Bay of Plenty Regional Council

Detailed Design Report
Rangitāiki Spillway Project

Norconsult NZ

June 2022

Norconsult

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Summary

The Rangitaiki River Major Scheme was devised by the Bay of Plenty Catchment Commission in the 1960s and constructed through the 1970s to alleviate flooding on the Rangitāiki Plains. A major feature of the scheme is the floodway, running parallel to the Rangitāiki River on the right bank from approximately 2 km upstream of Edgecumbe, following the alignment of Reid's Central Canal to re-join the river near the coast.

In response to the 2004 river flooding, BoPRC initiated works to mitigate flooding in the region, including the Rangitāiki Floodway Project which increases the conveyance capacity of the floodway and the proportion of flow down the floodway during high river flows. Construction works to widen the Rangitāiki Floodway and to provide an additional outlet are currently being completed (by Others).

Physical works to upgrade the spillway on the Rangitāiki River to pass more flow into the floodway has been programmed by BoPRC for summer 2022/23, following completion the downstream works and the realisation of additional floodway conveyance capacity. The spillway location, and floodway in relation to Edgecumbe township are depicted in Figure 1.

Key components of the Floodway project include:

1. Various sections of channel widening and raising of stopbanks to increase the capacity of the Floodway to 190 m³/s,
2. Potential offline storage areas to mitigate peak flows;
3. Upgrades to the existing fixed crest overflow weir spillway into the Floodway, to
   (i) begin operating at lower flood levels (>1/20 AEP event), and
   (ii) pass the design discharge of 190 m³/s when the flow in Rangitāiki River (upstream of the spillway, measured at Te Teko) is 804 m³/s, nominally a 1/100 AEP event.
4. A new gated spillway immediately adjacent to the overflow weir to provide 'contingency' release into the floodway in the event that the overflow weir does not develop the required design discharge.

Norconsult New Zealand Ltd. (NNZ) has been engaged to provide the detailed design of a new gated spillway and (ungated) overflow spillway to satisfy item 4 above. Riley Consultants (RILEY) has been engaged to provide geological and scour design inputs. RILEY are also responsible for the detailed design of the overflow spillway civil structure, based on hydraulic finished profiling provided by Norconsult.

The overflow spillway is to be capable of discharging 190m³/s at the design flood (nominally the 1:100 AEP flood event) with an additional, adjacent gated spillway being capable of discharging up to 40m³/s in the same event. The gated spillway is to be used as a 'contingency' measure to ensure that the spillway flow requirements are met should the overflow spillway discharge fall short of the required capacity. The maximum scheme discharge shall not exceed 190m³/s during the 1:100 AEP flood event.

Overflow Spillway

Development of the overflow spillway has progressed the preliminary engineering design (PED). The overflow weir and discharge system developed during the PED operated as a broad crested weir of 210m crest length at crest EL 5.90 m. The crest discharged to a 210m shallow sloping discharge channel that was assumed to be grass covered.

Early in the detailed design phase BoPRC reviewed the river rating and required Norconsult to adopt river rating data produced by Others. A review of the updated rating for XS36 highlighted a +244mm river level difference from the base case adopted at PED. The resultant detailed design comprises a crest width of 104m length and crest invert of RL6.0m at the upstream end of the overflow spillway with four equi-spaced steps of 0.05m raising up to RL 6.15m at the downstream end of the spillway.

The crest is finished in a concrete capping and contains an energy dissipation channel on the downstream side of the crest to contain the hydraulic jump and avoid unnecessarily onerous scour erosion works in the outlet channel.

The final civil/structural design of the overflow spillway is being handled by Others.

Gated Spillway

The gated spillway comprises a series of sheet pile walls for scour protection and water entrainment, a reinforced concrete gate structure complete with three identical, hydraulically actuated radial gates and a downstream energy dissipator, for the dissipation of gate discharge energy prior to flow entering the outfall channel.

This design report includes commentary on the design process, analysis and conclusions for the new spillways. Appendices are included covering a copy of the design criteria memorandum, design calculations and drawings The technical specification are provided separately to this report.

Contents

1 Introduction

The Bay of Plenty Regional Council (BoPRC) have engaged Norconsult NZ Limited (Norconsult) to provide engineering design services for a new spillway and contingency gates at the head of the Rangitāiki Floodway on the Rangitāiki River approximately 2km upstream of the Edgecumbe in the Bay of Plenty.

This report documents the detailed design phase of the project as partial completion of BoPRC contract *2020 0130 Rangitaiki Spillway Design Preparation dated 8 May 2020*.

1.1 Background

The Rangitaiki River Major Scheme was devised by the Bay of Plenty Catchment Commission in the 1960s and constructed through the 1970s to alleviate flooding on the Rangitāiki Plains. A major feature of the scheme is the floodway, running parallel to the Rangitāiki River on the right bank from approximately 2 km upstream of Edgecumbe, following the alignment of Reid's Central Canal to re-join the river near the coast.

In response to the 2004 river flooding, BoPRC initiated works to mitigate flooding in the region, including the Rangitāiki Floodway Project which increases the conveyance capacity of the floodway and the proportion of flow down the floodway during high river flows. Construction works to widen the Rangitāiki Floodway and to provide an additional outlet are currently being completed (by Others).

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Key components of the Floodway project include:

5. Various sections of channel widening and raising of stopbanks to increase the capacity of the Floodway to 190 m^3/s,
6. Potential offline storage areas to mitigate peak flows;
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   (i) begin operating at lower flood levels (>1/20 AEP event), and
   (ii) pass the design discharge of 190 m^3/s when the flow in Rangitāiki River (upstream of the spillway, measured at Te Teko) is 804 m^3/s, nominally a 1/100 AEP event.
8. A new gated spillway immediately adjacent to the overflow weir to provide 'contingency' release into the floodway in the event that the overflow weir does not develop the required design discharge.

The Rangitāiki Floodway project will reduce flood levels in the Rangitāiki River from upstream of Edgecumbe to the river mouth. This will in turn reduce pressure on Rangitāiki River stopbanks during large flood events.

Figure 1: Location of spillway, with floodway flow indicated.

1.1.1 Previous Work on Spillway Options

1.1.1.1 Concept Design Phase

Investigations into the spillway upgrade by Norconsult include the *Rangitaiki Spillway Analysis - Project Initiation Report* (Norconsult 2018), and the *Rangitaiki Spillway Analysis - Options Report* (Norconsult 2019), which provides conceptual options for increasing the spillway capacity and increasing the reliability by providing contingency capacity.

The conceptual spillway upgrade design considered a lowered fixed-crest of approximately 90 m width, with two or more contingency gates of approximately 10 m total flow width, located within the existing spillway (lowered stopbank length of approximately 230 m). The remainder of the existing spillway would be reinstated to the adjacent stopbank crest level. Two options for the spillway location are shown in Figure 2.

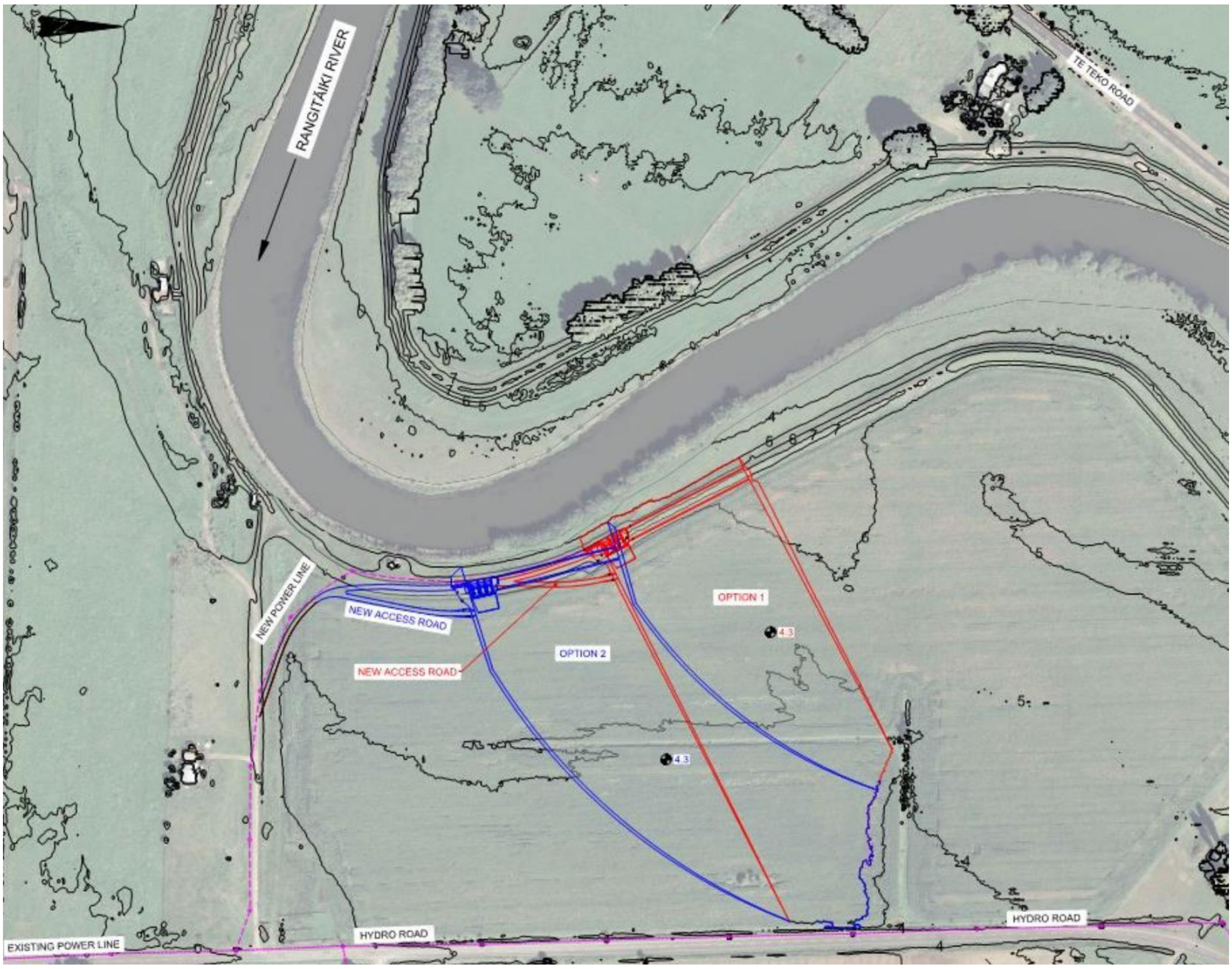

Figure 2: Conceptual spillway upgrade options from 2019 (Source: Norconsult drawing C-40-D-01, Rev C)

Key criteria and decisions for the spillway upgrade at that stage included:

- A nominal overflow weir discharge capacity of 190 m³/s with a flow of 804 m³/s in the Rangitāiki River upstream (provisionally 1/100 AEP flood peak river level);
- The spillway crest level shall be set to not pass water into the floodway for Rangitāiki River flows lower than 500 m³/s (provisionally 1/16 AEP);
- An additional discretionary 'contingency' capacity of 40 m³/s shall be provided at the 1/100 AEP flood peak river level. This capacity shall be provided by radial gates, which are considered to provide the best-value option to BoPRC in terms of reliability, cost and operational constraints, and provide the ability to close under gravity in an emergency situation.

Norconsult further refined the conceptual design to provide clarity and generate construction and engineering cost estimates for the works in mid-2019 (Doc ref: NNZ1827-G-00-M02-A and NNZ1827-G-00-M03-B respectively). The outcomes of these reports were discussed at a meeting between BoPRC and Norconsult in which a request for proposal for the detailed design of the works was made.

1.1.1.2 Preliminary Engineering Design (PED) Phase

Norconsult's proposal (document 1918-G-00-P01, Version A) was submitted 23 September 2019 for the detailed design, around the same time as the BoPRC Engineering Panel was under evaluation. As such, the detailed design scope was redefined to a PED which was awarded on 08 May 2020, with the intention that a detailed design scope would be commissioned at some future stage.

The PED phase comprised the following tasks as part of contract 2020 0130 Rangitaiki *Spillway Design Preparation*:

- Task 1: Project Management
- Task 2: Data Review
- Task 3: Design Criteria Memorandum
- Task 4: Hydraulic Assessment and Preliminary Design

Boundary conditions for the River and floodway were provided by River Edge Consulting based on 2009 river cross sections.

Model runs were undertaken iteratively using Telemac-2D software for revised spillway arrangements ('proposed conditions'), starting with a two-level fixed-crest overflow spillway, located in a similar position to the existing spillway. Spillway discharge for the 1/20, 1/40 and 1/100 AEP events (assuming the gated spillway is closed) were compared with the following design criteria:

1. The ungated spillway shall convey a nominal maximum of 190 m³/s into the Floodway during the Design Event, being a river flow of 804 m³/s.
2. The ungated spillway shall not pass more than 40 m³/s for the 1/40 AEP event.
3. The ungated spillway shall not pass any appreciable flow for the 1/20 AEP event (i.e. the spillway crest is set at the 1/20 AEP river level).

The preferred arrangement for the ungated spillway comprised a single level overflow weir of invert level 5.90m and 210m length. A single level spillway, rather than a two-level spillway, was adopted as it meets the capacity criteria and to simplify construction and maintenance. A single level spillway, rather than a sloping spillway (in the direction of the river axis), was selected based on guidance in May et al (2003)[1], to simplify construction and maintenance.

The gated spillway section was not evaluated as part of the PED with the intention that this would be refined during the detailed design phase.

The location of the ungated spillway and the gate structure is shown in the following figure.

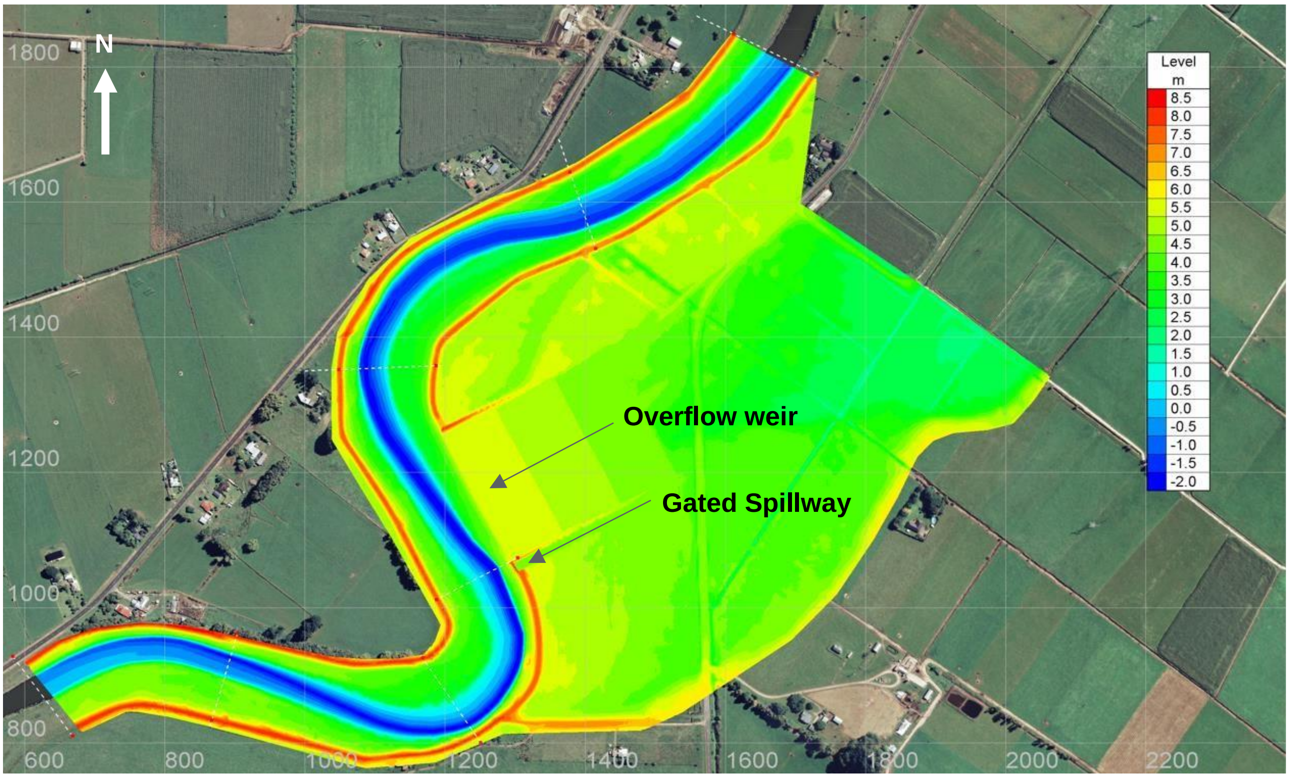

Figure 3: Model domain and geometry including preferred arrangement for ungated spillway and gate structure

[1] May R, Bromwich Y, Gasowski Y, Rickard C, *Hydraulic Design of Side Weirs*, Thomas Telford, 2003

The following deliverables were provided under this commission:

- Riley Consultants Ltd, *Rangitāiki Spillway Gate, Geotechnical Desktop Review*, Ref 200155-B, 10 July 2020
- Norconsult, Memo titled *Rangitāiki Spillway Length – Implications of Awa flood modelling*, Doc ref: 1918-C-30-M01-A, 3 September 2020
- Norconsult, Memo titled *Rangitāiki Spillway Arrangement Options*, Doc ref: 1918-C-30-M02-A, 19 November 2020
- Norconsult, *Rangitāiki Spillway Design Preparation, Hydraulic Assessment*, Doc ref: 1918-C-30-M03-A0, 12 February 2021
- Norconsult, *Rangitāiki Spillway, Design Criteria Memorandum*, Doc ref: 1918-C-30-M04-A, 4 March 2021
- Norconsult, *Rangitāiki Spillway, Design Criteria Memorandum*, Doc ref: 1918-C-30-M04-B, 31 May 2021
- x3 Preliminary Engineering Design drawings, Nos:
  - C-40-D-01-D
  - C-40-D-02-B
  - C-40-D-03-B

Version A0, 'Pre-Draft' of the PED report was issued 12 February 2021 to facilitate discussion of model parameters and assumptions with Phil Wallace, who undertook modelling of the downstream effects (flood volumes and depths within the Floodway). The report was subsequently updated with the first version released on 01 June 2021 (Doc ref: 1918-C-30-R001-1)[2].

The PED findings were presented to BoPRC in a workshop format on 09 June 2021.

1.1.1.3 Detailed Design Phase

Norconsults original proposal for the detailed design phased was revised and resubmitted 04 June 2021 following the completion of the PED phase for discussion in the upcoming PED workshop at BoPRCs request (Doc Ref: 2115-G-30-P01-A). Project award for the detailed design phase was received 07 July 2021.

The following Variations were also awarded during the course of the detailed design phase:

VO-01 (awarded November 2021): Assistance in the Registration of Interest (ROI) phase of the project, to assist in the scoring and progression of Civil and M&E Contractors into the RFP phase

VO-02 (awarded November 2021): Revised Hydraulic modelling work, resulting from a change of river hydrology by BoPRC during the course of the project. River hydraulic modelling and boundary conditions by RiverEdge were revised with more recent and detailed work by AWA, which was completed in parallel with Norconsults detailed design work for the spillway. The updated modelling work resulted in an increase in river water levels of about +260mm during the 1:100 AEP flood event.

VO-03 (awarded March 2022): Additional civil works design and hydraulic modelling resulting from an increase in river levels and previously unforeseen energy dissipation requirements. The new work comprises an upstream sheet pile training wall (previously concrete design), sheetpile transition wall, and energy dissipation structure downstream of the gated spillway.

A summary of the detailed design is presented in this report.

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[2] This report supersedes the findings in Norconsult's memo of 3 September 2020 (Norconsult 2020) and the memo of 19 November 2020 (Norconsult 2020a).

1.2 Scope of Detailed Design Works

Norconsults scope of work for the includes the following:

1.2.1 Detailed design

A defined detailed design nominally comprising technical specifications and drawings suitable for the Contractor to fabricate, construct and commission the works. The scope of the detailed design work includes:

- Hydraulic modelling of the site, to determine geometry of the overflow weir and gated spillway to
    - meet the design criteria
    - provide surface profiles for use in the design of the overflow weir civil structure (by Others)
    - contain the hydraulic jump on the downstream side of the overflow weir into a local energy dissipation channel, thus reducing the extent of downstream scour protection controls.
- Design of an upstream sheetpile training wall, to guide water into the gated structure whilst protecting the stopbank.
- A concrete gated structure and downstream energy dissipation structure, to provide contingency flood discharge to the overflow weir
- New radial gates, complete with embedded parts, to be installed within the gated structure
- A sheet pile transition wall between the gated structure and the overflow weir, to guide water to the overflow weir without causing an unwanted flow regime downstream of the weir
- A concrete slab suitable for receiving a new gate house for housing of the radial gate operating plant.

1.2.2 Reference Design

A reference design nominally comprising technical performance and quality specifications and reference drawings on which the Contractor is to base their detailed design. The reference design permits the Contractor to adapt the Norconsult design to better suit their experience, suppliers, and/or plant equipment capabilities on the basis that it meets or exceeds a defined set of performance criteria as set out in the technical specifications and drawings.

The scope of the reference design work includes:

- Radial gate hoisting system, inclusive of hydraulic cylinder, lines, fittings and hydraulic pack
- Electrical supply, controls and instrumentation
- Gate house procurement
- Security and safety fencing

1.3 Contract Packaging

The Works will be delivered in three contract packages at BoPRC's request. The Technical specification for each Contract sets out the respective design responsibilities and interface points between the various contracts. Where appropriate, interface points have also been provided on the Contract drawings.

<u>Contract Package 1 (CP1): Civil Works</u>

The manufacture and construction of:

- Sheet pile walls (upstream training wall, transition wall, downstream training wall)
- Gated spillway civil structure - primary concrete works

- Energy dissipation structure works
- Overflow weir civil works
- Mass earthworks and engineered back filling where appropriate
- Scour protection
- Gate house slab, ready to receive the CP2 Contractors gate house.

Contract Package 2 (CP2): Mechanical Works

The fabrication, transport to site, erection and commissioning of:

- Radial gates (3 off)
- Embedded parts (3 sets)
- Hoist and gate trunnion beams
- Foot bridge
- Guard rails

The detailed design, fabrication, transport to site, erection and commissioning of:

- Gate hoisting system
- Gate house

Contract Package 3 (CP3): Electrical, Controls and Instrumentation Works

The detailed design, fabrication, transport to site, erection and commissioning of:

- Electrical offtake from the nearby 11kV network
- Gate house power supply
- Control system
- SCADA system

1.4 Project Workstreams

Figure 4 depicts the various project workstream interactions for the project:

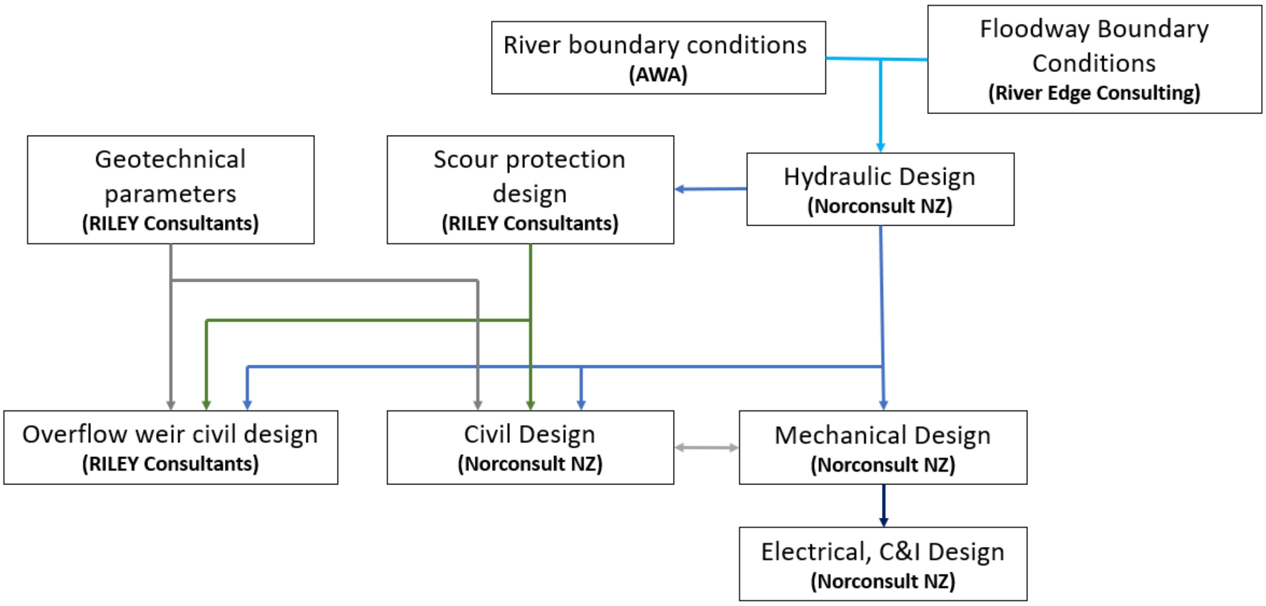


Figure 4: Project Workstream

2 Design Criteria Memorandum

The Design Criteria Memorandum (DCM) sets out the applicable industry standards comprising the general and particular design requirements, codes, standards, main references and assumptions applicable to the project and forms the basis for the detailed design. The DCM is a live document, periodically updated to reflect advancements in project definition, changes in scope and inclusion of new information.

A copy of the DCM, Revision D is included in Appendix A for reference.

3 Hydraulic Modelling

The objectives of the hydraulic detailed design modelling include the definition of geometric details for the spillway structures including the overflow weir and the gated structure. The requirements for energy dissipation at the terminus of each spillway structure forms part of the geometrical definition.

Norconsult has undertaken detailed numerical modelling using Flow3D software to establish the geometrical design required to meet BoPRC's design criteria and objectives for the Rangitāiki river spillway complex. Norconsult's design relies on key input data from other sources which will directly affect the spillway flow rates. The most significant of these external inputs include rating curves for the river at various cross-sections (by AWA) and for the floodway at the downstream boundary of Norconsult's model domain (by River Edge Consulting).

The typically accepted accuracy (computational margin) of fully 3D CFD modelling for spillway ratings when compared to good physical scale models of the same subject is +/- 5% in terms of flow rate.

The following subsections are taken from Norconsult (2022) with the intention of providing an overview of the hydraulic design process and outcomes for completeness. The reader is referred to the original text for a more detailed explanation and commentary.

3.1 Overflow Spillway

Development of the overflow spillway has progressed the preliminary engineering design (PED). The overflow weir and discharge system developed during the PED operated as a broad crested weir of 210m crest length at crest EL 5.90 m. The crest discharged to a 210m shallow sloping discharge channel that was assumed to be grass covered. Operation of the PED spillway could be described as "low-energy" since only a very limited portion of the crest passed through critical depth (i.e. a hydraulic jump).

The PED phase identified differences between the two, third party modelling agencies that provided input data with relevant ratings at the river surveyed cross sections. Table 15 of the PED Hydraulic Report[3] identifies there is a difference in river ratings between the River Edge[4] (base case) and Awa Environmental ('Awa'), with Awa modelling providing higher river levels. Data from the table identifies the river section critical to dictating headwater level for spillway discharge (downstream XS 36) has a level 83mm higher than the equivalent River Edge value. The PED report notes among other things, the sensitivity of the overflow weir to underestimation of river level and noted that 2D numerical modelling showed that if realised, an 83mm higher river level could increase overflow weir discharge by 31 m^3/s (+16%) over the targeted discharge of 190 m^3/s.

Early in the detailed design phase BoPRC reviewed the river rating and required Norconsult to adopt river rating data produced by Awa. A review of the updated rating for XS36 highlighted a +244mm river level difference from the base case adopted at PED. The detailed design phase testing of the original 210m length RL 5.90m elevation weir configuration using a 2D CFD simulation resulted in 330 m^3/s discharge of the overflow weir (i.e. +74% over the consented discharge). This issue resulted in a re-evaluation of the overflow weir arrangement as outlined below.

Noting the design criteria (including one amendment from the PED phase), the detailed design adopts:

- Nil (or near zero) discharge in a 0.05 AEP (1 in 20 year) flood, river flow at 437 m^3/s; and

[3] Doc number 1918-C-30-R001-1_A, Hydraulic Assessment - Rangitaiki Spillway Preliminary Engineering Design, Norconsult NZ Ltd, June 2021.
[4] Note River Edge Consulting Ltd is cited as 'Phil Wallace' in the PED report.

- capacity to pass 190 m^3/s in a 0.01 AEP (1 in 100 year) flood, river flow at 804 m^3/s; while
- the design review team removed any flow criteria related to 0.025 (1 in 40 year) flood

Increase in the rating for upstream cross section XS37 using Awa data required the overflow weir crest to be raised, noting the revised level for at XS37 for 437 m^3/s river flow is EL 5.93m. The design review team adopted a minimum overflow crest level of RL 6.0m, as this provides a margin on the 0.05 AEP flood level and is a convenient level for construction purposes.

Based on the revised weir crest level, the initial approach to achieving the 0.01AEP design discharge of 190 m^3/s was to reduce the overflow crest length. A crest length of 104 m was configured and achieved the discharge criteria, however the flow intensity (flow per metre of crest length) in conjunction with the higher headwater elevation resulted in a cross-wave pattern of hydraulic jumps randomly distributed over the runout channel downstream of the crest, i.e. the crest outflow regime was no longer generally subcritical and involved higher velocities than could be considered in the PED phase.

The design review team when incorporating comments from RILEY regarding erosion control, concluded that the overflow crest would require a concentrated form of energy dissipation to minimise the potential for erosion downstream of the weir within the spillway discharge channel. The result is a concrete surface for the weir crest. Norconsult has developed the necessary hydraulic profile for the weir and outflow channel on which RILEY is to base their design for the civil/structural works.

3.2 Gated spillway

The design condition for development of the gated spillway is 0.01 AEP (1 in 100 year) flood condition and the prevailing river level when the overflow weir is in operation, discharging 190 m^3/s. The gated structure must safely pass up to 40 m^3/s in the design condition.

The concept design and PED considered an articulated concrete block mattress (ACBM) immediately downstream of the gates structure (physically attached to the gates structure) to provide erosion protection and transition between the gates structure and the downstream channel. The detailed design of the downstream erosion works (by RILEY) adopts the use of riprap in preference of ACBM's due to concerns regarding availability in the New Zealand market.

Norconsult has undertaken a detailed CFD model simulation study to develop energy dissipation arrangements for the gates discharge to reduce and redistribute the energy of the flow to the downstream runout channel.

Development of the energy dissipator configuration is constrained by the following factors:

- The spillway is in low-lying terrain relative to the floodway
- Underlying ground conditions are of low quality in engineering/geological terms
- Supply of suitably large riprap (rock boulders) to the site is expensive and can be difficult to source[5]
- Articulated concrete block mattress (ACBM) erosion protection as included in the PED design is not readily available[6]
- BoPRC are aware and accepting of the potential need to undertake reinstatement of surfaces downstream of the spillways post operation, but spillways must be stable under design conditions[7].
- the applicable design ranges of standard USBR series energy dissipaters.

[5] Based on feedback from BoPRC.
[6] Consideration of the project design review group (BoPRC, Norconsult, RILEY).
[7] BoPRC comments in design phase and referenced in Design Review meeting 13 May 2022.

An assessment of requirements for energy dissipation options was undertaken, leading to the development of a bespoke dissipation structure as described in Section 3.2.1.

A 2D CFD simulation model was used to obtain headwater and tailwater levels for the spillway gates during the 0.01 AEP flood event following completion of the design of the gated spillway, overflow spillway and the energy dissipaters of each spillway; A 3D CFD simulation model was used to obtain a rating for the spillway gates.

3.2.1 Energy dissipation objectives

The arrangement should provide energy dissipation in conjunction with interruption of the moderately high velocities discharged from the spillway gates, such that the downstream channel erosion protection can be generally maintained during the design event. It is noted that:

- BoPRC have stated they would prefer an option to undertake future maintenance and remedial works downstream of the spillway structures after significant events, rather than design for every contingency in terms of erosion control as this could incur excessive construction-phase (capital) costs as previously noted.
- The characteristics and velocity distribution of discharge for the structure should be acceptable to the designer of the erosion protection works (Riley Consultants Ltd).
- Outputs from the CFD hydraulic modelling are provided as input to the erosion protection design via Appendix A of this report.

No specific criteria has been set down for the erosion protection design, however design review meetings have suggested subcritical flow conditions with depth-averaged velocities of no more than 2-3 m/s. The objective of the gates energy dissipation structures is therefore to reduce hydraulic energy and redistribute the velocity gradient to provide sufficiently calmed flow for the erosion protection design by Others.

The available energy for formation of a hydraulic jump is limited for the gated spillway structure which has an invert elevation of RL 4.5m, as noted above.

3.2.2 Gates discharge rating

The gated spillway discharge rating was established with a headwater level at the monitoring position of EL 7.07m, i.e. a head to the gate invert of 2.57m of thereabouts with the overflow weir passing 190 m^3/s and the gates passing 40 m^3/s. Prior to opening the gates and while the overflow weir is passing 190 m^3/s, the water level at the monitoring position was found to be EL 7.15 m. This leads to the point that when the gates are opened, water levels along the river adjust to develop the new equilibrium. The rating presented assumes a fixed water level in the river downstream of the overflow spillway, whereas in flood operation river level may be varying with changes in river flow and/or in response to gate operation.

The rating curve in was undertaken by iteratively repeating the model simulation while increasing the gate opening in the increments as plotted in the figure. All gates were operated at the same opening and each opening increment was run to steady-state condition. The river boundary conditions were not adjusted between simulations and prior to operation of the spillway gates the overflow spillway was discharging nominally 190 m^3/s. Key visual observations were made to:

- Check the hydraulic jump to ensure this was well clear of the gates for all openings
- Ensure the gate trunnions have good freeboard under all operating conditions

At river EL 7.07m a gate opening of 0.9m for all gates achieves the design discharge criteria of 40 m^3/s.

Note is made of the location of the hydraulic jump downstream of the gates as there is a requirement to ensure the hydraulic jump does not impact the rear of the gates to prevent gate vibration and oscillatory loadings on the gate elements.

It was found that unstable flow conditions develop at gate openings above 1.5m, meaning that the limit of hydraulic control that can be provided by the gates is between 0 and 57 m^3/s at the stated river level. If additional flow requires to be passed into the floodway, it is recommended to fully open the gates to avoid hydrodynamic vibrations being introduced onto the gate leaf.

Note: this is considered to be an emergency discharge and is beyond the design discharge criteria for the gated spillway. A maximum discharge of 67 m^3/s resulted from simulating all gates fully open at the above river level[8].

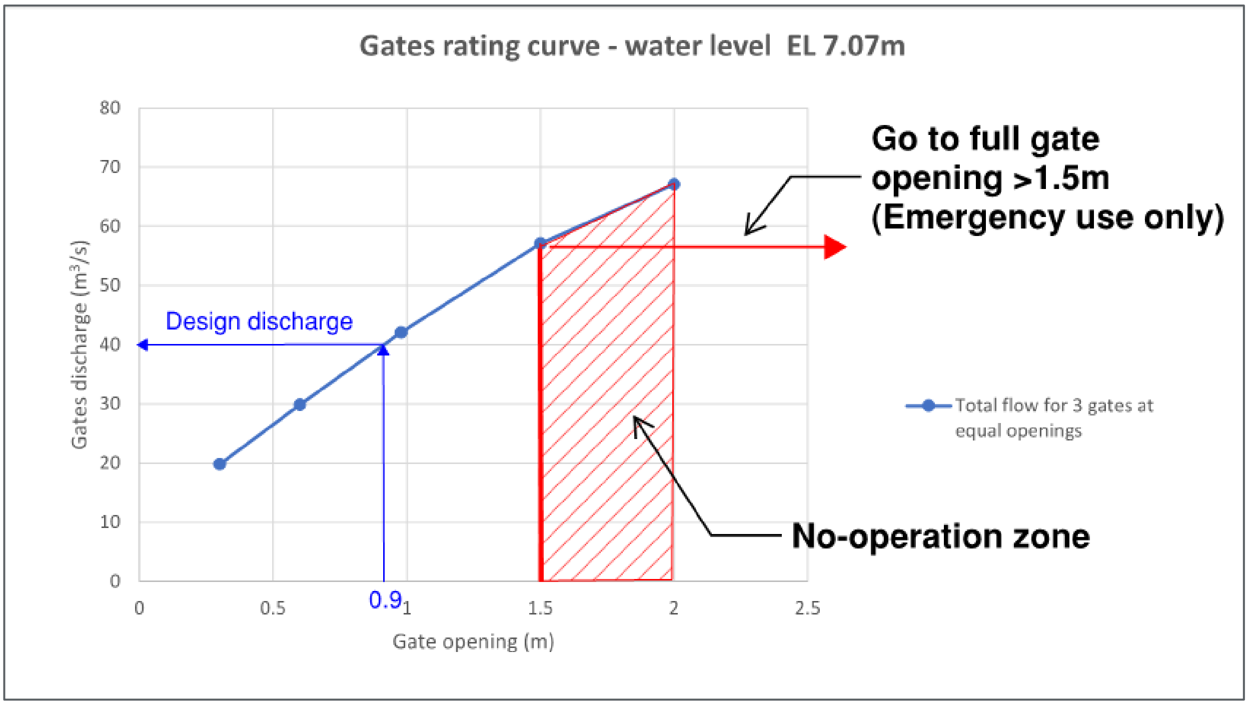


Figure 5: Gates rating curve, 3-gate parallel operation at river level monitor EL 7.07m

It is noted that the 2m opening position conforms to the fully-open position as hydraulic control of flow has been lost at this gate opening. The reason that openings greater than 1.5m (i.e. full-gate opening) is marked emergency use only is that there is reduced stilling basin effectiveness and the downstream riprap and runout channel lining may incur a greater degree of damage.

As the spillway gates have been designed as additional (contingency) spillway capacity, the design arrangement has focussed on a head range commensurate with 0.01AEP flood conditions. The rating equation and the characteristics of the stilling basin operation is expected to apply in the range of river levels EL 6.9 m to EL 7.2 m. It is noted that aside from the condition discussed in Section 3.2.3, other heads have not been assessed.

[8] The section below on Combined Operations discusses the influence of gate operation in reducing the river level and thus reducing discharge via the overflow weir.

3.2.3 Assessment of low-head gate operations

The gated spillway is designed to operate at the upper range of head prevalent at in the 0.01AEP flood design event, i.e., nominally at a river level of EL 7.0 m registered by the gates water level recorder. This is due to the restriction to flow passage through the energy dissipater, which will control the rate of flow instead of the gates, if the gates are opened at low upstream river level.

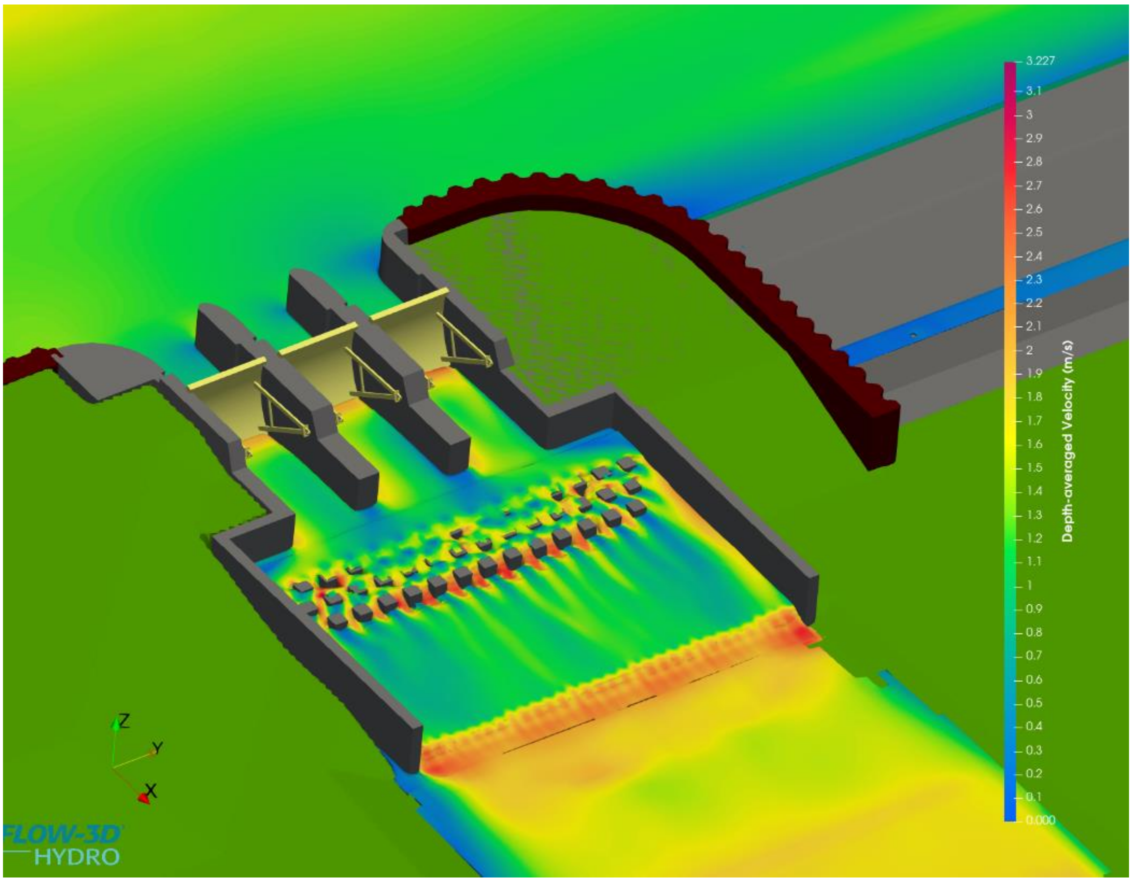
One example was generated using a fully 3D CFD model simulation as a check at 0.05AEP flood condition with three gates at 400mm opening. Modular flow is not achieved at this opening and the overall discharge control is the end sill of the stilling basin. The predicted gates discharge in this condition is 12.3 m^3/s and accordingly, there is minimal advantage in operating the gates for flow diversion purposes. Furthermore, there will be no tailwater generated by the overflow spillway, therefore the runout channel downstream of the gates would be directly exposed to the gate flow.

If the gates are opened for trial or training reasons, close observation of the gate flow and the downstream channel will be required to ensure:

- Modular flow exists at the gates, or the gates are lifted clear of flow; and
- Damage is not occurring to the runout channel downstream of the gates

Any low-head operations prior to the pooling upstream of and overtopping of Hydro Road should be undertaken cautiously and with close observation of the gates through to the downstream flow path.

Figure 6: 0.05AEP Flood with 400mm gate opening (DAV)



3.3 Combined Spillways Operation

Flow through the spillways lowers the downstream river level. Passing flow to the floodway using the gates will tend to have the following effects, assuming no blockage at the overflow spillway:

- Increase total spillway flows
- Reduction in downstream river levels
- Reduction in the amount of flow via the overflow spillway
- Increase of flows along floodway, with a corresponding increase in floodway water levels

Alternatively, if there is a blockage or insufficient flow via the overflow spillway causing the river to remain at a high level, opening the gates to pass flow to the floodway will tend to have similar effects to those listed above, albeit that the objective will be to lower the river level to meet the required downstream flood level parameters, ensuring the targeted 190 m^3/s is diverted from the river to the floodway.

3.4 Combined Spillways Model Downstream Boundary

A rating curve was adopted as the downstream boundary of the combined spillways model, noting this model boundary was set 225 m downstream of XS37 as shown in Figure 7. There is a notable upward swing in the rating curve at about 605 m^3/s and the river level rises abruptly[9] by 53mm at XS36 between river flows of 605 and 609 m^3/s which correspond to river levels of 6.97 and 7.04 EL m respectively.

The overflow spillway is sensitive to relatively small variations in head, noting that the overflow spillway flow increases by some 14 m^3/s while the downstream river flow increases by only 4 m^3/s. This ratio means that achieving an exact water balance in a fully 3D simulation model is extremely difficult because oscillation can easily occur in the model particularly as the steep portion of the river rating curve occurs near the design operating level of the overflow spillway. For this reason, the combined rating curve is undertaken on a manual calculation basis.

[9] This is a separate issue to the hysteresis in the rating data between rising and falling stages of the rating, and is present in all the rating curves provided by Awa (2021), i.e. in XS32 to XS39.

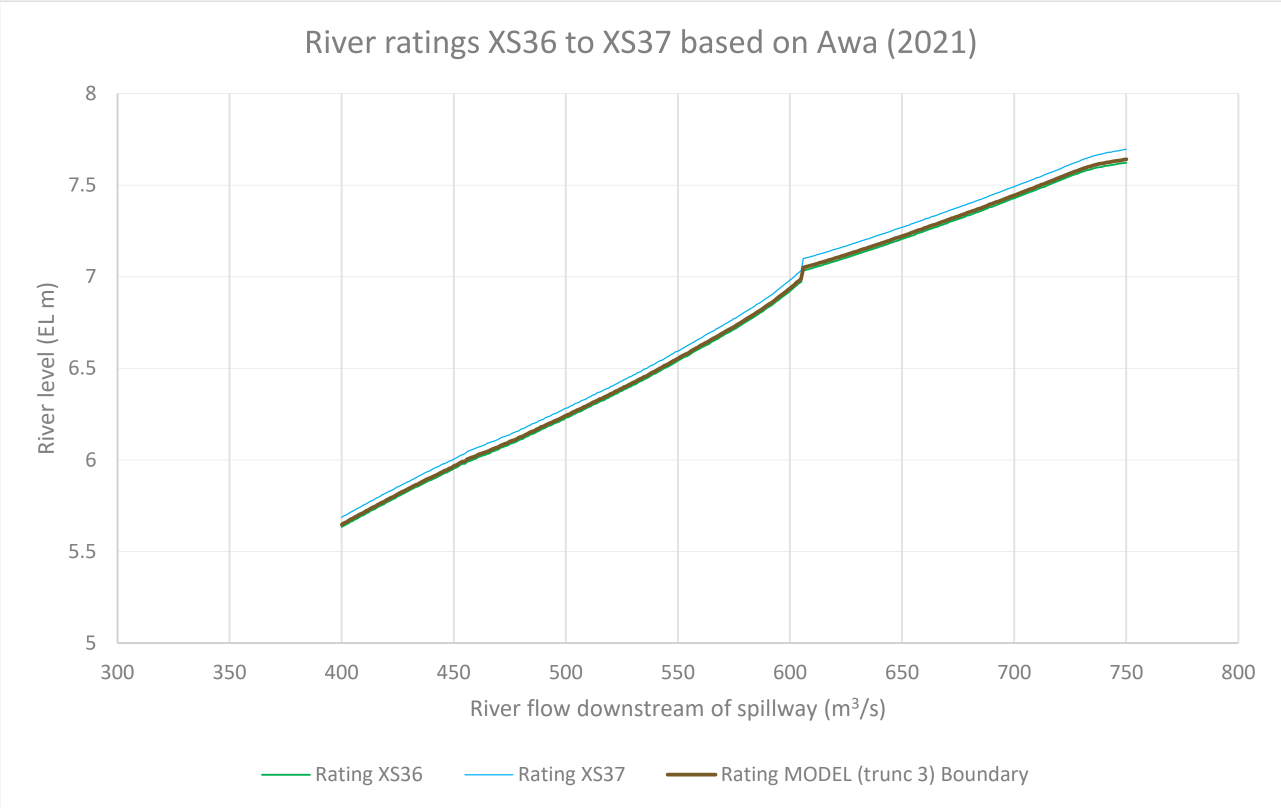


Figure 7: Downstream model boundary rating (combined spillways model)

3.5 Non-blockage Combined Operation of Spillways

Gated spillway discharge at the design discharge is insensitive to minor changes in river level in comparison to the overflow spillway. When the gated spillway is operated, the total flow will increase, but there will also be some decrease in the overflow spillway flow for any substantial operation of the spillway gates.

2D (river) and 3D (spillways) hybrid CFD model simulations were undertaken to assess the combined spillways discharge when the gates are additionally opened to release 40 m^3/s and to assess the maximum spill flows (i.e., if the gates are fully opened) in the 0.01AEP flood event. While exact water balance was not resolved in these models due to the effect of minor surging of water, the model simulations confirmed the principle of operation of the combined spillways.

Hand calculations were used to develop the relationship between river flow and overflow spillway flow without gate operation as presented in Figure 8: Overflow spillway operation (gates closed)

, for reference. In Figure 8: Overflow spillway operation (gates closed)

, the effect of opening the spillway gates is charted to represent the effect of opening the spillway gates (progressively) to 900mm as the 0.01AEP flood level rises to nominally EL 7.0 m. Note that the river levels in both Figure 7 and Figure 9 are based on the model boundary rating curve rather than the water level recorder position (upstream near gatehouse), because continuous rating data has not been developed for the latter location. Translating the data to the water level recorder position, the water level is expected to be nominally EL 7.16 m with spillway gates closed (applies to Figure 8: Overflow spillway operation (gates closed)

) and EL 7.03 m to 7.07m with the spillway gates open (applies to Figure 8: Overflow spillway operation (gates closed)

).

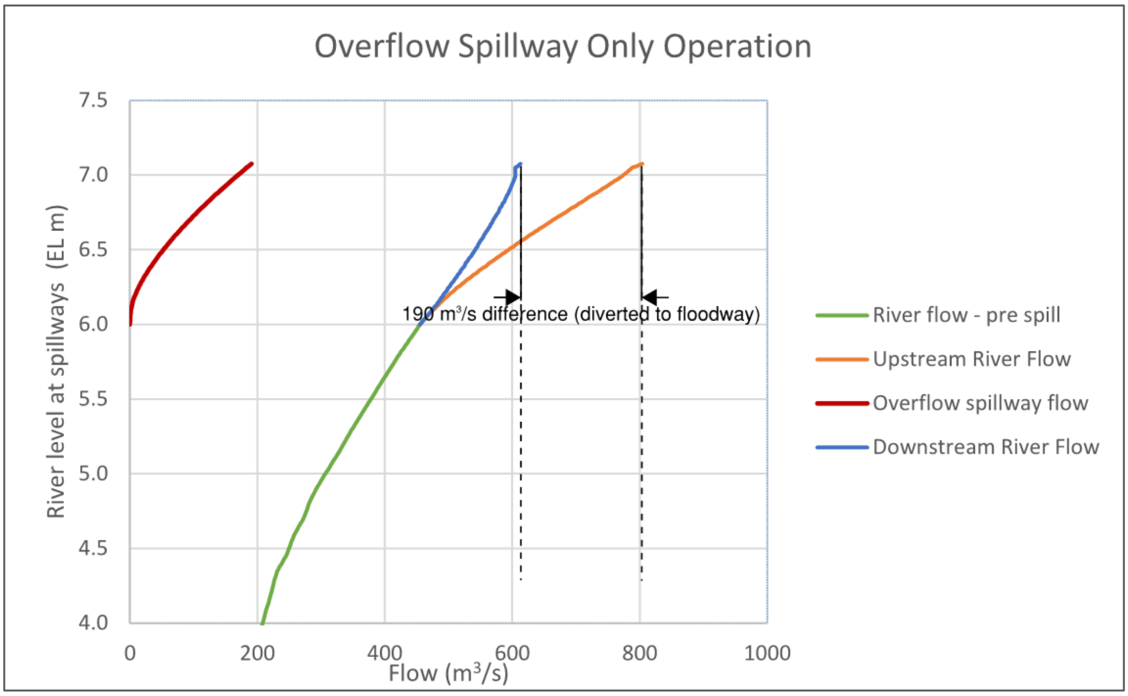


Figure 8: Overflow spillway operation (gates closed)

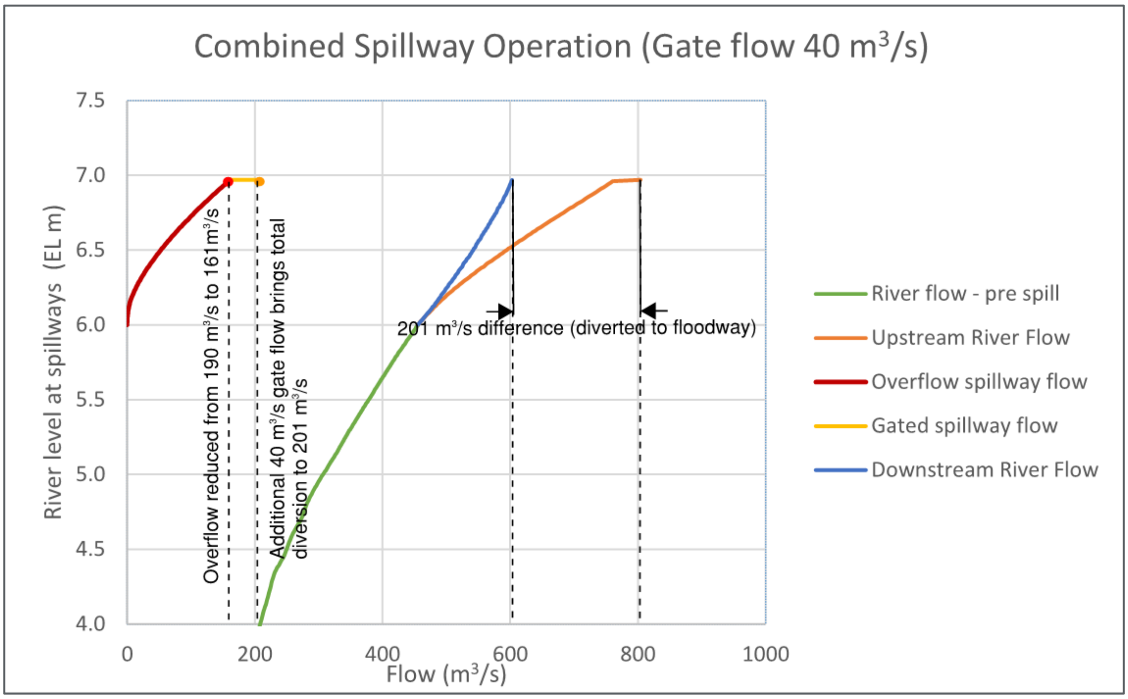


Figure 9: Combined spillway operation at gates design discharge

When the spillway gates are additionally opened to full gate opening, the total spill reaches up to a maximum of 210 m³/s. This scenario is represented in **Error! Reference source not found.**.

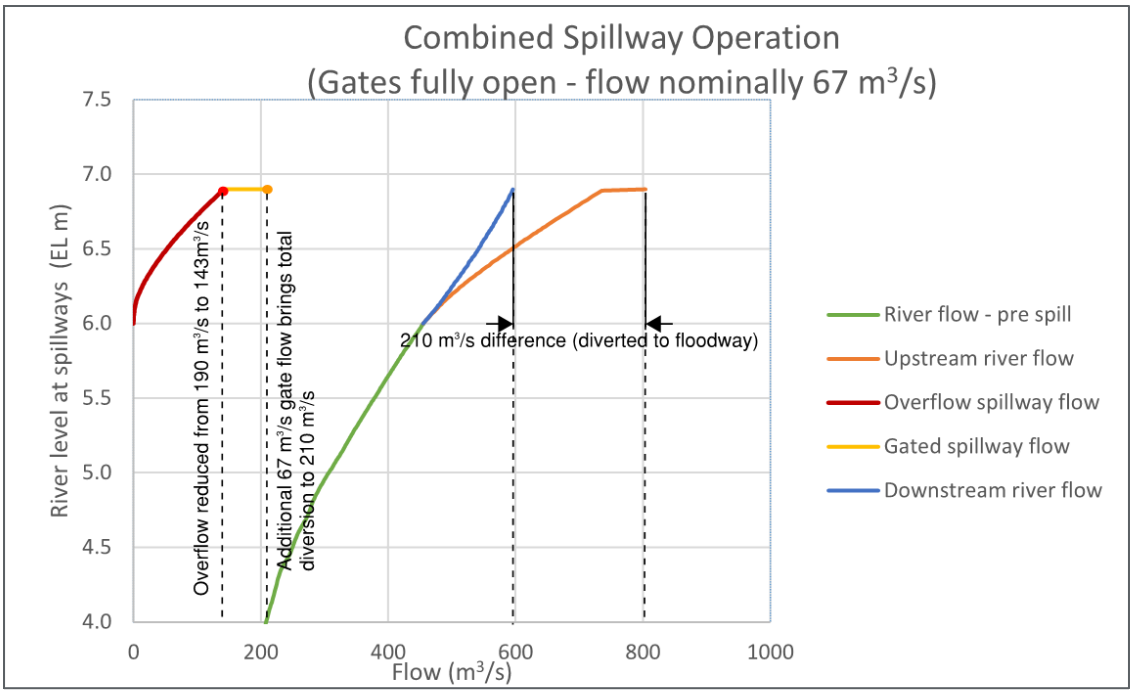


Figure 10: Combined spillway operation at gates fully open

Figure 10 highlights that the difference in total spillway capacity between the spillway gates releasing the design capacity of 40 m³/s to releasing 67 m³/s when fully open is only an additional 9.0 m³/s, because the overflow spillway discharge is reduced as the downstream river declines (no blockage conditions). By this mechanism there is a natural limit to the total flow that can be released via the combined operation of the overflow and gates spillways. Comparing Figure 9 and Figure 10, increasing the gates discharge swings the distribution of flows from the overflow spillway over to the gated spillway.

Note that full opening of the spillway gates can increase the risk of damage to the outflow spillway channel riprap and lining.

3.6 Partial Blockage of Overflow Spillway

The primary function of the gated spillway is to ensure that 190m³/s can be achieved even if the overflow spillway is partially impaired by debris, sedimentation influences or other forms of blockage. In an overflow spillway blockage scenario, the river is expected to stay at a high level (determined by the total flow that is released to the floodway).

Figure 11 presents a modification to the overflow spillway discharge rating curve that as a hypothetical 21% blockage (blockage 40 m³/s /190 m³/s).

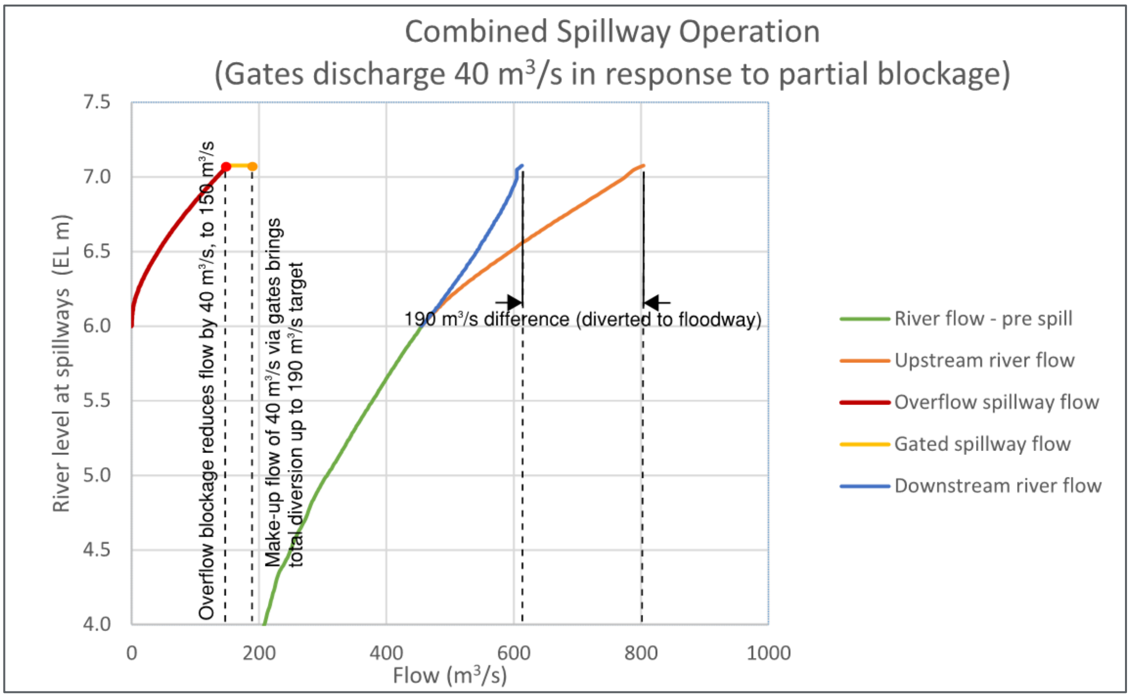


Figure 11: Overflow Spillway partial blockage scenario (40 m^3/s make-up flow via gates)

It is highlighted that in the spillway blockage scenario depicted in Figure 11, the river level remains similar to that shown in Figure 8: Overflow spillway operation (gates closed)

, because the downstream river flow is the same in these cases.

3.7 Recommendations

Norconsult has undertaken detailed numerical modelling to establish the geometrical design required to meet BoPRC's criteria and objectives for the Rangitaiki river spillway complex. Norconsult's design required key input data from other sources on which it is dependent in terms of outcomes such as the flow rates that will be achieved by the spillway at the design operating condition. The most significant of these external inputs include rating curves for the river at various cross-sections and for the floodway at the downstream boundary of Norconsult's model domain, noting each of these was sourced from differing external sources.

The typically accepted accuracy (computational margin) of fully 3D CFD modelling for spillway ratings when compared to good physical scale models of the same subject is +/- 5% in terms of flow rate. It is recommended that this value is used as a baseline, and it is further noted that the other parameters can additionally influence the calculation margin.

3.7.1 Confirmations performed via the macro hydraulic models:

Norconsult's modelling has incorporated a limited domain simulation model at a detailed fully 3D level, specific to the spillway location. To minimise the risk of the overflow spillway capacity being under or over predicted and to minimise the risk that floodway levels are under-predicted, the following tasks (by others) are recommended. This is to ensure that interaction of the spillways systems as developed are fully understood in terms of the responses of the river and the floodway, including identification of any related risks and confirmation of gate operational practice.

- Awa Environmental Ltd should undertake additional modelling of the 0.01 AEP (1 in 100 year) flood event. This work would incorporate the final spillway width and stepped-crest geometric design and is primarily focussed on verifying the design works as intended within the overall context of the river and floodway
- The satisfactory operation of the spillway in relation to the sudden rise in the spillway rating curve near river EL 7.0 m should be checked as part of this work.
- As part of the Awa modelling, it is recommended that operation of the gated spillway is included. The primary reason for this is to identify the time duration between gate operation and the river responding to a steady-state condition. This information should be developed to inform the BoPRC operator of the necessary waiting time between gate movements required to know the result of the movement (prior to making further adjustments. This modelling should also be used to check for floodway risks.
- Roughness of erosion protected areas provided by RILEY should be confirmed and reviewed in terms of the range that Norconsult has trialled within this report (refer Section **Error! Reference source not found.**).
- Confirmation of floodway performance by River Edge in terms of the most recent available survey data, the nominal discharge of the overflow spillway (190 m^3/s), the nominal discharge of combined operation of the overflow and gated spillways (recommended at 210 m^3/s + 10% margin) to confirm freeboard to stop banks, private property and accessways to private property. This work should incorporate the possible range of roughness of the floodway.

3.7.2 Erosion Protection Design

- Data provided by Norconsult as input to the civil designer for erosion protection design is provided in terms of flow depth and depth-averaged velocities (Appendix A refers), and will require appropriate design factors to be applied to account for typical variation between average and peak velocities at hydraulic surfaces.

3.7.3 River Monitoring and Surface Maintenance

- The existing process of period surveying of river hydrological cross-sections provides important data and should be continued with data reviewed by hydraulic engineer to assess for any adverse impacts on operation of the spillway scheme.
- The river within the stop banks should be well maintained into the future to avoid significant vegetation that could compromise roughness and adversely impact on flood levels and operating level at the overflow weir. The validity of the present design is dependent on the roughness as modelled by Awa, being maintained. This recommendation applies especially from the vicinity of XS39 upstream of the spillways complex to the river downstream.
- To maintain the design capacity of the spillways, the constructed spillway forebay/approach and outflow channels should be continuously maintained within the range of roughness parameters tested in this report (noting these are subject to further confirmation by RILEY).

3.7.4 Overflow Spillway – Vertical Construction Tolerance

- Construction tolerances for the overflow spillway crest levels should be set at the minimum range that can be practically constructed. Spillway discharge is sensitive to any increase of overflow head and excessive discharge is not controllable. Therefore, specification of crest construction tolerance should be given in positive (upward) direction only, with stated crest elevation on Norconsult drawings being adopted as the minimum allowable.

3.7.5 Overflow Spillway – Monitoring and Control of Settlement

- Since excessive settlement of the crest could impair control of spillway discharge, this aspect requires control at the design and construction phase and monitoring during the life of the spillway.
- It is recommended that spillway level checks be included in the regular surveying of river hydrological cross-sections.
- Design of the concrete crest should consider the possibility for future build-up of the concrete crest if this is required in response to settlement. A suitably qualified and experienced hydraulic engineer should be involved prior to any such undertaking. As identified in Section **Error! Reference source not found.**, an average settlement along the crest of 32 mm could result in an additional 5% discharge of the overflow crest in a 0.01AEP flood. The sensitivity of the floodway to additional flow may have a bearing on the amount of settlement that is permissible before a correction (build-up) of the crest is required.

3.7.6 Overflow Spillway Runout Channel Drainage

- The overflow runout spillway channel is formed as a level bench at RL 5.0 m to limit runoff velocities, on the assumption this area could be artificially drained. This aspect requires consideration when developing erosion protection in relation to the potential for sodden conditions, particularly in the winter season.

3.7.7 Gated Spillway Operation

- The key function of the spillway gates is to assure discharge of 190 m^3/s in total can be achieved, should function of the overflow spillway be limited for some reason.
- The gates should be operated symmetrically unless there is a problem with a gate, preventing its operation.
- The gates should not be opened to discharge greater than the design rate of 40 m^3/s total discharge unless it is an emergency, as there is a risk that higher discharges could result in damage to the runout channel lining (riprap and other erosion protection linings).

3.7.8 Monitoring of Control System Water Level Sensor

The water level sensor is a critical item of equipment as it provides the spillway operator with an input that may be used to estimate overflow weir and gate discharge in association with the rating curve equation for each of these.

- Regular inspection and clearance of debris from the water inlet should be undertaken, along with checks the water level transducer and related systems are working

3.7.9 Monitoring of Spillway Operations

By design, spillway systems including the gated spillway will be operated infrequently, noting the overflow spillway will not operate unless the river flow exceeds a 0.05AEP (1 in 20 year) flood and by design the spillway gates will not be required to operate unless the river flow reaches a 0.01AEP (1 in 100 year) flood[10].

- It is recommended that the flow data from the upstream Te Toki and downstream SH2 gauges be provided as inputs into the gates control system, so that these can be directly accessed by the BoPRC operator to assist with decision making.

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[10] BoPRC may choose to exercise the spillway gates and utilise the floodway to pass flow (subject to consent conditions) at a lesser flood.

- Observation and recording of any spillway operation should be undertaken as it will likely provide information of importance to train operators and to review spillway performance with engineers.
- Use any flood operational data obtained to validate the spillway rating curves, to ensure the as-built spillway systems operate as expected.

3.7.10 Post-operation Inspection and Maintenance

- It is highlighted that every flood operation of the overflow or gated spillway should be followed up by inspection for damage.
- In any significant spillway operation, some damage to riprap protection or other erosion protection could potentially and Norconsult highlights allowances should be considered for future maintenance of these items[11].

Spillway flows must cross Hydro Road to enter the floodway. Hydro Road forms a hydraulic control and is therefore subject to a head differential and some elevated velocities, in particular on its Eastern side. BoPRC has expressed its awareness of the need to control public access along the Hydro Road in flood conditions where the spillway could potentially operate[12]. Similarly, BoPRC is aware there could be damage to the road following a spill to the floodway[31].

- The condition of Hydro Road should be specifically inspected by engineers following each and every operation of the spillways and any necessary repairs to the road effected
- Should Hydro Road be realigned or culverted at any time in the future, the design phase should involve close hydraulic assessment in relation to the impact of spillway operations on the road arrangements and should also assess any impacts on floodway hydraulics.

[11] Norconsult acknowledges erosion protection is for design by others, and this statement is made because the exact behaviour of the flow in all locations downstream of the spillway structures can potentially vary from model results, thus specific impact on erosion protection linings can vary.
[12] General discussion, project design review meetings.

4 Site Geology

Norconsults civil works design is based on site geotechnical parameters provided by Riley (2021)[13].

The following excerpt is taken from the aforementioned report, for the readers understanding of site conditions.

> *"Published regional-scale geological mapping for the area indicates that the site is underlain by Tauranga Group swamp deposits (Q1a), comprising dark brown to black peat, organic-rich mud, silt, and sand.*
>
> *The [gated spillway] concrete structure is around 250m north of the active Edgecumbe Fault, with an active splay of the Edgecumbe Fault approximately 130m to the north. These ruptured at the surface during the 1987 M6.5 Edgecumbe Earthquake. The GNS Science Active Fault Database indicates that the Edgecumbe Fault has a recurrence interval of less than 2000 years."*

Rileys work also included a site investigation comprising of three new test pits near the gated structure location, the results of which are reproduced in the below table.

Table 4-1: Summary of Test pits

Test Pit ID	Location	Total Depth (m)	Soil Units encountered	
			Depth (m)	Simplified Description
TP01	10m upstream, 1m beneath crest of existing stopbank	4.90	0.0 – 0.9	FILL
			0.9 – 4.6	SAND and SILT mixtures. Local gravel and organics.
			4.6 – 4.9	SAND
TP02	10m downstream, at existing accessway	4.20	0.0 – 1.3	FILL
			1.3 – 4.2	SAND and SILT mixtures. Local gravel and organics.
TP03	Top of existing stopbank, footprint of concrete gated structure	0.30	0.0 – 0.3	FILL

The reader is referred to the RILEY (2021) for further information.

[13] "Rangitaiki Spillway Geotechnical Report", RILEY Consultants, Issue 1, 28 September 2021.

5 Salient Project Features

Salient features of the spillway project designed by Norconsult are presented below in Figure 12 below and on Drawing D011.

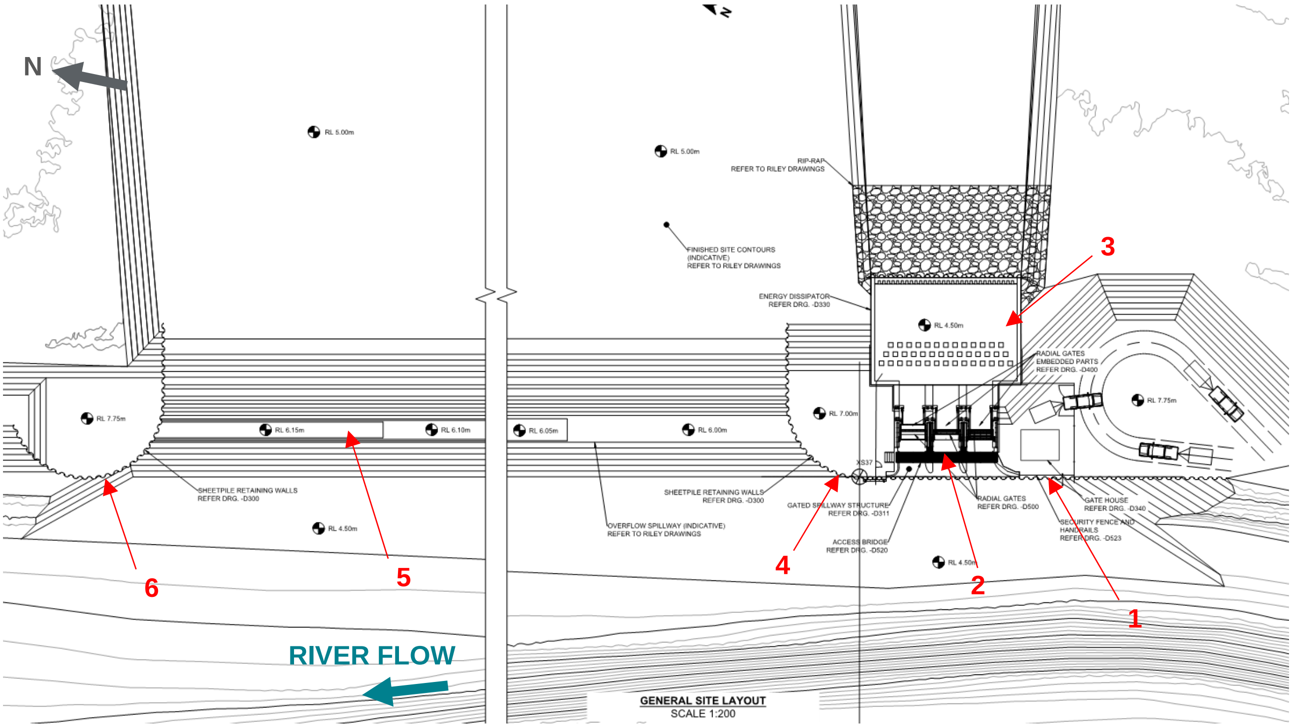

Figure 12: Salient project features

The salient features of the spillway upgrade works comprise the following (in order of upstream to downstream):

1. An upstream, sheet pile training wall, providing scour protection of the bank and assisting the conveyance of flood water into the gated spillway structure.
2. A new gated spillway structure, comprising
    - Three off identical radial gates
    - Hydraulic hoisting system for each gate
    - A gate house for the protection of the hoisting systems Hydraulic Pressure Unit (HPU), electricity distribution, control and SCADA equipment
3. An energy dissipator downstream of the gated structure, to dissipate energy release from the gate discharge and prevent undue scour downstream of the structure
4. A sheet pile transition wall immediately downstream of the new gated structure, to assist in the guidance of water past the gated structure (when not in use) and over the fixed-crest, side-discharge overflow weir
5. A new lowered fixed-crest, side-discharge overflow spillway (weir), approximately 114m in length with nominal crest level set at RL6.0m stepping up to RL 6.15m in four equal steps.
6. A sheet pile transition wall immediately downstream of the overflow weir, transitioning to the existing stopbank.

6 Sheet Pile Walls

6.1 Introduction

Sheet piles have been selected to:

- retain the embankment, and;
- Guide water into the gated structure and overflow spillway, and;
- to provide scour protection at the upstream end of the gated spillway and downstream end of the energy dissipator.

The sheet pile retaining walls on the upstream side of the gated structure, in front of the gated spillway and the transition wall are continuous for scour protection reasons (sections 1, 2 & 3 on drg D300). Site geological conditions required a driven retaining solution, rather than a more traditional reinforced concrete design resulting in a simpler construction and cost effective solution compared to other alternative retaining wall options.

The sheet pile wall upstream of the gated spillway (section 1) has a maximum retained height of 3.8m (assuming a non-effective depth of 0.5m). Section 1 includes a tie-back system consisting of tie-rods, waler beams and a continuous anchor sheet pile wall given its height (i.e. a cantilever wall is not feasible at this height).

The short straight section immediately downstream of the gated structure (section 3A) is also propped to limit deflections where it connects to the gated structure abutment. All other sheet pile wall sections are cantilever walls (i.e., unpropped).

The sheet pile walls leading into the fixed overflow weir at both ends (sections 3 & 5) have a radius of 10 m. The interlock swing required to achieve this radius is less than the typical maximum interlock swing angle of 5^{o}.

The sheet pile wall calculations are included in Appendix B.1. Details are shown on drawings D300 to D305 in Appendix D.

6.2 Corrosion Allowance

A design life of 50 years has been selected for the sheet pile walls. The design allows for loss of material due to corrosion over this period as no corrosion protection system is proposed for the sheet piles. A corrosion allowance of 1.0mm per steel surface has been adopted based on SNZ TS 3404 and the following corrosivity categories or exposure classifications. Corrosion rates at the upper end of the range were conservatively selected.

<u>Steelwork in contact with air</u>

- Surface-specific atmospheric corrosivity category: **C4** (based on external and wet for C2 (Low) macroclimate corrosion category)
- Atmospheric corrosion rate = 0.016 mm/steel surface/year

<u>Steelwork in contact with soil</u>

- Exposure classification for steel in soil: **Mild** (based on soil pH of 4 to 5 in soil condition B (high permeability soils that are in groundwater or all soils above groundwater))
- Corrosion allowance = 0.02 mm/steel surface/year

Multiplying the larger corrosion allowance of 0.02 mm per steel surface per year by 50 years gives 1.0 mm per steel surface for the design life.

No soil testing has been carried out to confirm the “Mild” exposure classification for steel in soil. This is however the same as the classification adopted for the East Bank Road stopbank sheet pile cut-off wall on the “land side”. At East Bank Road a “Severe” exposure classification was selected for the “river side” as the wall is approximately 1 km from the coast and retains brackish water. As the Rangitaiki spillway is located 11 km from the coast (in a straight line) the assumption is that brackish water is not present and a “Mild” exposure classification is appropriate for both sides of the sheet pile wall.

6.3 Pile Type

The design of the sheet pile walls is based on a U-type hot rolled SP3W profile with a yield strength of 390 MPa. This profile is common in New Zealand and has been used by BoPRC on other stopbank projects. The profile is 600 mm wide with a flange thickness of 13.4 mm and a web thickness of 9 to 10 mm.

Other similar profiles are available in New Zealand and could be considered by the Contractor. Z-type sheet pile profiles are also an option and have a higher capacity for the same cross section, as the interlocks are at the flange (rather than the centre of the web). Accordingly, no reduction is required to account for a possible lack of shear force transmission (EN 1993-5 cl. 5.2.2).

Regardless of the profile selected, a minimum steel thickness of 10 mm is specified to account for localised pitting corrosion (SNZ TS 3404 cl 2.5).

The ultimate limit state shear and moment resistance of the SP3W sheet pile profile was calculated in accordance with EN 1993-5 together with the UK National Annex.

Interlock reduction factors for the U-shaped sheet pile were selected from Table NA.2 of the UK National Annex assuming ‘Unfavourable conditions’ with enhancement achieved by providing a 300 mm long structural weld (minimum 6 mm fillet) near the top.

The design shear and moment resistances with and without the corrosion allowance of 1.0mm per steel surface are included below in Table 6-1.

Table 6-1: Design Resistance of SP3W Sheet Pile

	Without corrosion allowance	With corrosion allowance (estimate)
Design plastic shear resistance ($V_{pl,Rd}$)	1002 kN/m	784 kN/m
Design moment resistance ($M_{c,Rd}$)	508 kNm/m	426 kNm/m

6.4 Pile Length

The required sheet pile embedment depth and associated shear forces, bending moments and prop forces (where applicable) were determined in accordance with the methodology prescribed in the DCM and are summarised in Table . The design shear and moment design actions are less than the design resistance in Table 6-1.

Table 6-2: Summary of Sheet Pile Wall Embedment Depths & Design Actions

Sheet pile wall section	Critical case	Maximum retained height (m)	Embedment depth	Design shear, V* (kN/m)	Design moment, M* (kNm/m)	Design prop force (kN/m)
1	ULS Case 3: Flood	3.8[1.]	4.8m (RL -0.80m)	98	174	103
2	Base of sheet pile wall section 2 aligned with sheet pile wall section 1					
3 & 5	ULS Case 2: EQ	3.0	6.9m (RL -2.90m)	196	225	-
4	ULS Case 3: Flood	1.1[2.]	3.0m (RL 0.4m)	29	13	-

1. Including a non-effective depth allowance of 0.5m for scour or erosion protection material
2. Retained height corresponds to depth of erosion protection below top of energy dissipator base slab

The sheet pile wall at the upstream end of the gated spillway (section 2) extends to the same depth as the adjacent sheet pile wall section 1. This wall functions as both a scour protection wall and a seepage barrier.

The sheet pile wall at the downstream end of the energy dissipator (section 4) is designed on the basis that no lateral support is provided by the rip rap erosion protection apron (by RILEY) which has a depth of 1.1m below the top of the energy dissipator slab. As the scour protection requirements for section 2 (upstream end) are less than section 4 (downstream end) the embedment depth of section 2 (which exceeds section 4) is more than adequate for scour protection.

The design of sheet pile wall sections 3 and 5 assume that the retained material is engineered to limit differential groundwater levels due to rapid drawdown following a flood event to 0.5m. This is necessary as props are difficult to accommodate and significant differential groundwater pressure as defined in the DCM for the flood case (groundwater on retained side at 1/100 AEP level) are not tolerable for a maximum cantilever retained height of 3.0 m in the expected soil conditions. For a groundwater differential of 0.5m the required embedment and design actions are similar to ULS Case 2: EQ.

As the retained height of sheet pile wall sections 3 and 5 are typically less than the maximum retained height of 3.0 m, an embedment depth that varies depending on the retained height could be considered. For example the embedment depth required for a retained height of 2.0m is about two meters less than the embedment required for a retained height of 3.0 m.

The sheet pile wall upstream of the gated spillway (section 1) extends 0.5m above finished ground level for additional freeboard when the river is high. A fence or barrier along this edge is required for fall protection.

The top of sheet pile wall section 3 extends to the same height as the upstream section however as the retained finished ground level here is 0.8m lower the sheet pile wall also provides adequate fall protection (1.25m above finished ground level).

6.5 Tie-back System

The waler beams used in Sections 1 and 3A consist of a pair of 250 PFC's connected at the tie locations with a 200x100x6 RHS. Three pieces are proposed for the upstream wall and are joined with a bolted 200 PFC splice. The waler beams are bolted to the outside flange of the sheet pile walls and drainage holes are provided in the top PFC.

The Macalloy TB460 System is shown on the drawings and includes an M42 tendon with turnbuckle and couplers as required. Spherical nuts and washers at both ends of the tie mitigate the effects of possible bending in the anchor. This is good practice and based on EN 1993-5 cl 7.2.3 allows a higher tendon capacity to be adopted.

The anchor sheet pile wall and waler arrangement is similar to the retaining wall. These have been sized and are located a sufficient distance from the retaining wall to allow full development of passive pressures.

While a corrosion loss of 1.0 mm per steel surface may be tolerable for the tie back system, corrosion protection is recommended for these critical components. The specification includes a high build epoxy paint system for the walers, Denso tape for the tie-rods and Radolid caps with Denso void filler for the spherical nuts and washers.

Design of the tie-back system considers loss of a support anchor and prevention of progressive collapse as described in the Design Criteria Memorandum.

6.6 Interface with Gated Spillway and Energy Dissipator

According to the Rangitaiki Spillway Geotechnical Report (Riley 2022) the predicted settlement of the concrete structure could be 45 mm at 50 years with 30 mm occurring in the first month.

To accommodate potential differential settlement between the gated spillway structure and the sheet pile walls the design includes the following details:

- The top of the sheet pile scour protection walls at both the upstream and downstream ends of the concrete structure (sections 2 & 4) are set 100 mm below the top of the concrete. A bond break consisting of a polythene sheet between the concrete and the sheet piles is also specified to allow the concrete to settle relative to the sheet pile wall.
- The sheet pile wall is attached to the face of both abutments at the upstream end with chemical anchors. 100 mm long slots are provided in the sheet pile at these locations to also allow the concrete to settle relative to the sheet pile wall.

It is possible that gaps will form between the concrete and sheet pile walls due to a combination of settlement and concrete shrinkage. To minimise seepage through significant gaps it is likely that a sealant will need to be installed. The drawings indicate that the concrete / sheet pile wall interface is to be checked with the Engineer at least 30 days after completion of the spillway and energy dissipator concrete works (after which most of the settlement is expected to have occurred) and that where necessary gaps are sealed in accordance with the methodology agreed with the Engineer. Ongoing periodic monitoring of these joints will be required.

7 Gated Spillway & Energy Dissipator

7.1 Civil Structure

7.1.1 Introduction

The gated spillway structure is typically 15.2 m wide by 12 m long with 3No. 3.4m wide gate bays. It has:

- a 650 mm thick base with an additional 250 mm thickening below the gate sill
- 1000 mm thick piers with hydraulic profiling at the upstream end that vary in height from 2.5 m to 3.65 m.
- abutment walls 1000 mm thick at the base with a 1H to 10V batter on the outside face. The batter is a requirement of BoPRC (2021)[14] to ensure a firm contact with the fill. The abutments also vary in height from 2.5 m to 3.64 m.

The geometry of the structure is largely governed by hydraulic design and profiling requirements and minimum practical widths and thicknesses required to accommodate stoplog slots (200 mm deep by 500 mm wide), gate embedded parts and the cylinder and gate trunnion beams.

The energy dissipator structure is immediately downstream of the gated spillway. This structure is 20.6 m wide by 14 m long. It has:

- a 650 mm thick base similar to the gated spillway
- 2.5 m high by 500 mm thick walls
- 42No. 600 mm wide by 600 mm deep by 900 mm high baffle blocks

A waterstop is to be cast into the base and walls at the interface between the gated spillway and energy dissipator.

Both the upstream end of the gated spillway and downstream end of the energy dissipator are cast against the sheet pile scour protection walls with a polythene bond break between (refer section 6.6).

The gated spillway and energy dissipator calculations are included in Appendix B.2 and Appendix B.3. Details are shown on drawings D311 to D333 in Appendix D.

7.1.2 Stability

The stability of the gated spillway structure and energy dissipator has been assessed based on USACE EM 1110-2-2100 Stability Analysis of Concrete Structures as prescribed in the DCM.

The results of the stability assessment for the two structures are included in the Appendix B.2 and Appendix B.3. In summary, the sliding and flotation factors of safety and location of resultant and bearing pressures for both structures are less than the requirements of EM 1110-2-2100 except for the sliding factor of safety for the gated spillway under the extreme earthquake loading condition (1/2500 AEP).

The calculated factor of safety for the extreme earthquake loading condition is 1.0 compared to a required factor of safety of 1.1. Adoption of a seismic coefficient equal to 2/3 of the peak ground acceleration recognises that an acceptably small amount of lateral displacement will likely occur during a major earthquake. A factor of safety of 1.0 means that the additional lateral displacement may be marginally

[14] *"Stopbank design and construction guidelines"*, Version 1.2, BoPRC, March 2021.

higher. Given that the gated spillway structure does not typically retain water the marginal increase under an earthquake of this magnitude is considered acceptable.

7.1.3 Strength & Serviceability

The gated spillway and energy dissipator concrete reinforcing typically consists of H20 bars at 200 centres each way and on each face. Based on NZS 3106 this slightly exceeds the reinforcement required to limit early age thermal cracks to 0.2 mm in fully restrained members with a thickness greater than or equal to 500 mm.

Both the gated spillway and energy dissipator were modelled in Space Gass. The graphical input and finite element output is included in Appendix B. This shows that H20 bars at 200 mm centres are typically sufficient for strength. Where this is not sufficient H25 bars at 200 mm centres are provided. This includes the bottom transverse layer of reinforcing in the base and outer vertical layer of reinforcing in the abutments of the gated spillway.

H16 bars at 200 mm centres each way and on each face are adequate for the energy dissipator baffle blocks. While the drawings show conventionally reinforced concrete for the baffle blocks the Contractor may elect to precast these elements. Design of lifting inserts, additional reinforcing for handling, and connection to the slab is the responsibility of the Contractor.

7.2 Stoplog Slots

Stop log slots are provided on the upstream side of the gate, for use in the event that the gate bays need to be isolated for major maintenance works. Given the low operating head and normally dry service conditions, there are no embedded parts associated with the design of the stoplog slots for stoplog guidance or sealing. Instead, the design has allowed for a F5 concrete finish for resistance to cavitation and a good flatness for stoplog sealing.

Design of stoplogs themselves for these slots is beyond Norconsults scope of work and accordingly have not been designed.

7.3 Footbridge

A footbridge provides access across the structure and to the piers upstream of the gates. It has three similar spans each 4.39 m long by 1.2 m wide.

The main beams consist of 230 PFC's connected with 200 PFC cross members that support Webforge grating. A short section of grating spans between the footbridge and the piers and is removable to allow access to the stoplog slots if required. The footbridge is attached to the top of the gated spillway piers and abutments with chemical anchors.

The 230 PFC main beams were selected to accommodate the cylinder hydraulic lines. These are sufficient to support the 3 kPa uniformly distributed load or 3.5 kN concentrated load as per the standard referenced in the DCM (AS/NZS 1170.1, Table 3.1, Type B).

7.4 Gate Embedded Parts

The embedded parts comprise the side seal plates and sill seal plate assemblies which allow for gate lateral guidance and sealing against the civil structure.

Fabrication, erection and embedment of the seal plates is primarily by the CP2 Contractor, with the exception of a mounting plate left by the CP1 Contractor during casting of the primary concrete. The purpose of the mounting plate is to allow the CP2 Contractor a pre-embedded surface on which to readily weld adjusting studs to facilitate erection works.

An alternate arrangement would be that the mounting plate is removed and that the CP2 Contractor epoxy anchors his adjusting studs directly into the primary concrete. This option would require re-bar scanning to work satisfactorily and is not normally done for green field sites. The Instructions to Tenderers (ITT) should request the CP2 Bidders preferred mounting methodology and the 'For Construction' set of drawings and technical specifications updated as appropriate.

The side seal plates have been designed to allow line of sight and direct access of tooling onto the adjusting nuts prior to secondary concrete embedment, simplifying set up. The width of the sealing plates is sufficient to allow appropriate tracking of the side guide wheels and side seals under normal and exceptional operation events whilst providing sufficient rigidity in the event of side loading from the gate. Side seal plates are made up of three straight segments to facilitate ease of fabrication and erection as opposed to curved plates. The side guide plates extend past the top of the concrete by about 240mm to allow a minimum of 2 side guide wheel sets to remain within the side guides when the gate is in its fully open position (i.e., n-1 or emergency operating scenario), providing improved lateral guidance and resistance against wind loading etc. at these high openings.

Whilst embedded steelwork with >75mm cover is typically mild steel, the Rangitaiki Spillway design has adopted full stainless steel for all embedded steelwork at BoPRC's instruction to eliminate issues with welding of dissimilar metals and subsequent possibility of concrete spalling. The cost of changing the mounting plate and associated anchor material from painted mild steel to stainless steel is likely a relatively small cost relative to the total project.

Erection tolerances for the embedded parts are considered industry standard and are typically adopted from DIN19704[15], tolerance Class 1.

The specified secondary concrete is Sikagrout 212 which is a flowable grout with minimal shrinkage properties and strength properties that exceed that of the primary concrete. Pour height is limited to a maximum of 2m per pour to prevent distortion of the embedded parts. Reinforcing bar is provided to adequately tie the secondary concrete to the primary concrete.

7.5 Radial Gates

Each of the three radial gates are of identical design, measuring 3.4m in width, 3.3m height and of 4m skinplate radius.

Design static head is 3.3m (RL 7.8m), equivalent to the top of the civil structure and surrounding stopbanks, well in excess of the 1/100 AEP flood event (RL 7.2m). The gates have not been designed for overtopping.

Gate design operating head is set at the 1/100 AEP event being RL 7.2m with an opening margin of 40% over the gates combined factored friction and self-weight from the fully closed position (i.e. maximum water load). Gate operation at water levels greater than RL 7.2m is possible, but at reduced opening margins. Norconsult anticipates that gate opening beyond the 1/100 AEP event to be unlikely and, if required, the expectation would be that the gates are opened in advance of this river level being realised.

The gate has been arranged to be assembled in situ by bolted connection for the purpose of reducing transport and site craneage requirements. Bolted connections are located between (1) the gate skinplate assembly and gate arms, (2) the gate arms and gate trunnion assembly and (3) between the gate trunnion assembly and the gate trunnion beam. Each of the three bolted connections designs includes steel packing

[15] "Hydraulic Steel Structures", Part 2, DIN19704:1998

plates to facilitate site adjustment during assembly. Packers will require a machined face for adequate fit up and corrosion protection reasons.

The heaviest lift during gate erection is the gate skin plate assembly, with a weight of approximately 2.1t. The skin plate assembly comprises a 12mm thick rolled plate supported by five (5), 150 UB horizontal girders each welded to two vertical beams, one on either side of the gate. Intermediate stiffeners have been provided to support the skinplate against long term deformation. These are relatively inexpensive and provide additional structural margin (though are not strictly required).

Gate hydrostatic load is transmitted to the gate trunnions via two pairs of radial arms located on each side of the gate which results in two points of support for the vertical beam. The arms are a 125 x 125 SHS section whilst the vertical beams are built up from plate. Gate arm arrangement has been determined according to guidelines published by Erbisti[16] to provide equal bending moments on the main vertical girder supports. The gate arms are connected at their mid span by strut to reduce buckling length and provide improved long term confidence in the design.

The gate trunnion assembly comprises a 50mm thick steel bearing housing fitted with an off-the-shelf SKF spherical trunnion plain bearing.

7.5.1 Trunnion Bearings

The selected bearings are SKF plain spherical bearings complete with triple lip seals over the inner race to prevent ingress of debris over time. The bearing material is a self-lubricating, PTFE fabric material, requiring no greasing or maintenance over the course of its life. The bearing is slightly oversized for its application, in order to reduce trunnion pin bending stress requirements and thus give the Contractor material options come procurement (also refer Section 7.5.4).

Bearing rotation and misalignment is accommodated on a common spherical face located between the inner and outer races. The fit of the bearings inner race to the pin is a clearance fit (h6), to facilitate assembly and allow the inner race a small amount of axial float during operation. Spacers are also adopted to locate the inner race on the trunnion pin in the axial direction. The fit of the bearing OD to the housing ID is an interference fit to accurately locate the bearing.

7.5.2 Seal arrangement

The gates are fitted with side and sill seals. All seals are EPDM material, selected due to its superior resistance to UV exposure compared to other common seal materials.

The side seals are an angle seal design, selected due to their lower pre-set and hydrostatic reaction force compared to that of music note seals. Accordingly, side seal friction loads and thus hoist loads are also reduced.

The sill seal is of a rectangular profile, arranged to provide a 5mm protrusion below the bottom of the gate to encourage a single flow detachment point whilst maintaining suitable compressibility for sealing.

The sill seal sits flush against the face of the angle seal, thereby eliminating any leakage path.

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[16] *"Design of Hydraulic Gates"*, 2nd Edition, Paulo C.F Erbisti, 2014.

7.5.3 Side Guide Rollers

Three sets of side guide rollers have been provided down each side of the gate skinplate (i.e., six sets total per gate) for the purposes of providing lateral guidance over the full gate stroke. The clearance between the side guide wheels and the side seal plates is 10mm ±1mm.

The side guide wheel sets are aligned with the horizontal girders for load transfer reasons, with the positioning of the two most bottom wheel sets such that they remain within the side guides when the gate is in its fully open position. The side guide assembly is a full stainless-steel fabrication (unpainted) which is likely comparative in cost to prepping, blasting and painting the equivalent mild steel assembly given the small quantities involved.

The stainless steel wheel axle is supported by D-Glide self-lubricating bushings which rotate around a static pin. The bushings require zero maintenance and are provided sufficiently thick so as to elastically take up a small amount of misalignment between the wheel face and the track surface on contact.

The side guide assembly is separated from the mild steel gate structure by a 3mm thick sheet of nylon to provide galvanic isolation of the dissimilar metals.

7.5.4 Gate materials

Gate design has considered section and material availability in the current market, in an effort to minimise the use of materials which require international procurement. The gate structure is designed around the use of G300 mild streel plate and G350 steel sections, commonly available in NZ from a range of different steel suppliers.

Pin material for the trunnion bearing, hydraulic cylinder trunnion and connection pin are grade 17-4PH stainless steel, condition H1150, normally held in stock at Specialty Steels and Metals in Auckland. The technical specifications permit alternate pin materials subject to the Engineers review and non-objection, allowing the Contractor some freedom in sourcing alternate materials should the selected material not be available at the time of procurement.

7.5.5 Gate Hoist

7.5.5.1 General

All three gates are operated by twin, single acting hydraulic cylinders arranged to act in tension only, thus avoiding the possibility of rod buckling. The cylinders are supplied with pressurised oil from a common hydraulic pack located in the gate house. Whilst the hydraulic pack is common to all three gates, each gates operating system comprises separate valves, supply and return lines, thereby increasing system redundancy over a common header pipe arrangement. All valves are, as far as practical, located at the hydraulic pack, minimising field mounted components.

Gate position is by linear variable differential transformer (LVDT) that measures cylinder rod position. Proximity probe type limit switches are used to detect end of stroke (both fully open and filly closed positions). The gate control system will include an automatic shutdown command in the event that two cylinder read more than 5% difference (adjustable setting) to one another, indicating a gate jam and preventing further damage to the gate and/or side guide plates.

Gate operation is completely manual, either directly at the pack, from the gate house switch board or remotely, via the RiCADO SCADA system (BoPRC's operating system of choice).

7.5.5.2 Hydraulic Cylinders

The selected cylinders are from Victor Hydraulics Marine series, are of 3.5” diameter bore and 2” diameter rod with a stroke length of 1430mm. The cylinders are arranged to lift the gate from its fully closed position (bottom of gate at RL 4.5m) to fully open position (bottom of gate at RL 7.8m). A gate opening of just 0.9m is required for all three gates to pass the 1:100 AEP gate discharge of 40m^3/s. A greater gate opening has been allowed to accommodate n-1 gate operation under which the gates are required to open to

Opening and closing rates are both 250mm/min resulting in an opening/closing time of approximately 13 minutes. Opening time to the 1:100 AEP opening of 0.9m will take just over 3.5 minutes. Operating speed was selected to be consistent with that of the Kaituna gates to simplify operator understanding.

Each cylinder is rated to 8.82t at 3000 psi (206 bar). While higher hydraulic operating pressures are possible (with different cylinders), the selected 206 bar maximum improves the likelihood of off-the-shelf component availability at short notice compared to higher pressure systems should parts require replacing in an emergency.

The cylinder rod end is fitted with an external, off-the-shelf SKF rod eye permitting misalignment of the cylinder rod relative to the lifting pin at the gate. The cylinder rod eye is secured to a cantilevered gate lifting pin secured through the bottom of the vertical beam and radially aligned with the lower gate arm. The rod connection point has been selected to eliminate bending moment on the vertical beams and to provide a defined load path into the trunnion bearing. The rod eye is secured to the pin via external circlip.

The cylinder head lug is fitted with a spherical plain bearing allowing misalignment of the cylinder relative to cylinder trunnion pin.

7.5.5.3 Cylinder Arrangement

Selection of the cylinder mounting position on the civil structure is a trade-off between cylinder force and geometric constraints. Locating the cylinder nearer to the rod connection point requires a lower hoist force but at the expense of a higher mounting point to fit the cylinder when the gate is raised. Conversely, mounting the cylinder closer to the gate trunnion bearings requires a greater hoist and gate trunnion bearing load but allows a lower mounting position which in Rangitaiki’s case is favourable from a structure stability, overturning and cost perspective.

The relatively limited height of the Rangitaiki civil structure required the cylinder mounting position approximately 1.5m upstream of the gate trunnion. Accordingly, gate hoist load is relatively high for the gate size, but these high cylinder loads are only realised when the gate is near its fully open position which is anticipated to be a very rare occurrence for the reasons outlined in Section 7.5.5.2.

The gate lifting point is aligned with the lower gate arm, providing a direct load path for the transmission of operating forces into the trunnion bearings and eliminating bending of the vertical beams .

7.5.6 Finite Element Analysis (FEA)

Hand calculations for the gate design were verified through the use of FEA.

Inventor Professional (Autodesk) was used to produce a 3D model of the radial gate assembly. The gate assembly was simplified into a single-body model for use in *Inventor Nastran* (Autodesk FE software). Non-structural features of the gate were not considered, such as guide and seal assemblies and their relevant fastener holes on the skinplate. The trunnion bearing assemblies were simplified into solid blocks, with the outer race housing of the bearing merged into the gate structure block and the inner race housing merged into the trunnion base plate and pin. The bearing inner and outer race sliding faces were set as contact faces for FE modelling purposes to reflect real world conditions of the bearing arrangement.

The FEA model is depicted in .

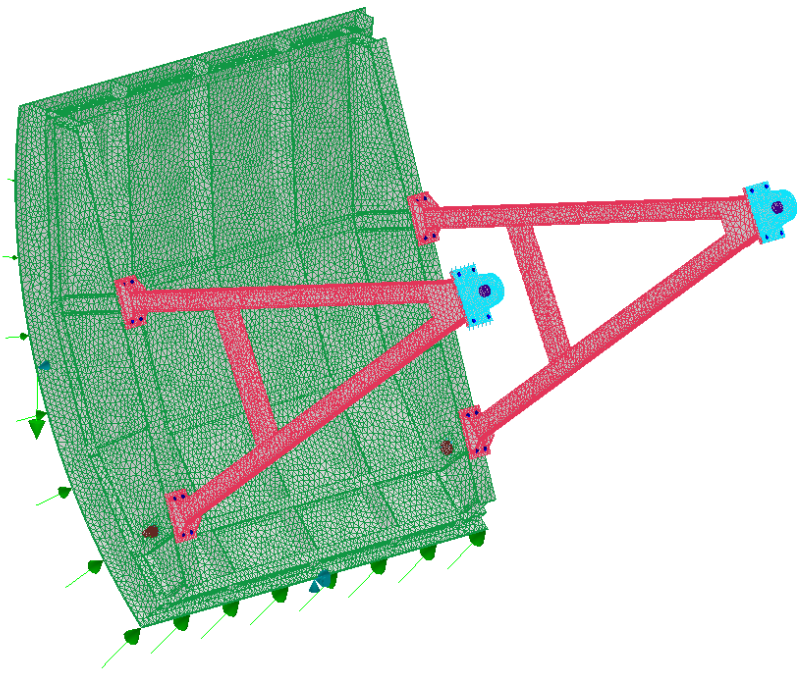

Figure 13: Isometric view of the Rangitāiki Spillway gate, with Nastran mesh, loads, and constraints shown

7.5.6.1 Load Application

The load cases assessed in the FEA are as follows:

Table 7-1 Rangitāiki Spillway Gate FE Load Cases

#	Load Case	Gate Action	Head on gate (m)	PGA (g)	Hoist Load (kN)	Load Classification (Erbisti)
LC1	Gate closed against maximum head in river (equivalent to height of gate)	Static	3.3	0.0	0.0	Normal
LC2	Gate closed, no hydrostatic pressure on gate, 1/2500 AEP EQ	Static	0.0	0.79	0.0	Exceptional
LC3	Gate opening from fully closed position 1/100 AEP flood event, PRV lift by hydraulic cylinders	Static	2.7	0.0	173.0	Occasional

#	Load Case	Gate Action	Head on gate (m)	PGA (g)	Hoist Load (kN)	Load Classification (Erbisti)
LC4	Gate in fully open position (no hydrostatic pressure on gate), PRV lift by hydraulic cylinders to hold gate open	Dynamic	0.0	0.0	173.0	Occasional

7.5.6.2 Gate Static Load Cases

The hydrostatic loads were applied as a pressure to the upstream face of the skinplate.

Hydrodynamic loading of the gate was not considered as the river level is normally below the gate sill. The combination of an earthquake and a 1/100 AEP flood event is considered a double jeopardy event and therefore not assessed.

7.5.6.3 Gate Dynamic Load Cases

Gate dynamic load restraints are depicted in Figure 14.

In the dynamic load cases (gate opening), seal friction and trunnion bearing friction forces were applied to the model to simulate gate opening conditions.

The side seal friction is applied at the side face of the skinplate. The loads were applied in the opposing direction to gate opening to simulate the friction loads.

The bearing trunnion friction load was applied at the bearing's sliding surface, to simulate the moment imposed on the bearing due to friction between the inner and outer bearing races during gate operation.

7.5.6.4 Model Constraints

In the static load cases the gate was supported with a frictionless vertical constraint at the skinplate sill and a transverse horizontal constraint applied. The trunnion bearing pins were fixed in all directions.

In the dynamic load cases (lifting of gate), the vertical sill constraint was removed, and a vertical constraint placed at the cylinder lifting pin (on the gate) with all other constraints as per the static load case models.

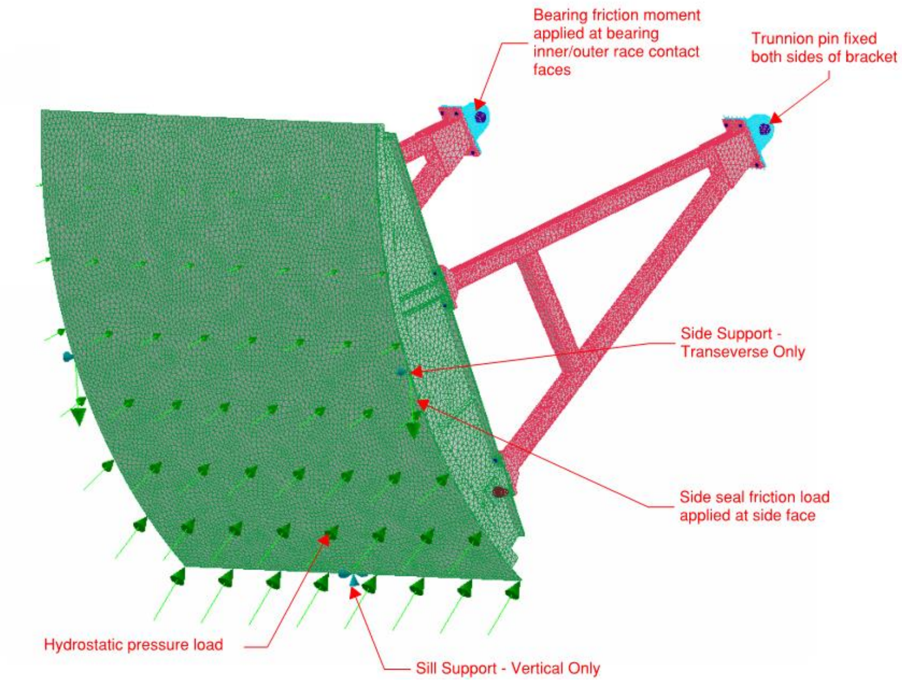

Figure 14: Isometric view of the model loads and constraints

7.5.6.5 FE Model Structural Element Results

FE Model results are depicted below in Figure 15 through Figure 18. Results showed very low overall utilisation.

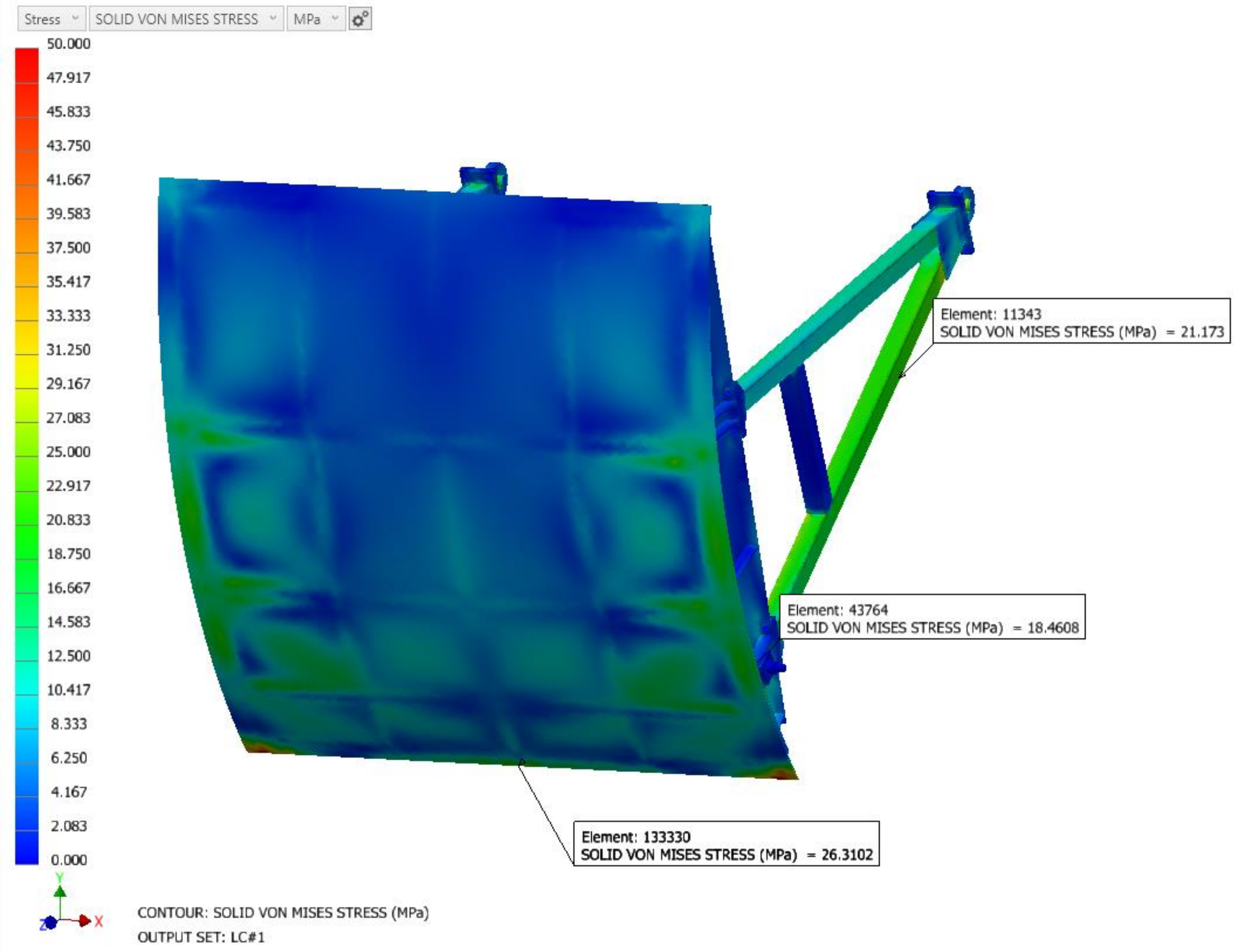

Figure 15: Load Case 1 - Combined Stress (Von Mises)

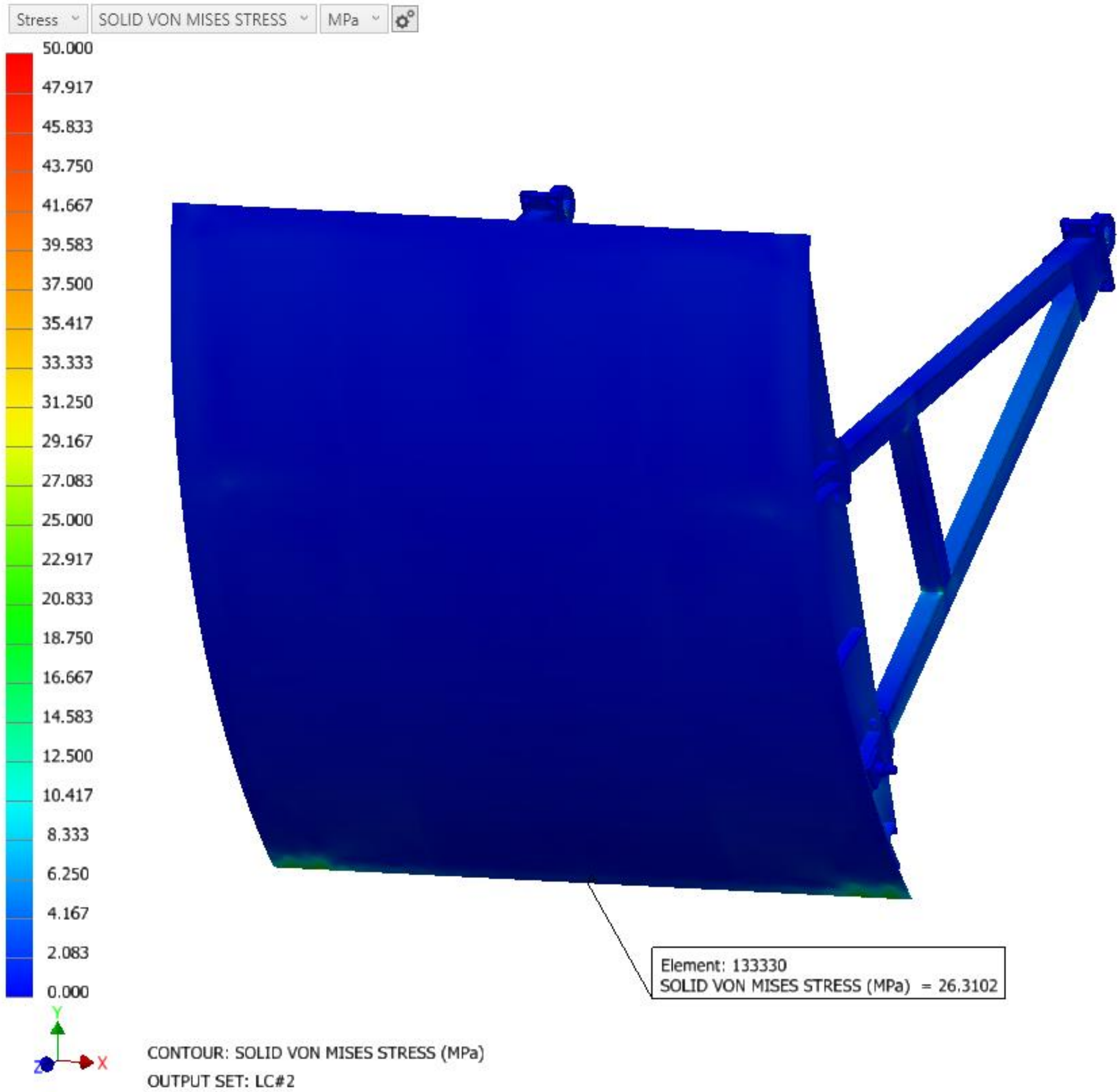

Figure 16: Load Case 2 - Combined Stress (Von Mises)

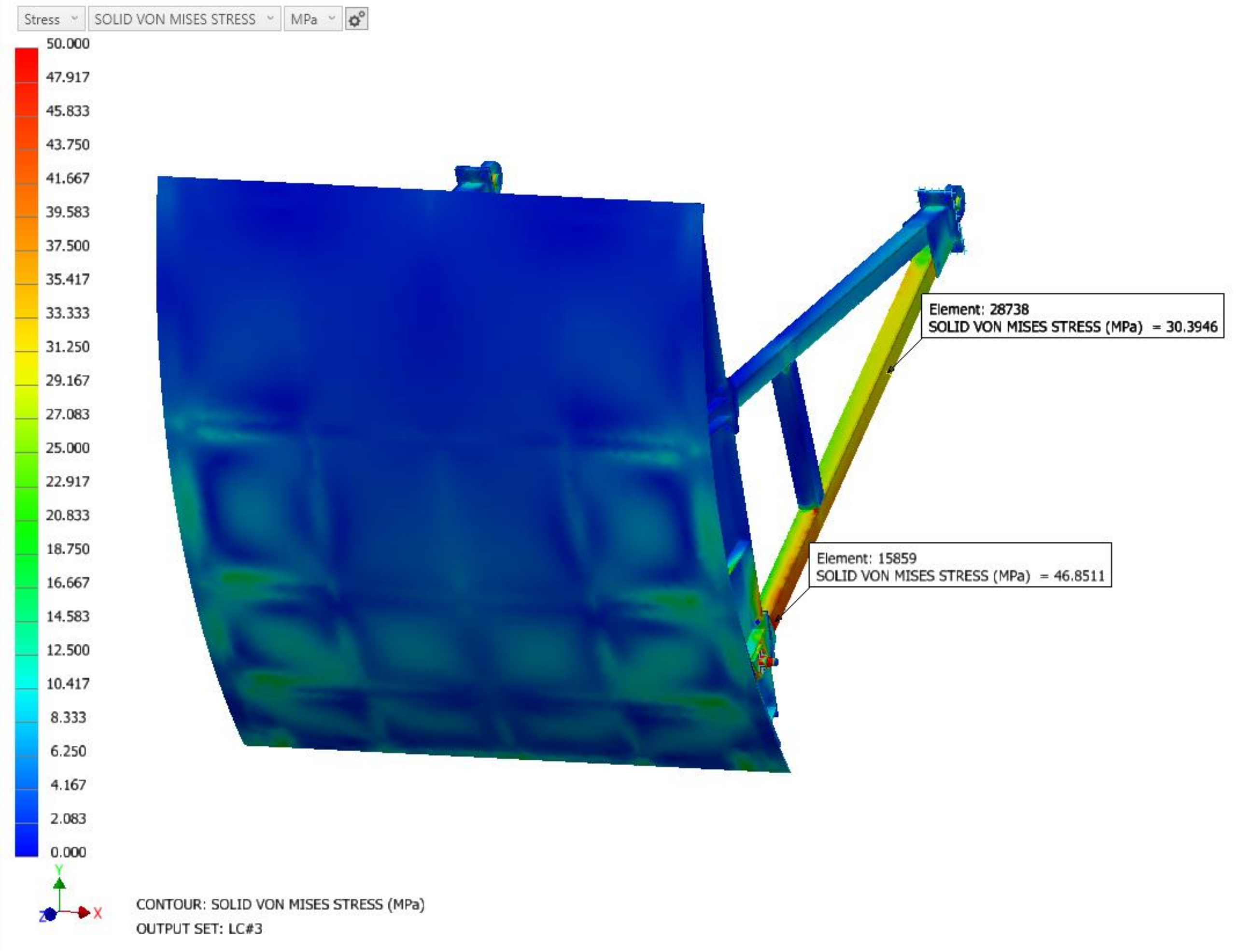

Figure 17: Load Case 3 - Combined Stress (Von Mises)

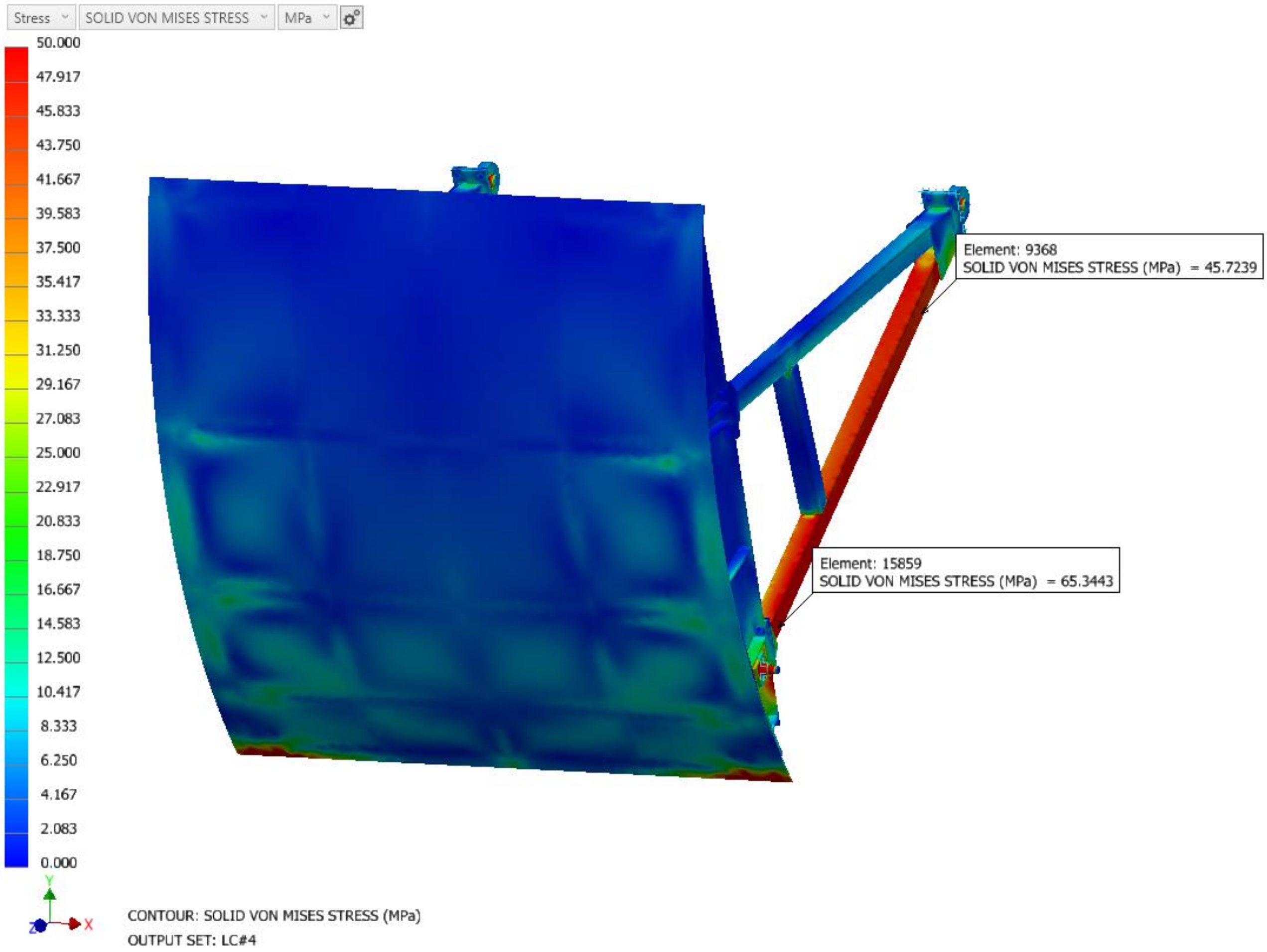

Figure 18: Load Case 4 - Combined Stress (Von Mises)

7.5.6.6 Model Validation

The FEA results depict a low loading to that of the hand calculations. Further modelling was undertaken of the radial gate to validate the FE model results. To ensure that the skinplate was not behaving in an unrealistic manner when combined with the structural framing (girders and stiffeners), an FE model of the gate without skinplate was assessed. Hydrostatic loading was removed from the FE model and inserted as uniformly distributed loads (UDL) along the length of the girders as shown in .

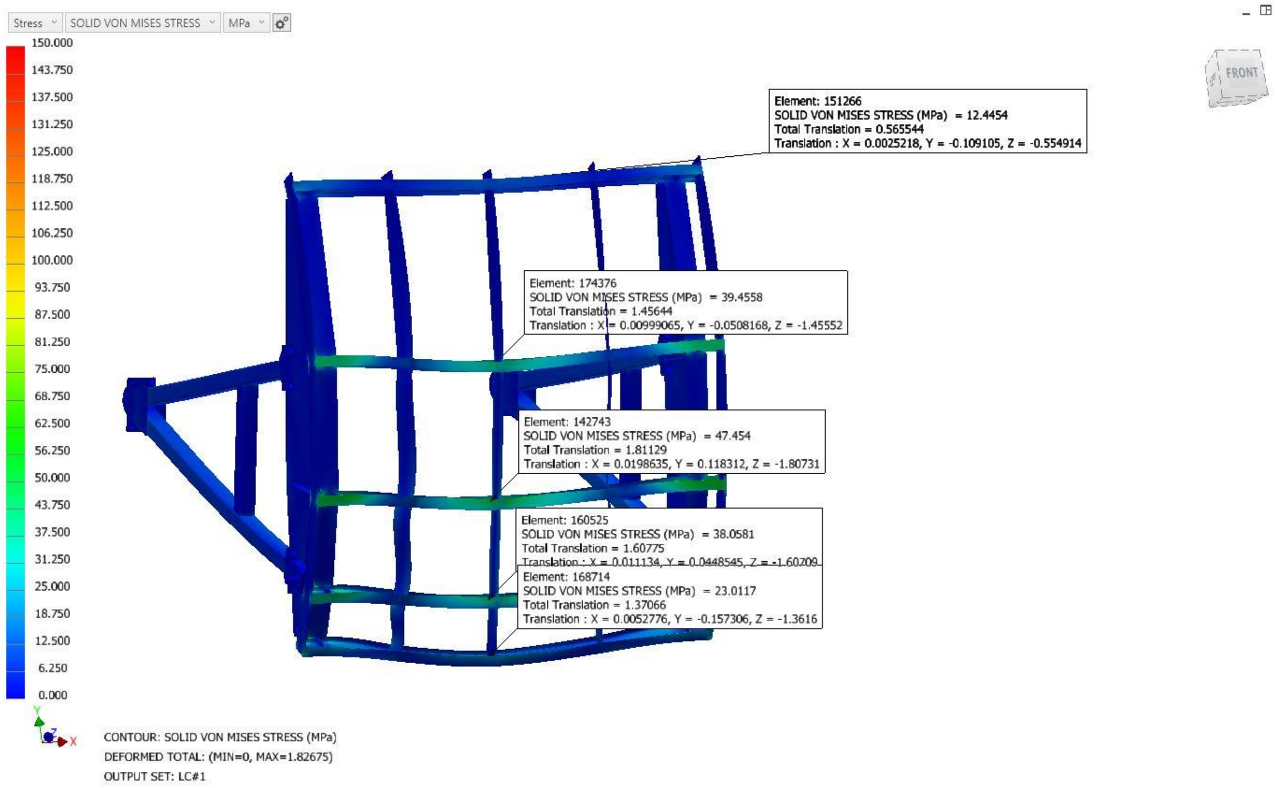

Figure 19: Radial gate FE model without skinplate

This model was further simplified to an FE model of UB girder 3 only with an equivalent UDL applied across the upstream flange of the girder to simulate the transfer of load from the skinplate into the girder.

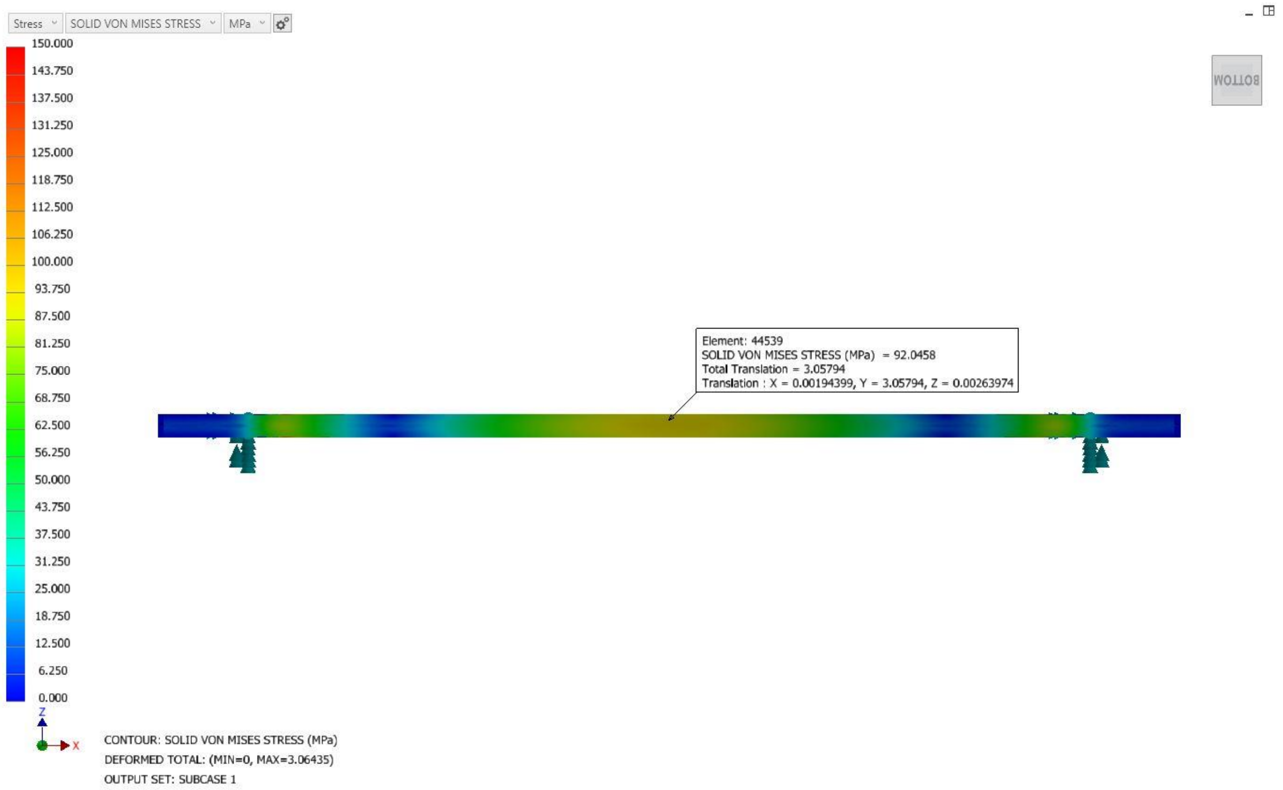

Figure 20: Radial gate FE model of girder 3

7.5.6.7 Results

The FE results show poor agreement with the first principle calculations, even for the simple beam analysis as depicted in Figure 20 (92MPa FEA vs 144MPa calculated) however both forms of analysis are within allowable limits.

The primary reason(s) for the differences is suspected to be a combination of (1) the radial stiffness component provided by the skinplate, (2) the increased section modulus of the girder due to the inclusion of the skinplate and some (3) two-way action in the skin plate, all of which are not adequately accounted for in the first principle calculations i.e., the calculations oversimplify the stress distribution.

Nonetheless, given that both calculation forms demonstrate suitable stress margins to allowable stresses, the design is considered suitable.

7.5.7 Gate & Cylinder Trunnion Beams

The gate and lifting cylinder trunnions are attached to trunnion beams designed to transfer the gate and cylinder loads into the gated spillway piers and abutments.

For both beams the loads are applied at the trunnion which is eccentric to the centre of the beam. Square hollow sections (SHS) have therefore been adopted as these provide suitable resistance to the resulting torsion – a 250 SHS for the gate trunnion beam and a 200 SHS for the cylinder trunnion beam. Each of the beams are attached to the reinforced concrete piers or abutments with cast-in M36, 2507 super-duplex anchors.

When assessing the loads on the beams and anchors three gate positions were considered– closed, 50% open and 100% open. As the beams on the piers support a gate and cylinder on both sides of a pier various combinations of different gate positions have been considered. Both beams have conservatively been designed for the PRV loads and a DIN 19704-1 partial safety factor of 1.35. Loads due to earthquake are significantly smaller than the hydrostatic or PRV loads and therefore have not been assessed.

The gate and lifting cylinder beams were modelled in Space Gass. Graphical output of the Von Mises stress envelope and maximum deflections is included in Appendix B.

The anchors have an embedment depth of 650 mm. While concrete breakout or shear failure is unlikely due the presence of the typical reinforcement in the piers and abutments, supplementary reinforcement is provided to ensure that the anchor shear and tension forces are reliably transferred into the concrete. For tension, the additional anchor reinforcement consists of a pair of H25 U-bars developed on both sides of the potential breakout surface. For shear, additional anchor reinforcement consisting of H16 straight bars are provided adjacent to the anchors near the upper surface of the concrete.

The anchors include adjusting nuts to assist with accurately locating the beams during installation. Following final positioning of the beams grout pads will be installed between the concrete and the underside of the base plates. The grout pads extend at least 220 mm from the anchor centre line in the direction of the predominant shear force (parallel to the face of the piers and abutments). This exceeds six times the anchor diameter, which based on the SESOC Design Guide – Anchor Bolts for Steel Structures, is the distance at which the grout can be assumed to be effective in resisting lateral bearing minimising bending moments in the anchors.

In addition, following final positioning, the technical specifications require the annulus between the anchors and base plate holes to be filled with epoxy mortar for shear transfer across all anchors.

The gate and cylinder trunnion beam calculations are included in Appendix B.2. Details are shown on drawings D510 to D512 and D531 to D533 in Appendix D.

7.5.8 Design for security

The gates and hoisting system have, as far as practical, been designed with security and potential for vandalism in mind. All critical bolted connections are fitted with shear nuts on top of the tensioned nut, which also double as locking nuts. Non-critical bolts, such as that fitted to the seal clamping plates for example, do not include a requirement for shear nuts.

Field instrumentation has been minimised, with most hydraulic valves being located at the hydraulic pack inside the gate house. Gate position sensing is internal to the hydraulic cylinders in the form of a linear variable differential transformer (LVDT) for the continuous measurement of gate position and proximity probe type limit switches for detection of end of stroke positions (i.e., as back up to the LVDT). Both instrument types offer high levels of reliability with minimal moving parts and are visually unenticing.

Each of the high pressure hydraulic supply lines fitted is fitted with a low-pressure switch for the detection of a broken line downstream of the gate house. The activation of the low-pressure switch closes the upstream distributor valve to prevent draining of the reservoir and oil spill into the environment. Field hydraulic lines are protected as far as practical via web-forge grating, thus enabling their inspection without removal of the covers.

8 Electrical, Controls and Communications

8.1 Power Supply

8.1.1 Overview

The electrical power supply for the Rangitāiki Spillway will be sourced from the 11kV network that runs along Hydro Road. The configuration of the power supply connection will require input from the local lines company, Horizon Energy. The following items will need to be confirmed by the Contractor during the detailed design phase with Horizon:

- Confirmation of the capacity limitations of the local 11kV network.
- Required upgrades to the local 11kV network to facilitate the Rangitāiki Spillway (if any).
- Confirmation of the transformer size for the Rangitāiki Spillway power supply.
- The location of the transformer. For example, either a new pole mounted transformer located on a power pole on Hydro Road, or a new pad mounted transformer located near the gate house.
- Requirements for the local lines company assets to the installed on the Rangitāiki Spillway site (power poles, transformer, etc.).
- The location of the 'point of supply'. The 'point of supply' defines the point where the ownership of the power supply changes from the lines company to the customer.

The 400V mains cable has been specified to be buried from the point of supply to the gate house. However, this shall be dependent on the confirmed location of the point of supply.

Back up 400V electrical power is provided via a 100kVA standby portable diesel generator and socket located at the gate house.

8.1.2 Indicative Electrical Loads

The table below lists the indicative electrical loads for the Rangitāiki Spillway. These electrical loads shall be confirmed by the Contractor during the detailed design phase of the power supply.

Table 2: Indicative Electrical Loads

Electrical Load	Power	Phases
HPU pump motor 1	2 kW	3 phase
HPU pump motor 2	2 kW	3 phase
Gate control system and general services	3 kW	1 phase
Indoor and outdoor lighting	3 kW	1 phase
Gate house heat pump (future if required)	3 kW	1 phase

Assuming that the three single phase electrical loads will be assigned to the different phases to balance the 3 phase load, the indicative total 3 phase electrical load for the Rangitāiki Spillway is approximately 7 kW.

Assuming that the two HPU pump motors are started Direct Online (DOL), the peak starting current of the motors is approximately 6x the rated motor current. The indicative starting electrical load is:

2kW (Motor 1) x 6 + 2kW (Motor 2) x 6 + 3 kW (balanced single phase loads) = 27 kW.

This would indicate that a 50kVA transformer is likely required for the electrical power supply to the gate house if the HPU pump motors were started DOL, and to provide some spare capacity. If the local 11kV network has come capacity restraints, the local lines company may request that the motor starting loads are reduced by the using Variable Speed Drives (VSDs), which would reduce the motor starting load to approximately 2x the rated motor load.

8.2 Control System

The gates shall be operated by a PLC based control system. The gate PLC shall be interfaced to a RICARDO SCADA system via an Ethernet connection, common on most BoPRC sites and familiar to operations personnel. Communication to the wider BOPRC SCADA system will be via a 4G router/modem.

The gate control system will enable the operation of the gates locally via manual pushbuttons and remotely via the RICARDO SCADA system. There will be no automatic gate operating functions. All gate operations shall be activated manually.

The gate points of control are listed below in order of precedence:

1. Manual levers on the hydraulic pack,
2. Push buttons on the gate house control panel,
3. Remotely via the SCADA system.

Local gate indications shall be presented as light indicators each consecutively arranged and labelled to correspond with the order of the gates as arranged on the spillway structure. Manual push buttons shall be adopted for local control. It is intended that a touch screen will be provided as a Human Machine Interface (HMI). However, no gate controls shall be via the HMI, only physical pushbuttons shall be used. The HMI will only be used to display the operating parameters of the gate system and alarms.

The gate control system shall be provided with a uninterruptable power supply (UPS) sized for 8 hours of control system function in the event of a power outage.

8.3 Gate House Electrical Plant

The gatehouse will house the power distribution switchboard, local control panel and possibly a separate motor control centre (if the HPU pump motor contactors cannot be housed in the power distribution switchboard).

8.3.1 Clearance Around Switchboards

Switchboards installed within the gate house must be located such that the minimum clearance requirements, as defined in AS/NZS 3000, around the switchboard (from the face of the switchboard and/or with the switchboard door open) must be met.

8.3.2 Local Services

The gate house will be provided with light fittings and power sockets for general services. If required, a heat pump may be provided to maintain an appropriate operating temperature for the plant housed in the gate house.

9 Gate Operation

The following text has been extracted from Norconsult (2022) and is included for completeness. The reader should refer to the original text for full context.

9.1 Operating Guidance

This section is not a comprehensive guide to the operator but rather provides a high level commentary on flow aspects wen controlling the gates.

The gates are fully manually controlled ad as such it is the operator's responsibility to ensure the discharge rates released from the spillway gates are suitable in flood situations. If the spillway gates are opened, this is assumed to be because of one of the following reasons:

- Pressure/issue with the downstream stop-bank has caused the operator to open spillway gates to release flow to the floodway (or additional flow if the overflow spillway is already in operation) into the floodway. This assumes the floodway has available freeboard.
- The overflow spillway is in operation but there is some impairment to capacity, i.e. it is not by itself releasing sufficient flow into the floodway.
- Rangitaiki river flow rates exceeding the 0.01 AEP (1 in 100 year) flow rate has required bypass into the floodway to reduce flow rate in river.
- Potential for gate operation by mistake (a consideration assessed by fully opening the gates, which could result in gates discharge exceeding the gate design flow criteria).

The operator should be well versed on the operation of the gates and their need for operation in the context of the wider scheme i.e. including combined operation with the overflow weir. Operators must also have a good understanding of the figures presented as these provide important information about the differing scenarios that may be encountered in flood conditions.

The gate structure discharge and the location of the hydraulic jump downstream of the gates can vary with the river level. In any flood event the river level changes over time. All operations of the radial gates should be monitored to avoid the following circumstances to protect the gates during operation:

- Hydraulic jump wave action in contact with back of gate
  - Action: fully close or fully open gate)
- Water level cyclically building up and dropping below gate (infers loss of control of flow at current gate opening);
  - Action: either reduce gate opening to re-obtain discharge control or fully open gate)
- Water touching gate at one side only
  - Action: either reduce gate opening to re-obtain discharge control or fully open gate)

It is noted that the hydraulic jump formation will tend to wash out if the spillway gates are fully opened and this may result in damage to riprap rock and channel linings in the downstream channel. For this reason, full opening of the gates should be undertaken as an emergency measure only.

9.2 Eccentric Gate Operations

The assessment checks performance of the gates energy dissipater for the cases where the non-symmetrical gate openings may be required, e.g. in the case of a gate malfunction or where a gate may have incurred a debris blockage.

In the 0.01 AEP flood with single gate operation at an opening of 900mm opening and a headwater level near the designated water level sensor location of EL 7.15m, discharge of 13.7 m^3/s was simulated, with well controlled velocity distribution as seen in Figure 21.

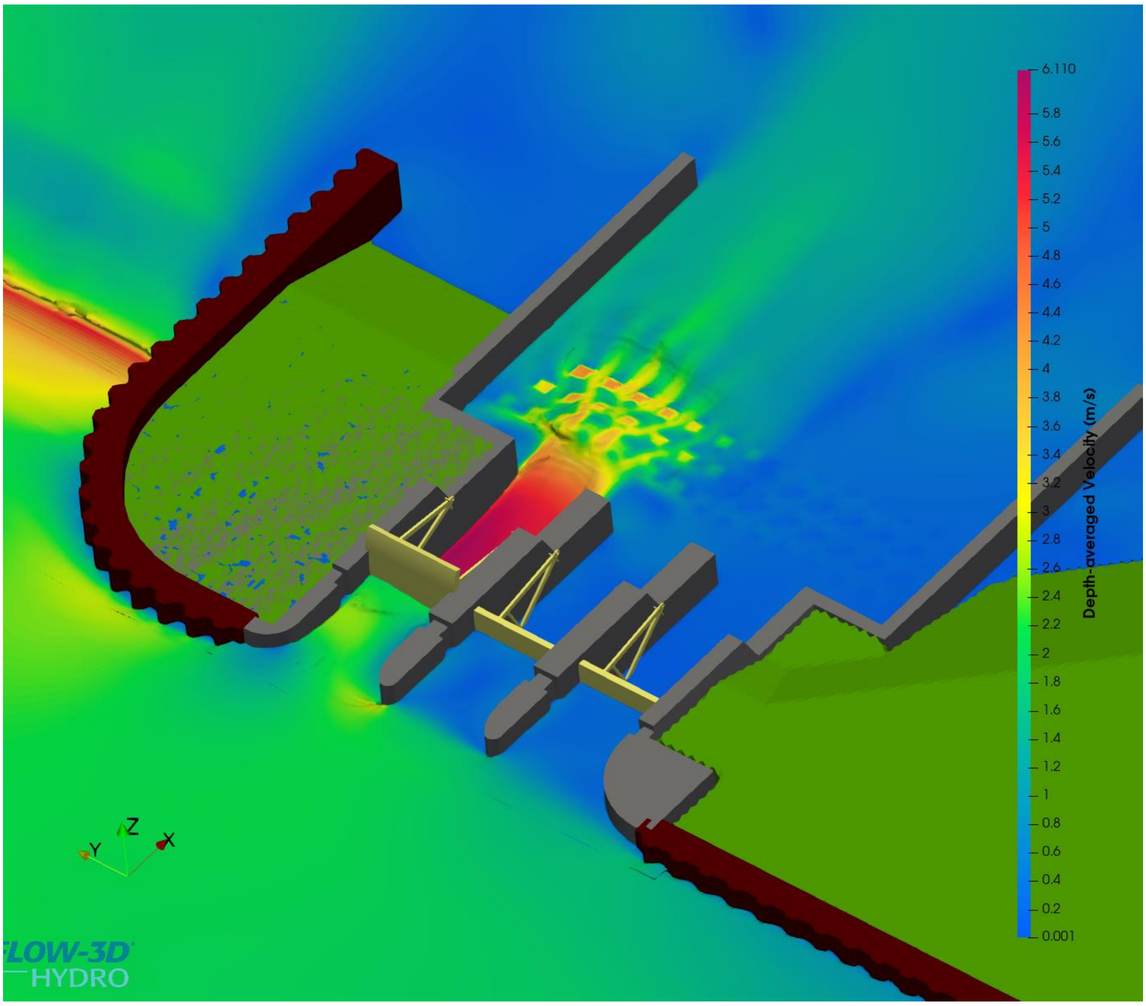

Figure 21: Single gate 'eccentric operation' (900mm opening) depth averaged velocity

Under the same conditions as in Figure 21, with one gate at full opening, 21.4 m^3/s is discharged and velocity distribution remains reasonable, as shown in **Error! Reference source not found.**.

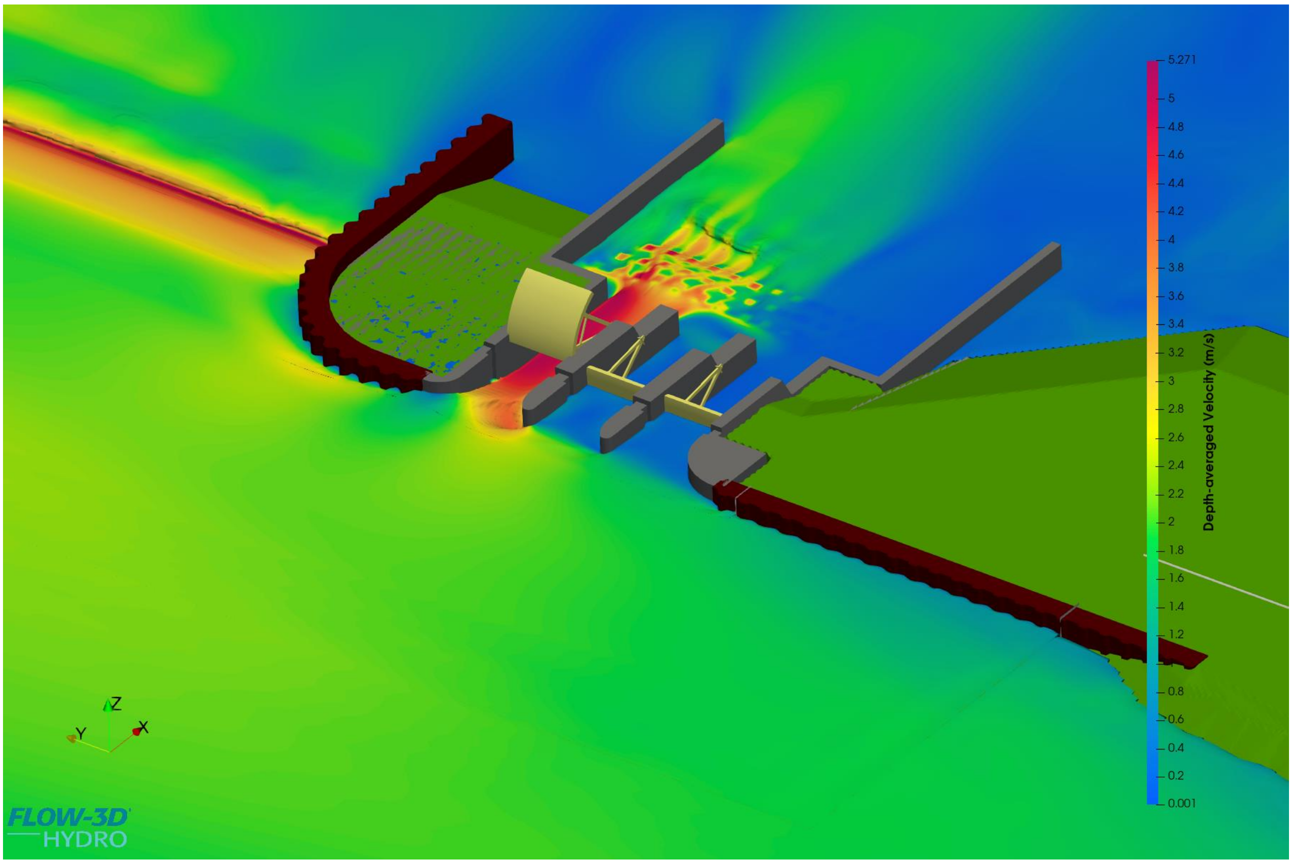

Figure 22: Single gate 'eccentric operation' (full open) depth-averaged velocity

Figure 23 shows the result of repeating the 0.01AEP flood simulation with two gates (gates 2 and 3) at full opening position, noting discharge from these gates totalling 42 m^3/s and the water level at the designated sensor position is EL 7.10 m. Velocity distribution resulting from the stilling basin operation looks reasonable, but there is usually some risk of damage to riprap rock and channel linings in eccentric operations. Symmetrical operation of the spillway gates is preferable as this provides the greatest opportunity for energy dissipation and velocity redistribution to calm the stilling basin outflow. While the model simulations indicate that eccentric operation of the gates is tolerable, eccentric gate operations should only be adopted as an emergency (n-1) operating practice.

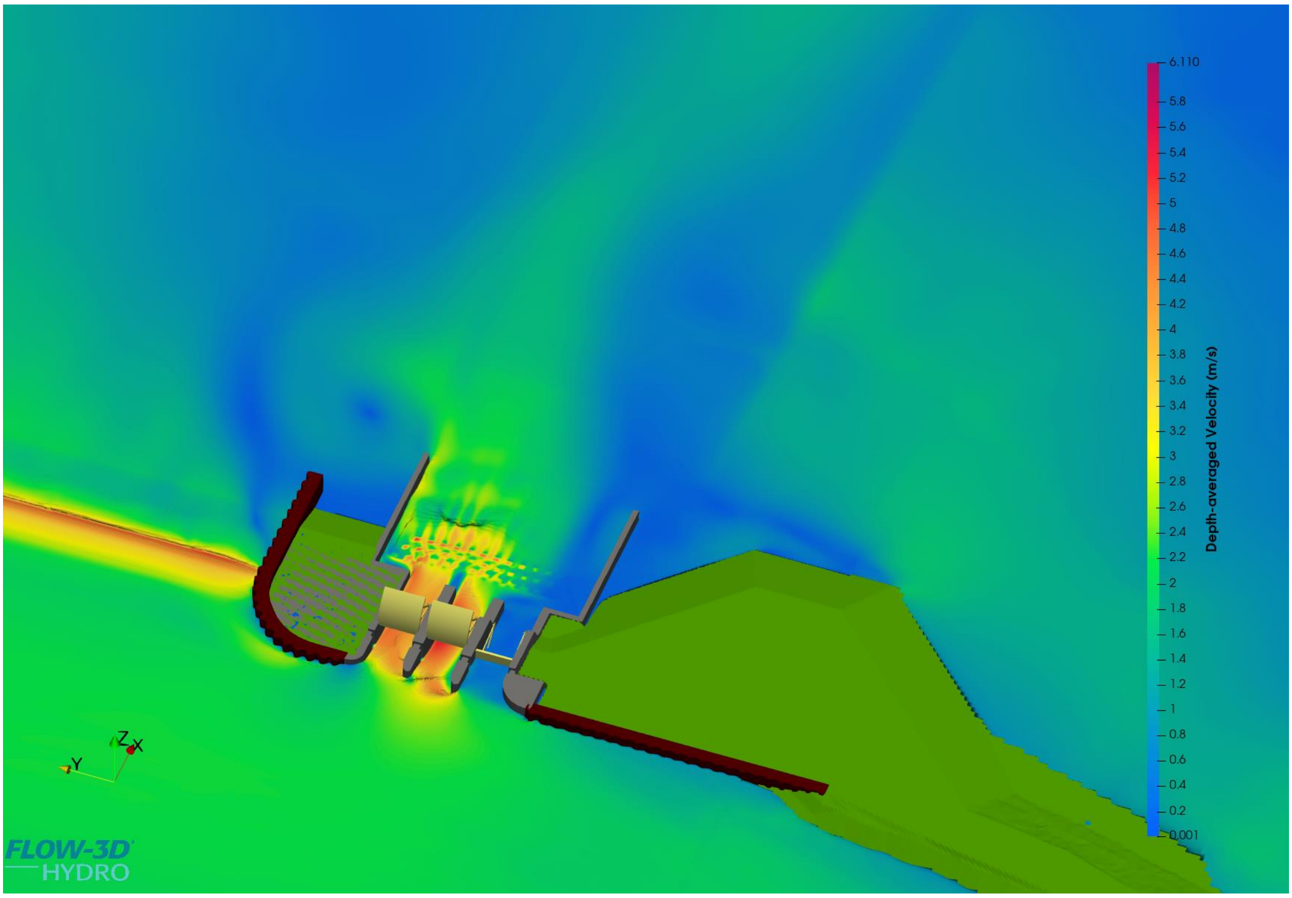

Figure 23: Two-gate eccentric operation (full open) depth-averaged velocity

9.2.1 Gates discharge rating

Refer Section 3.2.2.

10 References

Norconsult (2018)	*Rangitāiki Spillway Analysis - Project Initiation Report*
Norconsult (2019)	*Rangitāiki Spillway Analysis - Options Report*
Norconsult (2022)	*Spillway Hydraulic Report; Rangitāiki Spillway Project, Rev A, Norconsult NZ Ltd., 03 June 2022.*
Riley (2021)	*"Rangitaiki Spillway Geotechnical Report"*, RILEY Consultants, Issue 1, 28 September 2021.
BoPRC 2021	*"Stopbank design and construction guidelines"*, Version 1.2, BoPRC, March 2021.

Appendix A: Design Criteria Memorandum

Appendix B: Civil Design Calculations

Appendix B.1 Sheet Pile Walls

Appendix B.2 Gated Spillway

Appendix B.2.1 Gated Spillway Stability & Strength

Appendix B.2.2 Gate & Cylinder Trunnion Beams

Appendix B.2.3 Footbridge

Appendix B.2.1 Gated Spillway Stability & Strength

Appendix B.2.2 Gate & Cylinder Trunnion Beams

Appendix B.2.3 Footbridge

Appendix B.3 Energy Dissipator

Appendix C: Mechanical Design Calculations

Appendix D: Drawings