Scrivener Dam

Volume A: Dissipator Strengthening Options Report

National Capital Authority

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→ **The Power of Commitment**

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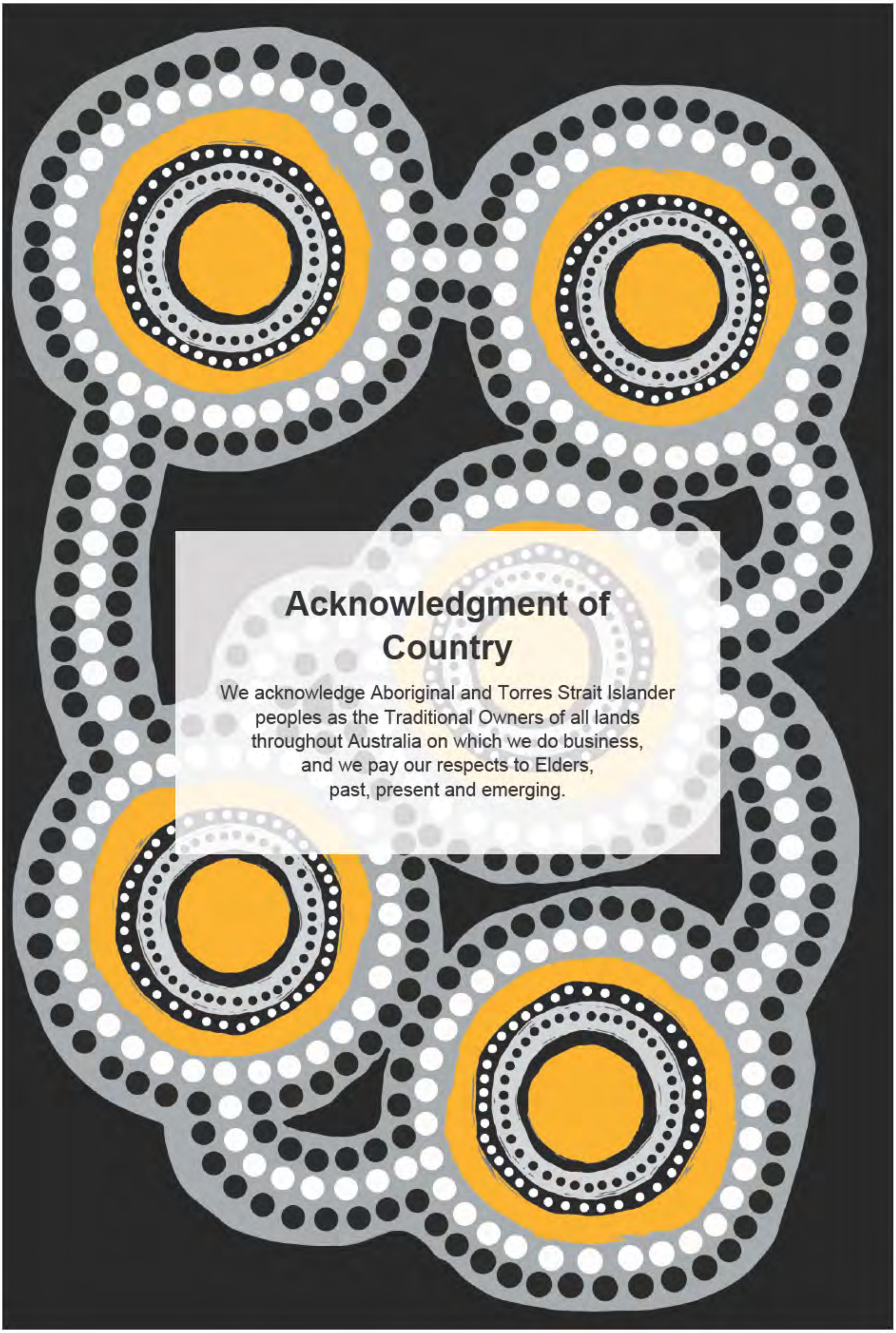

Acknowledgment of Country

We acknowledge Aboriginal and Torres Strait Islander peoples as the Traditional Owners of all lands throughout Australia on which we do business, and we pay our respects to Elders, past, present and emerging.

Executive summary

Background

In 2015/2016, a Design Review of Scrivener Dam (SMEC, 2016) identified that the stilling basin has several possible structural deficiencies, particularly relating to insufficient stability against uplift forces generated during spill events. The need to address these potential deficiencies was further supported by pre-existing concerns regarding the lack of waterstops in the stilling basin contraction joints, limited slab reinforcing and anchor lengths, the unknown condition of the anchors, and observations made of air-bubbling from the joints. A physical hydraulic model study was subsequently developed to further explore and quantify the potential hydraulic forces acting on the stilling basin over a range of discharges (WRL, 2021).

GHD was engaged in August 2021 to undertake an options assessment and concept design of upgrade works to the stilling basin to address the possible deficiencies. The options assessment (this report) included the following work:

- Review background documents on the project and develop an appropriate design criteria for the upgrade works;
- Identify a broad range of options to address the possible deficiencies in the stilling basin, and facilitate an options identification workshop to agree on the selection of three preferred options to further develop;
- Further refine the three preferred options, prepare preliminary cost estimates for them, review their advantages and disadvantages, and undertake a multi-criteria assessment of the options to identify the preferred option to take through to concept design.
- Facilitate an options selection workshop to agree on the preferred option to take through to concept design.

Design Criteria and Hydraulic Review

In the early phases of the project, design criteria were developed for the upgrade works. These criteria were largely based on dam engineering guidelines and reference documents, and information provided in the SMEC (2016) Design Review and the more recent physical hydraulic model study (WRL, 2021). As part of this process, a high-level review was undertaken of the hydraulic performance of the stilling basin, and the following key items were concluded:

- The proposed design flood was taken as the 1 in 100,000 AEP event, which is appropriate for a High C consequence category dam. This event equates approximately to a discharge of 8,200 m^3/s. Beyond this event, it is understood that the stability of the dam becomes marginal.
- The tailwater levels adopted in the analysis have been based on the SKM (2010) analysis, however the sensitivity analysis undertaken on tailwater levels in physical hydraulic model study runs was taken into account when developing loads for the stilling basin.
- The high-level review of the hydraulic performance of the dissipator suggested that the existing stilling basin is providing a relatively effective dissipation of energy, and does not require modification in the upgrade works. However, previous analysis indicates that there would be an improvement in flow conditions with changes to the gate operating rules (namely opening the outer gates earlier in the opening sequence).
- A review of the proposed hydraulic loads to be adopted in the stilling basin upgrade were undertaken. It was proposed that uplift beneath the slab be taken as a linear regression from headwater to tailwater level. It was proposed that pressures above the slab be taken as the mean less two times the standard deviation of the pressure transients measured in the physical model study. In cases where there are no waterstops in the contraction joints, increased uplift pressures were applied beneath the slab, equal to full reservoir head.

Option Identification

During the Options identification phase, nine key options were identified. High-level concepts for these options were developed, and an assessment was carried out to determine the ability for each option to meet the design criteria, and the pros and cons of each option. These options were presented during an options identification workshop, during which time, engineering judgment and experience was used to determine the three preferred options to take through to the option refinement phase. The list of broad options, and a summary of the reasons for selecting the three preferred options is provided in the following table.

Option	Description	Comments
1	Do Nothing	Eliminated – unlikely to meet performance requirements
2	Overlay slab with no anchors	Not preferred – slab likely to be very thick and may adversely impact the hydraulic performance of the basin.
3a	Anchored overlay slab (minimum slab thickness)	Preferred – Option offers a potentially robust solution, and meets design criteria. Moving forward, instead of considering three subset options, develop option to select an appropriate balance of slab thickness, anchors and connection between old and new slab.
3b	Anchored overlay slab (maximum slab thickness)	
3c	Anchored overlay slab (monolithic tie between existing and new slab)	
4	Retrofit Anchors into existing slab	Preferred – May not necessarily provide the robustness of other options, but likely to be a cheap solution so take forward to further development to challenge other options.
5	Lengthen stilling basin	Eliminated – unlikely to meet performance requirements unless done in combination with another option.
6	Tailwater control weir	Eliminated – as per Option 5
7	Change gate operating rules	Eliminated – as per Option 5
8	Deepening side bays of stilling basin	Eliminated – as per Option 5
9	Partial demolition of slab and reconstruct with anchored slab to original geometry	Preferred – Likely to meet performance requirements and provides a solution which maintains the current geometry.

Option Development and Preliminary Cost Estimates

The following three options were subsequently refined during the option development phase:

- Option 3 – Installation of a new, anchored overlay slab, resulting in the new stilling basin invert being slightly higher than the existing arrangement (refer to Figure 1);
- Option 4 – Retrofitting anchors into the existing slab to provide sufficient additional resistance (refer to Figure 2).
- Option 9 – Partial demolition of the existing slab and reconstructing a new anchored slab to the original geometry. This option is similar to Option 3, but maintains the same invert levels and geometry as the existing stilling basin.

For each of the three options, basic calculations were undertaken to size the key components of the works. As part of this work, the scope of works, advantages and disadvantages for each option were further refined.

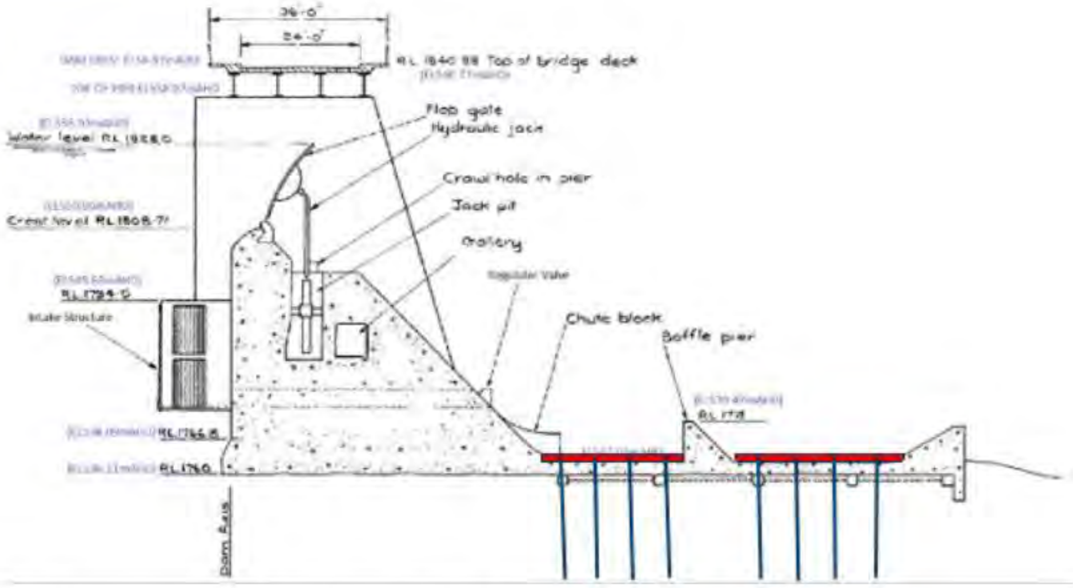

Figure 1 *Sketch of Option 3 (similar to Option 9)*

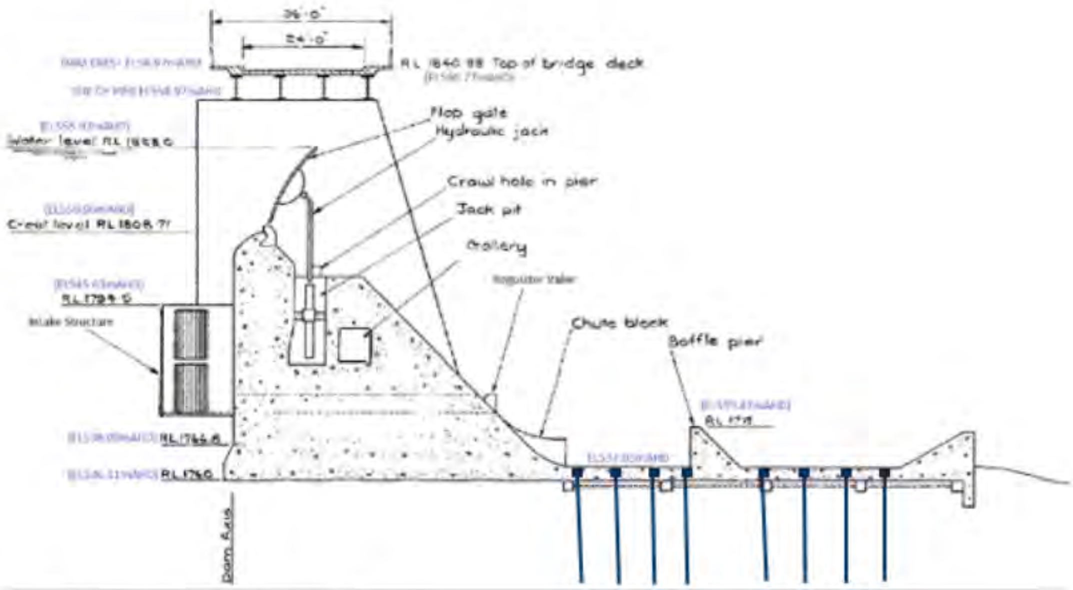

Figure 2 *Sketch of Option 4*

Preliminary comparative cost estimates were also produced for the three options. A summary of these estimates is provided in the following table.

Item No.	Description	Quantity	Option 3	Option 4	Option 9
			$	$	$
DIRECT COSTS					
	Preliminaries		[redacted]	[redacted]	[redacted]
	Flood Protection and Coffer Dams		[redacted]	[redacted]	[redacted]
	Stilling Basin Slab upgrade		[redacted]	[redacted]	[redacted]
	Stilling Basin Training Wall Raising		[redacted]	[redacted]	[redacted]
5	Miscellaneous		[redacted]	[redacted]	[redacted]

Item No.	Description	Quantity	Option 3	Option 4	Option 9
			$	$	$
TOTAL CONTRACTOR'S ESTIMATE					
	Total Direct Cost		[redacted]	[redacted]	[redacted]
	Contractor's Supervision and Site Overhead	35%	[redacted]	[redacted]	[redacted]
	Minor miscellaneous items not measured	25%	[redacted]	[redacted]	[redacted]
	Contractor's Estimate (excl contingency)		[redacted]	[redacted]	[redacted]
	Contingency allowance (including allowance for floods during construction)	40%	[redacted]	[redacted]	[redacted]
	Indicative Contractor's Estimate (excl. GST)		[redacted]	[redacted]	[redacted]
OTHER INDIRECT COSTS					
	Indirect Costs (excl contingency)		[redacted]	[redacted]	[redacted]
	Contingency allowance	40%	[redacted]	[redacted]	[redacted]
	Indicative Other Indirect Costs (excl. GST)		[redacted]	[redacted]	[redacted]
	Total Indicative Cost Estimate (exc. GST)		[redacted]	[redacted]	[redacted]

Multi-criteria Assessment

After developing the three options, a multi-criteria assessment (MCA) was used as a tool for identifying the preferred option to take through to the concept design. The work was presented during a second workshop, the 'Preferred Option Workshop', which included working through the MCA process with the participants. The MCA process used a number of different processes to assess the options, and included a sensitivity analysis which was undertaken following the workshop. Furthermore, an open discussion was held during the workshop to capture the participants' thoughts on which option was likely to be the preferred solution to upgrading the spillway. A summary of the MCA outcomes is presented in the following table.

	Option 3	Option 4	Option 9
A - Cost	7	7	6
B - Technical Merits	15	12	15
C - Construction	13	12	8
D - Operation and Maintenance	5	3	5
E - Other Aspects	14	16	10
Total Score	53	50	43

The MCA process identified that the preferred option was Option 3, and this choice was supported by the participants in the workshop. As such, it was recommended that Option 3 be progressed to concept design.

The concept development phase of the project is described in Volume B – Scrivener Dam Dissipator Strengthening Concept Design Report (GHD, 2021b).

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1. Introduction

1.1 Purpose of this report

The National Capital Authority (NCA) is embarking on a significant project to upgrade the stability of the stilling basin at Scrivener Dam, which is a culmination of several years of work. This work identified and, to a large extent through a recent physical hydraulic model study, quantified concerns regarding the potential for large differential uplift pressures to develop in the stilling basin. Under certain hydraulic conditions, these uplift pressures may lead to instability of the stilling basin slabs. Combined with pre-existing concerns regarding the lack of waterstops in the stilling basin contraction joints, limited slab reinforcing and anchor lengths, the unknown condition of the anchors, and observations made of air-bubbling from the joints, the need to address stability concerns is a prominent focus in the NCA's dam safety program.

The NCA engaged GHD in August 2021 to undertake an options assessment and develop a concept design for the proposed upgrade works. The key purpose of this consultancy is to identify the preferred upgrade option, and develop the concept to an extent where the following is achieved:

- The broad details of the preferred concept for upgrading the stilling basin are defined, with sufficient detail to prepare a Class 4 cost estimate and with the solution meeting the proposed design criteria, and being compliant with industry guidelines.
- The broad details of the associated flood management and constructability aspects required to construct the upgrade works are defined.
- A Class 4 Cost estimate (in accordance with AACE guidelines), which will ultimately be used by NCA to submit an application for funding of future stages of the project.
- Details of future works required to develop the design are identified.

Ultimately, the developed preferred option, and associated costings, will used by the NCA as supporting documentation for full remedial project funding applications.

1.2 Scope of this report

The scope of GHD's consultancy services has been separated into a number of activities. These activities are summarised in Figure 3.

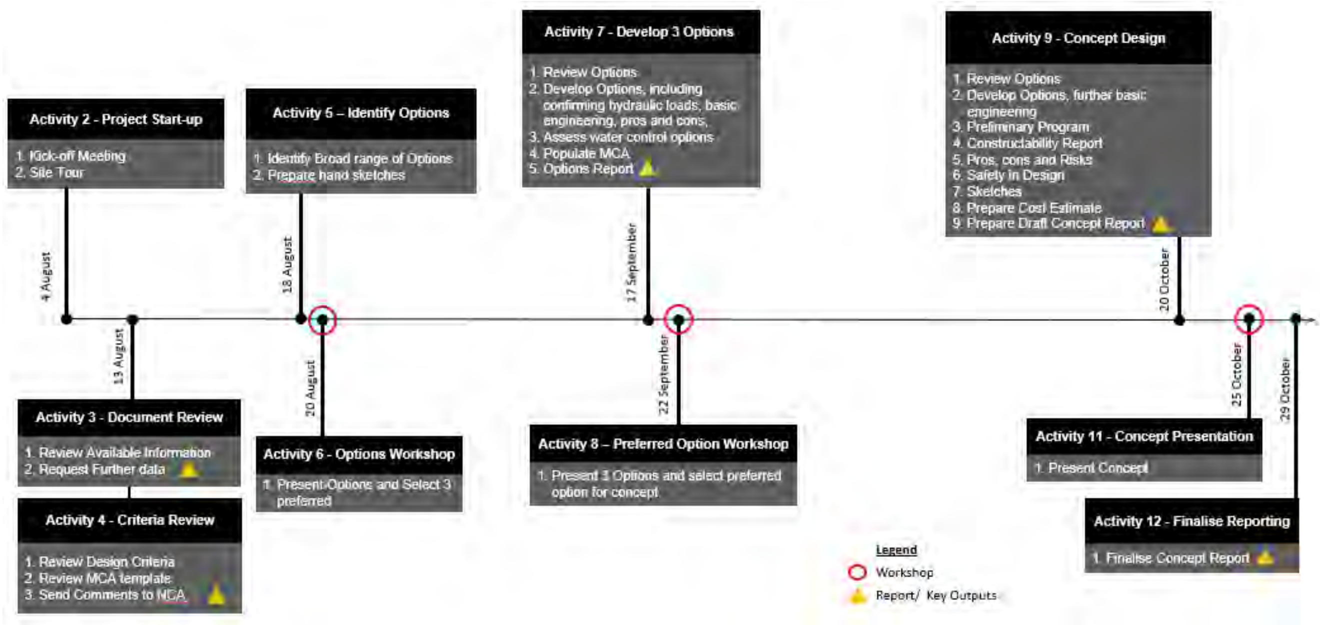

Figure 3 Proposed Design Process Flow

The activities are also summarised as follows:

– Activity 1 – Project award
– Activity 2 – Project start-up
– Activity 3 – Document review
– Activity 4 – Review of Design Criteria and Multi-criteria analysis template
– Activity 5 – Identification of options
– Activity 6 – Option Identification Workshop
– Activity 7 – Development of three preferred options
– Activity 8 – Option presentation and selection of preferred option workshop
– Activity 9 – Development of Concept Design
– Activity 10 – Collaboration with Construction specialist
– Activity 11 – Concept Design Presentation
– Activity 12 – Finalisation of the Concept Design Report.

This report presents the outcomes of Activity 1 to 8. A separate Concept Design Report will cover Activities 9 to 12.

1.3 Limitations

This report: has been prepared by GHD for National Capital Authority and may only be used and relied on by National Capital Authority for the purpose agreed between GHD and National Capital Authority as set out in section 1.1 of this report.

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The services undertaken by GHD in connection with preparing this report were limited to those specifically detailed in the report and are subject to the scope limitations set out in the report.

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The opinions, conclusions and any recommendations in this report are based on assumptions made by GHD described in this report (refer section 1.4 of this report). GHD disclaims liability arising from any of the assumptions being incorrect.

Any reports, drawings, memos, or other deliverables produced by GHD shall be produced in a traditional and generally accepted format. Accessible reports, drawings, memos, or other deliverables can be provided by GHD at an additional cost if necessary.

The opinions, conclusions and any recommendations in this report are based on information obtained from, and testing undertaken at or in connection with, specific sample points. Site conditions at other parts of the site may be different from the site conditions found at the specific sample points.

Investigations undertaken in respect of this report are constrained by the particular site conditions, such as the location of buildings, services and vegetation. As a result, not all relevant site features and conditions may have been identified in this report.

Site conditions (including the presence of hazardous substances and/or site contamination) may change after the date of this Report. GHD does not accept responsibility arising from, or in connection with, any change to the site conditions. GHD is also not responsible for updating this report if the site conditions change.

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GHD has prepared the preliminary cost estimates set out in section 6.6 of this report (“Preliminary Cost Estimates”) using information reasonably available to the GHD employee(s) who prepared this report; and based on assumptions and judgments made by GHD, as discussed in section 6.6.

The cost estimate has been prepared for the purpose of comparing the options and ranking for the multi-criteria assessment, and must not be used for any other purpose. It is noted that costings prepared in the future Concept Design stage of the project will be used for funding applications.

The cost estimate is a preliminary estimate only. Actual prices, costs and other variables may be different to those used to prepare the cost estimate and may change. Unless as otherwise specified in this report, no detailed quotation has been obtained for actions identified. GHD does not represent, warrant or guarantee that the [works/project] can or will be undertaken at a cost which is the same or less than the cost estimate.

Where estimates of potential costs are provided with an indicated level of confidence, notwithstanding the conservatism of the level of confidence selected as the planning level, there remains a chance that the cost will be greater than the planning estimate, and any funding would not be adequate. The confidence level considered to be most appropriate for planning purposes will vary depending on the conservatism of the user and the nature of the project. The user should therefore select appropriate confidence levels to suit their particular risk profile.

GHD has prepared this report on the basis of information provided by National Capital Authority and others who provided information to GHD (including Government authorities), which GHD has not independently verified or checked beyond the agreed scope of work. GHD does not accept liability in connection with such unverified information, including errors and omissions in the report which were caused by errors or omissions in that information.

1.4 Assumptions

A number of assumptions have been made during the development of upgrade options for the stilling basin at Scrivener Dam. These assumptions include:

- The dimensions used in the development of options has been based on existing drawings and survey data. It is assumed that this information is suitable for the development of options.
- The design loads proposed for the stilling basin are based on the assumption that the dam is a High C consequence category. It is noted that this consequence category has recently been reduced from a High B, based on work recently undertaken by SMEC (2021).
- The hydraulic loads used in the development of options have been based on data provided in the Physical model study report. It is understood that the physical hydraulic model was suitably calibrated and data has been adequately reviewed.
- The tailwater rating used in this option assessment has been based on that which was developed by SKM (2011) and used in the physical hydraulic model study report (WRL, 2021).
- The development of options has been based on the existing gate operating rules. It is noted that alternate gate operating rules may be adopted in the future.
- It is understood that debris loading (trees, logs etc. being flushed through the spillway) on the structure is not a significant issue at the site.
- It is assumed that the upgrade works will be constructed without full draining the storage (or with minimal drawdown).
- Specific assumptions relating to the development of cost estimates are described in Section 6.6.
- The options have been developed on the assumption that there are no significant environmental, heritage of approvals requirements for the project.

1.5 Reference Documents

A suite of reference documents have been provided to inform the development of upgrade options. These documents are summarised in Section 8.

2. Background

2.1 Scrivener Dam Design and Structure

2.1.1 Overview of the Dam

Scrivener Dam is located in the ACT, and retains one of the most famous storages in Australia, Lake Burley Griffin. The dam was originally proposed purely for aesthetic purposes and forms an integral part of the Canberra landscape, but it also serves the community for recreational purposes including non-motorised water sports. The dam is managed, operated and maintained by the National Capital Authority (NCA).

The dam is located on the Molonglo River, on Lady Denman Drive, Canberra. The reservoir has a capacity of 33,000 ML at FSL (EL 555.93 m) and a catchment area of around 1,870 km^2.

The majority of the dam is formed by a concrete gravity structure, however the left and right sides of the concrete dam are flanked by earthfill embankments. The concrete gravity portion of the structure incorporates a gated mass gravity spillway structure. The lake level is controlled by five large spillway gates (fish-belly flap gates) which fold down onto the concrete dam as the gates are opened. Flows from the spillway discharge into a USBR Type III stilling basin which has an invert at varying levels. The dam also features three low level outlets, which form an important part of managing the lake levels to the very tightly controlled +/-100 mm. A photo of the dam is provided in Figure 4.

Figure 4 Photo of Scrivener Dam from left abutment

The principal features of the storage are summarised in Table 1.

Table 1 Key Features of the Dam

Component	Item	Detail
Reservoir	Catchment Area	1870 km²
	Full Supply Level	555.93 mAHD
	Capacity at FSL	33,000 ML
Dam	Type	Concrete gravity
	Crest Elevation	560.80 mAHD (top non-overflow road level)
	Height	33 m
	Length (total) Concrete cut off walls Non-overflow section Flap gated spillway section	319 m 84 m 69 m 165 m (Including piers, 152.4 clear spillway width)
Spillway	Type of Spillway	Concrete gravity structure with gated spillway
	Discharge Capacity	8,200 m³/s (at level where spillway bridge restricts flow to orifice conditions)
	Type of Gates	Fish Belly Flap Gates
	Number gates	5 No.
	Height gates	5 m (approximate depth of water retained)
	Width gates	30.5 m
Outlet Works	Sluice Valves Number	3
	Capacity	55.8 m³/s combined when reservoir at FSL
	Gate dimensions	1.2 m x 1.2 m
Instrumentation	Piezometers	9 No.
	Fissure meters	12 No.
	Reservoir level gauge	1 No. Installed at site office

A suite of key drawings of the dam are provided in Appendix A, and a selection of photos is provided in Appendix B.

2.1.2 Concrete Overflow Section

The spillway section of the dam comprises 5 No. 30.5 m long spillway bays, with 3.05 m wide piers separating each bay (total length 165 m), as shown in Figure 5. The spillway is controlled by “fish belly” gates, which maintain the Full Supply Level (FSL) at EL 555.93 m AHD. The spillway bays are labelled 1 to 5 from right to left. The concrete section of the dam consists of a series of monoliths, separated by contraction joints. The length of the monolith blocks differ from the length of the spillway bays, and therefore the contraction joints do not align with the centre of the spillway bays or piers. The monolith blocks which make up the spillway section are referred to as Block 3 to Block 11, while Blocks 2 and 12 are partially in the spillway section and partially in the non-overflow portion of the dam.

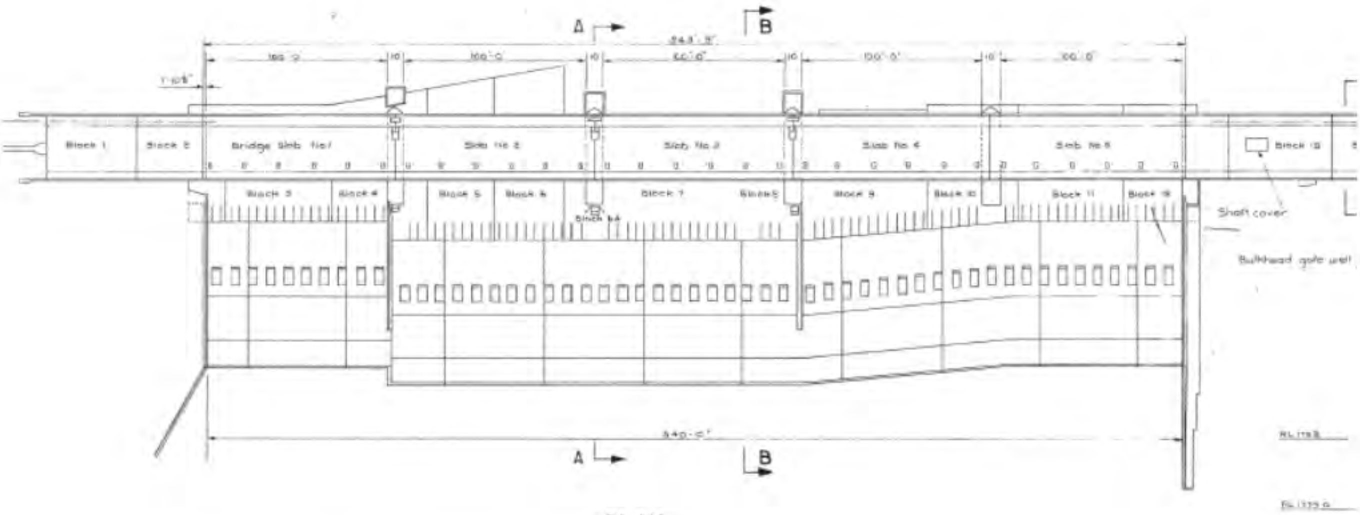

Figure 5 *Plan View of Overflow section of Scrivener Dam*

The concrete overflow section was designed to allow the spillway gates to fold onto the crest, and when fully open, the gates form an ogee-style profile on the top of the concrete section, as shown in Figure 6.

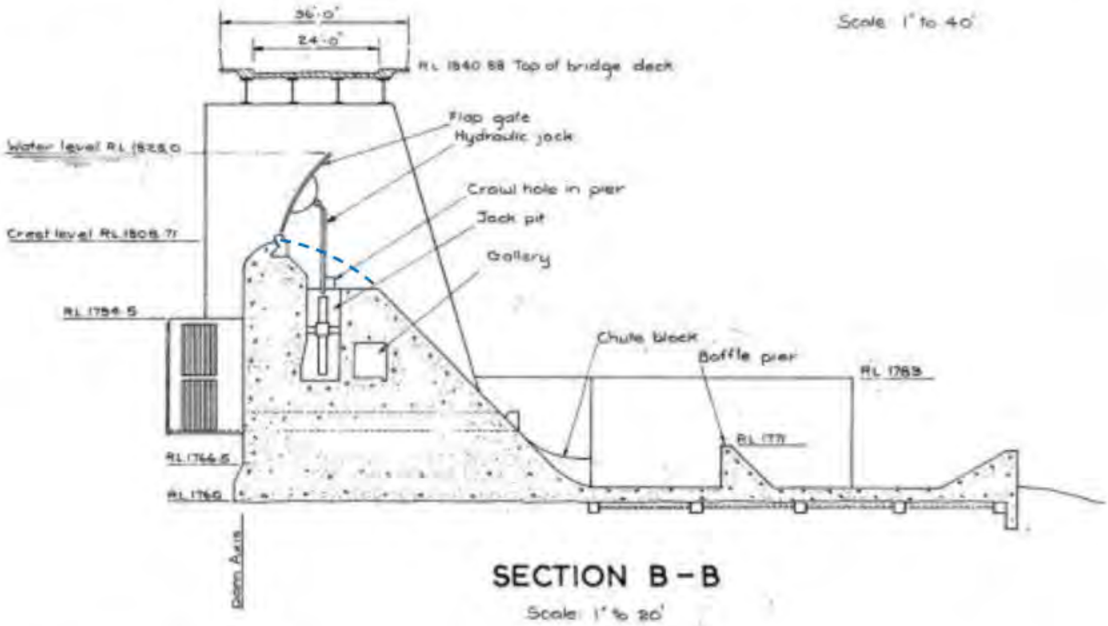

Figure 6 *Typical Overflow section*

The dam incorporates a gallery which runs the full length of the concrete section of the dam, and which houses the hydraulics, pumps and associated mechanical and electrical equipment required for the gate and outlet operation. The gallery is accessed from an adit on the upper left abutment. The gallery cross-section is 1.98 m wide and 2.13 m high except for the last 21.3 m where the width has been reduced to 1.22 m. An emergency exit from the gallery was retrofitted after construction to address recommendations made in a 1995 Design Review. The gallery drains into a sump located in the central part of the gallery which drains to the spillway basin via a submersible pump. It is noted, however that the gallery does not have a drainage function as far as the dam's foundations are concerned.

During construction of the dam, it became apparent that the central section of the spillway (namely Blocks 3, 4, 5 and 6) were founded on two significant intersecting faults. The first fault strikes across the river and dips downstream, and the second fault strikes roughly upstream-downstream and steeply dips towards the left bank.

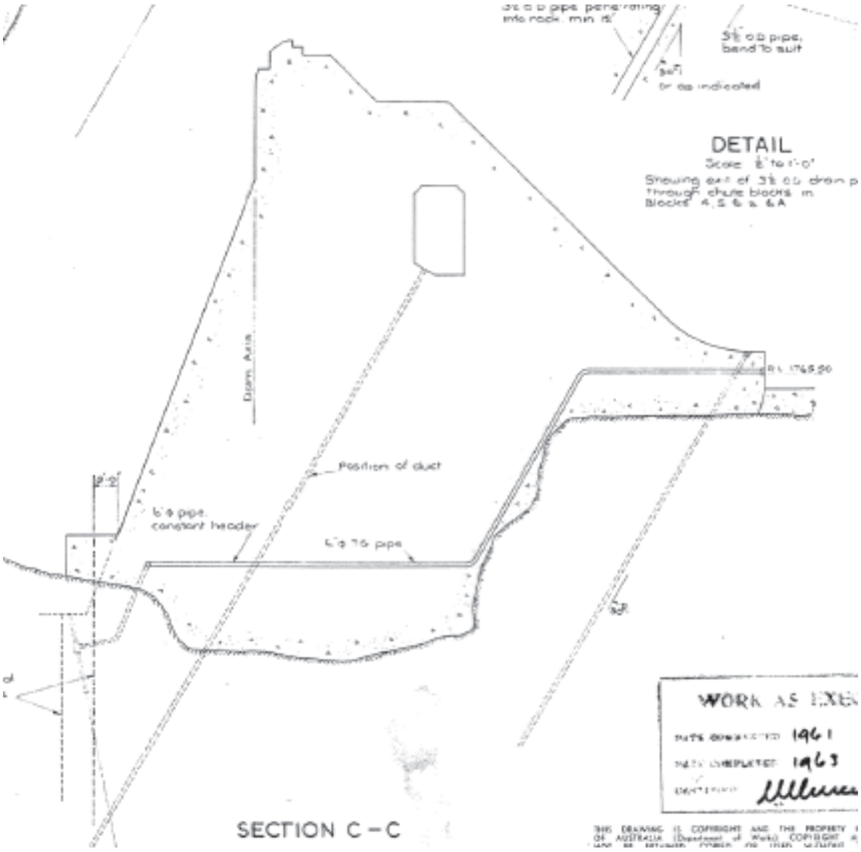

Figure 7 ***Deepening of foundation at fault***

The highly weathered and fractured rock between these faults was excavated and replaced with concrete. Due to the significant increase in depth of the foundation, post-tensioned anchors were installed through Blocks 4 to 6 to provide an increase in stability. These blocks are approximately 25m high while the remainder of the monolith blocks are generally 15 m high.

A rudimentary drainage curtain exists beneath the concrete section of the dam, as shown in Figure 8. The drainage system comprises a network of pipes set in the dam concrete and 75 mm diameter holes drilled into the foundation at the heel and toe of the dam. There are three drain holes per block near the upstream heel of the dam, and located at approximately 6.1 m spacing and 10.7m deep, inclined downstream at 23 degrees to vertical. The exception is Block 5, 6 and 6A which only have two drains per block. These drain holes are connected by header pipes and drained into the spillway basin. The toe drains were drilled vertically through the spillway chute blocks at approximately 6.10 m centres (every second chute block) to 4.57 m deep. In most cases these drains are not accessible to monitor seepage or to allow the embedded drainage pipes to be inspected and cleaned.

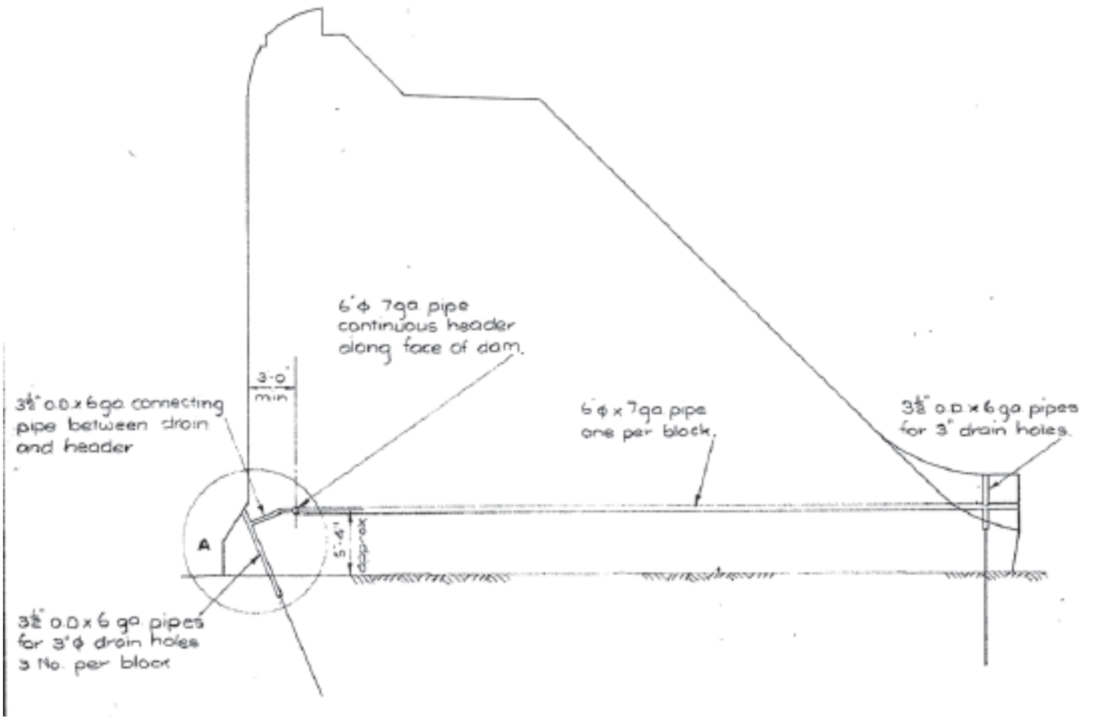

Figure 8 *Details of Foundation drainage curtain under spillway blocks excluding deepened section under blocks 4 to 6*

2.1.3 Spillway Gates

The five 'fish belly' spillway control gates are 30.48 m wide and 6.06 m chord length, and retain a depth of around 5m of water when fully closed. Seals are incorporated along the base and sides of the gates. Each gate is supported on four hydraulic jacks which are mounted on pivoting trunnions in jack pits, located below the crest of the concrete dam. Each gate pivots on hinges which attach the base of the spillway gates to the crest of the spillway. A major upgrade program on the gate hinges was undertaken around 2012 to 2014.

The hydraulic jack pump units, which operate the spillway gates, are located in the gallery of the dam. These units can be operated locally from within the gallery or from a control unit located in the entry adit to the gallery. The gates can also be operated from the Dam Office, but cannot be operated from any locations off-site.

2.1.4 Stilling Basin

Discharge from the spillway passes into a Type III USBR energy dissipator basin which discharges into the Molonglo River. The basin is 24.4 m long and 164.6 m wide, and incorporates chute blocks, baffle piers (blocks) and a downstream end sill.

The floor level of the stilling basin varies, as shown in the elevation in Figure 9. Bay 1 and 5 have an invert level of EL 540.08 mAHD, while the central, deepest, bays have a stilling basin invert level of EL 537.03 mAHD. Bay 4 has a sloping invert, varying from the level of Bay 3 to Bay 5.

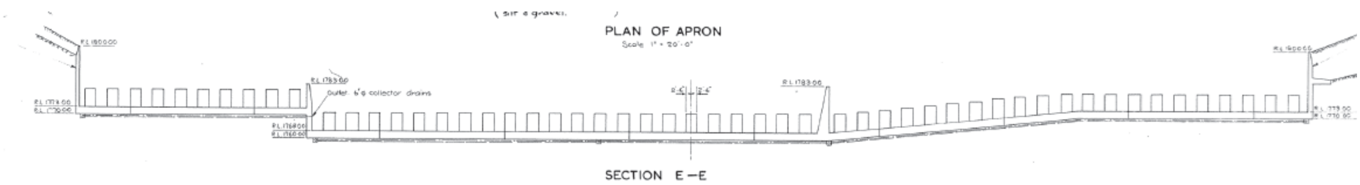

Figure 9 *Elevation of stilling basin*

The central bays are separated from the outer bays with divider walls which extend most of the length of the stilling basin. These walls have a top level of EL 543.13 m AHD which is 6.1 m above the lowest floor level of the basin. The training walls of the stilling basin extend well beyond the end sill, and have a top level of EL 548.3 mAHD which is approximately the 1 in 100 AEP tailwater level.

The stilling basin slab is 900 mm thick, and is anchored to the foundation via 28 mm bars on a 2.1m grid extending a minimum of 1.5m into rock, as shown in Figure 10. The slab is reinforced with one top layer of 12 mm bars at 300 mm spacing both ways. Upstream-downstream oriented contraction joints are located at variable spacing ranging from 13.8 to 19.8 m, and there are two cross-valley contraction joints, one located at the toe of the dam, and the other about 1.7m downstream of the central baffle blocks. None of the contraction joints in the floor slab incorporate waterstops.

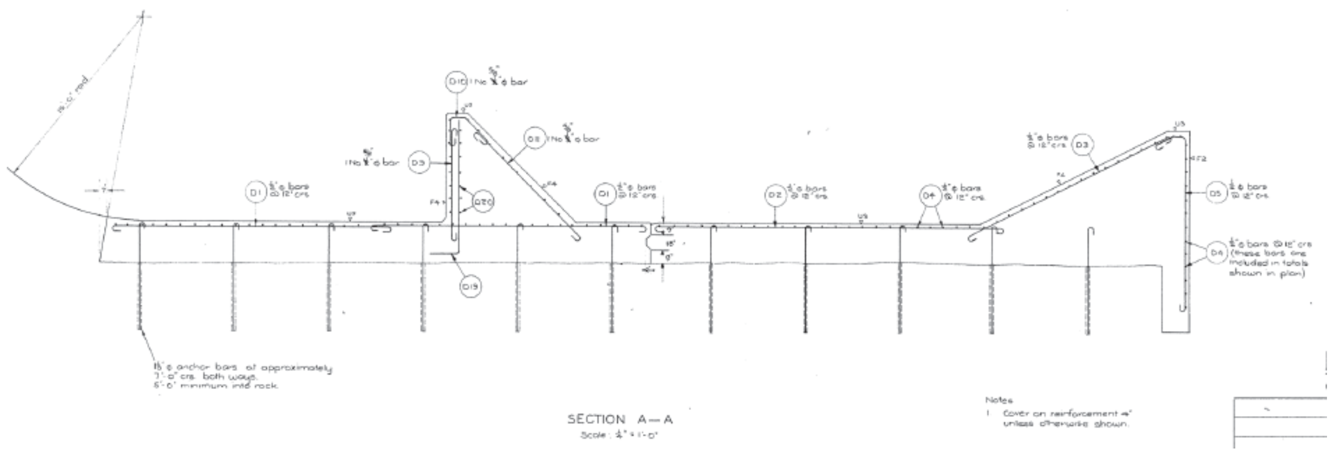

Figure 10 ***Reinforcement and anchor arrangement for stilling basin***

A subsurface drainage system was constructed beneath the slab, comprising a series of open-jointed pipes encased in no-fines concrete, as shown in Figure 11. Some of these drains are located directly under the transverse contraction joints. The stilling basin also has a 150 mm valved scour pipe through the downstream sill, although according to drawings the discharge tailpipe is raised about 1.6 m above the basin floor and hence could not fully drain the basin.

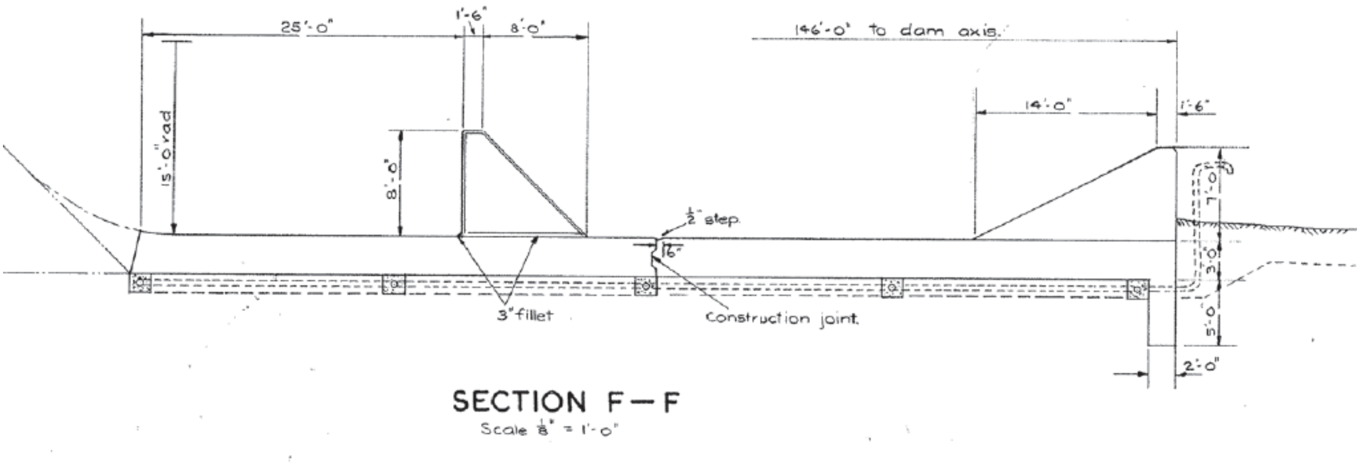

Figure 11 ***Cross-section of stilling basin showing sub-surface drainage***

2.1.5 Outlet

The three sluice outlets are located in the piers between Spillway Bays 1&2, 2&3 and 3&4. The outlets are 1200 x 1200 mm square section conduits (sluices) through the dam, controlled by hydraulically actuated sluice gates located within the body of the dam, as shown in Figure 12. Each sluice upstream intake is protected by trashracks and there is provision for the placement of a bulkhead gate (stoplog) to isolate the sluices. The total combined discharge of the three sluices is around 55.8 m³/s with a full storage.

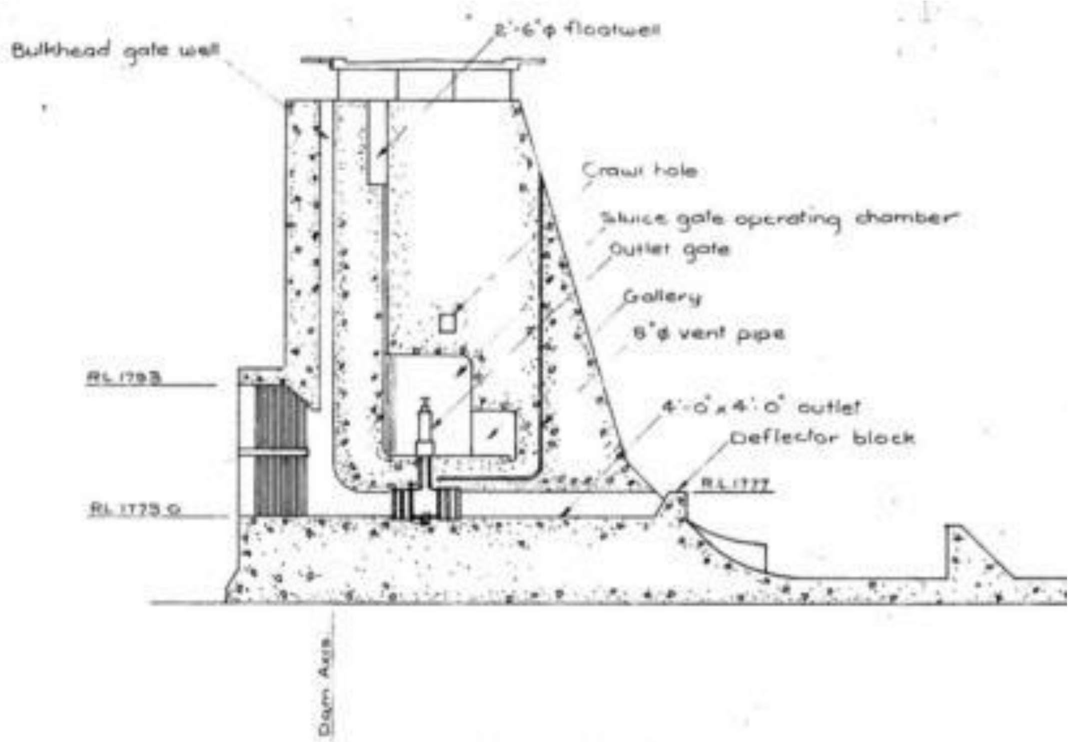

Figure 12 Typical section through outlet

Except for the sluice gates, the outlet conduits are unlined through the concrete. The conduits discharge into the central dissipator basin past a single energy dissipating baffle pier at the downstream end of each outlet. The square conduit is bell mouthed on the top at the upstream intake.

2.2 Previously identified perceived deficiencies

A number of potential deficiencies of the stilling basin have been identified in previous studies and investigations. These potential deficiencies include:

- The potential for high velocity flow to enter the open contraction joints and pressurise the underside of the stilling basin slabs; and
- The potential inadequate resistance of the stilling basin slab to buoyancy (net uplift forces), as a result of inadequate anchoring and weight of the slab arrangement, which is further exacerbated by the likely ongoing corrosion of the anchors.

During one of the routine dam safety inspections of the dam in 2015, it was observed that 'bubbling' was occurring from some of the joints in the stilling basin slab on the upper left side of the basin. Further observations were made of this bubbling over several years, however the exact cause and mechanism for this bubbling has never been fully understood. It is postulated to be related to water penetrating through the lower slab joints during operation of the sluices (and during larger spill events) which pressurises air in the sub-surface drains which subsequently releases in the higher sections of the slab floor. The observations of this bubbling highlight the fact that there are no waterstops in the contraction joints and there is a high potential for the underside of the slab to become pressurised during operation of the spillway.

In 2016, a Dam Safety Review (DSR) was undertaken on the dam and associated structures. One of the key deficiencies identified in the DSR was the lack of robustness in the design of the stilling basin. In particular, there was concern regarding the potential for uplift of these slabs due to potentially deficient and corroded anchors and lack of resistance to uplift, the lack of waterstops, and the ongoing observations of the bubbling. Due to the complex hydraulic behaviour of the stilling basin, it was necessary to develop a more comprehensive understanding of the hydraulic characteristics of the spillway using a physical hydraulic model. This study has now been completed, and has highlighted the possibility of certain flows to cause upward forces which may exceed the ability of the dissipator to withstand.

The key focus of the proposed upgrade options is to address these potential deficiencies in a manner that is appropriate for the site conditions and nature of the structure, and which will ensure the dissipator has a design life of 100 years. The recent model study (WRL, 2021) has been used as a key reference document in guiding the upgrade design.

3. Stilling Basin Upgrade Design Criteria

A set of high-level design criteria for the upgrade works on the stilling basin at Scrivener Dam have been developed. The process for developing the design criteria is summarised as follows:

– The design criteria were initially prepared by SMEC for the NCA, a copy of which was included in the Project Brief.

– In the early stages of the project, GHD reviewed the proposed design criteria, tabulated the criteria, and made suggestions on modifications to the criteria. A summary of the comments made by GHD are captured in the project 'Comments and Response Register' provided in Appendix C.

– The proposed modifications were subsequently discussed and agreed with NCA and its expert reviewers who were Gamini Adikari from SMEC and Graeme Bell (independent consultant). In later stages of the project, Mark Sinclair (SRG) was also used as an expert reviewer for the constructability and costing aspects of the project, but was not involved in reviewing the Design Criteria.

In order to develop basic dimensions for the upgrade options, it was necessary to further develop the hydraulic loading design criteria which required a detailed review of the hydraulic performance of the stilling basin. Aspects relating to the hydraulic performance of the stilling basin are discussed in detail in Section 4.

It is also emphasised that the following design criteria only provide basic details appropriate to an 'Option-level' stage of design. As the project progresses, the design criteria will need to be further reviewed and updated as necessary. The current agreed design criteria are summarised in Table 2.

Table 2 Design criteria table

Category	Sub-category	Design Criteria
Relevant Codes and Standards	ANCOLD (2000)	Guidelines on Acceptable Flood Capacity (2000 and Draft) Guidelines on Concrete Gravity Dams (2013) Guidelines on Dam Safety Management (2003)
	USACE	EM-1110-2-1602 – Hydraulic Design of Spillways EM-1110-2-2100 – Stability Analysis of Concrete Structures EM 1110-2-2104 - Strength Design of Reinforced Concrete Hydraulic Structures
	USBR	EM25 – Hydraulic Design of Stilling Basins
	Australian Standards	AS3600 – Concrete Structures AS1170 – Loading AS5100 – Bridge Design Code
Hydraulic Design Criteria	Energy Dissipation / Hydraulic Performance	The energy dissipation characteristics of the stilling basin need to achieve equal or better hydraulic performance (and exit velocities) than the existing basin, The stilling basin must display equivalent hydraulic performance to the existing structure. This may be demonstrated with empirical designs based on USBR. Future design will consider numerical and physical modelling.
	Hydraulic Load on the Stilling Basin	The adopted concept must be able to withstand the imposed loading in accordance with relevant guidelines such as USBR and USACE including: – Transient uplift loading: SMEC Pty Ltd undertook a model study to which resulted in prototype transient loading of the existing stilling basin structure. – Pressurisation through Contraction joints: Potential for positive pressure leakage directly into the foundation drainage system and into the jointed foundation – Static uplift – Impact pressure
	Design Flood Events	The adopted concept must demonstrate its resilience to all flood levels up to and including the design flood for a "High C" consequence category dam. In accordance with the fallback requirements for Acceptable Flood Capacity (AFC) (ANCOLD, 2000), it is proposed that the AFC should be between 1 in 100,000 to PMP-DF (PMPF).

Category	Sub-category	Design Criteria
		With this classification the proposed usual, unusual and extreme load cases are proposed to be: – Usual loading: 1:50 AEP flood (1500 cumecs outflow) – Unusual loading: 1:2,000 AEP flood (4,300 cumecs outflow) – Extreme loading: 1:100,000 AEP flood (8,200 cumecs outflow)
Structural/ Stability Design Criteria	Hydraulic Loading	As per Hydraulic design criteria
	Structural Adequacy	The stilling basin must meet acceptance criteria for strength and stability in accordance with USACE and Australian Standards.
	Vibration	The analysis must consider the potential for vibration as the structure responds to the imposed dynamic loading (noted in the SMEC Modelling Report p36).
	Waterstops	The waterstop configuration must be considered and addressed to prevent high pressures jets from pressurising through the joints and creating high uplift pressures beneath the slabs.
	Dam Stability	Alterations to the stilling basin may result in changes to the stability of the dam due to changes in uplift forces. The stability of the dam must be assessed, and any strengthening requirements must be included in the concept design. Mitigation of alterations to the main dam should be sought as a priority. The stability of the main dam must be considered and must comply with the ANCOLD guidelines.
Construction	Constructability	The adopted concept must be "buildable", preferable without the need to drain or drawdown the level of the reservoir. The adopted concept must be able to be constructed without compromising safety, the dam or reservoir – even if flood events occur during the construction phase.
	CAPEX Estimate	An AACE Class 4 Cost Estimate (15% Design) is to be produced for the Concept Design Report.
Operation and Maintenance	Operational Aspects	The upgrade option must minimise the need for ongoing operational intervention on the selected option.
	Maintenance Aspects	The upgrade option must minimise the need for ongoing maintenance. Consideration should be given in the design of the need to inspect critical components of the works.
	Design Life	100 years
Other	Dam Safety	Options must not impede the safety of the dam.
	Latent conditions	Relevant latent conditions of the dam must be considered and addressed.
	Fishway	No specific design criteria is required for fish passage
Regulatory	ACT Dam Safety Regulations	ACT Dam Safety Regulations are to be met in the design of the upgrade works.

4. Review of Hydraulic Performance

4.1 General

A high-level review has been undertaken of the hydraulic performance of the stilling basin. The review has focused on two distinct aspects, namely:

– The overall hydraulic performance and ability to dissipate energy (refer to Section 4.5)
– The likely hydraulic loadings applied to the stilling basin during the range of events (refer to Section 4.6)

In addition to these items, this section provides details on:

– Previous assessments undertaken on the hydraulic performance of the dam
– Summary of the proposed design floods for the stilling basin
– Summary of the tailwater levels used in the assessment.

It is noted that the discussions provided in this section should be read in conjunction with SMEC (2021) and WRL(2021).

4.2 Previous Assessments of Hydraulic Performance

The hydraulic performance of the stilling basin has been studied a number of times, including:

– Original physical model studies which included
    - 1:72 scale model of the full reservoir and dam, used to investigate the stilling basin. This model was developed by the Commonwealth Department of Works.
    - 1:24 scale model of the sluices, used to investigate the outlets. This model was developed by the Commonwealth Department of Works.
    - 1:32 scale flume model study undertaken by the gate manufacturer. This model was developed for the gate manufacturer by the Technical University of Karlsruhe.

  Less sophisticated instruments were used in these studies limiting the ability to understand pressure transients, and therefore the SMEC (2021) and WRL (2021) work has been used in the current review.

– Sunwater (2009) - A 1:40 scale physical hydraulic flume model study was undertaken by Sunwater in 2009, of one gate bay. Similar to the original model study, the recent model study is considered to be considerably more advanced than the Sunwater study, and therefore the current review is largely based on the recent model study.
– SMEC (2016) - As part of the Design Review of Scrivener Dam undertaken by SMEC in 2016, a review of the performance of the stilling basin was undertaken. This review included assessing the performance of the crest hydraulics and the stilling basin, and was largely based on empirical formula, reviews of earlier model studies and reviewing video and photographic footage of the stilling basin during recent floods.
– Most recently, the physical hydraulic model study undertaken by WRL and reported on in WRL (2021) and further reported on in SMEC (2021).

Key findings, specifically from the 2016 and 2021 studies, are documented in the following sections.

4.3 Design Floods

In 2021, SMEC undertook a review of the catchment hydrology to update the flood frequency curve for the storage. A summary of the flood frequency outputs from this analysis is provided in Figure 13.

AEP (1 in X)	Peak Discharge (m^3/s)		Critical Outflow Duration (hours)	Water Level (mAHD)
	Dam Inflow	Dam Outflow		
10	703	703	48	555.93
20	1,053	1,053	48	555.93
50	1,541	1,541	48	555.93
100	1,960	1,960	48	555.95
200	2,367	2,366	36	555.95
500	3,083	3,083	24	555.96
1,000	3,626	3,606	24	556.03
2,000	4,333	4,295	24	556.48
5,000	5,350	5,314	24	557.22
10,000	6,208	6,157	24	557.74
20,000	6,876	6,804	24	558.18
50,000	7,747	7,679	24	558.81
100,000	8,274	8,198	24	559.15
200,000	8,939	8,770	24	559.78
540,000	10,011	9,695	24	560.90
PMF	10,970	10,440	36	562.11

Figure 13 Flood Frequency Data (Extract from SMEC, 2021)

The original stilling basin was designed for a peak discharge of 5,660 m^3/s (SMEC, 2016). Previous assessments indicate that the spillway will enter orifice flow when the upstream reservoir level is at approximately EL559.9 m AHD (due to the constriction created by the spillway bridge).

The dam is now classified as a High C Consequence Category dam, in accordance with ANCOLD guidelines. The fallback requirements for Acceptable Flood Capacity (AFC) (ANCOLD, 2000) for a High C dam is between 1 in 10,000 to PMPDF or the 1 in 100,000 AEP flood (whichever is less). In the case of Scrivener Dam, the PMPDF is 1 in 540,000 AEP, therefore the upper limit of the fallback criteria is taken to be the 1 in 100,000 AEP. This upper limit is also consistent with the ANCOLD (draft, 2016) guidelines which recommend an upper limit of 1 in 100,000 AEP for High C dams. The outflow of the 1 in 100,000 AEP is around 8,200 m^3/s, with a corresponding reservoir level around EL559.15 m AHD.

ANCOLD (2013) provides guidance for flood design cases to be adopted for concrete gravity dams. While this criteria is largely irrelevant for the stilling basin (typically stilling basins are not designed for usual and unusual events), the following load cases are consistent with the Gravity Dam guidelines:

- Usual loading: 1:50 AEP flood (1500 m^3/s outflow)
- Unusual loading: 1:2,000 AEP flood (4,300 m^3/s outflow)
- Extreme loading: 1:100,000 AEP flood (8,200 m^3/s outflow)

The key item to note is that the proposed design event (8,200 m^3/s) is considerably larger than the original design flood, but slightly less than the events reviewed as part of the recent physical model study, which assessed discharges up to 9500 m^3/s. The reservoir level for the 8,200 m^3/s flood, at EL559.15, is also less than the assessed level of 559.9, at which orifice flow commences.

It is also noted that previous assessments of the concrete dam indicate that the stability of the dam does not meet acceptance criteria for the 1 in 100,000 AEP event.

4.4 Tailwater Levels

In 2011, SKM undertook a hydrology and dambreak assessment of Scrivener Dam. As part of this study, the tailwater rating curve was presented, as detailed in Table 3. As part of the physical model study, sensitivity analysis was undertaken on the impacts of the tailwater levels, and various runs were undertaken with the tailwater level 1m, 2m, and 4m lower than the values shown in Table 3.

Table 3 Tailwater Rating Curve

Total River Flow - m^3/s	Tailwater Elevation - RL mAHD
1.1	538.9
10.0	539.3
20.8	539.6
46.0	540.1
52.8	540.2
124.3	541.2
215.8	542.1
372.3	543.0
532.7	543.9
714.2	544.7
892.9	545.4
1148.9	546.3
1449.6	547.1
1899.9	548.1
2736.7	549.7
3228.6	550.6
3726.9	551.3
4974.6	553.1
5530.5	553.8
6380.8	555.0
7355.1	556.0
7675.4	556.5
8281.7	556.9

4.5 High-level Review of Hydraulic Performance

4.5.1 General

The overall hydraulic performance of a stilling basin will be reliant on several factors. These include:

- Energy Dissipation - The energy dissipating performance of the stilling basin needs to be sufficiently effective to ensure that discharges leaving the concrete-lined basin have low enough velocities (and stream power) to minimize the potential for downstream erosion, especially erosion near the downstream end of the stilling basin that might lead to an undermining of the basin floor. The acceptability of the energy dissipation characteristics, to an extent, will depend on the likely erodibility of the material in the river channel downstream of the stilling basin versus the exit velocities.
- Containment of the Flow – The concrete-lined stilling basin structure needs to have high enough training walls to be able to adequately contain the highly turbulent, high velocity discharges from the spillway so that the toe of the dam either side of the stilling basin is not threatened, for example, by undermining for flows up to the peak design discharge.
- Potential for undesirable recirculating flows – Many stilling basins will experience some form of recirculation flows in which flows exiting the concrete basin return back towards the spillway in a circular motion. These flow patterns have the potential to cause damage by bringing loose gravel/stones back into the basin, or by circulating towards the unprotected toe of the adjacent embankment dam. For example, if the training walls at either side of the stilling basin are not adequately 'tied into' the banks of the river at the downstream end, recirculation flows have the potential to erode material behind the training wall, back towards the dam toe. Furthermore, recirculation flows leaving the basin have the potential to mobilise loose rock downstream of the concrete-lining and carry material back into the basin. Any debris carried back into the concrete-lined stilling basin has the potential to damage the concrete lining (e.g. ball-milling), especially at the bigger floods.

The performance of the Scrivener Dam stilling basin with regard to these factors is discussed in the following sub-sections.

4.5.2 Energy Dissipation

4.5.2.1 USBR Empirical Assessment

The Scrivener stilling basin is based on a typical USBR Type III arrangement comprising chute blocks, baffle blocks and an end sill. SMEC (2016) undertook a review of the arrangement of the Scrivener stilling basin against USBR design recommendations with particular emphasis on determining whether the existing stilling basin meets design recommendations, for a full range of discharges. The key findings were:

- The expected velocities at Scrivener Dam (inflow velocities in the order of 15 to 17 m/s) are at the upper limit of those recommended for a USBR Type III basin (e.g. 15 to 18 m/s), and the specific (unit) discharge (up to 56 $m^3/s/m$ for a discharge of 8500 m^3/s) is well above the recommended limit of 20 $m^3/s/m$ (200 ft^2/s).
- The Froude number at the design flood (around 3.0) is lower than recommended for a USBR Type III basin (i.e. 4.0). This may result in a weak and unstable formation of a hydraulic jump, and lead to excessive turbulence and wave action beyond the concrete-lined stilling basin.
- The tailwater depth was reviewed to determine whether adequate 'sequent depth' in the tailwater exists to form the hydraulic jump. The analysis found that the central bays (with the deeper stilling basin invert) need to be operated preferentially to ensure adequate tailwater depth downstream of the storage. If the current gate operating rules are used, Bay 4 will not be operated until the discharge exceeds 320 m^3/s, and the outer bays (1 and 5) will not operate until the discharge exceeds 900 m^3/s. It is noted that SMEC (2021) proposed an alteration to the gate operating rules (discussed later – refer to Section 4.5.4), which would see Bays 1 and 5 operate when discharges exceed 390 m^3/s. For both the existing and proposed gate operating rules, there should be adequate available tailwater depth to form the hydraulic jump. The only exception to this situation would occur if one (or more) of the central gates is unable to be operated due to maintenance or gate failure, in which case the outer gates may need to be operated at smaller discharges. A summary of the review of tailwater/sequent depth is provided in Figure 14 (extract from SMEC, 2016).

Total Discharge	Nº Gates	Specific Discharge	Inflow Velocity	Froude Number	Sequent Depth	Available Tailwater	
Q (m^3/s)		q ($m^3/s/m$)	V_1 (m/s)	Fr	y_2 (m)	$y_{(central)}$ (m)	$y_{(outer)}$(m)
140	1	5	16	9.8	3.8	4.3	1.2
160	2	3	15	11.6	2.8	4.8	1.7
320	2	5	16	9.3	4.0	6.4	3.4
360	3	4	15	9.4	3.3	6.8	3.7
900	3	10	16	6.4	5.3	8.9	5.8
960	4	8	16	7.0	4.8	9.0	6.0
1600	4	13	16	5.6	6.1	10.4	7.3
2000	5	13	16	5.6	6.1	10.9	7.9
3250	5	21	16	4.3	7.6	12.3	9.3
5660	5	37	16	3.4	10.0	14.5	11.4
8495	5	56	17	3.0	12.3	16.8	13.8

** In cases where the sequent depth Y_2 is well exceeded by tailwater, V1 may be reduced to below the value shown.*

Figure 14 Hydraulic parameters at stilling basin for flood discharges within standard gate operating regime, extract from SMEC, 2016

4.5.2.2 Exit velocities

As part of the recent model study (WRL, 2021), exit velocity measurements were taken immediately downstream of the concrete-lined stilling basin to provide an indication of the likely energy dissipating performance of the stilling basin under a range of discharge, tailwater and gate operating conditions. The location of the velocity measurement points is shown in Figure 15.

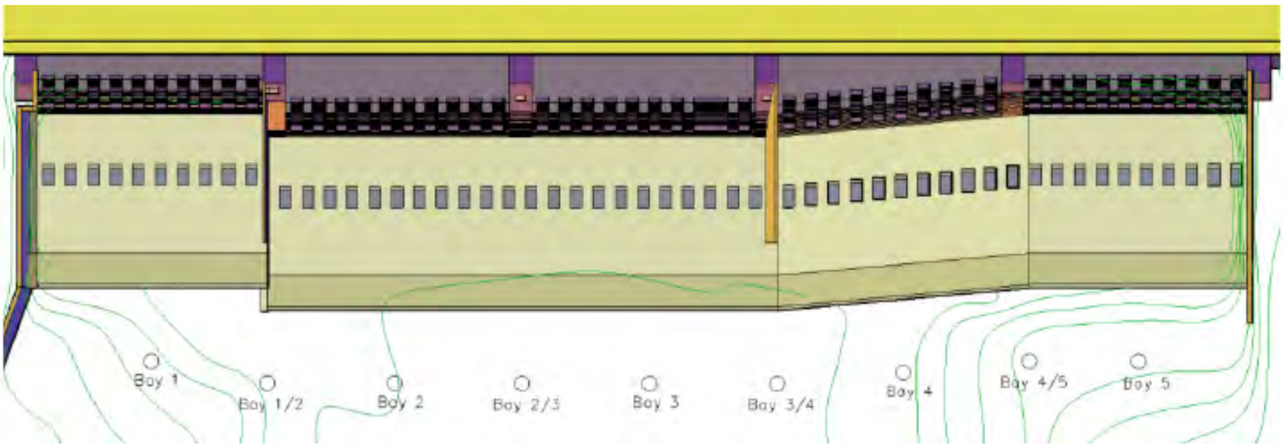

Figure 15 Location of Velocity measurements (WRL, 2021)

Table 4 provides a summary of the velocity measurements for various discharges using the 'best estimate' of tailwater levels and the current gate operating rules. The results indicate the following key aspects:

– The exit velocities for discharges up to 5000 m^3/s are typically around 1 to 2 m/s, and in all but four occurrences are less than 3.5 m/s.
– The four cases where exit velocities are greater than 3.5 m/s for discharges 5000 m^3/s or less, all occur in the parts of the stilling basin with higher inverts (i.e. Bay 1, 4 and 5) (refer to Test B03-Bay4/5, B04-Bay4/5, B06-Bay1 and B06-Bay5). In the case of Test B02, Bay 5 is not yet operating, and as such the peak velocity of 5.03 m/s between Bay4/5 and the 2.49 m/s in Bay 5 is a result of recirculation flow
– Velocities are highest for the extreme events, with velocities greater than 6 m/s for discharges of 7,500 and 9,000 m^3/s.
– Velocities are typically higher for Bays 1, 4 and 5 than corresponding velocities in Bays 2 and 3 for the same events. In other words, the high invert level of the stilling basin has an adverse impact on the resulting exit velocities (as to be expected).
– In the case of the extreme events with discharges of 7,500 m^3/s or greater, the peak exit velocities downstream of Bay 2 are surprisingly similar in magnitude to those downstream of Bay 5. It is unclear why Bay 2 starts to experience such high exit velocities, which in the case of Test B08 are double those experienced downstream of Bay 3.

Table 4 Velocity Measurements for Flows with 'best estimate' tailwater levels and current gate operating rules

Test Conditions		Gate Setting % (1)					Flow Velocity m/s (prototype)								
Test	Q*	1	2	3	4	5	Bay 1	Bay 1/2	Bay 2	Bay 2/3	Bay 3	Bay 3/4	Bay 4	Bay 4/5	Bay 5
B01	900	100	55	55	55	100	0.84	0.97	1.94	0.94	2.41	1.76	1.93	-	-
B02	1600	47	47	47	47	100	1.61	0.72	1.61	1.06	1.84	2.25	2.8	5.03	2.49
B03	1900	47	47	47	47	100	1.34	1.29	2.03	1.68	1.56	1.54	3.3	3.01	2.22
B04	2000	47	47	47	47	47	2.26	1.3	1.65	0.86	1.64	1.58	2.87	4.18	2.05
B05	3000	33	33	33	33	33	2.61	0.46	1.01	0.33	0.55	1.49	2.7	3.37	2.85
B06	5000	0	0	0	0	0	3.84	0.93	2.21	1.28	2.11	1.45	3.23	2.25	4.23
B07	7500	0	0	0	0	0	4.61	1.07	5.83	4.02	4.94	5.03	4.96	5.05	6.91
B08	9500	0	0	0	0	0	4.24	1.48	6.91	3.57	3.67	2.06	3.69	4.87	6.77

1) Represents percentage closed

In addition to the standard suite of tests undertaken for the existing conditions, a suite of sensitivity tests was undertaken with the tailwater level at 1m, 2m, and 4m lower than the estimated tailwater level, assuming the existing gate operating rules. The results of these tests are provided in Table 5. The key findings are:

– While the reduction in tailwater would be expected to result in less effective dissipation, and potentially inadequate formation of a hydraulic jump, this effect is not reflected in the velocity results. In fact, in some cases the highest tailwater level results in the highest velocities (e.g. refer to 7500 m^3/s cases).
– The highest velocity observed throughout the tests occurred for the 9500 m^3/s discharge, with an exit velocity of 8.34 m/s being measured downstream of Bay 5 with a tailwater level 2 m lower than the current estimated tailwater level. This peak velocity was most likely influenced by the geometry of the rock outcrop downstream of Bay 5.

Table 5 Velocity Measurements for Flows with different tailwater levels and current gate operating rules

Test Conditions			Gate Setting % (1)					Flow Velocity m/s (prototype)								
Test	Q*	TW	1	2	3	4	5	Bay 1	Bay 1/2	Bay 2	Bay 2/3	Bay 3	Bay 3/4	Bay 4	Bay 4/5	Bay 5
B03	1900	E	47	47	47	47	0	1.34	1.29	2.03	1.68	1.56	1.54	3.3	3.01	2.22
B09	1900	-1	47	47	47	47	47	1.58	0.52	2.42	1.58	2.45	2.84	3.99	5.05	2.46
B10	1900	-1	100	47	47	47	100	2.83	1.65	2.19	0.93	2.12	2.13	3.21	-	-
B13	1900	-2	100	47	47	47	47	0.13	0.43	2.27	1.73	2.81	3.21	5.03	5.18	2.02
B14	1900	-2	47	47	47	47	100	3.37	2.43	2.73	1.64	2.58	2.61	3.18	1.59	-
B06	5000	E	0	0	0	0	0	3.84	0.93	2.21	1.28	2.11	1.45	3.23	2.25	4.23
B17	5000	-2	0	0	0	0	0	3.35	0.86	2.06	1.82	1.9	1.46	4.3	4.46	4.37
B18	5000	-4	0	0	0	0	0	4.17	1.59	2.75	3.16	2.76	2.13	4.69	6.16	3.42
B07	7500	E	0	0	0	0	0	4.61	1.07	5.83	4.02	4.94	5.03	4.96	5.05	6.91
B11	7500	-2	0	0	0	0	0	3.88	0.78	4.64	3.72	3.99	3.39	3.69	4.64	6.43
B15	7500	-4	0	0	0	0	0	4.46	1.43	3.7	3.58	3.77	3.2	5.06	4.88	5.39
B08	9500	E	0	0	0	0	0	4.24	1.48	6.91	3.57	3.67	2.06	3.69	4.87	6.77
B12	9500	-2	0	0	0	0	0	5.04	1.84	6.43	4.78	6.29	5.89	5.58	7.25	8.34
B16	9500	-4	0	0	0	0	0	4.39	1.23	5.89	4.29	5.57	4.7	4.59	5.7	6.6

1) Represents percentage closed

Additional runs were also undertaken to assess the influence that the gate operating rules may have on the peak velocities observed in Bays 1 and 5. The results show that a substantial reduction in exit velocities can be achieved by opening the outer gates earlier in the operating rules, for discharges up to the point where all gates are fully open, as shown in Table 6. Opening the outer gates earlier in the sequence reduces the high velocities in the recirculating flows at the outer bays. In particular, the following is noted:

– Tests D03, B02 and D11 range in discharges from 1,450 to 1,700 m^3/s. Gate operating rules used for the 1,450 m^3/s (Test D03) and 1,700 m^3/s (Test D11) runs used proposed rules with outer gates opened earlier in the operating rules. Conversely, the middle discharge of 1,600 m^3/s (Test B02) adopted the existing operating rules. The velocity results for these tests show that the peak velocities observed in Bay 1 of 1.61 m/s , Bay 4/5 of 5.03 m/s and Bay 5 of 2.49 m/s for the existing rules (Test B02), all reduce in Tests D03 and D11, when the gates in Bay 1 and 5 are opened earlier in the operating rules.

– Similar results were observed at the 1,900 m^3/s discharge (refer Tests B03 and D08), and around the 2,900-3,000 m^3/s discharge (refer Tests D10 and B05) where peak exit velocities downstream of Bay 1 and 5 considerably reduced with the proposed gate operating rules.

Table 6 Velocity Measurements for existing and proposed gate operating rules

Test Conditions			Gate Setting % (1)					Flow Velocity m/s (prototype)								
Test	Gate Sequence	Q*	1	2	3	4	5	Bay 1	Bay 1/2	Bay 2	Bay 2/3	Bay 3	Bay 3/4	Bay 4	Bay 4/5	Bay 5
B01	Existing	900	0	55	55	55	100	0.84	0.97	1.94	0.94	2.41	1.76	1.93	-	-
D05	Proposed	900	74.5	63	63	63	74.5	0.43	0.28	2.15	0.63	2.1	1.41	2.18	-	-
D06	Proposed	1050	74.5	59	59	59	74.5	0.29	0.21	2.24	0.5	2.22	1.62	1.93	-	-
D12	Proposed	1150	74.5	55	55	55	74.5	0.2	0.17	2.21	1.13	2.13	1.84	2.1	1.77	0.07
D03	Proposed	1450	74.5	47	47	47	74.5	1.34	1.29	2.03	1.68	1.56	1.54	3.3	3.01	2.22
B02	Existing	1600	47	47	47	47	100	1.61	0.72	1.61	1.06	1.84	2.25	2.8	5.03	2.49

Test Conditions			Gate Setting % (1)					Flow Velocity m/s (prototype)								
Test	Gate Sequence	Q*	1	2	3	4	5	Bay 1	Bay 1/2	Bay 2	Bay 2/3	Bay 3	Bay 3/4	Bay 4	Bay 4/5	Bay 5
D11	Proposed	1700	73	40	40	40	73	0.22	0.3	2.12	1.03	1.84	1.73	2.72	0.85	0
B03	Existing	1900	47	47	47	47	0	1.34	1.29	2.03	1.68	1.56	1.54	3.3	3.01	2.22
D08	Proposed	1900	71	33	33	33	71	0.1	0.12	1.7	1.52	1.97	2.56	3.52	0.08	0
B04	Existing	2000	47	47	47	47	47	2.26	1.3	1.65	0.86	1.64	1.58	2.87	4.18	2.05
D09	Proposed	2150	63	29	29	29	63	0.23	0.2	1.98	2.13	1.68	1.69	3.22	1.29	1.26
D10	Proposed	2900	55	0	0	0	55	0.72	0.23	1.95	1.74	2.19	1.28	3.78	0.88	1.14
B05	Existing	3000	33	33	33	33	33	2.61	0.46	1.01	0.33	0.55	1.49	2.7	3.37	2.85

1) Represents percentage closed

4.5.2.3 Removable bed scour profiles

During the 2021 Physical Model Study, a removable bed downstream of the stilling basin was used to assess potential scour profiles downstream of the stilling basin for discharges of 7,000 m^3/s and 9,500 m^3/s. To undertake these tests, a portion of the concrete-lined riverbed of the physical model was removed and replaced with erodible material comprising 10 mm and 20 mm aggregate for tests undertaken with 7,000 m^3/s and 9,500 m^3/s discharge respectively. The gravels were spray painted in bands of different colours to aid with visual assessments of the scour and accretion, as shown in Figure 16. It is noted that a removable bed was not installed downstream of Bay 5, and only partially extended downstream of Bay 1. Each scour test was run for 1 hour.

Figure 16 Removable bed scour test, prior to run (note spray-painted gravels)

The results of the scour tests are shown in heat maps in Figure 17 and Figure 18.

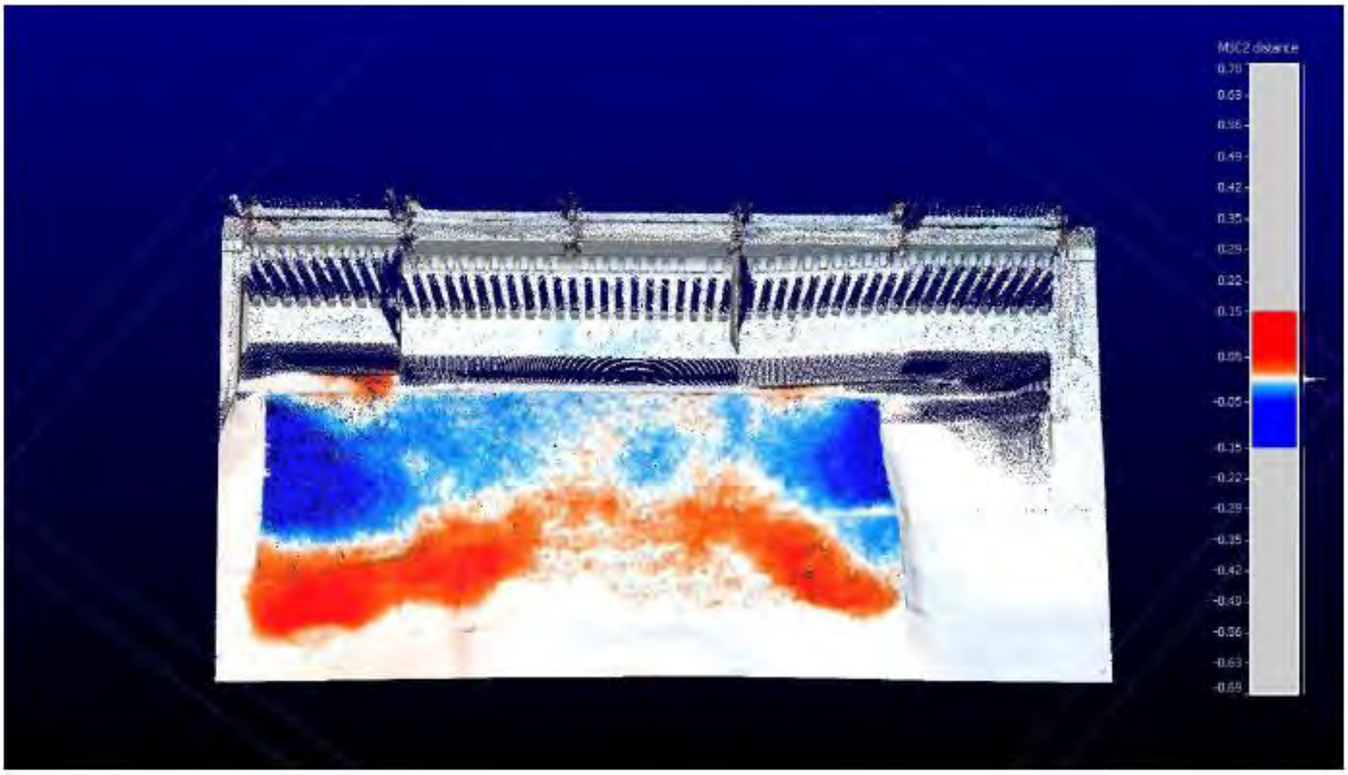

Figure 17 (2021) ***Heat map showing areas of scour (blue) and accretion (red) for 7,000 m^3/s discharge – Test F02) – extract from WRL***

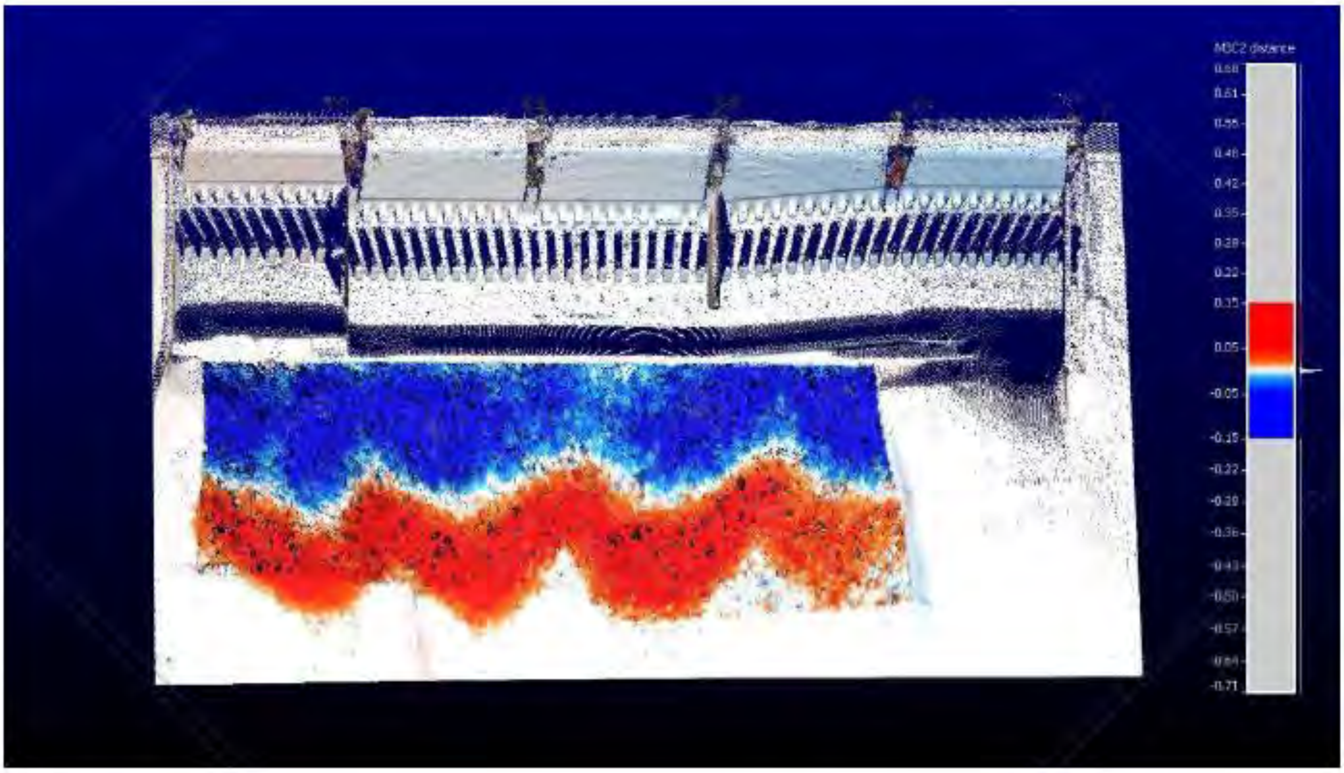

Figure 18 (2021) ***Heat map showing areas of scour (blue) and accretion (red) for 9,500 m^3/s discharge – Test F01) – extract from WRL***

Cross-section scour profile plots were produced for the two runs from the lower invert level of the stilling basin. The worst case scour profiles are shown in cross-sectional plots in Figure 19. Assuming the downstream material has similar erodibility characteristics at scale to the gravel material, the tests suggest that a scour hole in the order of 3-4 m depth below the lower end sill level may develop, suggesting a potential for undermining under high flood levels.

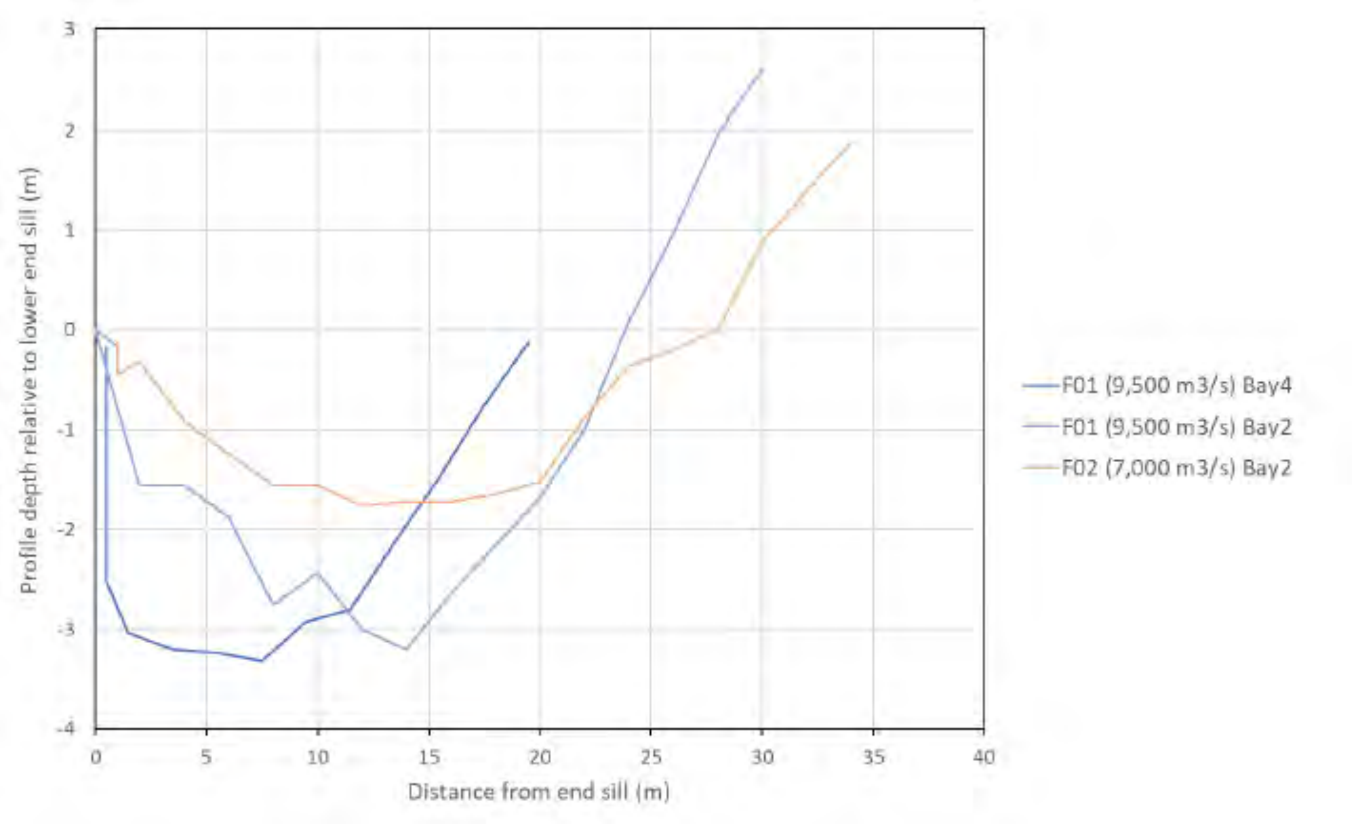

Figure 19 ***Worst case scour profiles (cross-sections downstream of concrete-lined stilling basin) - extract from WRL (2021)***

It is highlighted that the removable bed did not extend downstream of Bay 5 as there was less concern regarding the erodibility of the rock downstream of Bay 5 compared to the lesser known/understood geological conditions downstream of Bays 1 to 4. Nevertheless, given that high exit velocities were observed downstream of Bay 5 (refer to Test B07 and B08 in Table 4), it is possible that erosion would occur downstream of Bay 5. A better understanding will be required of the actual erodibility of material downstream of the stilling basin in the prototype to understand the applicability of the removable bed scour tests.

4.5.2.4 Conclusions of Energy Dissipating Performance

Key conclusions from the review of the data presented in Sections 4.5.2.1 to 4.5.2.3 include:

– The review of performance using USBR empirical assessments suggests that the outer gates have inadequate available tailwater downstream of the dam to ensure effective energy dissipation for discharges less than 360 m^3/s. The current and proposed gate operating rules require the central gates to be operated preferentially up to this discharge, and therefore there should be adequate tailwater to form a hydraulic jump under all conditions.

– The proposed gate operating rules appear to result in a considerable improvement (reduction) in downstream exit velocities, and in reducing the potential for recirculation flows. As such, these changes should be considered for implementation.

– Discharges greater than 7,500 m^3/s result in relatively high exit velocities. While these velocities may result in downstream scouring, it is likely that these high velocities will be limited in duration, and also given their very low frequency of occurrence, will therefore be acceptable. At this stage, it appears that the existing stilling basin geometry is performing relatively well in terms of dissipating energy, but further review will be required following a thorough geological assessment of the site.

– The scour profiles provide an indication of the possible depth of scour holes downstream of the basin. It is noted that the actual scour profiles will be influenced by many factors which are not easily captured in the physical model, and therefore should be considered as indicative only. Once a thorough geological assessment has been undertaken, the scour profiles should again be reviewed.

4.5.3 Containment of Flow (water surface profile)

It is common practice to install training walls either side of a stilling basin to contain discharges from the spillway, however in some cases, full containment would require extremely high training walls. In the case of Scrivener Dam, the training walls contain discharges for frequent events (up to around the 1 in 100 AEP), but extreme discharges have an associated tailwater level higher than the training walls.

The existing training walls either side of the stilling basin have a top elevation of RL1800 ft, or EL 548.31 mAHD, which is around 8.23 m higher than the stilling basin slab invert in Bay 1 and 5, as shown in Figure 20. When comparing this level to the tailwater rating curve (refer previous tailwater rating curve detail in Table 3), the tailwater will be higher than the top of the training walls when discharges exceed approximately 1,900 m^3/s (around the 1 in 100 AEP flood).

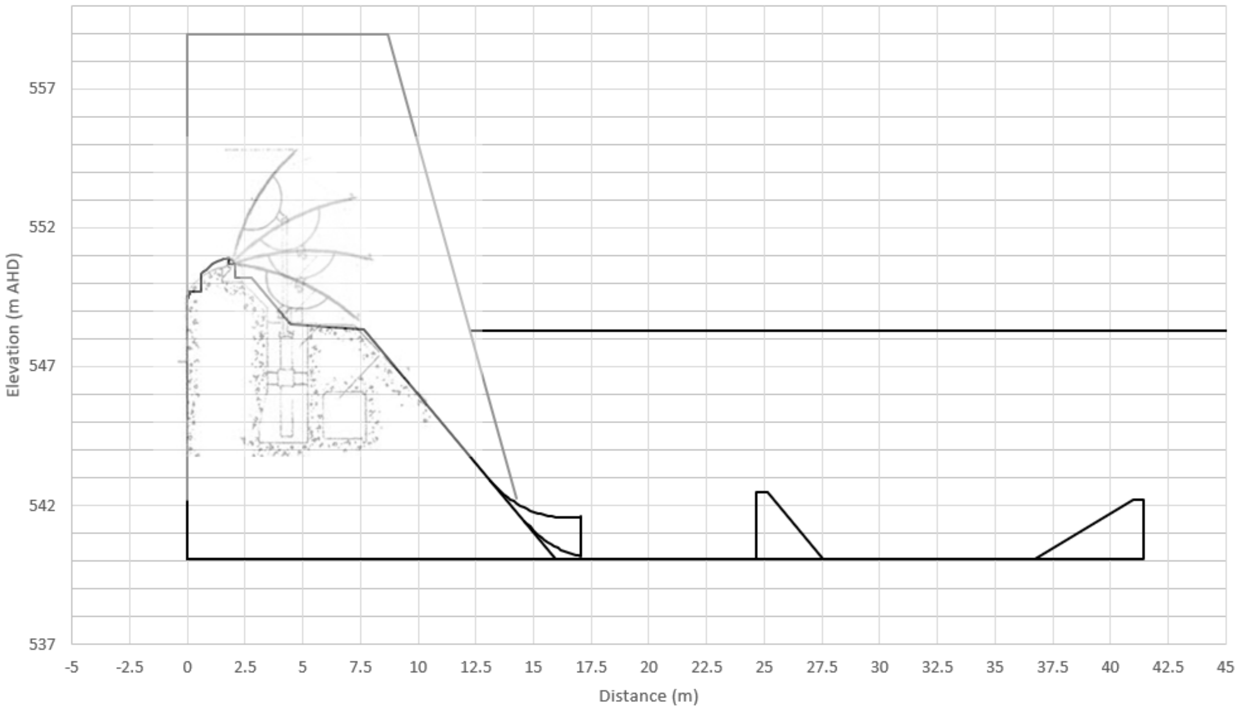

Figure 20 Cross-section of Stilling Basin showing height of training walls

In addition to the height of the training walls, the recent Physical Model study also highlighted the fact that under some frequent discharges (as the gates are opened), the training walls do not contain the spread of flow, as shown in Figure 21. Although the extent of impacting flow outside the training wall is relatively minor, it has the potential to occur at discharges more frequent than the 1 in 100 AEP flood. The area immediately behind the training walls appears to be protected on both sides of the stilling basin with a concrete slab (refer to Figure 22 and Figure 23), however, beyond this area it appears to be relatively erodible (soil) material.

Figure 4-1 Comparing flows in Bay 1 between prototype and model for 540 m^3/s

Figure 21 ***Discharge impact training wall during 540 m^3/s releases (extract from WRL, 2021)***

Figure 22 View of right training wall

Figure 23 View of left training wall

In terms of containment of flow, two potential improvements could be made to the training walls, namely:

- Extending the piers with a triangular profiled extension to better contain the water surface profile (and jet spread) up to the containment level of the rest of the stilling basin (refer to striped yellow triangle in Figure 24)
- Increase the height of the training walls to contain events greater than the 1 in 100 AEP flood (refer to solid yellow section in Figure 24), but not necessarily extended to contain the 1 in 100,000 AEP flood.

Figure 24 Potential Improvements to contain flow

4.5.4 Recirculation Flows

The potential for recirculation flows has been reviewed with specific focus on the two following aspects:

- Potential for recirculation flows to occur outside the training walls and erode material outside the stilling basin
- Potential for recirculation flows to scour rock and mobilise it back into the stilling basin in a recirculating pattern, leading to potential for ball-milling.

The 2021 Physical Model study investigated the potential for recirculation flows in detail, both in terms of flow patterns and the potential for ball-milling. Various plots of recirculation flow were produced, which also included estimates of area of accumulation of scoured material. An example of one of these plots is provided in Figure 25.

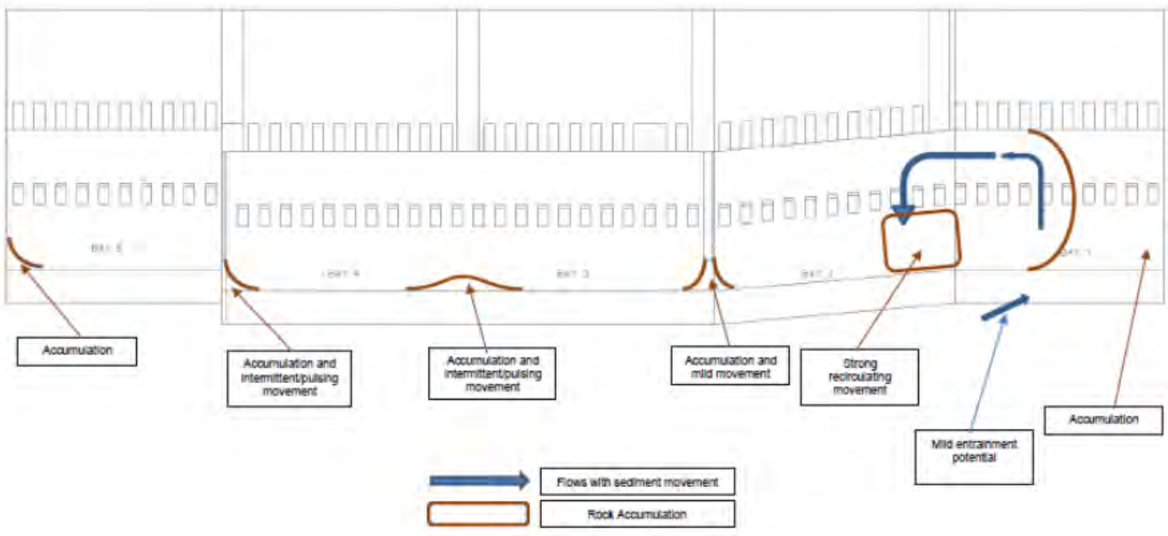

Figure 5-21 Test D04 ball milling assessment, Q = 1,040 m³/s

Figure 25 Example of test plots showing recirculation flows and potential for ball-milling (extract from WRL, 2021)

The key findings of these tests indicated:

- Significant recirculation flows occur within the stilling basin, particularly under the current gate operating rules
- Proposed changes to the gate operating rules (opening the outer gates earlier in the sequence) has the potential to significantly reduce the amount of recirculation (and thus reduce the amount of debris/particles being accumulated in the basin). In addition, the change in operating rules also appears to significantly reduce the peak exit velocities observed under frequent events.
- Recirculation flows outside the basin do not appear to be a significant concern.

A summary of the existing gate operating rules is provided in Table 7 and a summary of the proposed gate operating rules (SMEC 2021) is provided in Table 8.

Table 7 Current Gate Operating Rules (5 stages of operation)

Stage	Total Discharge	Gate Discharge for each Bay				
		1	2	3	4	5
Stage 1	80 - 140			80 - 140		
Stage 2	160-320		80 - 160	80 - 160		
Stage 3	360-900		120-300	120-300	120-300	
Stage 4	900-1600	240-400	240-400	240-400	240-400	
Stage 5	2000-3250	400-650	400-650	400-650	400-650	400-650

Table 8 Proposed Gate operating rules

Stage	Total Discharge	Gate Discharge for each Bay				
		1	2	3	4	5
Stage 1	80 - 140			80 - 130		
Stage 2	160-280		80 - 140	80 - 140		
Stage 3	360-460		120-130	120-130	120-130	
Stage 4	460-3500	80-700	100-700	100-700	100-700	80-700

4.6 Hydraulic Loadings on Slab

4.6.1 General

In addition to the hydraulic performance of the stilling basin, its structural performance also requires consideration for the range of discharges up to the design event. The hydraulic loadings which are typically used as the basis for structural design include:

– Uplift beneath the stilling basin – Seepage beneath the dam will result in an increase in groundwater pressure beneath the dam and stilling basin. The resulting uplift pressure beneath the stilling basin will need to be included in the structural design loads.

– Pressure transients above the concrete floor – Turbulence in the stilling basin will create transient pressures on the floor of the stilling basin slab. In some cases, these pressures will cause a downward pressure greater than the opposing uplift pressure beneath the slab (net downward force). At other points in time, pressure transients may result in downward pressures less than the opposing uplift (net uplift force). Factors which are important to consider when assessing these pressure transients include:
  - Areal effects of the pressure transients – Peak pressures may occur in a very localized portion of the slab, or a widespread area. Understanding the spatial extent of pressure peaks is important.
  - Magnitude of pressure 'spikes' from the average pressures – Pressure spikes may occur for a very short time, and the concrete slab may not have time to respond to these spikes.
  - Location of peak pressures – Peak pressures upstream and downstream of the baffle blocks may be significantly different, therefore requiring different structural solutions. In the case of Scrivener Dam where the stilling basin has a sloping invert, pressures will also differ between invert levels.
  - Discharge (and associated tailwater) conditions which result in the greatest pressures – The relationship between discharge and tailwater is important. Some stilling basins will experience their most severe hydraulic loading at discharges less than the design flood. It is important to understand which discharges (and corresponding tailwater conditions) produce the most severe loading.

– Potential for pressures to penetrate through joints in the slab – In cases where the stilling basin slabs do not have waterstops (as is currently the case at Scrivener Dam), there is potential for pressure transients, caused by the high velocity flows, to penetrate through the joints and into the drains, and pressurise the underside of the slabs and underlying jointed rock mass.

These aspects are discussed in the following sub-sections.

In addition to these factors, in future stages of the project it will be necessary to review the additional hydraulic loading and durability requirements:

– Pressure transients/impact loading on baffles and end sill – Pressure conditions upstream and downstream of the baffles will differ, and the resulting hydraulic loads on the baffles need to be considered.

– Erosion resistance of the concrete – Depending on the debris/sediment load circulating or passing the stilling basin, there is potential for the surface of the concrete to be eroded and perhaps cause damage by cavitation.

These two dot points have not been explored in detail in this section, but will need to be considered in the detailed design.

4.6.2 Steady-state Uplift beneath Stilling Basin

In the SMEC (2016) Design Review, an assessment was undertaken of the likely uplift pressures beneath the dam and stilling basin slab. Although the dam and stilling basin has a series of foundation drains, there is no means of maintaining the drains, and therefore it has been assumed that the drains are largely ineffective. On this basis, a typical triangular pressure variation has been adopted beneath the dam and stilling basin, transitioning from full reservoir head at the upstream side of the dam, to tailwater level at the downstream side of the stilling basin, as shown in Figure 26. The resulting uplift pressures, shown as a function of head (m) at the toe of the dam (refer location ‘A’ in Figure 26) and the downstream end of the stilling basin (refer location ‘B’ in Figure 26), are summarised in Table 9.

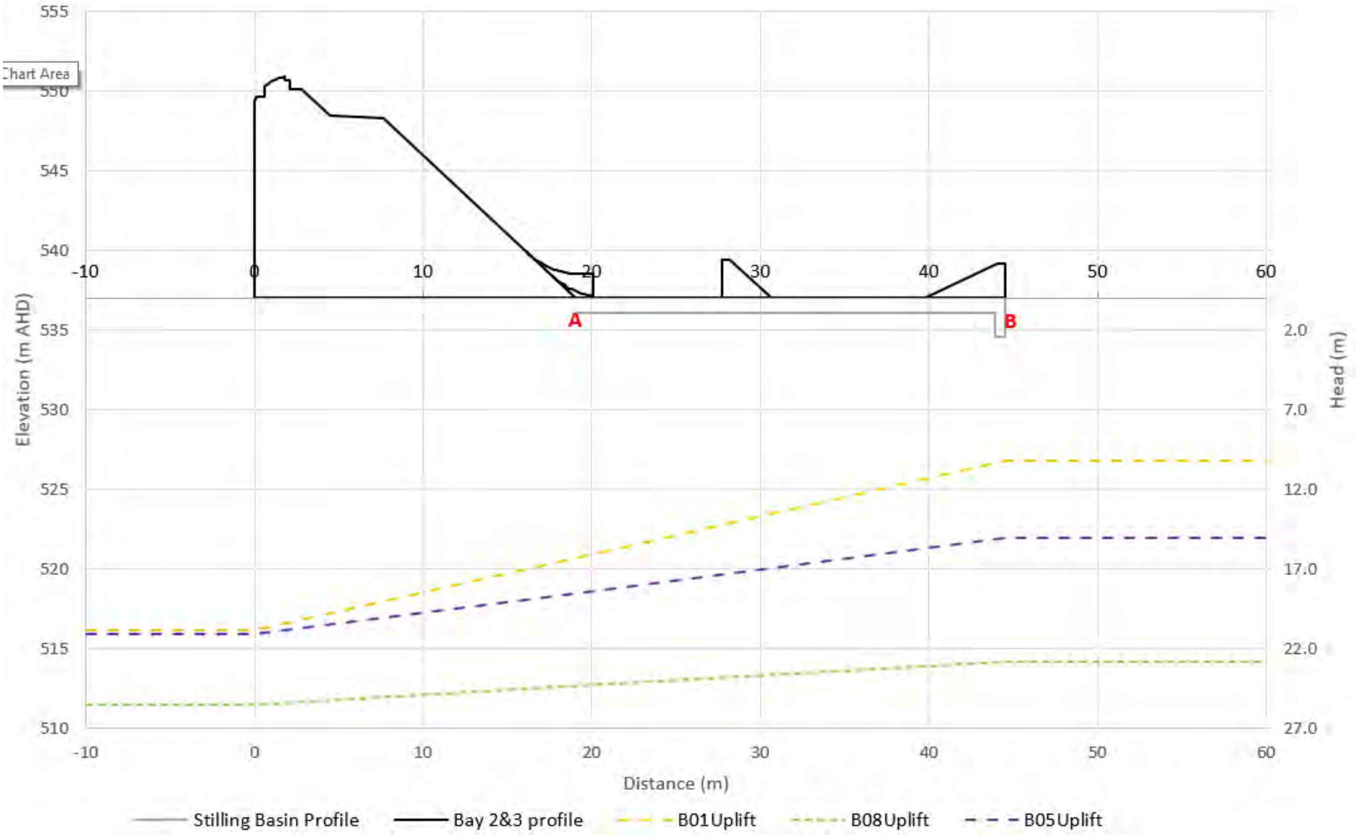

Figure 26 *Example of triangular uplift pressure for various combinations of headwater and tailwater levels*

Table 9 Proposed Steady-state Uplift Conditions on underside of stilling basin slab

Test conditions						Uplift Pressure Head at Underside of Slab (m)			
						Bay 2 and 3		Bay 1 and 5	
Test	Gate Sequence	Q*	HW	TW	TW	A	B	A	B
D07	Proposed	750	556.11	RC	544.78	15.4	9.3	18.5	12.3
B01	Existing	900	556.08	RC	545.39	15.4	9.2	18.5	12.3
D05	Proposed	900	556.11	RC	545.35	15.6	9.7	18.6	12.7
D06	Proposed	1050	556.12	RC	545.76	15.7	10.0	18.8	13.1
D12	Proposed	1150	556.07	RC	546.11	16.3	11.3	19.3	14.3
B02	Existing	1600	556.12	RC	547.37	16.1	10.8	19.1	13.9
D03	Proposed	1450	556.12	RC	546.95	16.4	11.5	19.4	14.5
D11	Proposed	1700	556.09	RC	547.60	16.8	12.0	19.9	15.0
B03	Existing	1900	556.56	RC	548.07	16.5	12.0	19.6	15.0
D08	Proposed	1900	556.05	RC	548.10	16.7	12.3	19.7	15.3
B04	Existing	2000	556.08	RC	548.36	16.7	12.5	19.8	15.6
D09	Proposed	2150	555.98	RC	548.63	17.6	14.2	20.7	17.2
B05	Existing	3000	556.32	RC	550.26	17.4	14.0	20.5	17.0
D10	Proposed	2900	556.11	RC	550.07	19.3	17.1	22.3	20.2
B06	Existing	5000	557	RC	553.23	21.4	20.0	24.5	23.1
B07	Existing	7500	558.6	RC	556.13	23.5	21.9	26.5	25.0
B08	Existing	9500	560.72	RC	558.02	16.4	11.0	19.4	14.0
B09	Existing	1900	556.48	RC-1	547.1	16.4	11.0	19.4	14.1
B10	Existing	1900	556.52	RC-1	547.12	16.0	10.0	19.0	13.0
B13	Existing	1900	556.6	RC-2	546.06	15.9	9.9	19.0	13.0
B14	Existing	1900	556.48	RC-2	546.03	18.4	15.1	21.4	18.1
B17	Existing	5000	556.92	RC-2	551.16	20.5	18.1	23.6	21.1
B11	Existing	7500	558.48	RC-2	554.21	22.3	20.1	25.3	23.1
B12	Existing	9500	560.04	RC-2	556.16	17.5	13.0	20.5	16.1
B18	Existing	5000	556.92	RC-4	549.14	19.6	16.1	22.7	19.1
B15	Existing	7500	558.44	RC-4	552.16	21.2	18.1	24.2	21.1
B16	Existing	9500	559.56	RC-4	554.2	15.4	9.3	18.5	12.3

4.6.3 Pressure Transients on concrete floor

Pressure transients were measured at 30 different locations in the model study, in Bay 3, 4 and 5. Pressure transducers were placed in the model upstream and downstream of the baffles. Transient pressures were measured for a range of flows up to 9,500 m^3/s with the corresponding 'best estimate' of tailwater level. Further sensitivity analysis was also undertaken for the same discharges with a lower tailwater level (namely, a reduction in tailwater level by 1, 2 and 4 m below the estimated level).

A typical example of a pressure transient record over time for the 9,500 m^3/s event (at location B3-12) is provided in Figure 27. The downstream tailwater level is shown in 'yellow', the average pressures is shown in a solid red line, and the standard deviation and 2x standard deviations are shown in dashed and dotted red lines respectively. The plot shows the following:

– the average pressure is less than the tailwater level
– Although the 2x standard deviation captures the vast majority of pressures, there are numerous pressure spikes that exceed this upper 2x standard deviation (mean+2σ) by up to 2m pressure, and by around 0.5 m below the lower 2x standard deviation (mean-2σ).

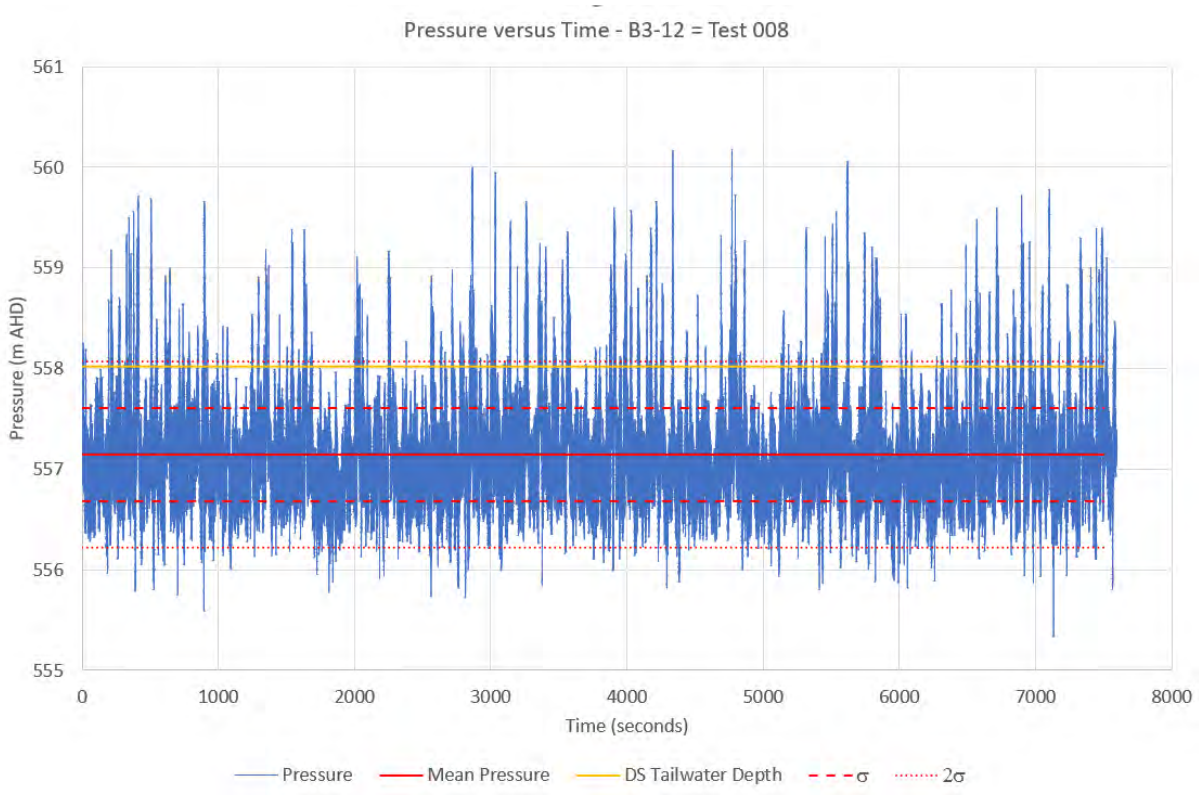

Figure 27 *Example of Pressure Transients versus time – B3-12 for 9,500 cumecs (Test 008)*

In order to accurately assess the performance of the stilling basin under the fluctuating loads, it may be necessary to develop a fluid-structural model and apply the pressure time history as an input load. This process will be time-consuming, particularly if all pressure time histories need to be assessed.

For the design of Scrivener Dam stilling basin remedial works, it has been decided to adopt a downward pressure equal to the average pressure transient minus two standard deviations (i.e. refer to -2σ in previous graph). Although this pressure does not capture the absolute minimum spikes in the data, it is postulated that the slab has insufficient time to respond to the infrequent spikes, and therefore the approach to adopting a design load of the mean-2σ, provides an appropriate design load to be adopted for the option assessment.

4.6.4 Pressure penetrating through joints

The ability for water to pressurise the underside of the slabs through pressurisation of contraction joints will largely depend on the arrangement of the contraction joints and sub-surface drainage system. For most of the upgrade options, it is proposed that double waterstops are adopted in all joints, and therefore the need to apply additional loads to the underside of the slab to account for this scenario is considered unnecessary.

For options which do not have double waterstops, or where it will be challenging to retrofit waterstops, it is proposed that the total uplift applied beneath the slab be equal to the reservoir head, which is a worst case scenario, and hence conservative (safe) assumption.

4.7 Conclusions

In summary, the following is proposed for the approach to hydraulic performance and loading for the design of the upgrade works:

- The proposed design flood is the 1 in 100,000 AEP event, which is appropriate for a High C consequence category dam. This event equates approximately to a discharge of 8,200 m^3/s. Beyond this event, it is understood that the stability of the dam becomes marginal. Furthermore, larger floods may result in orifice flow due to the presence of the spillway bridge.
- The tailwater levels adopted in the analysis have been based on the SKM (2010) analysis. The sensitivity analysis undertaken on tailwater levels and applied to the physical hydraulic model study runs will be taken into account when developing loads for the stilling basin.
- The high-level review of the hydraulic performance of the dissipator suggested that the existing stilling basin is providing a relatively effective dissipation of energy, and does not require modification in the upgrade works. However, previous analysis indicates that there would be an improvement in flow conditions with changes to the gate operating rules (namely opening the outer gates earlier in the opening sequence).
- A review of the proposed hydraulic loads to be adopted in the stilling basin upgrade has been undertaken. It is proposed that uplift beneath the slab be taken as a linear regression from headwater to tailwater level. It is proposed that pressures above the slab be taken as the mean less two times the standard deviation of the pressure transients measured in the physical model study.

  In cases where there are no waterstops in the contraction joints, increased uplift pressures will be applied beneath the slab, equal to full reservoir head.

It is noted that future stages of the design may require more complex modelling of the stilling basin, including additional physical hydraulic modelling (if the geometry substantially changes), and/or fluid-structural finite element modelling to provide a better basis for the design loads. However, it is considered that the loads previously discussed will provide an appropriate basis for use in the development of options.

5. Options Identification and Initial Option Screening Workshop

5.1 Options Identification and Evaluation

At commencement of the project, an internal workshop was held amongst senior team members to discuss the key perceived deficiencies of the stilling basin, and identify a broad range of potential upgrade works. During this workshop, nine different options were identified for upgrading the Scrivener spillway stilling basin.

This section provides a summary of the nine potential options, including a high-level sketch of the proposed arrangement.

5.1.1 Option 1 - Do Nothing

The first option considered was a 'do nothing' option. This option would only be considered if the net (uplift) pressure differential applied to the stilling basin slab is small enough that it can be confirmed that the existing slab and anchor arrangement can adequately withstand the design loads. Processing of the hydraulic design loads had not been completed at commencement of the 'option identification' phase, however based on earlier work completed as part of the Design Review (SMEC, 2016), it appeared unlikely that a 'do nothing' option would be acceptable.

Option 1 evaluation is presented in Table 10, which highlights that this option is unlikely to meet the requirements of the performance criteria.

Table 10 Option 1 Evaluation

Option 1 – Do Nothing	
Description of Option	No upgrade works are required
Ability to Meet Design Criteria	No
Pros	– Quick to implement – No capital cost
Cons	– Will not meet the requirements of the performance criteria – Potential failure of the stilling basin slab under flood loadings – Potential for ongoing maintenance/repair cost
Additional Considerations required to develop option	Are the fluctuation dynamic pressures adequately high to warrant an upgrade

5.1.2 Option 2 - Overlay Slab with No Additional Anchors

The second option identified comprised the installation of a concrete overlay slab on the existing slab. No new anchors were proposed as part of the new slab. This Option requires construction of a new thick reinforced concrete slab on the top of the existing slab as shown in Figure 28. The overlay slab would be designed to withstand the peak net pressure differential uplift forces without the need for passive anchors. Option 2 evaluation is presented in Table 11, which shows that this option can meet the requirements of the performance criteria.

Table 11 Option 2 Evaluation

Option 2 – Overlay slab with no additional anchors	
Description of Option	Construct a new thick reinforced concrete slab on the top of the original slab
Ability to Meet Design Criteria	Yes
Pros	– Easy to construct – Contraction joints can be adequately detailed • Waterstops (centre bulb and rear guard) will be added to the overlay slab • Dowels will be installed at the contraction joints of the overlay slab • Small chamfers away from flow to limit transmission of pressures – High strength and durable concrete will be used for the overlay slab – Will be adequately reinforced with two layers of the reinforcement at the top and bottom – Any defects on the original slab (e.g., crack, surface erosions) will be covered/protected by the overlay slab
Cons	– The overlay slab thickness should accommodate appropriate cover (top and bottom) and two mats of reinforcement – likely to be minimum 400 mm-500 mm thick – Surface preparation on the top of the existing slab – The overlay slab relies on its weight to overcome the uplift deficiencies – may not engage mass of existing slab and the existing anchors – Will impact the effectiveness of the baffle blocks and have potential to change the hydraulics of the spillway – Will create a cold joint between the overlay and original concrete slabs
Additional Considerations required to develop option	– Hydraulic modelling of the spillway – Calculation of fluctuation dynamic pressures infiltrating under the slab and the resulting uplift pressures

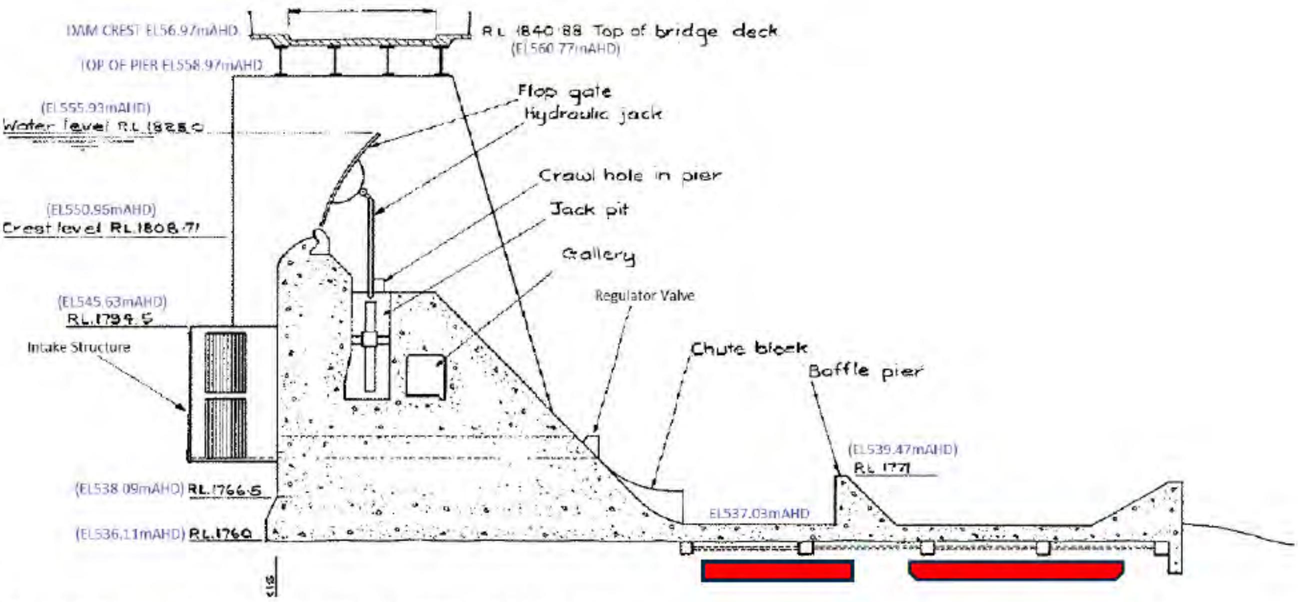

Figure 28 Overlay Slab with No Additional Anchors

5.1.3 Option 3a – Minimum Overlay Slab with Anchors

Option 3 comprises the installation of an anchored overlay slab. A number of subsets of this option were identified. Option 3a comprises designing the minimum thickness overlay slab possible, with a great reliance on anchors to resist the uplift loads as shown in Figure 29. The thickness of the new slab will be minimised to meet the structural requirements, and the number of anchors will be determined as required so that the new slab and additional anchors are able to take the peak net uplift pressure differentials.

Option 3a evaluation is presented in Table 12, which shows that this option can meet the requirements of the performance criteria.

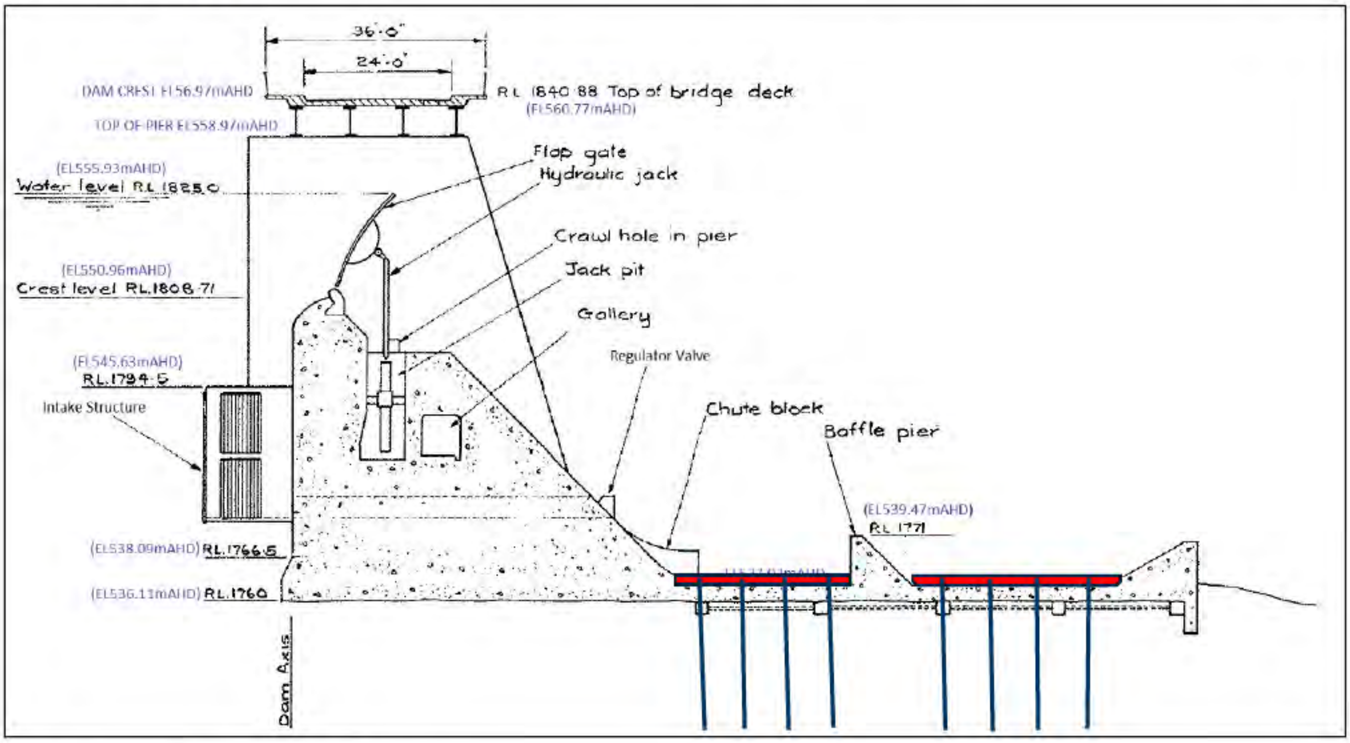

Figure 29 Overlay Slab with Anchors

Table 12 Option 3a Evaluation

Option 3a – Overlay Slab with Anchors (min slab thickness, rely on anchors)	
Description of Option	Construct a new thin reinforced concrete slab on the top of original slab with anchor bars
Ability to Meet Design Criteria	Yes
Pros	– The minimum slab thickness (likely more than 400 mm) – The contraction joints will be adequately detailed • Waterstop (centre bulb and rear guard) will be added for the top slab • Dowels will be installed at the contraction joints between the slab panels • Small chamfers away from flow to limit transmission of pressures – The top slab relies mainly on the anchors to resist uplift pressures – High strength and durable concrete will be used for the top slab – Will be adequately reinforced with two layers of the reinforcement at the top and bottom – The impact on the effectiveness of the baffle blocks will be reduced – The anchor head will be embedded in the top slab – Potential gap under the original slab can be filled through the new bore holes – Double corrosion protection anchors (expensive-Cons)

Option 3a – Overlay Slab with Anchors (min slab thickness, rely on anchors)	
Cons	– Big anchors depending on the dynamic uplift pressures – Development of the tensile capacity of the anchor within the overlay slab will need to be considered with respect to punching shear – Core drilling through the existing slab – The top of the original slab requires surface preparation – The construction period will be longer than option 2. – Will create a cold joint between the overlay and existing slabs
Additional Considerations required to develop option	– Might need hydraulic modelling of the spillway – Calculation of fluctuation dynamic pressures and the resulting uplift pressures

5.1.4 Option 3b – Thick Overlay Slab with Anchors

Option 3b is similar to Option 3a, however comprises a new thick slab on the top of the existing slab with a lesser number of anchors (as more reliance is placed on the weight of the slab to resist buoyancy) as shown in Figure 30. The thickness of the new slab will be maximised (within reason), and the number of anchors will be determined as required so that the new slab and additional anchors are able to take the net pressure differential.

Option 3b evaluation is presented in Table 13, which shows that this option can meet the requirements of the performance criteria.

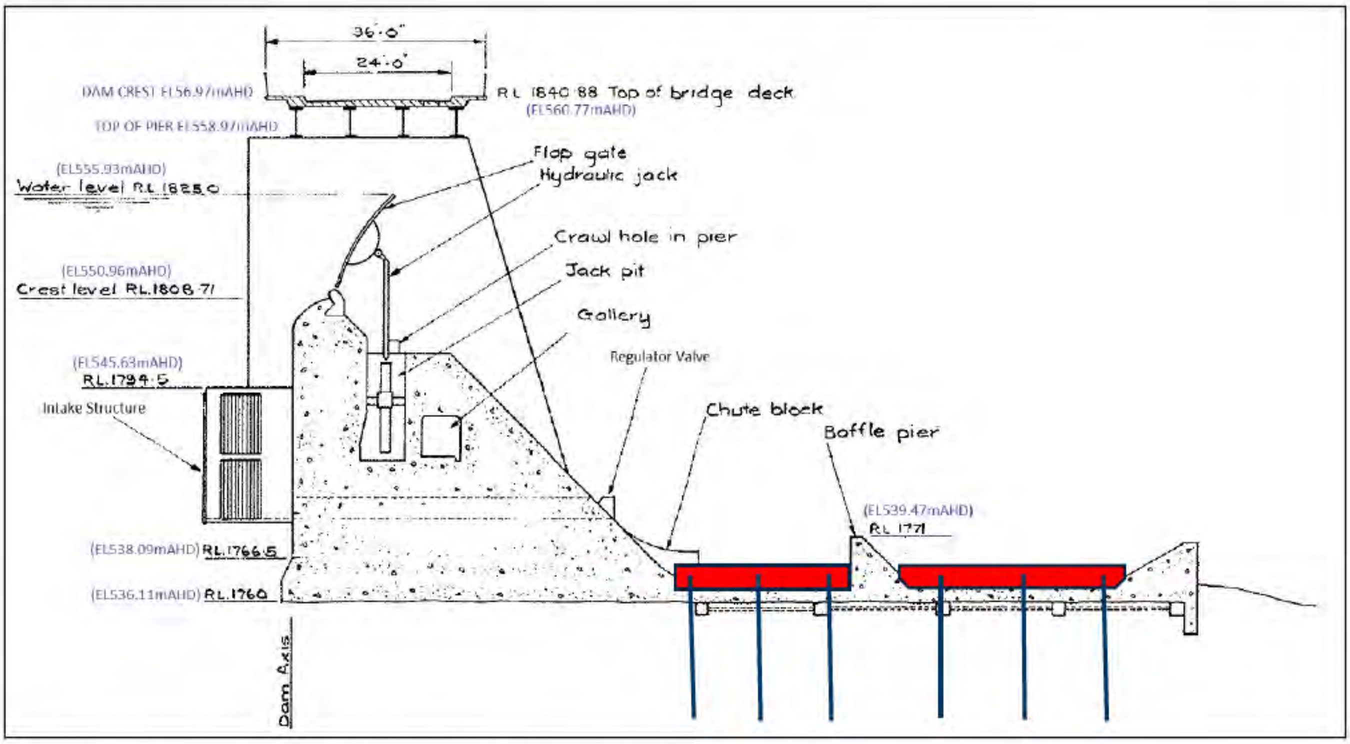

Figure 30 Overlay Slab with Anchors (Thick slab and less anchors)

Table 13 Option 3b Evaluation

Option 3b – Overlay Slab with Anchors (thick slab, less anchors)	
Description of Option	Construct a new thick reinforced concrete slab with less anchors
Ability to Meet Design Criteria	Yes
Pros	– Provide flexibility to optimise the construction cost – The contraction joints will be adequately detailed • Waterstops (centre bulb and rear guard) will be added to the top slab • Dowels will be installed at the contraction joints between the new slab panels – The top slab relies mainly on its weight to resist uplift pressures – High strength and durable concrete will be used for the overly slab – Will be adequately reinforced with two layers of reinforcement at the top and bottom – The anchor head will be embedded in the top slab – Less anchors – Double corrosion protection anchors (expensive-Cons) – Potential gaps under the original slab will be filled using the new anchor bore holes

Option 3b – Overlay Slab with Anchors (thick slab, less anchors)	
Cons	– The overlay slab thickness should accommodate appropriate cover (top and bottom) and two mats of reinforcement – likely to be minimum 400 mm thick – Core drilling is required through the original slab – The top of the original slab requires surface preparation – Will impact the effectiveness of the baffle blocks and have potential to change the hydraulics of the spillway
Additional Considerations required to develop option	– Hydraulic modelling of the spillway – Calculation of fluctuation dynamic pressures and the resulting uplift pressures

5.1.5 Option 3c – Overlay Slab with Anchors (Monolithic/structural connection to underlying slab)

Option 3c is also similar to Option 3a and 3b, but is designed with a monolithic tie-in detail between the existing and new slab. Specifically, this option requires construction of a new slab on the top of the existing slab with additional anchors and doweling of the top new slab to the existing slab as shown in Figure 31. The new slab will be doweled to the existing slab and the overall thickness will be minimised to meet the structural requirements. The number of the anchors will be determined as required so that the monolithic slab and additional anchors are able to withstand the peak uplift loads.

The main difference between Option 3c and Option 3a is that in Option 3c the new and existing slabs are monolithic in bending and shear. Option 3c evaluation is presented in Table 14, which shows that this option can meet the requirements of the performance criteria.

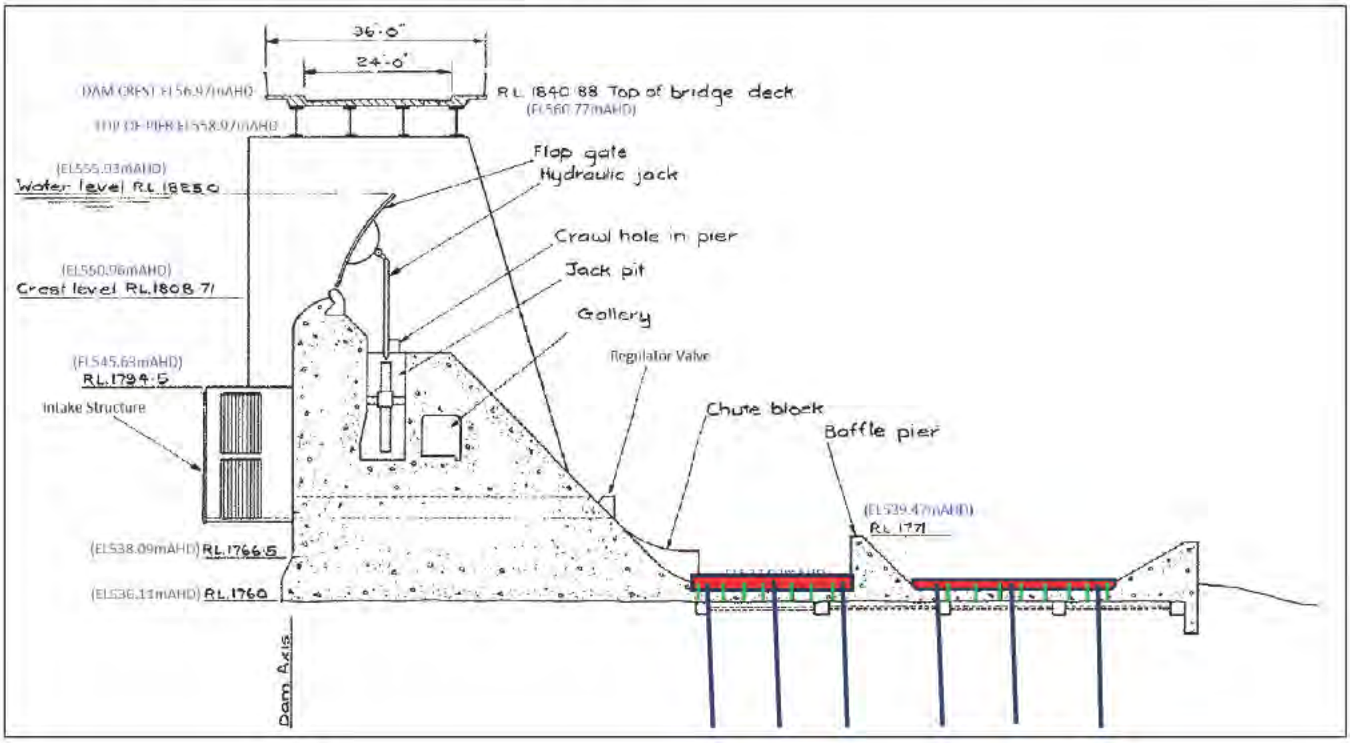

Figure 31 Overlay Slab with Anchors (monolithic/structural connection to underlying slabs)

Table 14 Option 3c Evaluation

Option 3c – Overlay Slab with Anchors (monolithic/structural connection to underlying slab)	
Description of Option	Construct a new reinforced concrete slab on the top of the original slab with anchor bars and dowel the top slab to the existing slab
Ability to Meet Design Criteria	Yes

Option 3c – Overlay Slab with Anchors (monolithic/structural connection to underlying slab)	
Pros	– Reduced overlay slab thickness compared with Option 3b. – The contraction joints will be adequately detailed – Waterstops (centre bulb and rear guard) will be added to the top slab – The new and original slabs create a monolithic structural slab to resist the uplift pressures – High strength and durable concrete will be used for the top slab – The new slab will be adequately reinforced with two layers of reinforcement at the top and bottom – The anchor head will be embedded in the top slab – Double corrosion protection anchors (expensive-Cons) – Potential gaps under the original slab will be filled using the new anchor bore holes – Reduced impact on hydraulics and effectiveness of baffle piers compared with Option 3b.
Cons	– Similar cons as the other Option 3s – No advantage over the other Option 3s as the dynamic uplift pressures infiltrates between the two slabs and will be resisted by the top slab – Additional drilling to install the dowels – Dowels will be exposed to corrosion and use of SS dowels will be required (expensive) – Construction period will be longer than the other Option 3s due to core drilling and installation of dowels
Additional Considerations required to develop option	Hydraulic modelling of the spillway Calculation of fluctuation dynamic pressures and the resulting uplift pressures

5.1.6 Option 4 – Retrofit Anchors (no new slab)

Option 4 requires retrofitting new anchors to the existing slab as shown in Figure 32. The number of the additional anchors will be determined as required so that the exiting slab and additional anchors jointly are able to take the peak design net uplift loads.

Option 4 evaluation is presented in Table 15, which shows that this option can meet the requirements of the performance criteria and most likely is the most cost-effective option.

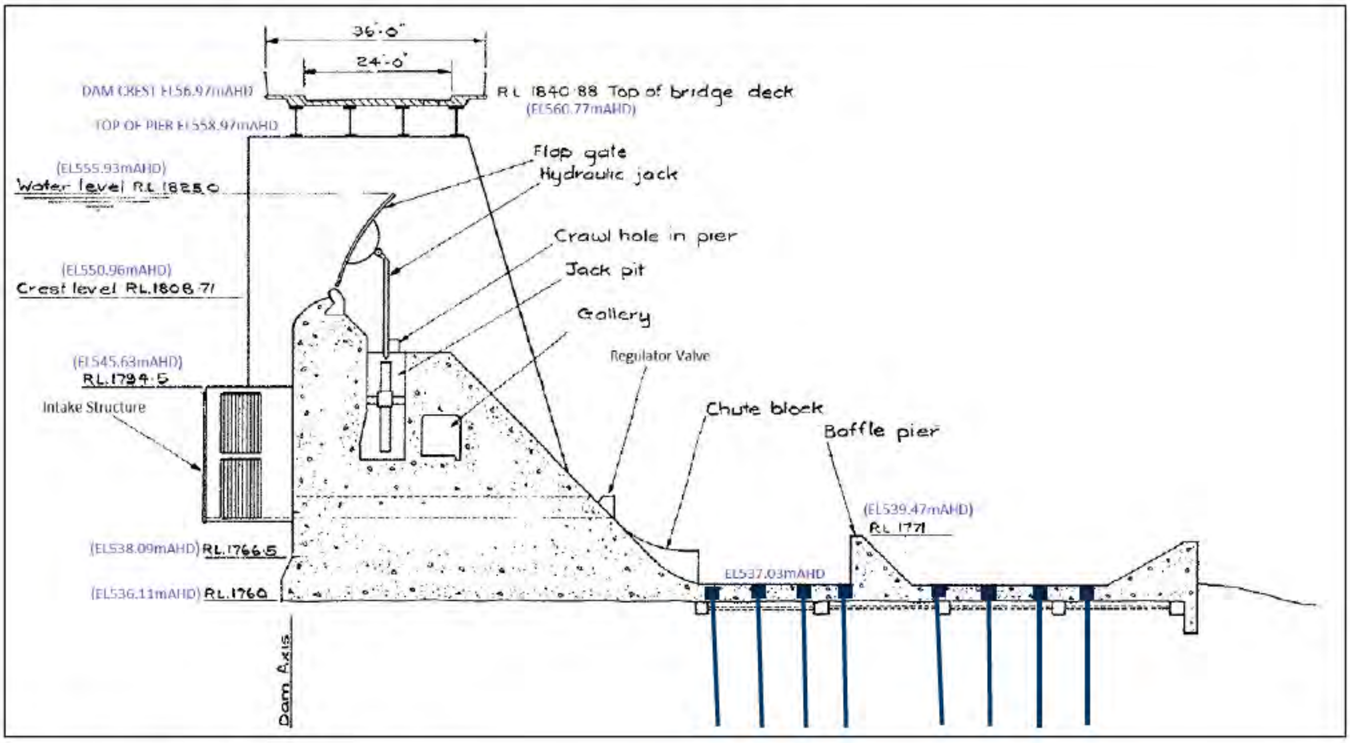

Figure 32 Retrofit anchors (no new slab)

Table 15 Option 4 Evaluation

Option 4 – Retrofit Anchors (no new slab)	
Description of Option	Install additional anchors
Ability to Meet Design Criteria	Yes
Pros	– New slab not required – Will not affect the baffle blocks and hydraulics of the spillway – Concreting will be minimum – Likely to be the most cost-effective option – Double corrosion protection anchors (expensive-Cons) – Potential gaps under the original slab will be filled using the new anchor bore holes
Cons	– Additional anchors should be big (or dense) to take all the fluctuation dynamic pressures infiltrating under the slab. – Waterstops will not be added to the existing slab joints – Core drilling of the original slab for embedment of the anchors head – Sealing of the joints for the anchor heads – Any defect (e.g., cracks, erosion and etc.) on the original slab will remain exposed (not covered by a new slab) – The original slab only contributes by its weight (not strength) to resist uplift pressures
Additional Considerations required to develop option	Calculation of fluctuation dynamic pressures and the resulting uplift pressures

5.1.7 Option 5 – Lengthening Stilling Basin

Option 5 requires lengthening the existing stilling basin by constructing a new reinforced concrete slab on the downstream of the exiting slab and removing the baffle blocks as shown in Figure 33. This option may need to be considered if fish passage through the spillway is a concern and for that reason it is proposed to remove the baffle blocks (noting that baffles are often the cause of high fish mortality rates for downstream passage).

Option 5 evaluation is presented in Table 16. At this stage, it is assumed that this option may improve some aspects of hydraulic performance (less baffles/ less turbulence/ less pressure differentials) but is considered unlikely to improve the conditions to a point where the stilling basin does not require stabilising. As such, it appears unlikely that this option would meet the performance requirement unless done in combination with another option (say Option 4).

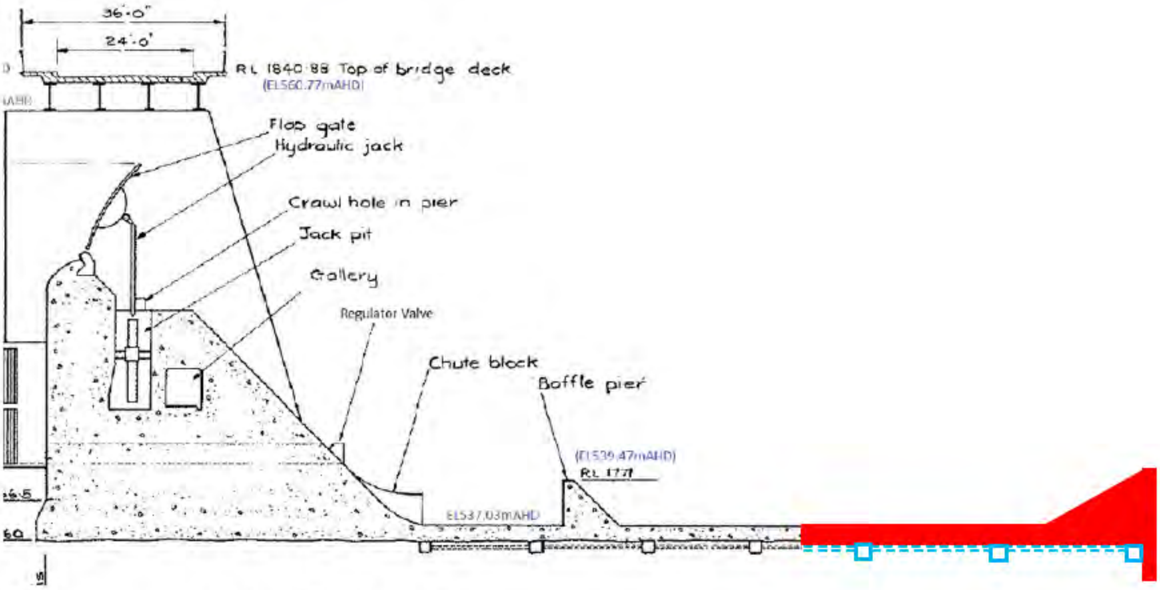

Figure 33 Lengthening Stilling Basin

Table 16 Option 5 Evaluation

Option 5 – Lengthen stilling basin	
Description of Option	Lengthen the existing stilling basin by construction of a new reinforced concrete slab on the downstream of the original slab
Ability to Meet Design Criteria	Potentially but unlikely– the existing slab still need stabilising
Pros	– May need to be considered if fish passage is a concern – Baffle blocks will be removed, potentially less turbulent flow – Waterstops will be added to the new slab panels preventing pressure transient penetrating through contraction joints

Option 5 – Lengthen stilling basin	
Cons	– Excavation of downstream for the extension of the slab – Demolition of the existing downstream apron and reconstruction of apron at the downstream end of the extended slab – No waterstops will be added to the original slab allowing pressure transient penetrating through the contraction joints – Foundation preparation and leveling concrete for the extended slab require significant work and cost – Drainage system should extend under the extended slab requiring significant work and cost – Unlikely to adequately mitigate issues of fluctuation dynamic pressures infiltrating under the existing slab – Construction cost and period will be significantly more than Options 1 to 4 – The static uplift pressures will be increased along the length of the extended slab
Additional Considerations required to develop option	– New hydraulic modelling – Calculation of dynamic fluctuation pressures and the resulting uplift pressures – Fish passage is an issue or not

5.1.8 Option 6 – Tailwater Control Weir on the Downstream of Stilling Basin

Option 6 requires constructing a tailwater control weir on the downstream of the existing stilling basin in an attempt to increase the available tailwater to improve energy dissipation characteristics, as shown in Figure 34.

Option 6 evaluation is presented in Table 16, which shows that it is unlikely to resolve the fluctuation dynamic pressure issues and is unable meet the requirements of the performance criteria.

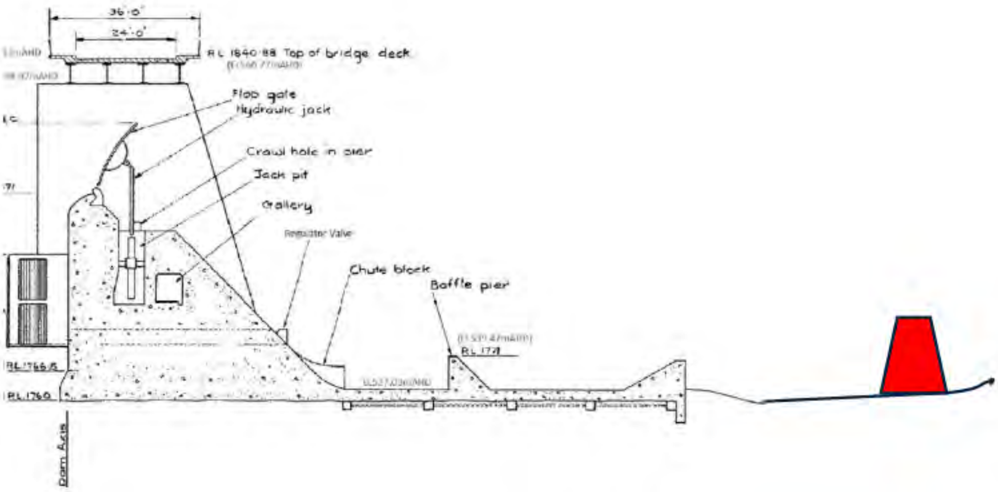

Figure 34 Tailwater Control Weir

Table 17 Option 6 Evaluation

Option 6 – Tailwater control weir to control hydraulic conditions in existing basin	
Description of Option	Construct a tailwater control weir on the downstream of the existing stilling basin to improve energy dissipation characteristics
Ability to Meet Design Criteria	Potentially but unlikely to resolve the fluctuation dynamic pressure issues
Pros	– More flexibility with operating the outer gates for small discharges (better energy dissipation for the outer gates under low flows) – May slightly reduce fluctuation pressures but unlikely to the extent required to achieve stability – Easy to construct
Cons	– Likely to cause issues with flooding the gallery adit – Unlikely to address the fluctuation dynamic pressure issues – Create higher uplift pressures under the dam due to the increased tailwater level
Additional Considerations required to develop option	New hydraulic modelling of spillway Calculation of fluctuation dynamic pressures and the resulting uplift pressures

5.1.9 Option 7– Change Gate Operating Rules (no physical changes)

Option 7 requires changing the gate operational rules to improve the hydraulic performance of the stilling basin. Option 7 evaluation is presented in Table 18, which shows that this option is unlikely to meet the requirement of the performance criteria unless it is progressed in combination with another option.

Table 18 Option 7 Evaluation

Option 7 – Change Gate Operating Rules (no physical changes)	
Description of Option	Change the gates operation rules to improve the hydraulic performance
Ability to Meet Design Criteria	No
Pros	– No physical change to the stilling basin – No construction cost – No further discussions as will not meet the performance criteria
Cons	– Will not mitigate the issue of dynamic uplift pressures
Additional Considerations required to develop option	– No further consideration is required

5.1.10 Option 8 – Deepen Stilling Basins in Bays 1, 4 and 5

Option 8 requires demolition of the existing slab in bays 1, 4 and 5 and excavation of their foundation to the same level of bays 2 and 3 and construction of a new slab as shown in Figure 35 for Bay 1, 4 and 5.

Option 8 evaluation is presented in Table 19, which shows that this option may slightly reduce pressures transient but unlikely to the extent required to achieve stability and meet the requirements of the performance criteria. As such, this option would most likely need to be done in combination with other options to meet the design criteria.

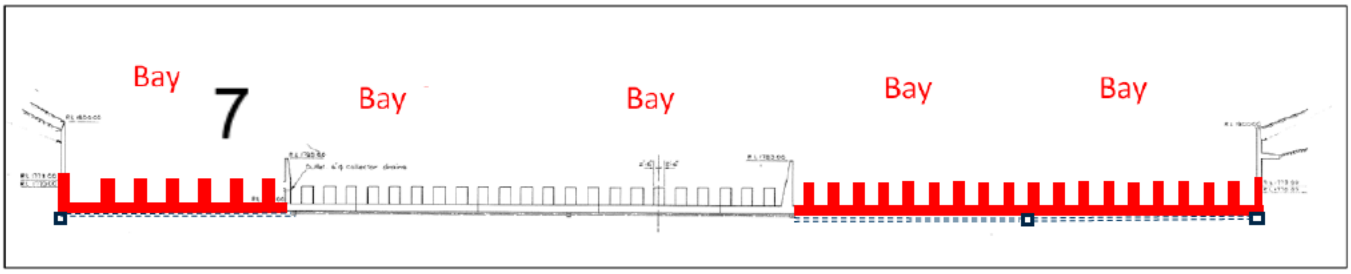

Figure 35 Deepen Stilling Basin in Bays 1, 4 and 5

Table 19 Option 8 Evaluation

Option 8 – Deepen Stilling Basins in Bays 1, 4 and 5	
Description of Option	Demolition and excavation of the original slabs in bays 1, 4 and 5 to build new slabs at the same level as of bays 2 and 3.
Ability to Meet Design Criteria	Yes, but unlikely to the extent required to achieve stability and meet the requirements of the performance criteria.
Pros	– More flexibility with operating the outer gates for small discharges (better energy dissipation for the outer gates under low flows) – May slightly reduce pressures transient but unlikely to the extent required to achieve stability – Sufficient anchors will be installed for the lowered slabs to resist the fluctuation dynamic pressures – Appropriate joint detailing and waterstops will be considered for the lowered slabs – The drainage system will be designed to reduce efficiently the uplift pressures in bays 1, 4 and 5
Cons	– Demolition of the existing slabs and drainage system in bays 1, 4 and 5 – Excavation of foundation to the same level as of bays 2 and 3 – Reconstruction of new drainage system under the new slabs in bays 1, 4 and 5 – Reconstruction of new slabs in bays 1, 4 and 5 – Remedial works are still required for bays 2 and 3 to fully mitigate the dynamic uplift pressures – Construction cost and period will be more than Options 1 to 4 – The divider walls should be extended vertically to the lower levels
Additional Considerations required to develop option	– New hydraulic modelling of spillway – Changing the gates operational rules – Calculation of fluctuation dynamic pressures and the resulting uplift pressures

5.1.11 Option 9 – Partial Demolition of Existing Slab, Adding New Anchors and Reconstruct Slab

Option 9 requires partial demolition of the top of the existing slab as required for the embedment of the new anchor heads and reconstruction of the slab to the original thickness as shown in Figure 36.

Option 9 evaluation is presented in Table 20, which shows that this option can meet the requirements of the performance criteria.

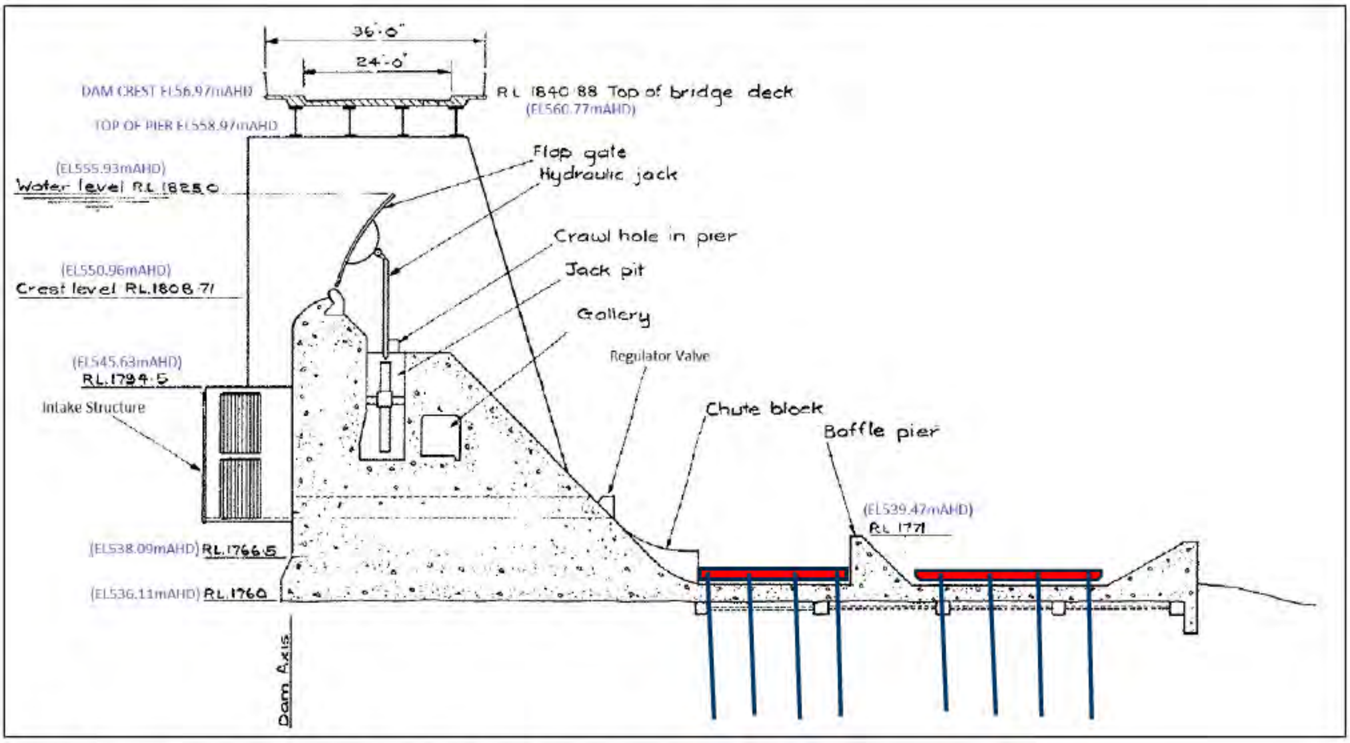

Figure 36 Partial Demolition of Existing Slab, Adding New Anchors and Reconstruction of Slab

Table 20 Option 9 Evaluation

Option 9 – Partial Demolition of original slab, new anchors, reconstruct slab	
Description of Option	Partial demolition of the top of the existing slab as required for the embedment of the new anchor heads and reconstruction of the slab to the original thickness.
Ability to Meet Design Criteria	Yes
Pros	– The contraction joints will be adequately detailed • Waterstops (centre bulb) will be added to reconstructed part of the slab • Dowels will be installed at the contraction joints between the slab panels where there are no shear keys – Anchors will be installed as required to resist dynamic uplift pressures – High strength and durable concrete will be used for the slab reconstruction – Will be reinforced appropriately with one layer of reinforcement at the top – No impact on the baffle blocks and hydraulics of the spillway – The anchor head will be embedded and protected in the reconstructed slab – Double corrosion protection anchors (expensive-Cons) – Potential gap under the original slab can be filled through the new anchor bore holes – Minor cracks and scouring on the existing slab will be fixed
Cons	– Partial demolition of the top part of the reinforced concrete slab will be difficult

Option 9 – Partial Demolition of original slab, new anchors, reconstruct slab	
	– Depending on the demolition method cracks can develop in the original unreinforced concrete slab – Will require big anchors depending on the dynamic uplift pressures – Will create a cold joint between the reconstructed and original concrete slab. – Core drilling through the original slab for the new anchors – The slab will have no bottom layer of reinforcement and still can only be relied upon its weight.
Additional Considerations required to develop option	Calculation of fluctuation dynamic pressures and the resulting uplift pressures

5.2 Option Screening

An initial option workshop with representatives from NCA, independent peer reviewer, GHD and SMEC took place on 27 August 2021 to discuss and shortlist the preferred options. The comprehensive minute of the workshop is included in Appendix D.

The workshop discussions and options evaluation resulted in shortlisting the original nine options to the three preferred options of 3, 4 and 9. The criteria for shortlisting were based on the technical advantages and meeting the requirements of the performance criteria. A summary of the outcomes are provided in Table 21.

Table 21 Initial Option Screening

Option	Description	Comments
1	Do Nothing	Eliminated – unlikely to meet performance requirements
2	Overlay slab with no anchors	Not preferred – slab likely to be very thick and may adversely impact the hydraulic performance of the basin.
3a	Anchored overlay slab (minimum slab thickness)	Preferred – Option offers a potentially robust solution, and meets design criteria. Moving forward, instead of considering three subset options, develop option to select an appropriate balance of slab thickness, anchors and connection between old and new slab.
3b	Anchored overlay slab (maximum slab thickness)	
3c	Anchored overlay slab (monolithic tie between existing and new slab)	
4	Retrofit Anchors into existing slab	Preferred – May not necessarily provide the robustness of other options, but likely to be a cheap solution so take forward to further development to challenge other options.
5	Lengthen stilling basin	Eliminated – unlikely to meet performance requirements unless done in combination with another option.
6	Tailwater control weir	Eliminated – as per Option 5
7	Change gate operating rules	Eliminated – as per Option 5
8	Deepening side bays of stilling basin	Eliminated – as per Option 5
9	Partial demolition of slab and reconstruct with anchored slab to original geometry	Preferred – Likely to meet performance requirements and provides a solution which maintains the current geometry.

The shortlisted options have been further developed so that they can be priced and compared for constructability, as discussed in the following sections.

6. Option Development and Selection

6.1 General

This section provides details of the development of the three preferred options, namely Options 3, 4 and 9. Specifically, this section includes details on:

– the proposed hydraulic loadings used in the development of the options,
– a description of each option including a summary of the key attributes, the proposed scope of works, and the perceived pros and cons of each option.
– the preliminary cost estimates prepared for each option; and
– the multi-criteria assessment of the options

Additional information to support this section is provided in the Appendices.

6.2 Design Approach specific to options

6.2.1 Hydraulic Loadings

Based on the review of hydraulic performance as described in Section 4, a summary of the required hydraulic assessments and loads was developed, specific to the three preferred options. This summary is presented in Table 22.

Table 22 Proposed Hydraulic Design Approach for Options

<table>
<tr><th>Hydraulic Performance</th><th>Option 3 – Overlay slab with anchors and waterstops</th><th>Option 4 – Retrofit anchors and waterstops</th><th>Option 9 – Partial demolition of slab and reconstruct to current dimensions</th></tr>
<tr><td colspan="4">Hydraulic Performance</td></tr>
<tr><td>Energy Dissipation</td><td>– Review performance against USBR Type III Basin (empirical)</td><td colspan="2">– No analysis – considered to be acceptable based on performance of current basin</td></tr>
<tr><td>Containment of Flow</td><td>– Invert of basin will be higher, therefore containment worse than existing. Need to raise training walls</td><td colspan="2">– Geometry of basin the same, therefore not worsening containment, but consider minor raising of walls.</td></tr>
<tr><td>Recirculation Flow</td><td>– Higher invert likely to exacerbate recirculation. Consider modifying gate operating rules.</td><td colspan="2">– Consider changing gate operating rules as proposed by SMEC (2021)</td></tr>
<tr><td colspan="4">Hydraulic Loading</td></tr>
<tr><td>Uplift Assumptions</td><td>– Linear distribution from reservoir head to tailwater level</td><td>– Refer to pressurisation of joints</td><td>– As per Option 3</td></tr>
<tr><td>Pressure on Top of Slab</td><td colspan="3">– Mean measured pressure less 2 x standard deviation for worst case</td></tr>
<tr><td>Pressure penetrating through slab joints</td><td>– None – assume that new waterstops are 100% effective in avoiding pressure transmitted through slab joints</td><td>– Waterstops unlikely to be 100% effective. Allow for pressure transmitted through joints equal to reservoir head.</td><td>– As per Option 3</td></tr>
</table>

Hydraulic Performance	Option 3 – Overlay slab with anchors and waterstops	Option 4 – Retrofit anchors and waterstops	Option 9 – Partial demolition of slab and reconstruct to current dimensions
Other			
Erosion of concrete	– Concrete strength, concrete cover and reinforcement design to consider potential for ball-milling/ scour	– Cracked and spalled concrete areas to be repaired	– As per Option 3
Other	– No negative impacts on concrete dam stability but consider improving drainage	– As per Option 3	– As per Option 3

Based on the proposed loading criteria to be adopted for each option (as described in the previous table), the results from the hydraulic model study were processed, along with assessments of uplift beneath the slab, to assess the critical peak design loads to be adopted for each option. These loads are detailed in Table 23. It is noted that although Bay 4 has a sloping invert, for simplicity the proposed loads for the Bay have been taken as those representative of the deepest location (i.e. the critical end of the slab). It may be beneficial in future stages of the project to adopt different loadings for the upper and lower sections of Bay 4.

Table 23 Design Loads

Bay	Location	Critical Load Case	Uplift Head acting upwards on underside of slab (m)			Design Head on top of existing slab (m)		
			Option 3	Option 4[1]	Option 9	Option 3	Option 4	Option 9
Bay 2, 3 and 4	Upstream of Baffles	900 m^3/s with normal TWL	15.4	20.0	15.4	-7.3	-7.3	-7.3
	Downstream of Baffles	900 m^3/s with normal TWL	13.6	20.0	13.6	-7.7	-7.7	-7.7
Bay 1 and 5	Upstream of Baffles	900 m^3/s with normal TWL	12.5	16.9	12.5	-4.5	-4.5	-4.5
	Downstream of Baffles	900 m^3/s with normal TWL	10.6	16.9	10.6	-4.4	-4.4	-4.4

1) *Pressures have been adjusted to accounted for transmission of high pressure jet through open contraction joints.*

In addition to the proposed loads on the slab, it is proposed that the training walls be raised to contain the 1 in 1000 AEP event. Beyond this discharge, flow will overtop the training walls. In the next stage of the project, the extent of wall raising and/or protection to the embankment toe should be further considered.

6.2.2 Acceptance Criteria

The three options were designed to meet buoyancy factors of safety using guidance presented in the USACE Engineering Manual for the Stability of Hydraulic Structures. Specifically, the following equation was used to assess the factor of safety for buoyancy:

$$FoS = \frac{Ws + Wc + S}{U - Wg}$$

Where:

- FoS = Factor of Safety
- Ws = Weight of the structure (assuming moist or saturated conditions as relevant)
- Wc = Weight of the water contained within the structure
- S = Surcharge loads
- U = Uplift forces acting on the base of the structure

– Wg = Weight of the water above the top surface of the structure

The following acceptance criteria were adopted for the FoS for buoyancy:

– Usual = 1.3
– Unusual = 1.2
– Extreme = 1.1

6.3 Option 3 Development

6.3.1 Description of the Upgrade

Option 3, which was selected during the Option identification phase as one of the preferred options, comprises the installation of a new anchored overlay slab. Preliminary sizing of this option has been undertaken, and the key features include:

– 500 mm thick overlay slab on the top of the existing slab
– N32 mm dia. anchors, 1.8 m x 1.8 m grid spacing on the upstream of the baffles and 2.1 m x 2.1 m grid spacing on the downstream of the baffles
– Waterstops and dowels at the contraction joints for the overlay slab
– 500 mm vertical raising of the chute blocks, baffle blocks and end sill
– 500 mm vertical raising of the divider walls
– Triangular extension and vertical raising of the left and right training walls (2.2 m)

A sketch of the proposed option is provided in Appendix E.

6.3.2 Scope of Works

A preliminary scope of works has been prepared for Option 3, and includes:

– Approvals and site establishment
– Dewater stilling basin and install cofferdam
– Surface preparation of the existing slab using hydro-demolition to expose aggregates, and full demolition of the baffle blocks to allow them to be raised to maintain their current heigh off the slab invert.)
– Core drill the existing slab and percussion drill foundation for installation of anchors (32 mm Dia. 8 m deep)
– Install double corrosion protection anchors
– Install waterstops and dowels at the contraction joints
– Install reinforcement to raise the slab, chute blocks, baffle blocks and end sill (500 mm)
– Concrete to raise the slab, chute blocks, baffle blocks and end sill
– Install reinforcement and concrete to raise the divider walls
– Triangular extension and raising the left and right training walls by 2.2 m to contain the 1 in 1,000 AEP flood.
– Site restoration

6.3.3 Review of Advantages and disadvantages

A more detailed review of the advantages and disadvantages of Option 3 has been undertaken. A summary of this review is provided in Table 24.

Table 24 Summary of Advantages and Disadvantages of Option 3

Item	Comments
Advantages	– Meets the requirements of the performance criteria – The required minimum slab thickness is 500 mm to develop the full tensile capacity of anchors – Waterstops and dowels will be installed at the contraction joints – The overlay slab relies mainly on anchors for uplift stability – Anchors are grade 500 MPa bars (design stresses will be kept low) – High quality and durable concrete will be used for the top slab – Slabs will be adequately reinforced with top and bottom reinforcement – The anchor heads will be embedded in the top slab – Potential gap under the original slab can be filled through the new anchor bore holes – All the existing defects on the slab will be covered and protected by the overlay slab – Double corrosion protection anchors will be used – The anchor heads will be anchored to the new slab either by 90 degrees bends or by using head plates – Any potential strength deficiency of the baffle blocks and end sill will be fixed.
Disadvantages	– Close spacing of anchors requires long embedment depth into the foundation (8 m) – Core drilling through the existing slab – Surface preparation for the existing slab, chute blocks, baffle blocks and end sill – Construction period will be longer than Option 4 – Lift joints between the overlay and existing slabs will not be fully bonded joints – Raising of the chute blocks, baffle blocks and end sill to keep the hydraulics the same

6.4 Option 4 Development

6.4.1 Description of the Upgrade

Option 4, which was selected during the Option identification phase as one of the preferred options, comprises the retrofitting of anchors through the existing slab to provide additional resistance to uplift. Preliminary sizing of this option has been undertaken, and the key features include:

- N36 mm Dia. anchors, 1.65 m x 1.65 m grid spacing on the upstream and downstream of the baffles, 12 m embedded into the foundation
- Triangular extension and vertical raising of the left and right training walls (1.7 m)

A sketch of the proposed option is provided in Appendix E.

6.4.2 Scope of Works

A preliminary scope of works has been prepared for Option 4, and includes:

- Approvals and site establishment
- Dewater stilling basin and install cofferdam
- Core drill the existing slab about 300 mm deep X 300 mm Dia. to accommodate the anchor head plates (36 mm Dia. anchors)
- Core drill the existing slab and percussion drill the foundation (12 m deep)
- Install double corrosion protection anchors
- Install vertical dowels around the anchor head plates and use mesh on top and repair the slab
- Triangular extension and raising the left and right training walls by 1.7 m
- Site restoration

6.4.3 Review of Advantages and disadvantages

A more detailed review of the advantages and disadvantages of Option 4 has been undertaken. A summary of this review is provided in Table 25.

Table 25 Summary of Advantages and Disadvantages of Option 4

Item	Comments
Advantages	– Meets the requirements of the performance criteria – No overly slab is required – No raising of the baffle blocks, chute blocks, end sill and divider walls are required – Concreting will be minimum – Potential gaps under the existing slab can be filled using the new anchor bore holes – The existing slab relies mainly on the anchors to resist the uplift pressures – Anchors are grade 500 MPa bars – Anchor head plates will be embedded in the existing slab – Double corrosion protection anchors will be used – The defects on the existing slab will be repaired (although, potentially not as effectively as the overlay options, see below) – Minimum construction time in comparison with the other options.
Disadvantages	– Bigger size and greater number of anchors (36 mm Dia. 1.65 m x 1.65 m spacing) – Anchors require deeper holes (12 m embedment into foundation) – Adding waterstops to the existing slab contraction joints most likely not viable – Repair around the anchor head plates will not be ideal (potentially a weak point) – Any existing defects on the slab will be repaired but remain exposed and may require future maintenance

6.5 Option 9 Development

6.5.1 Description of the Upgrade

Option 9, which was selected during the Option identification phase as one of the preferred options, comprises the partial removal of the existing slab, and replacement with a new anchored slab to the same geometry (and levels) as the existing slab. Preliminary sizing of this option has been undertaken, and the key features include:

– 500 mm depth demolition and reconstruction of the slab
– N32 mm dia. anchors, 1.8 m x 1.8 m grid spacing on the upstream of the baffles and 2.1 m x 2.1 m grid spacing on the downstream of the baffles, embedded 8 m into the foundation.
– Waterstops and dowels at the contraction joints of the slab
– Triangular extension and vertical raising of the left and right sidewalls (1.7 m)

A sketch of the proposed option is provided in Appendix E.

6.5.2 Scope of Works

A preliminary scope of works has been prepared for Option 9, and includes:

– Approvals and site establishment
– Dewater stilling basin and install cofferdam
– Partial demolition (500 mm deep) of the slab, end sill and complete demolition of the baffle blocks
– Core drill existing slab (~ 400 mm) and percussion dril foundation for installation of anchors (32 mm Dia. 8 m depth)
– Install double corrosion protection anchors

- Install waterstops and dowels at the contraction joints
- Install reinforcement on the slab, end sill and baffle blocks and reconstruct them all to the original level
- Triangular extension and raising the left and right training walls by 1.7 m

6.5.3 Review of Advantages and disadvantages

A more detailed review of the advantages and disadvantages of Option 9 has been undertaken. A summary of this review is provided in Table 26.

Table 26 Summary of Advantages and Disadvantages of Option 9

Item	Comments
Advantages	– Waterstops and dowels will be added at the contraction joints – The new slab (reconstructed) relies mainly on anchors to resist the uplift pressures – Anchors are grade 500 MPa bars – High quality and durable concrete will be used for the new slab – The new slab will be adequately reinforced with two layers of reinforcement at the top and bottom – The baffle blocks and end sill will be adequately reinforced – The anchor heads will be embedded in the top slab – Double corrosion protection anchors will be used – Potential gap under the original slab can be filled through the new anchor bore holes – The existing defects on the slab will be removed – The anchor bars are anchored to the new slab either by 90 degree bends or by using head plates
Disadvantages	– Significant concrete demolition works – The adopted demolition methods must prevent cracking/breaking of the existing concrete that is to be left in place, which makes it costly – Joints between the new and existing slabs will not be fully bonded joints – Core drilling is required through the existing slab – Close spacing of anchors requires long embedment depth into the foundation (8 m) – Construction period is the longest compared to the other options

6.6 Preliminary cost-estimates

6.6.1 General

This Section presents the preliminary cost estimates prepared for the three preferred upgrade options described in this report including:

- Assumptions made, and limitations in preparing the cost estimates
- Direct costs on identified construction items
- Mark-ups and allowances in the cost estimates
- Estimated dollar value for the three upgrade options
- A summary of the estimates

A schedule of quantities and breakdown of the cost estimate for each option is included in Appendix G.

6.6.2 Key Assumptions, limitations and accuracy

In addition to the assumptions for the different options previously described in this report, the following additional assumptions were made in the preparation of the preliminary cost estimates:

- The level of detail of the option designs and cost estimates of the present study correspond to a concept screening level study (or a 'Class 5 estimate in accordance with AACEi), which has been undertaken for the purpose of comparing options. This level of study is required where there is more than one business scenario, and it is necessary to determine which one is the best, both technically and financially.
- A preliminary construction program has been prepared for the three options, and has been used as a key basis for some components of the estimate. The program has been prepared using experience and engineering judgment.
- Indirect costs (engineering design, environmental and heritage studies, project management, supervision costs, etc.) are included as percentage mark-ups or lump sum items on the direct costs.
- The cost estimates are only for the implementation of the upgrade options and do not include any allowance for running or operational costs of the implemented options over the life of the dam. For instance, for monitoring or maintenance of the anchors, or ongoing dewatering and condition inspections of the stilling basin. This would be an additional cost over the life of the dam that should be considered in the MCA.

6.6.3 Development of Estimates

6.6.3.1 Direct Costs

The indicative cost estimates presented in this Chapter were developed by GHD's specialist contractor advisor, Mr. Asher Trounce. The estimates have been developed using the following data:

- Quantities applied in the cost estimates were based on data provided by GHD, and calculated using the preliminary option arrangements.
- Unit rates were developed by extrapolating recent similar project pricing, budget quotes from recent similar works, industry unit rates, as well as Mr. Trounce's experience as a specialist cost estimator on large infrastructure projects.
- Time-factored rates were assessed based on a preliminary construction program prepared by Mr. Trounce. A copy of the preliminary construction program for all three options is provided in Appendix F.

The estimates were based on the preliminary option designs and are subject to change as the design develops. A contingency allowance as described below has been added to the estimated costs of the works described in this report.

6.6.3.2 Mark-ups and allowances

The direct costs have been estimated based on approximate quantities and selected unit rates. There are however a number of items that cannot be directly measured. For those items the following percentage mark-ups have been included in the cost estimates, based on past project experience:

- Contractor's supervision, site overheads and profit: [redacted] of direct cost.
- Minor miscellaneous items not measured: [redacted]
- Engineering design (no allowance has been made for physical hydraulic modelling or geotechnical investigations as it is assumed the recently undertaken investigations and current planned geotechnical investigations will be sufficient): [redacted] for all options.
- Environmental, planning and heritage approvals: Assumed lump sum amount of [redacted]
- Project management and construction phase services: 1[redacted] of the contractor's estimate.

Considering the level of accuracy of the option designs undertaken for this study, a contingency allowance has been selected to reflect an 'indicative' or 'concept screening' level of assessment. Therefore a **contingency allowance of** [redacted] has been included in the cost estimates, prepared as part of this study.

6.6.4 Summary of Concept Screening Cost Estimates

A summary of the cost estimates prepared to a 'preliminary' level of refinement, namely for Option 3, 4 and 9, are provided in Table 26. More detailed breakdowns of these estimates are attached in Appendix G.

Table 27 Summary of Concept Screening Cost Estimates

Item No.	Description	Quantity	Option 3	Option 4	Option 9
			$	$	$
SECTION 1 – PRELIMINARIES					
1.1	Fixed and Time Related Charges		[redacted]	[redacted]	[redacted]
1.2	General Site Setup		[redacted]	[redacted]	[redacted]
	Subtotal		[redacted]	[redacted]	[redacted]
SECTION 2 – FLOOD PROTECTION/COFFER DAMS					
2.1	Flood Protection and Coffer Dams		[redacted]	[redacted]	[redacted]
	Subtotal		[redacted]	[redacted]	[redacted]
SECTION 3 – STILLING BASIN SLAB UPGRADE					
3.1	Stilling basin slab		[redacted]	[redacted]	[redacted]
3.2	Stilling basin baffle blocks, end sills and divider walls		[redacted]	[redacted]	[redacted]
	Subtotal		[redacted]	[redacted]	[redacted]
SECTION 4 – STILLING BASIN TRAINING WALL RAISING					
4.1	General Items		[redacted]	[redacted]	[redacted]
4.2	Chute wall triangular extension – L&R sides		[redacted]	[redacted]	[redacted]
4.3	Stilling Basin training wall raising L& R sides		[redacted]	[redacted]	[redacted]
	Subtotal		[redacted]	[redacted]	[redacted]
SECTION 5 – MISCELLANEOUS					
5	Subtotal		[redacted]	[redacted]	[redacted]
TOTAL CONTRACTOR'S ESTIMATE					
	Total Direct Cost		[redacted]	[redacted]	[redacted]
	Contractor's Supervision and Site Overhead	[redacted]	[redacted]	[redacted]	[redacted]
	Minor miscellaneous items not measured	[redacted]	[redacted]	[redacted]	[redacted]
	Contractor's Estimate (excl contingency)		[redacted]	[redacted]	[redacted]
	Contingency allowance (including allowance for floods during construction)	[redacted]	[redacted]	[redacted]	[redacted]
	Indicative Contractor's Estimate (excl. GST)		[redacted]	[redacted]	[redacted]
OTHER INDIRECT COSTS					
	Engineering design	[redacted]	[redacted]	[redacted]	[redacted]
	Environmental and planning approvals	[redacted]	[redacted]	[redacted]	[redacted]
	NCA's Management Costs (% of Contractor's Estimate)	[redacted]	[redacted]	[redacted]	[redacted]
	Other Indirect Costs (excl contingency)		[redacted]	[redacted]	[redacted]
	Contingency allowance	[redacted]	[redacted]	[redacted]	[redacted]
	Indicative Other Indirect Costs (excl. GST)		[redacted]	[redacted]	[redacted]
TOTAL PROJECT ESTIMATE					
	Indicative Contractor's Estimate (excl. GST)		[redacted]	[redacted]	[redacted]
	Indicative Other Indirect Cost (excl. GST)		[redacted]	[redacted]	[redacted]
	Total Indicative Cost Estimate (exc. GST)		[redacted]	[redacted]	[redacted]

A number of comments are made in relation to the preliminary cost estimates, as follows:

- The estimates have been based on complete demolition of the baffles, chute blocks and end sill for Option 3, as opposed to constructing a ‘skin’ to raise these features. This work adds a significant amount to the cost estimate. In the next stage of the project, it will be important to assess the merits of complete removal versus modifying the existing baffle and chute blocks.
- The current estimates assume all anchors will be hot-dip galvanized, in addition to providing double corrosion protection. Depending on the final arrangement, the black bar may be used.
- Option 4 has been designed for hydraulic loads allowing for full penetration of loads through open contraction joints. No allowance has been made in the cost estimate to allow for retrofitting of waterstops in these joints.
- Demolition works required in Option 9 are complex and time-consuming, and have the potential for substantial environmental impacts (noise, vibration etc.). The current cost estimates assume only hydro-demolition of the slabs, but it is noted that a combination of wire-cutting and hydro-demolition may be possible, and may result in a cheaper outcome. This aspect will require further consideration if Option 9 is selected as the preferred option.
- It is highlighted that the construction industry has experienced significant recent escalation in prices. For example, the price of steel has risen by around 12% in the last 3 months. The potential for on-going price escalation, and how to manage this risk, will need to be considered in future stages of the project.
- The preliminary cost estimates are intended for comparative purposes only, between the options, and should not be relied upon for budget purposes.

6.7 Preferred Option Workshop

A workshop was held on Friday 17th September 2021 to present and discuss the three preferred options, and select the preferred option for the Concept Design. During the Preferred Option Workshop, a Multi-Criteria Assessment (MCA) was undertaken with the participants to agree on the preferred option to take through to concept design. A detailed set of minutes were prepared during the workshop, a copy of which is provided in Appendix I, and the outcomes of the MCA process is described in the following section.

6.8 Multi-Criteria Assessment (MCA)

6.8.1 Process

A Multi-Criteria Assessment (MCA) was undertaken and was used as a tool for identifying the preferred option to be selected to progress to Concept Design. Although the MCA provided a structured process for assessing the options, it is noted that the final outcome was reviewed using engineering and business judgment to substantiate and endorse the overall selection of the preferred option.

This section provides details of the MCA process, including the following key steps:

- Selection of the criteria to be adopted in the MCA
- Determination of weightings to be used for each key criteria
- Development of a definition for scoring against each criteria
- Scoring process
- Results

A copy of the MCA forms used in the assessment are included in Appendix H.

6.8.2 Selection Criteria

In the early stages of the Option Development, a list of potential selection criteria was developed. The list of criteria was divided into five key subject areas, comprising:

- Cost
- Technical Merits
- Construction
- Operation and Maintenance, and
- Other Aspects

This list was provided to NCA, and was discussed between NCA and GHD during a meeting held on 23 August 2021. During this meeting, the criteria were further refined into a list of eleven criteria. These criteria are provided in Table 28. During the Preferred Option Workshop, further discussion occurred during the scoring process of the MCA regarding the applicability of some of the criteria. As a result, 'meets the design criteria' was removed from the criteria as it was agreed that all options need to meet the design criteria in order to progress to concept. As such, this item was removed to leave ten key selection criteria, as highlighted in Table 28.

Table 28 List of Selection Criteria and associated evaluation method

Blocks	ID	Criteria	Description	Method of Evaluation
A - Cost	1	Project Costs	Overall project costs for the upgrade works (design and construction costs)	Preliminary Cost Estimates
	2	OPEX	Likely on-going O&M costs	Engineering Judgement
B - Technical Merits	3	Meets Design Criteria	Ability to meet the design criteria, including specifically the ability to resist loads, design life, and overall likely hydraulic performance	Comparison with design criteria
	4	Robust & Durable Solution (beyond design criteria)	Additional design features beyond NCA requirements, or likely overall success of the upgrade to achieve a robust solution including risk to business	Identification of benefits beyond design criteria
	5	Benefits on Dam Safety	Whether the proposed solution improves the stability or safety of the dam (e.g. will the dam need to be upgraded to accommodate additional loads/ tailwater conditions)	Impact of the upgrade on dam Failure Modes (qualitative)
C - Construction	6	Construction simplicity/complexity	Ability to be safely constructed by a contractor with common industry procedures. The likely ease of construction, and associated timeline.	Constructability Review
	7	Flood Management during Construction	Ability to safely divert floods during the construction period without impact on the works or users - risk management of floods	Risk assessment
D - Operation and Maintenance	8	Maintenance Requirements	Requirements and ease of long-term maintenance on the stilling basin	Engineering Judgement
	9	Surveillance and Operation	Ability to observe, monitor and understand the condition of the stilling basin	Engineering Judgement
E - Other Aspects	10	Social and Environmental Impacts	Impacts to local community including recreational usage (expected predominantly during construction) - e.g. impacts on zoo Impacts to water quality impacts during the construction works (e.g. hydro-demolition, recycling material) Potential impacts to local ecosystem if existing, including features allowing fish passage	Identification of impacts
	11	OH&S Safety	Potential OH&S aspects associated with construction and O&M, and public safety	Safety in Design process

6.8.3 Criteria Weightings

In order to assign weightings for each of the criteria, a matrix-style weighting process was undertaken. The process involved listing the ten selection criteria long the top row (listed 'A' to 'K') and first column (listed '1' to '10') of a matrix table, as shown in Table 29. For each 'blue' cell within the table, the question was posed as to whether the row heading or column heading of the relevant cell was viewed to be more important in terms of selection criteria. For example, for cell 'F2', the question was asked, "Is Flood Management during Construction considered to be more important than the OPEX costs". In this particular circumstance, 'Flood Management during Construction' was considered more important, and therefore cell F2 was assigned this selection criteria. This process was undertaken for every cell in the table until each criteria had been reviewed against the remaining criteria. Once completed, the total number of cells returning each criteria was counted and converted into a percentage of the total number of cells to provide an overall weighting for each criteria. The final weighting table developed for the project is provided in Appendix H.

Table 29 Example of Weighting Matrix Process

		A	B	C	D	E	F	G	H	I	J
	Criteria	**Project Costs**	**OPEX**	**Robust & Durable Solution**	**Benefits on Dam Safety**	**Construction complexity**	**Flood Management during Construction**	**Maintenance Requirements**	**Surveillance and Operation**	**Social and Environmental Impacts**	**OH&S Safety**
1	**Project Costs**										
2	**OPEX**										
3	**Robust & Durable Solution (beyond design criteria)**										
4	**Benefits on Dam Safety**										
5	**Construction simplicity/ complexity**										
6	**Flood Management during Construction**										
7	**Maintenance Requirements**										
8	**Surveillance and Operation**										
9	**Social and Environmental Impacts**										
10	**OH&S Safety**										

6.8.4 Scoring descriptors

In order to provide guidance to the scoring process, a list of descriptors were developed for each criterion to described what would be viewed as a high versus low score for each criterion. In some cases, examples were developed for further description. A summary of these descriptors is provided in Table 30.

Table 30 Scoring Descriptors

Blocks	Criteria	No-Pass/1-3 score	Average Score/ 5	Pass/8-10 score
A - Cost	Project Costs	Cost disproportionate to the provided benefits and not in line with typical practice. *Example: Deduct one point for every 20% cost higher than average.*	Average cost	Strong cost justification. *Example: Add one point for every 20% cost lower than average.*
	OPEX	Disproportionate for a typical OPEX budget and not in line with common practice	Considered to be average OPEX	No significant costs associated beyond regular surveillance
B - Technical Merits	Robust & Durable Solution (beyond design criteria)	Parts of the solution are unlikely to meet a 100 year design life, or will require significant maintenance. *Example: Uncertainty that the retrofitted waterstops will be robust enough to withstand loads, and may need to be replaced after say 30 years.*	Design life of 100 years likely to be achieved with minimal maintenance	Additional benefits with a strong cost justification; improves business resiliency
	Benefits on Dam Safety	Upgrade works have an adverse impact on the safety of the dam. *Example: Stilling basin works likely to increase groundwater pressures beneath dam or inhibit discharge from existing drainage system.*	No changes to overall safety of the Dam (except for inherent improvements to the stilling basin)	Upgrade works is likely to have a beneficial impact on the overall safety of the dam. *Example: Drainage curtain/system installed in stilling basin likely to reduce phreatic surface beneath dam.*
C - Construction	Construction simplicity/complexity	Highly specialised contractors are required, with significant uncertainties on time and costs. *Example: Extensive hydro-demolition works requiring specialised sub-contractors and long program duration.*	Considered to be average in terms of construction complexity. *Example: Regular Tier-1 and Tier-2 contractors can undertake the work. Time and budget forecasts are reliable.*	Option has easier construction complexity than other options.
	Flood Management during Construction	Factors associated with the option make passing of floods more complicated or risky than other options. Significant risk of damages, delays and cost overruns; mitigating actions are not practicable. *Example: Removal or partial removal of existing slab during construction makes stilling basin vulnerable to overtopping events during construction.*	Ability to manage/pass floods during construction is similar to other options.	Factors associated with the option make passing of floods easier than other options. Floods not expected to cause damages or affect planning. *Example: Option leaves existing slab in place which means overtopping is lower risk, and construction works are very fast resulting in less exposure time.*

Blocks	Criteria	No-Pass/1-3 score	Average Score/ 5	Pass/8-10 score
D - Operation and Maintenance	Maintenance Requirements	Maintenance will require specialised means and periodic limitations on the dam's operation (ie regular dewatering and testing).	Maintenance can be carried out within a regular surveillance program. *Example: Maintenance would be required once every 30-50 years and would involve minor patching of scour/spalled areas.*	Maintenance likely to be minimal over design life. *Example: Maintenance likely to be very low (say <50 year intevals) and features are incorporated into the design to make dewatering very easy.*
	Surveillance and Operation	Condition of the stilling basin cannot be assessed without an investigation program	Condition of the basin can be monitored within regular surveillance program. *Example: Regular surveillance of upper parts of basin, and dewatering of submerged section required every 10-20 years.*	Surveillance requirements likely to be minimal over design life or made easier. *Example: Features are incorporated into the upgrade design to allow easy dewatering of the basin, and/or a suite of instruments are used to monitor performance in submerged areas.*
E - Other Aspects	Social and Environmental Impacts	Significant impacts. Community opposition to the project. Regulatory approval process likely to require management plans or changes to design and construction procedures.	Some impacts expected during construction, but are manageable and have a reasonable cost basis. *Example: Normal restrictions on working hours, works don't cause excessive vibration, noise or dust (particularly to zoo or Government House).*	Impact on the community is low or can be easily mitigated. Approval process expected to be straightforward.
	OH&S Safety	Significant safety concerns. Elimination or mitigation are not practicable or would impose limits on the operation of the dam.	Some OH&S aspects will need to be considered, but are manageable. *Example: SiD process doesn't identified any high-risk items, and OH&S durign construction can be managed through Contractor's safety plan and protocols.*	No significant safety concerns for public or operators

6.8.5 Assessment of Preferred Option

The MCA scoring spreadsheet was completed, using the key selection criteria, weightings and scoring definitions description in the previous sections. The results of the scoring is provided in Appendix H. The scoring spreadsheet provided in Appendix H also includes comments on the basis for the scoring of each option. A summary of the final aggregate of scores is provided in Table 31.

Table 31 Summary of MCA scores

	Option 3	Option 4	Option 9
A - Cost	7	7	6
B - Technical Merits	15	12	15
C - Construction	13	12	8
D - Operation and Maintenance	5	3	5
E - Other Aspects	14	16	10
Total Score	53	50	43

The MCA process determined that Option 9 scored vastly lower than Option 3 and 4. Some of the key drivers for Option 9 scoring so much lower than the other two options include:

- The option was estimated to cost considerably more than Option 3 and 4
- The option is considered to be significantly more difficult to construct than Option 3 and 4 due to the vast amount of demolition required.
- The associated social and environmental impacts of the demolition works are likely to result in significant challenges for the project (i.e. dealing with waste, noise and vibration, and associated impacts on the zoo and local community parklands).

Option 3 and 4 had more similar scores, with Option 3 scoring slightly higher than Option 4 (i.e. 53 versus 50). A high-level sensitivity analysis was undertaken on the scores for Option 3 and 4, firstly by adjusting the weightings applied to 'Project Cost' (i.e. increasing this weighting), and secondly by increasing the weighting on 'Robustness and Durability' (noting that a robust solution is likely to minimise long-term maintenance, surveillance and OPEX costs). In all cases where the weightings were viewed to be potentially arguable, changing the weightings did not result in the score for Option 4 exceeding that of Option 3.

As a further means of reviewing the scores, Option 3 and 4 were reviewed to determine which scored highest against each criteria (unweighted assessment). Table 32 provides a summary of the results. It can be seen that in six instances, Option 3 scored equal or better than Option 4, while Option 4 only scored higher or equal in five instances. As such, from this perspective, Option 3 continues to be the preferred option.

Table 32 Unweighted Assessment against criteria

Criteria	Best Option to Meet Criteria	Basis
Project Costs	Option 4	Option 4 is cheaper, as determined by preliminary cost estimates
OPEX	Option 3	Option 3 includes complete renewal of the slab, whereas Option 4 relies on using the existing slab (which is already 60 years old). As such, it is expected that Option 4 will require more ongoing maintenance (OPEX) than Option 3.
Robust & Durable Solution (beyond design criteria)	Option 3 and 9	Option 3 and 9 has more robust details, particularly around waterstops at contraction joints, and includes the construction of a new slab allowing existing slab defects to be addressed.
Benefits on Dam Safety	Option 3 and 4 had equal scores	Both options viewed to have similar merits.
Construction simplicity/complexity	Option 3	Option 3 was reviewed to have slightly easier constructability aspects, as Option 4 will require some complex details around the anchor head area.
Flood Management during Construction	Option 4	Option 4 was scored slightly higher than Option 3 as the program is likely to be slightly shorter resulting in less exposure time.

Criteria	Best Option to Meet Criteria	Basis
Maintenance Requirements	Option 3	Option 3 was scored slightly higher for similar reasons to those stated under 'OPEX'
Surveillance and Operation	Option 3	As per 'Maintenance Requirements'
Social and Environmental Impacts	Option 4	Option 4 was scored higher as it will use less materials (limited concrete works) than Option 3, therefore resulting in less truck movements and interruptions to the community.
OH&S Safety	Option 4	Option 4 was deemed to have less OH&S exposure due to less components being involved in the works, and the construction duration being slightly shorter.

Following the MCA review during the Preferred Options Workshop, a discussion was held with participants to consider the preferred option from an engineering judgment perspective. It was widely agreed by participants that irrespective of the MCA scores, Option 3 was viewed to be the preferred option. The key factors in this discussion were:

– There is concern that Option 4, while cheaper, does not provide sufficient robustness particularly in terms of the anchor head arrangement, and does not achieve the overall intent of the project. As such, Option 4 should be eliminated from further consideration;
– Option 9 has similar advantages to Option 3, but is significantly more expensive and creates a flood risk during construction due to the need to partially demolish the existing slab. As such, Option 3 is preferred over Option 9.

In summary, on the basis of the results of the MCA, a review of the sensitivity of results, and discussion with participants who attended the Preferred Options Workshop, it is proposed that Option 3 be selected as the preferred option for development in the Concept Design phase.

7. Conclusion and Recommendations

The option assessment undertaken on Scrivener Dam stilling basin has included an initial option identification phase, during which nine potential options were identified. These options were reviewed and discussed during an Option Identification Workshop, which resulted in three preferred options being selected for further refinements. The preferred options comprised:

– Option 3 – Installation of an anchored overlay slab
– Option 4 – Retrofitting of new anchors into the existing slab
– Option 9 – Partially demolishing the existing slab, and installing a new anchored slab to the original geometry.

These options were further developed to determine approximate sizing for the key features, and to identify key advantages and disadvantages in each option. As part of the options development, preliminary construction programs and cost estimates were prepared, and were used as part of an MCA process to identify the preferred option to take through to concept design.

This work was presented during a second workshop, the 'Preferred Option Workshop', which included working through the MCA process with the participants. The MCA process used a number of different processes to assess the options, and included a sensitivity analysis which was undertaken following the workshop. Furthermore, an open discussion was held during the workshop to capture the participants' thoughts on which option was likely to be the preferred solution for upgrading the spillway. The MCA process identified that the preferred option is Option 3, and this choice was supported by the participants in the workshop. As such, it is recommended that Option 3 be progressed to Concept Design.

During the development of the options, a number of items have been identified as requiring further consideration during the Concept Design. Items specific to Option 3 include:

– The current option assumed the minimum slab thickness with an initial layout of anchors. It is recommended that the arrangement of slab thickness and anchor layout be further refined, potentially including increasing the spacing between anchors
– Detailing of the anchor arrangement will be required, including preliminary detail of the double corrosion protection requirements, and selection of materials to be used in the upgrade works.
– The layout of contraction joints needs further consideration with the view to minimise the number of contraction joints.
– The preliminary cost estimates identified that complete demolition and rebuild of the chute blocks, baffles and end sill adds a considerable cost to the project. The concept design should investigate alternatives to complete demolition and rebuild.
– Further analysis of flood protection works will be required in the concept design phase.

The current level of cost estimating was considered adequate to provide valid comparisons of costs between the three short listed options. They are not however, intended to provide a valid project cost for budgeting purposes. A more detailed cost development, based on the refined design for Option 3, combined with a Monte Carlo simulation to better allow for cost uncertainty, will be required to provide the NCA with a likely project cost that can used for budgeting purposes.

8. References

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