BEFORE THE INDEPENDENT HEARING COMMISSIONER APPOINTED BY THE WEST COAST REGIONAL COUNCIL

UNDER	the Resource Management Act 1991 (RMA)
IN THE MATTER	of an application for resource consent under section 88 of the RMA by the West Coast Regional Council for stopbanks in the Waiho River

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STATEMENT OF EVIDENCE OF DR. DAI THOMAS ON BEHALF OF THE WEST COAST REGIONAL COUNCIL (AS APPLICANT)

12 July 2023

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Introduction, qualifications and experience

1 My name is Dai Benjamin Thomas.

2 I am a Senior Hydraulic Engineer and Geomorphologist at Tetra Tech and have 25 years' experience in hydrology, hydraulic engineering, fluvial geomorphology and water resources management. I have been involved in numerous river engineering projects throughout the United States and Asia-Pacific Region including New Zealand. My primary area of expertise and interest is in integrating dual specialties of water resources (hydrology, hydraulics and sediment transport) and geomorphology to solve complex landscape, river and environmental problems, including river and bank stabilization, flood management, water allocation and mined lands remediation.

3 I hold a PhD in Earth Resources (Fluvial Geomorphology) from Colorado State University (2014), a Master of Science in Civil Engineering (Hydraulic Engineering) from Colorado State University (1999) and a Bachelor of Engineering in Civil Engineering from University of Canterbury (1996). I am a member of the American Society of Civil Engineers and am a registered Professional Engineer (Civil, Colorado).

4 I have been asked by the West Coast Regional Council (**Council**) to provide independent expert evidence on its application for resource consent to raise the stopbanks on the true right bank of the Waiho River. In 2021, I was living in New Zealand and working for Tetra Tech Coffey, a subsidiary company of Tetra Tech, Inc. which I worked for before and after working for Tetra Tech Coffey. I have been involved in the project since May 2021 when first approached to assist in elements of the design by Land River Sea Consulting and the Council. I visited the site in June 2021 with Matthew Gardner of Land River Sea Consulting as part of pre-design investigations and am familiar with the surrounding location and site constraints.

5 Although this evidence is prepared for a Council hearing, I have read the code of conduct for expert witnesses contained within the Environment Court Practice Note 2023 and agree to comply with it. Except where I state that I am relying on the specified evidence of another person, my evidence in this statement is within my area of expertise. I have not omitted to consider material facts known to me that might alter or detract from the opinions I express.

Scope of evidence

6 My evidence addresses the following matters:

(a) A description of the proposal, including in respect of the stopbank design based on the modelling information and level of service sought by the Council;

(b) An explanation as to the effectiveness of the proposed stopbank design.

7 In preparing this evidence, I have reviewed and relied on the following:

(a) Application for resource consent for the use of land to construct stopbanks in the Waiho River by the West Coast Regional Council dated 20 March 2022 (**Application**);

(b) The requests for further information and subsequent responses in relation to the Application;

(c) Franz Josef Stopbanks Preliminary Design Report, Land River Sea Consulting Ltd, October 2021;

(d) Memo to Land River Sea Consulting (cc Gary Williams, Waterscape and Brendan Russ, West Coast Regional Council) dated 10 November 2021 (attached to my evidence as **Appendix 1**);

(e) Presentation by Matthew Gardner and Gary Williams to West Coast Regional Council regarding the alignment of the proposed NZTA stopbank (unknown date);

(f) The evidence of Matthew Gardner; and

(g) The evidence of Ben Pasco.

Executive summary

~~8~~ The Application includes raising the height of the existing stopbanks referred to as the Church, Helipad, and Havill Wall Stopbanks (collectively, the **Existing Stopbanks**). A new stopbank will be constructed in the riverbed that extends from the north end of the Helipad Stopbank and ties into the Havill Wall Stopbank near the Scenic Circle Hotel (the **Proposed Stopbank**). Note that the NZTA Stopbank, which runs parallel with SH6 (see map attached), will not be altered as part of these proposed works.

9 The existing Church, Helipad and Havill Wall stopbanks and the Proposed Stopbanks would extend continuously from State Highway 6 Waiho River bridge to the downstream end of the Havill Wall Stopbank, a total length of about 2358 m.

10 The Existing Stopbanks will:

(a) be raised by approximately 2 m to an average height of about 6 m;

(b) have rock protection placed on the river side, extending from the top of the existing rock to the top of the (raised) stopbank, with a median (D_{50})[1] size of 1.4 m and thickness of 2 m;

(c) maintain the existing slope (batter) on the river side of approximately 2 horizontal to 1 vertical (2H:1V) and

(d) have a 3H:1V batter on the landward side (apart from some locations with a 2.3H:1V batter as described in paragraph [41]).

11 The toe down depth[2] of the rock along the Existing Stopbanks is unknown; the Council indicated that it is thought to be between 4 and 6 m. Mr Pasco's evidence recommends inspection prior to construction, which I concur with.

12 The Proposed Stopbank design has the following features:

(a) a 2H:1V batter on the river side and 3H:1V batter on the landward side;

(b) a crest width of 6 m;

(c) rock (riprap) protection on the river side with a median (D_{50}) size of 1.4 m and thickness of 2 m;

(d) the rock will extend from 4 m below the existing river bed to the crest of the stopbank; and

(e) the core of the stopbank will be constructed from sediment (gravel to boulder sized) material excavated from the river.

13 The hydraulic conditions used as the basis for the design of the stopbanks in the Application is the hydraulic model output of the Waiho River performed by Matthew Gardner. Mr Gardner's modelling is based

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1 D_{50} is a term used to describe the median particle size.

2 The toe down depth is the depth below the river bed that e rock will be placed at.

on the river's geometry in 2021 and based on predicted sediment aggradation trends over a 20-year period.

14 The specified rock size and toe down depth of the rock (riprap) protection will provide the necessary protection for the stopbanks at the design discharge (2,500 m^3/s, as per Mr Gardner's report and evidence).

15 Due to ongoing aggradation of the Waiho River (and future aggradation which cannot be accurately predicted), the existing and new stopbanks have a finite lifespan and the proposed design is being progressed as a short term (i.e. 20 years) measure while longer term options are considered.

16 The average raise of 2 m for the Existing Stopbanks was selected to optimise the quantity of available rock, minimize impact on private property, and because it will provide sufficient protection at the design discharge including with the predicted 20-year aggradation.

Description of the proposed works

17 The proposed works will require removal of vegetation and soil from the stopbank footprint, followed by placement of gravel for the core of the stopbank and large rock on the river side of the bank.

18 The works include raising of the Existing Stopbanks from the State Highway 6 Waiho River bridge to the north end of the Helipad Stopbank, a new section of stopbank (the Proposed Stopbank) that extends from the Helipad Stopbank to Havill Wall Stopbank and the raising of the Havill Wall Stopbank to downstream of the wastewater treatment ponds.

19 The average raise in stopbank height is 2 m for an overall bank height of approximately 6 m.

20 The completed bank will be passable by vehicle with a crest width of 6 m.

21 For the Proposed Stopbank, rock riprap will be placed along the riverside slope from the crest of the stopbank to about 4 m below the riverbed to account for potential scour. This will require excavation along the river side toe of the bank to place the rock. During construction, the toe excavation will be constrained to a length which can be in-filled if severe adverse weather conditions are expected to create flood conditions. After construction, the excavation will be in-filled

to the original riverbed elevation which will cover the rock at the toe of the stopbank.

Description of the stopbank design based on modelling

22 Both the Proposed and raised stopbanks have been designed to convey the design discharge of 2,500 m^3/s with an assumed 20 years of channel aggradation. The design discharge was computed and provided by Matthew Gardner (Gardner, 2014) and is reported as having greater than a 100-year recurrence interval. The modelling work undertaken to inform the design of the stopbanks is referred to in more detail in Mr Gardner's evidence.

23 The alignment of the Proposed Stopbank was selected by me [Dai Thomas (Tetra Tech Coffey)], Matthew Gardner (Land Sea River), Gary Williams (Waterscape), and Brendan Russ (West Coast Regional Council) and is based on a similar alignment presented in a meeting to Council by Matthew Gardner and Gary Williams in 2015. The alignment was confirmed with Mark Healy from WSP who consulted with Waka Kotahi.

24 The Proposed Stopbank's alignment was selected to use the existing (Church, Helipad and Havill Wall) stopbanks. The alignment of the Proposed Stopbank, which connects the Helipad and Havill Wall Stopbanks will provide a smooth flow expansion between the constricted river section along the Church and Helipad Stopbanks to the wider river section at the Havill Wall stopbank.

25 Based on the investigations conducted and information presented to the Council, the Proposed Stopbank's cross-section geometry (including toe down depth, side slopes, top width), rock size, and available rock volume was suggested by the Council. Tetra Tech was requested to verify the specified rock size and toe down depths to ensure they provide the necessary protection to the stopbank.

Rock size and toe down depth

26 To verify the specified rock size (also referred to as riprap), a rock sizing analysis was performed. To verify the toe down depth, a scour analysis was performed. These analyses were based on hydraulic model output (e.g., water-surface elevations, depths, and velocities) provided by

Matthew Gardner from hydraulic modelling of the Waiho River from upstream of State Highway 6 bridge to the Waiho Loop.

27 I have read the evidence of Mr Gardner on the modelling process undertaken, and agree that this reflects the work undertaken. The model results indicated that the hydraulic conditions (e.g. velocity, depth and shear stress) which effect the scour and rock stability, are the most severe at the design discharge (2,500 m^3/s), and therefore these conditions were used for the rock sizing and scour analysis. Representative water depth and velocity values were selected at 50 m intervals along the artificial scour section near the toe of the stopbank. The selected velocities typically represent the highest predicted velocities along the stopbank, and therefore are the most conservative.

28 Rock sizing analysis was performed based on the following riprap equations:[3]

(a) Wallingford (1980)

(b) California Highways (1970)

(c) U.S. Army Corps of Engineers (1994)

(d) Jansen et al (low and high turbulence) (1978)

(e) Isbash (fitted and loose) (1936)

(f) USACE (1994)

(g) Aust Roads (2013)

29 Comparison of the average riprap sizes from the calculations (excluding the USACE method) indicated a median size (D_{50}) requirement of about 1.2 m along the Church and Helipad Stopbanks, which was very similar to the specified rock size of 1.3 m. The USACE method was excluded because the results were determined to be unreasonably conservative. The hydraulic conditions along the Proposed Stopbank and the Havill Wall Stopbank are less severe than along the Church and Helipad stopbanks. The same specified rock size was recommended for the Proposed and Havill Wall stopbanks which provides conservative rock sizing for these stopbanks.

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3 These are internationally recognised and accepted equations for this type of analysis.

30 Scour calculations were performed based on the following three methods: N.Z. Railways, Maza and Echavaria, and Blench method.[4]

31 For the scour calculations, a median bed material size (D_{50}) of 163 mm was used, based on sediment measurements collected in June 2021 using the pebble count method (Wolman, 1954).

32 The total scour depth predicted by the methods was marginally less than the proposed 4 m toe down depth, and therefore given the conservative hydraulic conditions applied to the analyses, the Council's proposed toe down depth was determined to be appropriate. Since the channel is aggradational, the toe down depth will become more conservative over time.

33 The hydraulic model was then modified by Matthew Gardner to represent the predicted aggradation over the 20-year period as estimated by Gardner (2021) and run at the design discharge of 2,500 m^3/s. The predicted water-surface elevations with 20 years of aggradation are higher compared to 2021 conditions and were used to design the crest of the raised stopbanks.

Stopbank profile and cross-section geometry

34 Stopbank design profiles were developed for 3 scenarios which included raising the Existing Stopbanks by an average height of 2 m, 2.5 m and 3 m.

35 The stopbank design profiles were developed by selecting a profile with a relatively uniform crest slope to match the predicted aggradation conditions and water-surface elevations.

36 Cross-sections were developed at approximately 50 m intervals that show the bulk fill, toe down rock and facing rock.

37 The volume of rock was computed for three scenarios and the resulting volumes were compared with the available volume of rock specified by the Council. Planform mapping was developed for each scenario to show the horizontal extents (particularly the bulk fill on the landward side) of the stopbank.

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4 As above, these are internationally recognised and accepted equations for this type of analysis.

38 Following discussion with the Council, the 2 m raise was selected for the following reasons:

(a) The rock quantity for the 3 m raise exceeded the available rock and therefore was not considered.

(b) Both the 2 m and 2.5 m raise encroach onto private land and the church near the Church stopbank, with the 2m raise having less encroachment.

39 In my opinion, the 2m stopbank raise is the appropriate design to accommodate the design discharge in this scenario.

40 Council requested that the stopbank crest near the upstream end of the Church stopbank near SH6 Bridge be raised by about 1 m to: (1) prevent the bulk fill on the landward side encroaching on to private property and the church property, and (2) the 1 m raise ties into the elevation of the SH6 road bridge. As a result, there is no freeboard at the upstream end under the predicted aggradation conditions at the design discharge.

41 Bulk fill will be avoided being placed on private property where practicable. As a result, the batter in some locations will be about 2.3H:1V (or 23 degrees) compared to the design specification of 3H:1V. However, I do not expect this to cause any adverse effects or stability issues.

42 The average freeboard (elevation between the crest of the stopbank and the water-surface elevation) along the stopbank at the design discharge and with the 20-years of aggradation is 1.1 m. However, there are two areas with freeboard less than 0.5m, which occur near the up- and downstream ends of the stopbank (as described above).

43 The freeboard of the Proposed Stopbank near the centre of the Church Stopbank is about 4 m (at the time of design). This area has experienced significant aggradation. Initially, the aggradation is expected to be greater at the upstream end compared to downstream. Therefore, the larger freeboard will provide an additional factor of safety.

44 The freeboard at the completion of the works along the Proposed Stopbank is about 4 m (at the time of design). The channel bed is relatively low in this area. It is anticipated that the channel will quickly aggrade, and as a result, the freeboard will eventually be reduced to similar values as the Havill Wall Stopbank.

Section 42A report

45 I understand that the section 42A report prepared by Selene Kane dated 5 July 2023 includes a comment that:

> While the purpose of the proposed works subject to this application are to provide flood protection there is always a risk that protection structures can fail. Should the stopbanks fail due to overtopping or damage there is potential for flooding up to the extent which could occur should no protection works be in place.

46 I agree with the statement that there is "…always a risk that protection structures can fail". I disagree that there is "…potential for flooding up to the extent which could occur should no protection works be in place." Stopbanks typically fail either by:

(a) erosion due to flow (near parallel) along of the stopbank;

(b) by overtopping; or

(c) by piping (where the flow goes through or under the stopbank).

47 All three failure modes can result in erosion of the stopbank leading to flow on the landward side. However, overtopping of the stopbank and piping does not necessarily lead to full failure. Overtopping of the stopbanks without erosion would result in flooding effects only from the amount of water flowing over the top of the stopbanks (i.e. not the full flood flow), and therefore even if the stopbanks are overtopped they would continue to provide some protection from the full flow.

48 Similarly, flow through or under the stopbank may not lead to failure, and the flows would be less than if no stopbanks were in place. A full breach in the stopbank (by either failure mode) would result in flood flows on the landward side of the stopbank, however, the flooding impact would probably be less than if no stopbanks were in place.

Conclusions on the effectiveness of the stopbank design

49 The proposed works will significantly improve the resilience of flood protection structures for the true right of the river below State Highway 6 and therefore reduce the risk of flooding to Franz Josef and infrastructure as far downstream as the wastewater treatment plant.

50 The Proposed Stopbank design was developed to provide flood protection at the design discharge (flows in excess of the 100-year flood)

and based on predicted aggradation of the riverbed over a 20-year period.

51 It is important to recognize that the aggradation patterns will vary with each flood, and future aggradation cannot be accurately predicted. Significant effort has been made to evaluate historic deposition patterns in order to predict future aggradation.

52 Rock sizing and bank construction will be similar to the existing structures with additional height to accommodate the 20-year aggradation scenario.

53 The proposed rock size and rock toe down depth proposed by Council was verified by performing rock sizing calculations and scour depths analyses to international standards. The specified rock size and rock toe down depths will provide the necessary stopbank protection for the design discharge.

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Dr. Dai Thomas

12 July 2023

References

Austroads, 2013. Guide to Road Design Part 5B: Drainage – Open Channels, Culverts and Floodways. Edition 1.1.

Blench, T., 1969. Mobile-Bed Fluviology. Edmonton. University of Alberta.

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U.S. Army Corps of Engineers (USACE), 1994. Hydraulic Design of Flood Control Channels. Engineer Manual No. 1110-2-1601. Washington, D.C. 20314-1000

Wallingford, 1980. Taken from US Army research in Charlton, F G & Farraday, R V; "Hydraulic Factors in Bridge Design", Hydraulics Research Station,

Appendix 1: Memorandum to Land River Sea

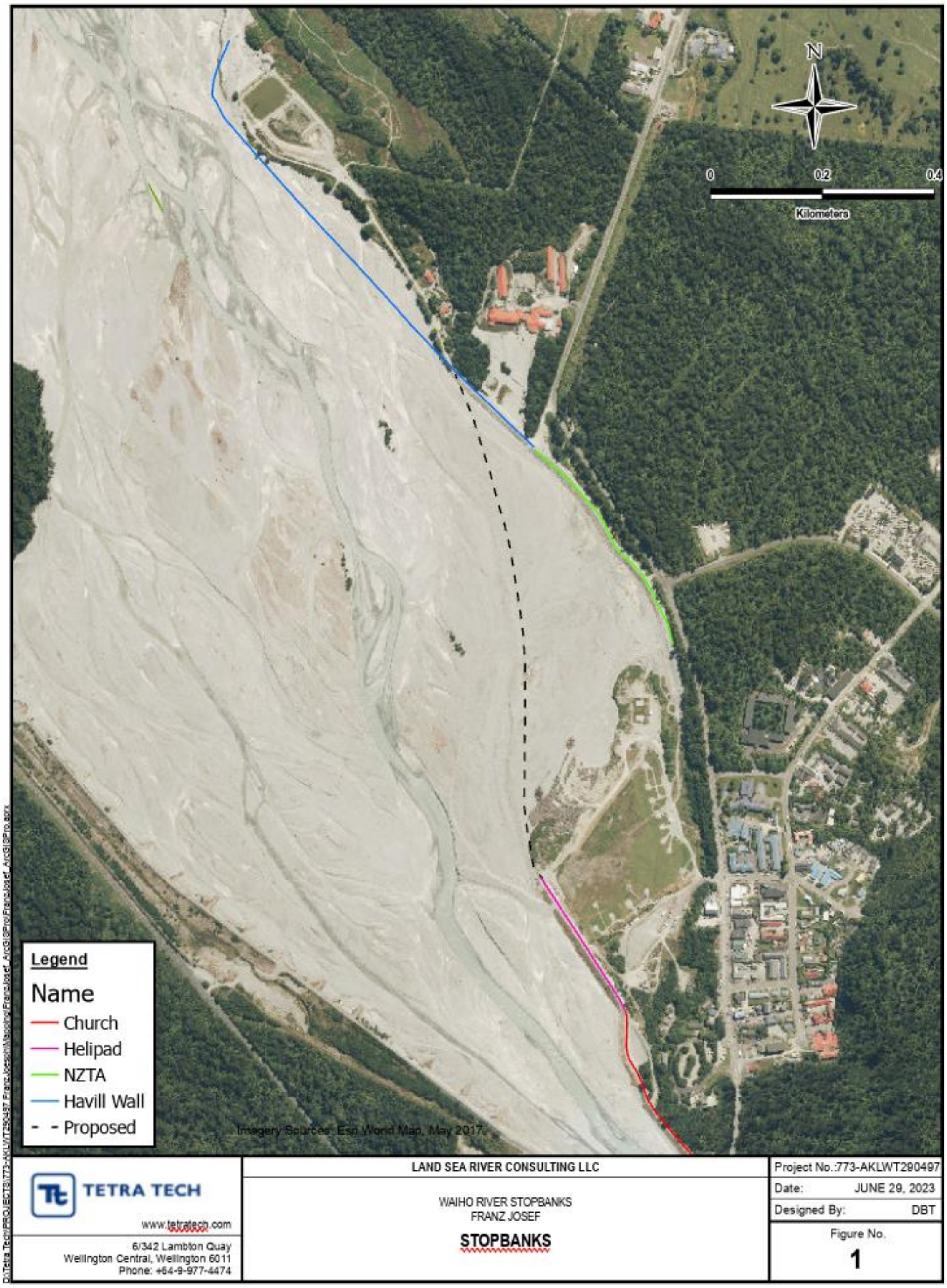

Figure 1 Location of the existing Church, Helipad and Havill Wall stopbanks and the Proposed stopbank.

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MEMORANDUM

Recipient name	Matthew Gardner	**Recipient company**	Land River Sea Consulting Ltd
Copied recipients	Gary Williams (Waterscape), Brendan Russ (WCRC)	**Memo date**	10 November 2021
Author	Dr. Dai Thomas		
Project reference	773-AKLWT290497 Franz Josef Stopbanks		
Memo subject	Franz Josef Stopbanks – Scour and riprap calculations, and stopbank design.		

Background

The West Coast Regional Council (WCRC) are proposing to raise the elevation of the Church, Helipad and Havill Wall stopbanks and construct the new NZTA stopbank that extends from the end of the Helipad stopbank to the Havill Wall stopbank (**Figure 1**). The WCRC requested a stopbank design with a 20-year design life. As discussed later in this memorandum, the stopbank was designed to convey the design discharge of 2,500 m^3/s and assuming 20-years of channel aggradation.

On Figure 1, the Church and Helipad stopbanks are shown as red circles and extend from chainage 0 to 750. The NZTA and Havill Wall stopbanks are shown as crimson squares and extend from chainage 0 to 1650

The existing stopbanks will be raised by an average of 2m and the new NZTA stopbank will be about 5m high. The Church, Helipad and Havill Wall stopbanks will be raised by placing bulkfill on top of, and on the landward side of the existing stopbanks, and extending the rock (riprap) up the face on the riverward side. The WCRC specified a batter of 3H:1V for the bulkfill on the landward side (**Figure 2**). On the riverward side, the batter of the raised section will match the existing batter with a maximum of 2H:1V (personal communication, Brendan Russ, WCRC, September 2021). The batter along the existing stopbanks varies from 1.9H:1V to 3.1H:1V. The bulkfill portion of the stopbank will be 6m wide at the crest.

The WCRC specified a rock gradation with a median weight (W_{50}) of 5.3 tonnes which is equivalent to a median size (D_{50}) of about 1.3m (**Table 1**). The rock gradation is the same as used for the raising of the Church and Helipad Stopbanks in 2016 (Gardner, 2016). The WCRC specified a toe down depth of 4m to account for scour along the toe (Figure 2).

Table 1 Approximate Quarried Rock Size Specification[1]

Percent Passing	Weight range (tonnes)	Approximate Size of b-axis (m)
50	4.5 – 6	1.3 – 1.4
35	3 – 4.5	1.1 – 1.3
1.5 - 3	1.5 - 3	0.9 – 1.1

[1]Provided by Land River Sea Consulting from 2016 Franz Josef stopbank design (Gardner, 2016)

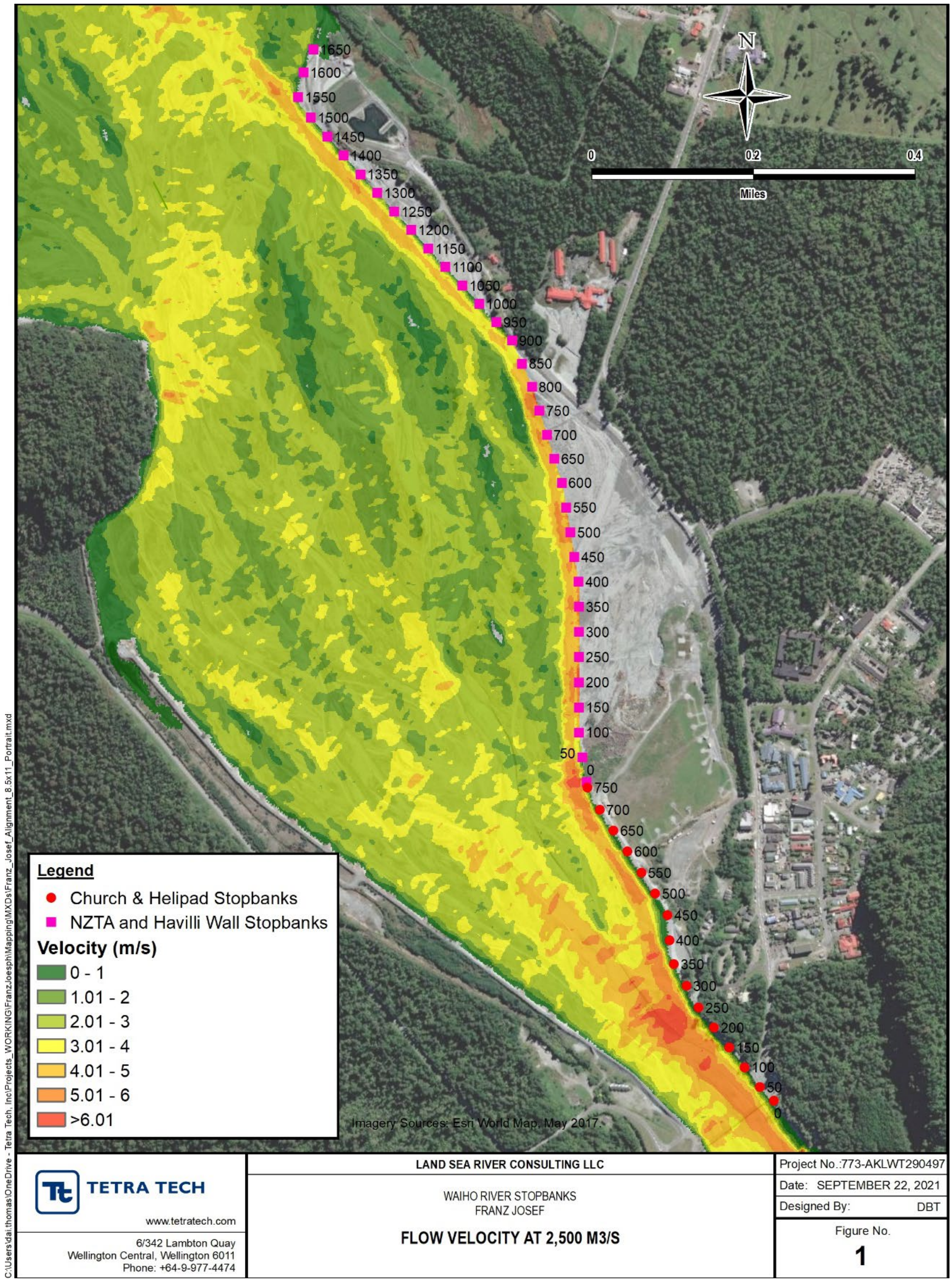

Figure 1 Stopbank alignment and predicted velocity at 2,500 m^3/s.

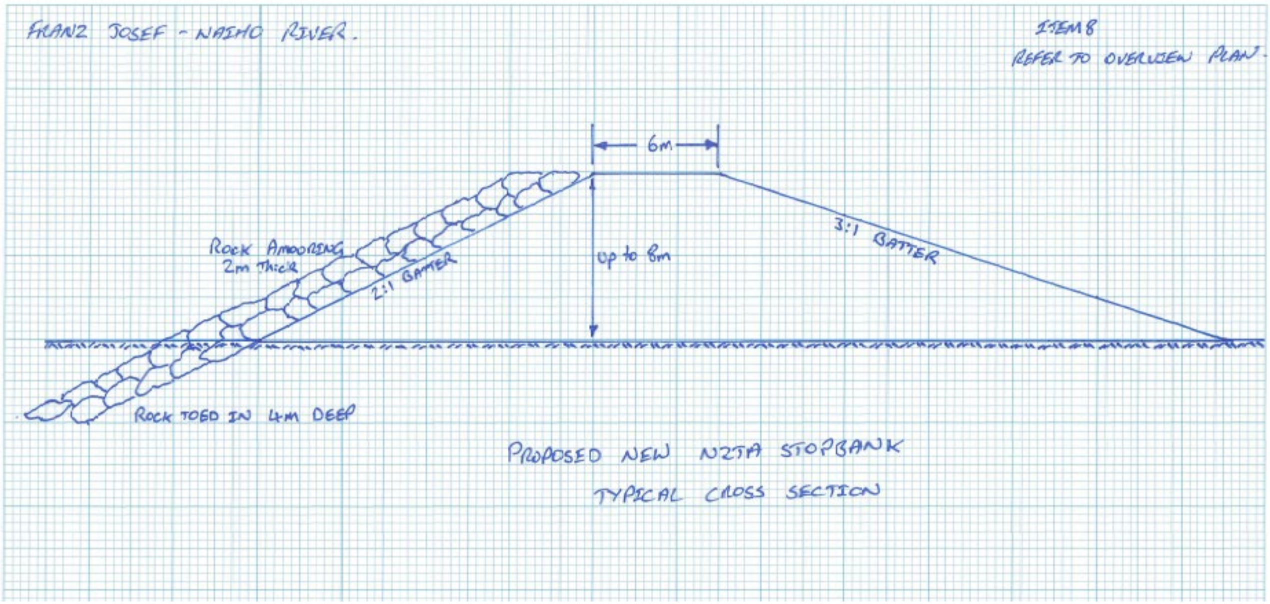

Figure 2. Typical cross-section geometry for the proposed stopbank upgrade (Figure provided by the WCRC)

The WCRC indicated that the rock along the existing stopbanks was toed down between 4 and 6m (personal communication, Brendan Russ of the WCRC and Mathew Garner from Land River Sea Consulting, September 2021). The toe down depth will be investigated by WCRC during the raising of the stopbanks scheduled for 2022.

The proposed alignment of the NZTA stopbank was selected by Matthew Gardner, Dai Thomas and Gary Williams in consultation with the WCRC. The alignment was also confirmed with Mark Healy from WSP who consulted with Waka Kotahi (N.Z. Transportation Authority) (personal communication, July 2021). Gardner (2021) performed an analysis of the historic aggradation rates along the Waiho River from the base of the Franz Josef Glacier to the Waiho Loop. The aggradation rates were determined based on repeat cross-section (CS) surveys at CS13 through 22 for the period from 1998-2021. The aggradation rate varied from 0.2 m at CS20 to 2.0 m at CS17,18 and 19, with an average rate of 1.5 m over the 20-year period. A representative aggradation rate of 0.2m/year was selected.

Tetra Tech-Coffey, under contract to Matthew Gardner from Land River Sea Consulting, performed a rock sizing analysis and scour calculations to verify the WCRC specifications. Gary Williams provided review of the scour and rock sizing analyses. Tetra Tech-Coffey also developed the design for the stopbank.

Hydraulic Analysis

Gardner (2021) performed hydraulic modelling of the Waiho River from upstream of the State Highway 6 bridge to downstream of the Waiho Loop. The model output was used to perform the rock sizing and scour analyses and to design the stopbank.

The hydraulic model geometry was based on the 2021 survey data and the proposed NZTA stopbank alignment. The model geometry was further modified to include a 2m deep by 30m wide channel along the toe of the right bank. A Manning's n-value of 0.05 was applied to the channel bed and the model was run for a series of flows from 500 to 3,500 m^3/s in 500 m^3/s increments. The design discharge is 2,500 m^3/s. A sensitivity analysis was performed by varying the Manning's n-value to 0.04 and 0.06 and comparing the predicted hydraulic conditions (depth, velocity and water-surface elevation) with the baseline conditions. As expected, the predicted velocities for the n=0.04 run are higher compared to the n=0.05 and 0.06 runs. The predicted velocities for the n=0.04 run were used in the scour and riprap calculations since they provide slightly more conservative results.

The hydraulic model was further modified to represent the aggradation over the 20-year period. The model was re-run at the design discharge and the predicted water-surface elevation was used, in part, to develop the top of stopbank profile.

Rock (Rock) Sizing

The riprap equations were developed for rivers and hydraulic structures that cover a range of hydraulic conditions (slope, velocity, depth). The rock sizing analysis was performed based on 8 riprap equations which include:

- Wallingford
- California Highways
- U.S. Army Corps of Engineers
- Jansen et al (low and high turbulence
- Isbash (fitted and loose)
- USACE (1994)
- Aust Roads (2013)

Gary Williams provided a spreadsheet that included the first five methods shown above. The spreadsheet was modified to include the USACE (1994) and the Aust Roads (2013) methods.

The riprap equations predict the rock sizing as either the D30 (30-percent of the rocks in the gradation are equal or smaller than the size), the D50 (median rock size), or the W33 (by weight). For comparison purposes, the D50 was calculated as 20% larger than the D30 size. For the W33, the D33 was computed as follows: D33 =W33 / $(0.85*\rho_S)^{0.33}$, where ρ_S = density of rock (2,650 kg/m^3). Note that the D33 size is approximately the same as the D30 size.

The parameters applied to the rock equations vary, but most include a combination of depth and/or velocity. The USACE (1994) method is more detailed and includes the ratio of curvature of the bend to the channel width, safety factor, stability factor and thickness coefficient.

The depth and velocity values were obtained from the hydraulic model developed by Gardner (2021) at the design discharge and the median bed material size (D_{50}) was 163mm based on the field sampling using the pebble count method (Wolman, 1954). Representative depth and velocity values were selected at 50m intervals in the artificial scour section near the base of the stopbank (Figure 1). The selected velocities typically represent the highest values along the stopbank, and therefore are the most conservative.

For the riprap equations, a velocity factor (usually about 1.5) is typically applied for velocities obtained from 1-D hydraulic model. Since the velocities were obtained from a 2-D model, and in the braid channel near the stopbanks, no velocity scaling factor was applied.

For the Wallingford, US. Army, Jansen and Isbash methods, the riprap was computed applying a batter of 1.5H:1V, and therefore, the results are conservative.

The predicted riprap sizes were evaluated for each equation and a representative size was selected for the Church and Helipad stopbanks, and the NZTA and Havilli Wall stopbanks. In general, the riprap sizes are reasonably consistent within each section with one or two outliers. The representative size was selected by excluding the outliers and choosing a size near the upper end of the predicted sizes (**Table 2** and **Table 3**). The representative values are summarized in **Table 4**.

Table 2 Comparison of the computed riprap sizes and representative sizes based for the Church (Distance 0 to 500m) and Helipad Reaches (Distance 500-750m).

Distance	VELOCITY	DEPTH	WAL/FORD (1.5H:1V)	CALIFORNIA HIGHWAYS		US ARMY (1.5H:1V)	JANSEN et al. (1.5H:1V)		ISBASH (1.5H:1V)		USACE (1994)	Aust. Roads
(m)	V (m/s)	D (m)	D50 (m)	W33 (kg)	D33 (m)	D30 (m)	D50 (m) Low Turb.	D50 (m) High Turb	D30 (m) Fitted	D30 (m) Loose	D30 (m)	D50 (m)
0	4.70	4.06	0.59	346	0.54	0.47	0.66	1.75	0.47	0.92	0.6	0.77
50	5.44	3.88	0.93	831	0.72	0.69	0.88	2.34	0.63	1.23	2.7	1.04
100	3.89	3.18	0.38	110	0.37	0.31	0.45	1.20	0.32	0.63	1.2	0.53
150	4.29	4.39	0.43	199	0.45	0.37	0.55	1.46	0.39	0.77	1.4	0.64
200	5.39	2.93	1.04	781	0.70	0.72	0.86	2.30	0.62	1.21	2.9	1.02
250	6.49	4.67	1.44	2,385	1.02	1.02	1.25	3.33	0.90	1.75	4.0	1.47
300	5.87	5.55	0.98	1,309	0.83	0.76	1.02	2.73	0.73	1.44	3.0	1.21
350	5.73	4.83	0.98	1,137	0.80	0.75	0.98	2.60	0.70	1.37	2.9	1.15
400	5.44	4.26	0.89	829	0.72	0.67	0.88	2.34	0.63	1.23	2.7	1.04
450	4.67	4.63	0.54	333	0.53	0.45	0.65	1.73	0.47	0.91	1.8	0.76
500	5.65	4.41	0.98	1,046	0.77	0.74	0.95	2.53	0.68	1.33	2.9	1.12
550	6.19	4.58	1.26	1,802	0.93	0.92	1.14	3.03	0.82	1.60	3.6	1.34
600	5.78	5.01	0.98	1,199	0.81	0.76	0.99	2.65	0.71	1.40	3.0	1.17
650	5.29	5.43	0.72	698	0.68	0.59	0.83	2.21	0.60	1.17	2.3	0.98
700	5.79	5.27	0.96	1,205	0.81	0.75	0.99	2.65	0.71	1.40	2.9	1.17
750	5.91	4.92	1.06	1,358	0.84	0.80	1.04	2.76	0.74	1.45	3.1	1.22
		Minimum	0.43	199	0.45	0.37	0.55	1.46	0.39	0.77	1.43	0.64
		Average	0.94	1,098	0.76	0.72	0.93	2.49	0.67	1.31	2.81	1.10
		Maximum	1.44	2,385	1.02	1.02	1.25	3.33	0.90	1.75	4.02	1.47
		Representative Values										
		D30 (m)	0.92		0.90	0.80	0.83	1.67	0.80	1.17	3.00	1.00
		D50 (m)	1.10		1.08	0.96	1.00	2.00	0.96	1.40	3.60	1.20

Table 3 Comparison of the computed riprap sizes and representative sizes based for the NZTA (Distance 0 to 900m) and the Havilli Wall Reaches (Distance 900 to 1650m).

Distance	VELOCITY	DEPTH	WAL/FORD (1.5H:1V)	CALIFORNIA HIGHWAYS		US ARMY (1.5H:1V)	JANSEN et al. (1.5H:1V)		ISBASH (1.5H:1V)		USACE (1994)	Aust. Roads
(m)	V (m/s)	D (m)	D50 (m)	W33 (kg)	D33 (m)	D30 (m)	D50 (m) Low Turb.	D50 (m) High Turb	D30 (m) Fitted	D30 (m) Loose	D30 (m)	D50 (m)
0	6.66	4.55	1.58	2,798	1.07	1.10	1.32	3.51	0.95	1.85	3.63	1.55
50	4.73	5.55	0.51	361	0.54	0.45	0.67	1.77	0.48	0.93	1.41	0.78
100	4.33	5.62	0.39	210	0.45	0.36	0.56	1.48	0.40	0.78	1.12	0.66
150	4.47	5.04	0.45	256	0.48	0.40	0.59	1.58	0.43	0.83	1.25	0.70
200	5.03	4.51	0.68	519	0.61	0.55	0.75	2.00	0.54	1.06	1.72	0.89
250	4.49	4.14	0.51	263	0.49	0.42	0.60	1.60	0.43	0.84	1.32	0.71
300	4.53	4.18	0.52	276	0.50	0.43	0.61	1.62	0.44	0.86	1.35	0.72
350	5.48	3.41	1.01	864	0.73	0.73	0.89	2.37	0.64	1.25	2.28	1.05
400	4.01	3.99	0.37	133	0.39	0.32	0.48	1.27	0.34	0.67	1.01	0.56
450	4.82	3.05	0.73	404	0.56	0.54	0.69	1.84	0.50	0.97	1.71	0.81
500	5.55	2.18	1.32	937	0.75	0.84	0.91	2.44	0.66	1.29	2.64	1.08
550	3.77	2.55	0.38	92	0.35	0.31	0.42	1.13	0.30	0.59	0.97	0.50
600	3.28	3.42	0.22	40	0.26	0.20	0.32	0.85	0.23	0.45	0.63	0.38
650	4.14	3.81	0.41	160	0.41	0.35	0.51	1.35	0.36	0.71	1.10	0.60
700	4.78	3.47	0.67	380	0.55	0.51	0.68	1.81	0.49	0.95	1.62	0.80
750	5.07	3.18	0.83	546	0.62	0.61	0.76	2.04	0.55	1.07	1.92	0.90
800	4.29	4.03	0.45	198	0.45	0.38	0.55	1.45	0.39	0.77	1.19	0.64
850	4.06	4.40	0.36	143	0.40	0.32	0.49	1.30	0.35	0.69	1.01	0.58
900	3.62	4.19	0.26	72	0.32	0.24	0.39	1.04	0.28	0.55	0.77	0.46
950	3.81	4.86	0.29	98	0.35	0.27	0.43	1.15	0.31	0.61	0.85	0.51
1000	3.86	5.38	0.28	106	0.36	0.27	0.44	1.18	0.32	0.62	0.85	0.52
1050	3.76	5.11	0.27	91	0.34	0.26	0.42	1.12	0.30	0.59	0.81	0.50
1100	3.87	5.60	0.28	108	0.36	0.27	0.44	1.19	0.32	0.62	0.85	0.52
1150	3.47	6.09	0.19	56	0.29	0.20	0.36	0.96	0.26	0.50	0.63	0.42
1200	3.65	4.79	0.25	76	0.32	0.24	0.40	1.06	0.28	0.56	0.76	0.47
1250	3.57	4.79	0.24	66	0.31	0.23	0.38	1.01	0.27	0.53	0.72	0.45
1300	3.71	4.97	0.26	83	0.33	0.25	0.41	1.09	0.29	0.57	0.78	0.48
1350	3.54	4.43	0.24	63	0.30	0.23	0.37	0.99	0.27	0.52	0.72	0.44
1400	3.98	4.33	0.35	128	0.38	0.31	0.47	1.26	0.34	0.66	0.97	0.56
1450	3.90	4.27	0.33	113	0.37	0.29	0.45	1.20	0.32	0.63	0.92	0.53
1500	4.28	4.54	0.42	196	0.44	0.36	0.54	1.45	0.39	0.76	1.15	0.64
1550	4.41	3.32	0.54	235	0.47	0.42	0.58	1.54	0.41	0.81	1.34	0.68
1600	2.86	4.32	0.13	17	0.20	0.13	0.24	0.65	0.17	0.34	0.64	0.29
1650	2.41	5.32	0.07	6	0.14	0.08	0.17	0.46	0.12	0.24	0.41	0.20
		Min	0.07	6	0.14	0.08	0.17	0.46	0.12	0.24	0.41	0.20
		Ave	0.46	297	0.44	0.38	0.54	1.43	0.39	0.76	1.21	0.63
		Max	1.58	2,798	1.07	1.10	1.32	3.51	0.95	1.85	3.63	1.55
		Representative Values										
		D30 (m)	0.58		0.70	0.70	0.75	1.67	0.50	1.10	2.30	0.92
		D50 (m)	0.70		0.84	0.84	0.90	2.00	0.60	1.32	2.76	1.10

Table 4 Comparison of the representative riprap sizes for each method with the average size and the specified riprap size.

Method	Church & Helipad Stopbanks		NZTA and Havilli Wall Stopbanks	
	D30 (m)	**D50 (m)**	**D30 (m)**	**D50 (m)**
WALLINGFORD	0.92	1.10	0.6	0.70
CALIFORNIA HIGHWAYS	0.90	1.08	0.7	0.84
US ARMY	0.80	0.96	0.7	0.84
JANSEN et al (Low Turbulence)	0.83	1.00	0.80	0.90
JANSEN et al (High Turbulence)	1.67	2.00	1.67	2.00
ISBASH (Fitted)	0.80	0.96	0.50	0.60
ISBASH (loose)	1.17	1.40	1.10	1.32
USACE	3.00	3.60	2.30	2.76
Aust Roads	1.00	1.20	0.9	1.10
Average (excl. USACE)	1.01	1.21	0.86	1.04
WCRC - Specified	1.30	1.33	1.30	1.33

Comparison of the representative sizes indicates that the USACE and the Jansen et al (High Turbulence) methods predict riprap sizes that are about three times and two times larger, respectively, than the other methods. The USACE method scales the velocity by a factor that is based on the channel width and radius of curvature. The equation was developed for conditions where the flow is around the outside of a meander bend, and not where flow is expanding such as along the stopbanks. The USACE method is sensitive to the velocity due scaling by the bend radius factor; as a result, it predicts very large rock. Removing the velocity scaling factor results in similar rock sizes predicted by the other equations.

Comparison of the average riprap sizes (excluding the USACE method) indicates the median size (D50) is about 1.2 m along the Church and Helipad stopbanks, which is very similar to the specified rock size of 1.3m. All the methods, except for the USACE, Jansen et al (High Turbulence) and Isbash (loose fitted) predict smaller rock sizes compared to the specified rock size. The Isbash (loose fitted) method predicts a rock size of 1.4m, which is close to the specified size of 1.3m. The Jansen et al method was sized for a batter of 1.5H:1V, which is steeper than the site conditions, and therefore, the result is conservative.

Along the NZTA and Havilli Wall stopbanks, the average predicted rock size is 1.0m. Similar to the upper section, all the methods, except for the USACE and Jansen et al (High Turbulence) predict smaller rock sizes compared to the specified rock size.

In summary, the rock sizes specified by the WCRC are larger than predicted by the riprap equations (except for the USACE and the Jansen et al. High Turbulence equations), and therefore, it is expected the specified rock will be stable up to the design discharge of 2,500 m^3/s.

Scour Analysis

The scour calculations were performed based on the following three methods:

- N.Z. Railways
- Maza and Echavaria
- Blench

Gary Williams provided a spreadsheet that included the scour equations. Similar to the riprap equations, the hydraulic values were obtained at 50m intervals near the stopbank from the existing conditions hydraulic model at the design discharge of 2,500 m^3/s (Gardner, 2021) (**Table 5**).

The following assumptions were made for the scour calculations:

- Since the hydraulic values were obtained from a 2-D model, a unit width (1 meter) was applied.
- The “rise” is the difference in depth between the low-water and flood conditions. A depth of 0.5m was assumed for the low-water conditions.
- The median (D_{50}) bed material size of 163 mm was applied based on the field measurement.

The scour formula predicts the total flow depth (D) which includes the scour depth (Ds). The predicted scour is shown for each method in the column labelled (Ds-D). The scour depths are separated into the two sections: (1) Helipad and Church stopbanks, and (2) proposed NZTA and Havill Wall Stopbanks.

For the Helipad and Church stopbank, the N.Z. Railways method predicts negative scour depths, which indicates that the predicted scour depths are less than the modelled conditions. It is important to note that the modelled conditions include a 2m deep channel along base of the stopbank. Therefore, the NZ Railways method predicts no additional scour. The NZR formula is based around asymmetry, and for the braid in a wide channel is likely to under-estimate the scour.

The Maza and Echavaria method predicts scour depths ranging from 1.5 to 2.4 with an average of 1.5m. The Blench method predicts scour depths ranging from 0.4 to 1.9m with and average of 1.2 m. Therefore, the predicted scour depths are less than the rock toe-down depth of 6m.

For proposed NZTA stopbanks (chainage 0 to 900m), the N.Z. Railways method predicts negative scour depths indicating that the predicted scour depths are less than the modelled conditions. The Maza and Echavaria method predicts scour depths ranging from -0.2 to 2.5 with an average of 0.8, and the Blench method predicts scour depths ranging from -0.1 to 1.9m to 2.2m with an average of 0.7 m.

For proposed Havill Wall stopbanks (chainage 900 to 1650m), the N.Z. Railways method predicts negative scour depths indicating that the predicted scour depths are less than the modelled conditions. The Maza and Echavaria method predicts scour depths ranging from 0.0 to 0.8 with an average of 0.3, and the Blench method predicts scour depths ranging from -0.3 to 0.7m with an average of 0.2 m.

In summary, the total scour depth (which includes the predicted scour depths and the 2m scour depth represented in the model) is less than the 4m toe down depth, and therefore, the WCRC toe down depth is appropriate. Since the channel is aggradational, the toe down depth will become more conservative over time.

Table 5 Summary of the scour calculations for the helipad and church stopbanks, and the proposed NZTA and Havill Wall stopbanks.

Chainage	FLOW	VELOCITY	WIDTH	AREA	MAX. DEPTH	MEAN DEPTH	RISE	MATERIAL	N.Z. RAILWAYS		MAZA & ECHAV.		BLENCH	
(m)	Q (m^3/s)	V (m/s)	W (m)	A (m^2)	D (m)	Dm (m)	R (m)	d50 (mm)	Ds	Ds-D	Ds	Ds-D	Ds	Ds-D
Helipad and Church Stopbanks														
0.0	19.1	4.7	1.0	4.1	4.1	4.1	3.6	0.2	1.8	-2.3	4.9	0.9	4.8	0.8
50.0	21.1	5.4	1.0	3.9	3.9	3.9	3.4	0.2	2.0	-1.9	5.3	1.4	5.2	1.3
100.0	12.3	3.9	1.0	3.2	3.2	3.2	2.7	0.2	1.4	-1.8	3.5	0.3	3.6	0.4
150.0	18.8	4.3	1.0	4.4	4.4	4.4	3.9	0.2	1.7	-2.6	4.9	0.5	4.8	0.4
200.0	15.8	5.4	1.0	2.9	2.9	2.9	2.4	0.2	1.7	-1.2	4.2	1.3	4.3	1.3
250.0	30.3	6.5	1.0	4.7	4.7	4.7	4.2	0.2	2.4	-2.2	7.1	2.4	6.6	1.9
300.0	32.6	5.9	1.0	5.6	5.6	5.6	5.1	0.2	2.4	-3.2	7.5	1.9	6.9	1.4
350.0	27.7	5.7	1.0	4.8	4.8	4.8	4.3	0.2	2.2	-2.6	6.6	1.7	6.2	1.4
400.0	23.2	5.4	1.0	4.3	4.3	4.3	3.8	0.2	2.1	-2.2	5.7	1.5	5.5	1.2
450.0	21.6	4.7	1.0	4.6	4.6	4.6	4.1	0.2	1.9	-2.7	5.4	0.8	5.3	0.6
500.0	24.9	5.7	1.0	4.4	4.4	4.4	3.9	0.2	2.1	-2.3	6.1	1.6	5.8	1.4
550.0	28.4	6.2	1.0	4.6	4.6	4.6	4.1	0.2	2.3	-2.3	6.7	2.1	6.3	1.7
600.0	29.0	5.8	1.0	5.0	5.0	5.0	4.5	0.2	2.3	-2.7	6.8	1.8	6.4	1.4
650.0	28.7	5.3	1.0	5.4	5.4	5.4	4.9	0.2	2.2	-3.2	6.8	1.3	6.4	0.9
700.0	30.5	5.8	1.0	5.3	5.3	5.3	4.8	0.2	2.3	-2.9	7.1	1.8	6.6	1.3
750.0	29.0	5.9	1.0	4.9	4.9	4.9	4.4	0.2	2.3	-2.6	6.8	1.9	6.4	1.5
Proposed NZTA and Havill Wall Stopbanks														
0.0	26.1	6.9	1.0	3.8	3.8	3.8	3.3	0.2	2.3	-1.4	6.3	2.5	6.0	2.2
50.0	26.3	4.7	1.0	5.5	5.5	5.5	5.0	0.2	2.0	-3.5	6.3	0.8	6.0	0.4
100.0	24.3	4.3	1.0	5.6	5.6	5.6	5.1	0.2	1.9	-3.7	5.9	0.3	5.7	0.1
150.0	22.5	4.5	1.0	5.0	5.0	5.0	4.5	0.2	1.9	-3.1	5.6	0.6	5.4	0.4
200.0	22.7	5.0	1.0	4.5	4.5	4.5	4.0	0.2	2.0	-2.5	5.6	1.1	5.4	0.9
250.0	18.6	4.5	1.0	4.1	4.1	4.1	3.6	0.2	1.8	-2.4	4.8	0.7	4.8	0.6
300.0	19.8	4.7	1.0	4.2	4.2	4.2	3.7	0.2	1.8	-2.3	5.1	0.9	5.0	0.8
350.0	18.1	5.3	1.0	3.4	3.4	3.4	2.9	0.2	1.8	-1.6	4.7	1.3	4.7	1.3
400.0	16.0	4.0	1.0	4.0	4.0	4.0	3.5	0.2	1.6	-2.4	4.3	0.3	4.3	0.3
450.0	14.7	4.8	1.0	3.0	3.0	3.0	2.5	0.2	1.6	-1.4	4.0	1.0	4.1	1.0
500.0	12.1	5.6	1.0	2.2	2.2	2.2	1.7	0.2	1.5	-0.6	3.4	1.3	3.6	1.4
550.0	10.2	4.0	1.0	2.5	2.5	2.5	2.0	0.2	1.3	-1.2	3.0	0.5	3.2	0.6
600.0	11.2	3.3	1.0	3.4	3.4	3.4	2.9	0.2	1.3	-2.1	3.2	-0.2	3.4	0.0
650.0	17.4	4.6	1.0	3.8	3.8	3.8	3.3	0.2	1.7	-2.1	4.6	0.8	4.5	0.7
700.0	16.7	4.8	1.0	3.5	3.5	3.5	3.0	0.2	1.7	-1.7	4.4	1.0	4.4	1.0
750.0	16.1	5.1	1.0	3.2	3.2	3.2	2.7	0.2	1.7	-1.4	4.3	1.1	4.3	1.1
800.0	18.7	4.6	1.0	4.0	4.0	4.0	3.5	0.2	1.8	-2.2	4.8	0.8	4.8	0.7
850.0	18.2	4.1	1.0	4.4	4.4	4.4	3.9	0.2	1.7	-2.7	4.7	0.3	4.7	0.3
900.0	15.0	3.6	1.0	4.2	4.2	4.2	3.7	0.2	1.5	-2.7	4.1	-0.1	4.1	-0.1
950.0	19.6	4.0	1.0	4.9	4.9	4.9	4.4	0.2	1.7	-3.1	5.0	0.2	4.9	0.1
1000.0	21.8	4.0	1.0	5.4	5.4	5.4	4.9	0.2	1.8	-3.6	5.5	0.1	5.3	-0.1
1050.0	21.2	4.2	1.0	5.1	5.1	5.1	4.6	0.2	1.8	-3.3	5.3	0.2	5.2	0.1
1100.0	24.3	4.3	1.0	5.6	5.6	5.6	5.1	0.2	1.9	-3.7	5.9	0.3	5.7	0.1
1150.0	24.9	4.1	1.0	6.1	6.1	6.1	5.6	0.2	1.9	-4.2	6.1	0.0	5.8	-0.3
1200.0	20.2	4.2	1.0	4.8	4.8	4.8	4.3	0.2	1.8	-3.0	5.1	0.3	5.0	0.2
1250.0	19.7	4.1	1.0	4.8	4.8	4.8	4.3	0.2	1.7	-3.1	5.0	0.2	4.9	0.1
1300.0	20.8	4.2	1.0	5.0	5.0	5.0	4.5	0.2	1.8	-3.2	5.3	0.3	5.1	0.2
1350.0	19.6	4.4	1.0	4.4	4.4	4.4	3.9	0.2	1.8	-2.6	5.0	0.6	4.9	0.5
1400.0	20.2	4.7	1.0	4.3	4.3	4.3	3.8	0.2	1.8	-2.5	5.1	0.8	5.0	0.7
1450.0	16.7	3.9	1.0	4.3	4.3	4.3	3.8	0.2	1.6	-2.7	4.4	0.1	4.4	0.1
1500.0	19.4	4.3	1.0	4.5	4.5	4.5	4.0	0.2	1.8	-2.8	5.0	0.4	4.9	0.4
1550.0	14.7	4.4	1.0	3.3	3.3	3.3	2.8	0.2	1.6	-1.7	4.0	0.7	4.1	0.7
1600.0	2.4	2.9	1.0	0.8	0.8	0.8	0.3	0.2	0.4	-0.5	1.0	0.1	1.2	0.4
1650.0	2.0	2.4	1.0	0.8	0.8	0.8	0.3	0.2	0.3	-0.5	0.8	0.0	1.1	0.2

Stopbank Design

To simplify the stopbank design, a continuous chainage was developed that extends from 0 to 2,358m. Note, that the chainage for the design is different than the chainage shown in Figure 1. Chainage 0 starts at the downstream end of the Havill Wall and chainage 2,358m is located at the upstream end of the Church stopbank.

The stopbank design was developed using the following method.

1. The hydraulic model was run for the 20-year aggradation conditions at the design discharge of 2,500 m^3/s.
2. Stopbank design profiles were developed for 3 scenarios which included raising the stopbank by 2, 2.5 and 3m.
3. The stopbank profiles were developed by selecting a profile with a minimum number of slope changes to match the predicted aggradation conditions and water-surface elevations.
4. Cross-sections were developed at approximately 50m intervals that included the bulkfill, toedown rock and facing rock.
5. The volume of rock was computed for each scenario using the end-area method and the resulting volumes were compared with the available rock. Planform mapping was developed to show the extents (particularly the bulkfill on the landward side) of the stopbank.
6. Following discussion with the WCRC, the 2m raise was selected for the following reasons:
    - The rock quantity for the 3m raise exceeded the available rock and therefore was not considered.
    - Both the 2 and 2.5m raise encroach onto private land and the church near the Church stopbank, with the 2m raise having less encroachment.
    - The WCRC are also planning to raise the stopbanks along the left side of the Waiho River. It is likely that the stopbanks along the left side will be raised to similar elevations as the right side. The amount of funding (and the quantity of available bulkfill and rock) is unknown, and correspondingly, it is not known how high the left stopbanks can be raised. The WCRC decided not to overbuild the right side without confidence knowing that the left side will be built to match, and therefore, the WCRC selected a 2m raise.

The proposed design profile is shown in **Figure 3** and **Appendix A.1**. The design planform and representative cross-sections are shown in **Appendix A.2 to A.4**.

Following are comments on the design.

- The freeboard varies along the length of stopbanks.
    - The average freeboard along the stopbank is 1.1m at the design discharge. There are two areas with freeboard less than 0.5m, which occur near the up- and downstream ends of the stopbank.
    - The WCRC requested that the stopbank crest near the upstream end (from about chainage 2,207 to 2,358m) be raised by about 1m to prevent the bulkfill on the landward side encroaching on to private property and the church property. The private property is located between Chainage 1,908 and 1,959m and the church between 2,257 and 2,308m (Appendix A.1). As a result, there is no freeboard at the upstream end under the predicted aggradation conditions at the design discharge..
    - Bulkfill will not be placed on the private property. As a result, the batter in this area will be about 2.3H:1V (or 23 degrees) compared to the design specification of 3H:1V. The angle of repose for cobbles is about 40 degrees and therefore, the 2.3H:1V batter is expected to be stable.
    - The freeboard near Chainage 2109m is about 4m. This area has experienced significant aggradation. Initially, the aggradation is expected to be greater at the upstream end compared to downstream. Therefore, the larger freeboard will provide an additional factor of safety.
    - The freeboard along the proposed NZTA stopbank is about 4m. The channel bed is relatively low in this area. It is anticipated that the channel will quickly aggrade, and as a result, the freeboard will be reduced to similar values as the Havill Wall.
- The calculated bulkfill and rock quantities are shown in **Table 6.**
- The bulkfill material will be sourced from the river bed.
- Typically, a filter layer is constructed between the bulkfill and the rock. The filter layer is intended to create a layer for placement of the rock and to prevent fines washing out of the bulkfill layer. Because

the bulkfill contains a large range of particle sizes including sands and gravels, it was determined that a filter layer was not necessary.

- Construction of the NZTA stopbank will create a low elevation area that will be bounded by the NZTA stopbank to the west and stopbank along state highway 6 to the east. At present, there is a stormwater drain that flows into this area that has the potential to cause ponding. The WCRC has indicated that the drain will be moved to prevent flows into the low elevation area.
- It is important to recognize that the aggradation patterns will vary with each flood, and future aggradation cannot be accurately predicted. Significant effort has been made to evaluate historic deposition patterns in order to predict future aggradation.
- The 20-year design life is intended to be an interim measure while the long-term plan is put in place. This assumes that the aggradation rates and locations are similar to the past 20 years, however there is no freeboard at the up- and downstream ends of the stopbank at the design discharge.

Table 6 Calculated rock and bulkfill quantities for the 2m stopbank raise.

Item	Quantity
Bulkfill (m^3)	208,420
Rock (tonne)	89,678
Rock[1] (m^3)	45,989

[1]Rock volume was calculated at 1.95 tonnes/m^3 as specified by WCRC.

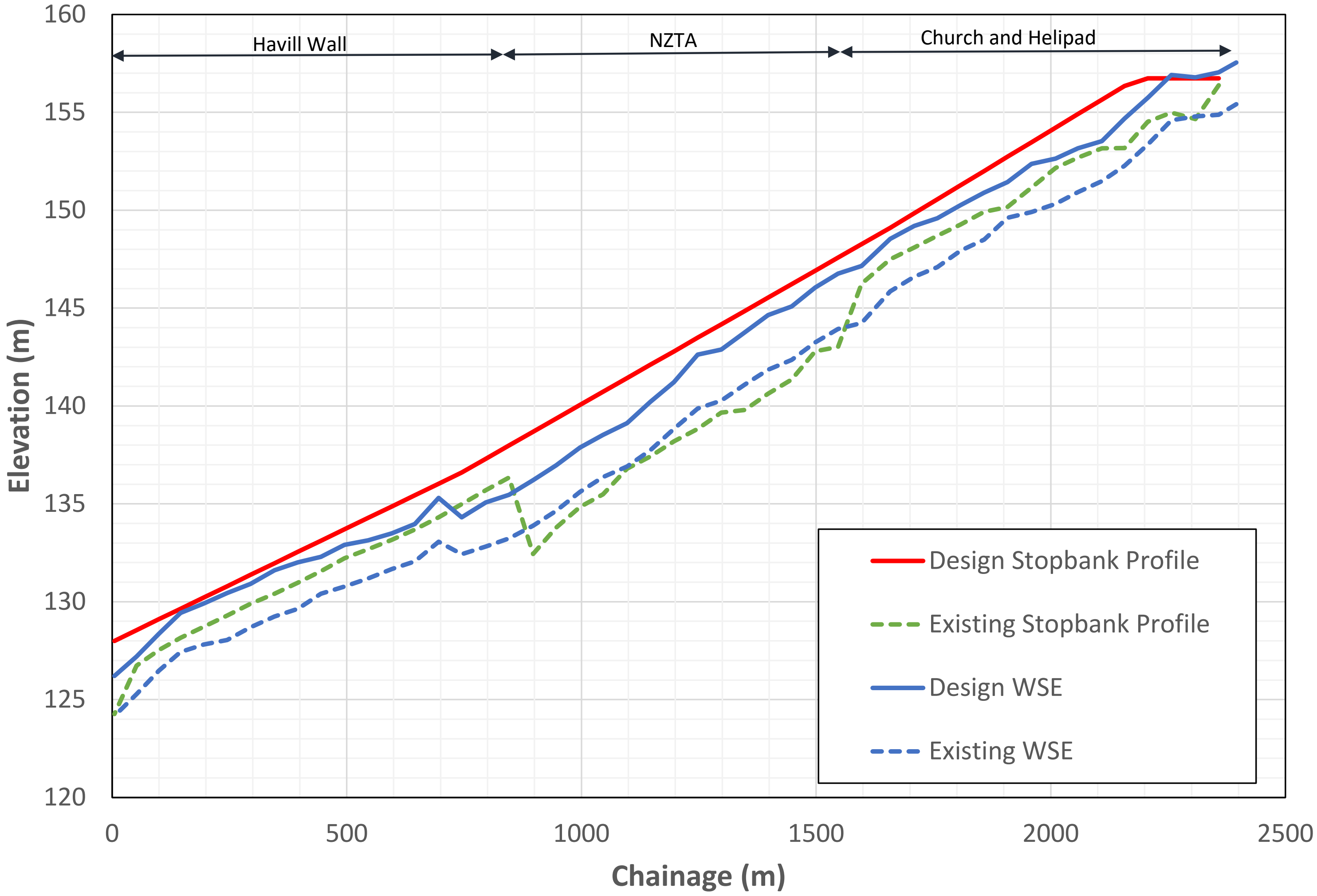

Figure 3 Comparison of existing and proposed stopbank profile for the 2m raise, and the existing and design water-surface profiles.

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