NTNU – Trondheim
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Study on the development of pumped storage hydropower project at Snåsa

Torstein Jørgensen Dahle

Hydropower Development
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Supervisor: Krishna Kanta Panthi, IGB

Norwegian University of Science and Technology
Department of Geology and Mineral Resources Engineering

NTNU
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TGB4910 Rock Engineering - MSc thesis

for

Torstein Dahle

STUDY ON THE DEVELOPMENT OF PUMPED STORAGE HYDROPOWER PROJECT AT SNÅSA

Background

The demand for green energy is increasing around the world. The sources of green are however limited to hydropower, wind, solar energy etc. The energy produced using wind and solar are uneven and vary over the period of the day, over the month and over the seasons. The balance may be made by developing pumped storage hydropower projects. Nevertheless, pumped storages projects required both head and tail reservoirs to store water so that it can be re-used when it is needed. It is not that easy to find such localities that consists natural reservoirs at different elevations that can be used as head and tail reservoirs. Norway on the other hand is an exception. There is a huge potential to convert Norway a green battery in the future. This can be done converting existing head and tail reservoirs pumped storage power plants.

MSc thesis task

Hence, this MSc thesis is to focus on the study of pumped storage project at Snåsa. The existing reservoir of Bogna hydropower project will be converted to head reservoir and Snåsa Lake as tail reservoir. The thesis shall address following main issues:

- Review existing hydropower and underground excavation for Norwegian hydropower
- Present two pumped storage project cases developed worldwide
- Present topographic, hydrological and geological condition of Snåsa (Bogna) area
- Assess maximum possible regulation potentials at Ytrebangsjø, regulation requirement at Snåsa and optimize installed pumped storage capacity
- Make a conceptual design and placement of underground structures for the optimized installed capacity

- Review engineering geological and construction aspects of the underground structures. Use field mapping and laboratory tested data for this evaluation
- Carry out stability assessment of the selected section using Phase 2 numerical modelling
- Discuss your findings and conclude your work.

Relevant computer software packages

Candidate shall use *roc-science package* and other relevant computer software for the master study.

Background information for the study

- Relevant information about the project such as reports, maps, information and data collected by the candidate.
- The information provided by the professor about rock engineering and hydropower.
- Scientific papers, reports and books related to Norwegian hydropower and tunnelling.
- Scientific papers and books related to international hydropower and tunnelling cases.
- Literatures in hydropower engineering, rock engineering and tunnelling.

The project work is to start on January 14, 2014 and to be completed by June 10, 2014.

The Norwegian University of Science and Technology (NTNU)
Department of Geology and Mineral Resources Engineering

January 14, 2014

Krishna Panthi

Dr. Krishna K. Panthi
Associate Professor of geological engineering, main supervisor

Preface

The goal of this thesis was to review the possibility of constructing a pumped-storage power plant between Ytter-Bangsjøen and Snåsavatnet.

Working with this has been interesting and educational. I wish to thank everybody that has assisted me in my work, especially my supervisor Krishna Kanta Panthi and my fellow students.

Abstract

An increasing power demand and a transition towards more renewable energy sources have led to an increasing development of wind farms. Wind farms produce unregulated power which may not be available when it is needed. Pumped storage power plants can have a stabilizing effect of the power grid as well as increase the utilization of the wind power by using surplus power to store water and produce power from this water when needed.

The two lakes Ytter-Bangsjøen and Snåsavatnet is suited for a pumped storage power plant. With the construction of a 1 km low dam the magazine capacity is increased to 260 mill m3 and a head of 296.8 m. The water tunnels that's need constructing will have a total length of 6.5 km which because of good rock quality and topography can be unlined.

A review of the topographic and geological conditions was done for the area, and the underground elements was located and orientated on basis of these findings in the area.

The headrace and tailrace tunnel was optimized towards an economical optimum, based on net present value and given values for the power price.

The selected tunnel profile and the power cavern were simulated with the finite element software Phase2 to investigate the stability of the openings. Results showed that only a 70 m stretch of limestone needs support.

The construction costs was calculated using NVE cost base for hydropower plants and gave a cost of 1122 mill NOK. For calculating profitability a series of sale power prices was set, and for each value a corresponding maximum buy price that gave a positive NPV was found. The result gave an equation that found the price variation in the power market to be too small for the project to be profitable from operating solely on the power market, and should rely on buying surplus wind power.

Sammendrag

Økende forbruk av kraft og et ønske om mer fornybar energi har før til en økt utbygging av vindkraftverk. Kraftproduksjonen fra disse er ikke regulert og styres av hvor mye vind det måtte være. Pumpekraftverk kan brukes til å øke utnyttelsen av denne kraften ved å pumpe vann til et høyere magasin når det er overskudd av kraft, for så å produsere kraft av dette vannet når det er mangel på kraft.

Ytter-Bangsjøen og Snåsavatnet er velegnet til bygging av et pumpekraftverk. Ytter-Bangsjøen kan med en 1 km lang demning med en høyde på 2-14 m øke magasinet fra 150 mill m3 til 260 mill m3. Nødvendige tilløps og avløpstunnel vil ha en total lengde på 6.5 km og kan på grunn av god fjellkvalitet og tilstrekkelig overdekning konstrueres uforet.

Det ble gjørt en vurdering av de topografiske og geologiske forholdene, for så å plassere tunnelene og kraftstasjonen etter disse funnene.

Tilløps og avløpstunnelen er økonomisk optimert på bakgrunn av netto nåverdi og gitte verdier for kjøp og salg av kraft.

Stabiliteten til det valgte tversnittet for tilløpstunell, avløpstunnel og kraftstasjonen ble simulert i dataprogrammet Phase2 for de forskjellige berartene. Resultatet viste at hele strekningen med unntak av 70 meter i kalkstein kan drives uten bergsikring.

Byggekostnadene er beregnet fra NVE kostnadsgrunnlag for vannkraftanlegg og ga en kostnad på 1122 mill NOK. Inntjenignspotensialet ble beregned ved å bestemme et sett med salgspriser for kraft og for hver av disse baregne høyeste innkjøpspris som ga en positiv netto nåverdi. Resultatet gav en ligning for nødvendig prisforskjell som viser at prisforskjellene i det frie kraftmarkedet er for små til å være lønnsomt, men med å kjøpe overskuds kraft fra vindfarmer kan det være mulig.

Table of Contents

Figures

Tables

Equations

1. Introduction

1.1. Background

Europe has decided to reduce their emissions and parts of this are decided to be accomplished by increasing the production of renewable energy. The demand for renewable energy has further increased after the 2011 earthquake in Japan with the following Fukujima meltdown which leads to Germany deciding to phase out nuclear power. Much of the renewable power is likely to come from solar and wind power. Several countries including Norway have planned to build large offshore wind farms. The main drawback with unregulated power sources like wind and solar is the uncertainty of when they will produce power. It can produce at full capacity when there is little use for the power, and can produce nothing at all when the need is greatest. This creates a need for balancing, that can be accomplished by creating pump storage power plants that can act as “batteries” which can store the surplus wind power by pumping water from a lower reservoir to a higher one, and utilizes the potential energy to deliver energy to the power grid when the wind is weak and the wind farms are unable to generate sufficient amounts of power.

“In Norway, with its long history of hydropower generation, we find half of Europe`s reservoir capacity. New pumped-storage power plants in connection with existing reservoirs could be part of the solution in securing a reliable energy system“ (Statkraft, n.d.).

Several major project are already under planning and construction on the Continent, in the North Sea and along the UK coast line (Statkraft, n.d.). Construction of a pumped storage power plant at Ytter-Bangsjøen would contribute to turning the energy production from this unregulated power source in to controlled reliable power.

1.2. Scope

The scope of this thesis is to look at the possibilities of establishing a pump storage power plant between Ytter-Bangsjøen and Snåsavatnet. It shall also look at the stability of the selected tunnel profile and turbine cavern.

The title of the assignment is: Study on the development of pumped storage project at Snåsa. And the thesis shall address the following main issues:

-Review existing hydropower and underground excavations for Norwegian hydropower

-Present two pumped storage project cases developed worldwide

-Present topographic, hydrological and geological conditions of Snåsa (Bogna) area

-Assess maximum possible regulation potential at Ytter-Bangsjøen, regulation requirement at Snåsavatnet and optimized installed pumped storage capacity

-Make conceptual design and placement of underground structures

-Carry out stability assessment of the selected section using Phase2 numerical modeling

2. Hydropower in Norway

2.1. Development history

The first Hydro power plant in Norway was built at Skien in 1885. It was a simple above ground construction for supplying power to a wood processing plant. The general design of that era was to bring the water down to the powerhouse through an above ground steel penstock. The First World War led to a shortage of steel and subsequently high prices. This led to the natural solution of bringing the water in a shortest possible way to the powerhouse. This was done by excavating an underground pressure shaft. The first of these shafts was still steel lined, but the length of lining needed was much shorter. The first power plants whit unlined pressure shafts where putt into operation in the years 1919-1921. One was a complete failure due to low overburden, but the remaining three remained in operation with some repairs doe to leakages.

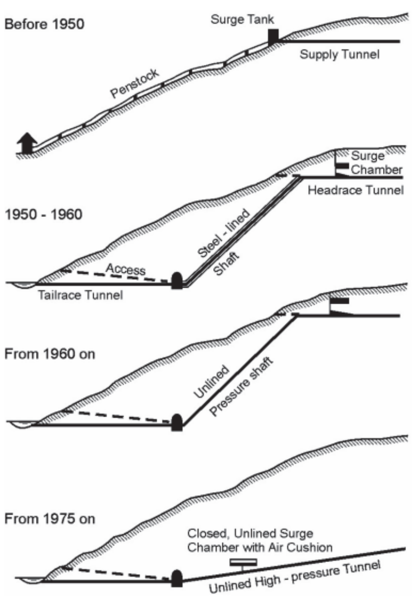

Figure 2.1 Evolution of powerhouse-pressure shaft design

During and after World War II underground constructions became the preferred way of constructing hydropower; this was because of wartime experience which led to the demand of improved wartime security. The advances in rock excavation techniques and equipment quickly led to placing the power plant and tunnels underground as the most economical solution. This also gave the designer freedom in planning the design quite independent of the surface topography. From the mid-70s the design of the layout changed from a close to level headrace tunnel and pressure shaft with an inclination of 45° to a direct tunnel with slight inclination that went more or less direct from the intake to the underground powerhouse. The next step in the development of hydropower came with the increasing calculation capacity of computers. It was now possible to use Finite Element Models (FEM) which meant it was now not only necessary to rely on rules of thumb, but could do advanced stress analyses. (Broch, 2005)

2.2. Use of underground for hydropower

In today's hydropower, close to all larger projects are constructed as an underground structure. This greatly decreases the restrictions of the design and gives good possibilities for shortening the waterways by construction them in a straight line and furthermore reducing the losses. One of the main reason for going underground was because of safety, and in some cases the only option because of terrain restrictions, but mostly it is placed under ground on the basis of being the most economical solution . In Norway the rock is primarily Precambrian and Paleozoic and can generally be classified as hard rocks, which in turn is favorable for water tunnels. Because of the generally good rock conditions Norway has a long history of using unlined tunnels and pressure tunnels. Most of the tunnels have only a 2-4% concrete or shotcrete lining. The reason for this low number is not only the good rock conditions in Norway, but to a great extent the philosophy of excepting some falling rocks during the operation period. A technology being more and more used is air cushioned surge chambers, which replaces the traditional solution of surge shaft and surge chamber and are in many cases and economically sound alternative. Since the 1970, none of the unlined pressure shafts that were built with heads of 150-1000 m have experienced any unacceptable leakages, which indicates that this is a technology that works. (Broch, 2005)

Because of the good experiences with unlined pressure shafts and tunnels alongside the air cushioned chambers, the technology is being adopted for some projects outside of Norway.

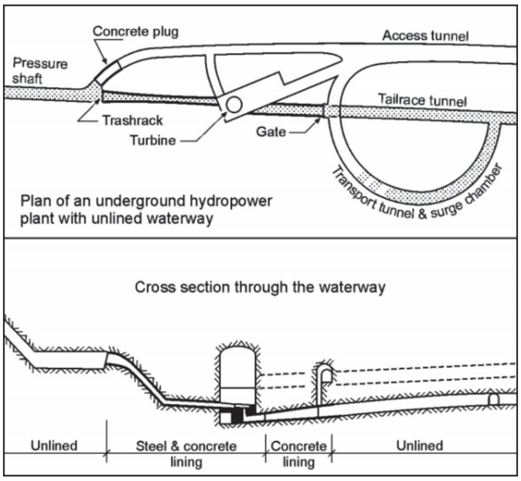

Figure 2.2 Principal underground powerhouse layout

2.3. Future development

Today in Norwegian hydropower the most attractive and economical hydropower project have already been developed. For the future, a changing power market and government incentives can make earlier uneconomical projects viable or make it economical to upgrade already existing plants. New technology can also be used to refurbish older power plants by for example replacing turbines, build new intakes, reduce friction losses or increase magazine capacity.

Some new technology is on its way, as wave power, tidal power and osmotic power. They are all promising technologies, but are still under development. At the moment the development of renewable power sources as wind and solar power is increasing. To compensate for their unpredictable power production and to utilize surplus power, the construction of pumped-storage power plants is necessary. In countries like Norway with its beneficial topography can also make it profitable to offer regulating services to nearby countries. This will again bring a need of expanding the power grid by constructing higher capacity transmission lines, which again will contribute to a more efficient power market (Vattenfall, 2013).

Another development that has been increasing, is the development of small and mini hydro. The development of more efficient turbines and new drilling technology from the experiences from the North Sea together with government incentives have made this more attractive (Jensen, 2008).

3. Developed pumped storage project cases

3.1. Limberg II

The Limberg II is a pumped storage power plant located in the Kaprun-valley in the central alpine region of Salzburg, Austria. The power plant was constructed, mainly to supply balancing power and handle the peak demands of the power grid, as well as increase the utilization of unregulated renewable power sources as wind and solar.

Figure 3.1Limberg II layout

The plant uses the two reservoirs Wasserfallboden and Mooserboden with a mean gross head of 365m and a volume of 81.2 and 84.9 million m3 which approximately equals 6.5 days of full production. (ILF Consulting Engineers, n.d.)

Most of the headrace located in rocks comprised of basalt and gneiss with a compressive strength on average of 100MPa and in some locations peaked at 150MPa.

The tunnel system includes of a 5.4 km headrace tunnel system which was excavated by TBM, a 0.4 km tailrace and a 5.5 km access tunnel that was excavated by drill and blast. The headrace system uses a 7m diameter for the pressure tunnel and a 5.8m diameter for the 45° pressure shaft. The pressure tunnel and tailrace tunnels have been concrete lined, while the pressure shaft has been fitted with a steel lining. The cross section for the access tunnel is between 30 to 40m2. The power station consists of a turbine cavern and a transformer cavern with the following dimensions:

	Turbine cavern	Generator cavern
Length	62 m	61 m
Width	25 m	15 m
Max. height	43 m	16 m
Excavation	56900 m3	12500 m3

Table 3.1 Limberg II Dimencions of cavens

The power plant has two 240 MW reversible Francis pump turbines with a maximum combined flow of 144 m3/s with an expected annual energy production of 1300 GWh (Dr Herbert, 2008).

3.2. Venda nova II

The Venda nova II is a pumped storage hydropower plant located in the north of Portugal. The plant was constructed to cover the need of peak power. The power plant utilizes the power production from thermal power plants and wind power plants to pump up water during off-peak hours and to produce power during peak hours. The plant uses the height difference of 420 meters between the Venda nova reservoir and the Salamonde reservoir.

The tunnel system consists of a 2.8 km 15 percent inclined unlined tunnel of 6.3 meter diameter, but with the stretch upstream of the power house is steel lined; a 1.4 km sub-horizontal unlined tailrace tunnel with the same cross section, a 420 meter vertical unlined shaft of 4.5 meter diameter to the surge tank 500 meters upstream of the powerhouse and a 1.5 km unlined access tunnel with a 11 percent slope of 8 meter diameter which also contains the power and control cables.

Because of good quality granite in the project area, rather than the traditional design used in Portugal of a reinforced concrete lining, the tunnels are only supported with fiber reinforced shotcrete and rock bolting which led to an increase in friction and hence the need for a larger cross section, but in all a reduction in time and construction costs. Also for the cavern the norm of cast-in-place reinforced concrete arc structure was abandoned in favor of cement-grouted rock bolts and fiber reinforced shotcrete. The powerhouse complex has an overburden of 350 meters and consists of two caverns, one for the transformers and one larger for the turbines that is 20 by 60 meters with a maximum height of 40 meters.

The surge shaft was intended to be excavate with traditional excavation methods of drill and blast, but was created with pilot hole drilling and back-reaming for safety reasons.

The plant has two 97.1 MW reversible Francis pump turbines with a maximum flow of 50 m3/s. Each of the turbines are coupled directly to a synchronous motor. The annual average power production is 220 GWh (Energias de Portugal, S.A, 2007)

4. Project case for this study

The project area is located approcimalely 35 km north of steinkjer. The planned pumped storrage plant will make use of the elevation difference between the Snåsavatnet lake and the Ytter-Bangsjøen lake. There is an existing power plant in operation between the two lakes, Bogna power plant. It has an installed capacity of 56 MW (Rosvold, 2010) and a yearly production of 145 GWh. (Nord-Trøndelag Elektrisitetsverk, 2012) It gets it water from the catchment draining into Ytter-Bangsjøen. As a result of this, the water draining into Ytter-Bangsjøen will be looked on as belonging to Bogna power plant and the project being planned will only use the water that it have already pumped up.

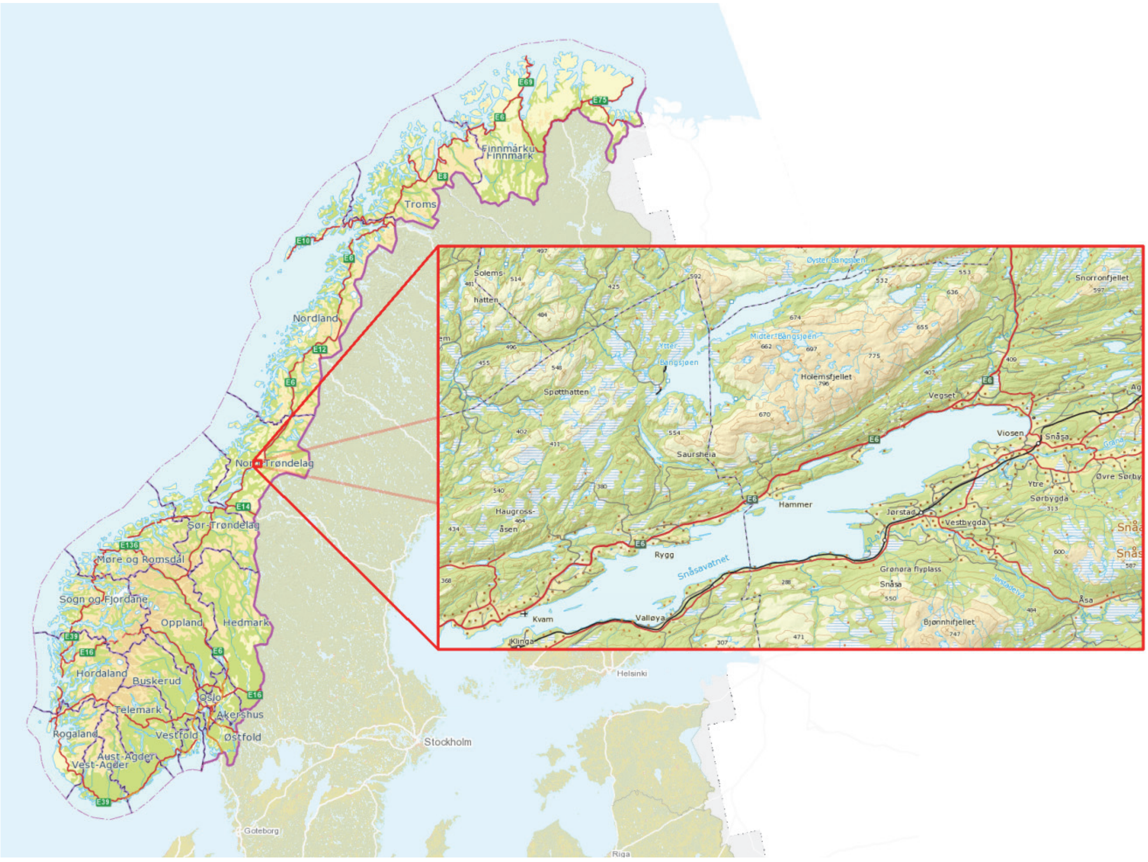

Figure 4.1 Project location

4.1. Tophography

The topography for the area is mostly gentle slopes between from 10° to 30° with some localized steeper hillsides linked to rivers and weakness zones. Between the intake and the outlet at Snåsavatnet there is a higher ridge with an elevation of around 640 masl close to tunnel alignment. The elevation at the upper lake Ytter-Bangsjøen is 315m at the highest regulated water level and the elevation at the lower lake Snåsavatnet is 22.43m at highest regulated water level which gives an elevation difference of 292.57m, and the horizontal distance between the two lakes is from 6 to 8 km depending on the decided tunnel placement. At the upper dam there are constructed 4 dams for the existing regulation. One is 250m long and the 3 others which are constructed in sequence are 270m, 60m and 80m. The two largest dams are rock fill dams, while the two smaller ones are concrete weirs. The area is accessible from the E6 which is runs right by the project. There is also existing transmission power lines running next to the project by a few hundred meters.

Figure 4.2 Topographic map with tunnel alignment

4.2. Hydrology

The catchment area is 143.4 km2 and the average runoff is 51.3 l/s/km2 which give a total runoff of 232 mill m3/year or 7.36 m3/s. (NVE, n.d.)The Bogna power plant uses the same lake as intake as the proposed pumped storage plant. The old power plant will operate as before since the new power plant will only rely on the water that it has pumped up. The new power plant may in some situations act as sort of a spillway when extreme flood event occur, and give an extra safety margin for the operation of the existing power plant.

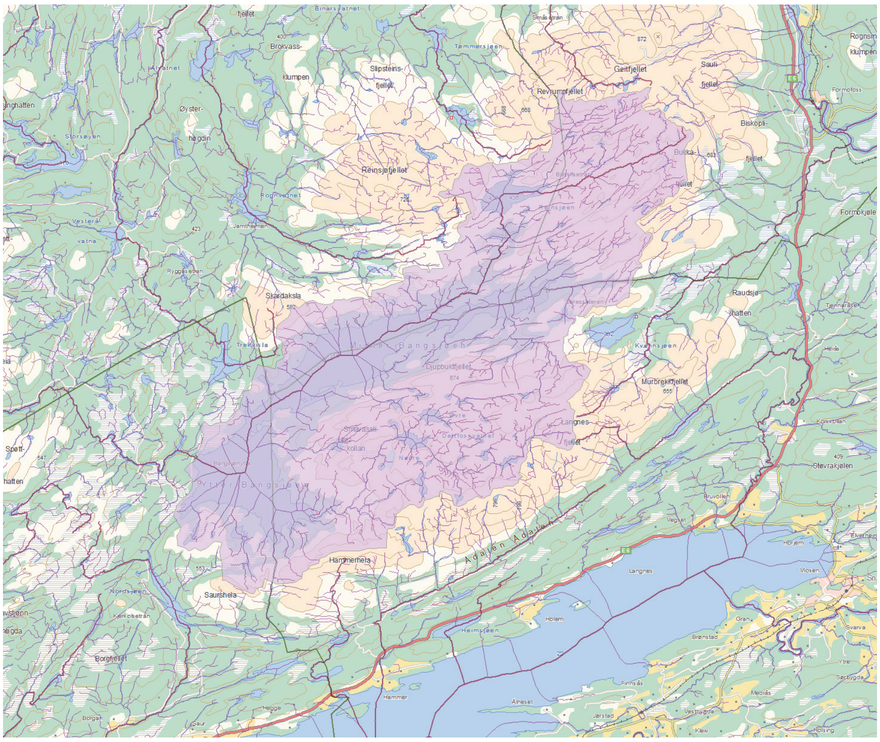

Figure 4.3 Map of the catchment for Ytter-Bangsjøen

4.3. Geology

4.3.1. Soil cover

The soil cover in the area is generally quite thin, with some local deviations. The thickness of the soil cover according to the NGU soil cover map is in the lower elevations between 0.2 and 0.5 meters thick, and in the higher areas, the soil cover varies between bare rock and insignificant cover. This matches with observations at the site.

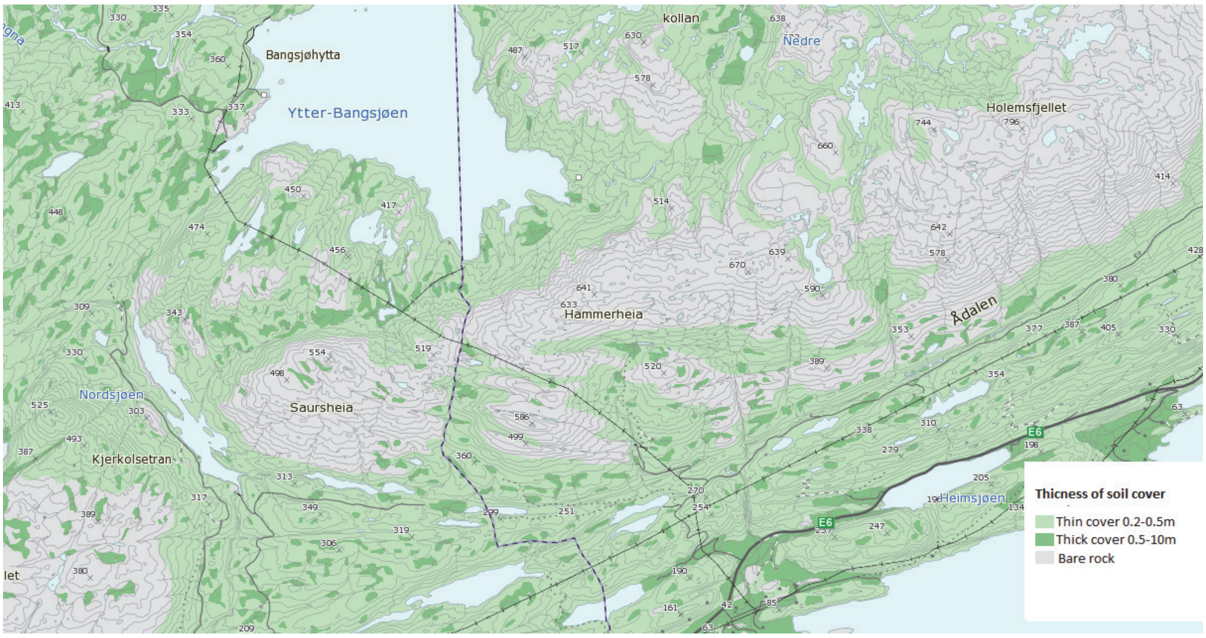

Figure 4.4 Soil thickness map

4.3.2. Rock

The rock types in the project area consist primarily of five kinds of rock, which all are quite massive. These are sandstone, greenstone, amphibolite, limestone/marble and granitic-gneiss. The rock types were mapped using bedrock maps from NGU.no in combination with data from on-site surface mapping. The map below shows the location of the different rock types: 1 is sandstone, 2 is greenstone, 3 is amphibolite, 4 is limestone and 5 is granitic gneiss.

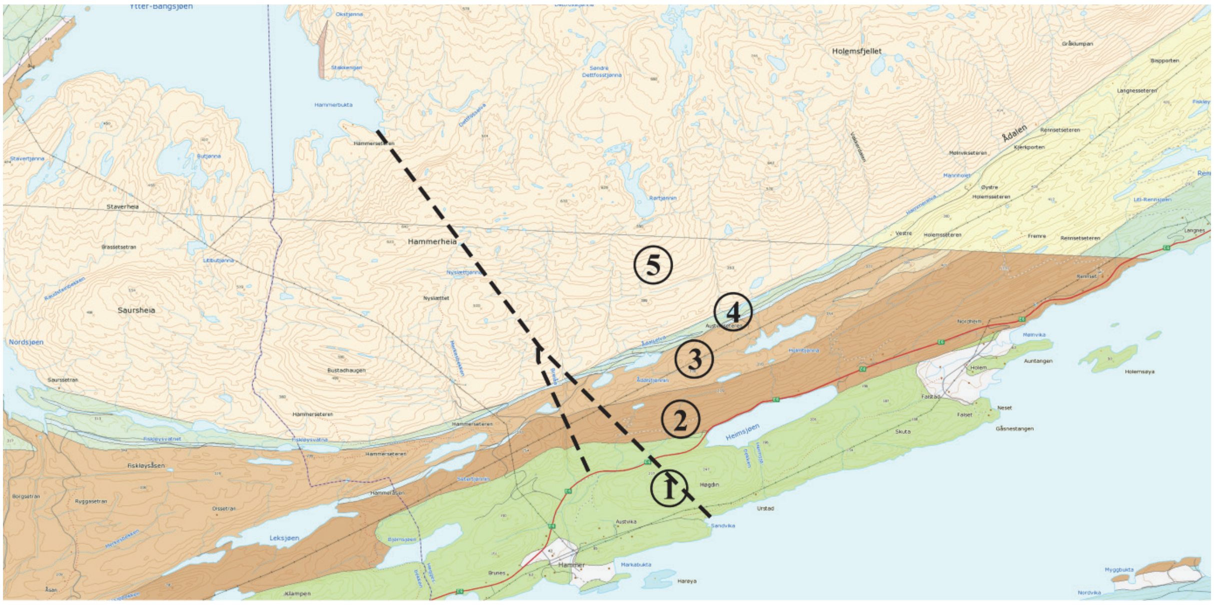

Figure 4.5 Map showing the bedrock

For the different rock types Q-values was estimated using the Q-system. There were done 2-4 samples per rock type, which were averaged. The location of the samples was taken 1-1.5 km west of the alignment.

The weight of the rocks were taken from the geophysics map service at NGU.no at locations close to the tunnel profile.

Rock type	Sandstone	Greenstone	Amphibolite	Limestone	Granitic-gneiss
Q-value	22	47	45	4	253
Density kg/m3	2709	2756	2902	2739	2598

Table 4.1 Q-values and rock densities

As for the direction of the principal stress, the direction was found at the web-site www.world-stress-map.org. The direction shown on the map is parallel to the direction of the tunnel.

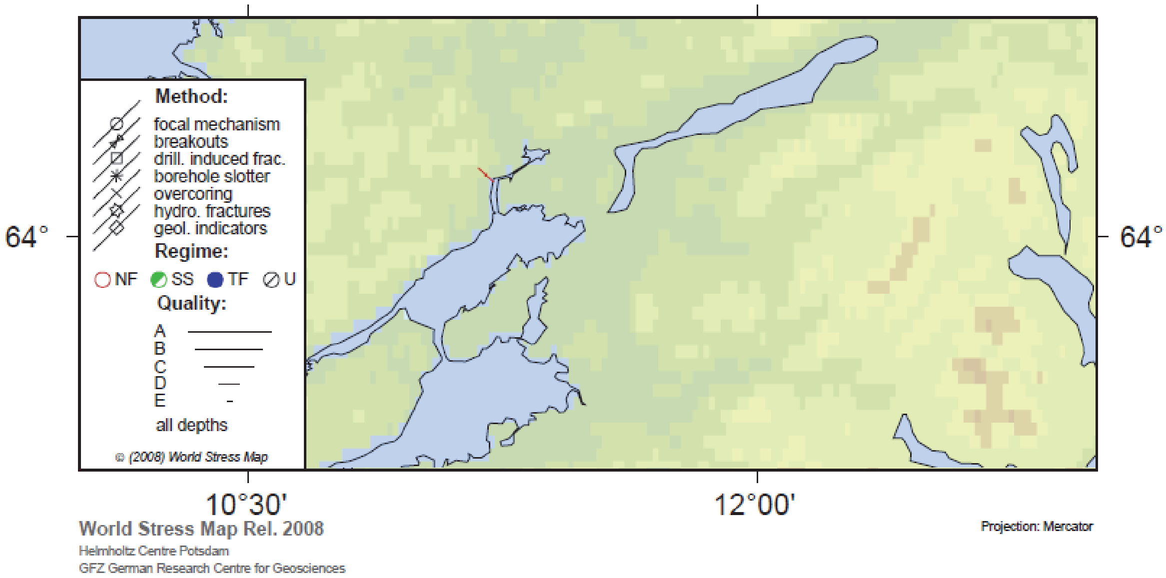

Figure 4.6 Direction of principal stress

4.3.3. Jointing

The joints were mapped along the Bogna power plant tunnel alignment. The mapped rock types are the same as for this project and the distance is from where the samples was taken to the proposed tunnel alignment is between a few hundred meters to a couple of kilometers and should therefore be representative.

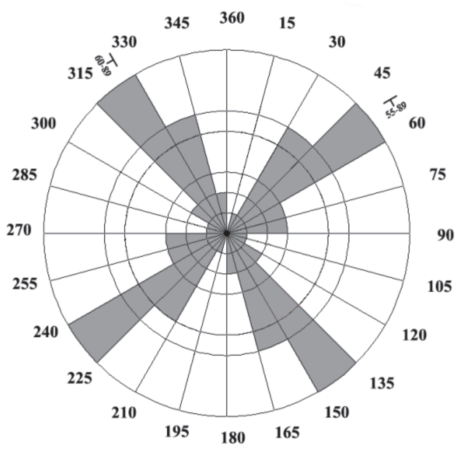

Figure 4.7 Joint rosette of all rock types

4.3.4. Faults and weakness zones

The weakness zones were identified from a combination of field visit, stereoscopic aerial photos, topographic maps and geological maps from NGU.no. It was found 5 weakness zones which are expected to have an influence on the excavation. The weakness zone between the granitic-gneiss and the lime stone is expected to be the one with the largest impact on the tunneling. It is a roughly 30 m wide zone of heavily weathered rock, expected to be of significant width at tunnel level. The weakness zones are expected to cause stability problems, possible water inflow and lead to the requirement of additional support work.

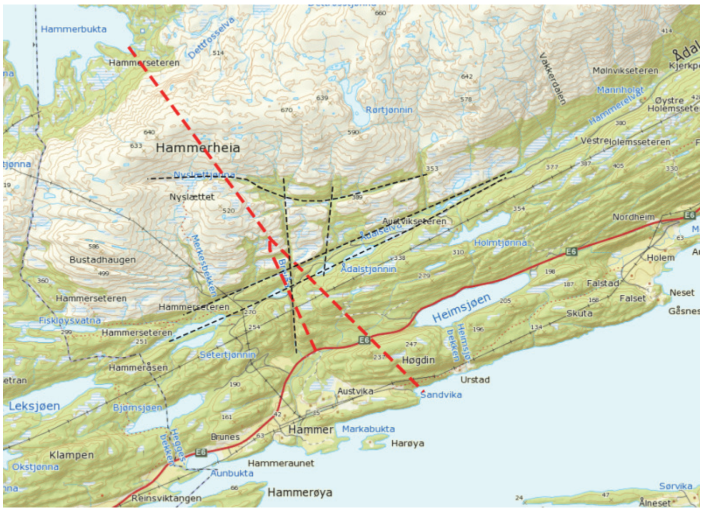

Figure 4.8 Map showing weakness zones and tunnel profile

5. Evaluation of regulation and pumped storage capacity

The project has a prerequisite of having a storage capacity/turbine size that gives a minimum of 30 days of full operation.

5.1. Assessment of maximum possible regulation potential at Ytter-Bangsjøen

Today the regulation of Ytter-Bangsjøen is between 315 MASL and 305 MASL. This gives a storage capacity of 150 mill m3 which is achieved by the existing dams. These dams comprises of 2 smaller rock fill dams and two concrete weirs. Most of the lake is surrounded by high hillsides, with exception for the so south-western side where the terrain in many places is not much higher than the HRWL and will require the construction of dams to increase the regulation. Another issue that will have to be dealt with is a number of cabins situated on the waterfront of the existing water level. From detailed maps there are discovered 20 to 30 cabins or boathouses that will be affected by a water level increase.

5.2. Regulation requirement at Snåsavatet

Today the regulation of the Snåsavatnet is between 22.43 MASL and 21.03 MASL. This gives a storage capacity of 160 mill m3. The regulation requirement at Snåsavatnet is assumed to be be equal that of Ytter-Bangsjøen to be able to handle the inflow when the proposed plant is operating. Without any further regulation, Snåsavatnet has a slightly higher storage capacity than Ytter-Bangsjøen and can cope with the fluctuations. For any further expansion the surface area of the Snåsavatnet is 6 times larger than that of Ytter-Bangsjøen so any increase in the HRWL at Ytter-Bangsjøen will result in a much smaller increase at Snåsavatnet.
The area close to the lake has more infrastructure and is denser populated than area close to Ytter-Bangsøen. No field visits have been done to assess the maximum potential increase in relation to the infrastructure, but from 1 m contour interval maps; and increase of up to 1 meters does not seem to cause any major problems. If the regulation exceeds 2-3 meter a great number of houses and boathouses will be affected. The railway on south side of the lake and the E6 on the north side of the lake will also so close to the new water level that it will most likely cause a problem.

5.3. Optimization of installed pumped storage capacity

The optimization process involves finding the cost to benefit optimum. This is usually presented in in an incremental cost/benefit curve. Because this process is linked to the wind power production for which no production data was found. The result only displays the cost of increasing the regulation level and the obtained volume. The calculation was done by first selecting locations for the needed dam structures up to the maximum height of the calculation. Then for each step the surface area was calculated which was further used to calculate the volume increase. The slope between the two subsequent surface areas is assumed to be linear. Then it was estimated where and how large dams that are needed for each water level would be. The result was then used with NVE cost base for hydropower plants for the dams over 8 meters and with the NVE Cost base for small-scale hydropower plants for the dams under 8 meters to calculate the cost of the dams at each water level.

The map were in details of 1 m contour intervals in the area of the dams, but was only in 10 m contour intervals in the areas covering most of the remaining lake. Therefor there were only exact measurements of the lake area for the water levels at 315, 320 and 330. These values were used to interpolate the values for 317.5 m, 322.5 m and 325 m.

It can be seen a flattening of the curve for the cost between 200 mill m3 and 250 mill m3. This is influenced by most of the dams at 260 mill m3 and down were less than 8 m high and calculated with NVE Cost base for small-scale hydropower plants.

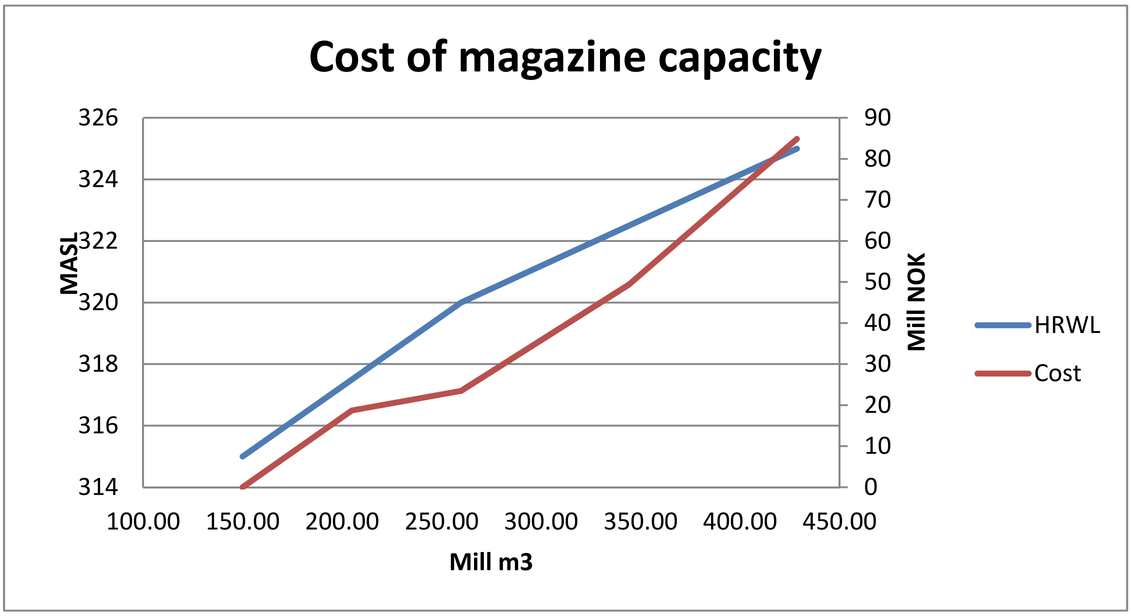

Figure 5.1 Curve showing magazine capacity related to regulation level and cost

Because of the inability of calculating the value of increased storage by simulating the power market, the further calculations in this project will be based on 5 m increase in HRWL and a magazine capacity of 260 mill m3 since this increase seems to be the most realistic water level increase.

6. Design and placement of underground structures

The goal for finding the optimal placement of the underground structures has been to find the shortest possible length of tunnel, but at the same time finding a desirable location for the power house cavern, access tunnel and surge shafts. In this chapter it will be looked closer at the main components: Headrace and tailrace tunnel, access tunnel, turbine cavern and surge shafts. This will be based on the geological conditions and the following physical parameters: Design flow Q=100 m3/s, gross head of 296.82 m, HRWL and LRWL at Ytter-Bangsjøen is 320 masl and 305 masl, HRLW and LRWL at Snåsavatnet is 23.18 masl and 21.03 masl.

6.1. Orientation

Several measurements of the jointing was taken from the powerhouse of the Bogna power plant which is placed in granitic-gneiss, the same rock and located little less than a kilometer south-west from the proposed powerhouse. Measurements from this cavern showed that the rock had two main joint directions which were N58-64°E and N130°E with corresponding dip of 84°SE and 88°SW. The orientation of the main cavern is chosen with respect to these findings, and placed direction N15°E.

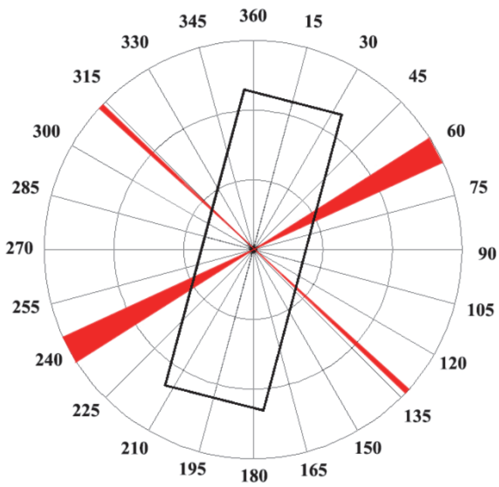

Figure 6.1 Joint rosette for the gneiss

The join-rosette used for the tunnel alignments contain several measurements of all of the rock types that the tunnel goes through. The orientation of the headrace tunnel and tailrace tunnels is N130.5E and N137.7E. The access tunnel is orientated N150.3E. These alignments are quite unfavorable compared to the joint sets found, but in this case the placement of the tunnels is heavily dictated by the topography. Experience from tunnels at Bogna power plant indicates that the unfavorable placement should not cause any significant problems regarding stability.

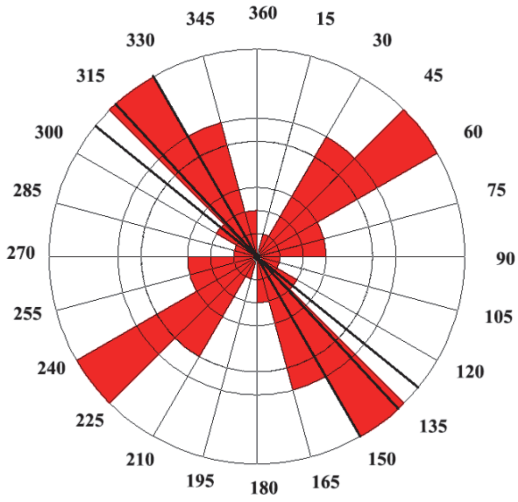

Figure 6.2 Joint rosette of all rock types with tunnel directions

6.2. Location

The powerhouse is located placed in the granitic-gneiss approximately 500m from the fault zone between the granitic-gneiss and the amphibolite and with an overburden of 322m. It was initially desired to have the cavern closer to the fault zone to reduce the length needed for the access tunnel, but the need of bringing the top of the surge shaft above the highest regulated water level forced the placement of the cavern closer to the intake.

Figure 6.3 Map showing tunnel placement

To assure that the overburden over the pressure tunnels would be sufficient, the tunnel cross section was checked with the rule of thumb equation by Bergh-Christensen and Danevig.

$$L > \frac{\gamma_w * H}{\gamma_r * cos\beta} \text{ (Broch, 2002)}$$

Equation 1 required overburden

The head were set to equal to the gross head of 297 meters plus the surge pressure of 16.6 meters and the slope of an average angle of 7.2°. The shortest length to the surface was measured to a modified average slope that is slightly lower than the actual terrain and was found just before the turbine cavern, and measured to be 299 meters. The minimum distance from the calculation is 199 meters.

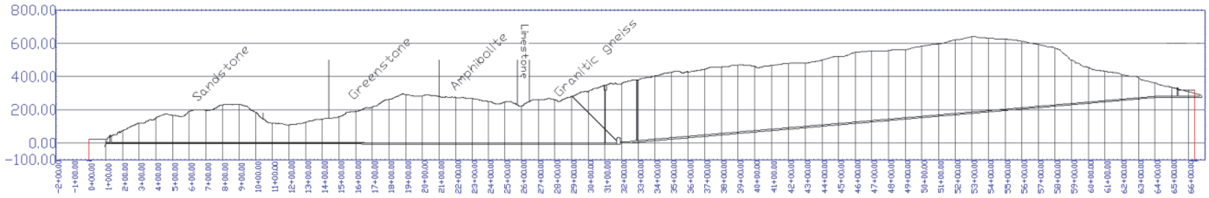

Figure 6.4 Longitudinal view of tunnel profile

6.3. Shape and size

6.3.1. Powerhouse cavern

The powerhouse cavern is not designed from the size of the physical dimensions of the turbine and generator because of problems obtaining the data. The dimensions were assumed based on the similar Portuguese power station Venda Nova II. The length is 60 m, width is 20 m and the height is 40 m. The volume of the cavern is 47219 m3 compared to the volume calculated by the formula ($Blasted\ volume = 78 \times H^{0.5} \times Q^{0.7} \times n^{0.1}$) from the NVE cost base (NVE, n.d.) which is calculated to 35890 m3, indicates that the dimensions are realistic for this project. The design of the power house does accommodate for an overhead traveling crane being installed on anchored concrete crane beams.

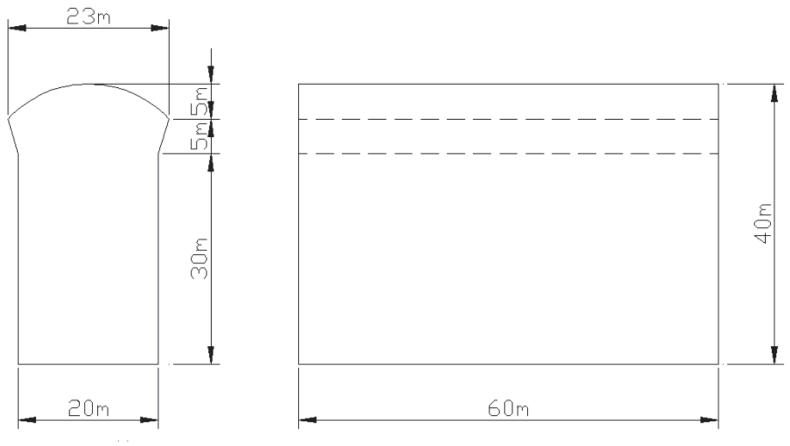

Figure 6.5 Powerhouse cavern cross section

6.3.2. Access tunnel

The access tunnel should be determined based on the size of the equipment needed to be transported through it. The height will usually be determined by the size of the generator while the turbine will be the determining factor for width. Because of the lack of data on the equipment size, the tunnel is assumed to be able to handle the same size equipment as the similar power plant the Venda Nova II. Based on that project the access tunnel cross-section is set to 57 m2 with an equal height to width design of 8m.

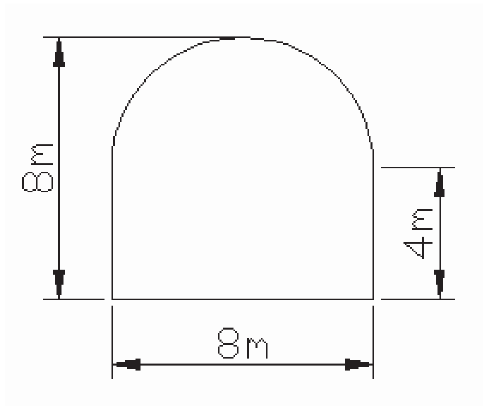

Figure 6.6 Access tunnel cross section

6.3.3. Headrace and tailrace tunnel

The size of the headrace and tailrace is based on the economical optimum. The calculations are heavily affected by the price selected for buying and selling power. Here the selected price for selling is based on the average price from Nordpool in the Trøndelag area from the last 10 years, which is 321.8 NOK/MWh. The price set for buying surplus wind power is set to 100 NOK/MWh. The assumption is also made that the power plant is producing at an average of 12 hours a day and pumping an average of 12 hours a day. The result from the calculation gives an optimum cross section of 63 m2 and a width and height of 8.4 m. The shape chosen is that of equal height to width since this is the shape that gives the greatest area to circumference and hence the least head loss while still retains good constructability.

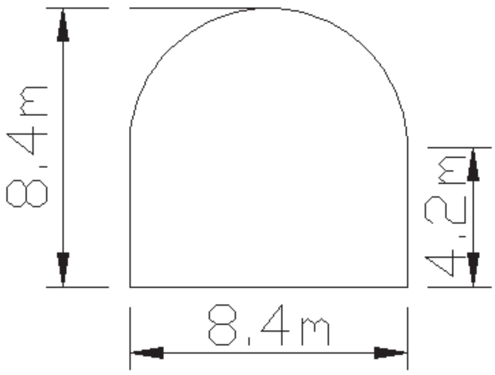

Figure 6.7 Headrace and tailrace cross section

The optimization is done by calculating the NPV for costs and benefits on a range of different tunnel cross sections, with a 50-year period and a 10% interest. For the benefits only the income from power sales was included. For the costs; costs of constructing the water tunnel with each cross section was included, together with 5% O&M for the same tunnel and the cost of pumping water.

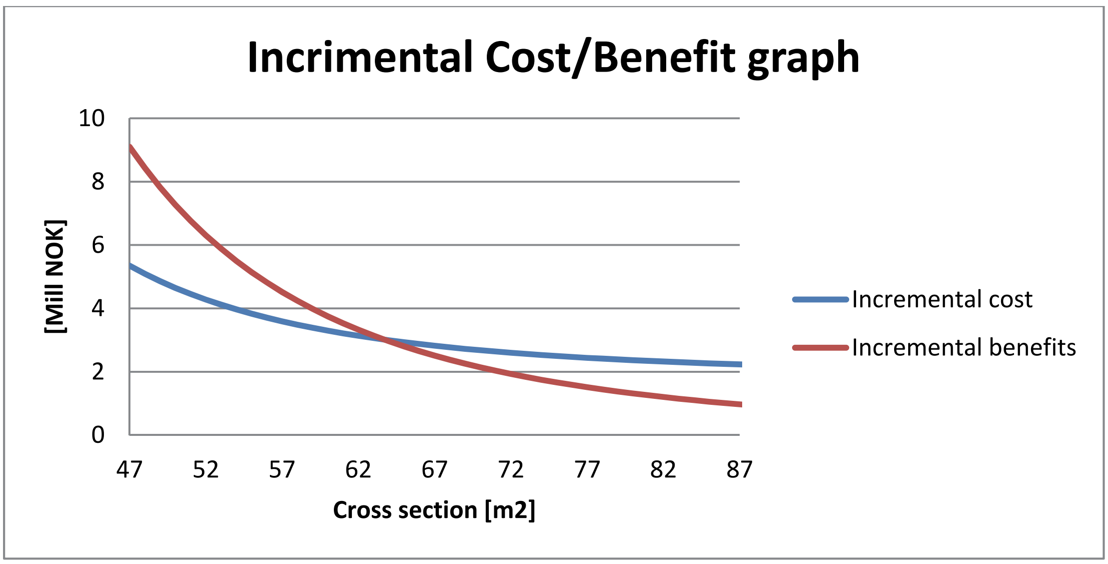

Figure 6.8 Curve showing headrace and tailrace tunnel optimum size

6.3.4. Dams

To accommodate for the increase HRWL the construction of three dams is needed. The dams will be constructed as rock fill dams with a side slope of 1:1.5 due to expected good ground conditions, and a freeboard of 3m. Figure 6.9 illustrates where the dams will be placed and figure 6.10 shows a lengthwise cross section of the dams. The height varies from as little as 1 m to 14 m and the total length of the dams are 1066 meters. The height of the dams is determined from a terrain profile along the suggested alignment.

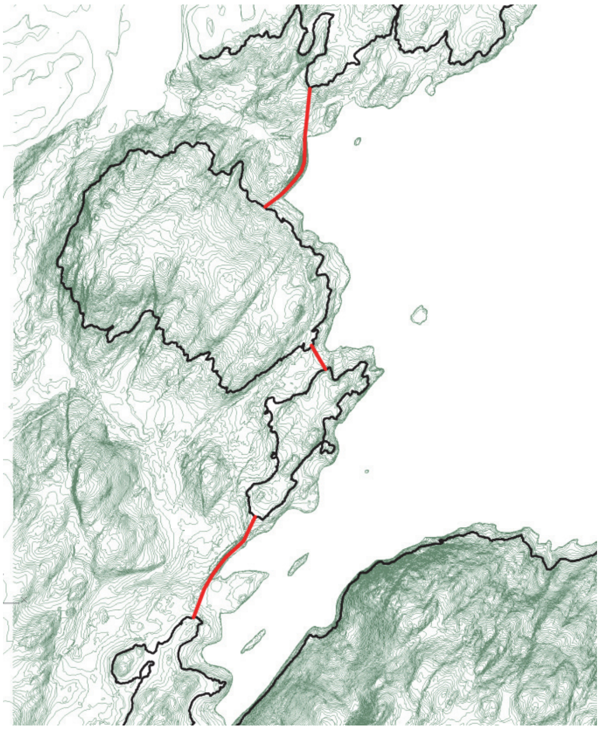

Figure 6.9 Placement of dams on map

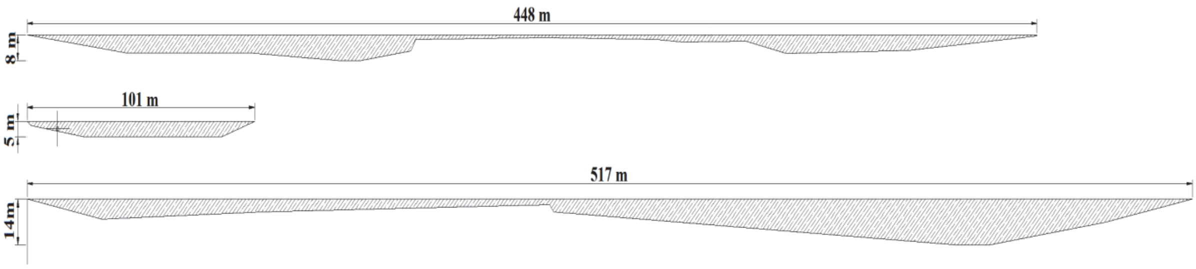

Figure 6.10 Longitudinal view of dams

6.3.5. Surge shafts

The water tunnels are more than 3 km in both directions of the power plant; because of this the plant is planned with two surge shafts. One is 68 meters downstream of the turbine cavern and the other one is100 meters upstream of the cavern. The upper and lower surge shafts have lengths of 365 m and 335 m. The diameter of the shafts is set to 3 m, and is set to 20 m for the surge tang which gives an area of 200 m2. The maximum surge height from a immediate stop is calculated to be 16.6 m which gives a maximum pressure at the turbine of 313.4 m

$$\Delta Z = \Delta Q * \sqrt{\frac{\sum l/_a}{g * A}} = 16.6m \quad \text{(Anon., 2006)}$$

Equation 2 Maximum surgepressure

7. Evaluation of construction aspects

The method of tunneling decided upon is drill and blast. This method gives much flexibility in the design and is not as affected by variations in the rock conditions as TBM. The drill and blast method of tunneling is a well-known and used method in Norway with much expertise and available equipment. It is not planned to provide any adits to allow excavation of the tunnels in several shorter sections. This is because of the tunnel being wide enough for direct loading of the dump trucks and allowing the trucks to pass at any point and leading to higher efficiency. For the surge shafts and the cable/emergency shaft the decided method is the pilot hole and back-reaming method. The reason for this is the much lower price compared to drill and blast for small cross-sections.

The construction plan is to start from two points, the access tunnel and through the tailrace tunnel via a short adit tunnel to the powerhouse. The crew from the access tunnel will most likely reach the powerhouse first and will continue to excavate the cavern. The crew working on the tailrace tunnel will bypass the powerhouse and continue to excavate headrace tunnel. The crew working on the powerhouse will also bore the cable/emergency tunnel. When the tunnel works permits it the work will start on the surge shafts.

The amount of access road needed is about 7-8km. To the access the cable/emergency tunnel there is only needed about 1 km of road in quite easy terrain. It will be an extension of the existing road to the adit for the headrace tunnel of the Bogna power plant. The road to the intake at Ytter-Bangsjøen is responsible for the rest of the distance needed to be constructed. This road will extend from the existing road at the north side of the lake Nordsjøen. It will be more challenging to construct since the terrain is steeper and the ground do in many places consists of marsh. The standard for the roads does not need to be high since they will only be used by heavy construction machinery. For both the roads building material can be obtained from the already excavated rock, and construction should therefor commence sometime after the initial tunnel excavation have started.

The spoils from the excavation need to be stored at an appropriate location. An approximation of excavated volume based on cross sections of tunnels, shafts and cavern and the length of these gives a volume of 640000 m3. With an expansion factor of 1.5 this will be 960000 m3. Roughly 50000m3 of this can be used as support filling for the dams, and some may be used for the construction of the temporary roads. For the remaining volume an investigation to find proper nearby locations for deposition or projects that could utilize the abundant rock should be initiated. During construction the 40000-60000 m2 area Korsvollmoen next to the entrance of the access tunnel could be possible temporary site for storing the excavated rock mass.

Under construction of the underground structures the inflow of some water should be expected. The tunnels are planned with a certain inclination which causes the water to collect at the excavation face. This means that the pumping equipment will have to move with the progression the tunnel. For the Headrace tunnel the water will drain towards the main cavern and can be pumped to the surface from there.

At a peak the project should expect to be able to accommodate up to 200 workers including engineers. Living quarters for the staff could practically be arranged at the city of Steinkjer. Transportation from Steinkjer to the construction site could be arranged with buss, and would be approximately half an hour drive. Facilities for all work related personal equipment should be located at site.

The project will need power during the construction period. High power electricity lines are located only a few hundred meters from both the excavation sites and can be accessed via on site transformers.

While mining some events should be expected to occur. In this project running into weakness zones with potential large inflow of water is a likely risk. Rock conditions could be different at the face than estimates. This may lead to changes in advance rates which can lead to deviations regarding the budget. Calculations show that there is not likely to be any problems with squeezing, and the moderate overburden should not lead to any spalling or rock bursts.

8. Construction cost and profit possibility

Determining the development costs are important to be able to conclude if the project is viable or not. It is common to base the choice of installed capacity on the amount of inflow from the catchment into the reservoir, which again has a ripple effect on other parts of the project and will be very influential when it comes to total cost. In this project it was decided that the power plant should be able to have full production for one month, and because of this the installed capacity is only based on magazine capacity.

For estimation of the construction costs the NVE cost base for hydropower plants with the price level dated to 1. January 2010 was used. The estimate is not including tax interest during construction.

8.1. Civil work

The cost in this section is considered to be an average with a 90% chance of being within the deviations listed for each part.

The parts making up the costs related to the civil work is:
The rock fill dams, headrace tunnel, tailrace tunnel and access tunnel. The drilled shafts for emergency/cable tunnel, both surge shafts and both gate shafts .Underground power station, roads and planning and construction management.

-The rock fill dam cost is estimated with a with an inclination of 1:1.5. For the stretches of dam being lower than 8m the NVE cost base for small hydropower is used since the regular cost base has a lower limit of this height. The dam was divided into different sections of equal height which was calculated and then the cost was summed. The uncertainty for the dam foundation is +70% to -30% and ±25% for the dam body.
-The headrace and tailrace tunnel are estimated from figure B.4.1 in the cost base and corrected for length. The cost of underwater tunnel piercing, plugs and cross cut plugs are also added with values from sections B.5.4.2 and B.5.2 in the cost base. The protection work is calculated as 45% of the basic price, and the uncertainty is stated as +30% to -20%.
-The access tunnel is estimated from the figure B.10.4 in the cost base, but 3500 NOK/m for concrete cable channel is subtracted since this is planned in a separate tunnel. The uncertainty is ±25%.
-All the shafts costs is estimated with the figure B.8:1 from the cost base and corrected for

length. The rock is assumed to be of medium drillability and the uncertainty is stated as ±30%. For the gate shafts the cost of civil work in the shaft is added from section B.5.3.3 in the cost base.

-The underground power station cost is calculated from section B.2.10.4 in the cost base. The cost is the excavated volume of the cavern multiplied with a factor of 2000 NOK/m3 for larger stations whicj this is assumed to be. The uncertainty for this estimate is stated as -50% to +100%.

-The temporary roads cost are taken the section B.12.1 in the cost base. They are classified as low standard roads, but is assumed to be built in difficult terrain. The cost base sets the cost of this kind of road to 1500 NOK/m. The annual maintenance is set at 10% of the cost of the road. Since the construction period is planned to be 2 years, the maintenance cost is added to the second year. The uncertainty for the roads is set to +100% to -50%

-The cost for planning and construction management is based on the section B.0.6 in the cost base. Because of this being a relative large plant, the cost is set to 15% of the total construction cost.

8.2. Electro technical work

The estimate includes the cost of transportation, insurance, installation and commissioning. The prices is stated to deviate ±10-20%

The parts making up the electro technical work includes the generators, transformers, high-voltage switchgear, control systems, auxiliary systems, cable systems and power lines. The costs is calculated as a combined total, and is taken from the E.8.2a graph.

8.3. Mechanical equipment

The cost of calculation of mechanical equipment includes the reversible pump-turbines, gates and miscellaneous equipment. The cost is have an accuracy of ±20% and includes transportation to the site, spare parts, installation and painting, casual labor assistance, the suppliers' technical service during installation and commissioning, and provisions during warranty period.

-The pump-turbines cost is estimated from the cost curves on figure M.1.B in the cost base. The curves apply for regular Francis turbines, but an factor of 1.25 is suggested to make up for the extra cost. Since it is planned with two identical turbine units the price of the second unit is calculated as 90% of the first.

-Gate costs are taken from the figure M.3.D in the cost base. Water pressure was set to 30m

for the lower gate and 50m for the upper gate. Because of the tunnel cross section being much larger than the largest possible gate, it was calculated with using five gates side by side.
-The miscellaneous equipment which includes machine hall crane, cooling and drainage systems, intake trash rack and draft tube gates is estimated from figure M.4.A in the cost base. The selected head is interpolated between the H=100 and H=500 line to get the correct head.

The total cost for the whole project is from this calculations 1121.8 mill NOK. With the stated deviations the cost could vary from 857.0 mill NOK to 1465.9 mill NOK. That is a deviation of +31% to -24%.

8.4. Profit possibilities

To find if the project can be profitable from using the price variations in the market, it is necessary to find the price difference between buying and selling power that gives a positive NPV. The calculation was set up as a 50-year NPV calculation. It was selected several different prices for selling power to the market and for each of these values it was found a value of buying power that gives a positive NPV. The calculation was given a total efficiency for the power plant of 85%, both for pumping and producing. The interest rate was set at 10% and the plant is assumed to be operating at an optimum of pumping 50% of the time and producing 50% of the time.

This results from the calculation is used to create a graph to find the link between the maximum price of buying to the sale price.

Figure 8.1 Formula for required price difference

The calculation gives the following formula for calculating the price difference needed. $Buy\ price = (0.7225 \times Buy\ price) - 119.7250$. When compared to the hourly prices from the last year (20.05.2014-20.05.2014) and find the price that gives equal amount of pumping and producing, there will only be on average 30 minutes of pumping and 30 minutes of production every day. If the same is done with the average daily values for the last three years, there will on average be 1 hour and 14 minutes of pumping and 1 hour and 15 minutes of production.

9. Stability assessment of the selected section

The stability assessment was done for the headrace tunnel, tailrace tunnel and powerhouse cavern. For this purpose the finite-element software Phase2 was used. Q-values for each rock type was estimated during a field visit and later converted to Geological strength values (GSI) to be used with the Hoek-Brown failure criterion in Phase2.

The Q-system support recommendations were used to compare the findings from the Phase2 calculation and to be used as a basis for support where needed.

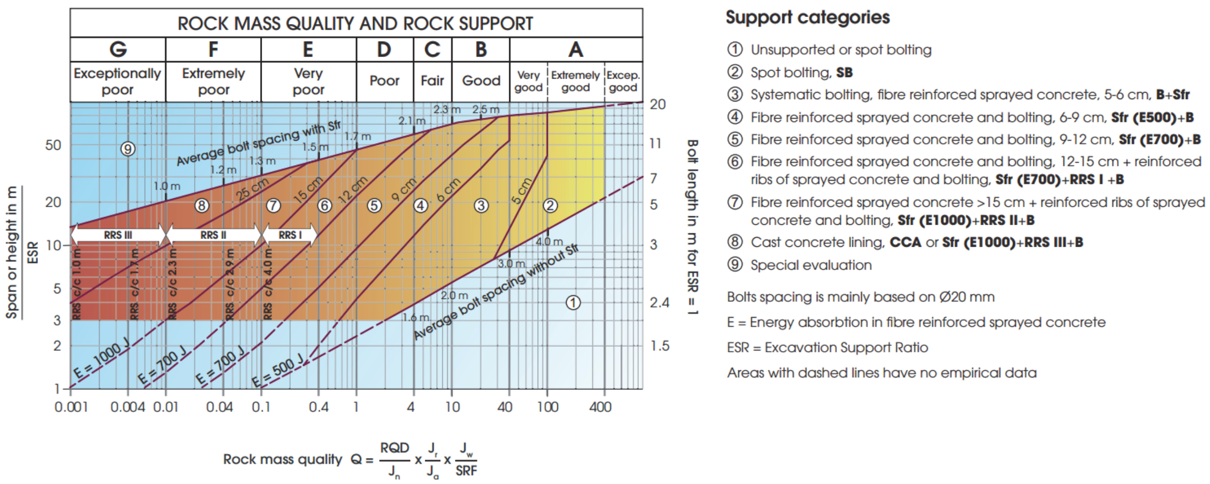

Figure 9.1 Q-method

$$GSI = 9log_e Q' + 44$$

Equation 3 Conversion from Q-value to GSI

(Bell, 2004)

Rocktype	Q-value	GSI
Sandstone	22.5	72
Greenstone	47.5	78.7
Amphibolite	45	78.3
Limestone	4.4	57.3
Gneiss	253	93.8

Table 9.1 Q-values and converted GSI values

For the strength parameters the material is set to be plastic and the failure criterion used is Hoek-Brown. The material is assumed to be isotropic.

Calculations are made for each of the different rock types at the place of maximum overburden to obtain the stability in the least favorable position. It was checked for sufficient strength factor and total displacement. The displacement seen on the figures are exaggerated by a factor of 500.

9.1. Result from analysis

9.1.1. Section in sandstone

Calculation was done with an overburden of 230 m. Lowest strength factor along the contour is 1.20 and the largest displacement is 1.98 mm. Results indicates that the rock mass is stable without additional support. Calculations with Q-system also recommend unsupported or spot bolting.

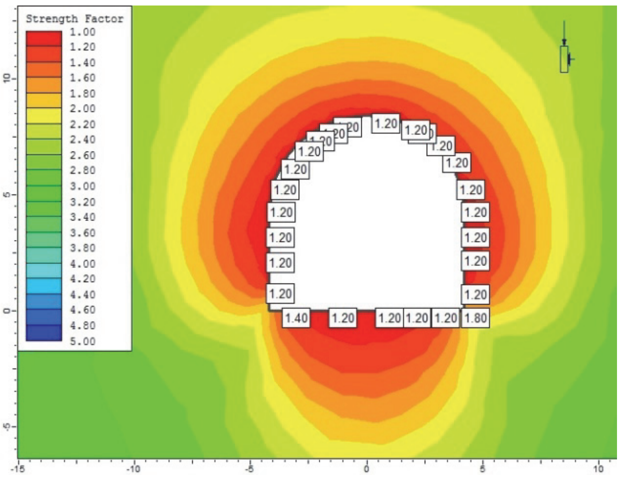

Figure 9.2 Sandstone strength factor

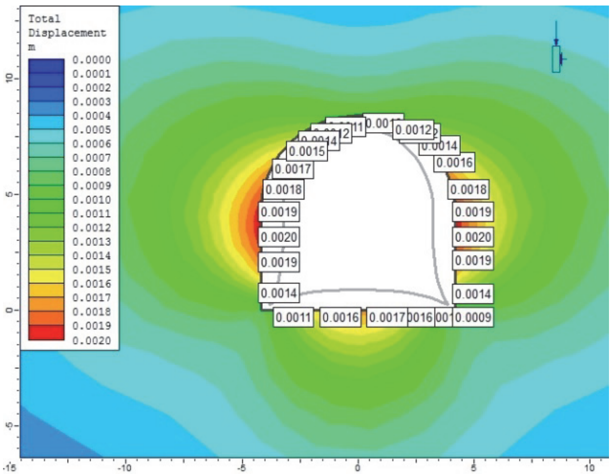

Figure 9.3 Sandstone total displacement

9.1.2. Section in greenstone

Calculation was done with an overburden of 292 m. Lowest strength factor along the contour is 1.20 and the largest displacement is 1.23 mm. Results indicates that the rock mass is stable without additional support. Calculations with Q-system also recommend unsupported or spot bolting.

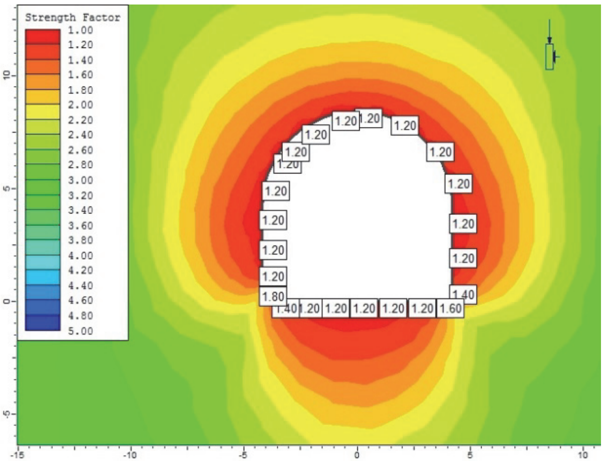

Figure 9.4 Greenstone strength factor

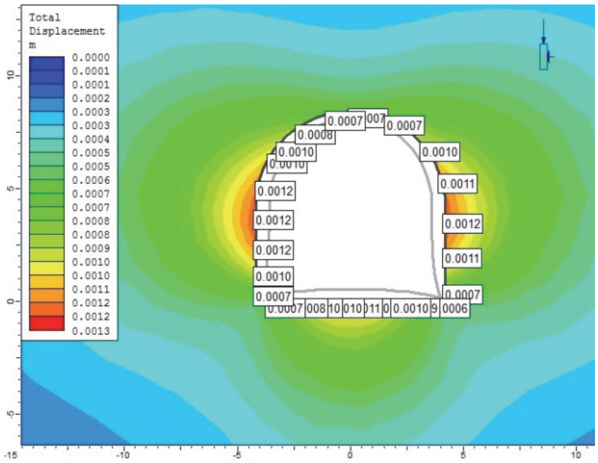

Figure 9.5 Greenstone total displacement

9.1.3. Section in amphibolite

Calculation was done with an overburden of 276 m. Lowest strength factor along the contour is 1.26 and the largest displacement is 1.14 mm. Results indicates that the rock mass is stable without additional support. Calculations with Q-system also recommend unsupported or spot bolting.

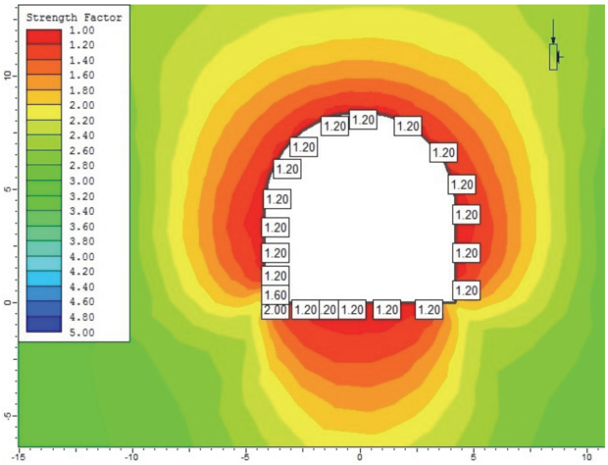

Figure 9.6 Amphibolite strength factor

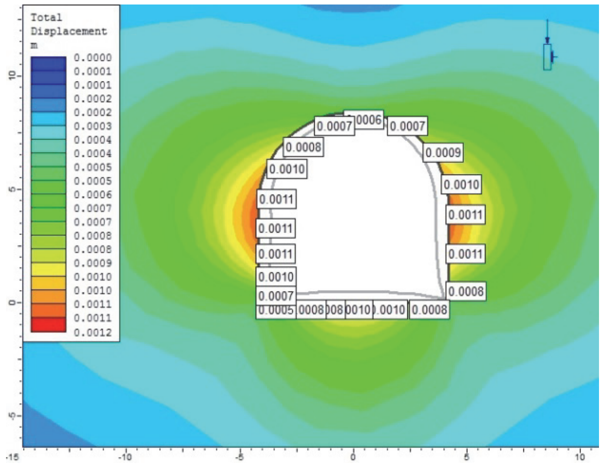

Figure 9.7 Amphibolite total

9.1.4. Section in limestone

Calculation was done with an overburden of 247 m. Lowest strength factor along the contour is 1.00 and the largest displacement is 1.78 mm. A strength factor of 1.0 is the lowest possible value for a plastic analysis and indicates that the forces acting on the rock are larger than its strength. To accommodate for this, support is installed as recommended by calculations with the Q-system. 5 cm layer of fiber reinforced shotcrete and 3 m long fully bonded rock bolts with 2.1 m spacing with a diameter of 20 mm to support the unstable rock. The strength factor is still too low, but the rock bolts do not yield and will hold the loose rock in place.

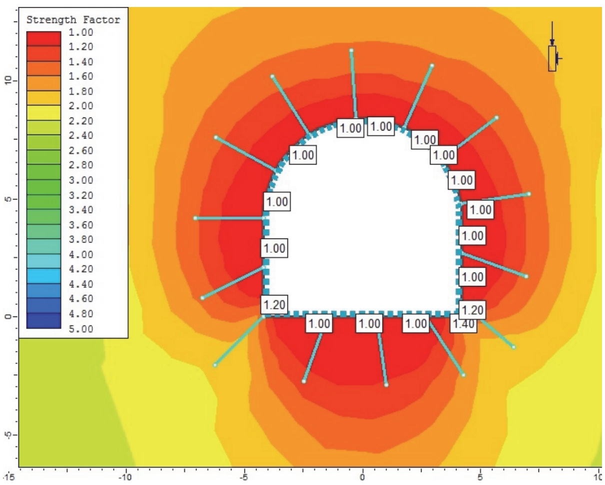

Figure 9.8 Limestone strength factor

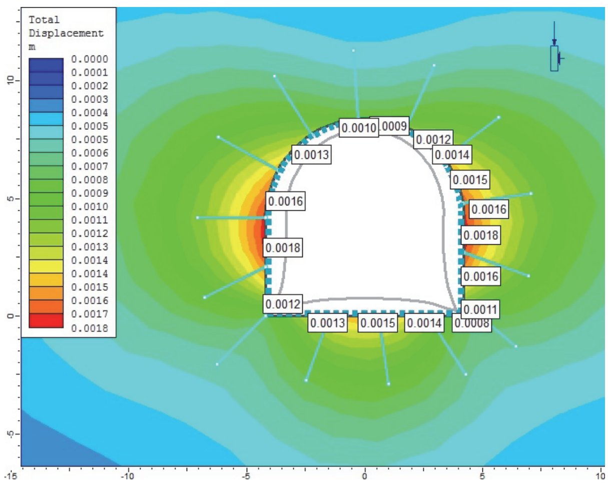

Figure 9.9 Limestone total displacement

9.1.5. Section in gneiss

Calculation was done with an overburden of 452 m. Lowest strength factor along the contour is 1.20 and the largest displacement is 1.01 mm. Results indicates that the rock mass is stable without additional support. Calculations with Q-system also recommend unsupported or spot bolting.

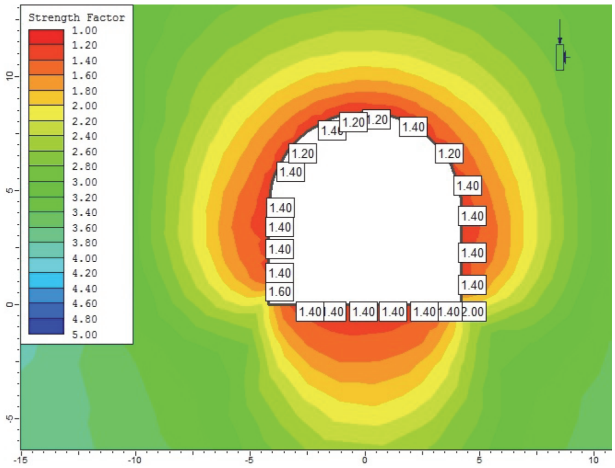

Figure 9.10 Gneiss strength factor

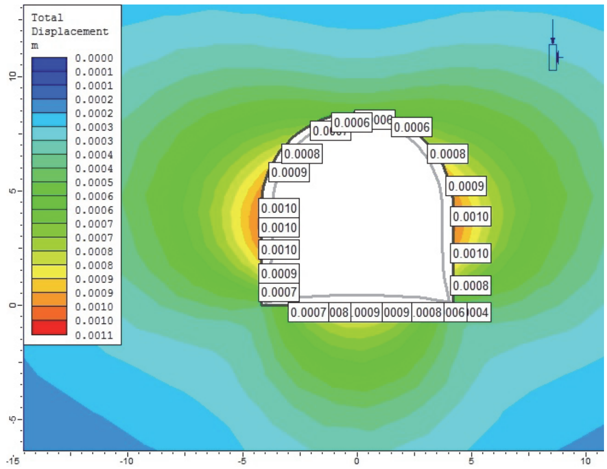

Figure 9.11 Gneiss total displacement

9.1.6. Powerhouse cavern

Calculation was done with an overburden of 321m. Lowest strength factor along the contour is 1.38 and the largest displacement is 3.65mm. Results indicates that the rock mass is stable without additional support. Calculations with Q-system also recommend unsupported or spot bolting.

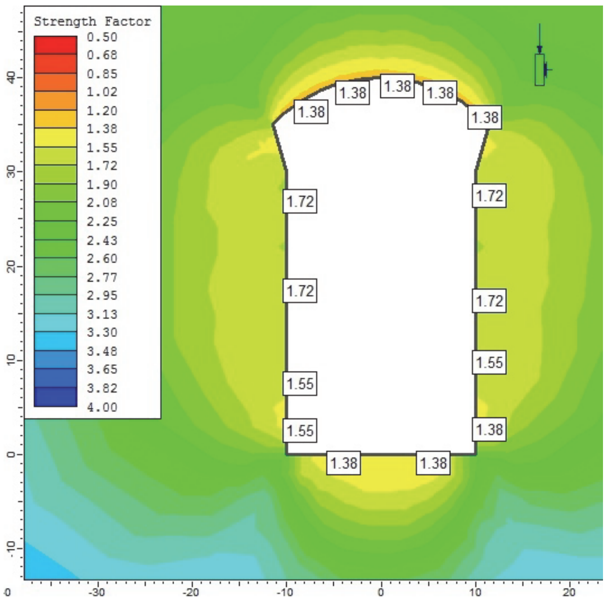

Figure 9.12 Cavern strength factor

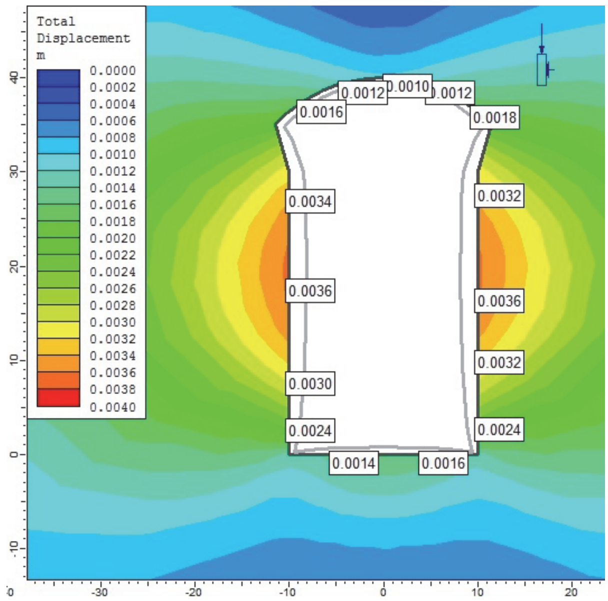

Figure 9.13 Cavern total displacement

10.Results and discussion

10.1. Placement and orientation

The tunnel system of a hydropower plant should be optimized in such a way that the total benefit versus cost is the greatest. This usually means placing the powerhouse and tunnels in a way that gives a shortest length of the water and access tunnels within the limitations given by for example the necessary rock cover and joint directions.

The placement of the intake and the outlet was to a certain extent given by the shape of the upper lake. For most of the lake the distance to the lower reservoir is about 10 km, but in the area of the chosen intake the distance is from 6 to 8 km. The final placement of the intake and outlet was based on getting a short stretch of tunnel, avoid crossing weakness zones in an unfavorable way, avoid areas with lakes and low overburden and having good placement possibilities for the powerhouse. When it comes to the orientation of the tunnels is in the least favorable position relative to the orientation of the joints, but layout was in any case determined by the location of the lakes.

The placement of the powerhouse and access tunnel was quite interdependent. The powerhouse was placed in position that gave the shortest stretch of access tunnel with an acceptable inclination from the desired entrance, but at the same time far enough into hill to have enough rock cover for the surge shafts to be higher than the highest regulated water level. The powerhouse cavern was rotated to not be in an optimal position relative to the two main joint directions. For the access tunnel the orientation is perpendicular to one of the joint directions and not optimal, but to place it in more optimal direction world require the tunnel to be almost twice as long.

10.2. Shape and size

The shape of the tunnels is the D-shape with a height to width of 1. This was chosen to get a tunnel shape that handles horizontal stresses well and gives a reasonable area to circumference while still offers enough height for machinery to operate efficiently. Given the size of the tunnel cross section having a slightly larger width than height might have been more preferable. The shape of the access tunnel should be designed to match the equipment that will be transported through it. Because of it not being possible to get specifications and dimensions for the equipment, it was decided to use the same shape as for the water tunnels. For the same reason the size of the access tunnel was taken from a project with similar characteristics.

The size of the headrace and tailrace was decided from an optimization process. The process involved doing cost/benefit calculations for a series of cross sections. For each section the head loss was calculated and a net head was found. A decision had to be made on how the power plant should operate since that will have a large effect on the size of the optimum cross section. Because of lacking information on the fluctuations in power production from the wind farms, this was set to the ideal state of pumping 50% of the time and producing 50% of the time. The value of power when producing was set to an average of the last 10 years and the price of buying surplus wind power was assumed. These values were used to calculate the yearly cost and benefit for pumping and producing power. Together with the cost of the excavated tunnel based on the NVE cost base for hydropower plants it was created an incremental cost-benefit graph to find the optimum size. Because of the rough approximations for the optimization and possibly optimistic figures the optimum cross section is believed to be slightly to large

The design of the powerhouse cavern is primarily dependent on the physical dimensions of the turbines and generators. As for the access tunnel, the main dimensions for the powerhouse cavern were taken from a similar project, but is expected to be close to what a design based on actual equipment dimensions would give.

10.3. Construction cost and profit possibility

The calculation of costs were conducted using the NVE cost base for hydropower plants and the NVE cost base for small-scale hydro based on the specifications decided upon in the shape and size chapter. The values from the cost base are based on statistics, and can only give a general picture of the expected cost. Even within the cost base expected deviation, the difference between the lowest and the highest estimated cost is almost a factor of 2. The dams were calculated in a quite rough way, which in this case probably leads to unrealistic low construction cost. For the profit possibility, the price difference needed between the price of buying and selling power is to grate compared to the variations in the power market, which leaves very few occasions where it is profitable to operate. But if the power plant buys its power as surplus production from wind farms, this price difference is more realistic to obtain.

10.4. Stability assessment

The stability assessment was carried out using the finite element computer program Phase2. It was found very little data on the rocks in the area, therefore most of the data came from Rocscience's database RocData, but supplemented with GSI values converted from estimated Q-values at site and the density of the rocks from NGU's online geophysics map service. The failure criterion chosen was Hoek-Brown and the material was set to be plastic. The rock stresses in the area was estimated based on statistics. The vertical stress was assumed to be equal to the weight and height of the overburden while the out of plane horizontal stress was set to be two times the vertical, and the in-plane horizontal stress was set to be equal to the vertical. This is not very exact, but based on statistics showing the horizontal stresses to usually being higher than the vertical in Norwegian projects and data from worlds-stress-map.org showing an over coring sample with the principle stress to be roughly in direction that correlates to the out of plane direction for the tunnels this should be a reasonable approximation.

For the actual simulation it was checked that the strength factor was not exceeded and that the deformation was within acceptable limits. In the cases where the strength factor was exceeded rock bolts and shotcrete was added and the simulation was run again to confirm that the strength of the rock bolts and shotcrete was not exceeded. The result was also compared with the recommendations from the Q-system. The results from a computer simulation does not give a 100% correct answer but an approximation to real life, since it is not possible to know about absolutely every detail of the rock mass, or for that matter to simulate them in such detail. In this case the quality of the data was quite low, so the result should only be looked at as an indication of what to expect.

To check for sufficient overburden, the rule of thumb by Berg-Christensen and Dannevig was used. It was drawn a revised and more uniform slope along the tunnel profile and checked at the most vulnerable places for each rock type. The results showed that the overburden was 50% higher than the minimum required and well within the limits..

11. Conlusion

Constructing the pumped storage power plant to operate solely on price variations in the power market is not viable. Basing its operation on surplus wind power on the other hand has greater potential, but in depth investigations on the wind production pattern should be done.

The stability of the underground openings is generally good, without the need of additional support, whit the exception of the short stretch of lime stone which will need systematic rock-bolting.

12.References

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Appendix

Cost calculations

Civil work

DAMS		Max	Min
Total cost of dams [mill NOK]	23.479438	29.3493	17.60958
HEADRACE TUNNEL			
Tunnel lenght [m]	3460		
Crossection [m2]	53		
Cost [mill NOK]	89.35	116.1555	71.48033
TAILRACE TUNNEL			
Tunnel lenght [m]	3074		
Crossection [m2]	53		
Cost [mill NOK]	77.825903	101.1737	62.26072
ACCESSTUNNEL			
Tunnel lenght [m]	1770		
Crossection [m2]	57		
Cost [mill NOK]	53.02035	66.27544	39.76526
SURGESHAFT UPPER			
Diameter [m]	3		
Lenght [m]	364		
Cost [mill NOK]	5.4006316	7.020821	3.780442
SURGESHAFT LOWER			
Diameter [m]	3		
Lenght [m]	335		
Cost [mill NOK]	4.8823905	6.347108	3.417673
CABLE SHAFT/EMERGANCY EXIT			
Diameter [m]	2.5		
Lenght [m]	410		
Cost [mill NOK]	5.4385988	7.070178	3.807019
GATE SHAFTS			
Shaft, upper inlet [m]	43		
Shaft,lower inlet [m]	36		
Diameter [m]	1.5		
Cost of shafts [mill NOK]	0.6776225	0.880909	0.474336
GATE SHAFTS, CICIL WORK			
Gate sealing, Cost [mill NOK]	1.00243		
Civil work in the gate shafts, Cost [mill NOK]	0.855		
Cost [mill NOK]	1.85743	2.414659	1.300201
UNDER WATER TUNNEL PIERCING			
Upper and lower reservoir			
Large tunnels (70 m2) 40-70 m pressure	4.8		
Large tunnels (70 m2) 40-70 m pressure	4.8		
Cost [mill NOK]	9.6	9.6	9.6

UNDERGROUND POWER STATION			
Blasted volume from plans [m3]	46295.46		
Cost [mill NOK]	92.59092	185.1818	46.29546
TEMPORARY ROADS			
Lenght [km]	8		
Cost for low standard difficult terrain [mill NOK]	13.2	26.4	6.6

Electro technical work

Total cost for Electro-technical equipment			
Speed nr: n=300			
Cost [mill NOK]	322.19301	386.6316	257.7544

Mechanical equipment

TWO REVERSIBLE PUMP-TURBINES			
Cost [mill NOK]	127.65299	153.1836	102.1224
GATES			
Number of gates, upper and lower intake	5		
Area of each gate [m2]	14		
Gate cost [mill NOK]	35.100333		
Cost [mill NOK]	37.635385	45.16246	30.10831
ADIT GATES			
Number of gates	2		
Gate size [m2]	6		
Cost [mill NOK]	2.5181893	3.021827	2.014551
PLUGS			
Lenght [m]	15		
Number of plugs	3		
Cost [mill NOK]	50.49	60.588	40.392
STEEL PIPES FOR ADIT GATES, TRANSITION TUNNEL/CAVERN			
Total lenght [m]	60		
Diameter [m]	3		
Cost [mill NOK]	0.30714	0.368568	0.245712
MISCELLANEOUS EQUIPMENT			
Cost [mill NOK]	65.885366	79.06244	52.70829
Total [mill NOK]	984.01	1285.888	751.7367
PLANNING AND CONSTRUCTION MANAGEMENT			
Percentage of the construction costs [%]	14		
Cost [mill NOK]	137.76081	180.0243	105.2431
TOTAL COST [mill NOK]	**1121.77**	***1465.91***	***856.98***

Calculations for required price difference

Selling price [NOK/MWh]	100	Buying price [NOK/MWh]	0	1	2	3	4	5
Income [mill NOK/year]	106	Pumping cost [mill NOK/year]	0	1.4625	2.9251	4.3876	5.8501	7.3127
		NPV [mill NOK]	-564	-576	-588	-600	-612	-624
Selling price [NOK/MWh]	200	Buying price [NOK/MWh]	25.10	25.10	25.10	25.10	25.10	25.11
Income [mill NOK/year]	211	Pumping cost [mill NOK/year]	36.71	36.71	36.71	36.71	36.72	36.72
		NPV [mill NOK]	0.03	0.02	0.01	-0.00	-0.01	-0.03
Selling price [NOK/MWh]	300	Buying price [NOK/MWh]	97.35	97.35	97.35	97.35	97.35	97.36
Income [mill NOK/year]	317	Pumping cost [mill NOK/year]	142.38	142.38	142.38	142.38	142.38	142.38
		NPV [mill NOK]	0.03	0.02	0.01	-0.00	-0.01	-0.03
Selling price [NOK/MWh]	400	Buying price [NOK/MWh]	169.60	169.60	169.60	169.60	169.60	169.61
Income [mill NOK/year]	423	Pumping cost [mill NOK/year]	248.05	248.05	248.05	248.05	248.05	248.05
		NPV [mill NOK]	0.03	0.02	0.01	-0.00	-0.01	-0.03
Selling price [NOK/MWh]	500	Buying price [NOK/MWh]	241.85	241.85	241.85	241.85	241.85	241.86
Income [mill NOK/year]	528	Pumping cost [mill NOK/year]	353.71	353.71	353.72	353.72	353.72	353.72
		NPV [mill NOK]	0.03	0.02	0.01	-0.00	-0.01	-0.03
Selling price [NOK/MWh]	600	Buying price [NOK/MWh]	314.10	314.10	314.10	314.10	314.10	314.11
Income [mill NOK/year]	634	Pumping cost [mill NOK/year]	459.38	459.38	459.38	459.39	459.39	459.39
		NPV [mill NOK]	0.03	0.02	0.01	-0.00	-0.01	-0.03

Optimum cross section calculations

Net head, m	288.6482	288.9949	289.3218	289.6302	289.9214	290.1966	290.4569	290.7034
Cost [Nok/m]	27415.86	27638.6	27861.34	28084.08	28306.82	28529.56	28752.3	28975.04
Cost [Mill Nok/6519m]	179.1352	180.5906	182.046	183.5014	184.9567	186.4121	187.8675	189.3229
Over all Efficiency [%]	90	90	90	90	90	90	90	90
Average hours of full production every day [hours]	12	12	12	12	12	12	12	12
Value of Power [Mill NOK/Year]	359.2034	359.635	360.0417	360.4255	360.7878	361.1303	361.4543	361.7611
Value of Power [NOK/MWh]	321.8	321.8	321.8	321.8	321.8	321.8	321.8	321.8
Interest	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1
Number of years	50	50	50	50	50	50	50	50
NPV of income [mill NOK]	2938.025	2941.554	2944.881	2948.02	2950.984	2953.785	2956.435	2958.944
Cost of power [NOK/MWh]	100	100	100	100	100	100	100	100
Cost of power [Mill NOK/Year]	137.8064	137.972	138.128	138.2752	138.4143	138.5457	138.67	138.7876
O&M of tunnel [%]	5	5	5	5	5	5	5	5
Tunell cross section	61	62	63	64	65	66	67	68
NPV of Cost + O&M	1355.865	1359.077	1362.211	1365.274	1368.269	1371.202	1374.077	1376.897
NPV of Income	2938.025	2941.554	2944.881	2948.02	2950.984	2953.785	2956.435	2958.944
Tunnel crossection Incriment	1	1	1	1	1	1	1	1
Cost Incriment	3.21225	3.134481	3.062308	2.995251	2.932874	2.874789	2.82064	2.77011
Income incriment	3.529613	3.3269	3.138777	2.963985	2.801397	2.649992	2.50885	2.377139

Dam cost calculation

2.5 m increase			5 m increase			7.5 m increase			10 m increase		
Lenght [m]	Height [m]	Cost [Nok]	Lenght [m]	Height [m]	Cost [Nok]	Lenght [m]	Height [m]	Cost [Nok]	Lenght [m]	Height [m]	Cost [Nok]
228	2	1995456	126	2	1102752	145	2	1269040	100	2	875200
127	3	1664589	110	2.5	1190365	100	2.5	1082150	474	2.5	5129391
61	4	1117886	175	3.5	2731488	110	3.5	1716935	144	4	2638944
11	5	268499	26	4.5	552747	27	4.5	574006.5	205	5	5003845
60	6	1881360	218	5.5	6054841	258	5	6297522	112	6	3511872
70	10	3937781	65	6.5	2284978	45	6	1411020	162	7.5	7030233
			24	7.5	1041516	94	6.5	3304429	185	8	8850770
			75	11	4728601	26	7	1018342	102	9	5044922
			50	13	3792151	56	7.5	2430204	94	10	5287878
						151	8	7224142	222	11	13996660
						61	9	3017061	40	12	2793681
						47	10	2643939	79	13	5991598
						30	13	2275291	23	17	2367804
						38	14	3110073	50	19	5829301
						70	16	6747931			
	[Mill nok]	10.86557		[Mill nok]	23.47944		[Mill nok]	44.12208		[Mill nok]	74.3521

Material cost		**Dam Height**	**Morain**	**Filter**	**Transition**	**Protection**	**Support**	**Fundation 1m Uncompacted**
Material	**Nok/1000 m3**		**1000m3 /m**	**1000m3 /m**	**1000m3 /m**	**1000m3 /m**	**1000m3 /m**	**1000 Nok/m**
Morain	166000	9	0.0625	0.068	0.068	0.095	0.089	15.7
Filter	159000	10	0.075	0.075	0.075	0.1	0.126	16.36
Transition	109000	11	0.0875	0.082	0.082	0.105	0.163	17.02
Protection	169000	12	0.1	0.089	0.089	0.11	0.2	17.68
Support	54000	13	0.11	0.096	0.096	0.115	0.23	18.34
		14	0.12	0.103	0.103	0.12	0.26	19
		15	0.12875	0.11	0.11	0.125	0.315	19.66
		16	0.1375	0.118	0.118	0.13	0.37	20.32
		17	0.15	0.126	0.126	0.135	0.3975	20.98
		18	0.1625	0.134	0.134	0.14	0.425	21.64
		19	0.175	0.142	0.142	0.145	0.4625	22.3
		20		0.15	0.15	0.15	0.5	22.96

Analysis information from Phase2 and rock data from RockData

Sandstone for tunnel simulation:

Phase2 Analysis Information
63m2 tunnel in sandstone

Project Summary

File Name: sandstone63.fez
Last saved with Phase2 version: 8.014
Project Title: 63m2 tunnel in sandstone

General Settings

Single stage model
Analysis Type: Plane Strain
Solver Type: Gaussian Elimination
Units: Metric, stress as MPa

Field Stress

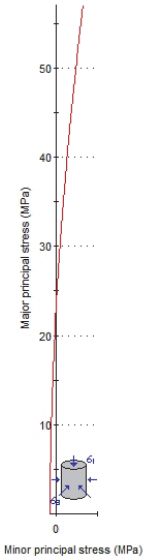
Field stress: gravity
Ground surface elevation: 230 m
Unit weight of overburden: 0.02658 MN/m3
Total stress ratio (horizontal/vertical in-plane): 0.2
Total stress ratio (horizontal/vertical out-of-plane): 0.2
Locked-in horizontal stress (in-plane): 6.11
Locked-in horizontal stress (out-of-plane): 12.22

Material Properties

Material: Material 1

Color	
Initial element loading	field stress & body force
Unit weight	0.02658 MN/m3
Elastic type	isotropic
Young's modulus	24305.5 MPa
Poisson's ratio	0.2
Failure criterion	Hoek-Brown
Material type	Plastic
Dilation Parameter	0
Compressive strength	115 MPa
mb parameter	6.25395
s parameter	0.0445514
Residual mb parameter	6.25395
Residual s parameter	0.0445514
Piezo to use	None
Ru value	0

Analysis of Rock/Soil Strength using RocData

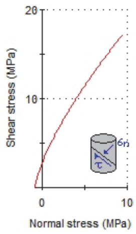


Hoek-Brown Classification
intact uniaxial comp. strength (sigci) = 115 MPa
GSI = 72 mi = 17 Disturbance factor = 0
intact modulus (Ei) = 31625 MPa
modulus ratio (MR) = 275

Hoek-Brown Criterion
mb = 6.254 s = 0.0446 a = 0.501

Mohr-Coulomb Fit
cohesion = 3.374 MPa friction angle = 57.35 deg

Rock Mass Parameters
tensile strength = -0.819 MPa
uniaxial compressive strength = 24.186 MPa
global strength = 42.009 MPa
modulus of deformation = 24305.48 MPa



Greenstone for tunnel simulation:

Phase2 Analysis Information
63m2 tunnel in greenstone

Project Summary

File Name: greenstone63.fez
Last saved with Phase2 version: 8.014
Project Title: 63m2 tunnel in greenstone

General Settings

Single stage model
Analysis Type: Plane Strain
Solver Type: Gaussian Elimination
Units: Metric, stress as MPa

Field Stress

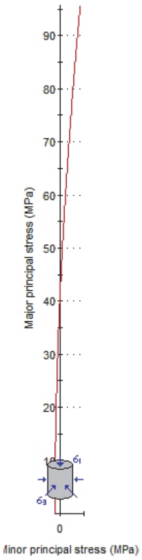
Field stress: gravity
Ground surface elevation: 292 m
Unit weight of overburden: 0.02704 MN/m3
Total stress ratio (horizontal/vertical in-plane): 0.2
Total stress ratio (horizontal/vertical out-of-plane): 0.2
Locked-in horizontal stress (in-plane): 7.8946
Locked-in horizontal stress (out-of-plane): 15.7892

Material Properties

Material: Material 1

Color	
Initial element loading	field stress & body force
Unit weight	0.02704 MN/m3
Elastic type	isotropic
Young's modulus	50840.2 MPa
Poisson's ratio	0.2
Failure criterion	Hoek-Brown
Material type	Plastic
Dilation Parameter	0
Compressive strength	130 MPa
mb parameter	12.2815
s parameter	0.096972
Residual mb parameter	12.2815
Residual s parameter	0.096972
Piezo to use	None
Ru value	0

Analysis of Rock/Soil Strength using RocData



Hoek-Brown Classification

intact uniaxial comp. strength (sigci) = 130 MPa

GSI = 79   mi = 26   Disturbance factor = 0

intact modulus (Ei) = 58500 MPa

modulus ratio (MR) = 450

Hoek-Brown Criterion

mb = 12.282   s = 0.0970   a = 0.501

Mohr-Coulomb Fit

cohesion = 5.000 MPa   friction angle = 61.01 deg

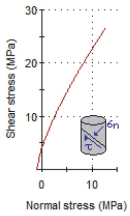
Rock Mass Parameters

tensile strength = -1.026 MPa

uniaxial compressive strength = 40.421 MPa

global strength = 67.294 MPa

modulus of deformation = 50840.20 MPa



Amphibolite for tunnel simulation:

Phase2 Analysis Information
Tunnel 63m2 Amphibolite

General Settings

Single stage model
Analysis Type: Plane Strain
Solver Type: Gaussian Elimination
Units: Metric, stress as MPa

Field Stress

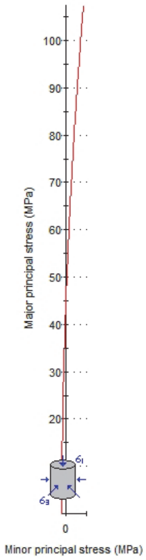
Field stress: gravity
Ground surface elevation: 276 m
Unit weight of overburden: 0.02847 MN/m3
Total stress ratio (horizontal/vertical in-plane): 0.25
Total stress ratio (horizontal/vertical out-of-plane): 0.25
Locked-in horizontal stress (in-plane): 7.86
Locked-in horizontal stress (out-of-plane): 15.72

Material Properties

Material: Material 1

Color	
Initial element loading	field stress & body force
Unit weight	0.02687 MN/m3
Elastic type	isotropic
Young's modulus	57850.2 MPa
Poisson's ratio	0.25
Failure criterion	Hoek-Brown
Material type	Plastic
Dilation Parameter	0
Compressive strength	150 MPa
mb parameter	13.6738
s parameter	0.0867743
Residual mb parameter	13.6738
Residual s parameter	0.0867743
Piezo to use	None
Ru value	0

Analysis of Rock/Soil Strength using RocData



Hoek-Brown Classification

intact uniaxial comp. strength (sigci) = 150 MPa
GSI = 78 mi = 30 Disturbance factor = 0
intact modulus (Ei) = 67500 MPa
modulus ratio (MR) = 450

Hoek-Brown Criterion

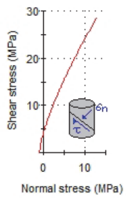
mb = 13.674 s = 0.0868 a = 0.501

Mohr-Coulomb Fit

cohesion = 5.181 MPa friction angle = 62.65 deg

Rock Mass Parameters

tensile strength = -0.952 MPa
uniaxial compressive strength = 44.110 MPa
global strength = 80.363 MPa
modulus of deformation = 57850.17 MPa



Limestone for tunnel simulation:

Phase2 Analysis Information

63m2 tunnel in limestone

Project Summary

File Name: limestone63.fez
Last saved with Phase2 version: 8.014
Project Title: 63m2 tunnel in limestone

Analysis Options

Maximum Number of Iterations: 500
Tolerance: 0.001
Number of Load Steps: Automatic
Convergence Type: Absolute Energy
Tensile Failure: Reduces Shear Strength
Joint tension reduces joint stiffness by a factor of 0.01

Field Stress

Field stress: gravity
Ground surface elevation: 247 m
Unit weight of overburden: 0.02687 MN/m3
Total stress ratio (horizontal/vertical in-plane): 0.25
Total stress ratio (horizontal/vertical out-of-plane): 0.25
Locked-in horizontal stress (in-plane): 6.64
Locked-in horizontal stress (out-of-plane): 13.27

Material Properties

Material: Limestone

Color	
Initial element loading	field stress & body force
Unit weight	0.02687 MN/m3
Elastic type	isotropic
Young's modulus	32561.1 MPa
Poisson's ratio	0.25
Failure criterion	Hoek-Brown
Material type	Plastic
Dilation Parameter	0
Compressive strength	80 MPa
mb parameter	1.72241
s parameter	0.00841468
Residual mb parameter	1.72241
Residual s parameter	0.00841468
Piezo to use	None
Ru value	0

Liner Properties

Liner: Shotcrete liner

Color	
Liner Type	Standard Beam
Formulation	Timoshenko
Thickness	0.05 m

Elastic Properties

Young's modulus	30000 MPa
Poisson's ratio	0.2

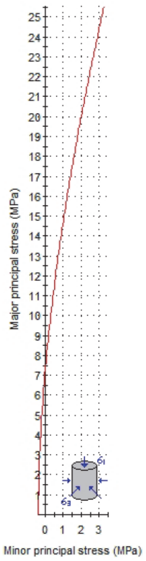
Strength Parameters

Peak compressive strength	35 MPa
Residual compressive strength	5 MPa
Peak tensile strength	5 MPa
Residual tensile strength	0 MPa

Bolt Properties

Bolt name	Rock bolt
Color	
Bolt Type	Fully bonded bolt
Diameter	20 mm
Young's modulus	200000 MPa
Tensile capacity	0.1 MN
Residual Tensile capacity	0.01 MN
Pre-tensioning	0 MN
Pre-tensioning force	Constant in install stage
Out-of-plane spacing	1 m
Allow Joints to Shear Bolt	Yes

Analysis of Rock/Soil Strength using RocData

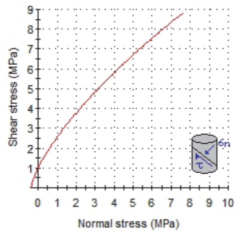


Hoek-Brown Classification
intact uniaxial comp. strength (sigci) = 80 MPa
GSI = 57 mi = 8 Disturbance factor = 0
intact modulus (Ei) = 72000 MPa
modulus ratio (MR) = 900

Hoek-Brown Criterion
mb = 1.722 s = 0.0084 a = 0.504

Mohr-Coulomb Fit
cohesion = 1.605 MPa friction angle = 45.17 deg

Rock Mass Parameters
tensile strength = -0.391 MPa
uniaxial compressive strength = 7.216 MPa
global strength = 14.756 MPa
modulus of deformation = 32561.11 MPa



Gneiss for tunnel simulation:

Phase2 Analysis Information
63m2 tunnel in gneiss

Project Summary

File Name: gneiss63.fez
Last saved with Phase2 version: 8.014
Project Title: 63m2 tunnel in gneiss

General Settings

Single stage model
Analysis Type: Plane Strain
Solver Type: Gaussian Elimination
Units: Metric, stress as MPa

Field Stress

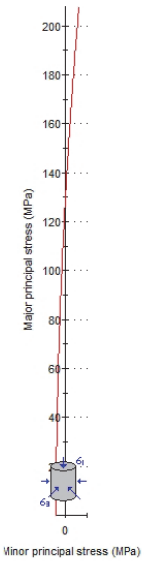
Field stress: gravity
Ground surface elevation: 452 m
Unit weight of overburden: 0.02549 MN/m3
Total stress ratio (horizontal/vertical in-plane): 0.2
Total stress ratio (horizontal/vertical out-of-plane): 0.2
Locked-in horizontal stress (in-plane): 11.52
Locked-in horizontal stress (out-of-plane): 23.04

Material Properties

Material: Material 1

Color	
Initial element loading	field stress & body force
Unit weight	0.02549 MN/m3
Elastic type	isotropic
Young's modulus	89717.4 MPa
Poisson's ratio	0.2
Failure criterion	Hoek-Brown
Material type	Plastic
Dilation Parameter	0
Compressive strength	175 MPa
mb parameter	22.5993
s parameter	0.513417
Residual mb parameter	22.5993
Residual s parameter	0.513417
Piezo to use	None
Ru value	0

Analysis of Rock/Soil Strength using RocData

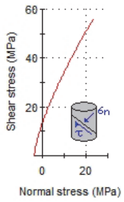


Hoek-Brown Classification
intact uniaxial comp. strength (sigci) = 175 MPa
GSI = 94 mi = 28 Disturbance factor = 0
intact modulus (Ei) = 91875 MPa
modulus ratio (MR) = 525

Hoek-Brown Criterion
mb = 22.599 s = 0.5134 a = 0.500

Mohr-Coulomb Fit
cohesion = 14.213 MPa friction angle = 62.49 deg

Rock Mass Parameters
tensile strength = -3.976 MPa
uniaxial compressive strength = 125.384 MPa
global strength = 144.800 MPa
modulus of deformation = 89717.43 MPa



Gneiss for cavern simulation:

Phase2 Analysis Information
63m2 tunnel in sandstone

Project Summary

File Name: sandstone63.fez
Last saved with Phase2 version: 8.014
Project Title: 63m2 tunnel in sandstone

General Settings

Single stage model
Analysis Type: Plane Strain
Solver Type: Gaussian Elimination
Units: Metric, stress as MPa

Field Stress

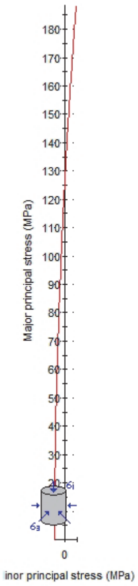
Field stress: gravity
Ground surface elevation: 230 m
Unit weight of overburden: 0.02658 MN/m3
Total stress ratio (horizontal/vertical in-plane): 0.2
Total stress ratio (horizontal/vertical out-of-plane): 0.2
Locked-in horizontal stress (in-plane): 6.11
Locked-in horizontal stress (out-of-plane): 12.22

Material Properties

Material: Material 1

Color	
Initial element loading	field stress & body force
Unit weight	0.02658 MN/m3
Elastic type	isotropic
Young's modulus	24305.5 MPa
Poisson's ratio	0.2
Failure criterion	Hoek-Brown
Material type	Plastic
Dilation Parameter	0
Compressive strength	115 MPa
mb parameter	6.25395
s parameter	0.0445514
Residual mb parameter	6.25395
Residual s parameter	0.0445514
Piezo to use	None
Ru value	0

Analysis of Rock/Soil Strength using RocData



Hoek-Brown Classification

intact uniaxial comp. strength (sigci) = 175 MPa
GSI = 94 mi = 28 Disturbance factor = 0
intact modulus (Ei) = 91875 MPa
modulus ratio (MR) = 525

Hoek-Brown Criterion

mb = 22.599 s = 0.5134 a = 0.500

Mohr-Coulomb Fit

cohesion = 13.759 MPa friction angle = 63.62 deg

Rock Mass Parameters

tensile strength = -3.976 MPa
uniaxial compressive strength = 125.384 MPa
global strength = 144.800 MPa
modulus of deformation = 89717.43 MPa