Hawke's Bay Regional Council

Wairoa Flood Mitigation Project

Developed Concept Design Report

10 December 2025

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WSP
Napier
Opus House
6 Ossian Street
Private Bag 6019
Napier 4110, New Zealand
+64 6 833 5100
wsp.com/nz

REV	DATE	DETAILS
1	6/08/2025	Final
2	21/11/2025	Updated floodway geometry
3	27/11/2025	Updated design report following CFD modelling results
4	10/12/2025	Minor updates to remove the fill site references

	NAME	DATE	SIGNATURE
Prepared by:	Robert Loomes Jean-Louis Dubois	10/12/2025	
Reviewed by:	Darren Rodd	10/12/2025	
Approved by:	Andrew Sowersby	10/12/2025	

This report ('Report') has been prepared by WSP exclusively for Hawke's Bay Regional Council ('Client') in relation to the Wairoa Flood Alleviation Design project ('Purpose') and in accordance with the Proposal letter dated 9 July 2025 and accepted on 18 July 2025. The findings in this Report are based on and are subject to the assumptions specified in the Report and the Proposal. WSP accepts no liability whatsoever for any reliance on or use of this Report, in whole or in part, for any use or purpose other than the Purpose or any use or reliance on the Report by any third party.

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1 INTRODUCTION

1.1 BACKGROUND

Hawke's Bay Regional Council (HBRC) engaged WSP to undertake the developed concept design for the proposed Wairoa floodway (alignment 1C+) and the proposed townside stopbank. Earlier work considered other floodway alignments (1C and 1D) but these were found to be not feasible because of land access limitations. The exit of floodway alignment 1C+ is in between the exits of options 1C and 1D.

Revision 2 of this report outlines updates since Revision 1 was delivered on 6 August 2025.

Previous investigations, optioneering and concept design reports are listed in Table 1.

Table 1: Previous Wairoa flood mitigation reports

Report Title	Issue date	Author Organisation	Comments
Wairoa Flood Scheme – Developed Concept Design Report	7 February 2025	WSP	Covered floodway options 1C and 1D, and Ski Club stopbank option 1.
Wairoa Floodway Options – Geotechnical Investigation & Material Suitability Report	5 February 2025	WSP	Presented the results of geotechnical investigations
Wairoa Flood Mitigation – Options Modelling Report	22 January 2025	WSP	A summary of the flood modelling results for options 1C and 1D
Wairoa Flood Alleviation – Developed Concept Design Cost Estimate	3 February 2025	WT Infrastructure	Based on the concept design in WSP's 7 February 2025 report
Memo – Wairoa Showgrounds Riverbank	7 June 2025	HBRC	Describes riverbank erosion at the Showgrounds
Wairoa Flood Mitigation Project – Developed Concept Design Report	6 August 2025	WSP	For alignment 1C+
Geotechnical Factual Report	10 October 2025	WSP	For alignment 1C+
Memo – Freeboard Optimisation	21 October 2025	WSP	Following investigation, freeboard can be reduced on most stopbank sections
Memo – Stopbank Configuration on Ruataniwha Road	20 October 2025	WSP	Proceed with option 3
Memo – Stopbank Geometry Optimisation	6 November 2025	WSP	Following investigation, crest width and side batters can be optimised

1.2 PURPOSE

The purpose of this report is to summarise the developed concept design. The report will be used:

- As a summary of the current concept design for a NIFF meeting to be held on 1 December 2025.
- As an input to the resource consent application which will be under an Order in Council (OiC) process. HBRC hopes to obtain the consent by the end of January 2026.
- To inform the ECI contractor.

This approach, while enabling the consenting and tendering processes to progress in parallel with the detailed design process, does come with associated risks (see Risk section). Some key considerations are also discussed in Section 11.4.

1.3 CONCEPT DESIGN DEVELOPMENT – REVISION 2

Following the delivery of Revision 1 of the developed concept design, some of the key assumptions were no longer valid and some important modifications to the floodway geometry had to be made.

The key updates since Revision 1 was issued are:

- The floodway outlet: Revision 2 is a narrower alignment due to land access constraints, which led to vertical concrete walls and a concrete floor/reinforced floor. These replace the revision 1 earth stopbanks.
- Ruataniwha Road – the geometry has been further refined following workshops with HBRC and WDC.
- Waihirere Road – flood gates are used so the horizontal alignment now stays the same.
- Shorter stopbanks are used beside Railway Road due to land access constraints.

Workshops undertaken during Revision 2 are listed in Table 2.

Table 2: Summary of concept design workshops contributing to Revision 2

Date	Topic	Key Outcome
18 August 2025	Floodway alignment discussion	Updated stopbank alignment over and beside the Clark property
29 August 2025	Road geometric design requirements	Design requirements for Ruataniwha Road and Waihirere Road
3 September 2025	Stopbank 2 alignment	Agreement on stopbank alignment beside the urupa and on top of the terrace (on and near Clark property)
5 September 2025	Floodway drainage workshop	Discuss the floodway geology and options for draining the floodway
9 September 2025	Railway Rd flooding	Further hydraulic modelling to be undertaken with the PNGL 'embankment' removed
24 September 2025	Stopbank alignments	Review and stopbank alignments and adjust as discussed
26 September 2025	Road alignments and cut/fill volumes	Review draft road alignments (Ruataniwha and Waihirere) and discuss cut/fill volumes including likely shortfall of fill

1 October 2025	Stormwater updates	Review the draft stormwater modelling results on the townside.
3 October 2025	Inlet and scour protection	Review the floodway inlet stabilisation options. Review the scour protection options, depending on the velocities in different locations.
3 October 2025	Narrow floodway outlet	Review the findings of the narrow floodway option (105m width and 0.5m cut below Waihirere Road)
16 October 2025	Railway Road stopbank length	Review the modelling results of a shorter stopbank on Railway Road (stops within the Clark property paddock)
17 October 2025	MSE wall facing options (Churchill Avenue)	Discuss the options for the MSE wall facing on the land-side of the stopbank on Churchill Avenue.
23 October 2025	Road alignments	Discuss options for Ruataniwha Road alignment. Discuss Waihirere Road alignment high level options.
29 October 2025	Cost estimate assumptions	Discuss assumptions and contingency approach for the cost estimate (with WT Infrastructure/HBRC)
29 October 2025	Flood gates	Discuss Waihirere Road alignment and the use of flood gates to avoid horizontal realignment of the road

1.4 CONCEPT DESIGN DEVELOPMENT – REVISION 1

The developed concept design was produced in a very short timeframe (two months). This meant that some assumptions had to be made so that the design could be developed while site investigations were ongoing, hydraulic model calibration and peer review were still being closed out, and landowner negotiations were ongoing.

It was agreed with HBRC that regular workshops would be held to discuss key items as the design progressed. The workshops were generally attended by representatives of HBRC, Wairoa District Council (WDC) and WSP. Table 3 summarises the workshops.

Table 3: Summary of concept design workshops contributing to Revision 1

Date	Topic	Key Outcome
2 July 2025	Floodway vertical alignment	Agreement on vertical alignment to be used in the concept design. This to be a 'worst case' so that the consent would cover the required volumes. (This consists of a deep cut as well as high stopbanks)
3 July 2025	Design Philosophy Statement	List of floodway and townside stopbank key assumptions agreed with HBRC
17 July 2025	Railway Road stopbank alignment	Stopbank to be formed in the road reserve which means that the road will be on top of the stopbank.

Date	Topic	Key Outcome
17 July 2025	Townside flood protection - floodwall concept design	The flood protection through Alexandra Park to be an earthen stopbank.
24 July 2025	Churchill Avenue MSE wall	Agreed that the Churchill Avenue side of the stopbank will have a steep batter. In concept design this will be a MSE wall, but may change during detailed design.
29 July 2025	Liquefaction risk and erosion risk	HBRC staff to share information with HBRC Governance Board about liquefaction risks. Erosion risk discussed. Proposed concept design approach discussed and agreed.
1 August 2025	Presentation of developed concept design drawings	HBRC informed on the concept design. Some items discussed have been incorporated into this report.

1.5 DRAWINGS

The developed concept design drawings are in Appendix A.

2 PROPOSED DESIGN - SUMMARY

2.1 DESIGN OBJECTIVES

- The design objective is to mitigate Wairoa from being flooded for any event up to a 100-year ARI event (no climate change).
- The objective is for physical works to start by early 2026.

2.2 DESIGN STANDARDS AND GUIDELINES

The design of the floodwalls will generally comply with the requirements set out in the following design documents, where applicable:

Table 4: List of standards and guidelines adopted for the project.

General	— Wairoa District Council Engineering Code of Practice (April 2022). — NZTA Bridge Manual (SP/M/022). — AS/NZS 1170 Structural Design Series (2002). — CIRIA Report 731 - The International Levee Handbook (2013).
Floodway Design	— CIRIA Report 116 - Design of Reinforced Grass Waterways. — Main Roads Australia – Floodway Design Guide. — FHWA (2006), "Hydraulic Design of Energy Dissipators for Culverts and Channels", Hydraulic Engineering Circular No. 14, Third Edition, Department of Transportation – Federal Highway Administration, Reference FHWA-NHI-06-086, July 2006." — FHWA (2005), "Design of Roadside Channels with Flexible Linings", Hydraulic Engineering Circular No. 15, Third Edition, Department of Transportation – Federal Highway Administration, Reference FHWA-NHI-05-114, September 2005."
Geotechnical Design	— MBIE/NZGS Earthquake Geotechnical Engineering Practice Modules 1-6 (2021) — HBRC Consultant Brief and Geotechnical Acceptance Criteria (draft 2025) — Bay of Plenty Regional Council Guideline 2021/1.2 – Stopbank Design and Construction Guidelines. — NZGS Slope Stability Guidance Units 1-4 (2024-2025).
Roading and Pavement	— Guide to Road Design Part 3: Geometric Design (2021). — Wairoa District Council's Engineering Code of Practice (April 2022) — New Zealand Standard (NZS) 4404:2010
Stormwater Design	— Wairoa District Council Engineering Code of Practice (April 2022). — Hawkes's Bay Waterway Guidelines - Stormwater Management (May 2009)

2.3 GENERAL SITE DESCRIPTION

Wairoa is located approximately 120 km north of Napier in the North Island. Wairoa's township is split into two main parts, each located on either side of the Wairoa River, approximately 1 km upstream of the river's mouth (Figure 1).

Wairoa's population is approximately 8,500 people.

In 2022, Cyclone Gabrielle caused major flooding in and around Wairoa. HBRC has received funding from the Crown to design and implement flood mitigation to protect the city for a 100-year ARI (no climate change).

The 100-year ARI predicted flood depth is shown in Figure 2.

Eagle Technology, Land Information New Zealand, GEBCO, Community maps contributors

Figure 1: Wairoa is located on both sides of the Wairoa River

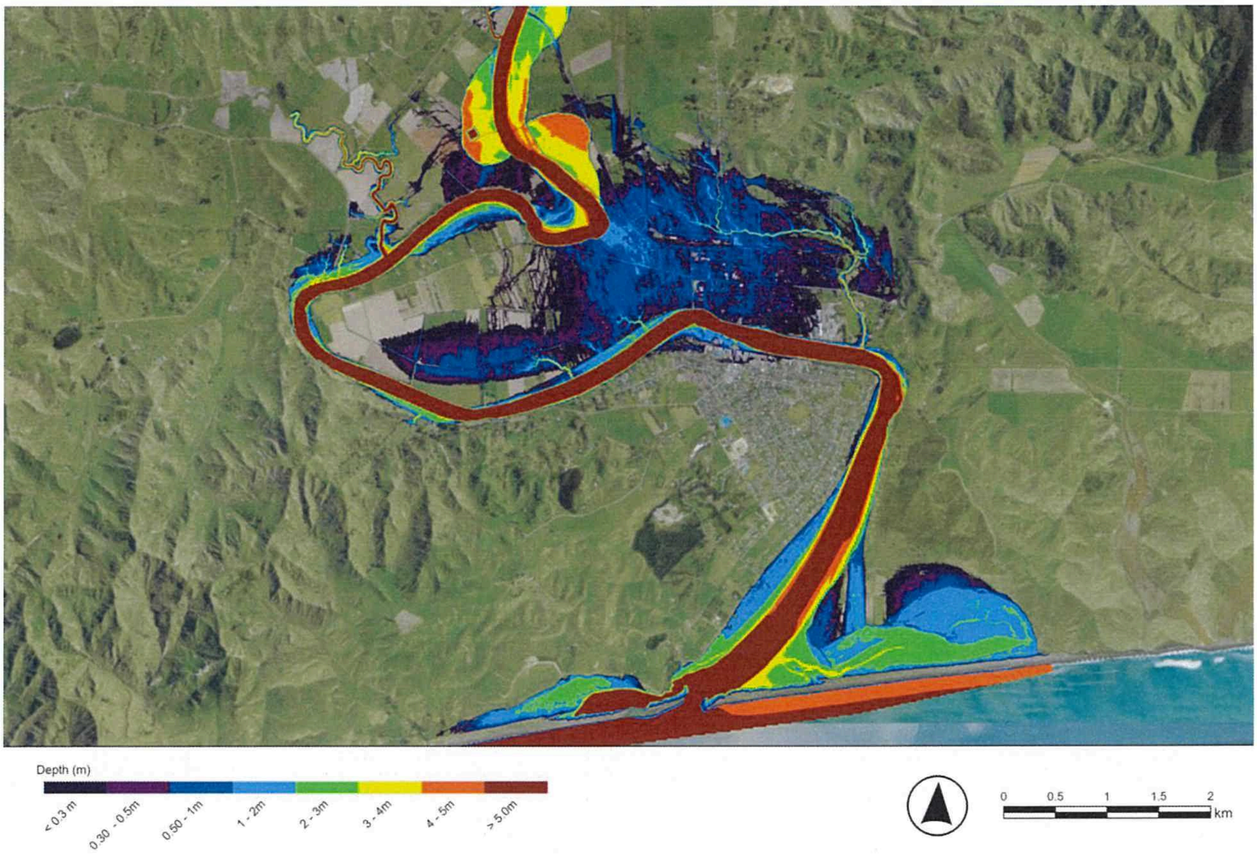

Figure 2: Predicted water depth for a 100-year ARI event with no mitigation

2.4 OVERVIEW OF CONCEPT DESIGN

The proposed alignment for the floodway is shown in Figure 3. The proposed design consists of:

- A floodway channel, over 1,000 m long, connecting two sections of the Wairoa River. The floodway comprises:
  - A grassed floodway channel (200-250 m width, over 800 m long)
  - A narrower floodway outlet (105 m width, over 200 m long) requiring a concrete structure (vertical walls and partial concrete floor).
- The floodway 'activates' at approximately a 30-year ARI flood event.
- The functioning floodway is designed to limit flooding, primarily to the North Clyde area, up to and including a 100-year ARI flood event.
- The floodway will be lined with grass and constructed with site-won materials (yielded from the floodway excavations, with the exception of the floodway outlet as detailed above and any other localised scour protection as required).
- Stopbanks are also required on the true right of the Wairoa River from Mitchell Road to the bridge (referred to as the 'townside stopbank'). These will also be constructed primarily with site-won materials.

The predicted 100-year ARI flood depth post-implementation of the project is shown in Figure 4 below.

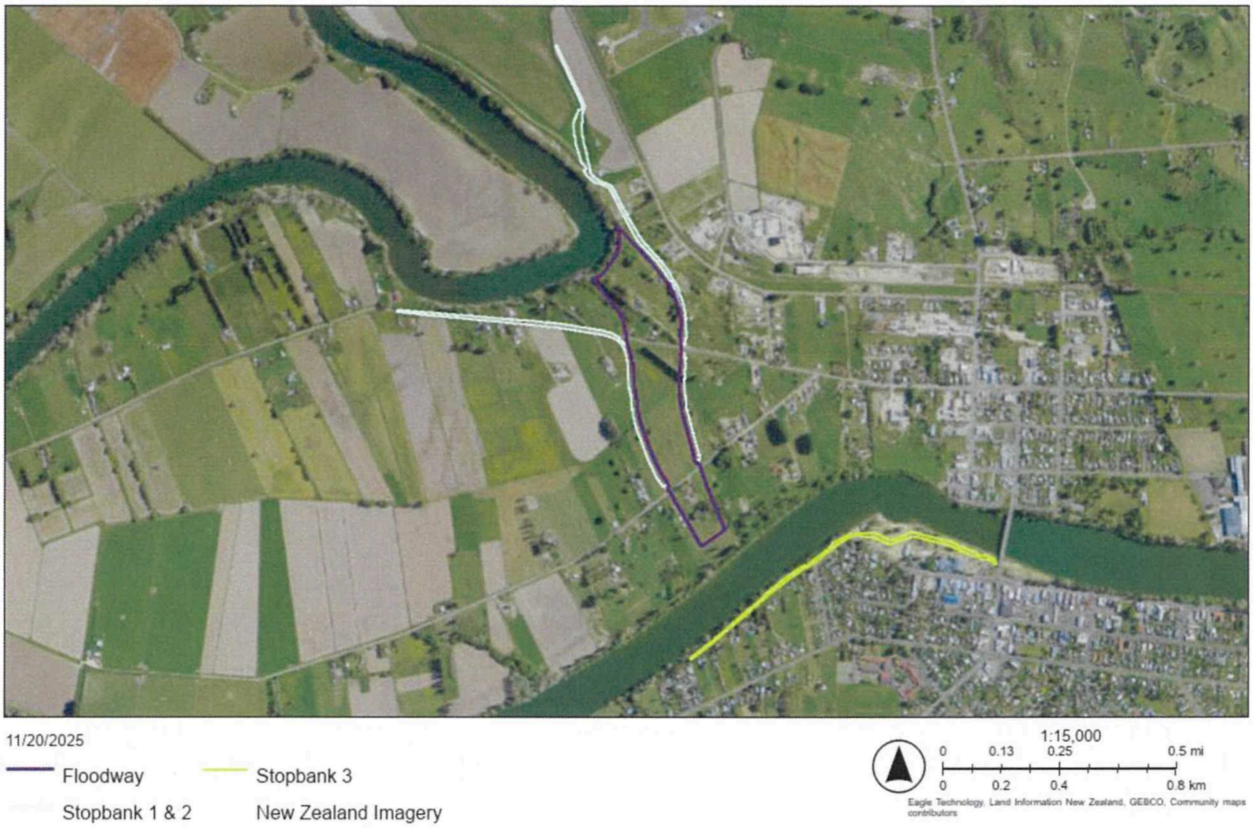

Figure 3: Proposed works. The 1C+ floodway is outlined in purple. Stopbanks 1 and 2 are light blue and Stopbank 3 (townside stopbank) is yellow.

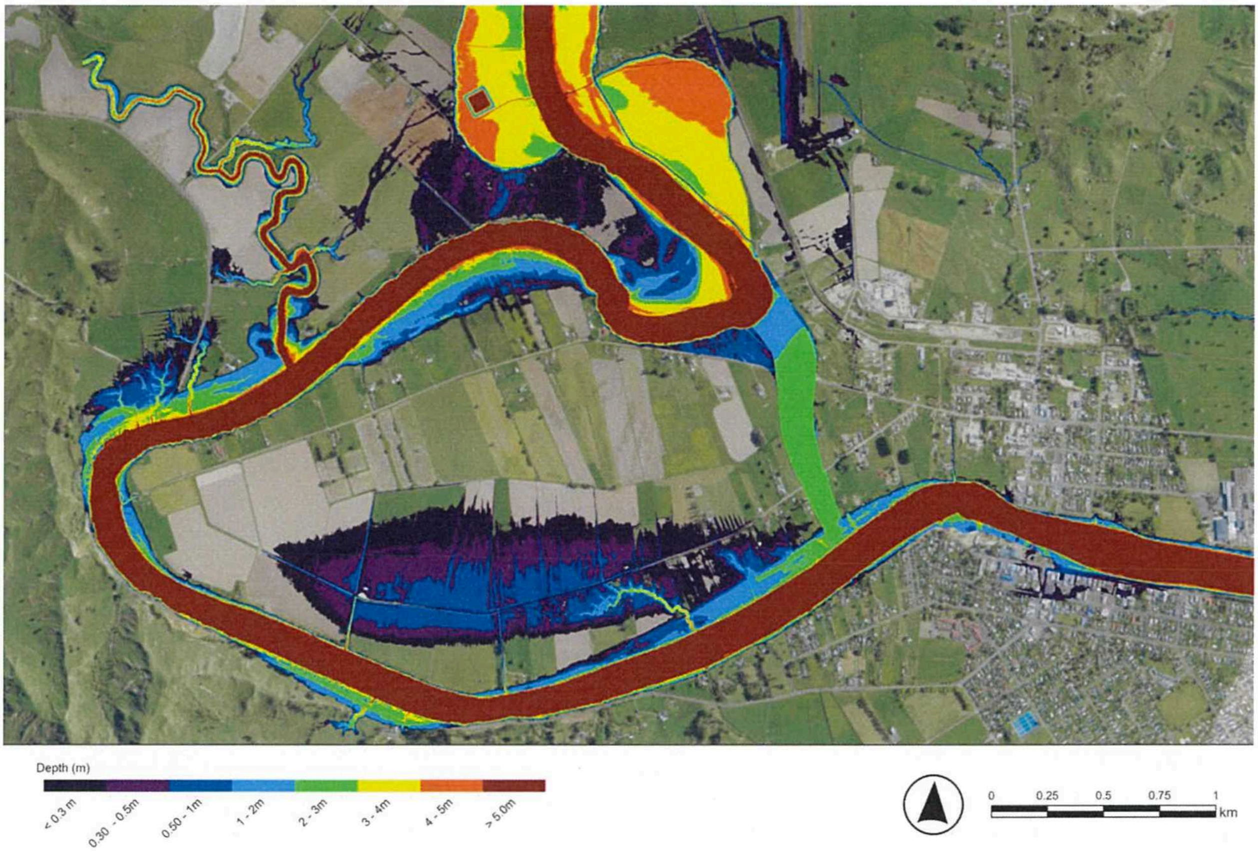

Figure 4: Predicted water depth for a 100-year ARI event post-floodway implementation.

2.5 DESCRIPTION OF EXISTING STRUCTURES

The floodway crosses existing assets as described below:

- Ruataniwha Road: it is proposed to maintain the general road alignment. The road will need to ramp over and down the stopbanks and will go through the floodway. The road will be flooded when the floodway is being used. Road barriers will prevent driving through the floodway during flood events.
- Ruataniwha Road: some of the road will be re-built on top of the stopbank (western end of the road). The alignment and the level of service will not be affected. The road elevation will simply be raised.
- Waihirere Road: it is proposed to maintain the general road alignment. The road will be lowered so it is approximately 1.5 m lower where it passes through the floodway. The road will be flooded when the floodway is being used. The flood gates will be closed during such an event which will block the road.
- Other services as described in Section 7.1 and 12.1.

The townside stopbank will affect the following structures:

- The skate park and the flying fox in Alexandra Park: these will need to be removed for the construction of the stopbank.
- Boat ramp access to the river: There are two existing boat ramps to the river. The existing concrete boat ramp will be abandoned, and a new concrete structure will be designed, replacing the previous two ramps.
- Ski club building: this building will need to be relocated.

Engagement with WDC and other stakeholders is ongoing regarding the required changes within Alexandra Park.

2.6 CLIMATE CHANGE

The design has no provision for climate change as instructed by HBRC.

2.7 DESIGN LIFE

The design life of the proposed design is as detailed below:

- Stopbanks: 100 years. Note that this excludes seismic performance (specifically liquefaction response), which shows damage to structures occurring on more frequent events (refer Section 3.6 below).
- Pavement: 25 years. This needs further consideration as discussed in the Pavement section.
- Scour protection: estimated between 25 to 30 years or longer depending on the products procured.
- Water and stormwater: 50 years.

2.8 BORROW MATERIALS

The design objective is to reuse material cut from within the floodway to build the stopbanks, to reduce the overall project costs. A geological model was prepared based on previous site investigations.

The project aims to reuse silt cut from the floodway to build the stopbanks. Quarry-sourced materials will be limited as much as possible, as it would be cost-prohibitive to source good-quality rocks, for example. Concept design volumes are detailed in Section 4.

2.9 HAWKE'S BAY REGIONAL COUNCIL REQUIREMENTS

The key requirements from HBRC are listed below:

- Stopbanks are of Importance level 2 (IL2).
- Stopbank crest width: 4m along the floodway.
- Stopbank batter slope: 3H:1V.
- Freeboard: as shown in Figure 5. (These figures are from the Freeboard Optimisation memo dated 21 October 2025. HBRC instruction was to adopt these figures.)
- Setback from waterways: 6m, measured from the crest of the riverbank to the toe of the floodwall. In some places, this minimum setback may not be feasible.
- Minimum setback from cadastral boundaries: 1 m, measured from the boundary to the toe of the floodwall.
- Floodway vertical alignment: the cut at the downstream end of the floodway is 2 m, which is approximately the level of the river terrace. The cut reduces to approximately 1.5 m cut where the floodway crosses Waihirere Road. From there, the cut reduces gradually to similar levels as for the Revision 1 design.

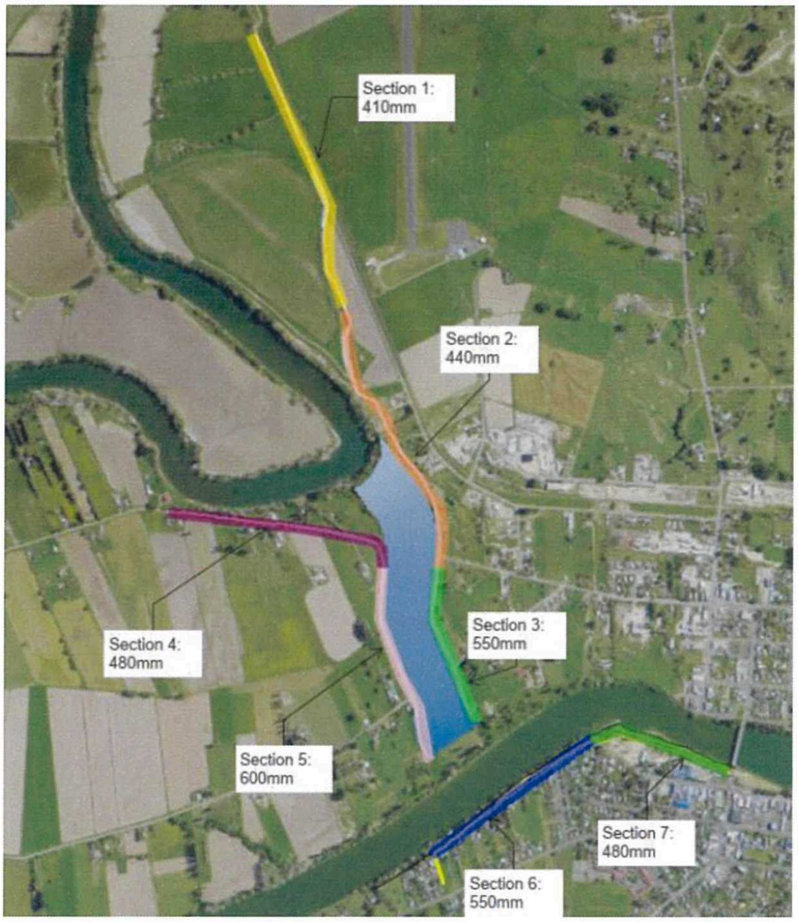

Figure 5: Stopbank freeboard heights

3 GEOTECHNICAL DESIGN

3.1 DESIGN CRITERIA

3.1.1 SERVICEABILITY

The geotechnical and structural design aims to incorporate:

- Long-term stability of stopbanks under operational and flood conditions.
- Acceptable total and differential settlements to maintain crest levels and freeboard.
- Control of seepage and piping through and beneath embankments.
- Constructability and maintainability of earthworks and drainage systems.

Performance thresholds are aligned with HBRC expectations.

3.1.2 SEISMIC DESIGN PHILOSOPHY AND APPROACH

The seismic design approach is to be based on a performance-based, Serviceability and Ultimate Limit State approach as outlined in NZS1170. All components of the project are assumed to fall under Importance Level 2 (IL2), which correlates to the return periods in Table 5. 'Intermediate States' have also been considered between the SLS and ULS state to assess the potential for a seismic step change at the site – intermediate states have no specific performance requirements for IL2 structures in accordance with NZS1170 and B1/VM1.

Table 5: Seismic response parameters for the site.

Importance Level 2 (IL2)

Limit State	Return Period (RP) - Years	Annual Exceedance Probability (AEP)	Peak Ground Acceleration (PGA)	Earthquake Magnitude (M)
SLS	1/25	4%	0.12	6.3
ILS	1/50	2%	0.18	6.6
ILS2	1/100	1%	0.28	6.8
ULS	1/500	0.2%	0.65	7.5

3.1.3 SEEPAGE, SLOPE STABILITY & FOUNDATION STRENGTH

Stopbanks will be designed to control seepage and prevent the potential for piping and boiling to occur through and around the stopbanks. Seepage shall be designed to the 100-year ARI event and achieve a critical hydraulic gradient ($I_{max)}$ limit of 0.5.

Slope stability of all structures shall meet or exceed the target Factor of Safety values outlined in the HBRC Geotechnical Acceptance Criteria for all cases, including prevailing conditions, rapid drawdown, ULS seismic, and developed seepage conditions.

Foundation soils must provide sufficient bearing capacity to support the structures placed upon them – a range of conditions should be considered i.e. static, saturated, and liquefied strengths.

3.1.4 LIQUEFACTION

Liquefaction potential for the site will be assessed using CPT, and laboratory testing data gathered across the site areas. This assessment will include the following performance metrics that may affect the integrity of the stopbanks and other structures:

- Post-seismic settlements.
- Lateral spreading.

Acceptability criteria for liquefaction response are typically dependent on the nature of the structure, consequences of structure damage, and cost of repair vs cost of mitigation. HBRC Geotechnical Acceptance Criteria suggests 200 mm vertical settlement, and/or 750 mm of lateral spread be considered 'damage' for the ULS case. If this magnitude of displacement is anticipated, then any SLS 'damage' should be negligible.

Should the liquefaction potential across the site area be variable, liquefaction 'zones' of varying severity may be created that will inform subsequent parts of the project.

3.2 PUBLISHED GEOLOGY

The published geological maps (GNS Map) indicate the site is underlain with Holocene River Deposits which comprise primarily of silts and sands, and clays (Figure 6). Late Pliocene sandstone and siltstone (Mangaheia Group) form the hills surrounding the site and are likely to extend beneath the site at significant depths (at least 30 m depth).

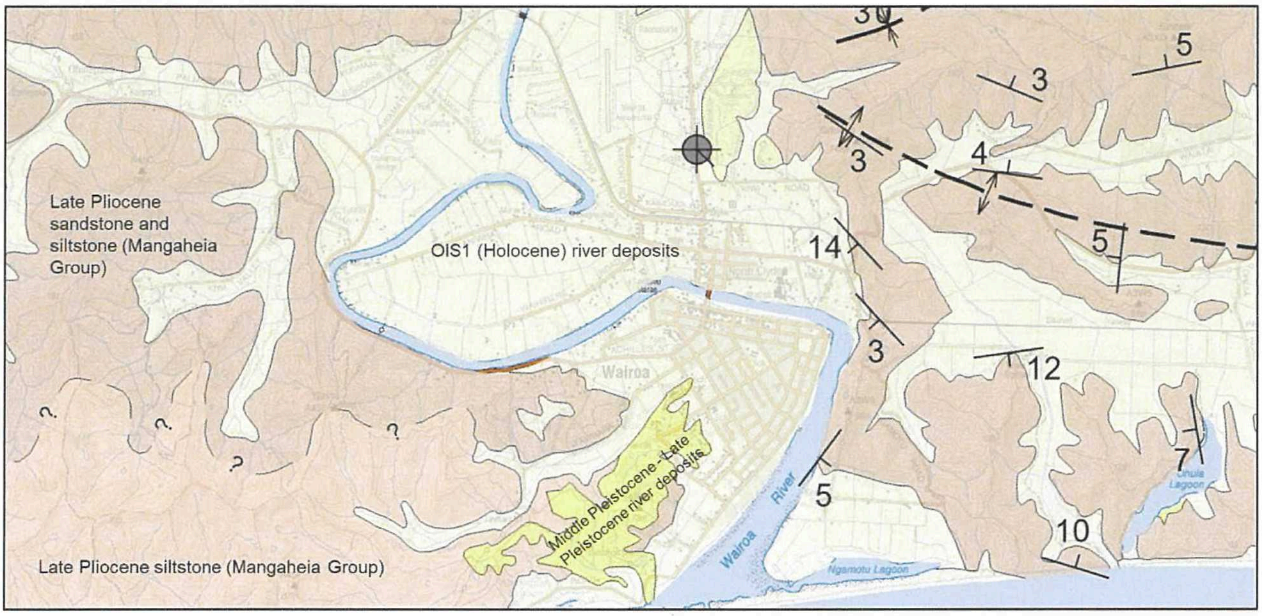

Figure 6: Geological map covering the project area. Source: GNS Geological Map; 1:250k.

3.3 SITE INVESTIGATIONS

A large number of investigations were undertaken across previous optioneering of potential floodway alignments. These can be found in the report *Geotechnical Investigation & Material Suitability Report, WSP, February 2025.*

A subsequent, ongoing, geotechnical investigation for the 1C+ alignment (floodway and townside) was conducted during July 2025. The Geotechnical Factual Report was delivered to HBRC on 10 October 2025.

3.4 METHODS OF ANALYSIS AND DESIGN

The following methods will be adopted:

- Liquefaction: Boulanger and Idriss (2014) method using CLIQ software.
  - Lateral displacements: Empirical method as per Zhang (2008) and Newmark Sliding Block (Bray and Macedo or similar).
- Slope Stability: Limit Equilibrium Method.
- Seepage: Finite Element Model.
- MSE wall (green facing) failure conditions: calculations as per Module 6 and B1/VM4.

3.5 PRELIMINARY GEOTECHNICAL ASSESSMENT

3.5.1 GROUND CONDITIONS

The site comprises two generalised areas, (1) Floodway and (2) Townside, separated by Wairoa River.

Each area predominantly comprises:

- Firm to stiff silts, with some clay and some sand to sandy. The silts were found to be typically low plasticity with limited cohesion properties.
- Sand layers, usually pumiceous, water bearing, and typically less than 1m thick.
- Soils at greater depth (>5m) was found to vary between the Floodway and Townside areas, with Floodway featuring more a consistent clayey SILT, and Townside featuring a dense layers course gravel.

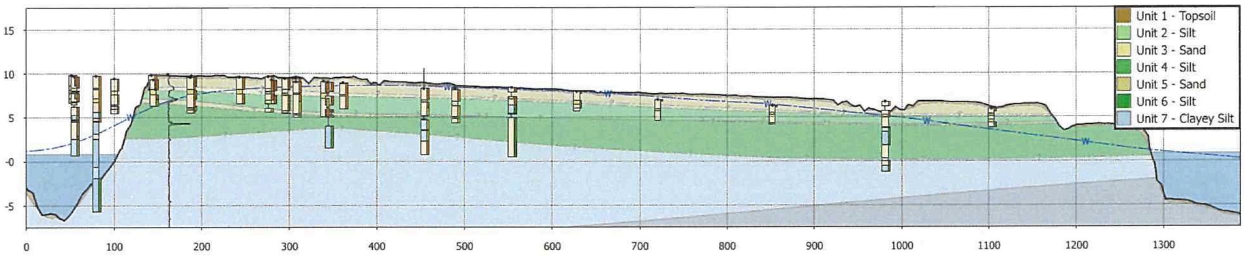

Figure 7: Long-section ground model through the floodway, showing the shallow silts, interlayered sands, and silt/clay at depth. A full section is included in Appendix E.

Groundwater has been measured directly at many points, and continues to be periodically recorded via several piezometers installed to monitor seasonal groundwater fluctuations across the site. The depth to groundwater was found to typically vary between 1-4m below ground level, being deeper on the townside compared to the floodway. The groundwater table is indicated to draw down towards the river level within closer proximity to the riverbanks. This groundwater gradient varies based on the general height of the riverbanks (with the riverbank being significantly higher on the northernmost edge of the floodway).

3.5.2 *STOPBANK FILL (BORROW MATERIAL)*

Near-surface, low plasticity silts across the site are proposed to form the stopbanks for the project. The stopbank materials are expected to be yielded entirely from the excavations required to form the floodway channel. The workability of the soils is indicated to be sensitive to moisture so careful construction planning will be required. Particular care with spread layer methodology during construction will likely be required in order to achieve optimal moisture levels for compaction.

3.5.3 *LIQUEFACTION SUMMARY*

Based on the results of our preliminary analysis, the liquefaction response across the site is indicated to be low during an SLS earthquake (1/25 year), with potential for settlement typically around 50-75mm. Any SLS damage to stopbanks is expected to be minor with insignificant levels of crest deformation that will not compromise their ability to withstand flooding. No lateral spreading is expected during an SLS earthquake.

The analysis shows a significant step change and much larger liquefaction response is occurring from around the 1/50 year event. Following this level of earthquake, practically all potential liquefaction effects are occurring across the site. At this level of earthquake, vertical settlements in the order of 100-200mm are indicated to occur across the majority of the site, likely resulting in moderate levels of crest deformations (but can still likely be relied on to withstand flooding following evaluation). However, lateral spreading is indicated to be severe for large portions of the stopbanks within 100-150m of the riverbank edge, with damage increasing closer to the riverbank crest. The magnitude of spreading damage could result in large sections of stopbank being significantly deformed and not able to withstand any significant flood event until physical repairs could be made. Stopbanks running parallel to the river comprise a significant portion of the proposed stopbanks that are at risk for the project. Any proposed scour protection assets close to the riverbanks will also consider the risk of spreading deformation. Note that any potential liquefaction is indicated to occur from only moderate levels of shaking (1/50 year). The liquefaction severity is not expected to increase significantly at higher level earthquakes, including the design ULS 1/500 year event.

It is understood that some seismic damage can be accepted in the design, and that HBRC are willing to accept and undertake stopbank repairs on a reactionary basis following a significant seismic event, on the basis that a major, localised earthwork is highly unlikely to coincide with a major flood event.

A summary of maximum ground motions is presented in Table 6 below. Further lateral spreading analysis will be presented at detailed design to more accurately gauge the potential for damage.

Table 6: Summary of maximum ground responses indicates following earthquakes following 1/50 year events and larger.

DEFORMATION TYPE	FLOODWAY/TOWNSIDE	SIGNIFICANT 'DAMAGE' TO STOPBANKS THRESHOLD
MAX. VERTICAL SETTLEMENTS	100-200 mm	≥ 200 mm
MAX. LATERAL SPREAD (within 50-100m of river edge)	1-4 m	≥ 750 mm

3.5.4 STABILITY AND SEEPAGE

Slope stability modelling has been undertaken on a range of representative stopbank heights (up to 3.5m height) and representative ground conditions across the Floodway and Townside areas. The model results suggest that at the design dimensions, the proposed stopbanks meet all target Factor of Safety (FoS) values for all design cases (e.g. static, rapid-drawdown, 100-year flood events and seismic). Lowest bound results are summarised in Table 7 below.

Table 7: Summary of stopbank stability Factor of Safety results (lowest indicative values presented).

Analysis Case	Target FoS	Floodway Stopbanks	Townside Stopbanks
Static	1.5	2.24	2.21
Rapid Drawdown	1.2	1.74	1.56
ULS Earthquake*	1.2	1.28	1.27
End of Construction	1.3	1.77	1.73

**Seismic coefficient of approximately 0.3 times the full regional PGAs has been adopted for the ULS case, which assumes acceptable seismic displacements in the order of 150-200mm. This follows conventional assessment guidance provided in NZGS Slope Stability Modules. As such, FoS values refer to stopbank resistance against these displacement thresholds.*

Other scenarios have also been checked to assess the sensitivity to failure in the relevant cases. These results also indicate satisfactory results. These cases include fully saturated stopbank and foundations (conservative case) and liquefied residual strengths.

As for the MSE wall, the overall stability of the slope also meets the requirements with the inclusion of geogrid reinforcement elements. Finalisation of grid elements requirements and internal stability assessment of the slope will be determined for during detailed design.

Transient seepage has been modelled up to a 24hr sustained flood duration, with the height being 600mm below the stopbank crest (freeboard allowance). The results indicate that for the duration of the design flood event, the phreatic surface would remain below the landward toe of the stopbanks. As such, no critical seepage is expected to occur, and therefore hydraulic gradients are practically zero. Extremely low flow rates into the landward toe zone may still occur due to capillary actions above the phreatic surface during flooding, however this flow is considered low enough to be controlled by evaporation only and should not result in any discernible seepage effects.

Note that the seepage assessment is highly sensitive to the assumed permeability characteristics of the stopbank material. Previous permeability tests of the borrow and foundation soils indicate a permeability range typically between 5e-06 to 5e-09 m³/s. We have assumed lower bound permeabilities for the purpose of this preliminary assessment. This will be refined based on further permeability results as they become available.

A typical output of the seepage analysis (and subsequent drawdown case) is shown in Figures 8 & 9 below.

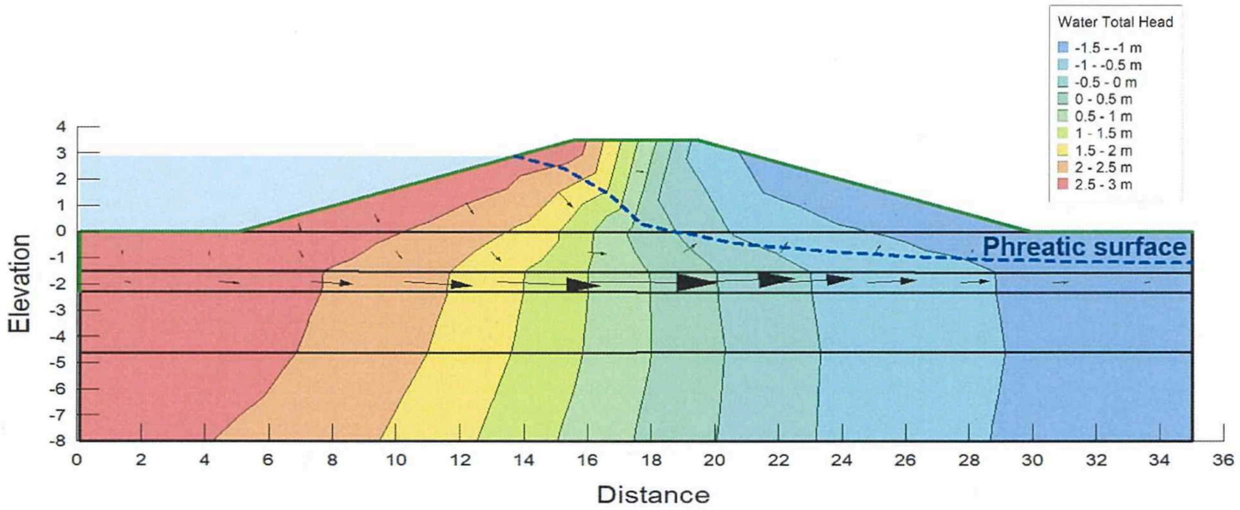

Figure 8: Representative stopbank in floodway, showing transient velocity vectors and phreatic surface (water line) development after 24hr flood peak flood duration. Note the phreatic surface still being at reasonable depth below the ground surface. Some very small arrow vectors can be seen above the phreatic surface, representing negligible volumes of capillary actions.

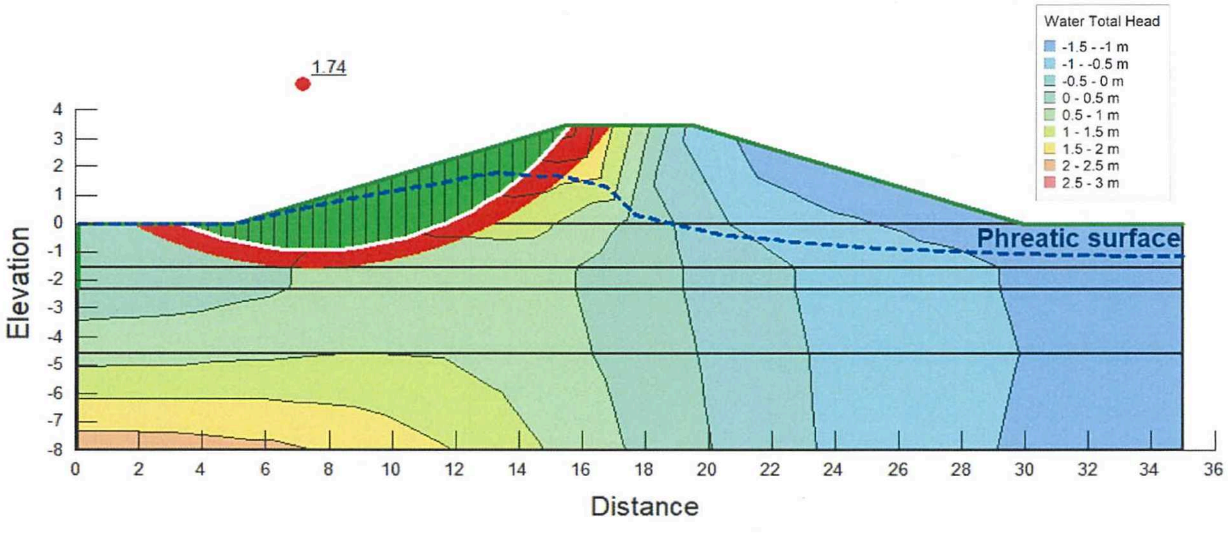

Figure 9: Rapid drawdown stability scenario showing the change in phreatic surface 1 hour after peak flood and temporarily lowering stability at toe of the floodway side of the stopbank.

3.5.5 SETTLEMENT

Estimated ground settlement (consolidation and groundwater drainage) based on the previous floodway alignment was deemed moderate: up to 100-150 mm directly beneath the stopbanks. Drainage effects are indicated to cause wider settlements up to 10mm extending a maximum distance of 25m away from the stopbank. Detailed settlement outputs are being prepared and will be presented at detailed design.

3.5.6 *BORROW MATERIAL COMPACTION TRIAL*

A limited compaction trial (results and report pending) has been recently undertaken in winter conditions at the site to assess the workability of the proposed borrow soils, namely compatibility and sensitivity to moisture. Although the site conditions at the time of trial were too wet for optimum compaction, this process has yielded a range of indicative results and useful information about the engineering properties and expected behaviour of the soils. A range of compaction machinery, lift heights, and density control methods were trialled, with results indicating the following:

Table 8: Summarised results of compaction trial (preliminary)

Trial Component	Indicative Result (preliminary)	Comments
Best Compaction Machinery	Sheepsfoot roller	Material is of limited cohesion so roller 'cleats' not holding shape very well. May improve when drier.
Optimum Number of Passes	2-3	Material appears prone to over compaction and pumping if worked too much. Will be much less sensitive when drier.
Optimum Layer Thickness	300mm	Machinery has difficulties forming good cleats with thinner layers and is destructive to finished layers. Unable to trial drying back of material in various layer thicknesses which may be a key component of constructability.
Most Reliable Density Test	Nuclear densometers and shear vanes	Cone penetrators unlikely to be very reliable for material type. Shear vanes will be useful for quick verification.

Based on the results of the trial and partial information processed to date, we believe that it should be possible to compact the soils optimally during fine summer conditions with conventional methodologies, however it is clear that the moisture sensitivity of the soils will be a key issue to manage during construction. This factor should be raised during a workshop with the physical works contractor to discuss methods and machinery that can be used to manage the soils and discuss possible contingency methodologies for any wetter-than-anticipated site conditions. Should the contactor have concerns on their ability to handle the soils, additional small-scale compaction trialling in summer conditions is well warranted to mitigate the risk of any delays and additional costs incurred during construction. This would also give the contractor an opportunity to witness the behaviour of the local soils with their proposed machinery.

3.5.7 *GEOTECHNICAL CONSIDERATIONS FOR DESIGN AND CONSTRUCTION*

The following is to be considered in subsequent design and construction tasks of the project:

- Borrow material is indicated to be highly sensitive to moisture and careful construction planning will be required. Material conditioning by spread layer drying and/or other moisture control methods (ripping etc) could be required to keep the material within optimal compaction range. A high level of geotechnical oversight will be required at regular intervals throughout the construction process.
- Some variation within the borrow material (currently simplified as a single unit) is present and should be expected.
- Groundwater is likely to be deeper than stopbank foundations in all areas, however consideration for wet foundations close to the water table may be prudent.

- The proposed Churchill Ave MSE wall (green facing) is expected to also be able to utilise the SILT borrow material, however drainage elements may be required to control an increased risk to seepage and will likely require important drainage material (gravel or similar).

4 FLOODWAY AND STOPBANK DESIGN

4.1 DESIGN PHILOSOPHY AND APPROACH

- Design event: 100-year ARI with no climate change.
- Damage during operation shall be limited such that the repair cost over a 100-year period does not exceed the cost of mitigation or result in avulsion of the main river channel.
- The floodway will function in approximately a 30-year ARI flood event.
- The grass will be used for bailage or low-intensity grazing (no permanent fencing), and grass height should be maintained between 75 and 200 mm.
- Floodway grass cover is to be reinforced where the likely repair costs will exceed the cost of reinforcement or it would compromise the floodway's operation.
- Floodgates to be used at the Waihirere Road crossing as per HBRC guidance (there are constraints related to road diversions).
- Floodway alignment (upstream end – beside Railway Road): horizontal alignment – hydraulic modelling showed that the stopbank could be shorter than the Revision 1 stopbank. Alignment was adjusted around a shed as per HBRC guidance.
- Floodway alignment (downstream end - between Waihirere Road and the river): horizontal alignment as per HBRC guidance (there are land access constraints) and cut no deeper than 2 m as per HBRC guidance (there are cultural and land access constraints).
    - This alignment requires the use of vertical walls to allow adequate cross-sectional area for the required flows.
- Floodway alignment (middle section): similar to Revision 1 alignment but slightly adjusted as required to match the upstream and downstream ends.
- The floodway constraints mean that significant erosion protection is required.

4.1.1 FLOODWAY STOPBANKS

The geometric design of the floodway is based on HBRC requirements outlined in Section 2.9.

Other geometric assumptions that have been incorporated into the Concept design are:

- Maximum footprint used is to the outside stopbank toe.
- From Waihirere Road to the end of the floodway: stopbank height increased by 1.2 m from the geometric details described above. This is to meet HRBC's requirements described in section 2.9.
- Taper the stopbank from Waihirere Road upstream of the floodway to meet the geometric design described above, at approximately 400 m from the inlet of the floodway.

4.2 TOWNSIDE STOPBANK

4.2.1 DESIGN PHILOSOPHY AND APPROACH

The geometric design of the Townside stopbank is based on HBRC requirements outlined in Section 2.9.

Along Churchill Avenue, HBRC requirements cannot be met. The proposed concept design consists of an MSE wall on one side (Mechanically Stabilised Earth) as follows:

- Riverside batter: 2H:1V
- Landside batter: 70° (with reinforced wall facing).
- The MSE wall will tie into stopbank on either side of the wall.

There are land access challenges where the townside stopbank traverses private properties on Mitchell Avenue. In this area, the stopbank geometry has been modified as follows:

- Stopbank crest width: 3 m
- Stopbank side batters: 2H:1V

4.2.2 INITIAL EROSION RISK ASSESSMENT

The proposed floodway alignment transfers flows that would otherwise have been more distributed along the riverbanks, and more concentrated further downstream, into a more concentrated discharge that occurs further upstream. This results in an increased flow rate within a section of the river when compared to the current situation. Since mean velocity depends on the sectional flow area and flow rate, the mean velocity increases. The momentum of water discharging from the floodway also causes the main river flow to be pushed through a narrower width, closer to the true right bank. While the former effect cannot be designed out, there remains potential to improve the latter by encouraging the discharging from the floodway to turn and flow downstream more rapidly, either by energy dissipation or through redirection.

Consequently, the proposed floodway alignment will inevitably raise the flow and mean velocity for a 100-year flood event between the floodway outlet and the river bend, as compared to the current situation. However, the mean velocity is a one-dimensional concept and does not necessarily represent the increased shear forces on the true right bank. Initial two-dimensional (2D) modelling provided indicative velocity estimates, but these are depth averaged and cannot fully account for turbulent mixing and the 3-dimensional effect of the two flows merging.

To better assess the actual distribution of velocities in the area and understand the associated erosion risk on the true right bank, further three-dimensional modelling was undertaken using Computational Fluid Dynamics (CFD). This work has now been completed and has confirmed there is a risk of increased velocity along the true right bank in line with the prior two-dimensional modelling. The CFD modelling also confirmed that the river naturally has higher velocities along this true right bank due to the downstream bend geometry. It is however still increased over existing.

The current design has been based on a minimum design approach, but given the confirmation of this risk, we believe the stopbank should be set back from the river as far as practical (more so than the default set back) to accommodate any potential bank slumping or erosion associated with the increased flow. Then further work can be completed to better define the scale of the risk if needed. This risk is due to the floodway alignment and cannot be designed out through changes to the floodway depth or width, though there may be some works that can lessen the effect.

The design requirements addressing this risk were confirmed during a workshop held on 3 July 2025.

4.2.3 COMPUTATIONAL FLOW MODELLING RESULTS

The CFD modelling was finalised last week, and our analysis of the results has just started. The details provided below are preliminary only.

The CFD modelling corroborated the velocity estimates from the 2D modelling, while providing greater clarity on how velocity is distributed across the channel. The results are presented in the figures below.

Early learnings:

- The 2m cut spillway option results in lower flow velocities than those observed in the CFD analysis for the wider and shallower spillway alternative.
- Velocities range between 2.5 m/s and 3.8 m/s, whereas the baseline (current scenario) displays velocities between 2 m/s and 3 m/s (3m/s is the maximum observed at the river bend only).

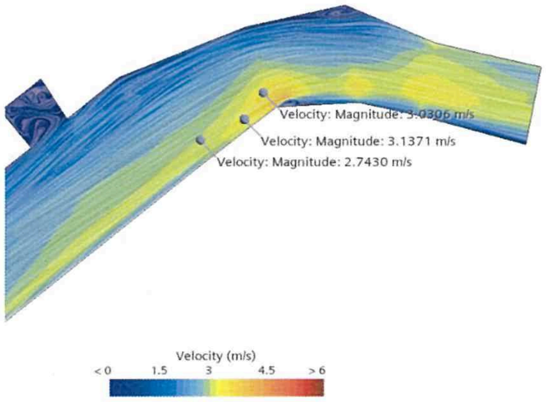

Figure 10: Estimated current velocities (Baseline).

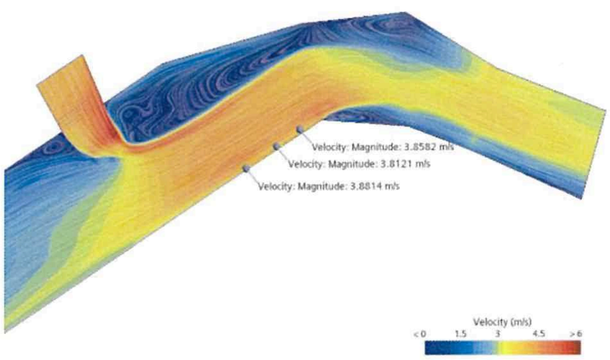

Figure 11: Estimated velocities with a 2m deep cut spillway with baffles

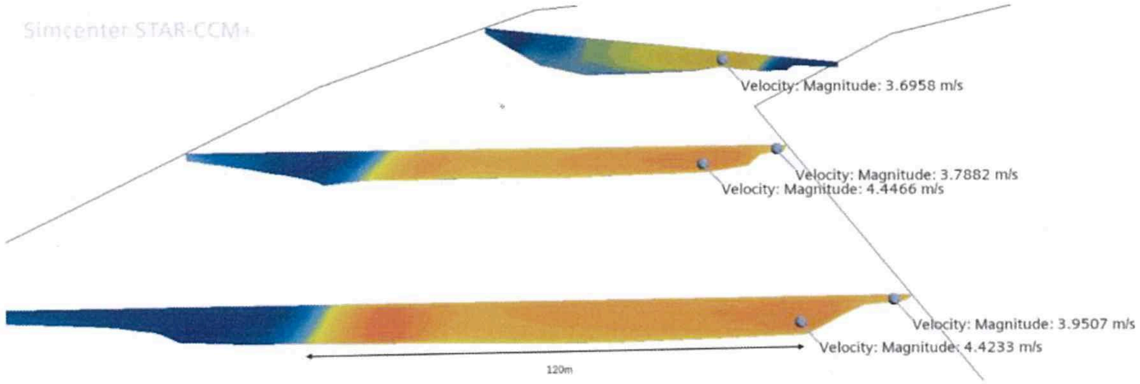

Figure 12: Estimated velocities with a 2m deep cut spillway with baffles across the river channel.

Disclaimer:

Modelling results are obtained from a numerical solution to the Navier Stokes equations and are dependent on the input values and physical model parameters. As a result, these results should be used only as a guide to engineering assessment. No experimental measurements were provided for calibration of the CFD output.

No other warranty, expressed or implied, is made as to the information and professional advice included in this report. It is not intended for use by other parties other than the initial recipients of this report.

4.2.4 UPDATED EROSION RISK ASSESSMENT

The CFD results presented above do not provide a quantitative assessment of the erosion risk (only a sediment transport or a scour-based model could provide that information). The projected increase in river velocity indicates that the channel will need to adjust to these new conditions, but the scale of the adjustment at this stage is unknown. The recommendations below are indicative only and are based on the comparison between the baseline results and the 2m cut deep (with baffles) spillway velocities. Those recommendations are based on expert judgement only and will need to be further refined once we have completed our analysis of the CFD results discussed in the previous section.

- When the estimated velocity is between 2 m/s and 2.5 m/s: it is recommended to increase the stopbank offset from 6 m to 10 m to enhance resilience and allow for future adaptive management in case signs of erosion appear.
- When the estimated velocity is between 2.5 m/s and 3 m/s: increasing the stopbank offset from 6 m to 15 m is advised.
- When the estimated velocity exceeds 3 m/s: the stopbank offset should be increased to 20 m.

It should be noted that these recommendations have not yet been discussed with HBRC at the time of this update (Revision 3). Consequently, the drawings have not been revised to reflect the proposed changes and continue to indicate a 6 m offset as previously agreed.

To better quantify the erosion risk, and how it could evolve over time, we recommend developing a sediment transport model. That would provide HBRC with a more robust risk assessment, but not a definitive answer as any model carries some limitations.

4.3 VOLUME OF EARTHWORKS

4.3.1 SITE GEOLOGY

A 3D geological model was prepared during the previous project phases. Additional site investigations (ongoing at the time of this report issue) will enable validating/updating this model during the detailed design phase, as required.

The report mentioned above describes seven geological units along the floodway. Two units, described as Silt A and Silt B, are deemed acceptable for the stopbank construction.

Other layers are considered unsuitable for the stopbank construction. Those layers will likely affect the contractor's productivity because of the required level of selectivity. This was taken into account in the construction cost estimate.

4.3.2 GEOLOGICAL VOLUMES

Using the proposed concept design model, the predicted geological unit volumes forming the concept floodway profile are described in the table below. Volumes include floodway cuts as well as undercutting required beneath stopbanks.

The floodway cut depth varies approximately from 1m at the inlet to 2m at the outlet.

Table 9: Predicted cut at concept design (numbers are rounded) within the floodway (geological units volume predicted)

Concept Floodway Excavations – Units Encountered	**Total Cut (m^3) - Indicative Yields***
Unit 1 - Topsoil	56,000 *(includes 15,000 from stopbank undercuts)*
Unit 2 – Silt A - ***proposed borrow material***	140,000 *(includes 10,000 from stopbank undercuts)*

Unit 3 - Sand	28,000
Unit 4 – Silt B	22,000
Misc. Allowance for Pavement and Drain Undercuts, Site Contouring	15,000
TOTAL	261,000

*Volumes shown in this table are from the geological model and have been rounded up. The volumes also include stopbank undercuts (25,000m^3) and miscellaneous cuts (15,000m^3). Floodway cut volume shown on the drawings (221,872m^3) plus the additional cuts (40,000m^3) matches the total in this table (261,000m^3).

4.3.3 *EARTHWORKS CUT/FILL VOLUMES*

Using the concept design model, the proposed cut and fill volumes are detailed in the table below.

Table 10: Volume of fill predicted at concept design.

Components	**Undercut – SILT A Fill Required (m^3) - Unfactored**	**Above ground - SILT-A Fill Required (m^3) - Unfactored**	**Total – SILT A Fill Required (m^3) - Unfactored**	**SILT- A Fill Required (m^3) – *Includes 30% Compaction and Selectivity Factor***
Floodway – Stopbank 1	9,000	14,000	23,000	29,900
Floodway – Stopbank 2	11,000	23,000	34,000	44,200
Townside – Stopbank 3	5,000	9,000	14,000	18,200
TOTAL	25,000	46,000	71,000	92,300

The total of cut material excess is the difference between the total cut volume (Table 9) and the total fill volume (Table 10), excluding the topsoil that will be placed on the stopbank batters - **112,700m^3** (261,000 - 56,000 - 92,300 = 112,700m^3).

5 STRUCTURAL DESIGN

5.1 OVERVIEW

The floodway design will include two key structural components: a reinforced concrete retaining wall and a reinforced concrete slab.

The reinforced concrete retaining wall will be designed to resist two primary loading scenarios: seismic loading when the floodway is empty; and hydrostatic pressure when water depth is at maximum flood level. It will stand 3.6 m high, accommodating a maximum water depth of 3.0 m while providing a 0.6 m freeboard. To facilitate this, approximately 2 m of soil will be excavated along the floodway, resulting in 2 m height of retained soil behind the wall.

The retaining wall system comprising the vertical wall, foundation slab, and downstand will have a thickness of 350 mm. The downstand is positioned at the front of the slab and is designed to mitigate erosion risks beneath the foundation slab.

The spacing between the two retaining walls will vary, with a minimum separation of approximately 100 m. At the upstream end, the wall will terminate within the stopbank section of the floodway. At the downstream end, the wall will terminate prior to reaching the river's edge. The only planned discontinuity between these two termination points is a floodgate crossing Waihirere Road

The reinforced concrete slab will span the full width of the floodway and extend longitudinally for 50 m. It will be 250 mm thick and incorporate a downstand at the upstream end to reduce erosion. Reinforcement continuity will be required across all construction joints. Baffles will be required to reduce water velocity and enhance hydraulic performance.

For the purpose of the concept design, we have allowed for a bearing pressure (SLS) of the founding soil of 100 kPa. This is to be confirmed during detailed design. Ground improvement measures may be required to enhance soil stability, mitigate liquefaction potential, and help to ensure structural resilience during flood and seismic events.

5.2 STRUCTURAL DESIGN CRITERIA

5.2.1 BASIS OF DESIGN

Importance Level	2
Design Life for Seismic Loads	50 years
Design Life for Durability	100 years

5.2.2 RELEVANT DESIGN STANDARDS

The following design codes and standards have been used for the design:

AS/NZS1170:2002	Structural Design Actions
NZS3101:2006	Concrete Structures Standard
NZS3106:2009	Design of Concrete Structures for Storage of Liquids
NZS3404:1997	Steel Structures Standard

5.2.3 DURABILITY

Based on NZS 3106, concrete exposed to stormwater under predominantly submerged and agitated or flowing conditions is classified as exposure class U. Therefore, a minimum concrete cover of 75 mm with 40 MPa concrete has been used.

5.3 LOADINGS

5.3.1 DEAD LOADS

Dead load refers to the weight of the structural components of the floodway, as well as any non-structural elements that are unlikely to change during construction and throughout the structure's use. Dead loads include the self-weight of the walls and base slab, and the retained fill.

5.3.2 LIVE LOADS

Live loads such as ground water pressure and construction surcharge shall be accounted for where appropriate.

Loadings from seismic and flood events will be applied independently of each other as they will not be expected to occur concurrently.

5.3.3 SOIL LOADS

For preliminary design, a maximum retained soil height of 2m has been assumed.

5.3.4 HYDROSTATIC LOADS

An internal stormwater level of 3 m has been considered. This has not been designed to occur concurrently with seismic loading. To be confirmed during detailed design.

5.3.5 SEISMIC LOADS

The floodway structural components have been designed to meet seismic loading requirements as per AS/NZS 1170.5. The seismic design parameters adopted are as follows:

Hazard Factor: 0.37, Wairoa, as per Table 3.3, NZS1170.5
ULS Return Period Factor: 1
SLS Return Period Factor: 0.25
Near Fault Factor: 1

5.3.6 WIND LOADS

As the reinforced concrete walls will be partially buried, wind loads are considered negligible and have been excluded from the design.

5.3.7 BUOYANCY

A buoyancy check has been carried out to ensure the reinforced concrete walls remain stable during flood events.

5.3.8 LOAD COMBINATIONS

Load combinations are applied in accordance with AS/NZS 1170 and NZS3106 for Serviceability and Ultimate Limit State design.

6 SCOUR PROTECTION

6.1 DESIGN PHILOSOPHY AND APPROACH

The proposed concept design looks at minimising the risk of erosion in the floodway, acknowledging that available materials and practicality in terms of cost and construction are key factors in the design process.

Following the delivery of Revision 1 of the developed concept design, some important modifications to the scour protection had to be made. These are as a result of narrowing of the floodway, changes to the floodway horizontal alignment and vertical geometry and the most recent geotechnical and hydraulic assessments:

- Increased amount of turf reinforcement matting (TRM) within the floodway due to an increase in water velocity and associated shear stress on the surface
- Inclusion of a reinforced concrete slab at the end of the floodway to accommodate baffles, higher average velocities and where the proposed concrete retaining walls terminate at the end of the floodway Inclusion of a buried grade control sill setback from the Wairoa Riverbank at the floodway inlet

Depth-averaged velocity and shear stress outputs from the 2D hydraulic model analysis are used to assess the performance of unreinforced grass protection and other treatment options within the floodway. Refer to Figure 13 below.

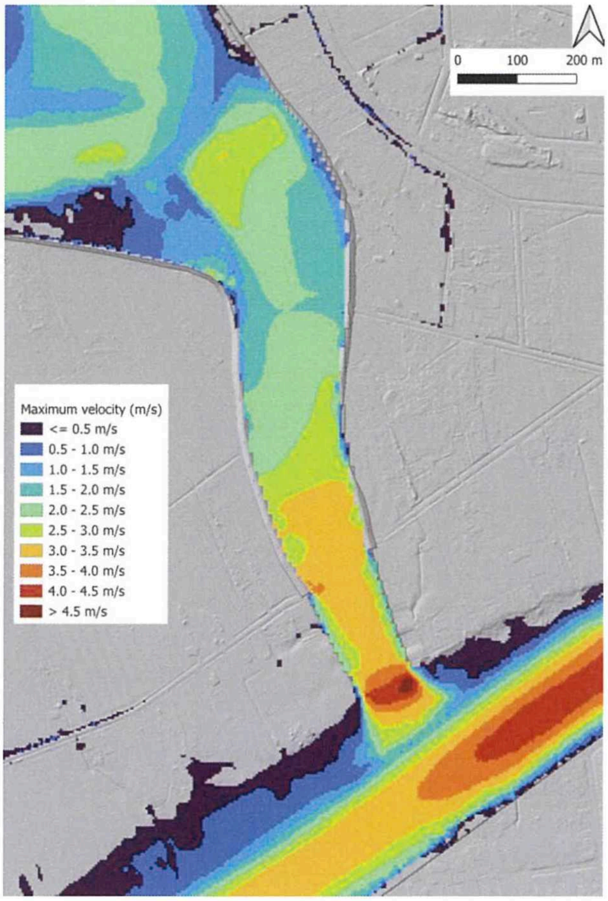

Figure 13: Depth-averaged velocity for flow through the floodway (2D hydraulic model, WSP)

Grass will be the primary form of erosion protection and may be damaged in some areas during flood events, so long as the extent of the damage is not significant / overly expensive to repair over the asset's life multiple times. The assessment of the grass is reliant upon good condition/coverage and that any insitu noncohesive materials such as sand are minimally present within high velocity areas. The assessment as per FHWA (2005) guidance found that the use of grass as erosion protection is stable up to velocities of 2.50 m/s.

Beyond this, erosion of the grass surface and underlying soils in excess of small localised scour failures can be expected to occur. For the concept design, velocities beyond 2.75 m/s require specific scour protection. This exceeds 2.5 m/s as it is understood that some damage following activation of the floodway is acceptable to HBRC and will reduce the scope of the capital works.

The overall critical points for scour protection are covered in the following sections 6.2 to 6.4.

Refer to Section 2.2 for a list of Standards and Guidelines adopted.

6.2 FLOODWAY TREATMENTS AND LEVEL OF SERVICE

Predominantly throughout the floodway where velocities exceed 2.75 m/s, Turf Reinforcement Matt (TRM) will be used in areas of higher velocity to strengthen the grass cover, where grass cover alone will not be sufficient. This is proposed to be a three-dimensional open non-woven mat, laid within the topsoil layer. Often the failure mechanism of geotextiles is the penetration of flows between the geotextile and topsoil causing uplift of the geotextile layer and grass. A well-designed system considers the following:

— Well anchored – woven fabrics or similar can struggle to maintain the tensile strength of the fabric with the necessary penetrations from pins that may create weak points

- Ensures the establishment of grass – woven fabrics or similar and grass species must be designed so that the grass density is not compromised and allows for the blade size to grow through. However, for open non-woven mats, the selected grass species needs to have deep enough root structure that it can anchor the mat to the surface
- Prevention of the buildup of pressure under reinforcement matting – products that are laid directly on seeded surfaces are not anchored into the root structure, eventually pins/matting will deteriorate and pressure will likely uplift the matting
- Softer and three-dimensional matting built into the topsoil surface will be more resilient to potential damage from pasture machinery and livestock

Stopbank areas that turn into the oncoming flow – flow will directly impact these areas and concentrate as they turn and run along the face. TRM is extended along the toe of the stop bank where this is occurring.

Supplementary bank vegetation is proposed to further reinforce the bank of the Wairoa River at key locations (the inlet, outlet, and bank opposite the outlet of the proposed floodway). This would consist of willow poles with ongoing management via coppicing, with some native species interspersed. Coppicing is a simple way of managing willow trees by cutting them back close to the ground. This encourages the willows to grow lots of new shoots from their base.

In areas of very high velocity (i.e. velocity > 4m/s), or critical points needed to allow correct floodway operation, wire-enclosed rock, riprap or similar will be used. These would be void-filled and topsoiled for additional protection and to hide their presence. Acknowledging that good quality quarry rocks are not available locally (or at a higher cost coming typically from the Taupo area), the following needs to be considered when confirming riprap or similar options:

- Design life: rock infilled wire mesh (Reno mattress or equivalent) may have a life as short as 25 to 30years, though it may also extend beyond this.
- Riprap used (if locally sourced) is likely to weather over time and may last 30 years to 100 years, depending on the rock source/rock properties.

6.3 FLOODWAY INLET AND ENTRY SILL

Maintaining the inlet crest level is important to ensure flow is evenly distributed and to govern the total flow rate. A buried concrete grade control sill is proposed at the floodway inlet crest currently, refer to **Error! Reference source not found.** below.

The concept design includes the following assumptions:

- The sill extends 1.0 m below the finished ground level
- 20 m of buried rock scour protection downstream of the sill to allow for flows overtopping the sill. The layer thickness of the rock protection, like the sill depth, is expected to be at least 1.0 m thick

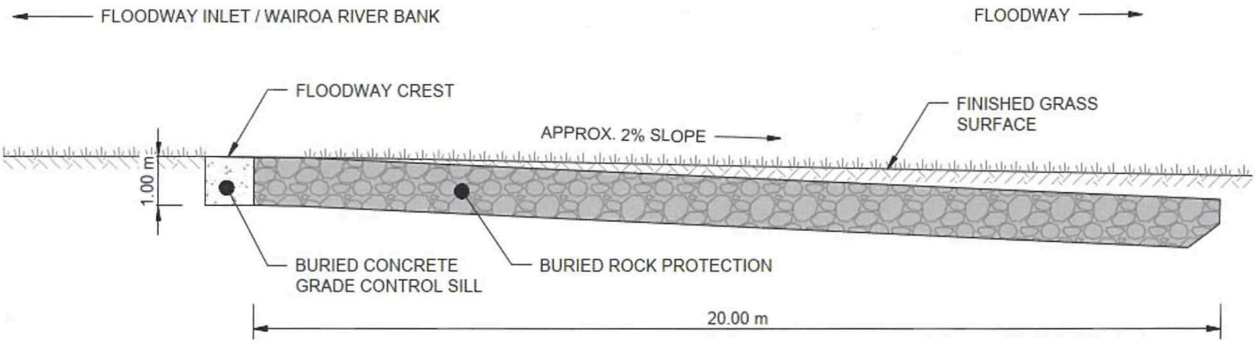

Figure 14: Proposed buried concrete grade control sill setback from floodway inlet

The proposed floodway inlet location has been subject to continual erosion associated with the bend in the river. Rapid drawdown is considered the likely cause (mass failures), possibly in conjunction with toe erosion. For now, the sill has been set back 80 m from the riverbank to allow for future erosion, based on inference from historic aerial imagery. Further assessment is required during detailed design confirm the scale of this setback.

6.4 FLOODWAY OUTLET AND TERRACE

Energy will be highest at the existing terrace at the downstream end of the floodway. Significant soil loss/erosion at this location could ‘head cut’ upstream, progressively increasing the total amount of erosion and cause damage throughout the floodway. Narrowing of the floodway for the revised concept design has predominantly occurred south of Waihirere Road and the revised floodway geometry has increased the averaged exit velocity of floodway flows at the confluence with the Wairoa River. The outlet meets the river at a 90-degree angle and flow is pushed to the true right (south) bank of the river at this transition where increased velocities and bed shear stress pose a risk to the riverbank. Preliminary Computational Fluid Dynamics (CFD) modelling has been undertaken to assess this effect and configure the floodway outlet to reduce the risk to the true right (south) riverbank. The full results of this modelling are still in progress and will be issued at a later date.

A reinforced concrete slab is proposed at the end of the floodway to accommodate the following:

- Inclusion of four rows of 0.5x1.0x1.0 m (H x W x D) rectangular concrete baffles no further than the end of the concrete retaining walls. This has been included in the CFD modelling and confirmed to reduce the scour risk to the true right (south) riverbank, refer to Figure 15 below.
- The concrete slab extends into the floodway upstream of the baffles, a total of 40 m past the end of the retaining walls. This is to allow for additional turbulence upstream of the energy loss through the baffles and provides flexibility during detailed design to further optimise the slab and floodway configuration to include a stilling basin and/or additional baffles if necessary.
- The concrete slab extends 20 m downstream of the end of the floodway onto the river terrace. This is where the highest velocities in excess of 4.5 m/s occur. The slab does not cover the full area where these velocities occur but enough to provide the highest level of resilience against scour and undermining of the concrete retaining walls. The areas in private land adjacent to this slab extension will not include scour protection, extending the slab further into the terrace may cause a future flow obstruction and exacerbate erosion to surrounding areas and also become susceptible to undermining.

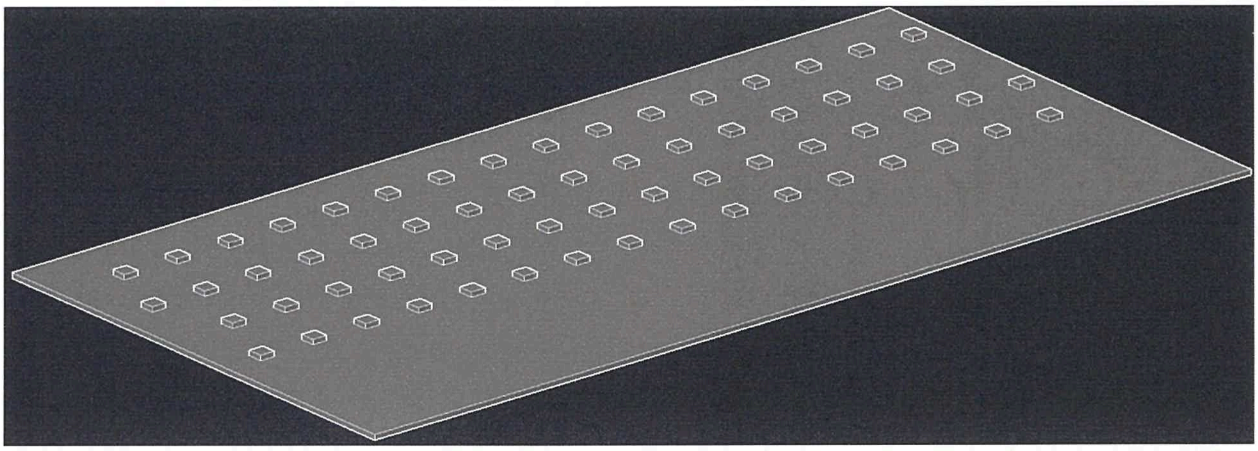

Figure 15: 3D modelled reinforced concrete slab and baffles used for CFD modelling

7 WATER SUPPLY

7.1 DESIGN PHILOSOPHY AND APPROACH

- To accommodate the proposed floodway, the watermains along Ruataniwha and Waihirere roads will need to be lowered to allow adequate cover and maintain service integrity. The intent is to trench those services as per the current alignments below the base of the floodway.
- The watermains will be laid with a gradual reduction in elevation to facilitate the passage of air bubbles without the need for air valve.
- A way to protect the watermains from scour risk will be assessed at detailed design (if deemed too high with grass only).
- Refer to Section 2.2 for a list of Standards and Guidelines adopted.

7.2 WAIHIRERE ROAD WATER SUPPLY PIPE

The Waihirere Road watermain is a one-way supply to a small number of rural properties on a no-exit road. The existing 50mm diameter steel pipe will be replaced with 110mm PE to future proof the section of pipe across the floodway. The lowered pipe will reconnect with the existing water pipe well outside the floodway stopbanks, to allow easy access for future repair works.

7.3 RUATANIWHA ROAD WATER SUPPLY PIPE

The Ruataniwha Road watermain is a one-way supply to a small number of rural properties on a no-exit road. The existing 50mm diameter PE pipe will be replaced with 110mm PE to future proof the section of pipe across the floodway. The lowered pipe will reconnect with the existing water pipe well outside the floodway stopbanks to allow easy access for future repair works.

The watermain running along Ruataniwha Road may need to be realigned because Ruataniwha Road will be at a higher elevation and this would increase the cover over the watermain, making it more difficult to access for repairs. This will be assessed during detailed design.

8 STORMWATER

8.1 DESIGN PHILOSOPHY AND APPROACH

- The stormwater design will be in accordance with Hawke's Bay Regional Council Stormwater Management guidelines and the Wairoa District Council Engineering Code of Practice.
- Floodway: Shaped to manage pluvial flow such that it does not cause regular nuisance or waterlogging that would prevent grass cutting operations.
- The stopbanks and concrete floodwalls shall not create nuisance flooding outside of the scheme boundary on other property in addition to existing during a 10-year ARI rainfall event and shall not result in flooding of any dwellings during a 100-year ARI rainfall event.
- External drainage shall be modified as required to maintain its function and to prevent water being trapped behind the stopbank. Where diversion is not practical, it shall be diverted into the floodway via a culvert installed with a non-return valve.
- The proposed Townside stopbank will be built over a number of stormwater pipes and overland flow paths that service the urban area and discharge to the Wairoa River. Additional drainage and pipe upgrades are required to collect and discharge stormwater that would otherwise be trapped behind the stopbank. A non-return valve is planned to be installed at the outlet of each culvert. This will be verified during the detailed design phase.
- Refer to Section 2.2 for a list of Standards and Guidelines adopted.
- The stormwater design will be investigated and finalised during the detailed design phase.

8.2 STOPBANK 1 AND STOPBANK 2 (FLOODWAY) STORMWATER DESIGN

8.2.1 SWALE DESIGN

Existing overland flow paths (OLFP) will be obstructed by the proposed stopbanks, necessitating the formation of a swale and a concrete box drain on the land side of stopbank 1 to intercept this flow and convey it to the end of the stopbank. It will then discharge at the bottom of the floodway and flow over land to the river. The details of this will be worked out in the detailed design. The swale and box drains are sized based on the contributing catchment area, delineated using GIS-based analysis of overland flow paths across the terrain to identify the area feeding into the swale and box drain, as seen in Figure 13. It is necessary to transition from a swale to a concrete box drain due to the restricted drainage footprint at the end of the floodway.

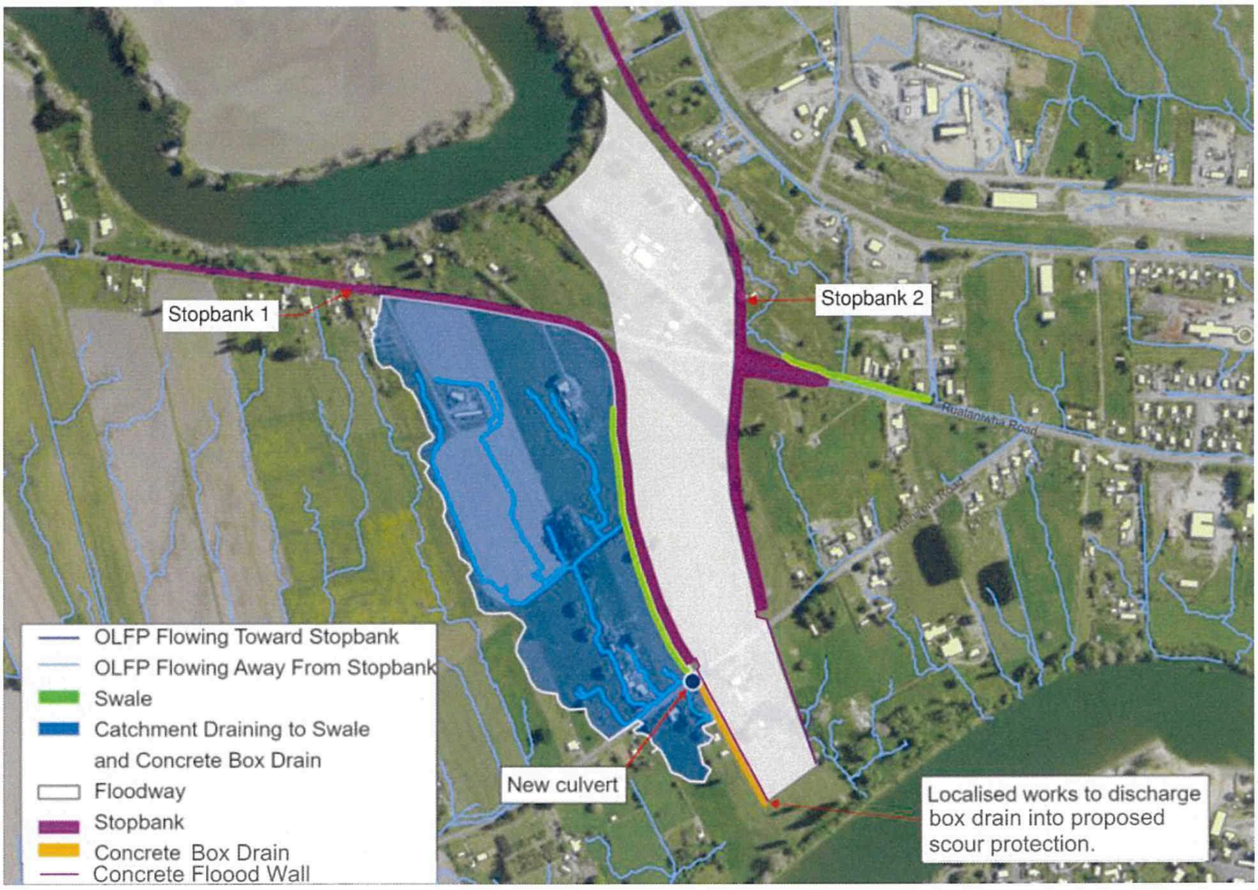

Figure 16: Overland flow paths and swale and box drain alignment for stopbank 1 and stopbank 2.

The swale and box drain will also pick up intersecting farm drains and roadside drains, which govern the size and invert of the swale (typically 0.5 m deep) and box drain (varies in depth up to 1.7 m). A typical section through the stopbank and the swale is shown in Figure 17. Figure 18 shows a typical section through the concrete floodwall and the box drain.

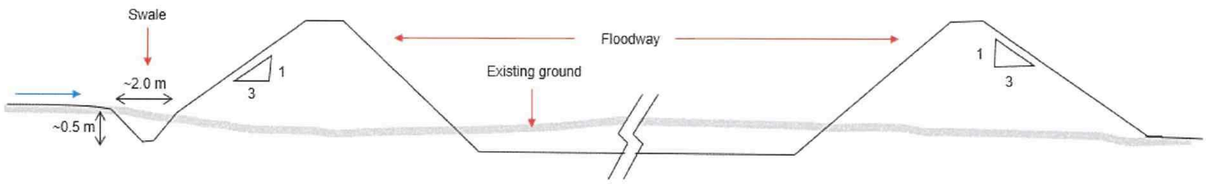

Figure 17: Typical section through the floodway, showing the stopbanks and the swale (view looking north/upstream).

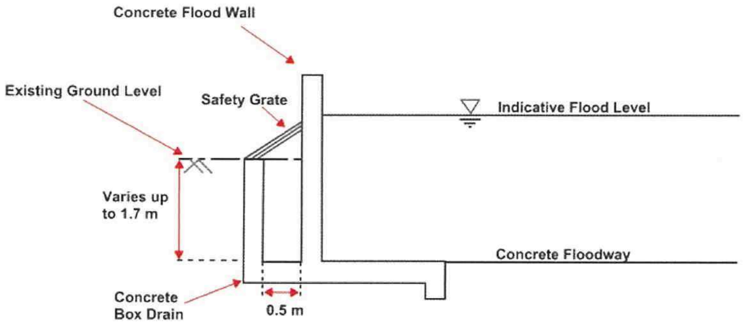

Figure 18:Typical section through the concrete section of floodway, showing concrete flood wall with concrete box drain (view looking north/upstream).

A culvert will be required where the swale crosses Waihirere Road on the side of stopbank 1. A small section of swale is required on the eastern side of the floodway along Ruataniwha Road (stopbank 2). The flow from the stopbank 2 swale then connects with the existing well-defined flow path that conveys runoff. This flow path will also discharge into the scour protection at the bottom of the floodway. It is expected that a new culvert will be required where the existing flow path crosses Waihirere Road on the side of stopbank 2. Both culverts will be sized during detailed design to prevent runoff building up and causing flooding behind the road embankment.

To view details of stopbank transition to the concrete flood wall, refer to drawings 2-T4441-WSP-03-DR-CIV-711 and 2-T4441-WSP-03-DR-CIV-712.

8.3 STOPBANK 3 (TOWNSIDE) STORMWATER DESIGN

8.3.1 PIPES / CULVERTS THROUGH THE STOPBANK

The proposed Townside stopbank will be built over a number of stormwater pipes and overland flow paths that service the urban area and discharge to the Wairoa River. Additional drainage and pipe upgrades are required to collect and discharge stormwater that would otherwise be trapped behind the stopbank. The Wairoa District Council's stormwater model is being used to size the stormwater upgrades required to avoid flooding behind the stopbank (i.e. when it rains but the river is not in flood). Sizing will be confirmed during detailed design and will allow for potential future network upgrades.

Refer to drawing 2-T4441-WSP-03-DR-CIV-060 for stopbank 3 placement.

8.3.1 HEADWALL AND FLAP GATE

All outlets discharging to the river through the stopbank will include non-return valve / flap gate to prevent high river levels back flooding the town. Outlets will also have a concrete headwall to provide some protection from vegetation and debris that could cause blockage as well as providing for easier inspection and maintenance.

9 ROADING GEOMETRIC DESIGN

9.1 DESIGN PHILOSOPHY AND APPROACH

- Ruataniwha Road and Waihirere Road are designed in accordance with the standards and guidelines outlined in Section 2.2.
- The cross section adopted for both roads are in accordance to the Cross-Section Guidelines for Verges on Rural Roads (Drawing No. C30) of the Wairoa District Council Engineering Code of Practice. The proposed carriageway widths will match the existing roads to maintain the current level of service.
- Both roads are designed to accommodate a Semi-Trailer 18 m Design Vehicle.
- The minimum K value of curves currently applied is for rural roads with streetlighting. During the detailed design, this will be improved and revised to rural roads without streetlighting.

9.1.1 DESIGN SPEED

According to Section 3.2.4 of AGRD3, the "design speed should not be less than the expected operating [...] speed for the road". A design speed of 60 km/h has been adopted for both roads.

- Ruataniwha Road complies with AGRD03 requirements as the operating speed is 50 km/h.
- Waihirere Road does not comply with AGRD03 as the current operating speed is 70 km/h.

The design speed for both roads may however need to be reduced during the detailed design phase to improve property access and other design elements. Appropriate traffic calming measures will need to be implemented to ensure vehicles slow down and maintain safe operating conditions.

9.1.2 SIGHT DISTANCE

Based on Table 5.5 from AGRD03, the desirable minimum Stopping Sight Distance (SSD) for cars, for a 60 km/h design speed, assuming a 2.5 second driver reaction time, is 81 m. However, applying this SSD would cause all stopbank configurations to encroach on property To avoid this, we adopted a reduced reaction time of 2 second, resulting in an SSD of 73 m. This is the minimum acceptable SSD for a 60 km/h design speed given the context of the road. We have also checked the minimum SSD for trucks, and the proposed road alignment meets those criteria as well.

SSD values may however need to be reduced during detailed design phase to improve property access. Further assessment and appropriate traffic calming measures will need to be investigated and implemented to maintain safe operating conditions.

10 PAVEMENT

10.1 DESIGN PHILOSOPHY AND APPROACH

- Austroads guide to Pavement technology Part 2: Pavement structural Design
- NZTA Specifications (TNZ B, TNZ M, TNZ P, TNZ T)
- Failure risk profile (from the New Zealand guide to pavement structural design) is "Less than $5x10^6$".
- Pavement drainage design will be prepared during the detailed design phase.

10.2 INSITU GROUND CONDITIONS

The geotechnical investigations presented in the factual report indicate that the in situ subgrade (Silt A) has a CBR value of 1%, which is considered very low. Due to the subgrade's limited strength, it is necessary to construct a platform to facilitate the placement and compaction of subsequent pavement layers. Six pavement options were evaluated. Based on high-level cost estimates prepared by WT Infrastructure, the typical details outlined in the following sections have been selected.

10.3 TRAFFIC LOADING DESIGN

Proposed traffic loading for Ruataniwha and Waihirere Roads are summarised in the table below.

Table 11: Traffic loading design.

Road	Ruataniwha Road	Waihirere Road
Parameters	Design Period	Design Period
	25 Years	25 Years
AADT (Mobile Roads est. 2023)	60	312
Traffic Growth Rate	2%	2%
AGF	31.5	31.5
Direction Factor	0.5	0.5
%HCV	16	8
TLD	Eskdale WIM	Eskdale WIM
ESA/ HV	1.8	1.8
Axle Groups per HV	3.1	3.1
Design Load (NDT)	1.71×10^{5}	4.45×10^{5}
Design Load (DESA)	9.92×10^{4}	2.58×10^{5}

10.4 RUATANIWHA ROAD - PAVEMENT TYPICAL SECTIONS

Figure 16 illustrates the standard pavement details suggested for Ruataniwha Road, situated outside the floodway where water exposure is not anticipated during its designed lifespan. In areas where the road is integrated with the stopbank, the stopbank's height (at the top of the freeboard) will align with the upper surface of the construction platform, which uses lean mix concrete. This lean mix is considered sufficiently impermeable to deliver the same performance as compacted silt (in terms of seepage), though this assumption will be validated in the detailed design phase.