sections: Comparing the 2008 and 2016 cross section surveys show the bed level at the thalweg in the vicinity of the Quarry Closure Bank generally decreasing while cross section area is increasing, indicating a degrading channel in this area.

f Heavy vegetation: Observations made during the site visit confirm heavy vegetation in the vicinity of the stopbank (particularly upstream and downstream ends) these result in high uncertainty in the stopbank LiDAR level and general ground levels in this area. This may explain why the model is not accurately predicting the overflow around the upstream side of the Quarry Closure Bank.

7 Recommendations

The following recommendations are made with regards to the different aspects of the findings of this report, these are presented as Management Options, for each issue identified in Section 6 above. The Management options can be considered together or separately.

7.1 Hydrology

1. That the design event peaks and hydrographs be adopted as it represents the latest and best estimates for the design events.
2. That in addition to the design peak flow rate, a design volume is established for each event. This is because the system is highly reliant on the amount of storage within the floodplain, and if this is exceeded, higher water levels are more likely.

7.2 Recommendations to improve Scheme performance and channel hydraulics

1. <u>Heuheu Parade</u>
   a. <u>Recommendation 1</u>: Carry out maintenance work on the timber floodwall to ensure gaps are closed/covered between wooden planks.
   b. <u>Recommendation 2</u>: Carry out a complete cross section survey in this area (to include the full extent as per the 2008 survey and compare the cross sections to establish channel trends.
   c. <u>Recommendation 3</u>: Carry out sensitivity analyses to investigate sensitivity of water level in this area when considering different lake levels, to better understand the risk of overtopping of the stopbanks in the area.
2. <u>Eastern Stopbank</u>
   a. <u>Recommendation 1</u>: Carry out maintenance work on the timber floodwall to ensure gaps are closed/covered between wooden planks.
   b. <u>Recommendation 2</u>: Confirm spillway levels through survey and if necessary consider lowering spillways back to original design level.
   c. <u>Recommendation 3</u>: Carry out a complete cross section survey in this area (to include the full extent as per the 2008 survey and compare the cross sections to establish channel trends.
   d. <u>Recommendation 3</u>: Carry out sensitivity analyses to investigate sensitivity of water level in this area when considering different lake levels, and bridge blockage to better understand the risk of overtopping of the stopbanks in the area.
3. <u>Western Stopbank</u>
   a. <u>Recommendation 1</u>: Clear fallen trees from stopbank structure.
   b. <u>Recommendation 2</u>: Ensure stopbank is clear of rabbit holes and any other defects which may affect its performance.
   c. <u>Recommendation 3</u>: Clear transects through the vegetation between the Western Stopbank and channel and undertake survey of these transects, and compare to the LIDAR data. Refer to Appendix I for suggested survey locations.
   d. <u>Recommendation 4</u>: Re-survey bed levels in the vicinity of cross sections 9, 10 and 11.
   e. <u>Recommendation 5</u>: Reassess the performance of the stopbank through hydraulic analysis through updating the existing 2017 model with the survey information from recommendation 5 and 6. Based on modelling results, consider the following options:

i Management Option 1: Topping up stopbank to bring to at least original design levels.

ii Management Option 2: Topping up stopbank to new design levels if higher than original.

4 Maniapoto's Bend – Maniapoto and Kiko Spillways and Bed Control Structure

a Recommendation 1: Carry out survey of the following areas:

i Maniapoto Spillway and area between Manaiapoto and Kiko Spillways. Complete ground topographical survey to confirm correct LiDAR levels in this area.

ii Bed Control structure.

iii Downstream of Bed Control Structure in the area of overflow, survey in this location will confirm both bed and bank levels

iv Erosion protection upstream and downstream of the Bed Control Structure.

b Recommendation 2: Undertake options modelling to determine the level to which the Maniapoto Spillway should be lowered (original design level or Kiko Spillway level).

c Recommendation 3: Lower the ManiapotoSpillway to a level based on recommendation 2.

d Recommendation 4: If survey results show that the left bank erosion protection near the Bed Control Structure and/or that the Bed Control Structure itself has been undermined, then reconstruct erosion protection in the area to design profile, using larger rock.

e Recommendation 5: Extend erosion protection at the left bank further downstream to relieve pressure on the left bank and reduce the risk of breakout to the lake at this location.

f Recommendation 6: If survey indicates a build-up of sediment and debris on the revetment, remove it. Confirm extent of upstream erosion protection.

g Recommendation 7: Consider extending upstream erosion protection further upstream to reduce risk of river directly entering the Kiko Spillway.

h Recommendation 8: In the 200 m upstream of the BedControl Structure, consider removing gravel from shoals in central area of river and placing to form a rudimentary stopbank along the left bank to discourage the river from overtopping and scouring the left bank with potential short cut to Kiko Spillway.

5 Quarry Closure Bank

a Recommendation 1: Clear vegetation on the stopbank immediately upstream and downstream to enable ground based topographical survey to confirm ground levels.

b Recommendation 2: If ground levels are different to those modelled to date, undertake further modelling to investigate the risk of the spillway not operating at its design event, and floodwaters outflanking the stopbank during the design events. Subject to model outcomes consider the following:

i Management Option 1: Lower the spillway crest to a new design level (subject to confirmation through modelling new design levels).

ii Management Option 2: Formalise the spill area upstream of the bank through constructing a spillway in this area designed to achieve the required spill rate at current river bed levels (subject to confirmation through modelling new design levels).

iii Management Option 3: Extend the upstream and downstream section of the Quarry Closure Bank to the original design levels to prevent outflanking in combination with Option 1. (subject to confirmation through modelling new design levels).

iv Management Option 4: Construct a Bed Control Structure (based on engineering design) to ensure water spills into the quarry at the correct level.

8 Applicability

This report has been prepared for the exclusive use of our client Waikato 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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