Preliminary Design For Seepage Remediation at a Small, Remote Dam – Case Study of Lac La Ronge Dam in Northern Saskatchewan

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ABSTRACT
The geotechnical challenges associated with aging infrastructure are forefront to an owner of 69 dams. The dam in this case study, Lac La Ronge Dam, was constructed in 1966 with a core constructed of locally sourced silty soils and a rock-fill shell. During annual inspections in the spring of 2017, Water Security Agency (WSA) staff identified three significant depressions in the crest of the dam. A subsequent geotechnical investigation confirmed evidence of piping which triggered an emergency response. Temporary measures were deployed to slow seepage while a permanent remediation could be designed and implemented. Clifton Associates Ltd. was retained to complete the remedial design work including: (a) preliminary design; (b) field investigation, (c) final design, and; (d) construction supervision. Five remedial options were considered: (i) grout curtain, (ii) vibro-compaction, (iii) geomembrane installation downstream of the dam core, (iv) slurry wall, and (v) soil mixing. After assessing these options, the recommended approach was to implement a seepage cutoff using proven technologies for the known site conditions. Conceptual designs were developed for two options: (1) downstream geomembrane cutoff and (2) sheet pile and grout curtain cutoff. A detailed field investigation program consisting of geophysical surveys, drilling, instrumentation installation, packer testing and tracer dye testing was completed to characterize the site. The objectives of the investigation were to obtain soil samples to characterize the subsurface, delineate the seepage area, define the groundwater conditions, and refine the geotechnical design parameters. Remediation work is scheduled for summer, 2019. This paper will discuss the case study of the Lac La Ronge Dam, from construction of the dam to the chosen remedial design.

RÉSUMÉ
Les défis géotechniques associés au vieillissement des infrastructures sont au premier plan pour un propriétaire de 69 barrages. Le barrage dans cette étude de cas, le barrage du lac La Ronge, a été construit en 1966 avec un noyau constitué de sols limoneux d'origine locale et d'une coquille d'enrochement. Lors des inspections annuelles tenues au printemps 2017, le personnel de la Water Security Agency (WSA) a identifié trois dépressions importantes dans la crête du barrage. Une enquête géotechnique ultérieure a confirmé la présence de tuyauterie qui a déclenché une intervention d'urgence. Des mesures temporaires ont été déployées pour ralentir les infiltrations, tandis que des mesures correctives permanentes pourraient être conçues et mises en œuvre. Clifton Associates Ltd. a été retenue pour mener à bien les travaux de conception corrective, notamment: (a) la conception préliminaire; (b) enquête sur le terrain, (c) conception finale, et; d) supervision de la construction. Cinq solutions ont été envisagées: (i) un rideau de coulis, (ii) un compactage vibrant, (iii) une installation de géomembrane en aval du noyau du barrage, (iv) un mur de suspension, et (v) un mélange de sol. Après avoir évalué ces options, l'approche recommandée consistait à mettre en œuvre une coupure des infiltrations à l'aide de technologies éprouvées pour les conditions de site connues. Des concepts ont été développés pour deux options: (1) coupure de géomembrane en aval et (2) coupure de rideau de palplanches et de coulis. Un programme d'investigation sur le terrain détaillé comprenant des levés géophysiques, des forages, l'installation d'instruments, des tests d'emballeurs et de colorants traceurs a été achevé pour caractériser le site. Les objectifs de l'enquête étaient d'obtenir des échantillons de sol pour caractériser le sous-sol, délimiter la zone d'infiltration, définir les conditions des eaux souterraines et affiner les paramètres de conception géotechnique. Les travaux d'assainissement sont prévus pour l'été 2019. Cet article abordera l'étude de cas du barrage du lac La Ronge, depuis la construction du barrage jusqu'à la conception d'assainissement choisie.

1 INTRODUCTION

Water Security Agency (WSA) owns and operates 69 dams in the province of Saskatchewan ranging in size from very large (Gardiner Dam, forming lake Diefenbaker), to small earth dams and in-stream weirs. WSA's dams are located all over the province with some remote and northern locations. WSA operates and maintains its structures in accordance with their internal dam safety management policy, based on Canadian Dam Association (CDA) Dam Safety Guidelines. The policy requires regular monitoring and periodic evaluation of performance with geotechnical instrumentation, annual inspections and dam safety reviews.

In the spring of 2017, WSA operations staff noted three significant depressions in the crest of Lac La Ronge Dam. A subsequent geotechnical inspection and investigation confirmed evidence of piping which triggered an emergency response. Temporary measures were put in place to slow seepage until permanent remedial measures could be designed and implemented.

Clifton Associates Ltd. (Clifton) was retained to complete the remedial design work. This paper will discuss the case study of the Lac La Ronge Dam, from construction of the dam to the remedial design, including the remedial option to be constructed in the summer of 2019.

2 BACKGROUND

2.1 Location and History

Lac la Ronge Dam is located in Northern Saskatchewan, approximately 56 kilometres northeast of the Town of La Ronge (Figure 2.1). The Dam was constructed between 1966 and 1967 to control water levels on Lac la Ronge and is only accessible by aircraft or boat.

A major rehabilitation of the concrete control structure and embankment was completed between 2005 and 2007. A bypass channel was constructed on the downstream side of the embankment to facilitate flow from the lake during the rehabilitation of the control structure. This bypass channel was converted to a rock riffle fishway in 2007 after the completion of the control structure rehabilitation. Figure 2.2 shows the major components of the dam, which include the embankment, 4 bay control structure, fishway channel and spillway approach channel.

2.2 Climate

Lac La Ronge Dam is located in a borderline subarctic climate (Köppen Dfc), slightly below the threshold of a humid continental climate. Winters are long, dry and very cold while summers are short, warm and wet. Precipitation is low, with an annual average of 497.1 mm consisting of 343 mm of rainfall and 154.1 mm of snow. The highest temperature recorded in La Ronge was 37.2 °C on August 23, 1929. The coldest temperature ever recorded was -52.2 °C on February 15, 1936. The average annual temperature is -1 °C, consistent with the southerly fringe of the discontinuous permafrost zone.

2.3 Stakeholders

The dam regulates the water level of Lac La Ronge, the fifth largest lake in the province and a popular vacation spot. Recreational activities include fishing, boating, canoeing, hiking, and camping. Lac La Ronge Provincial Park, one of the largest parks in the province, extends around the lake on three sides. The project area is in Treaty No. 10 territory, signed in 1906, and is the traditional land of the Woodland Cree and Dene First Nations (Barry 1999).

2.4 Geological Setting

The dam is founded on metamorphosed, Precambrian basement rock consisting of tectonic schists and mylonite, along the west bank of the river and to the south side of the dam. The east bank and north side of the dam has porphyritic and quartz monzonite and granite as per the surficial geology report for the area. This is an area of well defined lineations that govern the general surface topography of the area. Geophysical investigation indicated that the lineations are likely related to bedrock shear structures that have been preferentially eroded by glacial ice, creating higher permeability zones in the foundation.

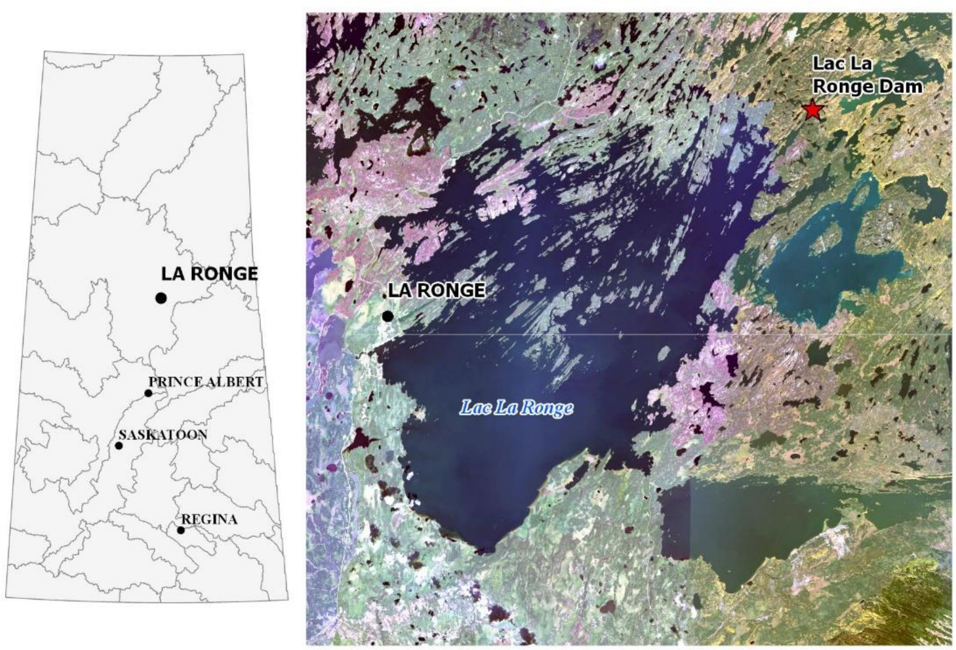


Figure 2.1. Location of the dam

Figure 2.1. Aerial view of the major components of the dam

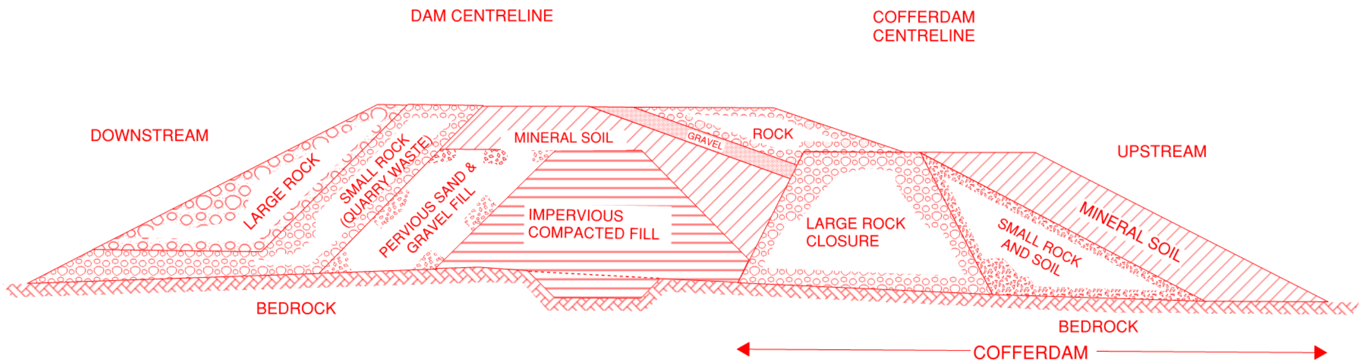


Figure 2.2. Typical Design Section

2.5 Dam Cross Section

The design cross section of the dam is shown in Figure 2.3. It is an impervious compacted silty fill core with a downstream gravel filter. The upstream and downstream faces of the dam are protected by a rockfill shell. No data book or construction records are available for this structure. It is reported that the maximum height of the embankment is 3.70 m.

2.6 Piezometric Regime

Piezometric conditions are controlled by water levels in Lac La Ronge, upstream of the dam, and Rapid River downstream of the dam. Piezometric monitoring reflects the expected gradients from headwater to tailwater. The Full Supply Level (FSL) for Lac La Ronge is 364.40 m while, downstream, Rapid River is typically at 362.8 m.

3 DAM SAFETY MANAGEMENT CHALLENGES

3.1 Management of Lac La Ronge Dam

Water Security Agency follows a dam safety program based on the CDA Dam Safety Guidelines. The program requires monitoring and inspection to ensure facilities are operated and maintained safely.

Monitoring frequencies of each structure are determined by several factors: (a) past performance and technical maintenance history, (b) consequence classification, (c) recommendations from dam safety reviews (DSR), and (d) the location of the dam.

Lac La Ronge is a significant consequence dam. It is scheduled to be inspected annually by WSA staff with the instrumentation monitored once per year.

3.2 Aging Dams

Lac La Ronge dam was constructed between 1966 and 1967. Critical documents such as as-constructed drawings, construction and material specifications, foundation information, etc. are not available for the dam. This situation is not unique to this dam and is common at many low hazard dams built around the 1960s. The lack of as-built records makes rehab works challenging and difficult to ensure that the geotechnical integrity of the dam is maintained.

The challenge of aging infrastructure is not unique to the dam industry and similar challenges are faced across North America. Much of the infrastructure is nearing or beyond design life. Maintenance costs increase, and questions arise about the integrity of subsurface components. The underlying dam safety issues with aging dams are usually technically complex and information about foundations, and even the materials from which the dam itself is constructed, is less than desirable in many cases (Bowles et al. 1999)

3.3 Remote Site Location

Lac La Ronge Dam is accessible by aircraft or boat only which makes data collection and maintenance activities challenging and costly. Remote monitoring over satellite networks are being considered to reduce monitoring costs.

When dam maintenance is required, the remote location limits equipment access by the size of the barge servicing the site and the number of trips the budget will support. Suitable earth materials for use as fill are limited and deforestation would be required to access them. Remedial work and maintenance need to be innovative and rapid as the costs are exponentially higher due to location, regardless of the consequence of the dam.

3.4 Lack of materials

Lack of suitable construction materials is a result of the regional geology which is typically shallow surficial non plastic or low plastic sandy gravelly tills overlying bedrock. Bedrock depressions typically form peat bogs (muskegs), with thin variable surficial soils, making it difficult to find a suitable quality and volume of soil for construction. The till is a clay-based matrix of fine uniformly graded sand and silt with numerous cobbles and boulders which is highly erodible. Great care must be taken in the design of filters with local materials to prevent the mobilization of embankment material due to seepage forces.

4 SEEPAGE PROBLEM AND TEMPORARY REPAIR

In the spring of 2017, a WSA maintenance crew noticed three depressions in the crest of the embankment near the upstream slope and a possible tension crack at the top of the rip rap on the downstream slope along the fishway channel.

Water Security Agency conducted a geotechnical investigation on October 12, 2017 to characterize the soils in the dam embankment and their susceptibility to piping. The disturbance caused by drilling was significant enough to generate a silt plume in the downstream fishway.

Uncontrolled seepage is a major problem for dam safety, often occurs without any apparent warning, which ultimately can lead to failure (Flores-Berrones et al. 2010; CDA, 2017). In response, WSA declared a hazardous condition and a temporary emergency repair of the dam was completed. The emergency repair of the dam included:

a) Filling of the existing depressions with imported soil;
b) Installation of a filter blanket on the downstream slope to reduce the loss of fines through the embankment; and,
c) Installation of steel sheet piles to increase the length of the flow path through the dam.

5 PRELIMINARY DESIGN

In the spring of 2018, Clifton was retained to complete the remedial design scope including:

- preliminary design;
- field investigation;
- final design; and,
- construction supervision.

With a lack of design and detailed soil testing information much of the internal dam structure, material properties and seepage pathways remain uncertain.

5.1 Site Inspection

A site inspection was performed on 07 June 2018 by Clifton personnel, accompanied by WSA staff. The dam crest did not indicate significant changes to the areas where the previous depressions had been backfilled in 2017. The water level on the upstream slope was approximately 1 m below the dam crest at the time of inspection. On the upstream slope rip rap was typically present, except at the north end of the steel sheet piles where a slump was observed. The water level on the downstream slope was approximately 2.5 m below the dam crest at the time of inspection, which assisted with observation of the downstream dam toe and points of seepage previously identified. Significant seepage, estimated to be 90 L/min, was observed exiting near the toe of the slope over a length of approximately 2 m along the dam at the same locations observed in 2017.

5.2 Preliminary Design Options

Preliminary design options were evaluated against the following criteria:

Low environmental impact: The dam is in a provincial park, on a river that is a highly valued resource that supports the way of life of local stakeholders. Any repair must minimize disturbance, both during construction and in the long-term.

Design life: The repair should last for the lifetime of the embankment, with little or no expected maintenance beyond routine inspections. Post-installation monitoring should be straightforward.

Ease of implementation (constructability): Construction solutions are constrained by site access and environmental concerns. Transportation of materials and equipment to the site will be by barge, further constraining potential solutions. Finally, there is limited space available onsite for construction activities due to the relatively narrow crest of the embankment and absence of adjacent work and laydown areas.

Abilities of local contractors: Local knowledge and resources, and the ability to execute the work locally, should be an important consideration in selection of options.

Availability of suitable materials: The materials needed to complete the repair must be readily and locally available to allow competitive pricing and reduce cost.

Since the principal seepage pathway was deemed to be through the fractured bedrock foundation, the following remediation concepts were considered (Clifton 2018):

1) **Upstream control of gradients** through the dam using a geomembrane and an extended upstream blanket to control hydraulic gradients to a safe level.
2) **Cutoff improvement** through the core and into the foundation.
3) **Downstream seepage control** from the core through a combination of a geomembrane cutoff and a downstream filter blanket to control groundwater flow exit gradients.

Within the framework of this design criteria and the three locations for remedial options, six different options were deemed reasonable for evaluation:

a) Grout curtain upstream of existing sheet pile wall installed to the fractured bedrock interface;
b) Vibrocompaction of the dam core;
c) Geomembrane installation on the downstream face of the core;
d) Soil cement bentonite slurry wall along dam core between two sheet pile walls;
e) Deep soil mixing with soil cement/bentonite replacement along dam core centreline; and,
f) Monitor structure, no remediation.

6 FIELD INVESTIGATION

6.1 Drilling

Five boreholes were drilled by Maple Leaf Ltd. (Maple Leaf) using a Geoprobe 7822DT track mounted drill rig from October 1-3, 2018. A 150 mm diameter solid stem auger was used to drill a pilot hole and cut through the layers of geotextile in the top 0.76 m of the boreholes. Below 0.76 m, the direct push method was used to drill an 82 mm diameter borehole and the rotary coring method using a 102 mm casing and NQ (76 mm) core barrel was used for drilling into the bedrock. Three boreholes (BH3510, BH5010 and BH5020) were drilled to direct push refusal (bedrock surface) which ranged from 3.3 to 4.5 metres below ground surface (mbgs). Two boreholes (BH4010 and BH4020) were drilled approximately 5 m into the bedrock to a total depth of 8.4 and 8.3 mbgs, respectively. The borehole layout is shown in Figure 6.1.

6.2 Instrumentation

In order to measure porewater pressures and assess the hydraulic gradient through the dam, each borehole had a nest of vibrating wire piezometers (VWP) installed as follows:

- Above or at the bedrock surface; and,
- Approximately 3 m into the bedrock.

The VWPs were grouted in place, protected at surface with a steel casing protector, and connected to a RST Instruments single channel data logger.

6.3 Packer Testing

Packer testing was completed in the bedrock for Boreholes BH4010 and BH4020 to understand the flow pathways through the dam. Successful packer testing was completed for two zones in each borehole. A summary is provided in Table 6.1. Additional packer testing depths/zones were unsuccessful because the packers were unable to seal against the core hole in the gravelly/highly fractured bedrock zone. Higher flows observed during the packer testing near the top of the bedrock coincide with lower RQD values in the core and observations of highly fractured zones during core logging.

Table 6.1. Summary of Packer Testing

Borehole ID	Packer Zones (mbgs)	Flow (Litres/minute)
BH4010	4.6 to 6.7	60
	6.7 to 8.4	Little to no flow
BH4020	4.6 to 6.7	4
	6.7 to 8.3	Little to no flow

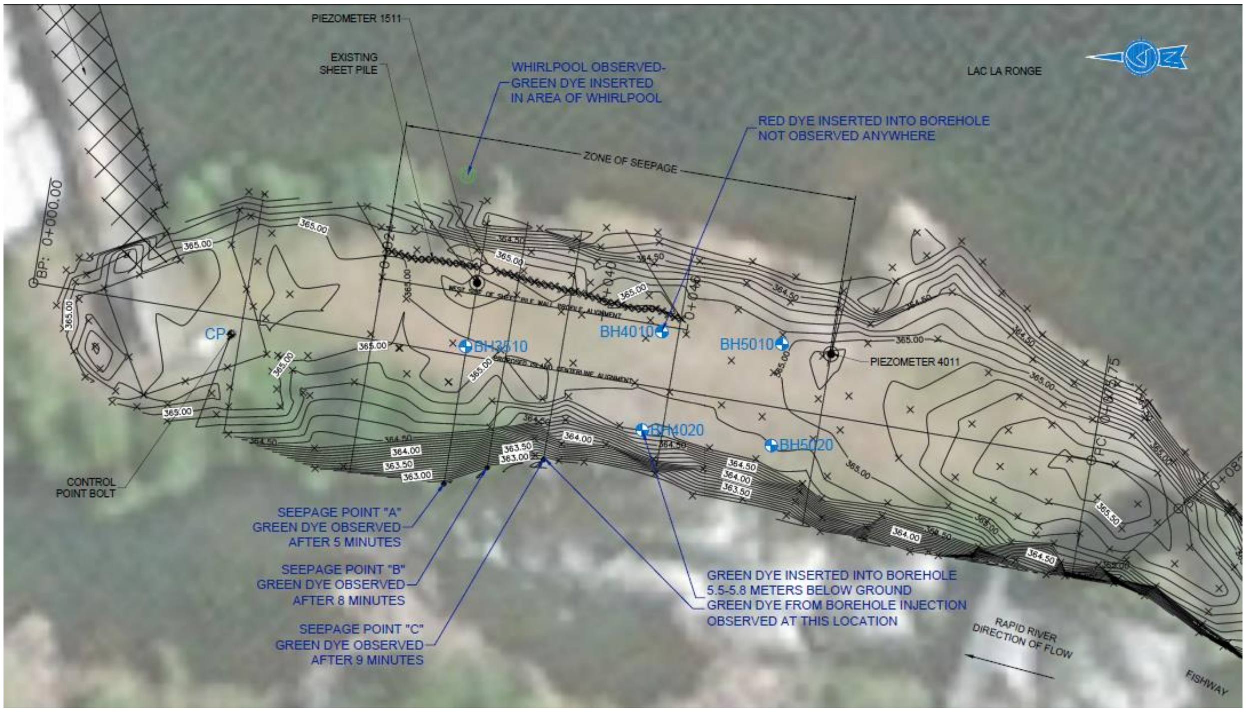


Figure 6.1. Plan view of Lac La Ronge Dam Embankment with Borehole Location

6.4 Tracer Dye Testing

Tracer tests are widely used in hydrogeology to asses the direction and velocity of groundwater (Battaglia et. al., 2016). During packer testing, different colours of tracer dye (red and green) were used to visually observe water flow through the dam structure. Observations were made on the left and right sides of the dam. Refer to Figure 6.1 for the following discussion.

A small whirlpool on the upstream right side of the dam had historically been observed and was observed during the October 2018 field investigation. Green tracer dye was placed in the small whirlpool and was observed at three separate locations along the downstream side of the dam. Tracer dye was seen at Location 1 (adjacent to the water/dam interface) after 5 minutes, at Location 2 (adjacent to the water/dam interface) after 8 minutes, and at Location 3 (south of the bedrock outcrop) after 9 minutes. Green tracer dye was also placed in Borehole BH4020 during drilling and was observed at Location 3 (south of the bedrock outcrop) when drilling was occurring at approximately 5.5 to 5.8 mbgs.

6.5 Geophysics

In October 2017, a geophysical investigation was initiated by WSA to map the bedrock surface and to determine the integrity of the bedrock.

Clifton deployed Ground Penetrating Radar (GPR), Seismic Refraction, and Refraction Microtremor (ReMi) surveys to profile bedrock depth. Data sets were georeferenced using differentially-corrected points returned from a Trimble GeoXH GPS with an expected sub-metre accuracy.

Interpreted depth to Canadian shield bedrock was approximately 1 to 3 mbgs. The interpreted depth to bedrock from the data sets generally correlated with information from the nearby sheet pile wall installation. Interpretation of the geophysical data also identified two areas of suspected bedrock fracturing.

The interpreted depth to bedrock from the GPR data set trended consistently when superimposed on the P wave velocity (Vp) and S wave velocity (Vs) cross-sections and generally followed the depth at which velocity from the overburden material (approximately 1000 m/s Vp and 500-800 m/s Vs) increased to the expected velocity range for near-surface bedrock in the area (greater than 2000 m/s Vp and greater than 1000 m/s Vs).

The Vp cross-section did not model variations in the Vp velocity structure beneath the interpreted bedrock surface. This is likely due to the “blind layer” limitation of the seismic refraction method. The contrast between the overlying material and bedrock surface likely represented a strong enough acoustic pulse refractor that imaging Vp contrasts within the underlying bedrock was not achievable.

The Vs cross-section modelled two areas beneath the interpreted bedrock surface with lower than expected shear wave velocities. These areas are roughly coincident with zones where the GPR data returned secondary signal reflections and diffractions and hence were interpreted to be related to bedrock weathering or fracturing. The primary GPR anomaly and low shear wave velocity zone occurs toward the southern portion of the metal sheet wall, another anomalous zone was identified approximately 10 m further south.

The geophysical data was combined with borehole information to show the relationship between the geophysics and the measured bedrock surface (Figure 7.1). The top of bedrock was within 1 metre of the interpreted bedrock surface. The highly fractured zones observed in the field also aligned with the geophysical data.

7 SEEPAGE REMEDIATION DESIGN

7.1 Groundwater Regime

Seepage was encountered in all boreholes during drilling. Where measured, the depths to seepage ranged from to 1.58 to 4.44 mbgs. Only Borehole BH5010 had sloughing of gravel materials and had to be re-drilled for instrumentation installation.

The preliminary design was based on groundwater levels collected at the existing boreholes in the dam, observed reservoir water levels and downstream water elevations. Preliminary seepage and stability analysis assumed an upstream water elevation equal to the top of the dam core and a downstream water level 1 metre lower than normal water levels. This was considered the greatest possible gradient, hence, most critical regime to use while simulating remediation measures for effectiveness in managing groundwater flow through the dam.

The water levels observed in the boreholes during completion of drilling were between the reservoir elevation and the fishway water elevation. For the boreholes completed downstream of the existing sheet pile wall, the water levels were lower than the upstream lake level, showing some head loss across the sheet pile wall. For the two boreholes at the southern end of the dam, where no sheet piling was installed, the water levels observed were much closer to the reservoir elevation.

The in-situ packer testing revealed two areas of flow beneath the dam, 4 L/min and 60 L/min, in addition to flow through the zone of gravel/highly fractured bedrock at the base of the dam. The gravel/highly fractured zone was not able to be sealed during packer testing to determine a flow rate. This would indicate the gravel/fracture zone was more permeable than the deeper bedrock zones where we were able to complete the in-situ packer testing.

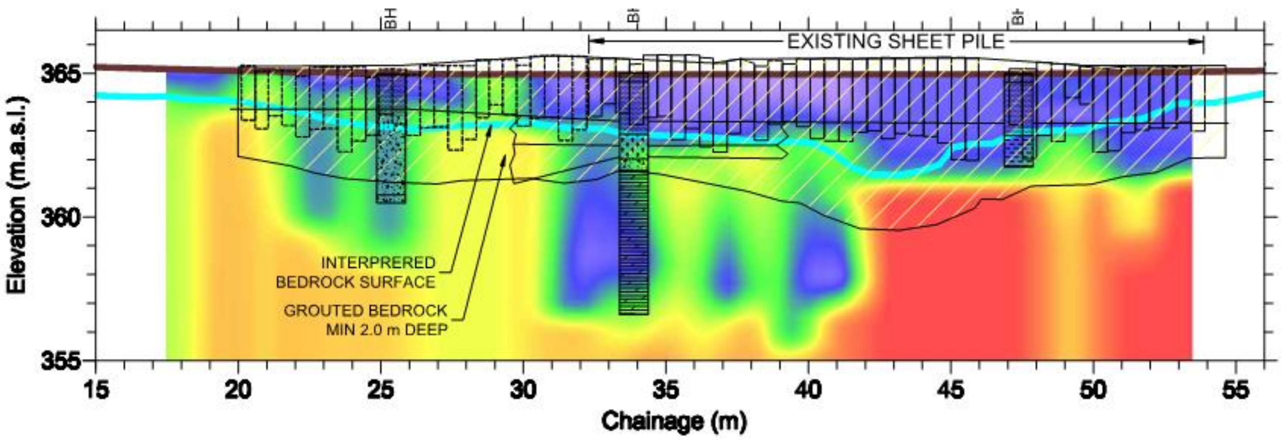


Figure 7.1. Geophysical comparison of dam stratigraphy with conventional drill program.

7.2 Stability and Seepage Analysis

Stability analyses of the embankment were performed to identify possible failure mechanisms, estimate the current factor of safety (FoS) and evaluate potential remedial options. The analyses were conducted using the software SLOPE/W. The Morgenstern-Price method with a half-sine side force function was used. The FoS objective for the dam embankment was based on typical objective of 1.5 under long term steady state conditions.

Seepage analyses were conducted with the objective of understanding potential seepage pathways and potential seepage induced failure mechanisms.

Existing conditions were modelled to estimate existing FoS and seepage conditions. The FoS was estimated to be 1.53 on the basis of assumed soil properties and piezometric conditions, with the critical failure only intersecting the downstream rock shell. The minimum FoS for a failure surface that intersected the dam core was estimated to be 2.39.

Material strength and seepage properties used in the analyses are presented in Tables 7.1 and 7.2 respectively.

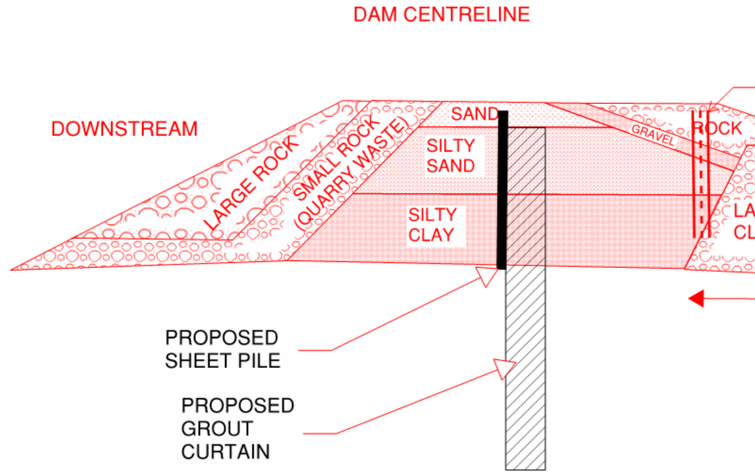
Table 7.1. Shear Strength Parameters

Material	Unit Weight (kN/m^3)	Phi (°)	Cohesion (kPa)
Dam Core	18.0	30.0	0.0
Gravel/Highly Fractured Bedrock	18.5	35.0	0.0
Rock Armour	17.5	40.0	0.0

Seepage analyses of the existing structure were based on observed seepage discharge locations, dam geometry (including existing sheet pile cutoff), piezometer tip elevations installed in the core (which were observed to be under suction) and recorded lake levels. The critical section intersects the 2017 sheet pile cutoff as shown on Figure 7.2; which also illustrates the probable location of the phreatic surface if no further work was completed.

Table 7.2. Seepage Modeling Parameters

Material	Ky/Kx Ratio	Vol. WC Function	Hydraulic Conductivity m/s
Dam Core	1	Impervious Core	$1x10^{-5}$ to $1x10^{-6}$
Gravel/Highly Fractured Bedrock	1	Gravel	0.0001
Fractured Bedrock	1	Rock	0.01
Sheet Pile Wall	1	Impervious Core	Impervious



Further modelling was undertaken to assess potential seepage mechanisms through the structure. This assessment was based on performance matching the seepage model with piezometer tip elevations in the core which provided an upper boundary of the potential phreatic surface. Two potential flow pathways were explored:

- A pathway through the core.
- A pathway through fractured foundation bedrock.

Given that the observed seepage flow was substantial but carried no fines would indicate the principal seepage pathway is not likely through the core but rather through the fractured bedrock. Accordingly, consideration of seepage control options focused on that pathway, acknowledging that gradients through the core must also be controlled.

7.3 Recommended Design Option

Figure 7.3 provides the recommended design option. The subsurface information from the field investigation confirmed the principal seepage pathway is at the interface of the dam structure with the fractured bedrock zone. The most effective method to control the seepage through this area is a sheet pile wall driven into the fractured bedrock with injection of grout upstream of the sheet pile wall that extends beyond the fractured bedrock zone to the more competent bedrock below.

The sheet pile wall will provide seepage control within the dam embankment while the injection of grout will seal from the lower portion of the sheet pile wall through the fractured bedrock zone to more competent bedrock below.

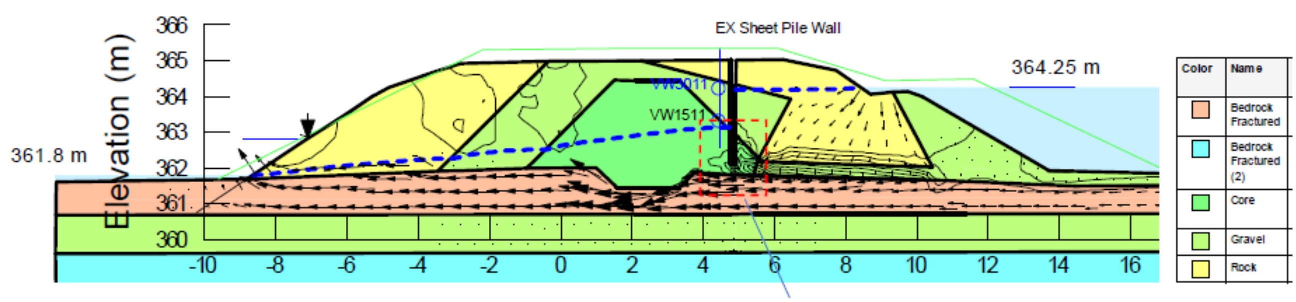


Figure 7.2 Phreatic surface through existing dam section prior to remediation

A multi-stage grouting program will be employed with a rapid setting and more viscous grout mixture first injected to manage the higher flows through the fractured zone followed by a less viscous but easier to manage grout mix to complete the seepage cutoff structure. The depth of grouting will vary between two to five metres into the upper bedrock, depending on the varying depth of the bedrock contact beneath the dam. The effectiveness of the grout curtain to cutoff seepage will be monitored during and post-construction with the additional piezometers now installed both upstream and downstream of the location of the seepage cutoff structure.

8 SUMMARY AND CONCLUSIONS

The dam stratigraphy as determined from the drilling investigation consists of the following units in descending order:

- Surficial sandy gravely fill, underlain by geotextile;
- Silty Sand core material;
- Silty Clay to Clay and Silt core material;
- Angular to sub angular gravel/highly fractured and weathered bedrock zone; and,
- Pelitic/granitic gneiss with potassium feldspar porphyroblasts, fractures and two distinct fracture zones.

The composition of the dam core transitioned from a silty sand to a silty clay for a total dam core that extend to between 3 to 4.5 mbgs. Even though there was a change of gradation within the dam core from silty sand to silty clay, the percentage of fine-grained material in the tested samples was consistent, averaging 73% clay and silt below a depth of 1.5 metres.

The presence of the 0.3 m to 0.6 m thick angular to subangular gravel/highly fractured bedrock zone beneath the core was not part of the original design and had the highest flow rates and poorest recovery in the boreholes. This layer is the most likely location for seepage beneath the dam structure, but given the nature of the material, it is not a concern for dam stability. However, there is the potential for piping at the interface of the dam core with the gravel/fractured bedrock zone.

The bedrock observed beneath the dam was reasonably competent but did have noted fracture zones. Visual observation of the rock cores confirms the noted observations for likely areas of high seepage flows that

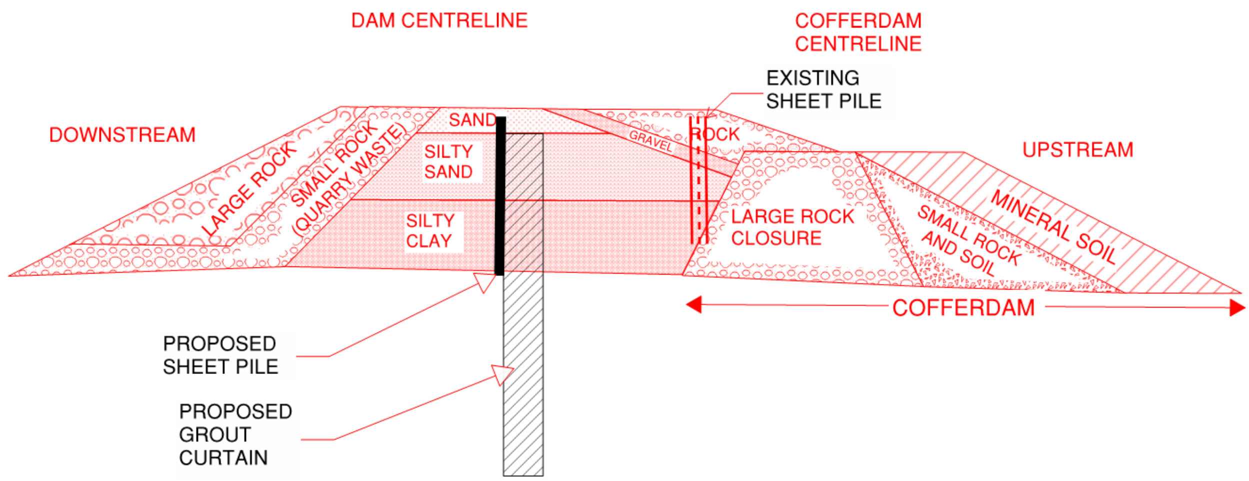


Figure 7.3. Recommended dam design section for seepage remediation

could be targets for the proposed grouting measures. The preliminary design suggested a target depth of two metres beyond the bedrock contact for grouting measures. The collected information suggests that deeper grouting may be required and will be confirmed during detailed design.

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