Fall Term Design Report

Project E4 - Tent Mountain Pumped Hydro Storage
Civil Engineering Design - ENCI 570

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Executive Summary

This report presents a preliminary design report for the Tent Mountain Pumped Hydro Energy Storage (TM-PHES) project, located near the Crowsnest Pass of Alberta. During the Fall semester, the primary objectives were to optimize a design to enhance the Tent Mountain Upper Reservoir and size the Lower Reservoir to enable a 16-hour continuous operation for generating 320 MW of clean power for Alberta. For the Upper Reservoir, the main goal was to estimate the water level required to satisfy the 16-hour operating window, where alternative locations for the intake structure and the subsequent changes in water level of the reservoir were evaluated. Additionally, seepage losses were modeled and quantified using GeoStudio and from there, a suitable grouting strategy was researched and selected to help combat these losses. The use of a vertical cutoff wall was identified as the best solution for preventing seepage through the waste material; for the non-waste material, polyurethane curtain grouting was selected. Furthermore, the elevation (El.) 1803.3 m Normal Maximum Operating Levels (NMOL) option was found to be the most cost-effective due to reduced exterior grouting and an associated shorter vertical cutoff wall.

As for the Lower Reservoir, the main goal was to size the reservoir to be able to accommodate the incoming volume from the Upper Reservoir during this 16-hour operation. It can be noted that the Lower Reservoir's location and sizing are constrained by its proximity to the Alberta-British Columbia provincial border and its encroachment onto Crown Land. This drove the decision to fix the South Dam location in place and explore alternative locations for the North Dam. From there, the subsequent changes in water level and required dam dimensions were evaluated for each option. Three NMOL options for the Lower Reservoir, ranging from El. 1500 m to El. 1509 m, were evaluated; the El. 1509 m NMOL option was selected primarily due to its minimal environmental and social impacts. Finally, three alternative rockfill embankment dam types for the North and South Dams, two with different cores (sheet pile, and asphalt) and a concrete faced dam, were researched and assessed for suitability. The dam type with the asphalt core was selected based on its constructability, reduced environmental impacts, and favourable costs.

Overall, for both major scopes, the costs and environmental impacts associated with each of the proposed options were analyzed to help guide our final selections. Furthermore, weighted decision matrices were utilized to summarize the comparisons between alternatives and support the final selection. The final design presented herein approaches sustainability holistically across environmental, social, and economic pillars. The next steps for the Winter semester include conducting a water balance, hydrological modeling, and stochastic and time series analyses to identify climate change impacts on the system.

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1 Introduction

1.1 Background

The Tent Mountain Renewable Energy Complex (TM-REX), formerly a coal mine in southwest Alberta, is undergoing evaluation for transformation into a pumped hydro energy storage (PHES) facility, jointly pursued by TransAlta and Evolve Power [1, 2]. The goal of this project is to transform the mine pit into a renewable energy storage facility, where water is discharged from the Upper Reservoir for power generation when demand for electricity is high, and water is pumped back up during periods of low electricity demand. This report focuses on designing a PHES system capable of sustaining continuous 16-hour operations, generating 320 MW of clean power for Alberta, while minimizing costs and environmental impacts. This duration aligns with potential peak electricity demand, ensuring efficient storage of renewable energy and promoting substantial long-term environmental and economic benefits.

1.2 Scope of Work

The scope of work undertaken for this project during the Fall semester entails optimizing the Upper Reservoir design and sizing the Lower Reservoir to enable this 16-hour continuous operation. First, the volume of water corresponding to a 16-hour operation was determined, where to achieve this amount of water storage, a combination of raising the Normal Maximum Operating Level (NMOL) and lowering the Dead Storage Level (DSL) associated with the penstock intake elevation was investigated. Additionally, due to the prevalence of seepage losses from the Upper Reservoir, these losses were modeled and quantified using GeoStudio, and appropriate grouting strategies were researched and suggested for implementation. As for the Lower Reservoir, it was sized to be able to accommodate the incoming volume from the Upper Reservoir during each cycle. Alternative NMOLs were investigated by fixing the South Dam in place and exploring alternative North Dam locations. In addition, alternative dam types for both the North and South dams were researched and assessed, and their corresponding dimensions were calculated. This report summarizes the alternative design options assessed for the Upper and Lower Reservoir of the Tent Mountain site with costs, sustainability, professional impacts, and engineering risks considered. The scope of work for the Fall Term is presented in *Table 1*.

Table 1 – Fall Term Deliverable Summary

Deliverable	Sub-deliverables	Constraints	Exclusions
Determine required water storage volume.	-	• Must suffice for 16-hour continuous generation.	• Short- and long-term water balance (reserved for Winter Term).
Design Upper Reservoir.	• Select Normal Maximum Operating Level (NMOL) and Dead Storage Level (DSL). • Investigate seepage from the reservoir. • Select seepage reduction strategies. • Perform high-level triple bottom line (TBL) cost analysis.	• NMOL and DSL must be compatible with required storage volume. • Avoid constructing dam-like structure if possible. • Avoid tunnelling for intake if possible. • Minimize environmental and economic costs and maximize constructability of seepage reduction strategy.	• Detailed design of cutoff wall and grouting. • Detailed design of intake structures and penstocks.
Design Lower Reservoir.	• Place South Dam. • Select North Dam placement and NMOL. • Select dam structure type. • Perform high-level TBL cost analysis.	• South Dam structure and construction footprint must be entirely within Alberta. • North Dam placement and NMOL must be compatible with required storage volume. • Minimize environmental and economic costs and maximize constructability of dam structures.	• Powerhouse location and design (assumed constant from prefeasibility phase). • Penstock intake elevation (assumed constant from prefeasibility phase). • Structural design of dams.

2 Required Storage Volume

Hatch's prefeasibility report specified a design discharge rate of 123.4 cms for the generation system it detailed [1]. To achieve 16 hours of continuous generation at this rate, the required volume was calculated at 7.1 million cubic meters:

$$Required\ Volume = 123.4\ \text{cms} * 16\ \text{hr} * \frac{3600\ \text{s}}{\text{hr}} = 7{,}107{,}000\ \text{m}^3$$

This volume represents the minimum live storage, or water storage above the penstock intake elevation. The penstock intake elevation is the DSL for both Upper and Lower Reservoirs.

3 Upper Reservoir Design Alternatives

The Upper Reservoir is an old coal mine pit that filled with water after the decommissioning of the mine in 1983 and has maintained a water level of around elevation (El.) 1780 m for decades [1]. This suggests any water entering the reservoir above this elevation escapes via seepage. The topography of the Upper Reservoir is such that the required water volume for 16 hours of electricity generation can be achieved by raising the NMOL or lowering the DSL, rather than by expanding the inundated area as with the Lower Reservoir. Security of water supply is dependent on the reduction of seepage from the reservoir walls. *Figure 1* illustrates the upper reservoir topography, the minimum and maximum water levels investigated, and the regions requiring seepage reduction [2]. Alternative designs for operating level and seepage reduction strategies are presented in Sections 3.1 and 3.2 respectively.

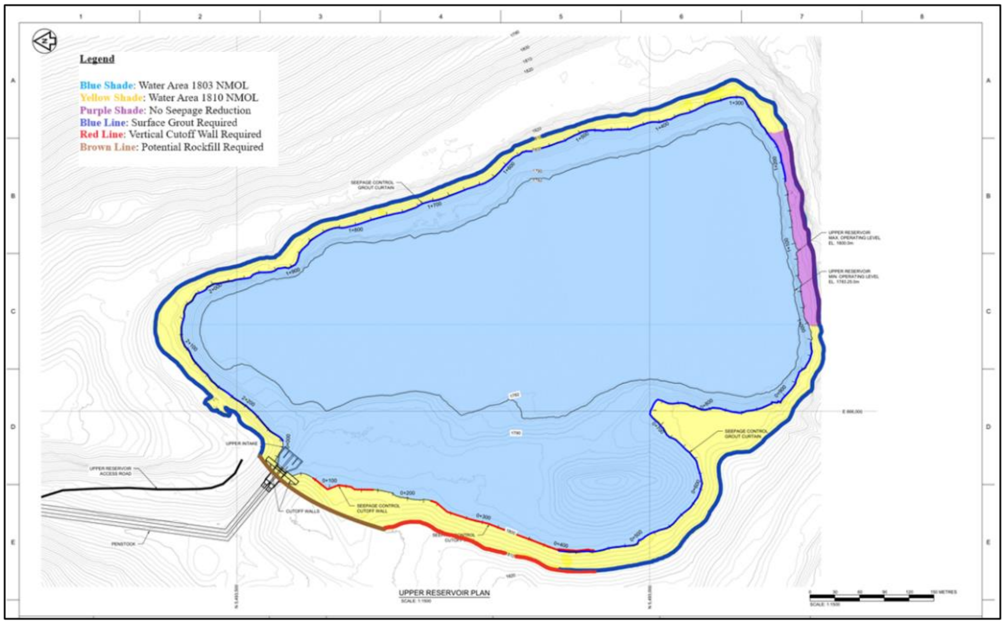

Figure 1 - Upper Reservoir Topography and Seepage Requirements

3.1 NMOL and DSL

Three NMOL-DSL pairs were investigated based on the constraints of the upper reservoir's topography (*Figure 1*). All options meet the live storage criterion of 7,107,000 m^3. The selected options along with their dead storage are summarized in *Table 2*.

Table 2 – Upper Reservoir NMOL-DSL Pairs

NMOL (masl)	Required DSL (masl)	Dead Storage Volume (m^3)
1803.3	1760	2,052,546
1807.0	1770	3,108,802
1810.0	1777	4,014,686

These options are associated with variations in the height of the grout wall, the depth of the penstock, and adjustments in the water surface area. Selection of the alternative water levels for the upper reservoir were based on constraints from the site's topography. Any NMOL above El. 1810 m would require a significant containment structure constructed along the west wall of the reservoir, which was deemed infeasible due to significant economic and environmental costs. The associated DSL is El. 1777 m. A bathymetric survey of the reservoir revealed a significant bench in the reservoir at El. 1760 m, with the deepest section of the reservoir at the south end of the pit [1]. Thus, an intake between El. 1777 m and El. 1760 m was deemed feasible; below El. 1760 m the tunneling requirements would increase costs exponentially. *Figure 2* presents the upper reservoir's stage-storage and stage-area curves along with each NMOL-DSL pair.

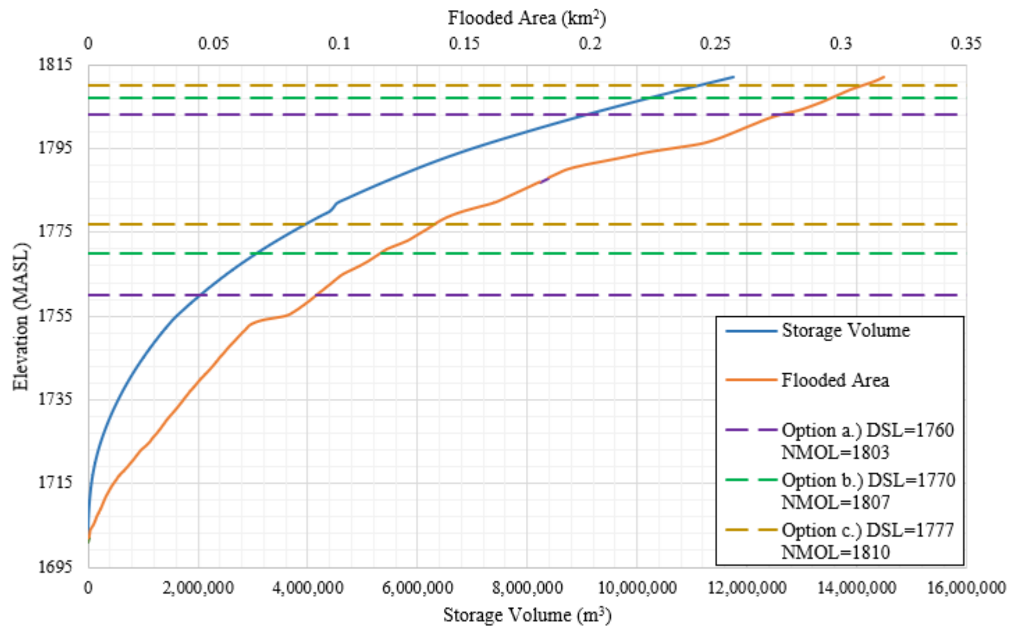

Figure 2 - Upper Reservoir Elevation-Storage and Elevation-Area Curves

Hatch's prefeasibility report established a positive economic benefit-cost ratio for raising the NMOL to El. 1810 m [1]. Lowering the penstock intake elevation was not explicitly considered during the prefeasibility stage, but it has been considered in detail herein for the following reasons:

- The required excavation volume to lower the intake to a minimum of El. 1760 m is significantly less than the material required to build a containment structure up to a maximum of El. 1810 m.
- Construction sequencing could be done such that the lower reservoir and the penstocks are completed before pumping out of the Upper Reservoir during construction.
- In the absence of pumping, the intakes could potentially be built using cofferdams or underwater construction techniques. While each of these have drawbacks, they are both at least comparable to building to increase the containment elevation economically.
- Lowering the final water level elevation reduces the seepage reduction required, which has environmental and economic benefits due to less grout being required.
- The lower the NMOL, the less risk of the reservoir overtopping due to the site's topography, and the lower the risk of slope failure due to a reduction in water pressure against the reservoir walls.

3.1.1. NMOL Design Options

Each of the three Upper Reservoir NMOL options are summarized below. The containment structure minimum elevation will be built to 3.5 m above the NMOL in each case. The decision matrix and rationale behind option selection are presented in Section 5.

El. 1803.3 m NMOL

This alternative has the lowest NMOL elevation, the shortest cutoff wall height, the minimum flux through the reservoir walls, and the smallest flooded area at its NMOL. Moreover, it minimizes dead storage volume, making use of the maximum proportion of retained water. Both low flux and flooded area enhance operational resilience against drought by reducing seepage and evaporation losses, respectively. The low cutoff wall height not only leads to the lowest construction costs by reducing the grouting required, but it also mitigates overtopping or slope failure risks due to additional topography surrounding the final water level and less force along the reservoir walls. The main drawback for this alternative is it would require tunneling and underwater construction for the intake.

El. 1807 m NMOL

With the median value for both NMOL and DSL, this option strikes a balance between required cutoff wall height and required excavation to achieve the correct intake elevation. It also has the median value for flux, flooded area, dead storage, and risk. The main benefit of this alternative is that it would not require tunneling for the intake or a rockfill structure to raise the western wall topography, making it the most constructable.

El. 1810 m NMOL

The primary benefit of this alternative is that it requires the least amount of excavation for the penstock intake, as the design elevation is only approximately 3 meters below the pre-construction water level. However, it ranks the lowest among the three options in terms of constructability, risk, flooded area, flux, dead storage, and overall cost. Consequently, it is associated with the highest potential for water losses (indicating lower security of supply) and the greatest cost.

3.1.2. Cost Analysis

A Class IV cost analysis was undertaken to compare the reservoir area preparation costs for each of the NMOL options. This analysis investigates high-level unit costs and equipment factored models to provide a comparison tool appropriate in the feasibility stage of a project [3]. The utility of this cost analysis is comparison, and as such only costs that were different between the options were considered and costs expected to be standard across options such as contingency and mobilization were ignored. The geometric properties of each alternative were derived through GIS analysis and the prefeasibility drawings [1]. The prefeasibility report was also used for tunneling costs as they are extremely site specific. The majority construction unit costs were obtained from the Government of Alberta 2023 Average Unit Cost Spreadsheet

[4]. Grout Specific Unit Costs were found in USD in 1999 [5], historical conversion rates were used to convert the price to CAD [6], and inflation was calculated using the Bank of Canada Inflation Calculator [7]. A grout curtain was used in this cost analysis for minor grouting, and a grout wall was used for the vertical cutoff; Section 3.2 explores the reasoning behind these selections. Results from the high-level cost analysis are summarized in *Table 3*, and the calculations are shown for each alternative in Appendix A Upper Reservoir Additional Figures.

Table 3 - Upper Reservoir Alternative NMOL Cost Analysis

Options (El. NMOL)	Exterior Grouting	Vertical Cutoff Wall	Rock Fill	Excavation	Tunneling	Total
1803.3	$34,956,924	$6,794,097	-	$1,995,933	$1,988,039	$45,734,993
1807.0	$39,224,252	$9,058,796	-	$1,995,933	-	$50,278,981
1810.0	$43,475,923	$10,611,099	$1,702,167	-	-	$55,789,188

Exterior grouting is the most expensive aspect of reservoir preparation due to high material unit costs (around $400/m^2) and a massive area requiring grout (between 86,000 m^2 to 107,000 m^2). The vertical cutoff wall is the second most expensive aspect of the project for similar reasons. While these seepage reduction aspects of the project are costly, they are imperative to the sustainability of the project in minimizing losses. Despite requiring both excavation and tunneling, the alternative with NMOL at El. 1803.3 m is the cheapest due to its reduced exterior grouting area and reduced vertical cutoff wall length.

3.2 Seepage Reduction

3.2.1. Investigation of Losses Due to Seepage Using GeoStudio

GeoStudio 2023's SEEP/W module was utilized to analyze seepage on developed Upper Reservoir models. The E-W and N-S profiles of the upper reservoir were developed using the Leapfrog geological model provided by TransAlta (see *Figure 3*) Hatch's Prefeasibility Study supplied permeability coefficients for Tent Mountain's rock formations, used in the GeoStudio model (see *Figure 4*, *Figure 5*, and *Figure 6*). In this simplified model, each material was considered isotropic. Additionally, a separate model was prepared to illustrate the seepage pathways observed through the waste dump pile around the intake structure, which has an approximate permeability of 10^{-3} m/s which is 4-5 orders of magnitude larger than the hard rock permeabilities. Therefore, the model shows all water seeping through the dump material as it's the path of least resistance. However, given that a vertical cutoff wall will be implemented to prevent seepage through the waste material, the models without the waste dump included (*Figure 4* and *Figure 6*) were utilized for all flux calculations to identify losses through the rest of the reservoir.

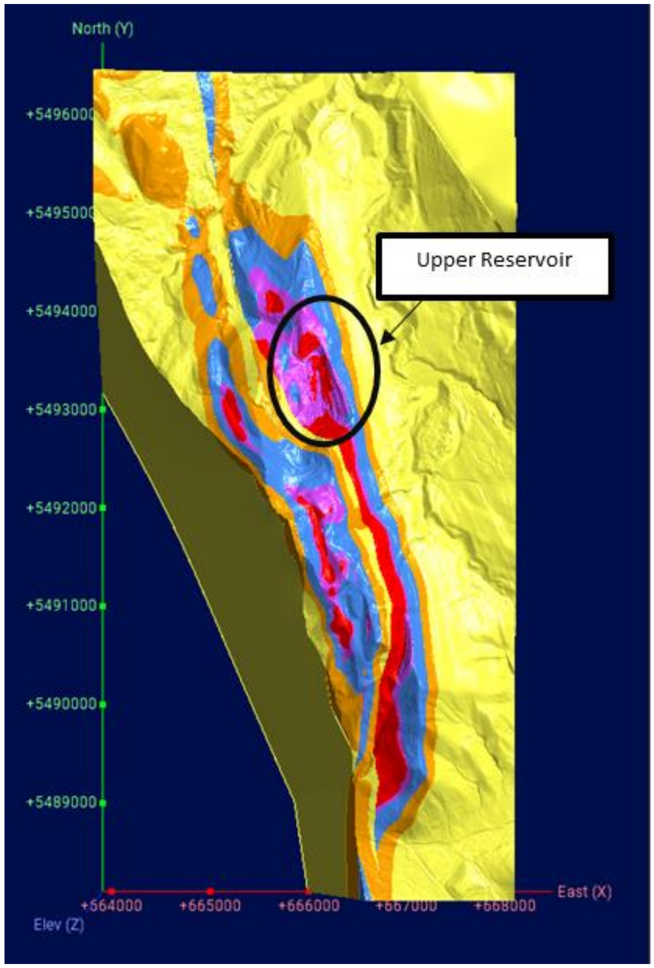

Figure 3 – Overall Leapfrog Model of the Tent Mountain Site (Plan View)

The model reservoir's total water head at El. 1810 m was used as a boundary condition. Only this highest level of the three alternatives was modeled as it maximized reservoir wall surface area and seepage losses. Another boundary condition was used to represent the potential seepage faces with a zero-flux boundary. The "Pit 4 Cross Section Lower Bound" in the E-W model considers a total water head of El. 1627 m, aligning with the natural water body elevation to the east of Pit 4 along the chosen cross section. For the N-S model, the Upper Reservoir level and zero flux boundary conditions are also utilized. The water level higher up the mountain (Pit 2) is included as a source of inflow to the upper reservoir; its value is based on monitoring well data. The elevation of the natural waterbody within the area of the proposed Lower Reservoir is also included as a total water head condition, valued at 1479 m, which approximately represents its minimum water level. Ultimately, the GeoStudio modeling provided an understanding of seepage pathways through the upper reservoir and to determine graphical water flux profiles that could be used to approximate total losses, (see Section 3.2.2).

Subsequent work is recommended to include changing the saturation regime to 'partially saturated' to characterize the groundwater table with a phreatic line that extends throughout the model (distinguishing the dry and saturated regions). Furthermore, 3D modelling software could be used for greater accuracy.

Color	Name	Hydraulic Material Model	Sat Kx (m/sec)
	JF	Saturated Only	4.7e-08
	JKKA	Saturated Only	1.7e-07
	JKKH	Saturated Only	7e-07
	JKKMm	Saturated Only	2.4e-07
	JKKMU	Saturated Only	1.8e-07

Color	Name	Category	Kind	Parameters
	Pit 4 Cross Section Lower Bound	Hydraulic	Water Total Head	1,627 m
	Upper Reservoir (Pit 4) Water Level	Hydraulic	Water Total Head	1,810 m
	Zero Flux	Hydraulic	Water Flux	0 m/sec

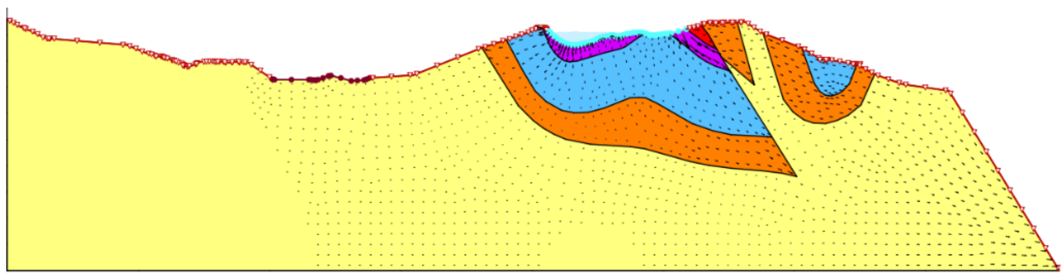

Figure 4 - GeoStudio SEEP/W Model of E-W Cross-Section

Color	Name	Hydraulic Material Model	Sat Kx (m/sec)
	JF	Saturated Only	4.7e-08
	JKKA	Saturated Only	1.7e-07
	JKKH	Saturated Only	7e-07
	JKKMm	Saturated Only	2.4e-07
	JKKMU	Saturated Only	1.8e-07
	Waste Dump Material	Saturated Only	0.001

Color	Name	Category	Kind	Parameters
	Pit 4 Cross Section Lower Bound	Hydraulic	Water Total Head	1,627 m
	Upper Reservoir (Pit 4) Water Level	Hydraulic	Water Total Head	1,810 m
	Zero Flux	Hydraulic	Water Flux	0 m/sec

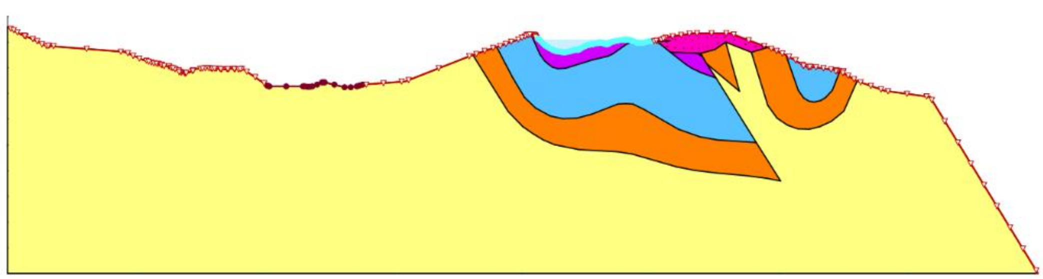

Figure 5 – GeoStudio SEEP/W Model of E-W Cross-Section with Waste Dump Pile

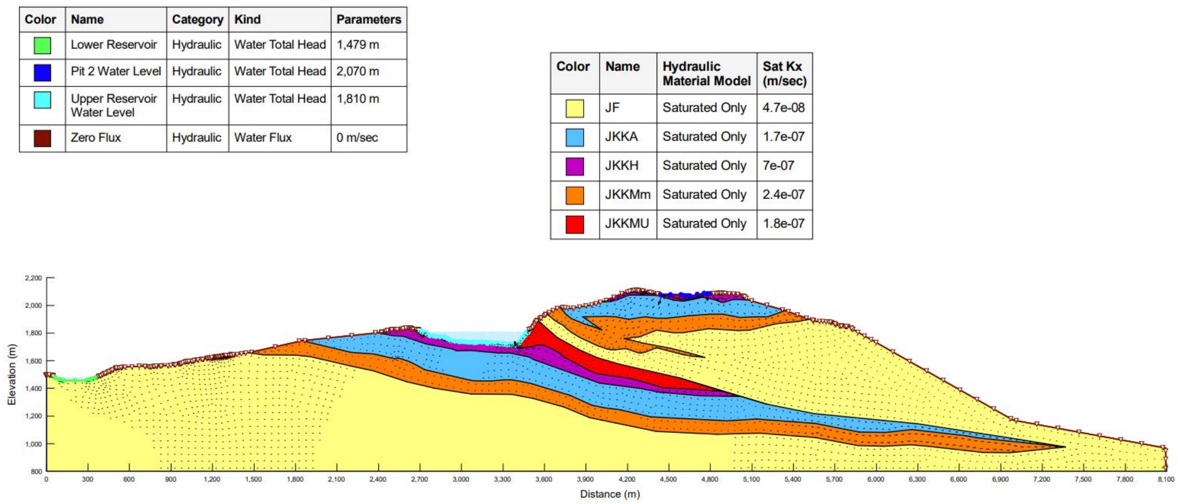

Figure 6 – GeoStudio SEEP/W Model of N-S Cross-Section

3.2.2. Flux Methodology

Flux calculations were utilized to quantify the decision matrix for three alternative designs of the upper reservoir. The analysis assumed exploration of flux above the current water level to the NMOL for each alternative design. Using N-S and E-W cross-section models, flux values ($m^3/s/m^2$) were computed, adjusting boundary conditions to reflect each design's NMOL. These values were averaged across the submerged reservoir surface, resulting in six flux values (see Appendix A Upper Reservoir Additional Figures). Flux discharge values (m^3/s) were then calculated using wall surface areas, which were determined for each wall and each option using Global Mapper software. Total discharge flux for each alternative was determined by summing these values, accounting for flux direction through the south wall, summarized in *Table 4*. The results seen below show that the flux values increase with the increasing NMOL of the alternative options.

Table 4 - Total flux for alternative design NMOL for the Upper Reservoir

	Option 1 (NMOL 1810 m)	Option 2 (NMOL 1807 m)	Option 3 (NMOL 1803.3 m)
Total Flux (m^3/s)	6.10 E-03	5.45 E-03	4.85 E-03

3.2.3. Investigation of Permeability Reduction Strategies

The upper reservoir will require localized grouting methods to reduce seepage through the rock mass, as well as additional works in certain areas to increase the reservoir height to satisfy the operational water level and ensure proper slope stability and watertightness.

Vertical Cutoff Methods in Waste Material

Cutoff walls work by providing a vertical low permeability surface which connects to a low permeability soil layer, reducing overall permeability and thus water seepage. Borehole records suggest the waste material at the site exists at an elevation of El. 1780 m [2]. This reinforces the finding that the waste material is responsible for the reservoirs constant water level, and it means any cutoff wall solution must extend down to an elevation of 1780m at a minimum. To achieve this, several options for vertical cutoff walls exist, with the primary categories being slurry walls, grouted barrier walls, sheet pile walls, and geomembrane walls.

Slurry walls are the most common vertical cutoff walls, and they are constructed by excavating a large trench to a low permeability layer, then filling the trench with a low permeability slurry and backfill. This method creates a surface with a permeability of 10^{-7} or less and can be constructed to a depth of 60 m. However, at depths greater than 15 m, specialized equipment is required for the excavation which greatly increases the costs associated with this method [8]. Since the depth of waste extends 40 m below the surface of the site, specialized installation would be required in this project. Construction would consist of a 1-2 m wide trench excavated via special slurry trenching clamshells with a slurry of bentonite and water pumped into the trench to prevent collapse. The clay lines the trench wall to keep the slurry solution in place, which pushes against the trench walls to keep the excavation open. This would be extremely expensive due to the depth of excavation and specialized equipment required, but using the aforementioned methodology it would be possible for this project.

Sheet pile and geomembrane vertical cutoff walls are both methods where impermeable panels are inserted into the permeable layer to reduce the layer's permeability. Sheet pile walls are very easy to transport to site and quick to install. Geomembrane walls are the most effective seepage reduction solutions, with the ability to reach permeability values as low as 10^{-12}. However, the installation of both methods requires either excavation or pile driving. The Upper Reservoir has waste material extending to El. 1780 m, while the surface in this region is around El. 1820 m, thus a depth of 40 m would need to be excavated for this installation. Alberta Health and Safety codes also require any excavation in loose materials to have 45 degrees sloping at minimum [9]. Thus, the width of the trench would legally be required to be 80 m, which is impossible due to the constrained nature of the site. Therefore, a sheet pile or geomembrane cutoff wall is infeasible for this project.

Grout barrier vertical cutoff walls are constructed by drilling holes into high permeability materials and inserting low permeability grout in the holes. The holes are drilled close together so that each grout column overlaps, and a solid grout wall is formed. This method's application is better suited for the project area, as it does not require any excavation to achieve the desired depth of the cut off wall and it can reach depths of 200 ft (610 m) [5]. The desired grout for the columns is based on the permeability of waste rock, which was on the order magnitude of 10^{-2}-10^{-4}, and that would allow for commonly used chemical grouts to be implemented [10]. Polyurethane (PU) chemical grout is a viable option in this case, as it is characterized by high expansion, durability, strong permeability resistance, and eco-friendliness [11]. The application of PU vertical cutoff walls for water containment have been explored and found that the PU grout permeability is on the magnitude of 10^{-12} m/s [12].

Ultimately, PU grout curtain vertical cutoff walls were the chosen solution to remediate the waste material. This is the most constructable option to implement in the waste material as the holes required to create the grout cutoff walls are small and can be drilled into the waste more feasibly than 40 m of excavation. Grout curtains are also the vertical cutoff method which creates the least waste and are often used to reduce seepage of hazardous materials on mine sites, so it will have a positive environmental impact by limiting the seepage of the selenium rich mine water contained in the reservoir with minimal waste produced. The grout curtain method is also required for the Lower Reservoir dams, so implementing the same solution for the Upper Reservoir will reduce economic and environmental costs of mobilization and site preparation.

Rock Grouting Methods in Non-Waste Material

As for the outer areas of the reservoir, grouting methods to deal with cracks in the non-waste material were investigated. The grouting methods will be used in order to lessen the hydraulic conductivity, stop internal erosion, and lower uplift pore pressure. The grouting methods of consolidation grouting, and curtain grouting have been researched and are described in detail below.

Curtain grouting refers to the installation of barriers in the reservoir siding to reduce seepage and permeability. Furthermore, sealing fractures at great depths can be accomplished via curtain grouting. These barriers are installed depending on the gradient desired and the grout curtains are typically designed so that primarily holes are filled first before repeating the process for secondary and tertiary ones [13]. These holes are spaced between 4.5-7.5 m apart and high-pressure grouting is used. As a result of this process, there will be progressively smaller grout volumes at each stage of the curtain construction.

Consolidation grouting is typically used to improve the characteristics of the foundation rock and increase its strength. This grouting occurs over the selected area of the reservoir siding and involves the drilling of shallow holes in a grid pattern. Due to this grid pattern, the grout injected into the shallow holes results in increased bearing capacity and lower permeability [13]. Additionally, the depths of holes range typically from 3-15 m with spacing between the primary holes of 12-30 m. In consolidation grouting low pressure grouting is used instead.

In summary, both exterior grouting methods were analyzed to find the best solution to filling the cracks in the site's non-waste material. Ultimately, curtain grouting was chosen for similar reasons to the grouted barrier wall. This method would result in the need for the least excavation, reducing costs and overall environmental impact. In addition to this, curtain grouting was the most appropriate technique for the required depth the project entailed, compared to the other method better suited for shallow work.

4 Lower Reservoir Design Alternatives

The Lower Reservoir's unique proximity to the continental divide (and the Alberta-British Columbia border) constrains its South Dam location. To a lesser extent, Crown Land further north in the valley constrains the North Dam location. Its DSL is also constrained at El. 1480 m by the prefeasibility placement of the powerhouse and intake/outlet. Live storage volume requirements can therefore be achieved by raising the NMOL or moving the North Dam northward. This project's remoteness and the surrounding geology greatly influence the feasibility of different dam types, reducing options to variations of rockfill dams. The South Dam location, as well as alternative designs for the North Dam location, the NMOL, and the dam type for the South and North Dams are presented in Sections 4.1 and 4.2.

4.1 Reservoir Sizing

To meet the requirement of 16 hours of continuous power generation, the live storage volume provided by the Lower Reservoir must match the established 7.1 million m^3. The following subsections describe how this is accomplished.

4.1.1. South Dam Location and Dam Crest Elevation

The placement of the South Dam will be consistent across all design options, primarily due to the requirement to maintain an appropriate distance from British Columbia border during construction. The minimum toe offset is 20 m. The South Dam crest elevation will be built to 4.0 m above the NMOL (0.5 m above the crest elevation of the North Dam) to ensure a primary northward failure direction in case of overflow. Although outside the scope of this report, it is recommended that the creation of wetlands between the South Dam and the continental divide be considered to offset environmental impacts, contingent upon geotechnical conditions and drainage characteristics.

4.1.2. North Dam Location, Dam Crest Elevation, and NMOL

Considerations for the placement of the North Dam include the following:

- Maximizing catchment area and thereby runoff volume.
- Minimizing total inundated area and environmental impacts.
- Minimizing material haul distance from the existing access road.
- Minimizing the average ground slope in the vicinity of the dam site. This parameter is weighted low as the slope at all potential dam sites is near 1%, which adds minimal difficulty to construction.
- Minimizing required dam width by leveraging the natural contours of the valley.
- Minimizing inundated Crown Land and thereby purchase or lease costs.

Three options were considered, with the dam crest elevation 3.5 m above NMOL for each. *Table 5* summarizes the characteristics of each of the following:

1. El. 1500 m NMOL with the North Dam placed 1.9 km north from the South Dam.
2. El. 1505 m NMOL with the North Dam placed 1.5 km north from the South Dam.
3. El. 1509 m NMOL with the North Dam placed 1.2 km north from the South Dam (approximately the same location as in the prefeasibility report [1]).

Table 5 – Lower Reservoir Elevation Alternatives

Option	El. 1500m NMOL	El. 1505m NMOL	El. 1509m NMOL
Drainage Area (km^2)	6.235	5.746	5.557
NMOL Flooded Area (km^2)	0.4870	0.4274	0.4064
Haul Distance (km)	1.3	1.6	1.9
Average Ground Slope (%)	0.62	1.11	1.60
Required Dam Width (m)	257	265	236
Inundated Crown Land Area (km^2)	0.26	0.15	0.09

Figure 7 shows each option projected on the DEM of the site.

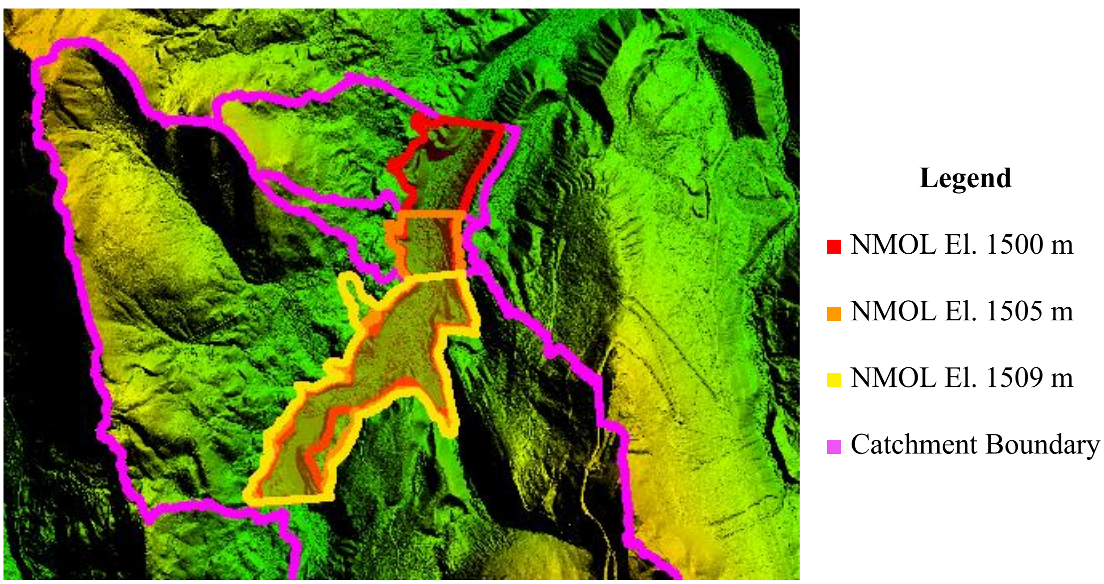

Figure 7 – North Dam Placement Options Plan View

El. 1500 m NMOL

This option encompasses the largest catchment area, the shortest haul distance from the main access road to the dam site, and the flattest ground at the dam site. These characteristics contribute to the largest potential runoff volume and high ease of constructability.

Ease of constructability is hindered by this option having the median dam width of the three options. It also inundates the most Crown Land and the most area overall, contributing to high construction costs and significant environmental externalities. *Figure 8* presents its elevation-storage and elevation-area curves.

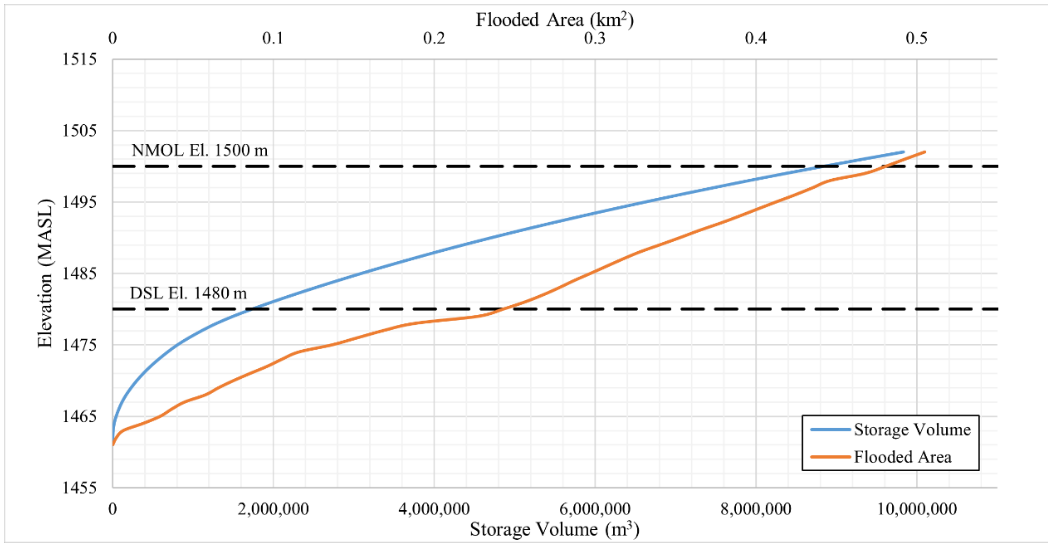

Figure 8 – Lower Reservoir Elevation-Storage and Elevation-Area Curves (El. 1500 m NMOL)

El. 1505 m NMOL

This middle option yields median values for catchment area, flooded Crown Land and total flooded area, haul distance, and average slope. Where it stands out is with the minimum required dam width, affecting low construction costs. *Figure 9* presents elevation-storage and elevation-area curves for this option.

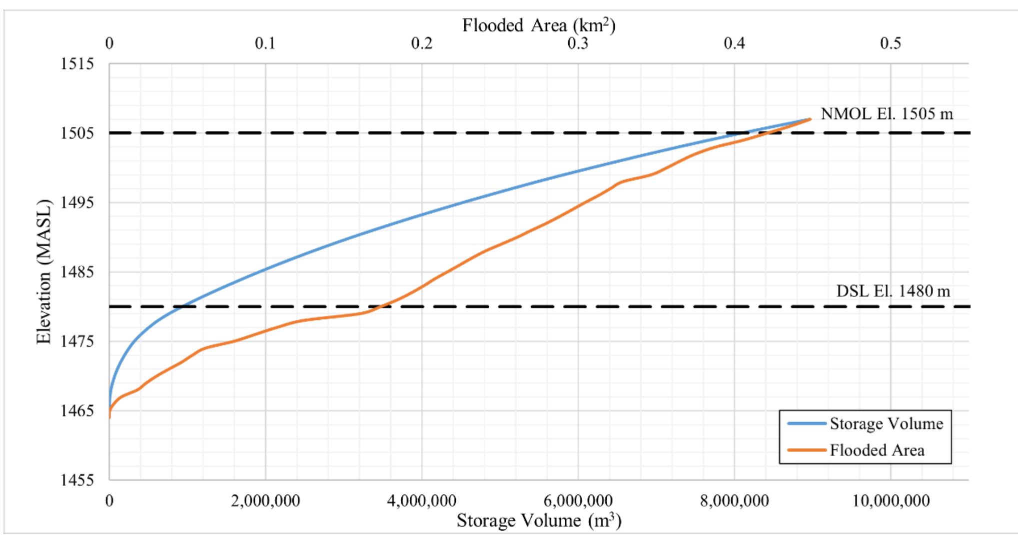

Figure 9 – Lower Reservoir Elevation-Storage and Elevation-Area Curves (El. 1505 m NMOL)

El. 1509 m NMOL

The final lower reservoir option involves the lowest catchment area, the longest haul distance, and the highest average slope. The low catchment area provides the lowest potential runoff volumes, but this effect is counteracted by this option's minimal flooded area, which minimizes evaporation losses. It also inundates the smallest amount of Crown Land, mitigating land acquisition costs. Due to the local topography, the dam width for this option is the widest, but it is only roughly 10% wider than the narrowest option. *Figure 10* presents this option's elevation-storage and elevation-area curves.

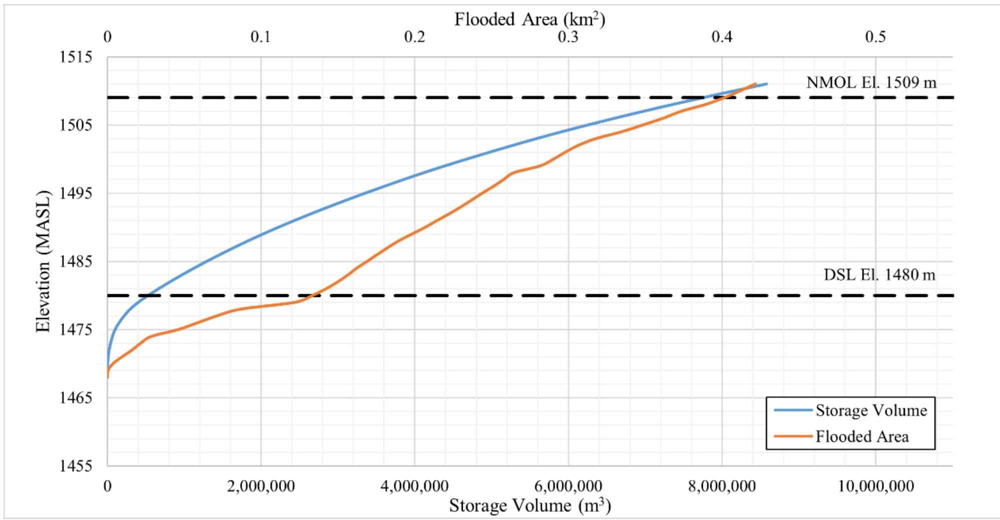

Figure 10 – Lower Reservoir Elevation-Storage and Elevation-Area Curves (El. 1509 m NMOL)

4.2 Dam Structure Type

Three dam structure options were explored based on the following considerations, the optimization of each of which is affected by several factors:

- Maximizing ease of material acquisition. Locally available materials were prioritized for all options, leaving out other potential options, such as clay core, altogether. Local geology is comprised largely of coarse granular overburden over sedimentary bedrock. This is convenient because all options require large amounts of these types of material.
- Maximizing sustainability of materials. The main detractor from sustainability of materials is associated with transportation, making long distance material hauling prohibitively unsustainable and necessitating the sourcing of local materials.
- Minimizing life cycle environmental impacts. Both core materials, (steel, and asphalt) as well as the concrete face, have significant life cycle emissions and other environmental externalities. Climate change resiliency and adaptability to changing weather patterns and more polarized extrema also reduce an option's life cycle environmental impacts by minimizing its likelihood of obsolescence.
- Minimizing construction and life cycle financial cost. Specific cost considerations were limited to material costs, but maintenance costs were considered at a high level.
- Maximizing constructability. Each option's compatibility with local geological and topographical conditions were explored.

4.2.1. Dam Dimensions

The ground profile and height requirements for the South Dam and each of the North Dam options were determined using the provided LiDAR data. As the structural design of the dam is outside the scope of this report, embankment slope and NMOL freeboard selections align with those established during the prefeasibility. Regardless of dam type, a minimum of 3.5 m of freeboard at the NMOL is provided for each option. The downstream face is sloped at 2:1 and benched at an elevation interval of 15 m. The upstream face is sloped at 2.25:1. A summary of the dimensions for the South and North Dams at each NMOL is presented in *Table 6*. The South Dam specifications in the table correspond to a NMOL of El. 1509 m; each parameter is proportionately lower for the other two options.

Table 6 – Summary of Dam NMOL Elevation and Dimensions for South and North Dam Options

Dam	Normal Maximum Water Depth (m)	Dam Height (m)	Dam Width (m)
South Dam	30.68	34.5	348
North Dam El. 1500 m NMOL	39.37	43.0	257
North Dam El. 1505 m NMOL	40.46	44.0	265
North Dam El. 1509 m NMOL	40.80	44.5	236

4.2.2. Dam Types

Embankment dams, and more specifically rockfill embankment dams, were primarily considered for their cost, feasibility, and compatibility with the surrounding geology. The embankment dam options explored included:

- Rockfill embankment dam with sheet pile core (*Figure 11*) [14].
- Concrete faced rockfill embankment dam (*Figure 12*) [14].
- Rockfill embankment dam with asphalt core (*Figure 13*) [15].
- Zoned rockfill (ultimately excluded due to challenges in clay transport to the site).

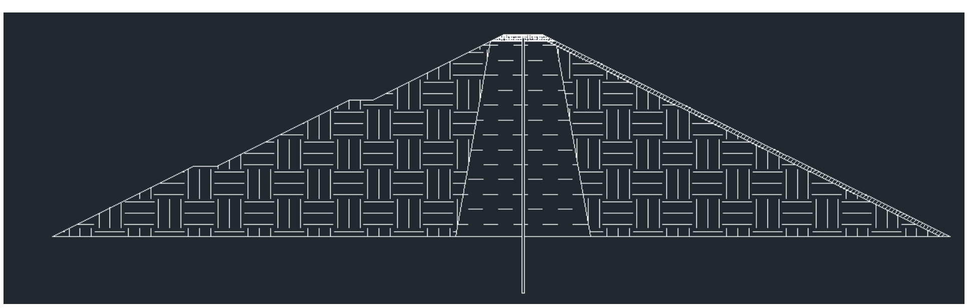

Figure 11 – Rockfill Embankment Dam with Sheet-Pile Core

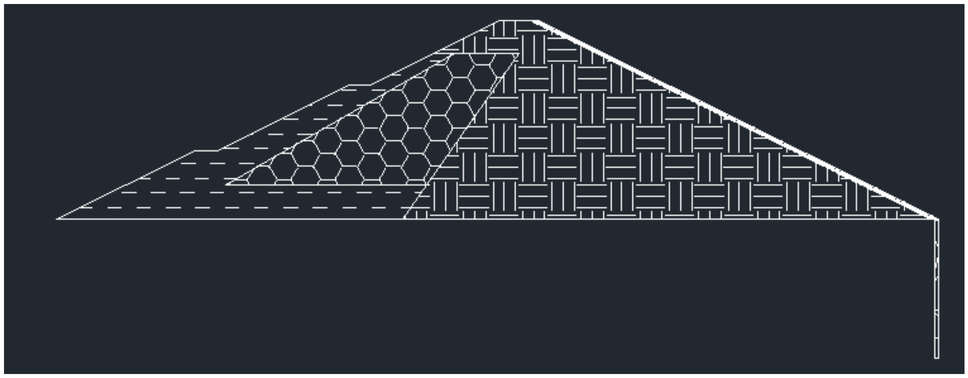

Figure 12 – Concrete Faced Rockfill Embankment Dam

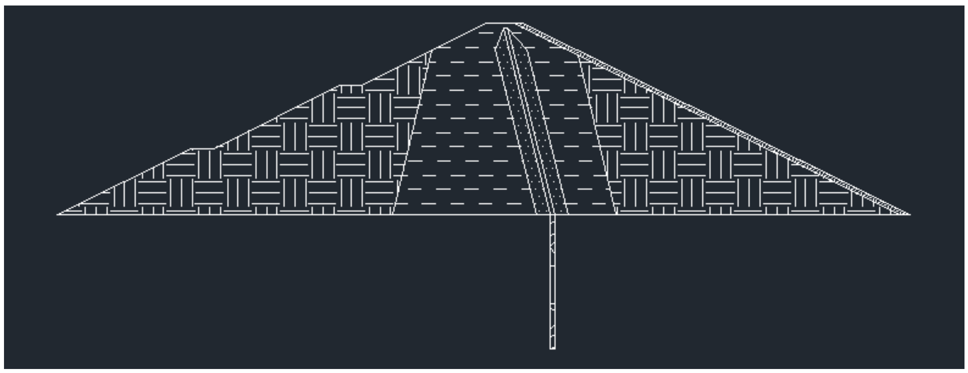

Figure 13 – Rockfill Embankment Dam with Asphalt Core

4.2.3. Cost Analysis

A Class IV cost analysis was completed for the lower reservoir alternative dam types. As this is a high-level cost analysis, the primary focus of this analysis was to find the material cost and use it as a differentiating factor between designs. As with the Upper Reservoir cost estimate, only costs that differed between options were included; labour, equipment, and hauling costs are assumed to be similar across options. Most construction unit costs were obtained from the Government of Alberta 2023 Average Unit Cost Spreadsheet [3]. Some grout curtain specific unit costs for the dams were found in AUD in 2014, which were then converted to CAD [6, 16]. Inflation was calculated using the Bank of Canada Inflation Calculator [7]. Any values from other sources were subjected to the same conversion and inflation as required [17]. Results from the high-level cost analysis are summarized in *Table 7*, and full calculations are shown for each alternative in Appendix B Lower Reservoir Additional Figures.

Table 7 – Lower Reservoir Alternative Dam Design Cost Analysis

Dam Type	Cost (CAD)
Rockfill embankment dam with sheet pile core	$42,919,919.54
Concrete faced rockfill embankment dam	$47,820,250.23
Rockfill embankment dam with asphalt core	$51,692,034.65

The cost analysis was performed by calculating the volume of each material based on the cross-sectional area of the material multiplied by the associated width based on the topography. To simplify calculating the mountainous topography, 5 m benched intervals were created to calculate the approximate width of every section of the reservoir. This volume was then multiplied by the material cost of each portion of the dam to form our final material costs.

5 Final Design Selection

A weighted decision matrix was used to select the best alternative in each of the design categories detailed herein. Where possible, objective values are used for comparison (e.g., distances or areas) but some metrics are rated subjectively based on available knowledge at the time of writing (e.g., dam constructability). Proportionalities are set up so that the maximum score is 1, and weights are assigned separately. For metrics for which the optimal result is maximized, the following equation was used:

$$\Delta metric \propto \frac{value}{value_{max}}$$

For metrics for which the optimal result is minimized, the similar equation below was used:

$$\Delta metric \propto \frac{value_{min}}{value}$$

where $\Delta metric$ is the difference in the relevant metric between each alternative and the optimal case for that metric, and $value$ is the objective or subjective assigned to each alternative. For quantitative metrics, the actual number for each metric was used as the value in the decision matrix. For qualitative metrics, each alternative was given a rank from 1 to 10 with 1 being the worst score an alternative could receive and 10 being the best.

The decision matrix and selection rationale for the Upper Reservoir Sizing, Lower Reservoir Sizing, and Lower Reservoir Dam Structure Type are presented in Sections 5.1, 5.2, and 5.3, respectively.

5.1 Upper Reservoir Sizing

The three upper reservoir NMOL alternatives and their corresponding DSLs were investigated in depth in Section 3. These alternatives were compared on the six weighted metrics below to select the final design solution. *Table 8* summarizes the weighted scores for each alternative, and Appendix A Upper Reservoir Additional Figures shows assigned values and in-depth scoring. Total weight across all categories is 1.0.

- Constructability: Each alternative was given a qualitative score for constructability. Higher constructability scores represent greater ease of construction. These ratings account for mobilization, the amount of equipment required for each method, and how each option's construction integrates with other construction on site. This criteria was assigned a weight of 0.2 because the ease of construction is imperative due to the isolation of the site.

- Risk: Qualitative scoring was also used for risk. Higher risk scores denote little risk of overtopping or slope failure. Lower NMOLs correspond to higher scores. Overtopping risk is reduced when there is available containment above the NMOL and reduced exposure to wind due to the site's topography. Slope stability risk is reduced when there is less force against the containment structure, and in the absence of constructed rockfill. This criteria was assigned a weight of 0.3 because it is imperative the final solution is safe and will not fail.
- Flooded Area: This is a quantitative metric representing final water surface area. A higher NMOL corresponds to a greater flooded area, which increases evaporation losses. It has a weight of 0.05 because the upper reservoir's flooded area is small and pre-developed, new environmental impacts are limited, and differences in evaporation losses are minimal.
- Flux from Reservoir: This is a quantitative metric obtained from seepage modeling (see Section 3.2). A higher NMOL corresponds to higher flux due to additional water contact with the rockface. This criteria was assigned a weight of 0.05 because seepage reduction techniques will reduce flux for all alternatives.
- Total Cost: This is a quantitative metric obtained form the cost analysis in Section 3.1.2. It has a weight of 0.3 because lower construction costs are vital when considering the economic feasibility of each alternative, and the likelihood the final solution will be constructed.
- Dead Storage: This is a quantitative metric representing the amount of water storage below the penstock intake which cannot be used for electricity generation. Less dead storage gives an alternative a better score on this metric as it reduces the time required to initially fill the reservoir. It has a weight of 0.1 because it is important to make efficient use of the resources already present on the site.

Table 8 – Upper Reservoir NMOL Decision Matrix

Metric	Weight	Weighted Score		
		El. 1803.3 m	El. 1807 m	El. 1810 m
Constructability	0.2	0.125	0.200	0.100
Risk	0.3	0.300	0.167	0.067
NMOL Flooded Area	0.05	0.050	0.047	0.045
Flux From Reservoir	0.05	0.050	0.044	0.040
Total Cost	0.3	0.300	0.273	0.246
Dead Storage	0.1	0.100	0.066	0.051
Total	**1.00**	**0.925**	**0.797**	**0.549**

The alternative with NMOL at El. 1803.3 m and penstock intake at El. 1760 m was selected. It achieved the best comparative score on all environmental, risk, and cost metrics. The tunneling requirement for this alternative lowered its constructability score but it remains the most suitable alternative for this project.

5.2 Lower Reservoir Sizing

Three locations and corresponding NMOLs were evaluated. The main differences between the options are as follows:

- Catchment Area: A lower NMOL corresponds to a greater catchment area, which will cause the reservoir to fill more quickly after construction and provide greater supply security against drought conditions.
- Flooded Area: A lower NMOL corresponds a greater flooded area, which will affect more evaporation losses, making the facility more susceptible to drought conditions. It will also increase the amount of forest cleared for construction of the reservoir.
- Constructability: Considerations include haul distance (defined as the distance from the dam centreline to the main access road), average ground slope in the vicinity, and required dam width.
- Required Crown Land encroachment: A lower NMOL corresponds to a greater encroachment into public land, incurring greater costs.

In the decision matrix, the shortest hauling distance, the least average slope, and the narrowest dam span were used as base values; furthermore, the difference in constructability was assumed to be proportional to the difference in each parameter. As all constructability parameters and the Crown Land encroachment seek to minimize, their proportionality formula resembles closely that of evaporation. *Table 9* presents the decision matrix used to evaluate the location/NMOL options.

Table 9 – Dam Location and NMOL Decision Matrix

Metric	Weight	Weighted Score		
		El. 1500 m	**El. 1505 m**	**El. 1509 m**
Catchment Area	0.25	0.250	0.200	0.175
NMOL Flooded Area	0.25	0.139	0.194	0.250
Haul Distance	0.075	0.047	0.066	0.075
Average Slope in Vicinity	0.05	0.029	0.043	0.050
Required Dam Width	0.125	0.094	0.094	0.125
Required Crown Land Area	0.25	0.167	0.222	0.250
Total	**1**	**0.725**	**0.819**	**0.925**

El. 1509 m scored the highest due largely to its performance with respect to the flooded area and required Crown Land area metrics.

5.3 Lower Reservoir Dam Structure Type

Three dam types were evaluated. The topography, ground characteristics, and access to materials is very similar at any of the three locations considered; therefore, it is assumed that whatever dam type is most suitable in one location will be most suitable in the others as well.

- Constructability: A higher value in this category represents greater ease of dam construction, with the focal points of this metric being the ease of material acquisition and feasibility of construction.
- Cost: This metric was calculated in Section 4.2.3. A lower cost was preferred.
- Environmental Impacts: A higher value in this category represents fewer negative impacts over the lifecycle of the dam, with the focal points of this metric being the ease of material acquisition, sustainability of individual materials and lifecycle impacts.

Table 10 – Dam Type Decision Matrix

Metric	Weight	Weighted Score		
		Asphalt Core	Sheet Pile	Concrete Faced
Constructability	0.25	0.219	0.125	0.250
Cost	0.4	0.286	0.400	0.343
Environmental Impacts	0.35	0.350	0.300	0.250
Total	**1**	**0.854**	**0.825**	**0.843**

Rockfill embankment dam with an asphalt core scored the highest due to its lifecycle environmental impacts and overall constructability. Although all designs were ranked similarly, the rockfill embankment dam with a sheet pile core was held back due to the constructability of installing such large pieces of sheet pile and the environmental lifecycle impacts of creating and transporting the steel to site. Additionally, the concrete faced rockfill embankment dam was held back by the wear and tear the concrete would face while exposed to the elements, so its lifecycle was considered shorter than other alternatives.

6 Sustainability, Engineering Risks, and Professional Impacts

6.1 Sustainability

Throughout the project, a comprehensive approach to the three pillars of sustainability was applied. Design choices to enhance environmental sustainability include minimizing its dam structure footprints using the valley's natural contours, reducing transportation-related emissions by utilizing on-site and locally available materials, and minimizing the inundated area. Social sustainability was upheld by minimizing impacts on Crown Land in the north part of the valley. TransAlta has also engaged in consultations within the Crowsnest Pass community and Treaty 7 Indigenous stakeholders. The project is also committed to hiring local and Indigenous workers for its construction and operation. Economically, the focus is on reducing transportation and material costs through using on-site and local materials. The TMPHES will also contribute sustainably to the local economy by providing renewable energy-based job opportunities in a region that has been reliant on coal for the past century.

6.2 Professionalism and Risk

The development of a Pumped Hydro Energy Storage system involves significant responsibility for professional engineers, especially concerning public safety, environmental protection, and the broader concept of public interest. To ensure the project's overall integrity and downstream safety, a geotechnical analysis and hydrological assessment was conducted. To ensure safety and ethical standards, the Canadian Dam Association rating will be classified as extreme before conducting a complete dam break analysis, given the potential consequences align with that classification. To foster continuous improvement and technological innovation, the concept of a 16-hour storage capacity was explored; aimed at enhancing system efficiency and ecological performance. Throughout the project, a key focus has been on maximizing the utilization of existing environmental features to incorporate with the project's requirements, while maintaining high level of environmental stewardship.

7 Conclusion

Table 11 summarizes the design of the PHES.

Table 11 – Design Summary

Metric	Value
Upper Reservoir	
Drainage Area (km^2)	1.311
Live Storage Volume (million m^3)	7.107
NMOL (masl)	1803.3
NMOL Reservoir Footprint (km^2)	0.2758
Dead Storage Volume (million m^3)	2.045
DSL (masl)	1760.0
Seepage Reduction Strategy	Grout curtain and grouted cutoff wall
Containment Structure Minimum Elevation (masl)	1806.8
Lower Reservoir	
Drainage Area (km^2)	5.557
Live Storage Volume (million m^3)	7.107
NMOL (masl)	1509.0
Dead Storage Volume (million m^3)	0.5268
DSL (masl)	1480.0
South Dam	
Dam Crest Elevation (masl)	1513.0
Dam Type	Asphalt core rockfill embankment
North Dam	
Dam Crest Elevation (masl)	1512.5
Dam Type	Asphalt core rockfill embankment

This design report has been provided for the exclusive use of TransAlta, Evolve Power, and the Schulich School of Engineering. All designs have been developed using the best information available at the time of writing. This report does not provide any guarantees that are not explicitly written herein. The following steps will be undertaken for the winter semester:

- Complete the water balance and hydrological modeling.
- Conduct a stochastic and time series analysis.
- Identify the impacts of climate change and variability on the system.

We look forward to continued progress on this project in the coming semester.

Sincerely,

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9 Appendix A Upper Reservoir Additional Figures

A1. Upper Reservoir Cost Analysis

1803 NMOL	
Properties	
Area North Wall (km^2)	0.00984
Area East Wall (km^2)	0.02996
Area South Wall (km^2)	0.0109
Area West Wall (km^2)	0.087
Length Total West Wall (m)	866.6666667
Vertical Cutoff Length West Wall (m)	450
Distance Between Grout Holes (m)	1
Estimated Grout Holes for Cutoff Wall	450
Average Depth of Grout for Cutoff Wall (m)	25
Area of West Wall Surface Grout (km^2)	0.041826923
Area of South Wall Surface Grout (km^2)	0.004459091
Area of East Wall Surface Grout (km^2)	0.02996
Area of North Wall Surface Grout (km^2)	0.00984
Total Area for Surface Grout (m^2)	86086.01399
Estimated Volume of Rock Fill Required (m^3)	0
Penstock Area of Excavation (m^2)	6666.666667
Penstock Depth of Excavation (m)	7
Estimated Volume of Excavation Required (m^3)	46666.66667
Area of Penstocks (m^2)	31.80862562
Length to DSL from Penstock Location (m)	83.33333333
Estimated Volume of Tunneling Required (m^3)	2650.718801
Unit Costs	
Grout for Grout Curtainand Cutoff Wall ($/m^2)	406.06972
Drilling (m)	197.85
Rock Transport ($/m^3/km) (assume 1km site travel)	1
Channel Excavation ($/m^3)	28.34
Rock Excavation Premium ($/m^3)	14.43
Tunneling ($/m^3)	750
Total Costs	
Exterior Grouting	$ 34,956,923.60
Vertical Cutoff Wall	$ 6,794,096.85
Rock Fill	$ -
Excavation	$ 1,995,933.33
Tunneling	$ 1,988,039.10
Total Cost for 1803 NMOL	$ 45,734,992.88

Grout Cutoff Cost Conversion	
Cost/ft^2 (1999 USD)	15
USD/CAD Rate	0.673
Inflation 1999-2023	1.6926
Conversion ft^2 to m^	0.0929
Cost/m^2 (2023 CAD)	406.07

Appendix A Figure 1 – Cost Analysis Upper Reservoir 1803.3 NMOL

1807 NMOL	
Properties	
Area North Wall (km^2)	0.01368
Area East Wall (km^2)	0.03572
Area South Wall (km^2)	0.01185
Area West Wall (km^2)	0.09399
Length Total West Wall (m)	910
Vertical Cutoff Length West Wall (m)	500
Distance Between Grout Holes (m)	1
Estimated Grout Holes for Cutoff Wall	500
Average Depth of Grout for Cutoff Wall (m)	30
Area of West Wall Surface Grout (km^2)	0.042347143
Area of South Wall Surface Grout (km^2)	0.004847727
Area of East Wall Surface Grout (km^2)	0.03572
Area of North Wall Surface Grout (km^2)	0.01368
Total Area for Surface Grout (m^2)	96594.87013
Estimated Volume of Rock Fill Required (m^3)	0
Penstock Area of Excavation (m^2)	6666.666667
Penstock Depth of Excavation (m)	7
Estimated Volume of Excavation Required (m^3)	46666.66667
Area of Penstocks (m^2)	31.80862562
Length to DSL from Penstock Location (m)	0
Estimated Volume of Tunneling Required (m^3)	0
Unit Costs	
Exterior Grout Curtain ($/m^2)	406.06972
Drilling (m)	197.85
Rock Transport ($/m^3/km) (assume 1km site travel)	1
Channel Excavation ($/m^3)	28.34
Rock Excavation Premium ($/m^3)	14.43
Tunneling ($/m^3)	750
Total Costs	
Exterior Grouting	$39,224,251.87
Vertical Cutoff Wall	$ 9,058,795.80
Rock Fill	$ -
Excavation	$ 1,995,933.33
Tunneling	$ -
Total Cost for 1807 NMOL	$50,278,981.00

Grout Cutoff Cost Conversion	
Cost/ft^2 (1999 USD)	15
USD/CAD Rate	0.673
Inflation 1999-2023	1.6926
Conversion ft^2 to m^2	0.092903
Cost/m^2 (2023 CAD)	406.0697

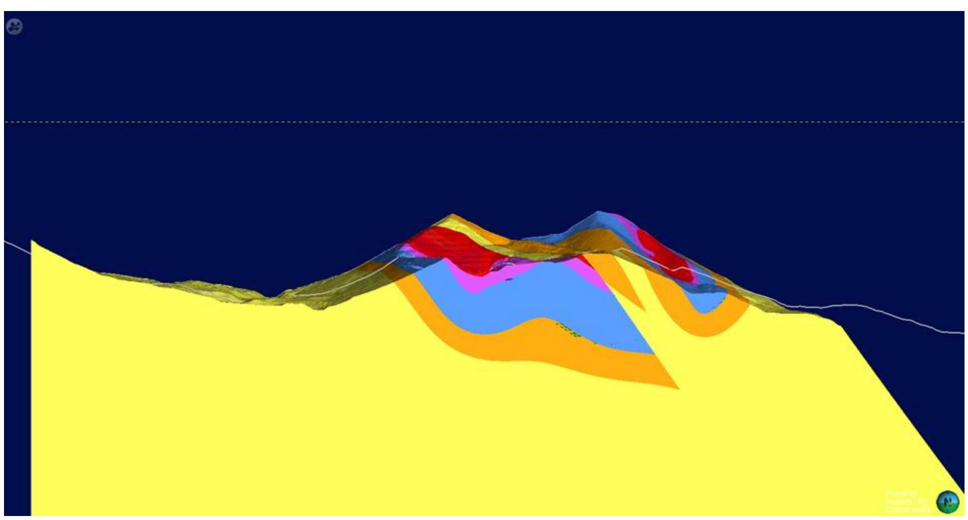
Appendix A Figure 2 - Cost Analysis Upper Reservoir 1807 NMOL

1810 NMOL	
Properties	
Area North Wall (km^2)	0.01687
Area East Wall (km^2)	0.0393
Area South Wall (km^2)	0.01466
Area West Wall (km^2)	0.09843
Length Total West Wall (m)	950
Vertical Cutoff Length West Wall (m)	516.6666667
Distance Between Grout Holes (m)	1
Estimated Grout Holes for Cutoff Wall	517
Average Depth of Grout for Cutoff Wall (m)	34
Area of West Wall Surface Grout (km^2)	0.044897895
Area of South Wall Surface Grout (km^2)	0.005997273
Area of East Wall Surface Grout (km^2)	0.0393
Area of North Wall Surface Grout (km^2)	0.01687
Total Area for Surface Grout (m^2)	107065.1675
Estimated Volume of Rock Fill Required (m^3)	38888.88889
Penstock Area of Excavation (m^2)	0
Penstock Depth of Excavation (m)	0
Estimated Volume of Excavation Required (m^3)	0
Area of Penstocks (m^2)	31.80862562
Length to DSL from Penstock Location (m)	0
Estimated Volume of Tunneling Required (m^3)	0
Unit Costs	
Exterior Grout Curtain ($/m^2)	406.06972
Drilling (m)	197.85
Rock Transport ($/m^3/km) (assume 1km site travel)	1
Channel Excavation ($/m^3)	28.34
Rock Excavation Premium ($/m^3)	14.43
Tunneling ($/m^3)	750
Total Costs	
Exterior Grouting	$43,475,922.58
Vertical Cutoff Wall	$10,611,098.71
Rock Fill	$ 1,702,166.67
Excavation	$ -
Tunneling	$ -
Total Cost for 1810 NMOL	$55,789,187.96

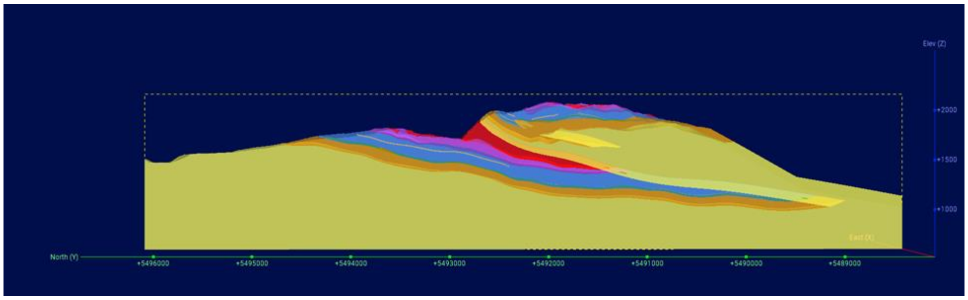
Grout Cutoff Cost Conversion	
Cost/ft^2 (1999 USD)	15
USD/CAD Rate	0.673
Inflation 1999-2023	1.6926
Conversion ft^2 to m^2	0.092903
Cost/m^2 (2023 CAD)	406.0697

Appendix A Figure 3 - Cost Analysis Upper Reservoir 1810 NMOL

A2. Leapfrog Model Cross Sections



Appendix A Figure 4 - E-W Cross Section from the Leapfrog Model (South View)



Appendix A Figure 5 – N-S Cross Section from the Leapfrog Model (East View)

A3. Flux Result Table

	Option 1 (MOL 1810m)				Option 2 (MOL 1807m)				Option 3 (MOL 1803m)			
	North Wall	South Wall	West Wall	East Wall	North Wall	South Wall	West Wall	East Wall	North Wall	South Wall	West Wall	East Wall
Area (km^2)	0.01687	0.01466	0.09843	0.0393	0.01368	0.01185	0.09399	0.03572	0.00984	0.0109	0.087	0.02996
Flux (m^3/s/m^2)	2.53E-08	2.53E-08	4.39E-08	4.39E-08	2.32E-08	2.32E-08	4.17E-08	4.17E-08	2.15E-08	2.15E-08	4.17E-08	4.17E-08
Flux (m^3/s)	4.27E-04	3.71E-04	4.32E-03	1.72E-03	3.18E-04	2.75E-04	3.92E-03	1.49E-03	2.11E-04	2.34E-04	3.63E-03	1.25E-03
Total Flux (m^3/s)	6.10E-03				5.45E-03				4.85E-03			

Appendix A Figure 6 - Flux Results and Calculations

A4. Upper Reservoir Full Decision Matrix

		Values			Scores			Weighted Scores		
	Weight	1803	1807	1810	1803	1807	1810	1803	1807	1810
Constructability (Rank)	0.2	5	8	4	0.63	1.00	0.50	0.125	0.200	0.100
Risk (Rank)	0.3	9	5	2	1.00	0.56	0.22	0.300	0.167	0.067
NMOL Flooded Area (km^2)	0.05	0.28	0.30	0.31	1.00	0.94	0.90	0.050	0.047	0.045
Flux Out of Resevoir (m^3/s)	0.05	4.85E-03	5.45E-03	6.10E-03	1.00	0.89	0.80	0.050	0.044	0.040
Total Cost ($)	0.3	$45,734,993	$50,278,981	$55,789,188	1.00	0.91	0.82	0.300	0.273	0.246
Dead Storage (Millions m^3)	0.1	2.05	3.11	4.01	1.00	0.66	0.51	0.100	0.066	0.051
10 is best, 1 is worst	1.00			Total			Total	0.925	0.797	0.549

Appendix A Figure 7 - Upper Reservoir NMOL Complete Decision Matrix

10 Appendix B Lower Reservoir Additional Figures

B1. Lower Reservoir Full Decision Matrix

	Weight	Values (Ranked)			Weighted Score		
		1500	1505	1509	1500	1505	1509
Catchment Area	0.25	10	8	7	0.250	0.200	0.175
NMOL Flooded Area	0.25	5	7	9	0.139	0.194	0.250
Haul Distance	0.075	5	7	8	0.047	0.066	0.075
Average Slope in Vicinity	0.05	4	6	7	0.029	0.043	0.050
Required Dam Width	0.125	6	6	8	0.094	0.094	0.125
Required Crown Land Area	0.25	6	8	9	0.167	0.222	0.250
10 is best, 1 is worst	1			Total	0.725	0.819	0.925

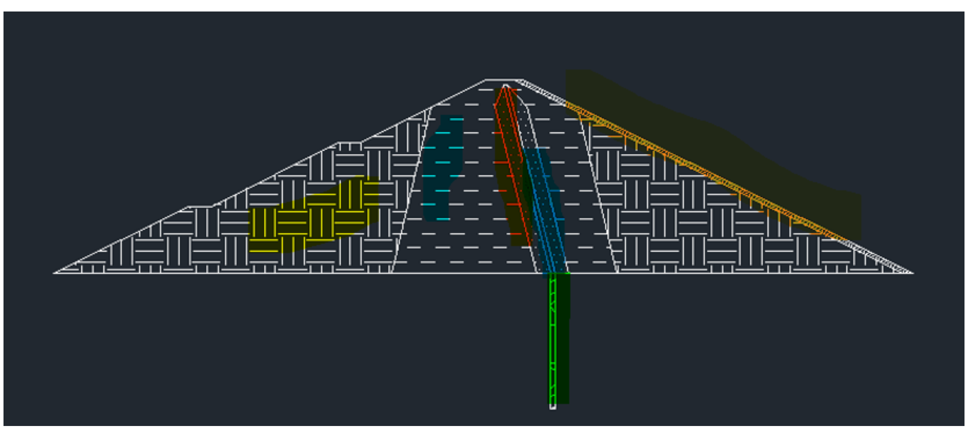
Appendix B Figure 1 - Lower Reservoir NMOL Complete Decision Matrix

B2. Lower Reservoir Dam Types Full Decision Matrix

	Weight	Objective Values (/10)			Weighted Scores		
		Asphalt Core	Sheet Pile	Concrete Faced	Asphalt Core	Sheet Pile	Concrete Faced
Constructability	0.25	7	4	8	0.219	0.125	0.250
Cost	0.4	5	7	6	0.286	0.400	0.343
Enviromental Impacts	0.35	7	6	5	0.350	0.300	0.250
10 is best, 1 is worst	1			Total	0.854	0.825	0.843

Appendix B Figure 2 - Lower Reservoir Dam Types Complete Decision Matrix

B3. Lower Reservoir Dam Types Detailed Cost Analysis



Appendix B Figure 3 - Color Coded Rockfill Embankment Dam with Asphalt Core

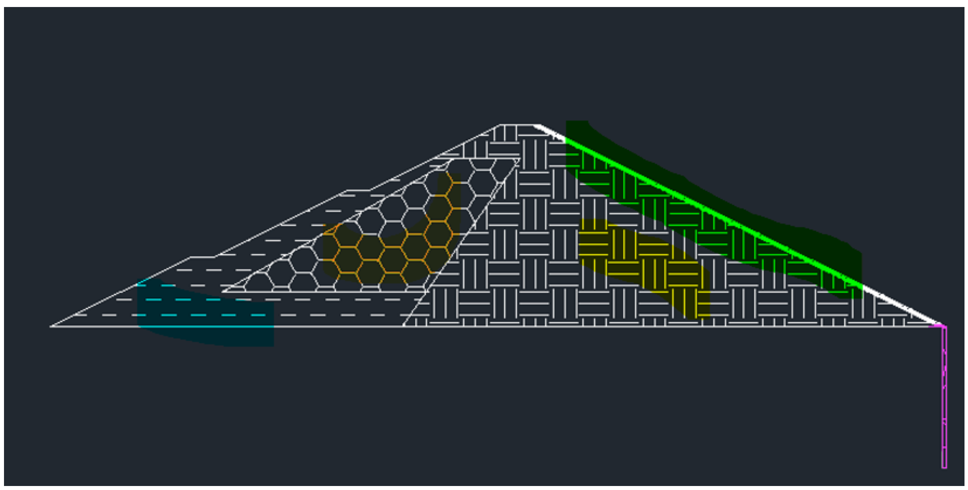
Asphalt core

Bench (top down)	Rip rap	Grout curtain	Filter coarse drain	Rock fill	Filter fine drain	Asphalt
1	9.0	0.0	58.5	0.0	7.6	3.5
2	10.0	0.0	136.3	6.3	29.6	5.0
3	10.0	0.0	163.7	82.5	31.0	5.0
4	10.0	0.0	171.5	205.7	31.0	5.0
5	10.0	0.0	179.2	304.7	31.0	5.0
6	10.0	0.0	186.9	403.8	31.0	5.0
7	10.0	0.0	194.7	527.9	31.0	5.0
8	10.0	0.0	202.4	627.0	31.0	5.0
9	10.0	0.0	210.2	726.0	31.0	5.0

cost

Average Price ($)	368.00	1650	70.00	61.00	21.00	341.00

Appendix B Figure 4 - Color Coded Area and Cost for Rockfill Embankment Dam with Asphalt Core



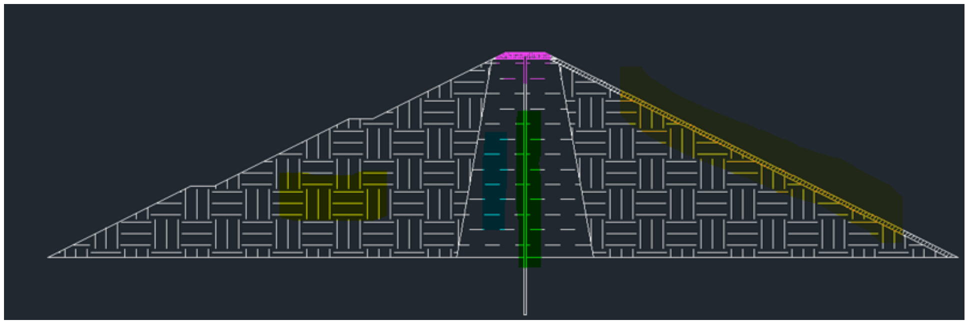
Appendix B Figure 5 - Color Coded Concrete Faced Rockfill Embankment Dam

Concrete faced

Bench (top down)	Grout curtain	Rock fill	Concrete	Secondary rockfill	Downstream rock fill
1	0.0	71.8	6.8	0.0	0.0
2	0.0	135.9	7.5	34.2	9.5
3	0.0	164.8	7.5	95.2	24.7
4	0.0	238.4	7.5	121.7	55.6
5	0.0	312.0	7.5	148.2	62.3
6	0.0	385.6	7.5	174.8	69.0
7	0.0	459.1	7.5	201.3	100.7
8	0.0	532.7	7.5	110.6	224.6
9	0.0	606.3	7.5	0.0	368.4

Average Price ($)	1650	61	1200	21	12

Appendix B Figure 6 - Color Coded Area and Cost for the Concrete Faced Rockfill Embankment Dam



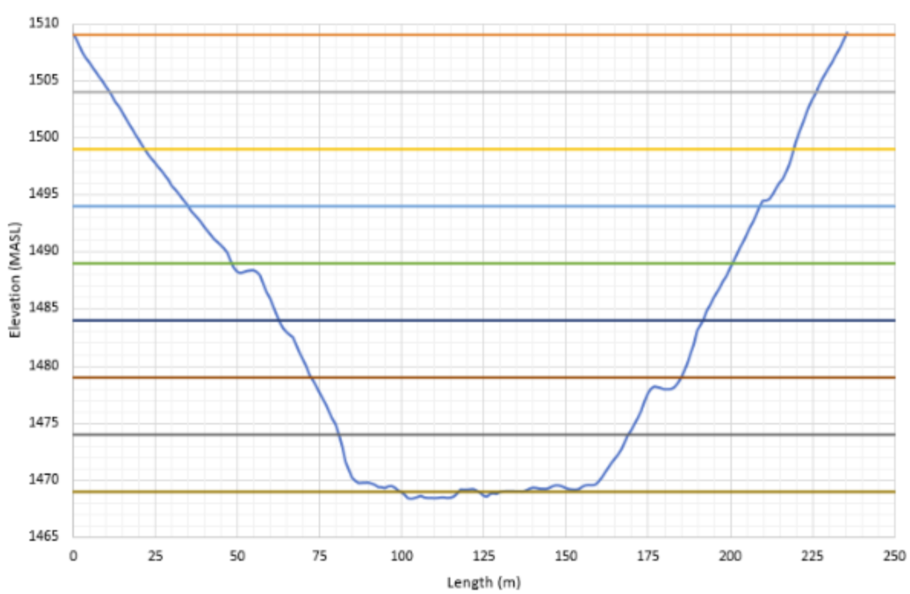
Appendix B Figure 7 - Color Coded Rockfill Embankment Dam with Sheet Pile Core

Sheetpile core

Bench (top down)	Rock fill	Sheet pile	Filter coarse drain	Rip rap	Concrete
1	12.3	3.8	41.8	6.0	16.3
2	95.7	5.0	78.4	10.0	0.0
3	191.8	5.0	87.5	10.0	0.0
4	313.7	5.0	96.4	10.0	0.0
5	411.5	5.0	105.5	10.0	0.0
6	509.3	5.0	114.0	10.0	0.0
7	632.1	5.0	123.5	10.0	0.0
8	729.9	5.0	132.5	10.0	0.0
9	827.7	5.0	141.5	10.0	0.0

Average Price ($)	61	6.04	70	368	1200

Appendix B Figure 8 - Color Coded Area and Cost for Rockfill Embankment Dam with Sheet Pile Core



Appendix B Figure 9 - Benched Width on 1509 NMOL Lower Reservoir Topography

Volume

Bench (top down)	Benched Width
1	235.28
2	215.02
3	197.75
4	174.27
5	152.61
6	129.15
7	112.80
8	88.18
9	37.07

Appendix B Figure 10 - Associated Values for Benched Width at 1509 NMOL

Asphalt core

Bench (top down)	Rip rap	Grout curtain	Filter coarse drain	Rock fill	Filter fine drain	Asphalt
1	$ 779,247.36	$ -	$ 963,471.60	$ -	$ 37,550.69	$280,806.68
2	$ 791,273.60	$ -	$2,051,505.82	$ 82,632.19	$133,656.43	$366,609.10
3	$ 727,720.00	$ -	$2,266,017.25	$ 995,176.88	$128,735.25	$337,163.75
4	$ 641,306.24	$ -	$2,092,087.34	$2,186,662.58	$113,448.47	$297,126.94
5	$ 561,590.08	$ -	$1,914,289.66	$2,836,441.94	$ 99,346.51	$260,193.23
6	$ 475,279.36	$ -	$1,689,695.62	$3,181,246.23	$ 84,077.95	$220,204.16
7	$ 415,115.04	$ -	$1,537,392.09	$3,632,470.93	$ 73,434.75	$192,329.12
8	$ 324,487.68	$ -	$1,249,277.57	$3,372,467.47	$ 57,402.58	$150,340.08
9	$ 136,410.24	$ -	$ 545,418.55	$1,641,593.45	$ 24,131.27	$ 63,200.94
underground	$ -	$ 11,682,000.00	$ -	$ -	$ -	$ -
total	$ 51,692,034.65					

Appendix B Figure 11 - Full Cost Analysis for the Rockfill Embankment Dam with Asphalt Core

Concrete faced

Benching (top down)	Grout curtain	Rock fill	Concrete	Secondary rockfill	Downstream rock fill
1	$ -	$1,030,479.34	$1,919,884.80	$ -	$ -
2	$ -	$1,782,494.30	$1,935,180.00	$154,427.36	$ 24,512.28
3	$ -	$1,987,941.20	$1,779,750.00	$395,341.80	$ 58,613.10
4	$ -	$2,534,274.96	$1,568,412.00	$445,376.73	$116,271.61
5	$ -	$2,904,397.39	$1,373,454.00	$474,940.39	$114,088.25
6	$ -	$3,037,861.68	$1,162,368.00	$474,091.16	$106,937.86
7	$ -	$3,159,059.30	$1,015,227.00	$476,852.12	$136,311.15
8	$ -	$2,865,252.67	$ 793,584.00	$204,797.58	$237,651.96
9	$ -	$1,370,934.03	$ 333,612.00	$ -	$163,870.21
underground	$ 11,682,000.00	$ -	$ -	$ -	$ -
total	$ 47,820,250.23				

Appendix B Figure 12 - Full Cost Analysis for the Concrete Faced Rockfill Embankment Dam

Sheet pile core

Benching (top down)	Rock fill	Sheet pile	Filter coarse drain	Rip rap	Concrete
1	$ 176,530.58	$ 5,400.15	$ 688,429.28	$519,498.24	$4,602,076.80
2	$ 1,255,222.25	$ 6,493.60	$1,180,029.76	$791,273.60	$ -
3	$ 2,313,635.45	$ 5,972.05	$1,211,218.75	$727,720.00	$ -
4	$ 3,334,740.17	$ 5,262.89	$1,175,960.46	$641,306.24	$ -
5	$ 3,830,639.51	$ 4,608.70	$1,126,995.31	$561,590.08	$ -
6	$ 4,012,403.93	$ 3,900.39	$1,030,632.96	$475,279.36	$ -
7	$ 4,349,469.35	$ 3,406.65	$ 975,181.94	$415,115.04	$ -
8	$ 3,925,939.41	$ 2,662.92	$ 817,832.40	$324,487.68	$ -
9	$ 1,871,552.20	$ 1,119.45	$ 367,158.54	$136,410.24	$ -
underground	$ -	$42,763.20	$ -	$ -	$ -
total	$ 42,919,919.54				

Appendix B Figure 13 - Full Cost Analysis for the Rockfill Embankment Dam with Sheet Pile Core