CULTANA SEAWATER PUMPED HYDRO-ELECTRIC ENERGY STORAGE PROJECT – GEOTECHNICAL OPTIONEERING DESIGN

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ABSTRACT

This paper presents the concept geotechnical design carried out for the optioneering stage (feasibility stage business case) on the Cultana Seawater Pumped Hydro-Electric project. The site is located 10km south-west of Port Augusta in South Australia. The site spans east to west from the banks of the Spencer Gulf (RL +2m AHD) to a hill located west (RL+250m AHD). The site is underlain by varying geology ranging from Alluvial sands more than 30m thick near shore to interbedded Shale, Sandstone and Quartzite formations located within 2m of ground level at the hill.

The objective of this project is to provide between 100MW and 250MW of dispatchable peak electricity to the National Electricity Market in South Australia, as well as providing a range of grid support and ancillary services.

This paper discusses the geotechnical concept design for a 40m deep bunker style powerhouse structure located nearshore, a 3.4km onshore transfer pipeline connecting the powerhouse to the hill penstock pipeline, the penstock pipeline that transverses up and down the steep hill to a height of approximately 220m above ground level and a 2.9GL reservoir located on a flat plateau at the top of the hill.

Each of the four structures presented above had its own unique geotechnical risks relating to dewatering, foundation design and reservoir wall design. The powerhouse structure required a permanent retention system that was fully waterproof. A diaphragm wall was considered the most appropriate solution. The transfer pipeline intersected highly variable geology with sections comprising deeper soils, shallow caprock and bedrock. Trench excavations with rock breaking or hydraulic excavation was recommended. The penstock pipeline loads, and its serviceability criteria required deep foundations. The steep hill slope made access using typical piling machinery a challenge. A group micropile option was considered the safest solution. The reservoir was designed for daily rapid drawdown which would result in potential wall instability and erosion. A crushed sandstone wall with a clay core was designed to mitigate this risk. Each option was selected based on hydraulic considerations, cost effectiveness, constructability and safety in design.

1 INTRODUCTION

The Cultana Seawater Pumped Hydroelectric Energy Storage (SPHES) Project is a grid scale electricity storage solution aiming to produce between 100MW and 250MW of dispatchable peak electricity for the National Electricity Market (NEM) grid in South Australia. The proposed development site is located within the Department of Defence (DoD) military land and Crown Land. These two areas are divided by Shacks Road.

The project was divided into four major structures requiring geotechnical design; a bunker style buried powerhouse with an intake / outlet structure connecting to the Spencer Gulf, a transfer pipeline connecting the powerhouse to the inland, a penstock pipe across a steep hill and a reservoir located on an elevated plateau at the top of the hill. Figure 1 shows the location of each of these structures. The reservoir, penstock and the transfer pipeline are located within the DoD land. The powerhouse, water intake / outlet structure and a section of the transfer pipeline is located within the Crown land.

A desktop study and a preliminary geotechnical investigation comprising test pit excavations, borehole drilling, laboratory testing and geotechnical mapping was carried out during the optioneering stage. The results of these investigations indicated highly variable ground conditions across the site. Subsequently, each structure has been designed to specific ground conditions. Access along the hill slope was unsafe, as a result only surface mapping of the hill slope was carried out. The preliminary geotechnical investigation provided enough information to highlight the key geotechnical risks for each structure. It is anticipated a detailed geotechnical investigation will be carried out during the detailed design stage of this project.

The purpose of this geotechnical optioneering stage was to provide viable concept design options and associated costs for the feasibility stage business case. A reasonably high level geotechnical design was carried for each of the four structures. Discussions have taken place with civil, structural and mechanical engineers, industry civil contractors and geotechnical construction specialists in developing each option.

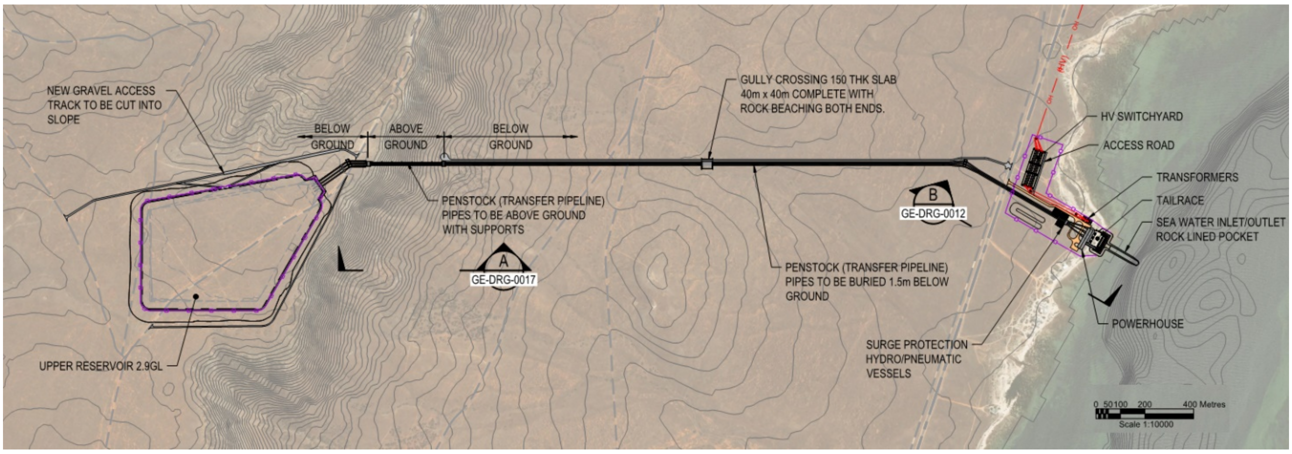

Figure 1: Project overview showing the four key structures

2 GEOTECHNICAL INVESTIGATION

2.1 SCOPE

The preliminary geotechnical investigation comprised underground services and Unexploded Ordnances (UXO) locating, geotechnical mapping of the hill, 35 no. test pit excavations and 4 no. boreholes. These works were carried out in accordance with AS1726-2017.

- 24 no. test pits for reservoir design and borrow area
- 11 no. test pits for transfer pipeline and penstock design
- 2 no. boreholes for reservoir foundation design
- 2 no. boreholes for powerhouse retention and foundation design

2.2 GEOLOGY

2.2.1 Hill Slope and Reservoir

Published geological information by the Government of South Australia (2017) suggests the elevated plateau comprises Proterozoic aged Tent Hill Formation, which includes laminated Shale, Quartzose Sandstone and Quartzite. This formation is likely to be overlain by relatively thin surficial Aeolian and Alluvial soils and residual soils of the Tent Hill Formation.

2.2.2 Transfer Pipeline and Powerhouse Structure

Published geological information by the Government of South Australia (2017) indicates the lower lying areas from the toe of the hill to the shoreline are underlain by Quaternary recent aged Pooraka Formation, which comprises gravels, sands and clayey sands with clay lenses. The Pooraka Formation is likely to be underlain by Quaternary aged Hindmarsh Clay.

Based on the limited geological data to date and considering the proximity of these lower lying areas to the hill, it is likely the Proterozoic aged Tent Hill Formation underlies the Pooraka Formation however it is not known whether any additional units lie in between.

2.2.3 Groundwater

Published groundwater data by the Location SA Map Viewer (2017) indicates standing groundwater levels between 2m and 5m Below Ground Level (BGL) within the coastal region, between 5m and 10m BGL across the DoD land and between 10m and 20m BGL around the reservoir.

2.2.4 Acid sulphate soils

The ASRIS (2017) database classifies the near shore area as A2: High probability / Moderate confidence and the DoD land as C4: Extremely low / Very low confidence.

3 GEOTECHNICAL CONCEPT DESIGN

3.1 RESERVOIR

3.1.1 Location and geometry

The proposed reservoir is located at the elevated plateau and will be constructed above existing ground levels. The reservoir is diamond shaped and is approximately 2km circumference. The maximum height of the reservoir is 15m, with reservoir walls likely to vary between 7m and 15m. Excavations up to 3m below existing ground level are anticipated.

3.1.2 Ground conditions

The encountered ground conditions based on the preliminary geotechnical site investigation are presented in Table 1.

Table 1: Summary of ground conditions - Reservoir

Geological Unit	Description	Inferred strength	Depth to top of unit (m)	Thickness of unit (m)
Topsoil	Silty Clay and Clayey Silt	Soft to Firm	0m	0.1-0.3m
Aeolian and Alluvial Soils (Pooraka Formation)	Clay, Silt and Clayey Sand, with Ironstone Gravels	Firm to Hard	0.1-0.3m	0.5m
Sandstone Cap rock / Cemented Sands *(localised)*	Iron and quartz cemented soils	V. low strength	0.5m	1-2m
Residual Soils (Weathered Proterozoic Tent Hill Formation)	Silty Clay and Clayey Gravel, trace Ironstone Gravels	Stiff to Hard	0.6-0.8m	0.5m
Proterozoic Tent Hill Formation	Interbedded Shale, Quartzose Sandstone and Quartzite	Low to extremely high strength	1-1.5m	Not established

3.1.3 Excavation conditions

The proposed excavation is likely to terminate in low to extremely high strength Proterozoic Tent Hill Formation. Conventional excavation equipment such as a 30T excavator is considered adequate to excavate through soils. A D10 / D11 bulldozer was recommended to excavate through low strength bedrock with the use of a rock breaker or blasting through high strength bedrock.

3.1.4 Concept design

Two concept designs were considered for the reservoir wall design:

- Option 1: Crushed sandstone embankment with a clay core. The feasibility of this option was dependent on the durability of the sandstone upon daily water flow and during rapid drawdown. A concrete cover was installed where inflow and outflow pipes connect to the reservoir. This option was considered more economically viable considering majority of the clay was available locally. This wall design will require further assessment on its impact under surge loading and water hammer action.
- Option 2: Crushed sandstone – cement mix embankment with a clay core. This was considered a better design where rapid drawdown was a concern.

Other concepts were considered, such as a full concrete wall, balanced cut and fill earthworks, and EPDM liners, however, the limitations of each of these concepts was considered too great to incorporate at optioneering stage. A polyethylene liner was recommended along the reservoir walls and compacted clay liner along the base of the reservoir.

Figure 1 and 2 show typical drawings of the crushed sandstone – clay core embankment and concrete cover around the penstock pipes.

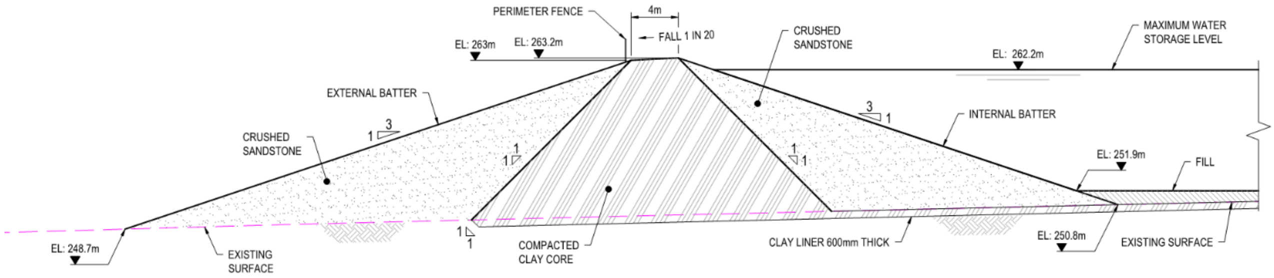

Figure 2: Typical embankment profile showing crushed sandstone and clay core

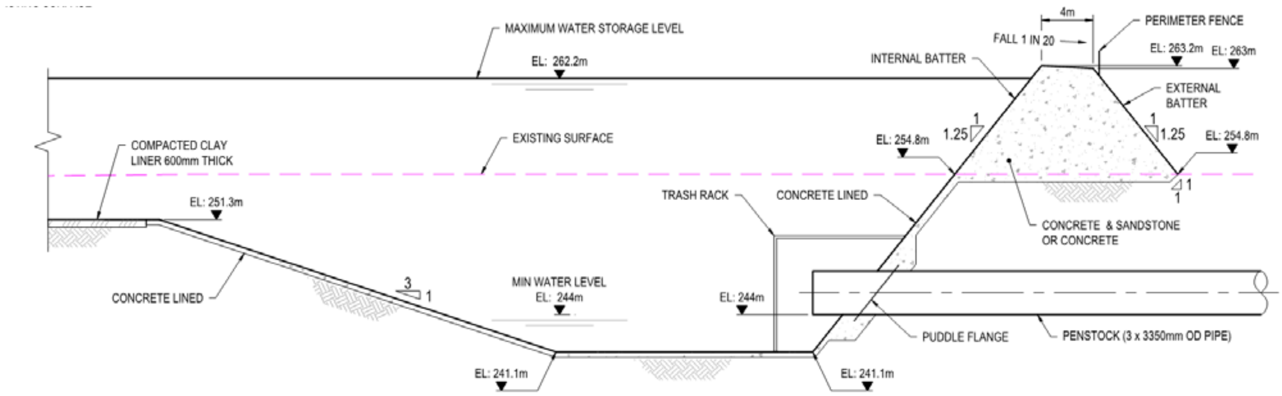

Figure 3: Typical embankment profile showing reservoir wall interphase at pipe inlet / outlet

3.2 PENSTOCK

3.2.1 Location

The penstock pipeline is located along the hill slope spanning from approximately 40m AHD to approximately 250m AHD.

3.2.2 Ground conditions

The geotechnical model was based on test pit and borehole data, geotechnical mapping of the cliff face and data from published geological maps. Due to unsafe access along the hill slope, geotechnical investigations were only carried out along the toe and crest of the penstock pipeline. A summary of the geotechnical model is presented in Table 2. A detailed geotechnical investigation will be carried out during the detailed design stage.

Table 2: Summary of ground conditions - Penstock

Geological Unit	Description	Inferred strength	Depth to top of unit (m)	Thickness of unit (m)
Topsoil	Silty Clay and Clayey Silt	Soft to Firm	0m	0.1-0.3m
Colluvium	Silty Gravelly Clay	Firm to Hard	0.1-0.3m	5m-10m
Proterozoic Tent Hill Formation	Interbedded Shale, Quartzose Sandstone and Quartzite	Low to extremely high (Inferred)	Not established	Not established
Corraberra Sandstone (Inferred)	Red-brown Micaceous sandstone	Low to extremely high (Inferred)	Not established	Not established
Tregolana Shale (Inferred)	Purple and brown Sandy Siltstones and Shales	Low to medium strength (Inferred)	0.3-0.4m (toe of hill)	Not established

3.2.3 Concept design and geometry

The mechanical and hydraulic design of the reservoir governed the geometry and total number of penstock pipes. A total of 3 no. identical pipes, each 3.35m dia., were designed to transfer seawater to the reservoir. Three foundation systems were considered:

- Option 1: Truss bridge with penstock pipeline resting on bridge girders. The span between girders was envisaged between 45m and 120m subject to further structural design during the detailed design stage.
- Option 2: A pipe on pier arrangement. This comprised 6 spans, each approximately 45m long.
- Option 3: A vertical shaft from the elevated plateau to the toe of the slope. This shaft would then feed into a horizontal tunnel that then connects to the transfer pipeline at ground level.

Only Option 1 and 2 were assessed during the feasibility design stage. The large uncertainty in ground conditions and whether a road header or Tunnel Boring Machine (TBM) would be better suited limited option 3 design. Adopting option 3 would mean fewer transfer pipelines compared to option 1 and 2, and as such, this would have resulted in re-design of the transfer pipeline and powerhouse turbine system. Furthermore, construction costs were highly variable as the hill geology and excavation technique was largely unknown. However, it is likely option 3 will be feasible and cost-effective and as a result, this option will be explored further in the detailed design stage.

Option 1 had a longer span than option 2 which gave the benefit of reduced geotechnical investigations in subsequent design stages. However, the longer span resulted in significant stiffening of the pipes between supports which increased pipe procurement costs. Longer spans also resulted in larger axial loads and bending moments at the supports which resulted in increased pile lengths or additional piles. As a result, option 2 was selected the most cost-effective option.

The design surface level required earthworks along the hill slope to flatten the undulating terrain. It was anticipated fill of up to 8m would be required, particularly around the mid-span. Carrying out earthworks would have required construction access tracks across the slope which would have been difficult to construct due to the highly undulating terrain. Furthermore, the undulating terrain comprised of ridges and gullies which would have contained loose screen (cobbles and boulders) creating an unsafe environment to drive on. As a result, the perceived difficulty in constructing these access tracks dictated the type of recommended foundation support. Lowering of plant and machinery from the elevated plateau was considered a more feasible option in installing piles.

3.2.4 Foundation design

The ground conditions across the width of the pipes were assumed to be identical considering the three pipes spanned 10m wide centre to centre. The foundation design presented here represents one pipe alignment and can be replicated for the other two pipes.

Table 3 presents the foundation design ground model which adopts the higher bound layer thicknesses presented in Table 2. It is highly likely the design ground model is conservative which results in longer pile lengths and/or greater number of piles.

The adopted geotechnical design parameters presented in Table 4 are based on experience, conservative design assumptions and heavily reliant on published geological information. For a conservative design, the ground model ignored the Corraberra Sandstone unit.

Table 3: Foundation design ground model

<table>
<tr><th>Pier Location</th><th>Thickness of Fill (m)</th><th>Thickness of Colluvium (m)</th><th>Proterozoic Tent Hill Formation (m)</th></tr>
<tr><td>Pier 1 & 2</td><td>-</td><td>-</td><td>Outcrop at surface</td></tr>
<tr><td>Pier 3 & 4</td><td>5</td><td>10</td><td rowspan="4">Not established</td></tr>
<tr><td>Pier 5</td><td>8</td><td>10</td></tr>
<tr><td>Pier 6</td><td>7</td><td>10</td></tr>
<tr><td>Pier 7</td><td>-</td><td>10</td></tr>
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Table 4: Adopted geotechnical design parameters

Parameter	Fill	Colluvium	Proterozoic Tent Hill Formation
Unit Weight (γ’)	20	18	22
Friction Angle (Φ’)	30	30	40
Shear Strength (Su)	-	100	n/a
Poisson’s Ratio (ν)	0.5	0.5	0.2
Unconfined Compressive Strength (UCS)	n/a	n/a	< 40
Intact Youngs Modulus (Ei)	20	40	200
Skin friction – compression (fs) (kPa)	40	40	250
End bearing – compression (fb) (kPa)	n/a	n/a	3,000

A bored pile design and a micropile design were considered as a foundation system. Due to uncertainty in successfully and safely constructing access tracks along the hill slope, a micropile foundation system was adopted. The intention was to use a relatively light piling rig lowered from the elevated plateau using cranes to access each penstock support. Lowering large bored piling rigs or large excavators was considered unsafe. However, it is likely if option 2 is selected at detailed design, a construction access track will be designed and as such, bored piles will be better suited.

Both bored pile and micropile foundations were designed under axial (compression) loading only. Ultimate Limit State (ULS) loading was provided by the structural engineers. Thrust blocks were designed by the structural engineers under full lateral loading and uplift which was generated through pipe hydrostatics, surge pressures and wind loading.

The bored pile design was carried out using AS2159-2009 and Rowe and Armitage (1986). Pile diameters of 0.6m, 0.90m, 1.05m and 1.50m were considered. A final pile diameter of 1.05m and concrete strength f’c = 50 MPa was adopted to minimise excessive piling. It was assumed conventional drilling rigs would be adequate to drill through Proterozoic Tent Hill Formation assuming no hole collapse in the overlying soils. The bored piles were designed to be rock socketed into the Proterozoic Tent Hill Formation, which was assigned Class III and Class IV HW Shale foundation design parameters in accordance with Pells at al, (1998). For a preliminary design, a geotechnical strength reduction factor $\varphi_g = 0.45$ was adopted in accordance with AS2159-2009 as it was assumed no pile testing will be carried out. The contribution to shaft friction by the engineered fill and top 2m of colluvium was discounted.

The applied structural loads resulted in multiple bored piles acting in a group. A minimum spacing of one pile diameter + 0.3m was recommended for bored pile block design. This resulted in a group efficiency factor $\eta = 1.0$.

Table 5: Design output for bored piles

Pier Location	ULS Loading (kN)	Min. rock socket[1]	Total pile length (m)	Factored single pile capacity (kN)	No. of piles in group
Pier 1	5,500	9	9	≈ 3025	2
Pier 2	11,100	9	9	≈ 3025	4
Pier 3 & 4	11,100	5	20	≈ 3188	4
Pier 5 & 6	11,100	5	23	≈ 3247	4
Pier 7	5,500	5	15	≈ 3069	2

The micropile design was based on FHWA (2005) guidelines. The micropiles were designed to be fully cased in soil and uncased in rock. A structural steel casing of outer diameter 273.1mm, wall thickness 16mm and yield strength fy = 241 MPa was used. A grout strength f'c = 25 MPa was adopted. The failure of the steel casing due to grout failure was calculated as 600 MPa when adopting a steel young's modulus E = 200 GPa. A 510mm^2 single rebar was used with a design yield strength fy = 420 MPa.

The governing structural capacity of the micropile was calculated as the minimum of the structural casing yield strength (fy = 241 MPa), grout failure (fy = 600 MPa) and rebar yield strength (fy = 420 MPa). The yield strength of the structural casing was found to be governing and as a result the allowable compression of the micropile ($P_{C\text{-}Allowable}$) was based on a yield strength fy = 241 MPa. Chemically passive grout was recommended as it was assumed ground conditions were non-aggressive. As such, no reduction in casing diameter was allowed for due to corrosion.

The following equation was used to calculate the allowable micropile compression ($P_{C\text{-}Allowable}$)

$$P_{c-allowable} = [0.4f'_{c-grout}\ x\ A_{grout} + 0.47F_{y-steel}(A_{bar} +\ A_{casing)}] \quad (1)$$

The allowable geotechnical bond capacity $P_{G\text{-}allowable}$ was calculated using the following equation and was re-arranged to determine the bond length L_b

$$P_{G-Allowable} = \frac{\propto_{bond}}{FS} x\ \pi\ x\ D_b\ x\ L_b \quad (2)$$

$$L_b = \frac{P_{G-allowable}\ x\ FS}{\propto_{bond}\ \ x\ \pi\ x\ D_b\ x\ L_b} \quad (3)$$

The bond length within the rock was calculated by setting the allowable geotechnical bond capacity, $P_{G\text{-}Allowable}$ to equal the allowable micropile compression capacity, $P_{C\text{-}Allowable}$. A grout to rock ultimate bond strength α_{bond} = 385 kPa for HW Shale and a Factor of Safety, FoS = 2.0 were adopted in accordance with FHWA (2015). This resulted in an average uncased bond length, L_b = 12m in the Proterozoic Tent Hill Formation. A micropile capacity of 1975 kN was calculated using equation 1 and 2. Table 6 presents the design micropile lengths. Figures 4 and 5 show a typical cross section of the micropiles and penstock arrangement.

Table 6: Design output for micropiles

Pier Location	ULS Loading (kN)	Min. rock socket (m)	Total micropile length (m)	No. of micropiles in group
Pier 1	5,500	12	12	3
Pier 2	11,100	12	12	6
Pier 3 & 4	11,100	12	27	6
Pier 5 & 6	11,100	12	30	6
Pier 7	5,500	12	22	3

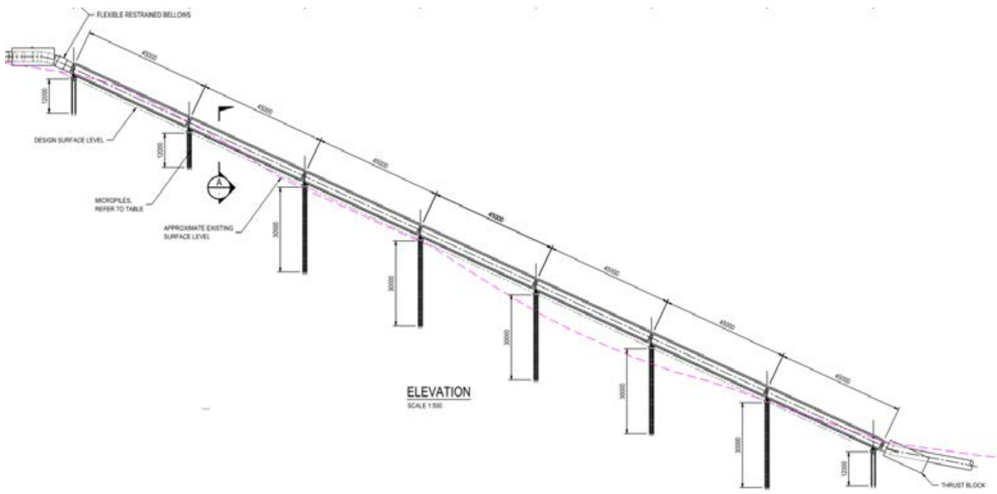


Figure 4: Penstock layout across 6 spans

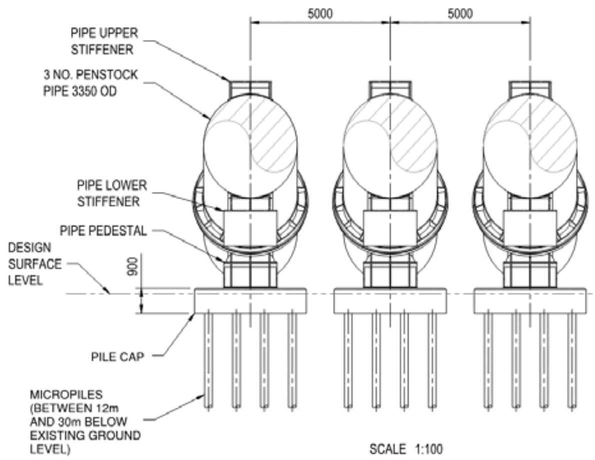


Figure 5: Typical cross section of the penstock foundation system

3.3 TRANSFER PIPELINE

3.3.1 Location

The transfer pipeline is located within the DoD and Crown land and spans from the toe of the hill slope to the powerhouse near the shoreline.

3.3.2 Ground conditions

The ground conditions along the transfer pipeline are likely to be variable and were based on the test pit data collected and published geological maps. The design ground conditions are presented in Table 7.

It is highly likely the “Whyalla Sandstone” layer noted in the table above is a caprock layer or a cemented bed and that the Pooraka Formation is likely to extent beneath this layer. Excavation refused on the caprock layer / cemented bed as a result is it difficult to confirm the extent of soils until borehole drilling or larger excavation machinery is used in the detailed geotechnical investigation.

Table 7: Summary of ground conditions – Transfer Pipeline

Geological Unit	Description	Inferred consistency / strength	Depth to top of unit (m)	Thickness of unit (m)
Topsoil	Silty Clay and Clayey Silt	Soft to Firm	0m	0.1-0.3m
Pooraka Formation - Aeolian and Alluvial Soils	Silty Clay, Clayey Silt and Clayey Sand, with Ironstone Gravels	Firm to Hard	0.1-0.3m	0.5-1.0m
Weathered Whyalla Sandstone – Residual Soils	Silty Clay and Clayey Gravel, trace Ironstone Gravels	Stiff to V. Stiff	0.6-1.3m	Up to 1.5m
Whyalla Sandstone (Inferred)	Fine to coarse grained	V. low to Medium Strength	0.5-1m (Western half) 2-5m (Eastern half)	Not Established

3.3.3 Concept design

The proposed transfer pipeline is to be located a nominal 5.35m below ground level using conventional trenched excavations. Recommended backfill material was crushed rock and re-worked in-situ clay borrowed from site excavations. A 1.0m thick layer of compacted clay overlying a 0.5m thick crushed rock layer was adequate cover to resist the largest surface loads comprising army vehicles.

DoD required minimal ground disturbance as such a shoring box with internal props were used in areas where the excavations did not meet the OHS (2004) and OHS (2017) guidelines. This allowed personnel entry into the excavations for pipe lowering and fitting connections. In addition, temporary shoring and propping reduced spoil volumes and made the design more cost effective.

Considering the presence of a hard caprock or cemented layer, the construction costs allowed for the use of a D10 or D11 bulldozer and rock blasting and the use of excavators with hydraulic rock breakers. It was assumed the detailed geotechnical investigation will identify the most appropriate excavation technique.

An alternative concept involved installing the transfer pipeline through a tunnel. This option may prove to be cost effective if tunnelling equipment is mobilised for the construction of the horizontal tunnel instead of the penstock however due to the lack of comprehensive geotechnical investigation data, the tunnelling option was difficult to price accurately.

Figure 5 shows the design cross section of the transfer pipeline.

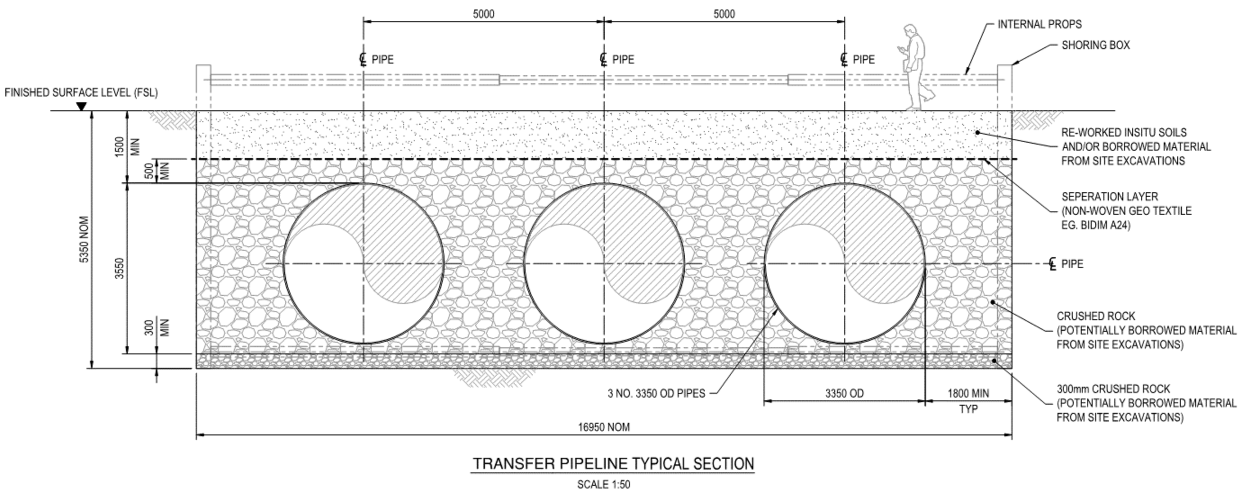


Figure 6: Typical cross section of pipeline excavation

3.4 POWERHOUSE AND WATER INTAKE / OUTLET STRUCTURE

3.4.1 Location and overview

The powerhouse is located within the Crown land approximately 150m inland from the shoreline. The powerhouse was designed as a bunker type structure that houses the pump turbines, bridge crane, ancillary support systems, electrical equipment and operators. The powerhouse connects to the waterway via a water intake / outlet structure. The inland section of the powerhouse connects to the transfer pipeline that leads up to the penstock. The intake / outlet structure consists of metal screens to protect marine fauna entering the powerhouse development.

3.4.2 Ground conditions

The ground conditions at the powerhouse are based on two boreholes drilled during the preliminary geotechnical investigation and published geological maps. A summary of the design ground conditions are presented in Table 8 below.

Table 8: Summary of ground conditions – Powerhouse

Geological Unit	Description	Inferred consistency / strength	Depth to top of unit (m)	RL to top of unit (m AHD)	Thickness of unit (m)
St Kilda Formation – Aeolian, Alluvial and Estuarine Sediments	Sands, Clayey Sands, Sandy Gravels, Sandy Clay	Loose to medium dense, dense to v. dense gravel	0.0m	+4m AHD to +8m AHD	Approx. 10m
Pooraka Formation – Alluvial Soils	Sandy Clay	Stiff to V. Stiff	0m to 10m	-2m AHD to -6m AHD	3-4m
Hindmarsh Clay – Alluvial Soils	Clay, with beds and lenses of sands and gravels	Stiff to V. Stiff	13-14m	-6m AHD to -10m AHD	23-24m
Whyalla Sandstone	Fine to coarse grained	Medium to V. High Strength	37m	-29m AHD to -33m AHD	Not established

The gravels encountered within the St. Kilda formation are likely to be caprock or cemented beds similarly to within the DoD land.

Standing water was encountered between 1m and 2m BGL however it was not clear whether this was the groundwater level or a confined aquifer. For a conservative design, the encountered standing water was assumed to be the groundwater level at high tide.

3.4.3 Concept design and geometry

The design for the powerhouse retention system was based on similar sea water pumped hydro projects carried out in other parts of the world. The geometry of the powerhouse was influenced by the mechanical design of the turbines. A 40m deep structural retention system was required to house three turbines. Each turbine connected to one transfer pipeline that led to the elevated reservoir.

On the landward side, the transfer pipeline connects to the powerhouse at RL -25m AHD and on the shore side, a tail race swan neck connects the powerhouse to the intake/outlet screen. The invert of the tail race swan neck is at RL -34.2m AHD. The toe of the powerhouse excavation is located at RL -30.6m AHD (≈ 37m deep) and allows for surface earthworks to match grade with support structures.

An allowance was made for the top of Whyalla Sandstone being encountered at RL -29m AHD and, as such, the powerhouse excavation is likely to extend into the Whyalla Sandstone by 1.6m. For preliminary design of the excavation retention system, a retaining wall embedment of 3m into the Whyalla Sandstone was allowed for. It was understood, this detail will be refined in the detailed design stage.

A tanked retention system was designed considering the shallow groundwater level and proximity of the excavation to the shoreline. Permanent (long-term) dewatering was ruled out as an option due to environmental concerns around discharging water, costs and maintenance associated with dewatering and the potential ground movements caused by dewatering particularly in the surficial sands.

An open battered excavation was not possible due to the extensive dewatering required and the high likelihood of slope failures resulting in an unsafe work environment for site personnel. There were also concerns that the batter slopes would encroach into environmentally sensitive areas around the nearshore and that protecting marine fauna entering the excavation during construction would be a challenge.

Three permanent retention systems were considered for the powerhouse excavation. These are outlined below:

- Option 1: Secant piled wall with shotcrete infill panels. One major limitation of this retention system was the difficulty to maintain verticality in pile lengths in excess of 25m. This system would have required permanent anchors, and its access and long-monitoring would be difficult and costly. However, there was an option to use robust electrical load cells and long-term monitoring would be dictated by the relevant anchor installation code used and / or project specifications. Furthermore, lateral pressures developed by a 37m deep excavation would be significant and such a retention system would have required significantly long permanent anchors bonded in Whyalla Sandstone. Additional measures such as using internal props and struts would be required to counter large lateral pressures. Creating a permanent seal / water-proof retention system on the secant piled wall is difficult at depths greater than 25m without the use of a water proofing barrier such as shotcrete, which would not have been suitable to resist hydrostatic pressures at such depth.
- Option 2: Rectangular or circular diaphragm wall. A diaphragm wall was assessed as the most appropriate solution considering it can be designed as a water tight, permanent structural retention system. A vertical diaphragm wall would have eliminated the need for open excavations and verticality would be better controlled than a secant piled solution. Prior to preliminary design, an internal risk analysis was carried out and it was concluded the mitigation of risk across its entire design life justified the high cost. The circular cross section would result in the diaphragm wall panels acting in a hoop system where they resolve lateral forces in compression. This would have minimised or eliminated the requirement of permanent anchors. However, one of the major limitations of the circular diaphragm wall was achieving a large enough diameter to house three turbines. Constructing three separate diaphragm walls could be possible however interaction between the three was a concern along with increase in cost. The rectangular diaphragm wall offered a better solution of accommodating all three turbines however the geometry of the rectangular structure would have required significantly large permanent rock anchors to tie back the diaphragm wall panels. The use of permanent anchors posed the same risk as indicated in option 1 above.
- Option 3: "Peanut" shaped diaphragm wall. This design was based off a successful "peanut" shaped diaphragm wall constructed for the MTR Shatin to Central Link project in Hong Kong (MTR, 2017). A 40m deep diaphragm wall was constructed at Fenwick Pier for the launching of a Tunnel Boring Machine (TBM), construction of permanent egress points, construction of permanent cut & cover tunnels and construction of a permanent railway. In this project, a "peanut" shaped shaft design was adopted due to space constraints caused by an existing 10m deep Intake Cell basement, an existing operating rail line and an existing old seawall. The shaft comprised of a 24m dia. twin circular shaft with two middle cross beams acting as a permanent "strut". The lack of a conventional permanent strut saved the project approximately 2,000 tonnes of steel work. There were no interface issues between strut installation and bulk excavation works, and consequently, no interface issues between strut removal and permanent construction works. The "peanut" geometry provided additional working room for the TBM gantry and design analysis showed ground settlement was reduced by approximately 60% compared to a traditional rectangular shaft. These benefits were replicated for the Cultana SPHES project. The MTR Shatin to Central Link project also made a comparison between a traditional rectangular shaft and a "peanut" shaped shaft and found the "peanut" shaped shaft better fit for purpose. The "peanut" shaped diaphragm wall comprised of two cross walls that acted as a permanent strut providing lateral resistance. Furthermore, allowance for a circular cross section and cross walls eliminated the need for permanent anchors. The "peanut" shaped diaphragm wall also helped reduce the wall area compared to the three circular diaphragm walls and the rectangular diaphragm wall presented in option 2, thereby saving on construction costs.

3.4.4 Diaphragm wall model

The geotechnical design of the "peanut" shaped diaphragm wall was carried out using *PLAXIS 2D*. The model type adopted was axisymmetric which was taken to model the support provided by the cross walls and therefore eliminate the need for permanent ground anchors. The design ground conditions are presented in Table 9 below. The axisymmetric model was used to reflect the distribution of hoop compression forces along the circular members.

The first phase of the construction sequence comprised installation of the 40m deep diaphragm wall. The diaphragm wall was modelled as a plate element. Considering the model adopted was axisymmetric, only half the diameter of the circular shaft was modelled. The diaphragm wall was modelled as unreinforced (fully concrete). The resulting maximum bending moments were provided to the structural designers for reinforcement cage design. In reality, such a diaphragm wall would be heavily reinforced. It was assumed the diaphragm wall installation would be segmental and expandable water-stop joints will be included to prevent groundwater entry in the long-term case. A diaphragm wall panel width of 1m and panel thickness of 0.8m was adopted based on discussions with ground improvement specialist contractors and after a few iterations using *PlAXIS 2D.*

Following installation of the diaphragm wall, the 37m deep excavation was modelled in stages. The excavation terminated on the Whyalla Sandstone layer. The diaphragm wall extended a further 3m into the Whyalla Sandstone. The staged excavation and 3m embedment limited deformations of the diaphragm wall.

Temporary dewatering spears were installed in rows along the diaphragm wall as excavation proceeded. A six-month excavation program was anticipated and, as such, dewatering within this period was considered feasible and economically viable. The groundwater conditions for each lift excavated and the lift below were modelled dry and the two lifts directly underlying were interpolated between dry and global groundwater conditions. It was assumed dewatering would be effective enough to mimic this design scenario. An alternative option considered but not modelled in *PLAXIS 2D* was the use of a rectangular water cut-off barrier such as a cement-bentonite wall or a soil-bentonite wall installed around the perimeter of the diaphragm wall.

Following bulk excavation, a 2m thick reinforced concrete slab was installed at the base. Temporary dewatering was anticipated to continue for another 6 months until construction of the powerhouse structure was complete. The *PLAXIS 2D* output indicated the load from the building and the reinforced concrete slab were adequate in preventing uplift caused by upward groundwater pressures.

Table 9: *PLAXIS 2D Input*

Geological Unit	Depth from (m)	Depth to (m)	Model	γ' – kN/m^3	Φ' - degrees	Su - kPa	ν'	E' - MPa
St Kilda Formation	0m	12m	Drained	18	34	-	0.25	20
Pooraka Formation	12m	22m	Undrained	18	-	50	0.50	20
Hindmarsh Clay	22m	37m	Undrained	20	-	100	0.50	40
Whyalla Sandstone	37m	70m (assume)	Drained	22	45	20	0.20	200

The following diaphragm wall properties were used in *PLAXIS 2D*:

- Model type: Axisymmetric
- Model: Plate element
- Material type: Elastic
- End Bearing: No
- Panel Width: 1m
- Panel Thickness: 0.8m
- Depth of panel: 40m
- 28-day Concrete f'c: 32 MPa
- Concrete Youngs Modulus: 30,100 MPa
- EA: 24×10^6 kN/m
- EI: 1.3×10^6 kPa/m
- Excavation dimensions: 37m deep x 15m wide (half width)

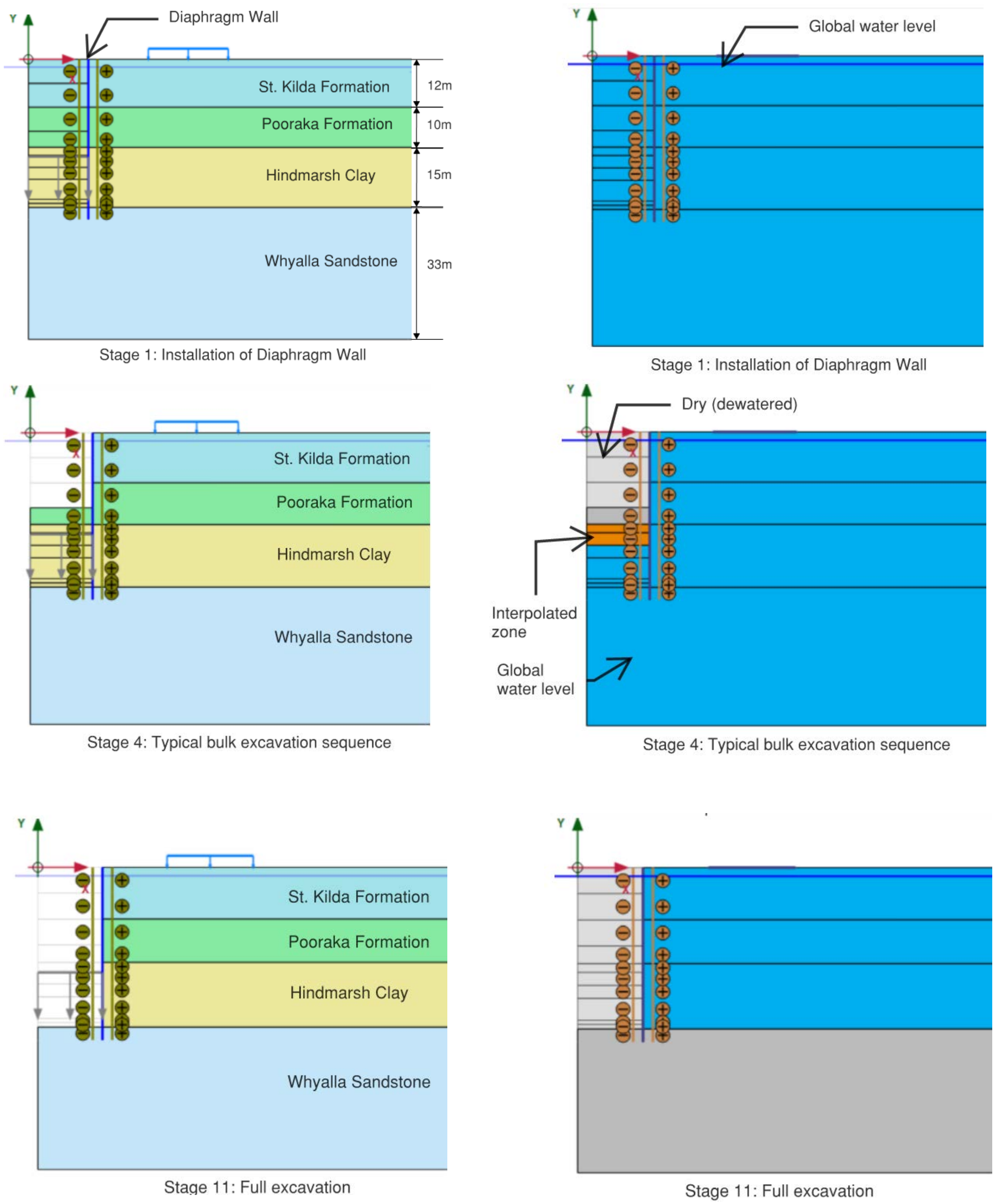
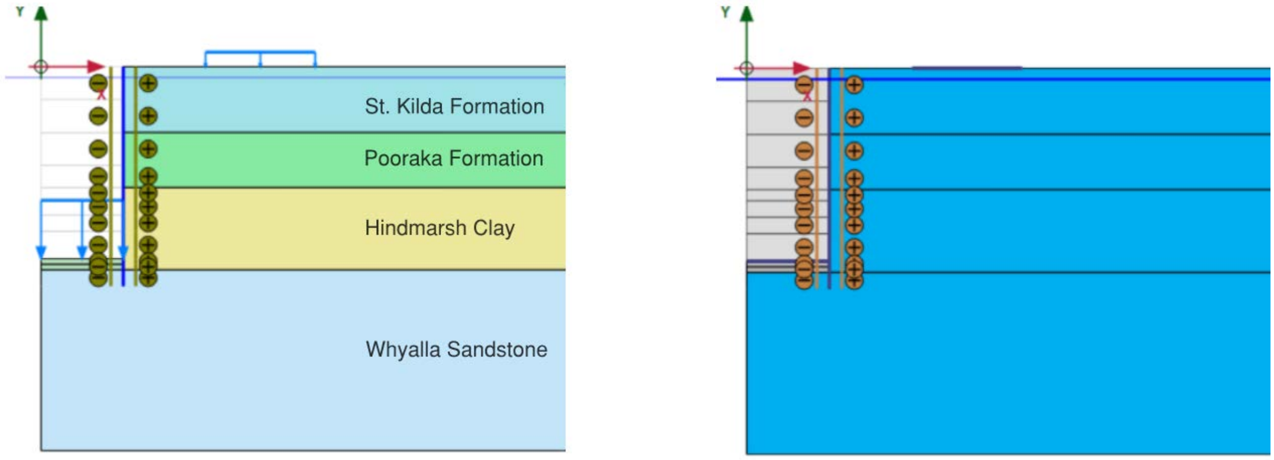


Figure 7: *PLAXIS 2D* construction staging and flow conditions



Stage 12: Installation of concrete base slab and construction of powerhouse structure

Stage 12: Installation of concrete base slab and construction of powerhouse structure

Figure 7: *PLAXIS 2D* construction staging and flow conditions *(continued)*

Post-construction total displacements

Post-construction bending moments

Post-construction axial forces

Post-construction hoop forces

Figure 8: *PLAXIS 2D* output

3.4.5 Intake / outlet structure

A preliminary design of the intake/outlet retention system was not carried out as part of this feasibility study. Discussions were held with ground improvement specialist contractors on possible solutions.

The depth of the intake / outlet retaining structure ranged from 37m (where it connects to the powerhouse structure) to approximately 10m (point where dredging commences). Considering a permanent retention system was required, permanent diaphragm walls were designed as a retention system.

The intake / outlet structures comprise marine screens which would be held on either side by diaphragm wall panels. Its installation would comprise construction of a full-length diaphragm wall followed by localised breakthrough of the diaphragm wall to install marine screens.

An internal risk analysis was carried out and the decision to use diaphragm walls was attributed to its flexibility and cost effectiveness considering diaphragm wall equipment was already mobilised for the powerhouse retention system. The major objective of the marine screen diaphragm wall was to provide a structural retention rather than water tightness.

3.4.6 Connection to transfer pipeline

A preliminary design of the transfer pipeline connection was not carried out as part of this feasibility study. Internal discussions were held with senior civil and geotechnical engineers and specialist contractors on possible solutions.

Connecting the pipeline to the powerhouse on the inland side would have required a pipe jacking method or tunnelling considering the deepest section of this connection is at 37m depth. A tunnelling method was considered ideal if tunnelling machinery was mobilised to construct a horizontal tunnel through the hill. The presence of the caprock or cemented bed layer could have created some difficulty with pipe jacking.

Bulk earthworks with a temporary retention system was proposed if pipe jacking or tunnelling was not adopted. A soldier piled wall with shotcrete infill panels, temporary anchors and drains were proposed where retaining heights were less than 15m. Secant piles with shotcrete and temporary anchors were adopted for retaining heights between 15m and 25m and diaphragm wall panels at depth in excess of 25m. Construction of a cement-bentonite wall or a soil-bentonite wall was proposed around the outer perimeter of this retention system to divert and minimise groundwater flow into the excavation. These retention systems were design as temporary structures considering the excavation would be backfilled once the transfer pipeline had been laid.

Pipe jacking was adopted at concept stage due to its relative cost effectiveness compared to the tunnelling and excavation retention option. This option was heavily discussed with a pipe jacking specialist contractor. However, there was a risk in the pipe jacking method intersecting the caprock or cemented bed layer. It was recommended the preferred installation method be decided after completion of the detailed geotechnical investigation so the caprock or cemented bed layer could be better defined.

Figures 9 to 12 below show typical details of the powerhouse and intake / outlet structure and a concept model of the "peanut" shaped diaphragm wall.

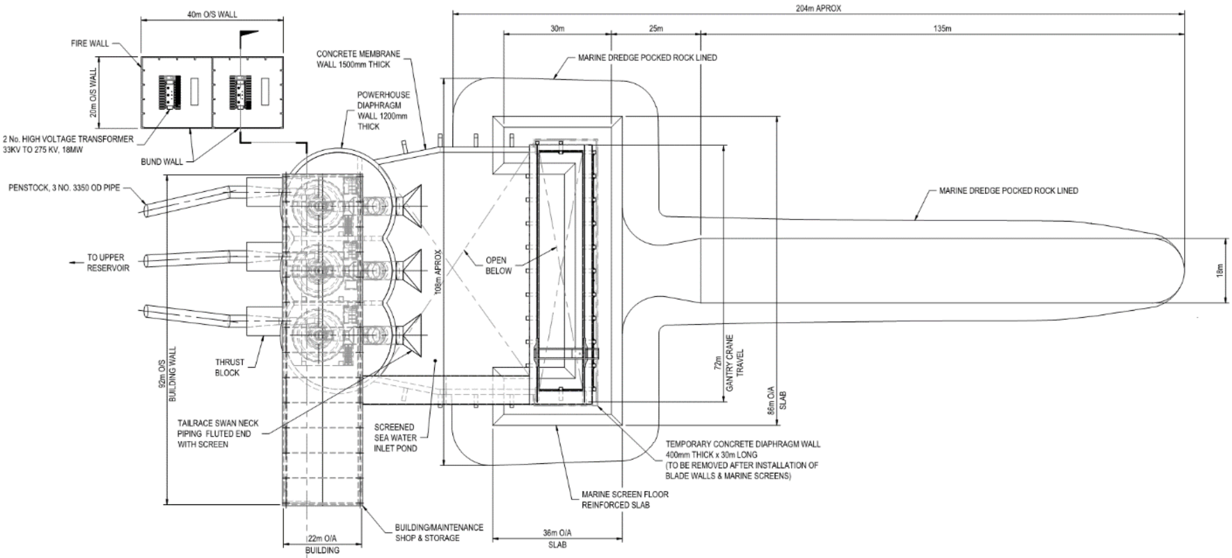


Figure 9: Plan view of the powerhouse and intake / outlet structure

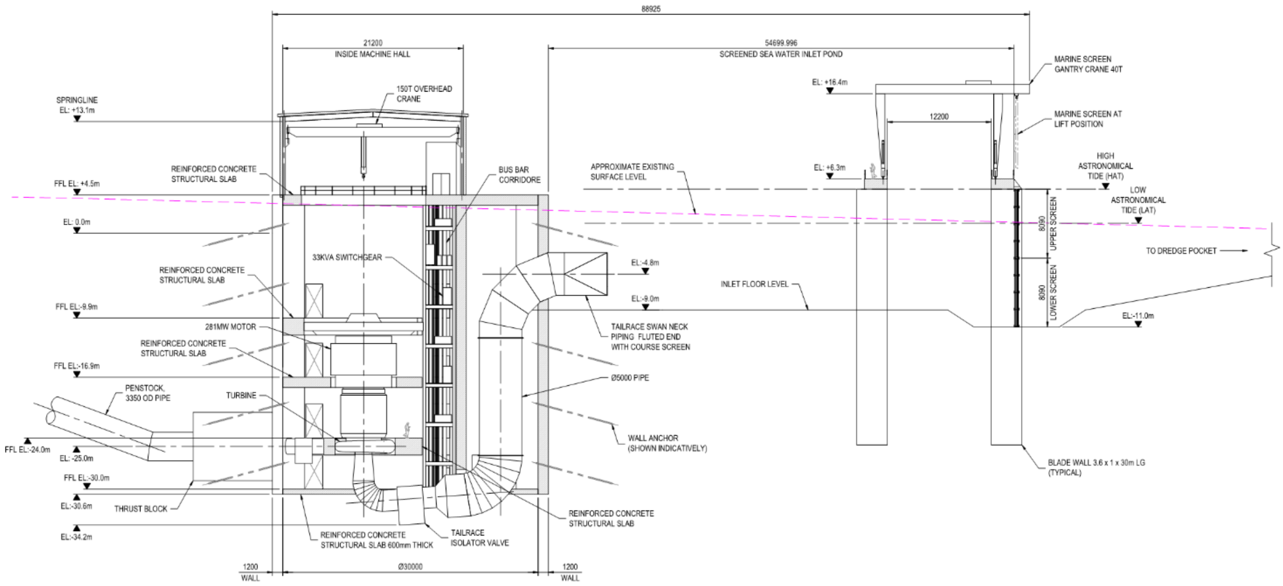


Figure 10: Cross sectional view of the powerhouse and intake / outlet structure

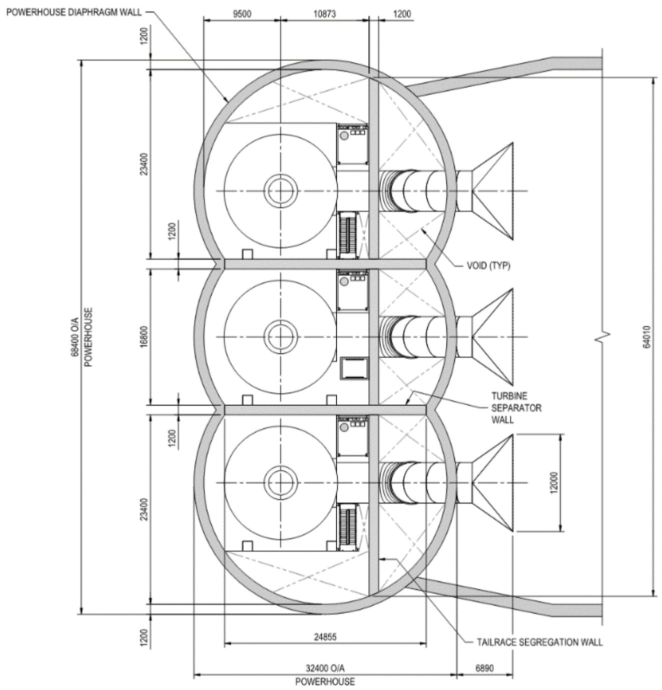


Figure 11: Plan view of the concept "peanut" shaped diaphragm wall

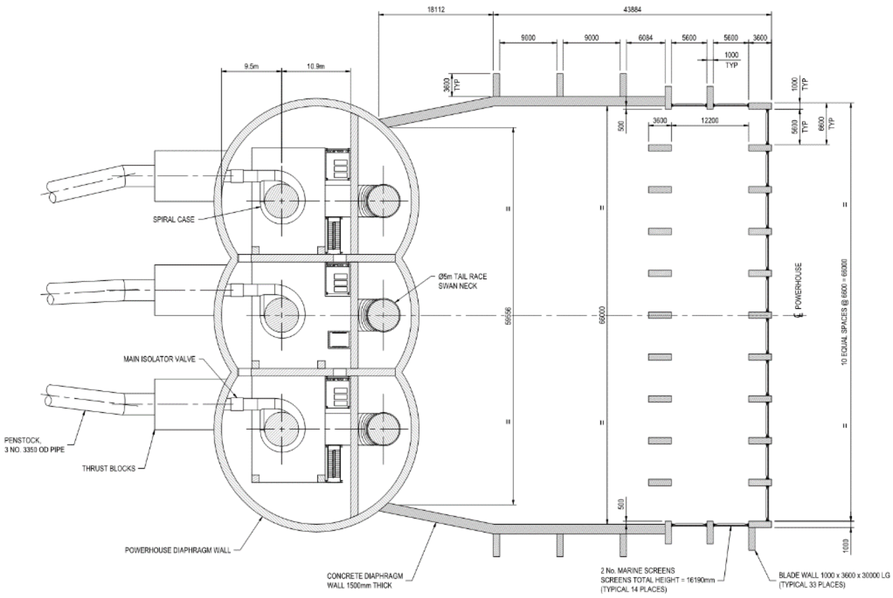


Figure 12: Plan view of the concept "peanut" shaped diaphragm and intake/outlet structure interface

4 CONCLUSION

The Cultana SPHES project comprises the construction of a pumped hydro scheme capable of producing between 100MW and 250MW of dispatchable peak electricity to the NEM grid in South Australia. From a geotechnical perspective, four major structures have been identified, these are the powerhouse and water intake / outlet structure, the transfer pipeline, the penstock pipeline and the reservoir located on an elevated plateau. The site has highly variable ground conditions which ranges from deep Aeolian and Alluvial sands and gravels near shore to shallow bedrock at the evaluated plateau. As such, each of these four structures have been identified as located in different ground conditions.

The feasibility phase of this project comprised providing high level, viable preliminary design options for each of the four structures. Each concept option has been based on construction feasibility, anticipated cost effectiveness, safety in design, long-term maintenance requirements, impacts on marine fauna and overall project risk.

The reservoir walls comprised of a crushed sandstone face with a clay core. A concrete wall was designed at the reservoir and pipeline interface due to high surge loads. The reservoir walls are anticipated to be between 7m and 15m high with maximum reservoir depth of 15m. The reservoir will be constructed largely above ground with excavations up to 3m below ground level anticipated. The governing design criteria for the reservoir were the stability of the embankment and its permeability, particularly in areas where the crushed sandstone face is subject to rapid water drawdown.

The penstock pipeline comprised a 6-span bridge with equally spaced piers. The pipeline rests directly on the piers and the piers are supported by micropiles up to 30m deep. A micropile design was adopted due to uncertainty of safe access along the hill slope. The intention was to use a crane to lower down a light micropiling rig at pier locations. A bored pile solution was preferred if a maintenance track was proposed along the hill slope. A vertical shaft below the reservoir leading to a horizontal tunnel at the toe of the slope has been considered as an alternative, however its costs are considered excessively high.

The transfer pipeline connects the penstock to the powerhouse structure. Preliminary design has allowed for this 3.4km long pipeline to be installed to a maximum depth of 5.35m below ground level. Bulk excavations will be carried out using D10 / D11 bulldozers with rock blasting or hydraulic rock breakers. The Department of Defence required minimal ground disturbance, as such the intention was to excavate vertically and install temporary shoring boxes and internal props to enable personnel entry.

The preliminary design of the powerhouse structure comprised bulk excavations up to 37m below ground level. The powerhouse is located approximately 150m inland from the shoreline with standing groundwater level between 1m and 2m below ground level. A diaphragm wall design was adopted for preliminary design considering a permanent, structural retention system was required. A "peanut" shaped diaphragm wall was used taking advantage of the distribution of hoop compressive forces. Two full length cross walls were installed which eliminated the need for permanent anchors.

The intake / outtake structure was located near shore and required dredging works to allow water entry into the powerhouse. Large screens were designed to prevent marine fauna entry into the powerhouse. The intention was to construct a full-length diaphragm wall followed by local breakthrough to install the marine screens. Diaphragm walls were adopted for the intake / outtake structure considering 10m to 37m deep permanent retention was required.

The transfer pipeline connected to the powerhouse on the inland side. A soil-bentonite or cement-bentonite water cut-off wall was installed to minimise water entry into the pipeline and powerhouse connection. Pipe jacking was used for installation and connection of the transfer pipeline to the powerhouse based on internal discussions and advice provided by a specialist pipe jacking contractor. Other options such as soldier piles with shotcrete panels and secant piling were considered if pipe jacking was not suitable and bulk excavations had to be carried out.

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