Detailed Design and Energy Calculations of Snowy 2.0 Project

Executive Summary

This report advances an investment-ready 2,000 MW pumped-hydro scheme linking Tantangara and Talbingo to the point where detailed design and procurement can proceed with confidence. The aim is explicit and measurable: store surplus renewable energy at low prices, then provide the stored energy back to the grid during peak demand, or when electricity supply is low.

The layout includes six reversible Francis turbines in an underground cavern. A long headrace feeding a 25° unlined inclined pressure shaft, splitting into six penstocks and a graded tailrace to Talbingo. We utilise about 650 m of gross head and existing reservoirs to curb greenfield risk while retaining deep storage. Upsizing the principal waterway from 7.7 m to 9.0 m holds velocities near 5.5 m/s, trimming losses and improving controllability under dynamic dispatch.

Safe excavation and pressurisation are determined by measured rock mass conditions. The alignment skirts the Long Plain Fault Zone. Confinement checks for the unlined shaft against Norwegian criteria return factors of safety above 1.5, contingent on disciplined mapping, grouting, and adaptive support classes during advance.

Hydraulics are quantified end to end. Effective turbine head lands in the mid 500 m range at design flow. Total discharge is roughly 350 m³/s, about 56 to 59 m³/s per unit. Active storage near 240 GL enables multi-day operation at nameplate and genuine scheduling flexibility.

The project's reversible Francis turbines are expected to achieve turbine efficiency near 89 percent and overall efficiency around 85 percent. Round-trip efficiency is assessed at roughly 78%. Head fluctuation remains within acceptable limits for long-life Francis operation.

Construction will be sequenced to suit underground work, with units coming online in stages and first operation targeted for 2029. An engineering, procurement and construction management approach keeps scope flexible as conditions change. A mix of public and private funding maintains control while reflecting the wider benefits of large-scale storage. The full system value depends on new transmission being ready, especially the HumeLink project, which needs to be in place before energisation and commercial operation.

Key risks are identified with clear responses. Uncertain ground conditions are addressed through focused site investigations and adaptable support during excavation. Pressure surges are managed by sizing the surge tanks correctly and confirming performance with modelling. Exposure to price swings is limited by operating the plant flexibly to capture value rather than always chasing nameplate output. Overall, the project is feasible and useful to the National Electricity Market. Immediate tasks are to complete the transient and fatigue checks, finalise surge tank and valve specifications, plan construction near the Long Plain Fault Zone carefully, and keep transmission delivery aligned with plant readiness.

1. Management & Integration of Snowy 2.0 by General Manager (20%)

1.1 Site Context and Initial Assumptions

Snowy 2.0 is a 2,000 MW pumped hydro energy storage (PHES) facility to connect the Tantangara Reservoir (upper) and the Talbingo Reservoir (lower) in the Snowy Mountains Scheme. The site was selected because of the substantial elevation difference 682.3 existing water storage infrastructure, and strategic location within the National Electricity Market (NEM).

Upper Reservoir: Tantangara (FSL 1,228 m).

- **Lower Reservoir:** Talbingo (FSL 543 m).
- **Design discharge:** ~350 m³/s
- **Active storage volume:** ~240 GL.

During off-peak periods, water is pumped from Talbingo (lower) to Tantangara (upper). During peak demand, water flows downhill via a headrace tunnel, pressure shaft, and six penstocks, driving reversible Francis turbines before discharging into Talbingo. See **Error! Reference source not found.** for the Snowy Hydro Concept.

Figure 1: Snowy Hydro 2.0 Concept

1.2 Financing Models & Contract Frameworks

One of the key difficulties for large-scale PHES projects is reconciling their expensive initial capital and uncertainty of subsurface works. Snowy 2.0 was financed through a hybrid scheme of a AUD $1.38 billion Commonwealth Government equity investment supplemented by Snowy Hydro Limited retained earnings. This approach preserved government ownership of a project of national importance without loss of operating control.

The contractual model employed was EPCM (Engineering, Procurement and Construction Management) rather than the standard fixed-price EPC (Engineering, Procurement and Construction). The EPCM path was employed because it offers greater flexibility during the building process, allowing design amendments and adaptive procurement where unforeseen ground conditions arise. Geotechnical problems in relation to tunnelling through high-strength granite or excavating the 1.45 km inclined pressure shaft posed risky extremes, so the employment of EPCM was advisable.

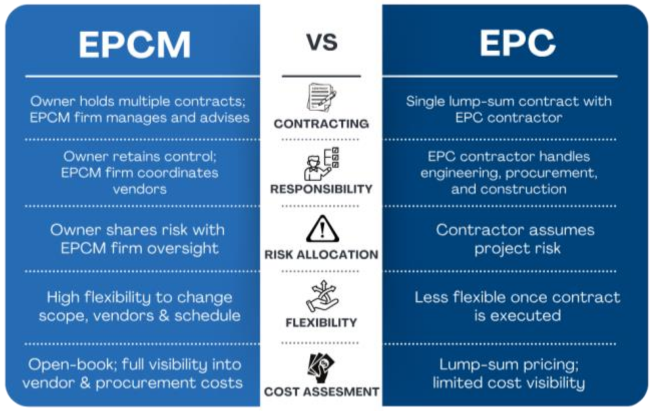

Figure 2: EPC vs EPCM diagram

1.3 Project Timeline & Milestones

The Snowy 2.0 project is Australia's most ambitious project of infrastructure development, aimed at expanding the renewable energy reserves in the country through pumped hydro energy storage (PHES). The project may be divided into several distinct stages, which include crucial design, construction, and commissioning aspects. The below is a step-by-step description of the project timeline and major milestones.

Feasibility & Environmental Studies (2017-2019)

The project initiated with a thorough feasibility study and environmental studies. These were conducted from 2017 to 2019 to analyse the technical, environmental, and economic viability of the project.

- Environmental Impact Assessments (EIA) were completed in 2019 to make sure that the project was following Australia's stringent environmental standards.
- Preliminary technical studies were conducted on site selection, reservoir analysis, and geotechnical studies.

Construction Phase (2019-2028)

Construction of Snowy 2.0 was planned carefully to install all the elements efficiently and safely.

- 2019-2020: Initial work was focused on prioritizing access tunnels, geotechnical investigation, and preparation for civil excavation. Early tunnel construction was planned to create the infrastructure required for main excavation activities.
- 2020-2023: Major tunnel works began, including the installation of Tunnel Boring Machines (TBMs) for headrace, tailrace, and pressure shafts. Cavern excavation of the power station, surge tanks, and penstock development also belonged to this phase.
- 2023-2025: The project continued with the completion of more than 50% of the underground power station cavern. This phase also witnessed the initiation of turbine and generator installations, in addition to ongoing work on auxiliary structures like transformers and valves.

Commissioning & Testing (2028-2029)

Commissioning will see the turbines and related infrastructure installed and commissioned sequentially. All the turbines will be online sequentially, allowing for testing, optimization, and connection to the National Electricity Market (NEM).

- 2028-2029: Snowy 2.0 must be up and running by 2029, with all turbines and systems running and connected into the grid, providing the necessary storage capacity to allow renewable energy production.

- Ongoing Operations (2029 and beyond)

Once it is in operation, Snowy 2.0 will store energy for more than 70 years, facilitating Australia's transition to a low-carbon future. The plant will be maintained, monitored, and optimized continuously to guarantee long-term operation and efficiency.

Milestone	Date
Feasibility & Environmental Studies	2017-2019
Construction Phase (Early Works)	2019-2020
Major Tunneling & Excavation	2020-2023
Power Station Cavern Completion	2023-2025
Commissioning & Full Operation	2028-2029
Ongoing Operations (Design Life)	2029 and beyond

Figure 3: Gantt-style timeline

1.4 Integration into National Electricity Market (NEM)

Snowy 2.0's strategic rationale stems from its position in the National Electricity Market of the future. In the Australian Energy Market Operator's (AEMO) Integrated System Plan, the "Step Change" scenario estimates renewable penetration at greater than 80% by 2040. To accommodate this transition, there needs to be large-scale, dispatchable storage. Snowy 2.0 fits directly into the process of making this transition by firming up renewable output, saving surplus solar and wind generation in low-demand periods and delivering it in peak demand periods.

In addition to energy arbitrage, Snowy 2.0 also provides valuable system services like synchronous inertia, frequency control, and reactive support that all stabilize an increasingly inverter-dominated grid. The project depends significantly on transmission enhancement, especially the Hume Link 500 kV line that will connect Snowy 2.0 with major demand hubs in Sydney and Melbourne. Independent market modelling has demonstrated that Snowy 2.0 is the lowest-cost large-scale storage asset on the NEM due to its size, central location, and long design life.

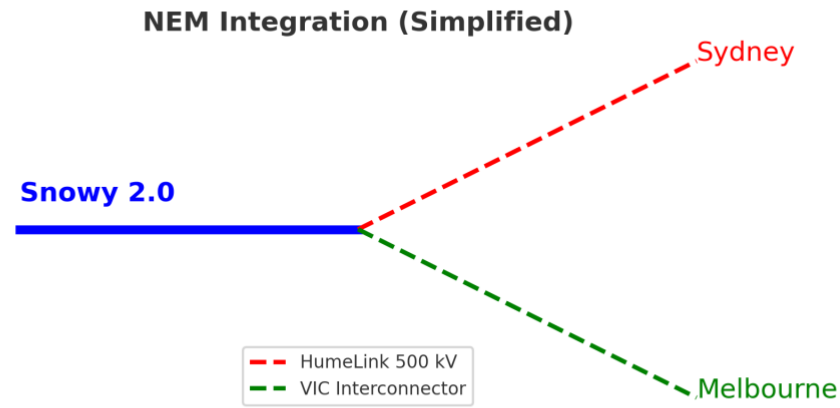

Figure 4: Simplified schematic showing Snowy 2.0's integration into the NEM via HumeLink to Sydney and Melbourne.

Figure 4: NEM Integration

1.5 Benchmarking – Snowy 2.0 Feasibility Study

Benchmarking against both Snowy 2.0 feasibility study and world's best practice international PHES projects is required to confirm the scheme's viability. The most significant technical and performance measures are:

Table 1: Benchmarking Snowy 2.0 Against Global PHES Projects

Parameter	Snowy 2.0 Feasibility	Global PHES Benchmarks
Net Head (m)	~626	200 – 700
Round-trip Efficiency	67 – 76%	65 – 80%
Storage Capacity (MWh)	350,000	50,000 – 400,000
Design Life (years)	70+	50 – 100

These figures affirm Snowy 2.0's technical feasibility. At the same time, the project highlights important lessons for future PHES developments, particularly in the management of geotechnical risks, the value of contractual flexibility through EPCM frameworks, and the necessity of aligning project delivery with market and transmission readiness.

2. Civil Works Design by Civil Manager (20%): Steps 1–4, 9

The Snowy Hydro 2.0 PHES project is to be designed to connect the upper Tantangara reservoir to the lower Talbingo Dam. The PHES consists of the intake headrace tunnel that connects to the upper reservoir, the inclined pressure shaft, 6 penstock tunnels connecting to the power generator and a tailrace tunnel connecting to the lower reservoir (see Figure 5**Error! Reference source not found.**).

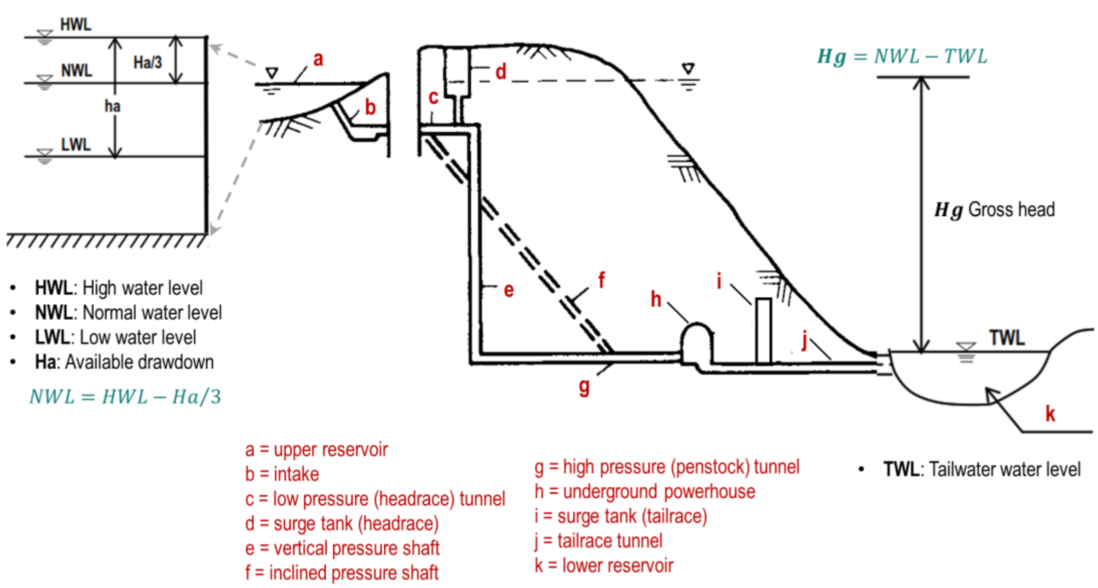

Figure 5: Schematic of the pumped hydro power energy system

The placement of PHES elements was chosen to optimise the head across the two reservoirs and increase the efficiency of the power generators. From a civil works perspective, an upper and lower water level was assumed based on the 2017 study. All relevant tunnels, penstocks and shafts were designed in accordance with these levels to satisfy relevant criteria.

For PHEs projects, the ideal site choice is dependent on the elevation difference between the upper and lower reservoir bed levels (H) as well the horizontal length of the waterway between the two reservoirs (L).

Based on the existing site profile from Gnomes et al, the tantangara reservoir riverbed invert is 1100m and the lower reservoir is 500m. From this, the riverbed elevation is greater the 400m which is the minimum ideal for the PHES sites. An ideal L/H ratio for PHES sites is less than 6, however the Snowy Hydro site dramatically exceeds this. While this is not ideal, the chosen site utilises existing reservoir infrastructure and existing storage systems which makes the project feasible.

2.1 Reservoir Levels & Gross Head

To determine the best alignment for the PHES project, topological and geomorphological data was analysed.

Topography

From a topological perspective, the alignment that best fits the project allows for the maximum gross head while maintaining adequate pressure in the incline shaft and maximises turbine efficiency. Data taken from ELVIS was used to determine the existing surface conditions and dictate the length and angle of proposed PHES infrastructure, see Figure 6.

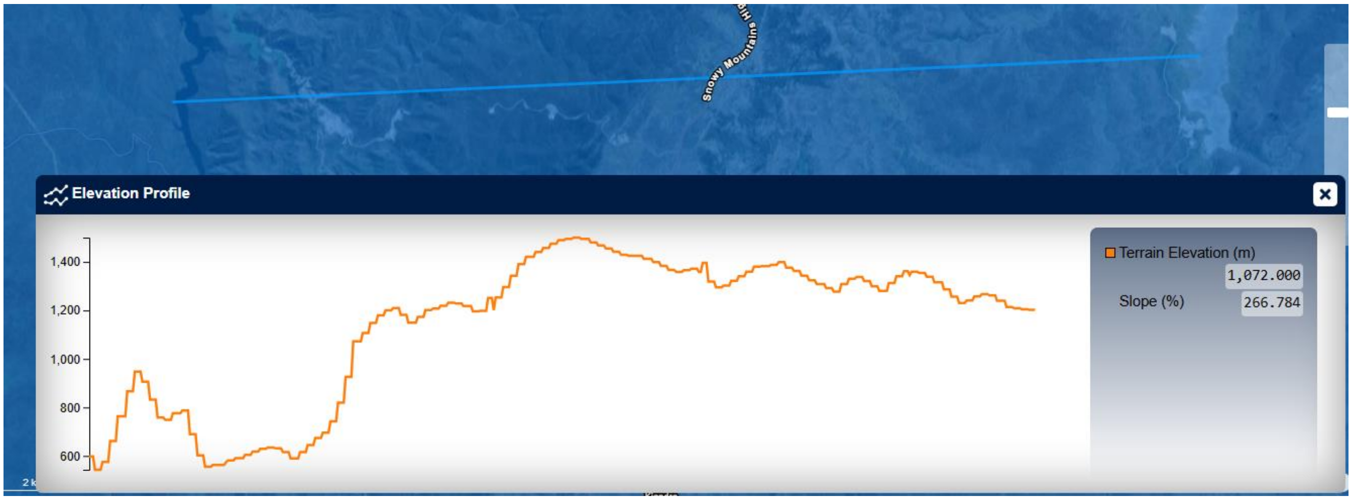

Figure 6: Topological data (ELVIS, 2025)

ELVIS data was also used to verify the HWL for both the tantangara reservoir and talbingo reservoirs as 1228.7m and 543.2m respectively in accordance with the 2021 feasibility study. These water levels were therefore adopted for this study.

Geomorphology

The geomorphology of the region consists of mostly stable rock that is suitable for tunnelling (Snowy Hydro 2.0 feasibility study Chapter 6, 2017). However, there is a large fault line as seen in Figure 7 that traverses the region between the upper and lower reservoirs.

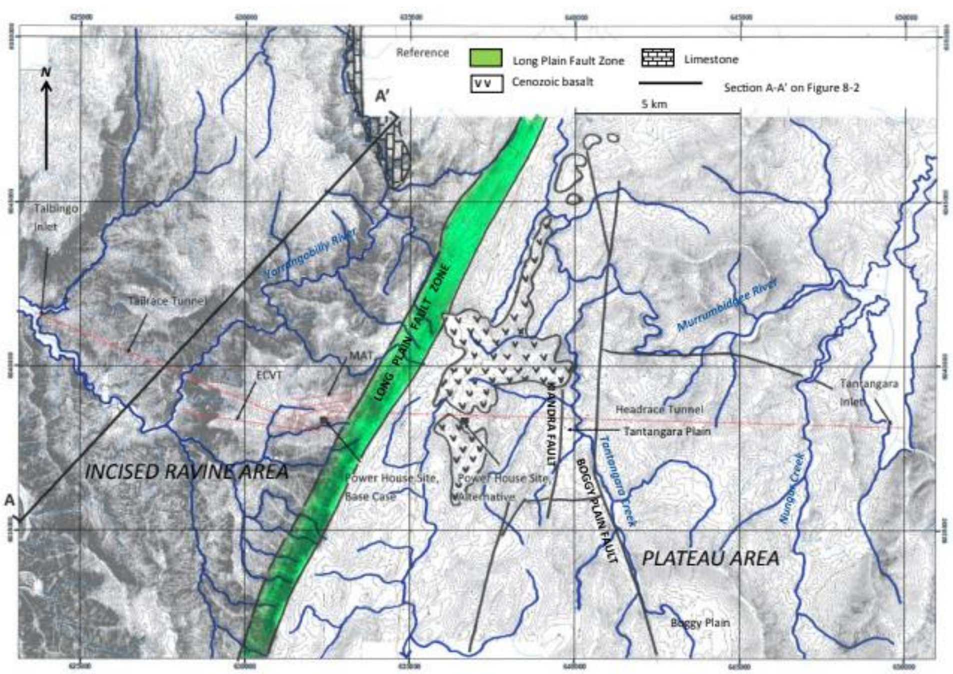

Figure 7: Geomorphology of the region (Snowy Hydro 2.0 feasibility study Chapter 7, 2017)

The long plain fault zone (LPFZ) presents tunnelling issues as well as high in-situ stress conditions that are not ideal for pressure shafts. The inclined shaft is unlined and therefore has been designed to avoid the long plain fault as seen in Figure 8 with the proposed shaft alignment.

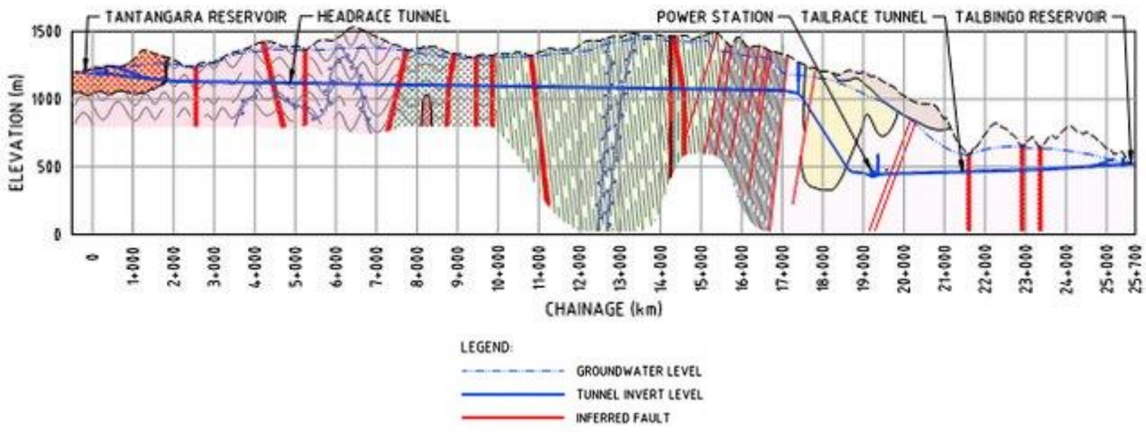

Figure 8: Snowy 2.0 geotechnical longitudinal section (Gomes et al. '2021 Paper')

Reservoir Levels

As the water level and hence head across each reservoir is constantly changing with generation, pumping, infiltration, evaporation and rainfall the PHES system must be able to operate over a range on operating environments. The High Water Level (HWL) of the upper reservoir (Tantangara) is governed by the spillway height and is set at 1228.693m as per the 2021 study and elevation data. The Low Water Level (LWL) is the minimum required water level to maintain safe and reliable operation the PHES system and to achieve adequate head for operating conditions. The intake pipe for the upper headrace must be set below the LWL of the upper reservoir to ensure no air bubbles enter the headrace, see Figure 9.

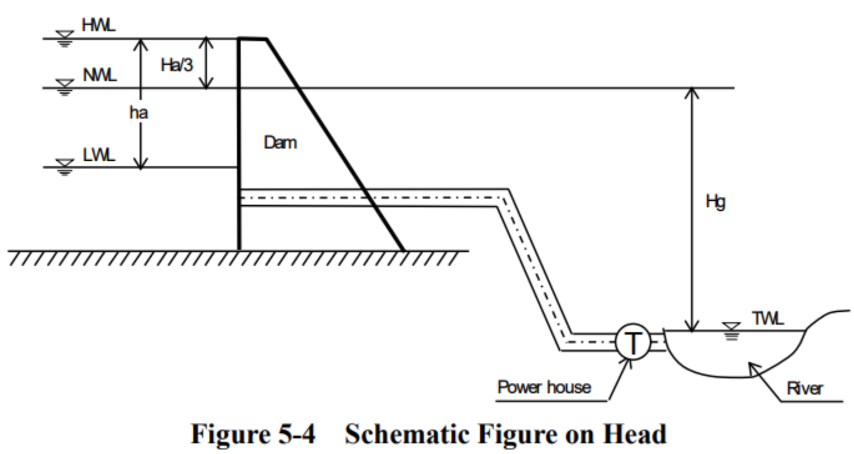

Figure 9: Water levels and Gross Head Schematic (Guideline and Manual for Hydropower Development, 2011)

When pipes are set below the water level, there is a level of drawdown immediately around the intake and a cone of depression forms (see Figure 10). The available drawdown (Ha) is the difference between the HWL and LWL as this is the operational range for the PHES. In steady state conditions during generation, the normal water level (NWL) is set at the bottom of the cone of depression or 1/3 of the available drawdown.

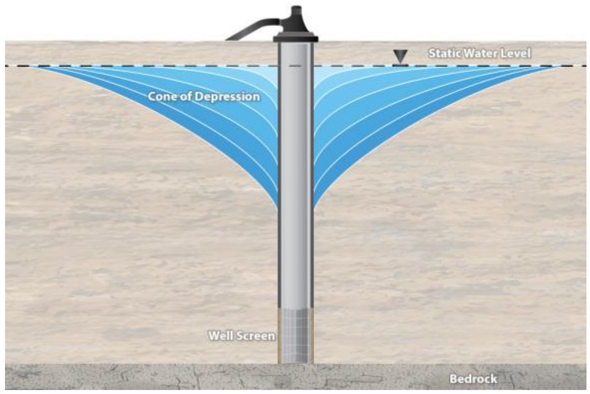

Figure 10: Available Drawdown and cone of depression diagram (Solonist, N/A)

For the lower Talbingo reservoir, the Tailwater level (TWL) is set as the average between the HWL and LWL and governs the gross head across the reservoirs.

A summary of the equations governing the gross head are seen below and values are summarised in Table 2.

$$Ha = HWL - LWL$$

$$NWL = HWL - \frac{Ha}{3}$$

$$TWL = \frac{HWL_{lower} + LWL_{lower}}{2}$$

$$Hg = NWL - TWL$$

Table 2: Summary of Reservoir Levels

Tantangara reservoir	HWL (m)	**1228.69**
	LWL (m)	1205.83
	Ha (m)	22.86
	NWL (m)	1221.07
Talbingo Reservoir	**HWL (m)**	543.19
	LWL (m)	534.35
	TWL (m)	538.77
	Hg (m)	**682.3**

The gross head of 682.3m is within the minimum and maximum levels based on the feasibility study for Snowy Hydro 2.0 and can be adopted for this study.

2.2 Reservoir & Dam Design

The flow through the waterway from headrace to power generators depends significantly on the turbine efficiency and gross head across the upper and lower reservoirs.

To find the design discharge for the PHES system between tantangara reservoir and talbingo reservoir, an initial turbine efficiency of 85% was assumed. The gross head (Hg) between the reservoirs is required to find the operating flow. To ensure the flows through the turbine remain within an acceptable range, a minimum and maximum gross head of 662.6m and 694.3m respectively was adopted based on the 2017 feasibility study for the design discharge. The calculations required to determine the design flow can be found below using Equation 1.

Equation 1

$$Q_{des} = \frac{P_{des}}{\rho g \eta H_g}$$

Where:

P_{des} = design power (2000MW as seen in section 1.1)

ρ = unit weight of water (1000kg/m^3)

G = gravity (9.81m/s^2)

H_g= gross head (682.3m, see Table 2)

Finding other relevant values in relation to the reservoir are the reservoir energy and the duration of generation at design power, see Equation 2 and Table 3.

Equation 2

$$E = \rho g \eta H_g V$$

$$T = V/Q_{des}$$

Where:

V = Active Storage Volume (240GL as per the 2017 feasibility study)

E = Energy

T = Duration of maximum power output

Table 3: Flow design parameters

Q_{des} (m^3/s)	**349.6**
E (J)	1397089890
T (h)	190.7

2.3 Waterway Profile & Pressure Tunnel

Tunnel Design

The design of each tunnel the internal diameter was estimated based on the design flow.

As the maximum plant discharge is dependent on two waterways, the maximum plant discharge is considered to be half of the design discharge of 349.6m^3/s (see Table 3).

The average flow rate can be estimated using the capacity factor and Equation 3.

Equation 3

$$Q_{ave} = Q_{max} \times CF$$

The annual capacity factor (CF) of the plant is assumed to be 0.25, to find an average flow rate of 87.4m^3/s.

Using half of the average flow rate and interpolating from Figure 11, the inner headrace and tailrace diameter is estimated to be 7.2m. The lining thickness for the pressure tunnels is assumed to be 500mm, therefore the diameter of the headrace and tailrace is estimated to be 7.7m.

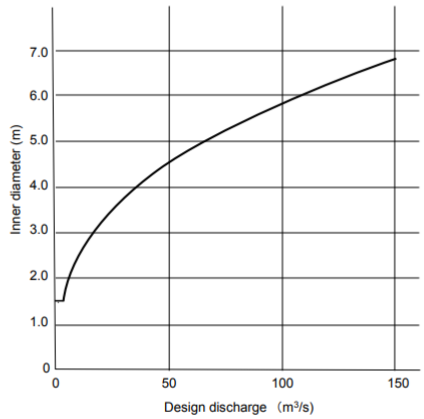

Figure 5-22 Discharge and Tunnel Inner Diameter

Figure 11: Inner Tunnel Diameter and discharge relationship (Guideline and Manual for Hydropower Development, 2011)

The velocity (v) in the tunnels must be checked to ensure it is within an acceptable range by rearranging Equation 4. For tunnels the velocity is typically between 2– 6m/s, while for shafts and penstocks it is usually 4-7m/s.

Equation 4

$$D = \sqrt{\frac{4Q_{max}}{\pi v}}$$

Substituting relevant values obtained above, the maximum velocity for the 7.7mm headrace and tailrace tunnels is 7.58m/s.

As this value exceeds the typical range, an alternative diameter was tested to meet all design criteria. Assuming an external diameter of 10m, the velocity using Equation 4 was found to be 5.5m/s. This satisfies velocity criteria for the headrace and tailrace tunnels as well as the incline shaft.

The new average flow rate was estimated as ~150m^3/s using extrapolation from Figure 11. This equates with an actual CF of 0.43.

Headrace design

The headrace intake is set about 1.5-2 times the diameter of the headrace pipe below the NWL as shown in Figure 12.

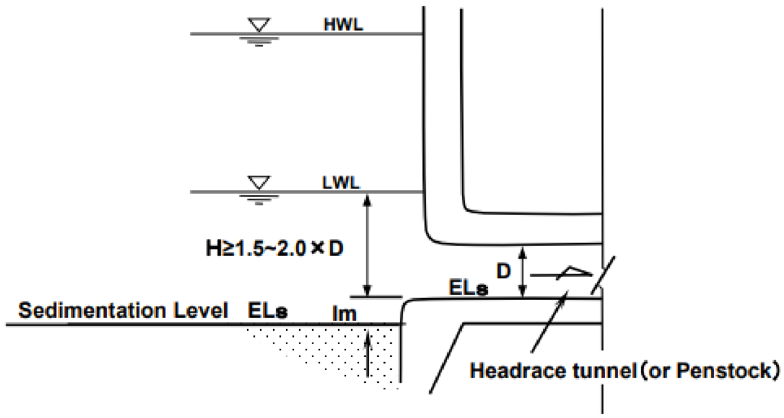

Figure 12: Headrace Invert level (Guideline and Manual for Hydropower Development, 2011)

Knowing the LWL of the Tantangara reservoir to be 1221.07m and the headrace diameter to be 9m, the invert for the headrace can be calculated as 1187.8m AHD.

Tailrace design

Turbine elevation is lower the LWL for the lower reservoir so that adequate potential energy harnessed through the generation process can be achieved. Consequently, the draft head (Hd) is found as the difference between the turbine centerline and the LWL of the Talbingo reservoir.

The draft head can be estimated using the assumed maximum pumping head of 694.3m as per the 2027 feasibility study.

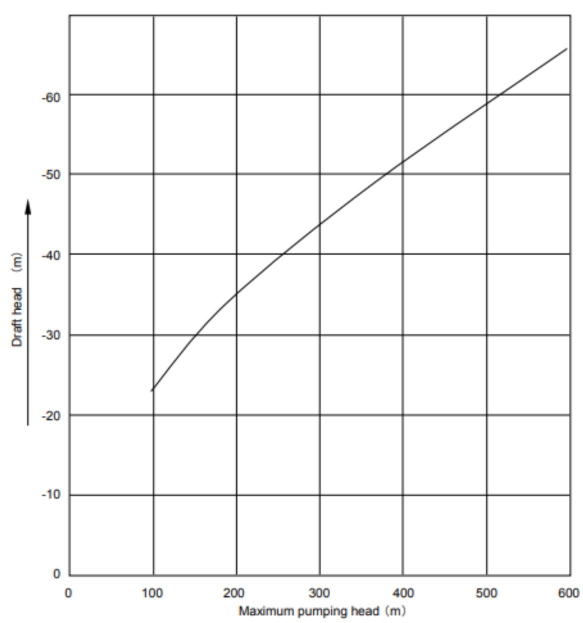

Figure 5-36 Draft Head

Figure 13: Draft head and pumping head relationship (Guideline and Manual for Hydropower Development, 2011)

Extrapolating from Figure 13 the draft head for the tailrace is estimated to be 72m. Using known values, the turbine centerline is found using Equation 5 to be 462.35m.

Equation 5

$$CL = LWL_{lower} - h_d$$

PHES infrastructure design

To avoid the incline shaft traversing the LPFZ, the headrace is designed to extend beyond the fault line for a total length of 15,000m. The incline shaft is assumed to be graded at 25° to the horizontal down to the turbine centerline elevation of 462.35m and the penstocks and headrace are designed with a flat 0% grade. The tailrace has a slight (0.83%) grade up to the LWL of the Talbingo reservoir.

Using known elevation and design values, the PHES infrastructure design can be seen in Figure 14 with a full calculation breakdown in Appendix A.

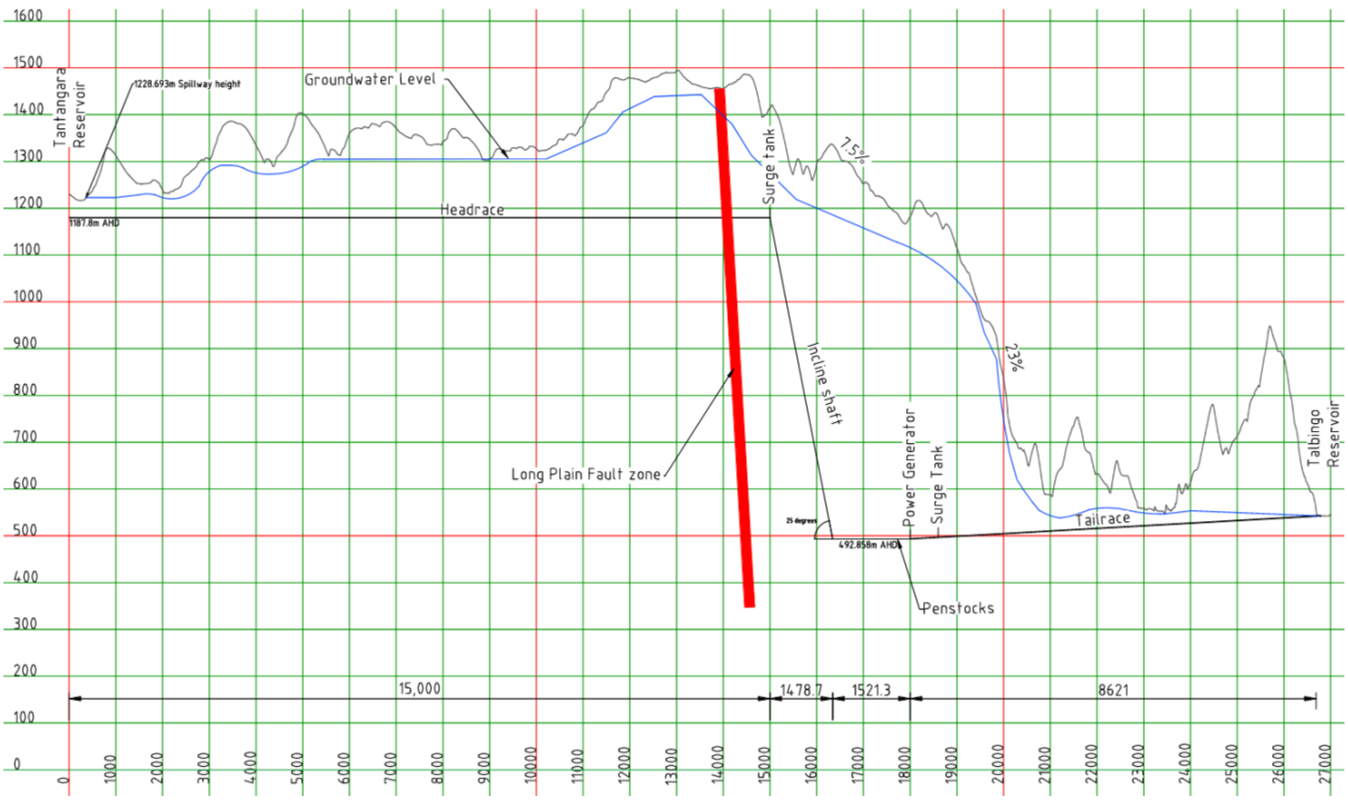

Figure 14: Design of Pressure Tunnels and Shafts

Please note that angles may appear distorted due to vertical exaggeration.

2.4 Confinement Check – Norwegian Criteria

The inclined pressure shaft is assumed to be unlined to help reduce costs unlike those associated with concrete or steel lined tunnels (Henki Ødegaard & Bjørn Nilsen, 2018). The Norwegian confinement criteria therefore must be applied to the inclined shaft to ensure no hydraulic failure occurs due to the pressure exerted on the surrounding in situ soil (Bikash Chaudhary, 2023). The Norwegian confinement check can be found as a factor of safety incorporating the groundwater head, minimum rock cover and vertical rock cover as seen in **Error! Reference source not found.**.

Equation 6

$$F_{RV} = \frac{C_{RV}\Upsilon_R \cos(\alpha)}{\Upsilon_w h_s}$$

$$F_{RM} = \frac{C_{RM}\Upsilon_R \cos(\beta)}{\Upsilon_w h_s}$$

Where:

F_{RV} = Vertical Factor of Safety

F_{RM} =Minimum Factor of Safety

C_{RV} = Vertical cover (see Figure 30

C_{RM} =Minimum Factor of Safety (see Figure 30

α = angle of pressure shaft (assumed to be 25°)

β = angle of existing surface

Υ_R = unit weight of in-situ rock (25kN/m^3)

Υ_w = unit weight of water (9.81kN/m^3)

h_s = static head (m)

To ensure that the confinement criteria is satisfied throughout the whole length of the incline shaft, the factor of safety was checked at multiple locations (see Figure 15).

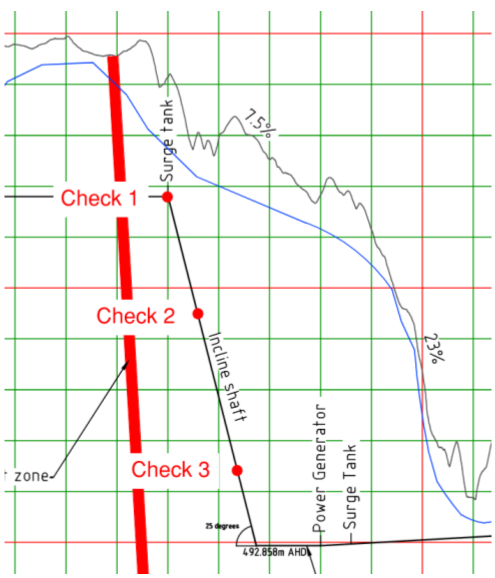

Figure 15: Norwegian Criteria check locations

To achieve “safe” in situ conditions, all factors of safety must be greater than 1.5. Table 4 demonstrates the parameters required for each confinement check and that all points along the incline shaft achieve safe soil conditions.

Table 4: Norwegian Confinement Criteria Check

	Check 1	Check 2	Check 3
CRM	**1005**	**3503**	**4002**
CRV	**225**	**350**	**650**
hs	**18.8**	**255.8**	**555.8**
β	**7.5%**	**23%**	**23%**
Frv	**27.6**	**3.2**	**2.7**
Frm	**30.4**	**3.4**	**2.9**

2.5 Underground Powerhouse Cavern Design

The underground cavern design incorporates the machine hall (MH), the transformer hall (TH) and the IPB galleries to house the powerhouse and transformers for electricity generation. The design of these components is important as they are effected by the insitu soil and groundwater conditions as well as size of the equipment they house.

The power generator is located after the penstocks and harnesses the potential energy storage in the upper reservoir. The general arrangement for the powerhouse and transformers is shown below in Figure 16.

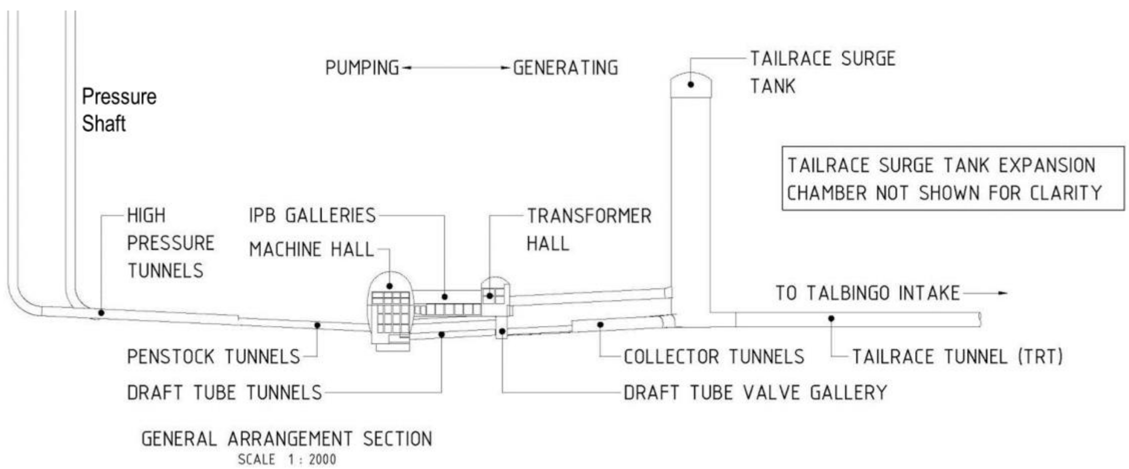

Figure 16: Tailrace surge tank section (Page 78 of 114 in Chapter 9 in ‘2017 Study’)

The machine hall houses the turbines and pumps required to operate the PHES system as well as other equipment required for power generation. The IPB galleries are a series of halls that connect the machine hall to the transformer hall. This this design six IPB galleries are required due to the 6 penstocks. The transformer hall contains all the transformers required as well as all transmission lines and draft tube valves.

The machine hall cavern can come in a variety of shapes include, Mushroom shaped, Horseshoe shaped and Elliptical (see Figure 17). Design of the machine hall is ultimately dictated by the stress distribution in the surrounding soil. Mushroom shaped caverns are typically required in high overburden stress conditions and have extra capital costs associated with the large reinforced arch. An elliptical shaped cavern uses the concept of the arch and takes it further by distributing the stress caused by the weight of the rock along the length of the curve and is ideal for high stress environments or where soil quality is poor. The horseshoe shaped cavern is the most commonly used in hydro plants.

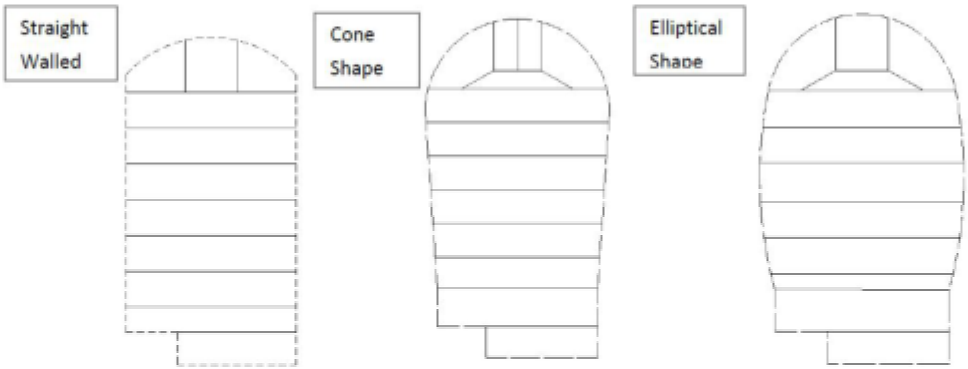

Figure 17: Types of Machine Hall shapes

Section 2.1 discussed the geomorphology of the Site and Figure 14 shows the location of the power generator. The soil in this location ranges from good to fair quality, therefore extreme measures such an elliptical shaped cavern is not required to counteract the effects of the stress in the existing soil. A conventional horseshoe shaped machine hall was chosen with vertical sidewalls. The cavern is sized to accommodate for cranes, turbine and working space to be 25x180x40m (see Appendix B for assumptions).

The transformer hall is required to accommodate step up transformers to allow for high voltage transmission across the NEW network. The lower portion of the hall houses the draft tubes and the cavern overall allows for ventilation and cooling during the power generation process. The transformer hall has been sized to be 18x82x30m (see Appendix B for sizing assumptions).

The IPB galleries are not only used to connect the transformer and machine hall but vitally provide a mechanism of separation between the caverns from a fire safety perspective. International Society for Rock Mechanics and Rock Engineering (ISRM) and International Commission on Large Dams (ICOLD) recommends that the size of the rock pillars separating the halls is dependant on the quality of rock. For this design, the IPB galleries is 30m based on the fair rock quality established in section 2.1 (see Appendix B for sizing assumptions). This pillar thickness is also in accordance with fire and electrical safety recommendations.

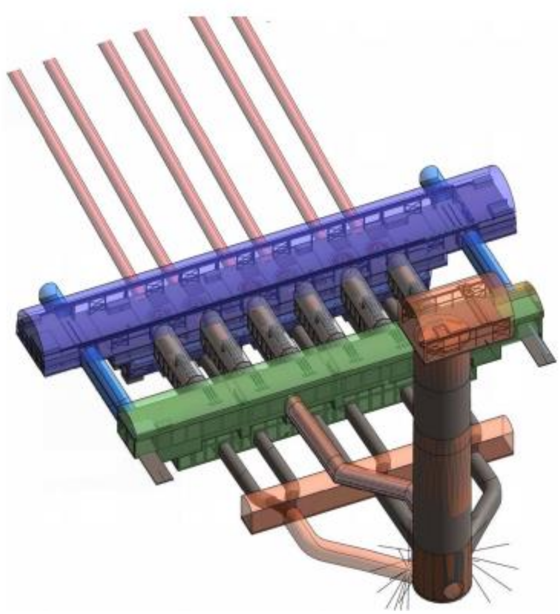

Figure 18: Power station complex details (Alexandre et al, 2021)

The stresses on the powerhouse are based on in situ stress conditions including the overburden height and governed using Equation 7.

Equation 7

$$\sigma_v = \Upsilon h_e$$

$$\sigma_h = \sigma_v \times k$$

Where:

Υ = unit rock weight (25kN/m3)

He = Vertical rock cover above cavern (m)

K = soil stiffness (assumed to be 2.4 for the region)

The using the height of the caverns, the known turbine centreline elevation and the existing surface profile, the rock cover of the powerhouse was found to be approximately 700m.

This is used to find the horizontal and vertical stresses on the cavern as 41,859kN/m3 and 17,441kN/m3.

To further develop the design of the powerhouse cavern, a Finite Element Analysis (FEA) must be undertaken to model the cavern stress distribution in the surrounding soil. FEA is a complex analysis that requires highly rigorous computational models to model the existing soil and the deformations formed with the construction of the cavern hall. Figure 19 demonstrates the complex nature of the FEA including the vectorisation of forces acting on the cavern hall.

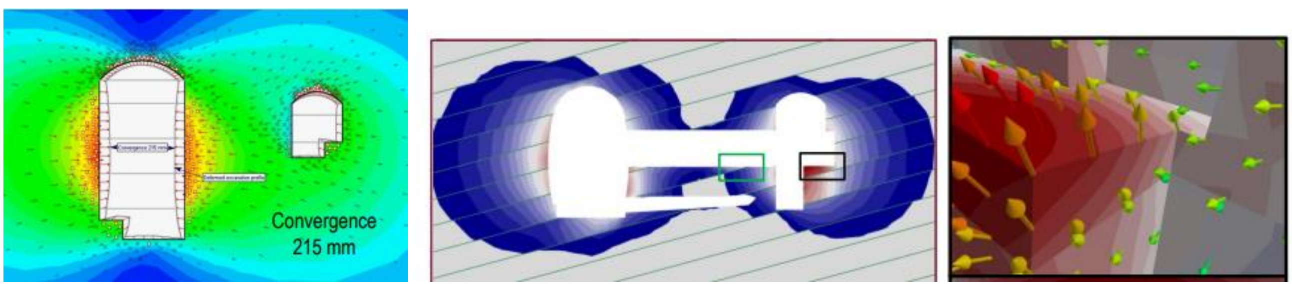

Figure 19: FEA Example distribution

3. Hydraulic Structure and Stress Analysis by Mechanical/Hydraulic Design Manager (20%): Steps 5–8

- The design of pressure tunnels and shafts is critical for ensuring the efficient transport of water between the reservoirs. The **effective head** is the actual usable water head after accounting for **head loss** due to friction in tunnels, valves, and other components.

- **Head Loss Calculation**: Apply principles of fluid mechanics to calculate friction losses using the Darcy-Weisbach equation and related methods.
- **Effective Head Calculation**: Subtract head losses from the total water head to determine the **effective head**, which directly impacts the energy generation potential.

3.1 Head Loss Analysis & Net Head

3.1 Head Loss Analysis & Net Head

Net head calculations are critical to correctly designing the pressure tunnels & shafts. The net head, or **effective head**, is the water head available after accounting for **head loss** due to friction in tunnels, valves & other components. Effective head can be found by subtracting the major head loss (H_{major}), minor head loss ($H_{f,minor}$) & draft head (H_d) from the gross water head represented by H_g.

Equation 8

$$H_e = H_g - H_d - H_{f,major} - H_{f,minor}$$

Major head loss can be calculated using the Darcy-Weisbach equation xx after first finding the Reynolds number, equation xx & friction factor given by the Swamee-Jain equation xx. The major head loss accounts for friction along the tunnel surface of the headrace, penstock & incline shaft.

Equation 9

$$H_{f,major} = f\frac{L}{D}\frac{v^2}{2g}$$

$$f = \frac{0.25}{[\log_{10}(\frac{\varepsilon}{3.7\,D} + \frac{5.74}{Re^{0.9}})]^{\,2}}$$

$$Re = \frac{4Q_{total}}{\pi N_{pen} D v}$$

Where:

Q_{Total} = Total discharge

N_{pen} = Number of penstocks

D = Pipe diameter

v = kinematic viscosity of water

ε = absolute roughness

L = Length of pipe

D = Diameter of pipe

g = gravity (9.81 m/s^2)

The major head losses of the headrace, inclined shaft & penstocks are 22.13m, 20.71m, & 2.85m, respectively. This totals to a 49.696m major head loss upstream of the turbines.

Minor head loss is based on the friction caused by bends, valves, expansions & contractions, where a friction coefficient (k-value) is assigned based on the amount & type of these components. The minor loss in each of the headrace, penstock & incline shaft is calculated with Equation 10.

Calculating the K-value requires identifying and evaluating all components of the plant piping upstream of the turbine. Figure 20: details where the components lie along the piping, and Table 6: Design Summaryis used to assign each of the piping components reasonable K-values.

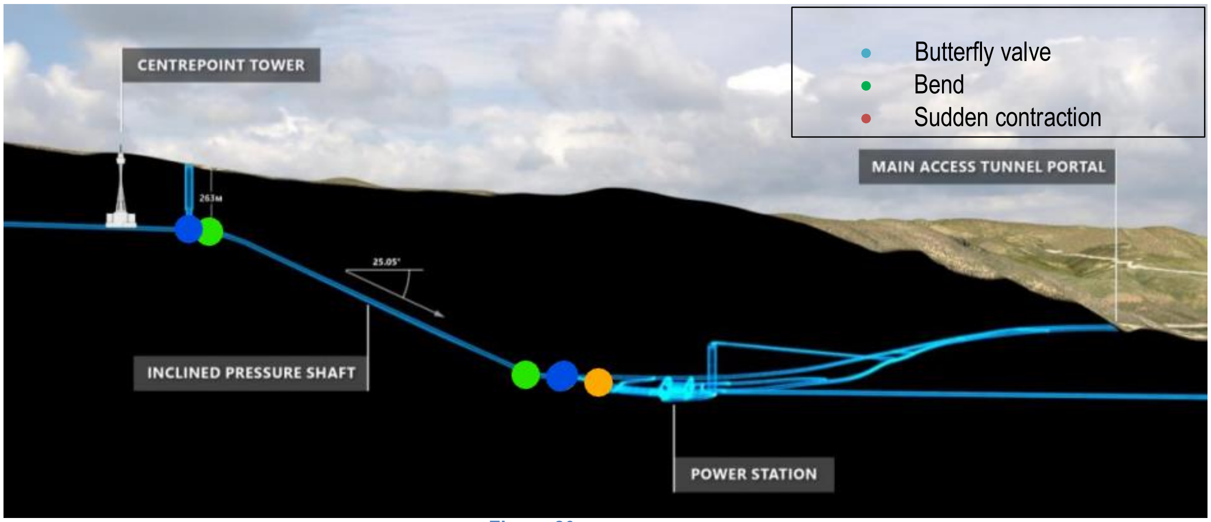

Figure 20:

Fitting / Component	Typical K-value
Sudden contraction	0.4 – 0.8
Sudden expansion	≈ 1.0
Smooth bend (90°)	0.2 – 0.4
Gate valve (fully open)	≈ 0.15
Butterfly valve (partially open)	≈ 2.0

Table 5: Flow design parameters

The two bends in the pipe surrounding the inclined pressure shaft are assumed to have a K-value of 0.2; the angle is less than 90°, which is why it is on the smaller end of the smooth bend K-value range. The surge tank in the headrace has a butterfly valve, resulting in a K-value of 2.0. The change between the inclined pressure shaft and penstocks includes two 90° bends & an expansion from one 10m diameter pipe to six 5m diameter penstocks.

The total minor head is calculated based on the individual K-value of the component & the velocity of the water where the component is along the piping, based on Equation 10. The total minor head loss is 4.5m.

Equation 10

$$H_{f,minor} = \sum K \frac{v^2}{2g}$$

Where:

K = K-value of bends, valves, expansions & contractions

v = Velocity of water

g = Gravity 9.81m/s2

Using gross head of 682.3m & draft head of 72m as found previously, the effective head totals to 560.1m.

3. .2 Penstock Design & Diameter Verification

Designing the penstock requires finding an appropriate penstock diameter that satisfies the velocity target, head-loss target & empirical chart.

The penstock diameter must relate to a velocity within 4-6m/s, based on the penstock flow rate. The total flow rate through the penstocks relates to the design power & effective head found in 3.1 as shown in equation xx. The velocity verification requires the flow rate of individual penstocks given by Equation 11.

Equation 11

$$Q_{Total} = f\frac{P_{design} * 10^6}{\rho g H_e \eta_{overall}}$$

$$Q_{Penstock} = \frac{Q_{Total}}{N_{Penstock}}$$

Where:

P_{design} = Design power (MW)

$N_{penstock}$ = Number of penstocks

g = gravity (9.81 m/s^2)

H_e = Effective head

ρ = Water density $(1000\ {}^{kg}/_{m^3})$

$\eta_{overall}$ = Overall efficiency

The $Q_{Penstock}$ is 69.13 ${}^{m^3}/_{s}$. For this value, the diameter must be between 4.7m & 3.75m, using equation x to relate velocity to diameter & flow rate.

Equation 12

$$v = \frac{4Q}{\pi D^2}$$

Where:

$Q_{penstock}$ = Flow per penstock

D = Penstock diameter (m)

The empirical target check involves using a D-Q sizing chart extrapolation using equation xx to find an estimated penstock diameter.

The estimated diameter given the $Q_{penstock}$ value is 5.13m. This is over the diameter given in the velocity range of 3.75-4.7m. The diameter chosen for the penstocks is 4.7m as it fits within the range of the velocity check while being close to the estimated value from the empirical target check.

The head loss target must also fit for this diameter; the total head loss caused by the penstocks is 27.52m, which fits within the required range.

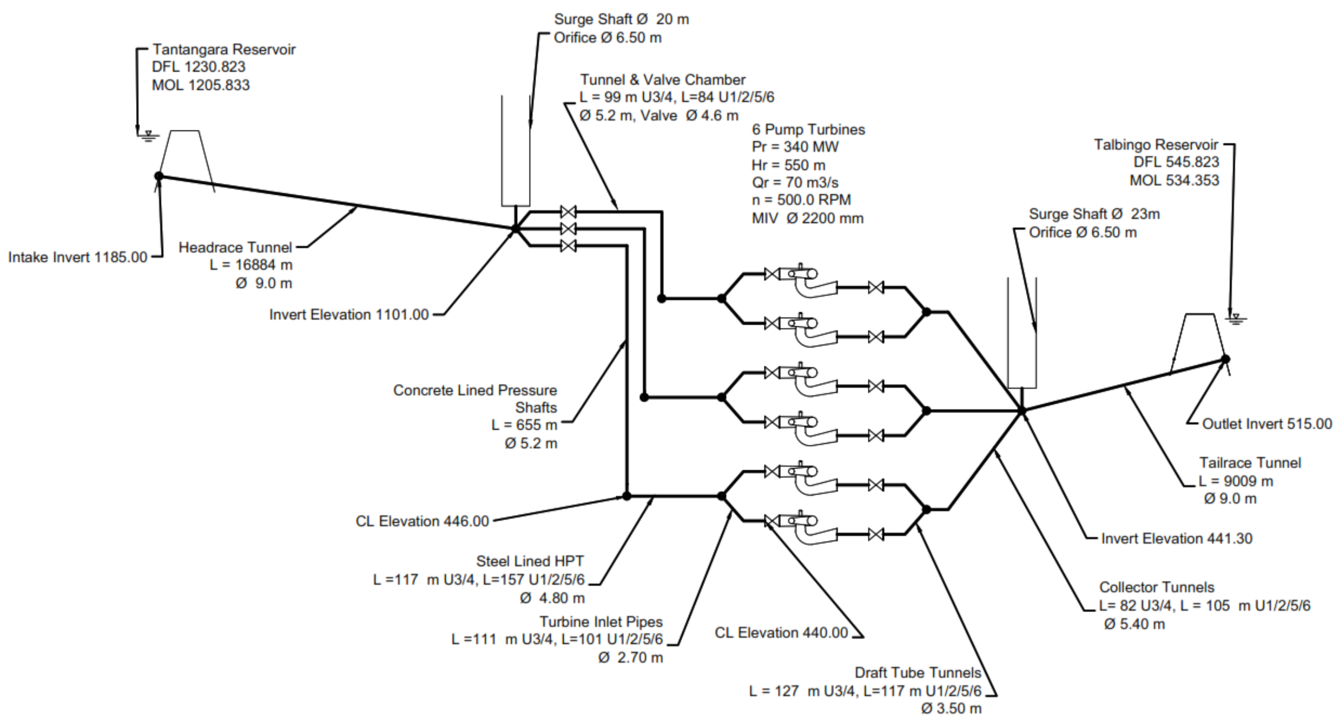

Figure 21: Schematic of power waterway system – Ravine (Page 10 of 114 in Chapter 9 in '2017 Study')

3.3 Penstock Design & Diameter Verification

The inclined pressure shaft (ISP) has been analysed to determine if the pressure in the pipe lining ever surpasses the maximum safe value of 6-7MPa. The inner & outer radius of the pipe is found to be 5m & 5.5m respectively based on previous diameter calculations using Equation 13 and Equation 14.

Equation 13

$$r_i = D/2$$

Equation 14

$$r_o = r_i + t_{lining}$$

D = Diameter (10m)

r_i = Inner radius (5m)

r_o = Outer radius (5.5m)

t_{lining} = Lining thickness (0.5m)

From these radii, the internal & external pressures can be calculated using the unit weight of rock & water. The internal pressure due to water weight is 5.45 MPa, while the external pressure is 6.18 MPa.

Equation 15

$$p_i = \gamma_w h_s$$

$$p_o = \gamma_r h_w$$

Υ_w = unit weight of water (9.81kN/m^3)

h_s = static head (m)

Υ_R = unit weight of in-situ rock (25kN/m^3)

h_w = groundwater head (m)

The boundary load at the external surface is found to be 290.5 kPa. Boundary load is found using the pressures found as well as the material properties of concrete & rock, as shown in Equation 16.

Equation 16

$$p_F = \frac{(p_e - p_i)\left[\frac{2(2-\nu_c)}{(r_o/r_i)^2-1} + \frac{1-2\nu_c}{1-r_i/r_o}\right] - 3p_e\dfrac{E_c(1+\nu_r)}{E_r(1+\nu_c)}}{3\left[\frac{2(1-\nu_c)}{(r_o/r_i)^2-1} + \dfrac{E_c(1+\nu_r)}{E_r(1+\nu_c)} + 1 - 2\nu_c\right]}$$

P_e = external pressure (kPa)

P_i = internal pressure (kPa)

E_c = elastic modulus of concrete (30000000 kPa)

E_r = elastic modulus of rock (60000000 kPa)

ν_c = poisons ratio for concrete (0.2)

ν_r = poisons ratio for rock (0.250)

The tangential & radial stress distribution across the lining is found using equations xx & xx. By observation, the maximum stress will be found at the innermost point, with the value r = 5. This maximum stress is 6.21MPa for tangential stress & 3.35MPa for radial stress, which is considered acceptable.

Equation 17

$$\sigma_\theta(r) = \frac{(p_e - p_i)}{2(1-\nu_c)} \cdot \frac{1-(r_o/r)^2}{(r_o/r_i)^2 - 1} + \frac{(p_e - p_i)}{2(1-\nu_c)} \cdot \frac{\ln(r_o/r) + 1 - 2\nu_c}{\ln(r_o/r_i)} + p_F \cdot \frac{1+(r_o/r)^2}{(r_o/r_i)^2 - 1} + p_F$$

$$\sigma_r(r) = \frac{(p_e - p_i)}{2(1-\nu_c)} \cdot \frac{1-(r_o/r)^2}{(r_o/r_i)^2 - 1} + \frac{(p_e - p_i)}{2(1-\nu_c)} \cdot \frac{\ln(r_o/r)}{\ln(r_o/r_i)} + p_F \cdot \frac{1-(r_o/r)^2}{(r_o/r_i)^2 - 1} + p_F$$

3.4 Surge Tank Design

The height of the tailrace surge tank must account for the rise & drawdown of headwater levels. There is also a freeboard & submergence allowance built into the formula for any extra changes. The rise & drawdown of the headwater is calculated as 15% of the gross head H_g. These allowances are used to calculate H_{stop}.& $H_{s,bottom}$ where the difference between them is the total height of the shaft H_{shaft}.

Equation 18

$$H_{s,\text{top}} \approx H_0 + \Delta h_{\text{rise}} + \text{freeboard}$$
$$H_{s,\text{bottom}} \approx H_0 - \Delta h_{\text{drawdown}} - \text{submergence}$$
$$H_{\text{shaft}} \approx (\Delta h_{\text{rise}} + \text{freeboard}) + (\Delta h_{\text{drawdown}} + \text{submergence})$$

The cross-sectional area must be within a safe range of difference between the total areas of the penstocks. The recommended ratio of A_s to A_p is roughly 8. This ratio fits for a diameter of 33m and an accompanying cross-sectional area of 855.3m^2. There is a further stability rule of thumb that relates the ratio of A_s to A_p to the ratio between penstock length & effective head, where A_s/A_p must be larger than $L_{penstock}/H_e$.

There are also stability equations that must fit with the chosen diameter, which are based on the flow rate, water wave speed & effective head, as shown in equation xx. Unfortunately, the values found for this equation would suggest a tank diameter of 2m, which does not logically make sense with the previous calculations and is assumed to be incorrect.

Equation 19

$$A_s = \frac{Q_0}{\omega H_e}, \quad \omega \approx \frac{\pi a}{L}$$

Thus, the final surge tank design is 33m diameter by 169 meters height as shown in Figure 22

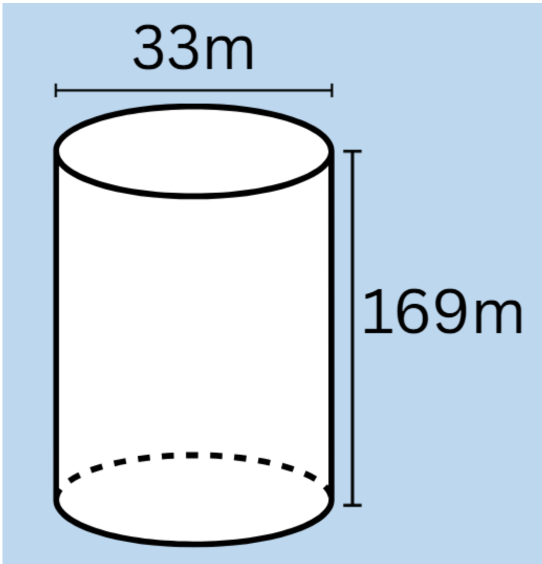

Figure 22: Surge tank design

4. Turbine Efficiency and Energy Calculation by Electrical Manager (20%): Steps 10–12

4.1 Turbine Type Selection

The turbine type selection takes into account the head and per unit flow rate through the turbine. There are four main types of turbine, each operating at different pressures and discharges. These turbines fall into two broader categories: Impulse and Reaction type. Impulse turbines, like the Pelton, are driven by the kinetic energy of a fast moving jet of water, rather than it being submerged in a flow. Impulse turbines operate in air. On the other hand, reaction type turbines are turbines that are fully submerged in the water and enclosed in a pressure casing. In addition to the kinetic energy of the water, profiled runner blades cause pressure differences across the turbine to also contribute to rotating the turbine. Examples of Reaction type turbines include Francis, Kaplan and Bulb turbines.

Pelton

Suitable for very high heads and low flow rates, Pelton turbines have a series of buckets mounted on a turbine runner. As an impulse type turbine, they are rotated by a high velocity jet of water directed through a nozzle or nozzles into the buckets. Pelton type turbines maintain a high efficiency under partial load, so are suitable for flexible opFeration.

Figure 23:

Francis

Francis turbines are suitable for moderate heads and flow rates and are the most widely used hydropower turbine type. They have a high efficiency across a broad operating envelope, with water entering radially through stay vanes and wicket gates, and exiting axially through the draft tube. Francis turbines are reaction type turbines.

Figure 24:

Kaplan

With their propellor-like appearance, Kaplan turbines are axial flow, reaction type turbines, ideal for low heads and very high flow rates. They are common in run of river plants and low head PHES schemes.

Figure 25:

Bulb

Bulb turbines are a more compact variant of the reaction type, axial-flow Kaplan turbine. They are designed for very low heads and very high discharges. Bulb turbines are rarely appropriate for PHES schemes due to the unusually low head and extremely high flow operating conditions.

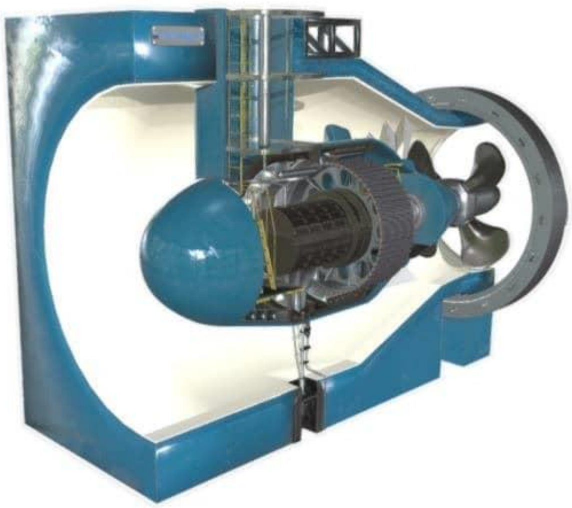

Figure 26:

Turbine Selection

To select the most appropriate turbine type for this proposed PHES scheme, the effective head and flow rate per unit were calculated to be 566-574m and 56-59m^3/s respectively. These values were then compared to the operating conditions of each turbine type. Since the effective head was between 50m and 700m, and the flow rate was higher than 50m^3/s per turbine, the Francis (Reaction type) turbine was selected. This selection was verified using the below reference chart which confirmed that a Francis type turbine was indeed best suited to the operating conditions specified.

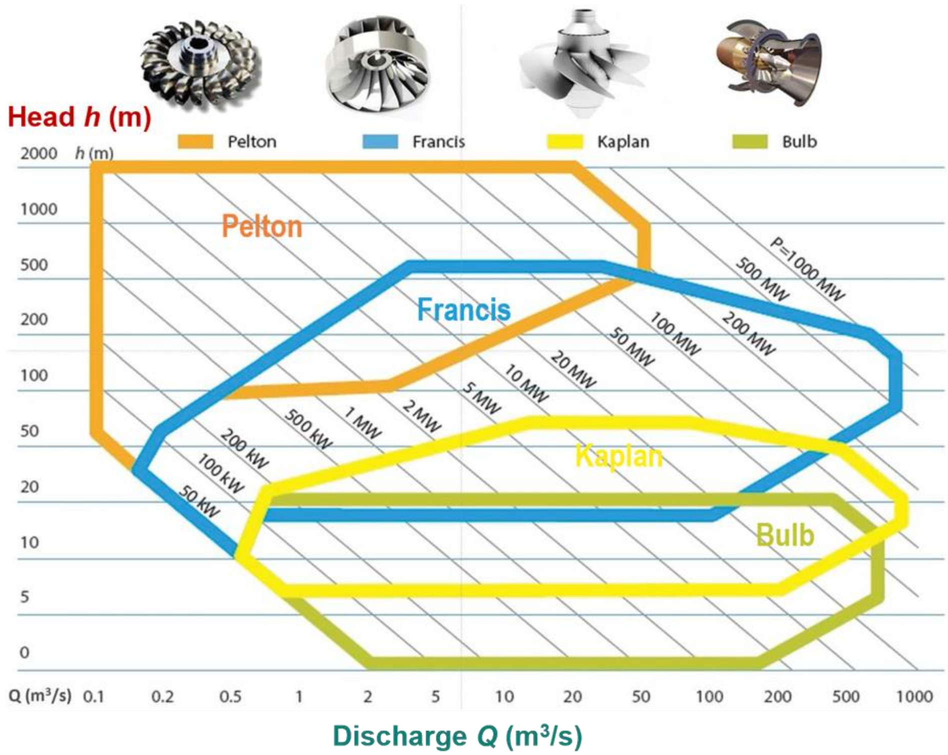

Figure 27:

4.2 Turbine Efficiency Estimation

The turbine efficiency was determined using the specific speed method. $N_S = \frac{N * \sqrt{P_{Unit}}}{H_e^{\frac{5}{4}}}$, with P in horsepower and H in feet.

N was interpolated from the chart below for Francis Turbines to be approximately 375rpm, using the effective head of 566-574m and the flow rate 56.99m^3/s per unit.

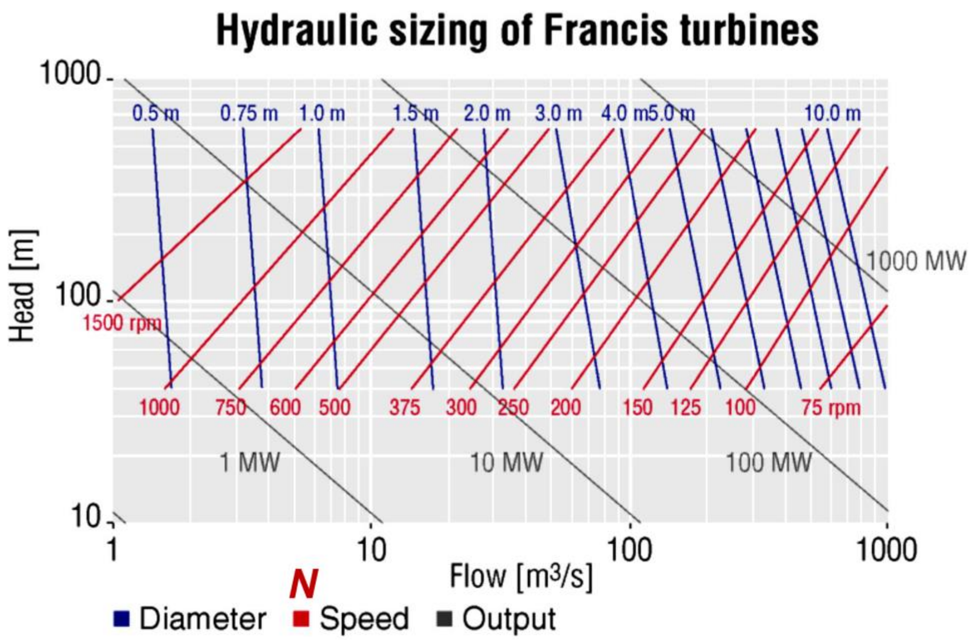

Figure 2628:

The appropriate unit conversions were made for effective head and power per turbine, and N_s was calculated to be 18.27rpm. The below reference chart was then used to interpolate the turbine efficiency. As a Francis type turbine with N_s of 18.27rpm, the turbine efficiency was found to be approximately 89%.

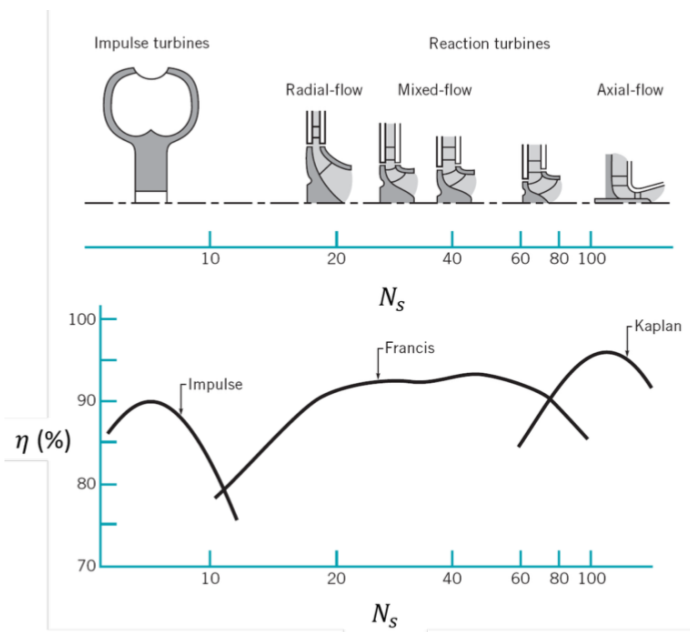

Figure 2729:

In addition to turbine efficiency, generator efficiency and transformer efficiency also play a role in overall efficiency. Since specific implementation of generators and transformers are beyond the scope of this report, generator efficiency was estimated to be 97%, in line with typical values, and transformer efficiency was estimated to be 99%, as this is in the typical range. The overall plant efficiency is the product of these three efficiency values, so it was found to be 85.47%.

4.3 Power Generation & Pumping Cycle

Using the efficiency, flow rate and Effective head values previously calculated, as well as the properties of water and gravity, the Power generated in generation mode, and the power required in pump mode can both be calculated.

$$P_{gen} = \frac{\rho g Q H_e \eta_{Overall}}{10^6} \text{ (MW)}$$

$$P_{Pump} = \frac{\rho g Q H_e}{\eta_{Pump} * 10^6} \text{ (MW)}$$

Power generated was calculated to be 1778.68MW.

Estimating η_{Pump} to be approximately 0.91, it was calculated that P_{Pump} is 2286.97 MW.

Due to energy losses, the energy gained during generation is always less than the energy lost during pumping.

$$Round\ trip\ efficiency = \frac{Energy\ gained\ during\ generation}{Energy\ required\ for\ pumping}$$

$$Round\ trip\ efficiency = \frac{1778.68}{2286.97}$$

$$Round\ trip\ efficiency = 77.78\%$$

The round-trip efficiency of the plant is the percentage of energy that goes into the pumps in pumping mode, that will be transformed back into electrical energy when the plant is in generation mode. Compare this to Wivenhoe PHES Plant, which in 2016 had a round-trip efficiency of 66%, and the projected efficiency of this project is quite high. It must be noted, however, that Wivenhoe was plagued by a number of issues. The average round trip efficiency of PHES Plants in the U.S. was 79% in 2019, almost exactly the same as the projected round-trip efficiency of this proposed plant.

With the chosen turbine being a Francis turbine the head fluctuation rate must be checked to ensure that the turbine can operate across a given range of operating environments. The head fluctuation rate (HFR) or the change in gross head for a Francis turbine must be greater than 0.7 and can be found using equation.

$$HFR = \frac{LWL_{upper} - TWL_{lower}}{HWL_{upper} - TWL_{lower}} \geq 0.7$$

$$HFR = \frac{1205.833 - 538.773}{1228.693 - 538.773} \geq 0.7$$

$$HFR = 0.967 \geq 0.7$$

Using these known values, the HFR was found to be 0.967 which is adequate for the Francis Turbine.

5. Design iterations and summary

The design process involves iterations to so that all relevant criteria can be refined and assumptions can be verified. After undertaking the relevant design of reservoir levels and head losses was well as PHEs infrastructure and turbine design, alll values have be re calculated with the relevant calculated efficiencies, gross head and flow rate. Table 6 provides a design summary of the Snowy Hydro 2.0 PHES design.

Table 6: Design Summary

Description	Unit	2017 Study (Ravine)	Your Design (47% incline)
Installed capacity (generation and pumping)	MW	2,000	2,000
Number of units	#	6	6
Type of pump-turbines	–	Reversible Francis	Francis
Number of synchronous units	#	3	6
Number of variable speed units	#	3	
Min / max gross head differential	m	662.6 / 694.4	682.3m
Headrace Tunnel: diameter x length	m	9.0m x 16.9 km	10.0m x 15 km
Tailrace Tunnel: diameter x length	m	9.0m x 9.0 km	10.0m x 8.6km
Headrace surge chamber: diameter x height	m	21.6 x 215	33 x 169
Tailrace surge chamber: dimensions x height	m	25 x 160	33 x 169
Number of pressure shafts	#	3	1 Inclined
Diameter of pressure shafts	m	5.2	9
Number of penstock tunnels	#	6	6
Machine hall dimensions (max. span x length x height)	m	33 x 190 x 55	25x180x40
Transformer hall dimensions (max. span x length x height)	m	20 x 180 x 43	18x82x40
Main Access Tunnel (MAT) dimensions (diameter x length)	m	9.0m x 4.3 km	10m x 3.4 km
Emergency, Cable and Ventilation Tunnel (ECVT) (diameter x length)	m	9.0m x 4.43 km	10m x 4.43 km
Hydraulic head losses at 2,000 MW*	m	68	–
Round Trip Efficiency at 2,000 MW	%	67%	78%

Conclusion

The Snowy 2.0 project is a game-changer for the future of Australia's energy, providing a significant solution to support the growing utilization of renewable energy resources. The report has thoroughly examined the design, energy calculations, civil works, turbine selection, and integration with the National Electricity Market (NEM) of the project. The technical feasibility and innovation illustrated in this report underscore Snowy 2.0's potential to deliver 2,000 MW of stored power, firming up the network and enabling the firming of renewable generation.

The funding arrangement of hybrids, where government equity is supplemented by Snowy Hydro's retained profits, enables the project to remain in public ownership while benefiting from operating flexibility. The use of the Engineering, Procurement, and Construction Management (EPCM) structure provides the project with the operating flexibility required to manage advanced geotechnical challenges, facilitating unhindered progress under unforeseen conditions.

Snowy 2.0 is anticipated to be completed by 2029, with the project design life for more than 70 years, so it will be a long-term asset in the energy infrastructure of Australia. It will significantly enhance grid stability during peak periods of demand when in operation and provide positive system services like frequency control and synchronous inertia. Its achievement will not only be crucial to Australia's renewable energy goals but also serve as a model for future pumped hydro schemes, both domestically and worldwide.

Snowy 2.0 is a key part of Australia's transition to a cleaner, more secure energy system, and its realization will shape the nation's energy storage and grid operating future for centuries to come.

6. References

- Include key resources by using the APA, Harvard, or Vancouver for the citation format.

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Alexandre, et al. "Development of the Geotechnical Baseline Report for the Snowy 2.0 Pumped Storage Project." *Australasian Tunnelling Conference | ATS 2020+1 | Melbourne, Victoria | 10–13 May 2021*, 12 May 2021, www.researchgate.net/publication/351624288_Development_of_the_Geotechnical_Baseline_Report_for_the_Snowy_20_pumped_storage_project.

7. Appendices

- Please upload any drawings and working excel sheets to support your report.

Appendix A – General Manager

Appendix B – Civil Manager

Working spreadsheets

Reservior & Dam design		
Assumed Inputs		
η,cycle	0.85	
Hg, max	694.3	
Hg, min	662.6	
g	9.81	m/s2
ρ	1000	kg/m3
Active Storage (both reservoirs)	240	GL
	240000000	m3
P,design	2000	MW
Final iteration		
Outputs		
Qtotal,des	340.482	m3/s
E	1434561199	J
T=V/Q	704883.4122	s
	195.801	h

Final calculated Inputs	
η	0.87759
Hg	682.3

Q generating (all values must be between the min/max q	
330	max head
418	min head

First iteration

Outputs (max head)		
Qtotal,des	345.458	m3/s
E	1389460932	J
T=V/Q	694730.466	s
	192.981	h

Outputs (min head)		
Qtotal,des	361.985	m3/s
E	1326021624	J
T=V/Q	663010.812	s
	184.170	h

	A	B	C	D	E	F	G
1	**Reservoir Levels & Gross Head**						
2	**Tantangara**				**Talbingo**		
3	HWL-Tantangara	1228.693	m		HWL-Talbingo	543.193	m
4	LWL-Tantangara	1205.833	m		LWL-Talbingo	534.353	m
5	Ha	22.86	m		TWL-Talbingo	538.773	m
6	NWL	1221.073	m		Ha		m
7	TWL				NWL		m
8	Hg	682.3	m				
9	HFR	0.966866					
10							
11							
12	invert upper	1100	m (from map)				
13	invert lower	500	m (from map)				
14	H	600	m > 400m				
15	L	26621					
16	L/H	44.37					
17	while site selection L/H exceed the recommend site ratio, the existing reservoir futilised existing						

Waterway Profile

Turbine Elevation		
LWL-Talbingo	534.35	m
Max pumping head	694.3	
hd	72	m
CL	462.35	m

v	tunnels	shaft/penstocks
min	2	4
max	6	7

Incline shaft		
Qdes (min head)	340.48	m3/s
Qdes, (max head)	340.48	
D	10.00	m
v (max)	4.34	m/s
v (min)	4.34	m/s

Headrace/tailrace-final iteration		
Qdes (min head)	340.48	m3/s
Qdes, (max head)	340.48	
D	10.00	m
v (max)	4.34	m/s
v (min)	4.34	m/s

Headrace/tailrace-first iteration		
Qdes (min head)	361.99	m3/s
Qdes, (max head)	345.46	
D	7.80	m
v (max)	7.58	m/s
v (min)	7.23	m/s

Assume		
Qmax	340.48	m3/s
CF	0.25	
Qave	85.120	m3/s
Qdes	170.241	
Dinner	7.2	m

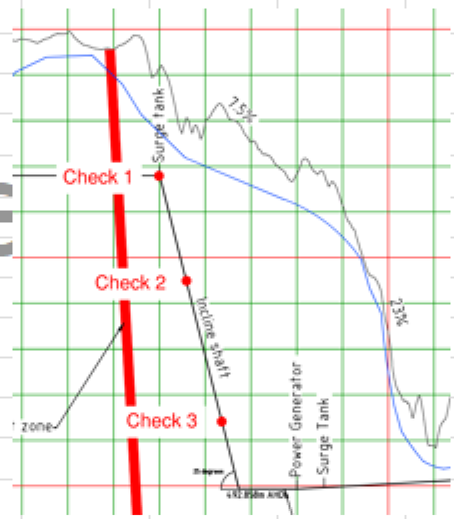
LWL (tantangara)	1205.83
D (headrace)	10.00
Invert (headrace)	1185.83

Empirical chart check

D	10.2494

Let		
Dinner	9	m
Qdes	300	
Qave	150	
Qmax	340.48	m3/s
CF	0.4406	

Confinement check						
Y,r	25	kN/m3	** range is 25-27, is conservative to take 25			
Y,w	9.81	kN/m3			0.906308	
α	25	°			0.997564	
β1	4	°				
β2	12	°				
Check 1					LWL upper	1205.833
Crv	225					
Crm	1005.0					
hs	18.8					
Frv	27.59					
Frm	30.37					
Freq	1.2					
Check 2						
Crv	350					
Crm	3503.2					
hs	255.8					
Frv	3.16					
Frm	3.41					
Freq	1.2					
Check 3						
Crv	650					
Crm	4002.81					
hs	555.83					
Frv	2.701					
Frm	2.915					
Freq	1.2					



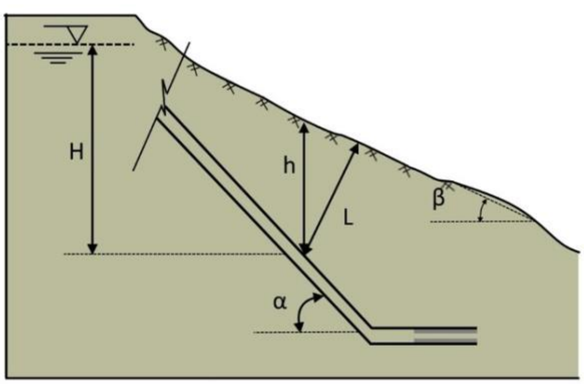


Figure 30: Norwegian confinement criteria parameters

Cavern hall assumptions

Machine hall

- Assumes 6 turbines with 25 spacing between as well as 30m clearance
- Assumes horseshoe shaped to have a height to .width factor of 1.6 to allow for crane movement
- Width is assumed 25m to account for the turbines, generator footprint, cranes and clearance

Transformer Hall

- Assumes 6 transformers for the 6 turbines with a length of 12m each and 30 clearance
- Maintenance clearance of 1.5 and a transformer width of 1.5 is used to determine the total width of 18m
- The height of the crane is assumed to be 15m + 5m clearance
- The draft tube gallery is assumed to be 20m bringing the total height of the transformer hall to 40m

IPB Galleries

$$t_{\text{pillar}} = \begin{cases} \max(20,\ 0.5B_{\text{MH}}), & \text{Good rock} \\ \max(30,\ 0.8B_{\text{MH}}), & \text{Fair rock} \\ \max(40,\ 1.0B_{\text{MH}}), & \text{Poor rock} \end{cases}$$

Underground power station cavern design							
Pdes	2038.398	MW			Machine hall		
N	6				B	25	
CL	462.35	m			L	180	
Y,r	25	kN/m3			H	40	
B,cavern	25						
L,cavern	180				Transformer hall		
he	697.65	m			B	18	
Type	horseshoe				L	82	
H,cavern	40				H	40	
z,crown	502.35						
k	2.4	*assumed					
hs	697.65		>600m				
h,vertical	697.65	m					
σ,v	17441	kN/m2					
σ,h	41859	kN/m2					
B,transformer	18	m					
h,transformer	20	m					
h, draft tube	20	m					
L,transformer	82	m					
t,pillar	30						
Punit	339.73	MW					

Appendix C – Hydraulic manager

Eq. 14 - Reynolds Number		
Inputs	Value	Unit
Q_total	340.482	m3/s
N_pen	6	
D_pipe	9	m
Kinematic viscocit	0.000001	m2/s
Output	Value	Unit
Reynolds Number	8028054.46	
Eq. 15 - Friction factor		
Inputs	Value	Unit
Reynolds Number	8028054.46	
D_pipe	9	m
e looking squiggle	0.003	m
Output	Value	Unit
Friction factor	0.015402834	
Eq. 16 - Major Head		
Inputs	Value	Unit
Headrace	22.13099961	
Penstock	25.89257624	m
Incline shaft	2.998874368	m
Output	Value	Unit
Major Head	51.02245022	
Eq. 17 - Minor Head		
Inputs	Value	Unit
Headrace	2.203114011	
Penstock	1.629663609	m/s2
Incline shaft	0.67051296	m/s
Output	Value	Unit
Total minor head	4.50329058	
Eq. 19 - Effective Head		
Inputs	Value	Unit
h_f Minor Head	4.50329058	
h_f Major Head	51.02245022	
H_g Gross Head	682.3	
h_d Draft Head	72	
Output	Value	Unit
Effective Head	554.7742592	

Headrace		
Eq. 14 - Reynolds Number		
Inputs	Value	Unit
Q_total	340.482	m3/s
N_pen	1	
D_pipe	10.00	m
Kinematic viscocity	0.000001	m2/s
Output	Value	Unit
Reynolds Number	43351494.08	
Eq. 15 - Friction factor		
Inputs	Value	Unit
Reynolds Number	43351494.08	
D_pipe	10	m
e looking squiggle	0.003	m
Output	Value	Unit
Friction factor	0.014966979	
Penstock		
Eq. 14 - Reynolds Number		
Inputs	Value	Unit
Q_total	340.482	m3/s
N_pen	6	
D_pipe	5	m
Kinematic viscocity	0.000001	m2/s
Output	Value	Unit
Reynolds Number	14450498.03	
Eq. 15 - Friction factor		
Inputs	Value	Unit
Reynolds Number	14450498.03	
D_pipe	5	m
e looking squiggle	0.003	m
Friction factor	0.01745463	
Output	Value	Unit
Friction factor	0.01745463	

Incline shaft		
Eq. 14 - Reynolds Number		
Inputs	Value	Unit
Q_total	340.482	m3/s
N_pen	6	
D_pipe	10	m
Kinematic viscocity	0.000001	m2/s
Output	Value	Unit
Reynolds Number	7225249.014	
Eq. 15 - Friction factor		
Inputs	Value	Unit
Reynolds Number	7225249.014	
D_pipe	10	m
e looking squiggle	0.003	m
Friction factor	0.015085396	
Output	Value	Unit
Friction factor	0.015085396	

Eq. 16 - Major Head		
Headrace		
Inputs	Value	Unit
Friction Factor	0.015402834	
D_pipe	10.00	m
L_pipe	15000	m
Gravity	9.81	m/s2
Velocity	4.34	m/s
Output	Value	Unit
Major Head	22.13099961	m
Penstock		
Inputs	Value	Unit
Friction Factor	0.01745463	
D_pipe	4.00	m
L_pipe	1456.42	m
Gravity	9.81	m/s2
Velocity	3.65	m/s
Output	Value	Unit
Major Head	4.315429373	m
Incline shaft		
Inputs	Value	Unit
Friction Factor	0.015085396	
D_pipe	10.00	m
L_pipe	1704.717726	m
Gravity	9.81	m/s2
Velocity	4.34	m/s
Output	Value	Unit
Major Head	2.463305739	m

Eq. 17 - Minor Head		
Headrace		
Inputs	Value	Unit
K	2.3	
Gravity	9.81	m/s2
Velocity	4.34	m/s
Output	Value	Unit
Minor Head	2.20311401	m
Penstock		
Inputs	Value	Unit
K	2.4	
Gravity	9.81	m/s2
Velocity	3.65	m/s
Output	Value	Unit
Minor Head	1.62966361	m
Incline shaft		
Inputs	Value	Unit
K	0.7	
Gravity	9.81	m/s2
Velocity	4.34	m/s
Output	Value	Unit
Minor Head	0.67051296	m

K-Value			
Headrace			
Component	Quantity	K for one	K-total
Bend	1	0.3	0.3
Butterfly valve (sur	1.00	2.00	2
Output			Total
K-Value			2.3
Incline shaft			
Component	Quantity	K for one	K-total
Bend	1	0.3	0.3
Contraction (to pe	1.00	0.40	0.4
Output			Total
K-Value			0.7
Penstock			
Component	Quantity	K for one	K-total
Bend	1	0.4	0.4
Expansion	1.00	2.00	2
Output			Total
K-Value			2.4

Eq. 20 - Q_total - Design Discharge		
Inputs	Value	Unit
P_design	2000	MW
H_e Effective Head	555.3098278	
ρ - density	1000	
Gravity	9.81	
μ_overall	0.87759	
Output	Value	Unit
Q_total	418.3444028	
Eq. 21 - Q_penstock		
Inputs	Value	Unit
Q_total	418.34	
N_penstock	6	
Output	Value	Unit
Q_pen	69.72	
Eq. 6.5 Velocity check		
Inputs	Value	Unit
Q_pen	69.72	
D_pen	4.70	
Output	Value	Unit
Velocity	4.02	4-6m/s
Empirical chart check		
Inputs	Value	Unit
Q	69.72	
Output	Value	Unit
D_pen estimate	5.13	

Eq. 7.1 Inner & Outer Radius		
Inputs	Value	Unit
D_ptun	10.00	m
Output	Value	Unit
r_i	5	m
Inputs	Value	Unit
r_i	5	m
t_l (lining thick)	0.5	m
Output	Value	Unit
r_o	5.5	m
Eq. 7.2 Internal & External Pressures		
Inputs	Value	Unit
gamma_w	9.81	kN/m3
h_s	555.83	m
Output	Value	Unit
p_i	5452.72173	kPa
Inputs	Value	Unit
gamma_w	9.81	kN/m3
h_w	630.00	m
Output	Value	Unit
p_e	6180.3	kPa
Eq. 7.3 Boundary Load p_F		
Inputs	Value	Unit
p_e	6180.3	kPa
p_i	5452.72173	kPa
r_i	5	m
r_o	5.5	m
v_c	0.2	ratio
v_r	0.25	ratio
E_c	30000000	kPa
E_r	60000000	kPa
Output	Value	Unit
num_l	17274.78692	
num_r	9656.71875	
denom	26.21964286	
p_F	290.548129	

Eq. 7.4 Stress Distributions		
Inputs	Value	Unit
p_e	6180.3	
p_i	5452.72173	m
r_i	5	m
r_o	5.5	m
v_c	0.2	m/s
v_r	0.25	m/s
E_c	30000000	
E_r	60000000	
Output	Value (Pa)	Mpa
1_eq	-454.736419	-0.454736419
2_tangential	3317.409134	3.317409134
2_radial	454.7364187	0.454736419
3_eq	3057.673167	3.057673167
Stress_tangential	6210.894012	6.210894012
Stress_radial	3348.221296	3.348221296

Eq. 8.1 Penstock cross-sectional areas		
Inputs	Value	Unit
D_pen	4.70	m
Output	Value	Unit
A_p,single	17.34944543	m^2
Eq. 8.2 Penstock cross-sectional areas		
Inputs	Value	Unit
A_p,single	17.34944543	
N_pen	6	
Output	Value	Unit
A_p	104.0966726	
Eq. 8.3 Surge tank radius		
Inputs	Value	Unit
A_p	104.0966726	
A_s	855.2985999	
Output	Value	Unit
R	8.216387506	
Eq. 8.4 w		
Inputs	Value	Unit
pi	3.141592654	
a	1000	
L	1456.42	
Output	Value	Unit
w	2.157065032	
Eq. 8.4 Stability formula		
Inputs	Value	Unit
Q_0	418.3444028	
w	2.157065032	
H_e	555.3098278	
Output	Value	Unit
A_s	0.349249155	
Eq. 8.7 Equivalent tank diameter		
Inputs	Value	Unit
pi	3.141592654	
A_s	0.349249155	
Output	Value	Unit
D_tank	0.666841687	

Eq 8.5 Rule of thumb stability		
Inputs	Value	Unit
A_s/A_p	8.21638751	
L/H_e	2.62271605	
Over 0?	Value	Unit
	5.59367145	

Eq. 8.8-8.10 Surge Tank		
Inputs	Value	Unit
H_0	555.3098278	MW
h_rise	83.29647417	
h_drawdown	83.29647417	
Freeboard	1.5	
Submergance	1.5	
Output	Value	Unit
H_s,top	640.106302	
H_s,bottom	470.5133537	
H_shaft	169.5929483	

Appendix D – Electrical Manager

Turbine Efficiency Estimation	
	Final head
q	0.814549852
N	375
P (hp)	447006.622
Ns (Gross Head)	16.28312354
Ns (Net head)	21.08927036
Turbine Efficiency (interpolated)	0.9
Estimated Generator efficiency	0.98
Estimated Transformer efficiency	0.995
Overall Plant Efficiency	0.87759

Power Generation and Pumping		
	Min Head	Max Head

P (Water)	997	997
g	9.81	9.81
Q	340.48	340.48
He	555.31	555.31
Overall Efficiency	0.87759	0.873873
Pump Efficiency	0.92	0.92
P_{gen}	1622.87527	1616.001642
P_{pump}	2010.044393	2010.044393
Round Trip Efficiency	0.8073828	0.80396316