Graz University of Technology
Institute of Hydraulic Engineering and Water Resources Management

Case Study for an Underground Pumped Hydro Storage for the City of Graz

Master´s Thesis

of

Clément Boehler, BSc

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Supervisor
Ass.Prof. Dipl.-Ing. Dr.techn. Helmut Knoblauch

Co-Supervisors

Dipl.-Ing. BSc Franz Georg Pikl
Dipl.-Ing. Wolfgang Richter

Graz, November 2019

Master´s Thesis Supervisor:

Ass.Prof. Dipl.-Ing. Dr.techn. Helmut Knoblauch

Master´s Thesis Co-Supervisors:

Dipl.-Ing. BSc Franz Georg Pikl
Dipl.-Ing. Wolfgang Richter

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Abstract

Nowadays, as global warming has become a worldwide issue, every country in the world has to rapidly develop a renewable energy system including storage on a massive scale. It has been proven by researches that batteries and hydrogen fuel are not an economic and efficient storage solution on the long-term view. One obvious solution to fix this problem is to build large-scale underground pumped hydro storage near the big consumers. It is the aim of this work to propose a feasibility study for such a facility for Graz, the second largest city in Austria, which could serve as example for other countries worldwide.

Several researches and feasibility studies on underground pumped hydro were conducted in the 1970s, however, none of these led to the construction of a functional facility due to the dropping costs of fossil fuels at that time. This thesis provides a detailed 3D model of an entire power plant, based on former studies as well as experience gained during the last century with conventional pumped hydro storage, and taking into account hydraulic, geotechnical, geological, and logistical aspects. This model is an important groundwork to optimise step by step the design of underground pumped hydro storage through CFD simulations or physical model tests.

The result of the present study indicates that the construction of an underground pumped hydro in the vicinity of the city of Graz is clearly feasible and constitute an efficient and economical way to store vast amounts of electrical energy from renewable sources. The proposed design is a two-stage underground power plant with a total gross head of 1600 m, a capacity of 1000 MW, and an active hydraulic storage of 4,000,000 m^3. Furthermore, suggestions regarding the different parts of the power plant made in this work have to be considered during the design of such a facility in a near future. Adaptations and optimisations of the different parts of the power plant are to be done considering in situ conditions of each chosen construction site.

Generally speaking, this study shows that underground pumped hydro storage could be an economic and strongly reliable worldwide solution to store renewable energy worldwide, and thereby reduce humanity's ecological footprint on the earth.

Kurzfassung

Da der Klimawandel zu einem weltweiten Thema geworden ist, muss die erneuerbare Energiegewinnung sowie die Speicherung erneuerbarer Energien weltweit in großem Umfang, und rasch ausgebaut werden. Studien und die praktische Nutzung zeigen, dass chemische Speichertechnologien und synthetisch hergestellter Wasserstoff oder Methan auf lange Sicht keine effiziente Lösung zur Speicherung elektrischer Energie sind. Eine kostengünstige und ökologische Lösung zur Behebung dieses Problems wäre der Bau von großtechnischen unterirdischen Pumpspeicherkraftwerken in der Nähe der Vebraucherzentren. Ziel dieser Arbeit ist es, eine Machbarkeitsstudie für eine solche Anlage für Graz, die zweitgrößte Stadt Österreichs, vorzulegen, die auch als Beispiel für andere Länder dienen könnte.

In den 1970er Jahren wurden intensive Forschungsarbeit und Machbarkeitsstudien über unterirdische Pumpspeicherkraftwerke durchgeführt, die jedoch aufgrund der damals sinkenden Kosten von fossilen Kraftstoffen nicht zur Realisierung einer vollständigen Anlage, insbesondere mit unterirdische Speicherkavernen, geführt hat. Diese Arbeit bietet ein detailliertes 3D-Modell eines gesamten Kraftwerks, basierend auf ehemaligen Studien sowie auf Erfahrungen aus dem letzten Jahrhundert mit konventionellen Pumpspeichern, unter Berücksichtigung hydraulischer, geotechnischer, geologischer und logistischer Aspekte. Dieses Modell ist eine grundlegende Ausgangsbasis, um das Design von unterirdischen Pumpspeicherkraftwerken durch CFD-Simulationen oder Modellversuchen künftig zu optimieren.

Das Ergebnis der vorliegenden Studie zeigt, dass der Bau eines unterirdischen Pumpspeicherkraftwerks in der Nähe der Stadt Graz machbar wäre, und eine effiziente wirtschaftliche und langfristige Möglichkeit zur Speicherung von elektrischer Energie aus erneuerbaren Quellen darstellt. Aus diesem Grund sollten technische Vorschläge bezüglich des Designs des Kraftwerks, die in dieser Arbeit enthalten sind, bei der Planung einer solchen Anlage berücksichtigt werden. Der vorgeschlagene Entwurf ist ein zweistufiges Untertagekraftwerk mit einer Gesamtbruttohöhe von 1600 m, einer Leistung von 1000 MW, und einem aktiven Speichervolumen von 4.000.000 m^3. Anpassungen und Optimierungen sind, unter Berücksichtigung der örtlichen Gegebenheiten, jedoch vorzunehmen.

Generell zeigt diese Studie, dass unterirdische Pumpspeicherkraftwerken eine weltweite Lösung sein könnten, um erneuerbare Energie zu speichern, und damit den ökologischen Fußabdruck der Menschheit auf der Erde zu reduzieren.

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Definitions

Intake:

The term “intake” is used to describe the inlet as well as the outlet structure for a given reservoir. As this work deals with pumped hydro power plant, this structure can be considered as an inlet while producing energy and as an outlet while pumping water up or vice versa.

Confusor/Diffusor:

The terms “confusor” and “diffusor”, for a given reservoir, describes the same structural element. As this work deals with pumped hydro power plant, a given transition betwwen two different cross section can be seen as confusor while producing energy and as diffusor while pumping water up or vice versa.

Air pressure compensation system:

The term “air pressure compensation system” is a global term to describe the air galleries and air shafts in the entire system.

Underground Pumped Hydro Storage (UPH):

Underground pumped hydroelectric storage (UPH) is a technique for storing baseload electrical energy and releasing it to satisfy peak load requirements. The concept incorporates reversible pump-turbines or single function pumps and turbines and an underground reservoir mined from hard rock. Pump-turbines are coupled to motor-generators within a subterranean powerhouse that converts the energy of a water column flowing from a surface reservoir to the subterranean reservoir and then pumps the same water mass back to the surface. The powerhouse is located below the lower reservoir to maintain pump submergence. The UPH concept is like that of conventional surface pumped storage except that the subterranean reservoir eliminates dependence upon natural topography and enables design for higher head operation. A typical plant would have a capacity of 1000 to 3000 MW, an operating head of 1000 to 2000 m, and reservoir storage capacities equivalent to 8 to 10 h of generation at full load [1].

1. Introduction

Nowadays, as climate change has become a worldwide issue, every country in the world have to rapidly develop renewable energy storage on a massive scale in order to avoid irreparable damages to the earth. The reserves of fossil fuels (coal, Oil and natural gas) are rapidly shrinking and therefore not a sustainable solution for our future [2]. Therefore, renewable energy is, since some decades, the only alternative to the fossil fuels. The problem with most of the renewable energy sources is, that the production follows an intermittent scheme, which must somehow be rendered reliable and dispatchable for human demand and the industries. It has been proven in many researches that batteries and hydrogen fuel are not a solution on long-term view, either being uneconomical or polluting the environment [3]. The simplest solution to fix this problem would be massive storage facilities for energy near the load point.

Historically this has meant pumped hydroelectric storage, a technology that is well-known, reliable and inexpensive on long-term view compared to other storing solutions. However, suitable reservoir sites in alpine areas are limited, as well is the building of such infrastructures in ecologically sensitive high-altitude mountains. The obvious solution would be to construct the entire power plant underground. The lower reservoir could be excavated several hundreds of meters below surface and the upper reservoir could either be constructed at the surface, immediately above the lower one, or as well underground. In such a system, the water can be exchanged in a closed loop between both reservoirs either to create or to store energy, depending on the demand.

In the late 20th century, a lot of work was done on Underground Pumped Hydro (UPH), especially in the U.S.A., with technical reports, journal articles as well as feasibility studies accompanied by site investigation and rough cost estimations [2]. Unfortunately, this has never led to the construction of a functional UPH facility. At that time, the prices for other peaking power sources like natural gas remained favourable, which could be a reason why such a project has never been accomplished. Nevertheless, it can be proven that UPH can be a very economical solution of energy storage if the underground conditions are appropriate. In Addition, enough experience with surface pumped hydro storage facilities and deep excavations has been gathered during the last decades to enable the realisation of such a project. To finish with, conventional pumped hydropower storage (PHS) are nowadays more and more criticized. Numerous articles report the bad social, economic and ecological outcomes of such projects on the region where it is planned to be built [4]. In the last decades, several projects which were in the design phase were aborted because of strong opposition. Most of the arguments against PHS are not applicable to UPH, making the necessity of such facilities even greater.

The aim of this Master Thesis is to provide a case study of an UPH for the city of Graz in Austria. The European Union has always been a leading entity regarding efforts made to reach a global Climate agreement, and must therefore be a model for other parts of the world to reach the Paris agreement of 2015 regarding CO2 emissions cut. In this case study, a preliminary design of such a power plant will be provided for the city of Graz. Several aspects like the possible site location, geology and geometry will be taken into account. The study will include a detailed design of each part of the power plant and different technical aspects linked to them.

2. Overview of the historical development of Underground Pumped Hydro

2.1 First concept

The first idea of building an UPH facility came in the beginning of the 20th century. In fact, the famous north American researcher called R.A. Fessenden obtained the first official patent for a “system of storing power” [5] which can be seen on Figure 1. On this figure, a wind mill is represented as the power source to pump water from an underground to a surface reservoir.

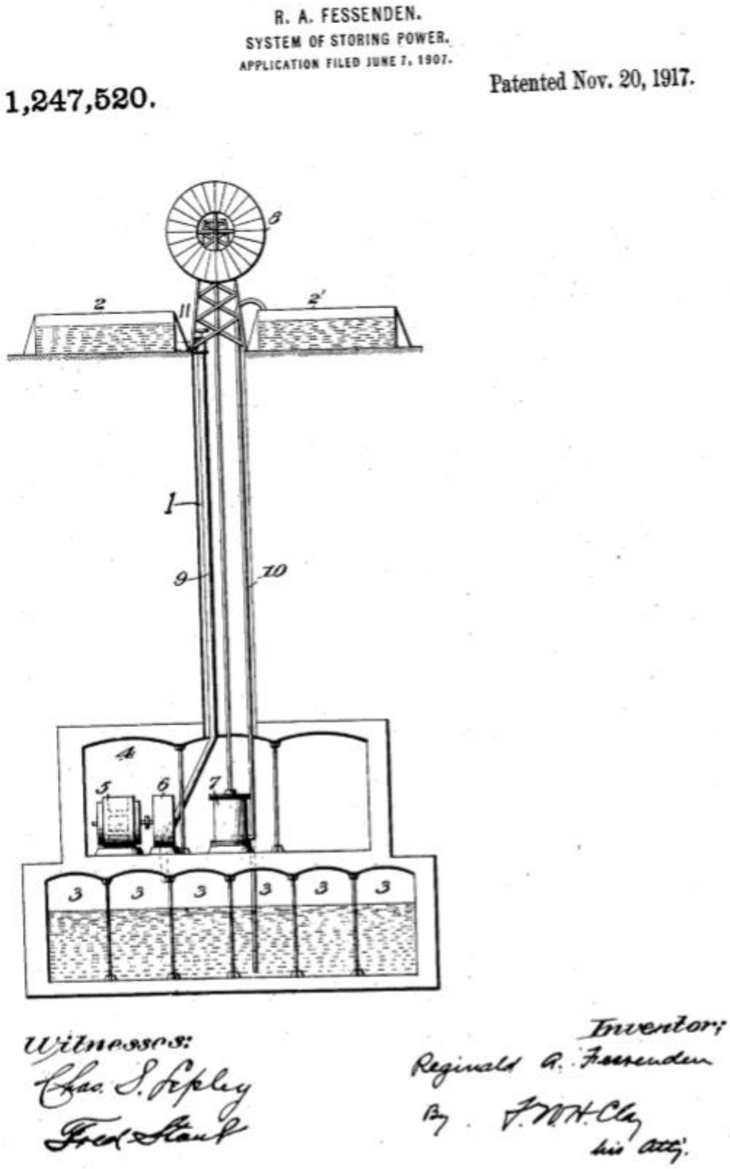


Figure 1: R.A. Fessenden: First Underground Pumped Hydro Storage patent [5]

The idea was to use intermittent sources of power, especially natural sources, to pump water from a lower to an upper reservoir and store it in potential energy. During peak hours, when energy was needed, water would flow through the turbine and produce energy. Already at that time, people recognized that in a near future, mankind will face a shortage of power unless some measures were taken to store the energy from intermittent sources.

In 1917, it was also proven that this kind of energy storage could be very cheap and reliable. It was estimated, that the cost per horse power per year of storing would be arount twenty cent with the UPH system, while the cost of generation by coal is in the neigbourhoud of fifty dollars per horse power per year [5]. Once such a plant would have been build, it could be in operation for many years, providing stability to the energy grid.

2.2 Research and development in the late 20th century

Between 1920 and 1970, not much research was conducted on UPH, and also no such plant which was built. Also, very few conventional PHS projects were realised during this period. One reason could be the two world wars and their consequences on the people. Nevertheless, experience with conventional PHS brought people to the following conclusion: “location and transmission are of key importance and the pumped-storage plant must be near the load centre. Otherwise, an expensive situation involving transmission facilities used only a few hours a day will results” [6].

After 1970, the activity on UPH was in full swing. Numerous journal articles and technical reports appeared, unfortunately unaccompanied by actual construction leading to functional UPH facilities [2]. Progresses were made in terms of concept and constructional approaches. It was shown that “The cost of excavation does not significantly increase with depth” [7] whereas the stored potential energy increases linearly. Further, some important statement about the required geology were made. “Competent rock, requiring a minimum of structural support, will be essential for the economic viability of this type of project” [8].

Technical and constructional aspects were discussed at that time. One of the focus was the operating mode. which could either be in a single stage or multiple stage layout. In a single-stage mode, the head would have been around 750 m, directly limited by the capacity of reversible pump-turbines. In the double-stage mode, a head of 1200-1500 m was possible with an intermediate reservoir and two stages of pump-turbines. The intermediate reservoir brings also flexibility to the operation system. Figure 2 and Figure 3 show layouts from the seventies of the two different modes of operation.

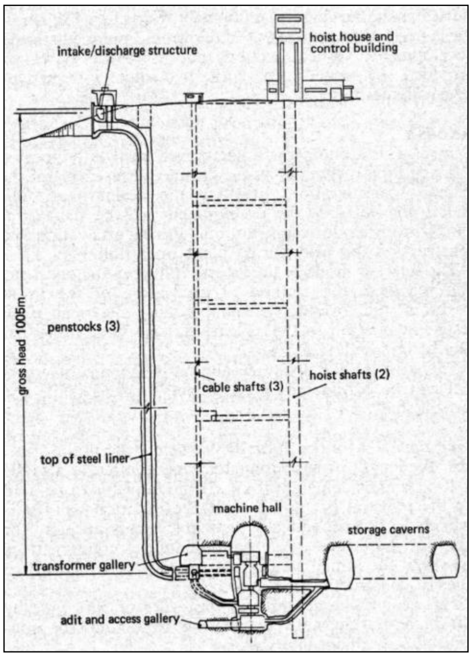


Figure 2: Single-stage Underground Pumped Hydro Storage layout from the 1970s [9]

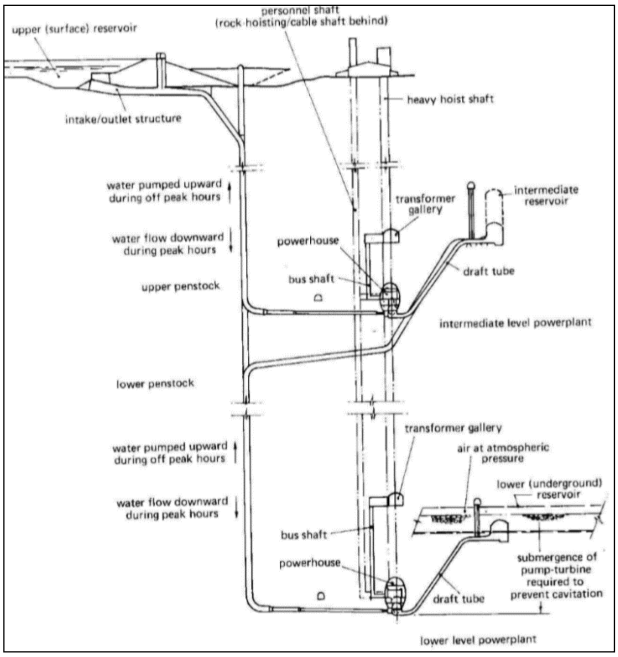


Figure 3: Double-stage Underground Pumped Hydro Storage layout from the 1970s [7]

As it can be seen, the conceptual and constructional concept of UPH was already rather well defined. An upper reservoir at the surface, a vertical pressure shafts to bring the water to the turbines, hoist shafts, air vent shafts, powerhouse and lower storage cavern. Further, the state of the art at that time would have enable the construction of such a power plant. The construction technics were well suited for the excavation of the shafts, powerhouse and lower reservoir, and the experience with conventional PHS was also available to assess all necessary hydraulic aspects.

Many other feasibility studies including detailed underground investigation campaigns were done in the U.S.A at that time. The U.S. Department of Energy commissioned a feasibility study for a UPH in 1984 [1]. Several locations were considered, taken into account the geology, distance to load centre, pump-turbine type, design and constructional aspects as well as a rough cost estimation showing the clear viability of such a project. Unfortunately, in the end of the 20th century, the price of natural gas remained favorable for peaking power, and no UPH were built [2]. Even construction of conventional PHS tapered off.

2.3 Nowadays: Underground Pumped Hydro becomes a necessity

Nowadays, the energy demand of the world population is greater than ever. Developing countries from the late 20th century become step by step industrial countries and want to provide themselves a higher life standard. Without an efficient management and distribution of the energy sources, the human kind might ruin the planet. Therefore, it is of primary importance to rethink about the way of producing and storing energy in order to optimise the integration of renewable energy sources.

It is already well known, that energy production by renewable sources like wind and solar energy alone is not viable for the future. In the U.S.A, scientists criticize the very rapid transition to renewable energy which have been done during the last decade [10]. They give the example of California, where between 2009 and 2015, 150 billion US Dollar were invested to build huge solar and wind farms. These farms are covering a significant amount of land and have a rather big impact of the nature. Additionally, huge transmission lines have been built to bring the energy from the desert to the city. The idea on long term view was to convert hydroelectric dams into PHS to create storage possibilities. Unfortunately, only a few dams were converted during this period of large wind and solar energy growth. Another challenge is that there are other uses for water in this area, like irrigation. In California, the water from the reservoirs is growing increasingly scarce and unreliable due to climate change. As a consequence, cities had to stop the electricity coming from the solar farms into the city because there was an excess supply, or pay the neighbouring states to take that electricity. The alternative was to suffer from blowouts of the grids.

One solution to face the upcoming energy and climate challenges is to restructure our energy production in a smart way. By using wind and solar in smaller and more local scale, and UPH to store the excess energy, each region of the world could generate and regulate energy for its own demand through smart technical solution suited to the in situ environment and climate. In such a scenario, the renewable intermittent sources of energy might vary from land to land, but UPH would be necessary worldwide to stabilise the grids. All the problems linked with conventional PHS like conflicts regarding water use, unreliability of the resource, risk for the populations in case of failure of a water impounding structure and impact on the nature are not applicable for UPH. Therefore, it is of great interest for the present world population to take a closer look at such power plants.

To finish with, William F. Pickard estimated roughly the global energy demand in 2100 and the related number of UPH needed to provide enough energy storage. An amount of 0.20 TJ (= 55.555 kWh) per person per year and a world population of 7.5 billion people could be assumed for the prediction. This would result in an annual energy demand of 1.5 x 10^{21} J (= 4.17 x 10^{14} kWh) or a daily desire for 4.1 x 10^{18} GWd = 50 x 10^{3} GWd. “Since prudence would seem to call for at least four days of storage and the standard reservoir introduced previously holds 2 GWd, this implies over 200,000 GWd of backup storage of some sort held in 100,000 reservoirs scattered over the planet’s surface” [2]. In such a short period of time, this number seems to be unrealisable. One has to keep in mind, that it is related to the wish of 4 days of energy backup for the entire planet, which portrays a rather conservative scenario.

3. General Aspects of Underground Pumped Hydro

3.1 Above or underground upper reservoir?

Which type of upper reservoir would be most suitable for an UPH? In fact, above ground and underground upper reservoir have both their advantages and drawbacks. Therefore, it is one´s choice to adopt one design or the other and try to deal as well as possible with the associated opportunities and problems. In this part, several aspects of each variant will be compared to see which one seems to be the best suited.

3.1.1 Aspects for above ground reservoirs

Above ground upper reservoirs have the advantage that they do not need any special geological or topographical features to be built. In contrast with alpine reservoirs for PHS, they can be built anywhere on a flat region with enough space, like a typical water retaining basin. The underlying soil does not need special properties regarding bearing capacity and permeability. Watertightness can be achieved by installing a dam impervious blanket made of several synthetic sheets. Underground drainage galleries have to be built to inspect possible leakages. The bearing capacity can be achieved by compacting the underlying soil layer.

Above ground upper reservoirs can be constructed out of excavation material from the shaft and underground reservoir. This material is more than sufficient to form the structural enclosure for the upper reservoir. By using the excavation materials, the construction costs are strongly reduced, making the project more viable.

One negative aspect about above ground upper reservoir is ironically that it is located at the surface of the earth. A lot of different debris can potentially end up in the reservoir like branches, stones, dead animals, synthetic material, etc. As it is not wanted to get all these things flowing through the turbines and pumps, additional constructional and monitoring measures have to be undertaken to avoid it. Further, the fact that the water is exposed to daylight will favour the development of life in the reservoir. The development of species like fresh water mussels for example, could be a real problem for the entire power plant. In the U.S.A., dams situated downstream from the great lakes recently started to get problems with different mussel species. "When established the mussels are ubiquitous, clogging intakes, trash racks, cooling water intakes, cooling water pipes and filters" [11]. Luckily, no such problems have been recorded until now for Austrian hydropower plants. Nevertheless, if such a project comes into consideration in other countries, this eventuality has to be taken into account during the design phase.

The main disadvantage about above ground upper reservoir is the fact that thermal energy cannot be stored. In the past, this was presented as an advantage, the water which is pumped from the lower reservoir to the surface will then be able to cool down so that no overheating in the system occurs. Recent studies [12] showed however, that the key to get a maximal efficiency on such a project is the thermal energy storage. This point will be discussed more in detail in chapter 3.1.2.

Above ground upper reservoir seems to be a rather economical and practical solution without important limitations. Nevertheless, the last aspect mentioned above restricts drastically the possible general efficiency of an UPH. In the late nineties, the cycle efficiency of a UPH was estimated to be up 80 % with an above ground reservoir [13]. However, an efficiency of 80 % is already very high. It will be shown in the next part that a much better efficiency can be obtained.

3.1.2 Aspects for underground upper reservoir

Obviously, the construction efforts required to build an underground upper reservoir are quite high. First, all the machinery and workers required for the blasting of the rock has to be brought underground. This implies also the building of adequate access and supporting structures to assure the safety of the underground works. Then, the excavation material as to be disposed from the site by mean of excavators, conveyor belt, and different means of transport (trucks, trains, etc.).

These factors are obviously increasing the costs of the underground variant compared to the above ground one. The in situ rock quality is a very important factors which can induce a strong cost variation. Therefore, it is necessary to choose a site were strong rock mass is available and close to the surface (200-300 m deep), in order to use the least amount of additional support possible.

The main advantage of the underground upper reservoir is the possibility to additionally store thermal energy in the water mass. By choosing this construction method, the entire system is an underground system working in a closed loop without exchanges with the outside air. The water will not anymore cool down at the surface. As rock mass is a very good insulator, the heat produced in the system (hydraulic losses, geothermal energy, etc.) can be stored in the water mass and used as thermal energy. Friction "losses" will not anymore be considered as losses. This energy is simply converted to heat and stays in the system. Recent works also showed that it would be possible to connect the UPH with the district heating system [12] and use it as a seasonal heat storage, which could increase the overall efficiency of the power plant up to 98 %. In long term view, this solution would be very economical.

It seems that, despite the higher construction efforts, building the upper reservoir underground is the best solution to maximise the efficiency of UPH. Therefore, this solution will be adopted for the further pre-feasibility study.

3.2 Location

An important task to perform is to find the best location to build the power plant.

First, it must be an area where good rock conditions can be found close to the surface. It would allow to excavate the upper reservoir cavern system without major difficulties. Further, only few support measures would be needed in case of local heterogeneities (kinematic blocks, disruption zones, poor rock mass quality, faults, etc.). It would speed up the construction time and reduce the project costs.

Then, it must be an area where the ground surface is not densely inhabited. According to the code of civil law in Austria (Bürgerliches Gesetzbuch), if one owns a piece of land, this property extends above the ground surface in the air and under the ground surface in the underlying soil mass. Therefore, for each piece of land situated above the planed underground structure, agreements have to be handled with the owners of the properties. Further, the excavation of underground structures could generate vibrations (blasting of the rock mass), noise (traffic and site facilities) and surface settlements. As the construction time can be expected to be very long, it is fundamental to reduce as much as possible the number of surrounding properties which will be affected in any kind of way by the construction.

The location which seems to be the most suited for this project is the Plabutschkogel on the west side of Graz, precisely underneath the Fürstenstand summit. To begin with, it is located very close to the city, load point of the electricity demand. This will enable a rapid distribution of the produced power and the possibility to easily connect the power plant with the district heating system. Then, previous experiences with the geology of this area have already been done in the past with the construction of the Plabutschtunnels. The first tunnel was finished in 1987 and the second one in 2004. Experiences from these construction sites represent a huge advantage to understand and assess the ground conditions of this area. The geology, water conditions and geotechnical properties of this area will be developed more in detail in chapter 3.3. Finally, there are almost no habitations on the ground surface, which reduces strongly the administrative efforts to get the agreement of potential owners and the risk of disturbing them.

3.3 Geology and Groundwater conditions

To assess the geology underneath the Fürstenstand summit, experiences from the construction of the Plabutschtunnel were used for a first rough estimation. Figure 4 shows the schematic geological section around the tunnel and the different soil layers [14]. The company Geoconsult was in charge of the geological investigation and assessment of the underground conditions during the project. The report states that around 6 km of the tunnel, round about 71.2 % of the entire length, were excavated in hard dolomite. This rock constitutes a very strong and stable ground. So called “braun rocks” and “barrandei-limestone” were found on a length of 1200 m in total. These rocks are also quite strong and stable, however less than the dolomite formation.

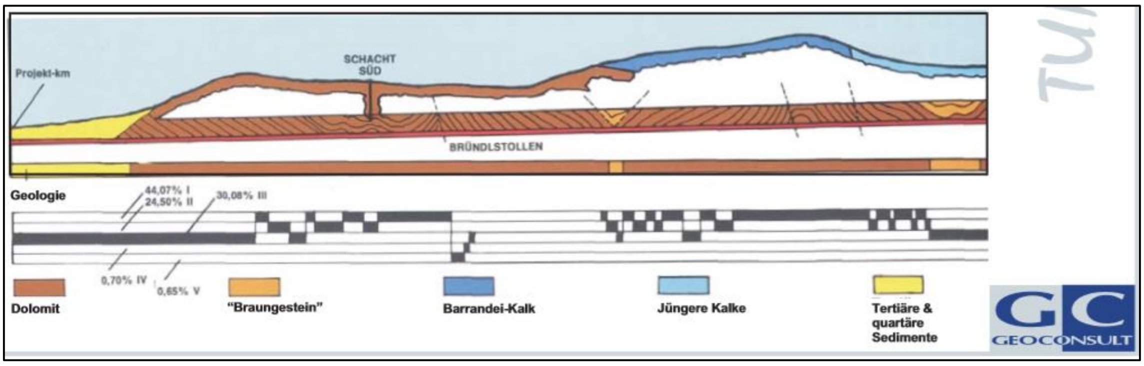


Figure 4: Geological section of the Plabutschtunnel area [14]

In the “Baugrundatlas Graz”, some further information about the geology of the Plabutsch can be found [15]. In the northern hillside of the Plabutsch, green schist, limy schist as well as different kind of Phyllite are the most present rock types. The formation indicates a strong clastic depositional environment which was influenced by volcanic activity. By favourable stratification, this formation has a high bearing capacity and constitute a hydrogeological water-impermeable horizon. On the eastern hillside of the Plabutsch, Sandstone formations can be found. It is an alternation of bedded to massive quartz-abundant sandstone, dolomite-sandstone and dolomite. The thickness of the formation is around 250 m and it has a very good bearing capacity. Directly underneath the Plabutsch and up to the Buchkogel, dolomite formation composed of dolomite and dolomite schist can be found. The thickness of the formation varies between 250 and 500 m and is very stable. Nevertheless, the reports states that some areas might be weathered and some karstic features could be found. This has to be taken into consideration when it comes to the water tightness of the upper reservoir.

Unfortunately, there is no mention of groundwater table in any literature. It can be assumed, that only a surficial groundwater is to be found. As the rock mass is of good quality, it is likely that, during rain events, the water directly seeps toward the city. However, this is only a hypothesis. Therefore, further geological and hydrological investigations are strongly recommended before the project starts.

3.4 Logistical aspects

Find a strategical place for site facilities and access tunnel into the mountain is of prior importance for such a huge underground project. It must be located close to existing roads or transportation facilities and as far as possible from habitations in order to avoid acoustical disturbance. The red rectangle on Figure 5 shows a possible location for the site facilities and access Tunnel. It is an old quarry in Graz located at the eastern hillside bottom of the Plabutsch.

Figure 5: Location of the site facilities and access tunnel [16]

3.4.1 Access Tunnel

The location and design of the access tunnel into the mountain is of prior importance to assure a fast connection between underground and surficial activities. The entrance of the tunnel could be located on the west rock face of the quarry showed by the blue arrow on Figure 5. One can see that it is located directly next to the Fürstenstand, the summit of the Plabutsch. As it was shown in chapter 3.2, the upper reservoir could be excavated directly underneath this summit. It would be possible to build a very short connection from there into the hill to bring the required equipment underground and convey the excavation material out of the hill. The elevation of this area is around 420 m a.s.l., approximately 327 m lower than the Fürstenstand summit (747 m a.s.l.). If the upper reservoir can be built at almost the same elevation, the access tunnel can be built quite flat.

On the left end of the quarry, jointed blue dolomite can be seen. Dolomite is very similar to limestone, however, the biggest part of the lime is represented by Magnesia. On the right side, it is possible to see the overlaying rock layer, which is a sandy dolomite with apparent clayey cleavages. Finally, a Sandstone with embedded black cleavages is laying on top of it. The entire formation is dipping at approximately 45° toward the north-west direction. The sandstones are showing detachments perpendicular to the bedding planes, directly linked with the orogenesis. The quarry is showing another rather curious phenomenon. The straightening up layer at the bottom of the quarry are pushing up flat horizontally laying rocks. Heritsch who discovered this feature sees here rightfully a failure plane because of the apparent offset between upper and lower layer [17]. Pictures of the quarry can be seen on Figure 6 and Figure 7.

The geological features described above are matching satisfyingly with the description of the eastern hillside of the Plabutsch given by the “Baugrundatlas Graz”. These geological features have to be taken into account when it comes to the structural design of the access tunnel. Nevertheless, the in situ conditions seems to be suitable for the construction of the access tunnel and upper reservoir.

An inspection of the site was done in the frame of this work to assess the in situ conditions. The daylighting rock face looks strong and stable and no ground water extrusion can be seen. Further geological mapping and investigations of the site are highly recommended.

Figure 6: Left side of the quarry showing blue dolomite [18]

Figure 7: Right side of the quarry showing sandy dolomite and sandstone [18]

3.4.2 Site facilities equipment

The site facilities equipment is an important interface between the underground activities and the city. Its location is shown on Figure 5 by the red rectangle. The brown/yellow area is mostly filled with material from the quarry. The green area next to it is a rather flat area covered with vegetation. An old access road can as well be found. Some earth works will be needed to remove the trees and level the area. One advantage is that the remaining material from the slurry can be used to for this purpose.

As it can be seen on the scale, the place would be around 200 x 200 m, which is enough to install containers, store material and install electricity cables and water pipes. The green dot is showing the goods station which is very close to the proposed site. It would be a suited area to store and transport the excavation material. A conveyor belt could be built in the air between the access tunnel and the goods station to transport the material. This would avoid truck traffic in the small streets between the site and the goods station. Another possibility would be the construction of an underground access to the station. The goods station could also be used to bring material to the site, which would avoid a lot of additional disturbance in the city. The orange rectangles are showing possible interface areas between the goods station and the access to the site facilities equipment.

Nevertheless, it can be seen that several habitations are located very close to the proposed site. An evaluation of the noise level has to be done in advance as well as prior agreements with the residents. As the construction will mostly take place underground, one can suppose that it will not create much disturbance for the people living in the surrounding.

3.5 Cavern System

For the upper and lower reservoir, a cavern system has to be excavated. The dimensions depend on the quantity of water that is needed to achieve a desired energy output. For the upper reservoir, the main factors influencing the layout are the geological conditions and the topography. First, the cross section has to be well designed to be self-bearing. Then, the layout will depend on the available space underneath the mountain with enough overburden. For the lower reservoir, space is not a problem anymore, therefore the layout is more flexible. Nevertheless, the deep excavation costs are an important factor, almost 20-30 % of the project expenses [1], therefore it has to be done as efficiently as possible.

Several layouts for the lower reservoir were already proposed in the late 1970s. Two of them are presented on the Figure 8 and Figure 9. In old reports, it can be seen that the upper reservoir was always designed at the surface.

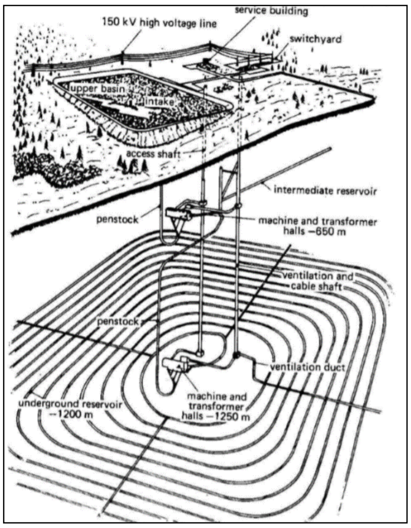


Figure 8: Spiral layout of a lower Underground Pumped Hydro Storage reservoir [9]

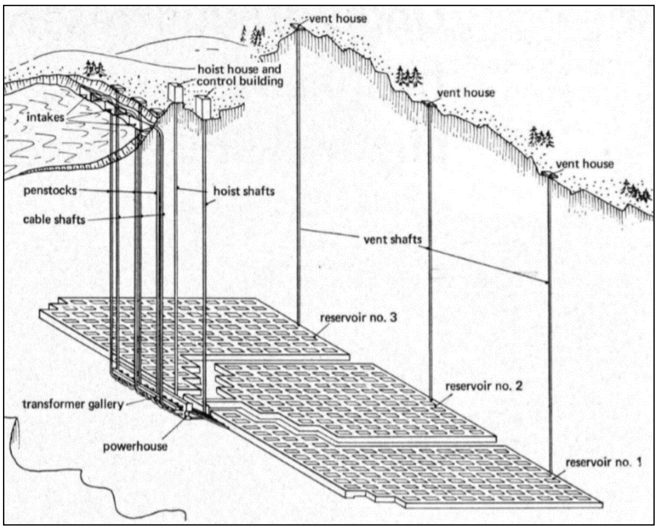


Figure 9: Raster layout of a lower Underground Pumped Hydro Storage reservoir with three independent parts. [7]

The first one is a spiral layout with four longitudinal caverns heading to the deepest point in the middle. It is pretty simple to excavate and has a favourable hydraulic shape. The second one is showing a more complex structure with a raster layout. The cavern system is basically made of straight caverns crossing each other and forming a structure similar to a honeycomb weave. One big advantage of this layout is, that there are three different parts in the system that can be isolated. If one part needs maintenance, or if the reservoir needs to be extended, the concerned part can be closed and the UPH do not have to stop its operation. A further advantage is the wave-breaking effect that such a layout has. As it is not one longitudinal tunnel but interconnected galleries, the wave propagation will be much harder, reducing at the same time the amount of freeboard required.

In principle, any kind of layout can be imagined. The in situ ground characteristics have to be taken into consideration, as well as the available space and the planed excavation technics. For a specific project, one layout could be very economic, but for another one unthinkable because of different boundary conditions. Further research about the layout for the case study will be discussed in chapter 4.

3.6 Required storage volume

The required storage volume is one key parameter for such a project. Combined with the head difference between upper and lower reservoir and the production time per day, it will give the output of the power plant. Or the other way round, if the desired output is given, a combination of a certain water head and a certain water volume will be required. The formula below is showing the relationship between these parameters.

$$P = Q \cdot \rho \cdot g \cdot H \cdot \eta \tag{1}$$

With:	P … Energy output	[W]
	Q … Discharge	[m^3/s]
	ρ … Water density	[kg/m^3]
	g … Gravitation constant	[m/s^2]
	H … Hydraulic head	[m]
	η ... Efficiency factor of the turbine	[-]

The discharge Q is calculated by dividing the active storage volume by the time corresponding to the full-load hours.

3.6.1 Upper and lower reservoir

According to “Energie Graz”, an active storage volume of 4 million cubic meters of water would be enough to connect the reservoir with the district heating system and use it as a seasonal heat storage for the entire city [19]. In the future, this number could rise up to 5 million. Therefore, the only parameters which can vary are the head difference, the output and the daily production time.

Additionally, a certain margin on top of the required 4 million cubic meters is needed. 2.3 % additional volume is required for “safety” storage to prevent over-filling of the reservoirs, and a further 0.3 % for freeboard [1]. As the intact excavated rock will be kept unlined, the bench of the caverns will be very rough and discontinuous. Therefore, 15 % additional volume is needed as “dead storage” for electricity storage puprose. Finally, the volume variation of the water due to heat variations has to be considered. This additional amount has already been calculated in a completed Master Thesis at the TU Graz and is 3.56 % [12]. All in all, the additional volume needed, considering safety, freeboard, dead storage and volume change because of temperature variation is 21.16 % of the total active storage volume. Therefore, the volume which needs to be excavated for the upper and lower reservoir is 4,846,400 m^3.

3.6.2 Intermediate reservoir

In case of a two-stage storage facility arrangement, an intermediate reservoir is required to cut the entire head into two separate drops. Therefore, two machine units have to be built, one at the intermediate level and one at the lower level. “The volume of water contained between the operating level limits in the intermediate reservoir allows some measures of unbalanced flow to or from this limited storage” [13]. All units of one plant (intermediate or lower) must be able to discharge during about 15 minutes, without compensating operation in the other. The intermediate reservoir must therefore be able to store the corresponding amount of water. At this point, it is not possible to calculate exactly how large the intermediate reservoir has to be, as it depends on the pumping and flow rate through the turbines.

3.7 Output, head difference and production time

3.7.1 General Aspects

As the reservoir volume has been given, the only parameters which can vary are the output, the head difference and the daily production time Figure 10 shows a graphical representation of the reservoir volume in millions of m^3 as a function of the Gross Head in m for some given output and daily production time combinations. The geothermal gradient of the rock mass is also indicated as well as an estimation of the required diameter of the pressure shaft.

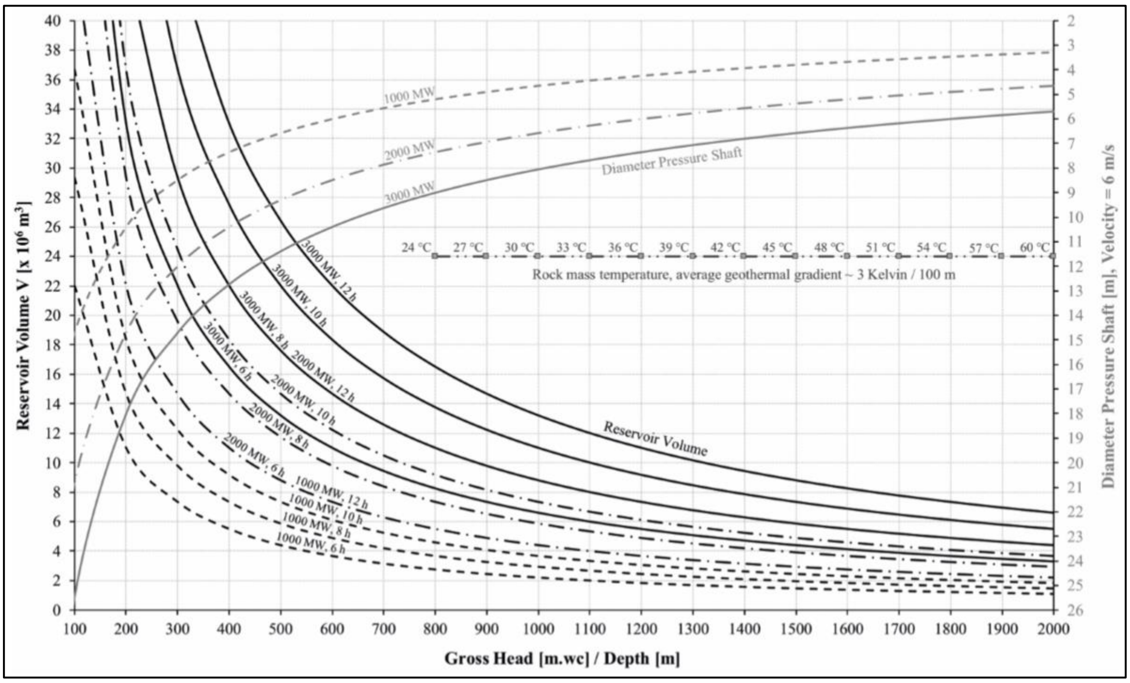


Figure 10: Reservoir volume as a function of the gross head for a given energy output and full load hours for the production process of a Pumped Hydro Storage [12]

For example, if one takes a reservoir volume of 4 million cubic meters, an output of 1000 MW and a daily production time of 12 h, the gross head required would be 1100 m. The corresponding diameter of the pressure shaft is around 4.5 m for a given flow velocity of 6 m/s. The velocity in the pressure pipe must be kept low in order to avoid high friction losses and turbulences, therefore 6 m/s has been assumed. These kind of charts are very useful to do a first approximation of the scale of such a project. Depending on which parameters are given as a boundary condition, it is possible to make the other parameters vary to find the optimum characteristics of the power plant.

The graph is clearly showing that the deeper the lower reservoir is built, the less storage volume is needed. For very low gross head (left side of the graph), the reservoir volume needed for a given output is on average very high. As the excavation costs represents 20 to 30 % of the overall costs, this factor is unneglectable. For very high gross head, (right side of the graph), the reservoir volume needed for a given output is on average very low. As a conclusion, it can be stated that such a project becomes economical for gross heads between 1000 and 1500 m. Further, depending on which of the input parameters are given as a boundary condition, different graphs can be used.

3.7.2 Input parameters for the feasibility study

For the feasibility study, some boundary conditions have to be assumed. The reservoir volume has already been given and is 4,000,000 m^3 of active hydraulic storage. The power plant will be designed with an intermediate reservoir and two stages of pumps and turbines. The total head which seems to be reasonable for the city of Graz is 1600 m with two stages of 800 m. A total installed capacity of 1000 MW will be provided by two turbines of 250 MW in each stage. With such boundary conditions, the power plant would be able to produce by full load during 16h if the upper reservoir is completely filled. The corresponding discharge is 70 m^3/s, which can easily be determined by dividing the active storage volume by the production time in second. This value will be used for the further dimensioning.

Table 1: Input parameters for the feasibility study

Output [MW]	**Hydraulic Head [m]**	**N° of stages [-]**	**Max. production time [h]**	**Discharge [m^3/s]**
1000	1600	2	16 h	70

4. Conceptual feasibility study

In this chapter, a 3D model has been done to assess the spatial feasibility of the project and propose a technical concept based on hydraulic and geotechnical aspects. This model is the first of this kind. It gives a feeling about how an UPH in Graz, or anywhere in the world, could look like. It will not be used for further hydraulic calculations in the frame of this Master Thesis. However, it can be used in the frame of future works in order to improve the global concept of an UPH through CFD simulations or model tests. The program Rhinoceros 5 has been used for the drawings.

4.1 Topography of the chosen site location

The first task was to implement the topography of the Plabutsch in the software Rhinoceros. To do so, Google Earth and the program Sketch up were used. In the “File” menu in Sketch up, there is the option “Geo-location” and “Add location”. It allows the user to select a squared area anywhere on the world map and import it in Sketch up. The size of the area that can be imported is limited. However, the program allows, by following the same procedure as described above, to “Add more imagery”. Basically, it is possible to insert as many squared areas as wanted and put them together to model an area with the wished size. The only requirement is an overlapping of the images. The steps to follow are shown on Figure 11.

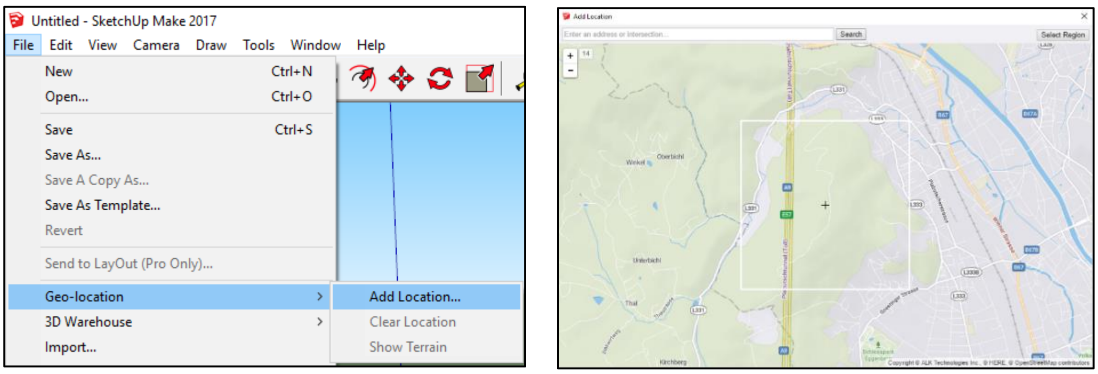


Figure 11: Import geolocation in Sketch up

The centre of the first selected square is the origin of the coordinate system. The GPS coordinate of the point is given as decimal degrees (DD) and can be find in “Window”, “Model Info” in the sub-category “Geo-location”. The origin of the model was taken at the highest point of the Plabutsch in order to have a well-known point. It is very important for the upcoming geometry implementation in Rhinoceros to know exactly the coordinates of the origin. Table 2 shows the given information.

Table 2: GPS coordinates of the origin of the 3D model

Country	**Location**	**Latitude [DD]**	**Longitude [DD]**
AUT	Graz, Styria	47.090148N	15.38560E

In the same Window, under the sub-category “Units”, one has to make sure that the “Format” is “Decimal” and the unit “mm”.

In a further step, it is possible to display the terrain topography, again in the menu “Geo-location”, by clicking on “Show terrain”. The file has to be saved as a SketchUp Version 8 file. Figure 12 shows how it the terrain model looks like.

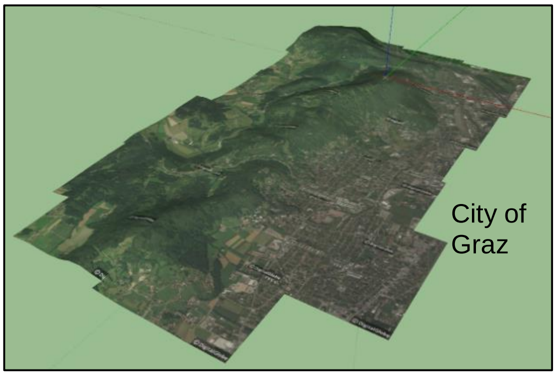


Figure 12: 3D terrain model of the Plabutsch in SketchUp

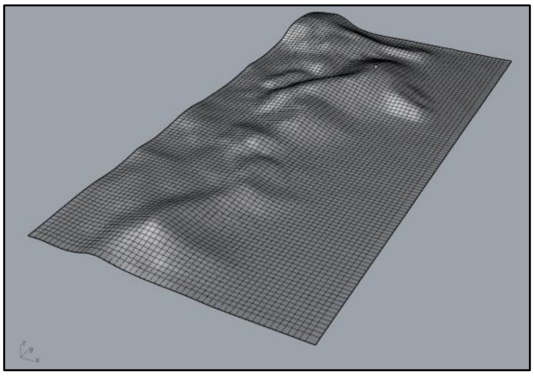


Figure 13: 3D terrain model of the Plabutsch in Rhinoceros

The program Rhinoceros can be started. The user has to make sure that the unit is millimetre. To do so, click on the “data” menu, “properties” and go in the sub-category “units” to be sure that the right unit has been chosen. Then, it is possible to import the terrain model in Rhinoceros by going in the “Data” menu, click on “Import” and select the SketchUp File. All the overlapping patches can now be seen. The flat surfaces coming from google maps are as well in the model and have to be deleted.

To add the origin point, go in the command and type “Point”, press “enter” and type the coordinates “0,0” to create it. A point will appear in the centre of the first quad, at the highest point of the Plabutsch. As the exact GPS coordinates of this points are known, it will allow in further steps the implementation of existing structures in the model.

Then, use the command “Drape” and drag a window that stays inside the boundaries of the quads over the area of interest. It will create a single surface of the selected area. The 3D terrain model in Rhinoceros as well as the origin point can be seen in Figure 13. The north is shown by the y-axis and the city of Graz is on the right hillside of the mountain. The dimensions of the model are around 9 by 4.5 km.

4.2 Implementation existing underground structures

The Plabutschtunnel is an approximately 10 km long road tunnel underneath the Plabutsch on the west side of Graz. Two tunnels going from north to south assure a safe and rapid underground crossing of the city. As no spatial conflict between the power plant and existing structures is allowed, it is of primary importance to implement the geometry of both tunnels in the 3D model. Further, it will help to find the optimal geometry of the UPH upper reservoir. To implement accurately both tunnels in the 3D model, google earth was used. A pin was placed at the entry and exit of each tunnel to know their GPS-coordinates and height in m a.s.l. Then, several other points were placed along both tunnel axes. Two cross section were made linking the points of each tunnel to know the distance between them. The slope is assumed to be constant, therefore it is possible to back calculate the elevation in m a.s.l. of each point. With the GPS-coordinates and the height of each point, the coordinates of each point were back calculated in the local coordinate system the 3D model. The results are shown in Table 3.

Table 3: Coordinates of the implementation points of the Plabutschtunnel in the 3D Model

Point Name	Decimal grad	Grad, minute, second	Distance [m]	Height [m a.s.l.]	X-Coordinate [m]	Y-Coordinate [m]	Z-Coordinate [m]
Origin	47.090148 15.385602	N47° 5'24.533" E15° 23'8.167"	0.0	747.0	0.0	0.0	0.0
Plabutsch Tunnel, Entry North	47.108631 15.378083	N47° 6'31.07" E 15°22'41.10"	0.0	369.0	-570.0	2058.0	-378.0
Plabutsch Nord-South 1	47.105590 15.379252	N47° 6' 20.124" E15° 22' 45.307"	369.0	368.2	-481.0	1719.0	-378.0
Plabutsch Nord-South 2	47.045409 15.378444	N47° 2' 43.472" E15° 22' 42.398"	7240.0	353.7	-543.0	-4980.0	-393.3
Plabutsch Nord-South 3	47.041593 15.378769	N47° 2' 29.735" E15° 22' 43.568"	7670.0	352.8	-518.0	-5405.0	-394.2
Plabutsch Nord-South 4	47.03765 15.380967	N47° 2'15.54" E15°22'51.48"	8150.0	351.8	-351.0	-5844.0	-395.2
Plabutsch Nord-South 5	47.033786 15.386864	N47° 2'1.63" E15°23'12.71"	8790.0	350.4	96.0	-6274.0	-396.6
Plabutsch Nord-South 6	47.032252 15.394336	N47° 1' 56.107" E15° 23' 39.61"	9400.0	349.1	662.0	-6445.0	-397.9
Plabutsch Tunnel, Exit South	47.032998 15.406895	N47° 1' 58.793" E15° 24' 24.822"	10400.0	347.0	1615.0	-6362.0	-400.0
Plabutsch Tunnel, Entry South	47.033169 15.406853	N47° 1' 59.408" E15° 24' 24.671"	0.0	347.0	1612.0	-6343.0	-400.0
Plabutsch South-Nord 1	47.032337 15.394919	N47° 1' 56.413" E15° 23' 41.708"	990.0	349.0	707.0	-6435.0	-398.0
Plabutsch South-Nord 2	47.033072 15.390614	N47° 1' 59.059" E15° 23' 26.21"	1330.0	349.7	380.0	-6354.0	-397.3
Plabutsch South-Nord 3	47.035207 15.385374	N47° 2' 6.745" E15° 23' 7.346"	1810.0	350.7	-17.0	-6116.0	-396.3
Plabutsch South-Nord 4	47.038551 15.381275	N47° 2' 18.784" E15° 22' 52.59"	2320.0	351.7	-328.0	-5744.0	-395.3
Plabutsch South-Nord 5	47.041375 15.379878	N47° 2'28.95" E15°22'47.56"	2650.0	352.4	-434.0	-5429.0	-394.6
Plabutsch South-Nord 6	47.105234 15.379999	N47° 6' 18.842" E15° 22' 47.996"	9982.0	367.4	-425.0	1679.0	-379.6
Plabutsch Tunnel, Nord Exit	47.107925 15.378961	N47° 6'28.53" E 15°22'44.26"	10300.0	368.0	-503.0	1979.0	-379.0

The points were implemented in Rhino starting from the origin of the model which is the point placed at the summit of the Plabutsch. After the points were placed, a rough cross section of the tunnel was drawn and a 3D object for each tunnel was created. The cross section was taken from a paper about tunnel constructions in Styria and the dimensions from a recent paper about the geologie encountered during the construction [20, 14]. The dimensions of the tunnel were taken from another paper about the The cross section and implementation of the tunnels is shown on Figure 14 and Figure 15.

As it can be seen on the figures above, the tunnel is going the all way from north to south throw the Plabutsch. Its position is relatively central, slightly on the west side. The next step will be the study of possible layouts for the upper reservoir of the UPH in order to fit next to this already existing underground structure.

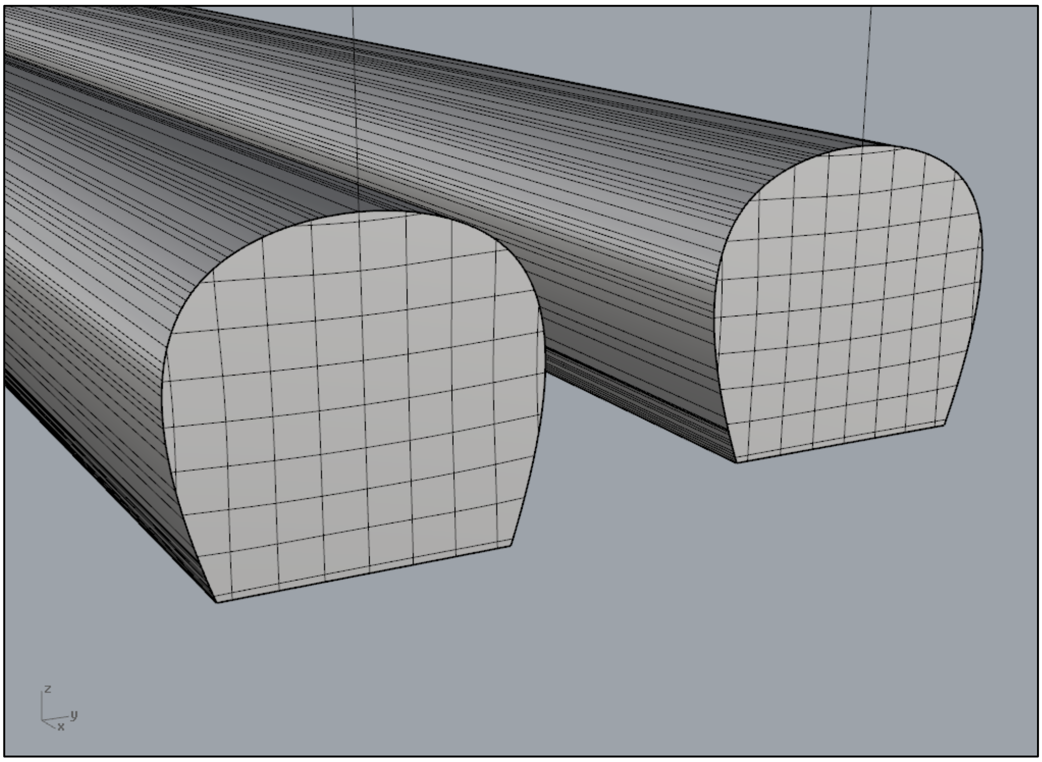


Figure 14: Isometric view of the cross section of the Plabutschtunnels

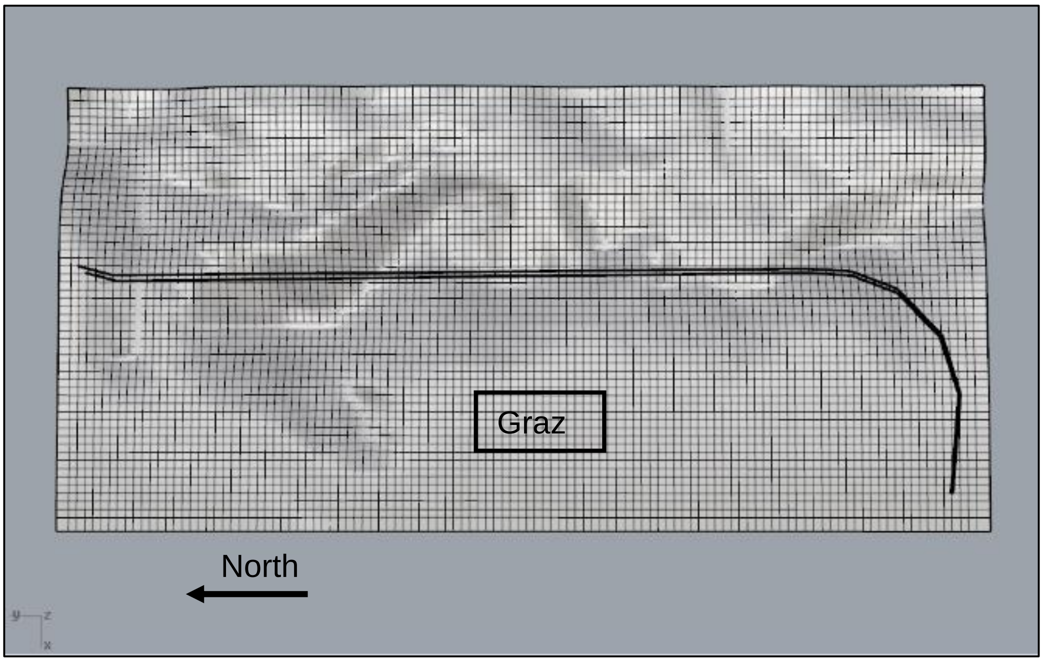


Figure 15: Bottom view of the Plabutschtunnel in Rhinoceros (north: left / east: bottom)

4.3 Layout study for the underground reservoir system

4.3.1 Cross section

A cross section of the tunnels which will constitute the reservoirs had to be designed. It is preferable to have a system of self-bearing caverns with a favourable geomechanical shape. An old feasibility study made in the US in 1984 recommends tunnels with 15 m width and 25 m height. The spacing between the tunnels could be 60 m, creating 45 m thick pillars between the tunnels. These recommendations were done for lower reservoirs. Nevertheless, the spacing between the tunnels have to be adapted, depending on the in situ stress and the rock quality. It is likely that the spacing of tunnels from the lower reservoir will be smaller than the one from the upper reservoir, as the rock quality deep underground is expected to be better than at the surface. For the sake of simplicity, the tunnels will be implemented with a spacing of 100 m for both reservoirs. Further, it provides an additional safety margin for the stability of the system. The Figure 16 shows the cross section chosen for the future modelling.

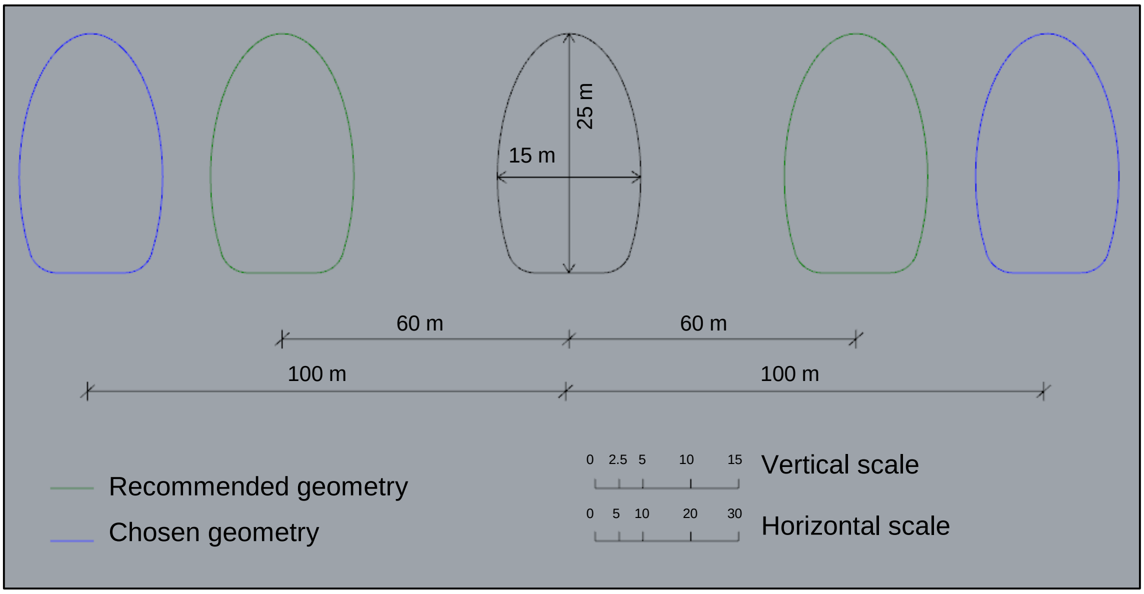


Figure 16: Cross section of the cavern system for the Underground Pumped Hydro Storage

An “egg profile” for the cross section is the best suited form, allowing a rather smooth stress redistribution along the internal walls after the excavation. This shape is only one possibility among others. However, sharp edges have to be avoided. They are not very favourable and would be weak zones in the system. The bottom edges were rounded in order to improve the hydraulic flow conditions.

This cross section has an area of 313.2 m^2. In order to achieve a storage volume of 4,850,000 m^3, a cumulated tunnel length of roughly 15.5 km is required. It can be used for the upper, intermediate and lower reservoir. After investigation of the geomechanical rock parameters, a more detailed study of the cross section is strongly recommended in order to find the most stable configuration.

4.3.2 Spatial assessment and variant study

It is necessary to assess whether it would be possible to find a cavern layout with a storage capacity of around 5 million cubic meters for the upper reservoir fitting underneath the Plabutsch. Basically, different kinds of layouts are possible (raster-shaped, spiral-shaped, etc.). Raster-shaped layouts are suited for traditional drill and blast excavation, whereas spiral-shaped layouts can be excavated mechanically. Three different rough layouts were studied in this part. The first and second one are squared raster of respectively 800 by 1000 m and 500 by 1500 m. The third one is a longitudinal tunnel system of 4 tunnels with a length of 4000 m. Figure 17, Figure 18 and Figure 19 show the different variants in the 3D model. The Plabutschtunnels are represented in brown and the reservoir in blue.

Variant 1 and 2 are quite similar and have the big advantage to be compact systems. A lot of storage capacity is offered on a restrained area. Another advantage of the raster layout is the wave propagation. In fact, while pumping the water up, the height of the waves induced by the water flowing in the reservoir will be much smaller because of the complex geometry. The overburden is always equal or higher than 100 m which is assumed to be sufficient if the rock quality is high enough. Such layouts can be extended rather easily by excavating further from one side.

Variant 3 is not a very compact solution, but such tunnels are straight forward to build because of the very simple arrangement. Regarding wave propagation, this system is more critical than the previous ones. The long linear caverns will induce higher waves while pumping the water up. Another problem is the very thin overburden at the edges of the tunnels, which is not more than 20 to 30 m. This could cause stability problems and induce greater costs for supporting measures. The extensibility of such a geometry is quite limited.

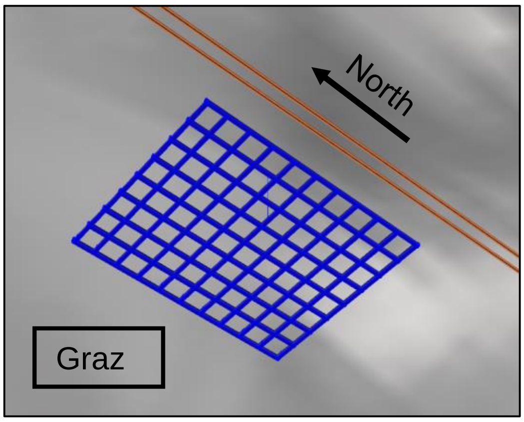


Figure 17: Variant 1, raster layout 1000 by 800 m

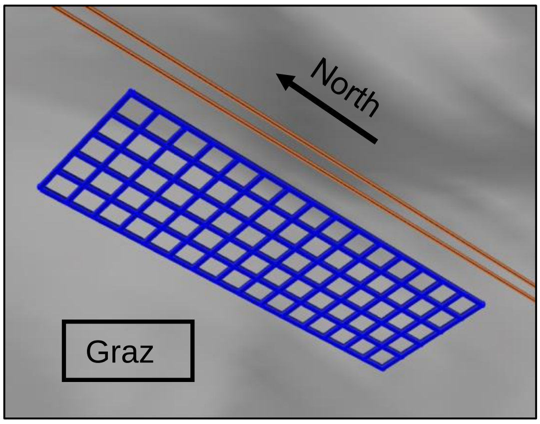


Figure 18: Variant 2, raster layout 1500 by 500 m

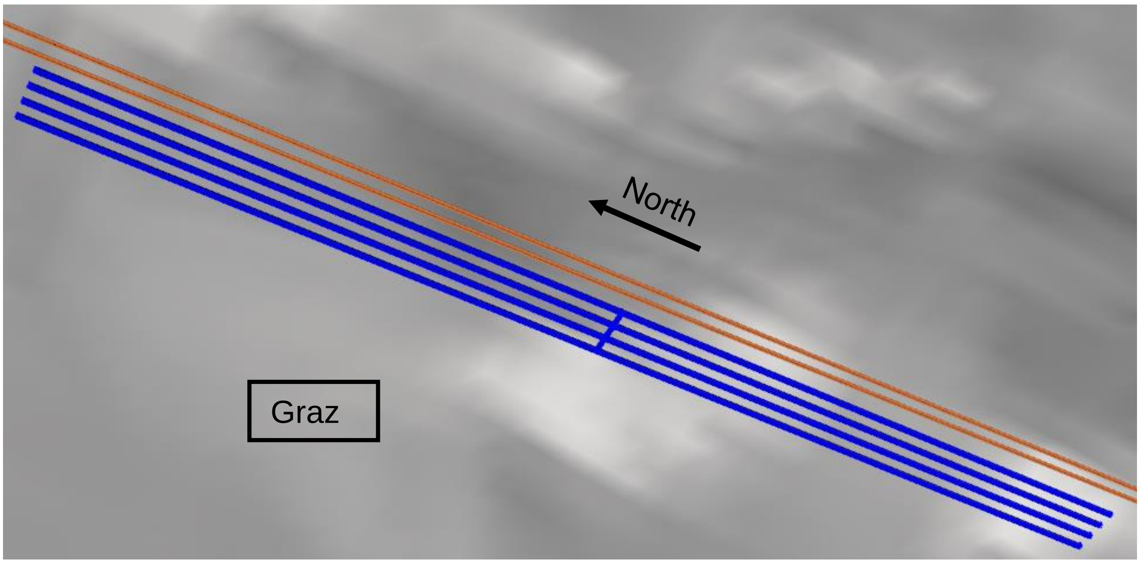


Figure 19: Variant 3, 4 tunnels with 4000 m length

The aim of this part was mostly to find out whether there is enough space available under the Plabutsch to place a water reservoir with a capacity of round about 5 million cubic meters or nor. The 3D model is clearly showing that there is enough space, and different layouts would be possible, each having advantages and drawbacks. In a further step, a more elaborated reservoir layout will be presented.

4.3.3 *Final layout of the cavern system*

As already explained in a previous part, one advantages of building both reservoirs underground is a high geometrical flexibility of the layout and the extensibility of the cavern system, even during power plant operation. Therefore, the geometry cannot be kept as simple as a squared raster or a few parallel tunnels. For the upper reservoir, it has to take the topography into account and be excavated in a way that is using the available space as good as possible. To be able to expand the reservoir without stopping the operation of the power plant, it must be possible to isolate one part of the cavern system during the further construction work. One scientific paper from the 1973 is presenting a rather interesting layout [7], which could be very well suited for the present feasibility study. This layout is shown on Figure 20.

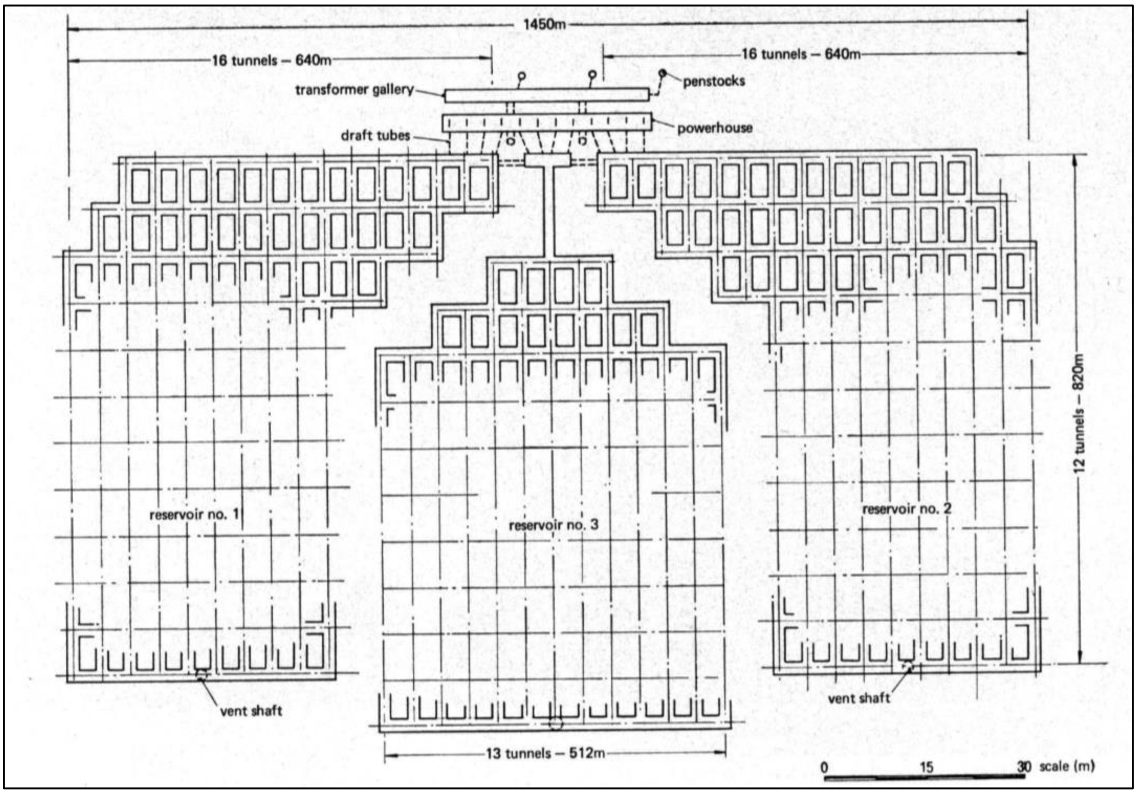


Figure 20: Layout of a lower reservoir [7]

The reservoir is composed of three independent part, all connected at the central point. This point will be at the lowest elevation and host the pressure shaft going vertically down to the turbines and pumps. Each part will be a flexible arrangement of interconnecting caverns. In order to get a natural water flow toward the central point, an inclination of 0.5 % in each direction is necessary. The spacing between each tunnel axis is 100 m. The total volume of the cavern system for this layout would be around 5.5 million cubic meters. The length is 1600 m and the width is 600 m. It can be seen on Figure 21.

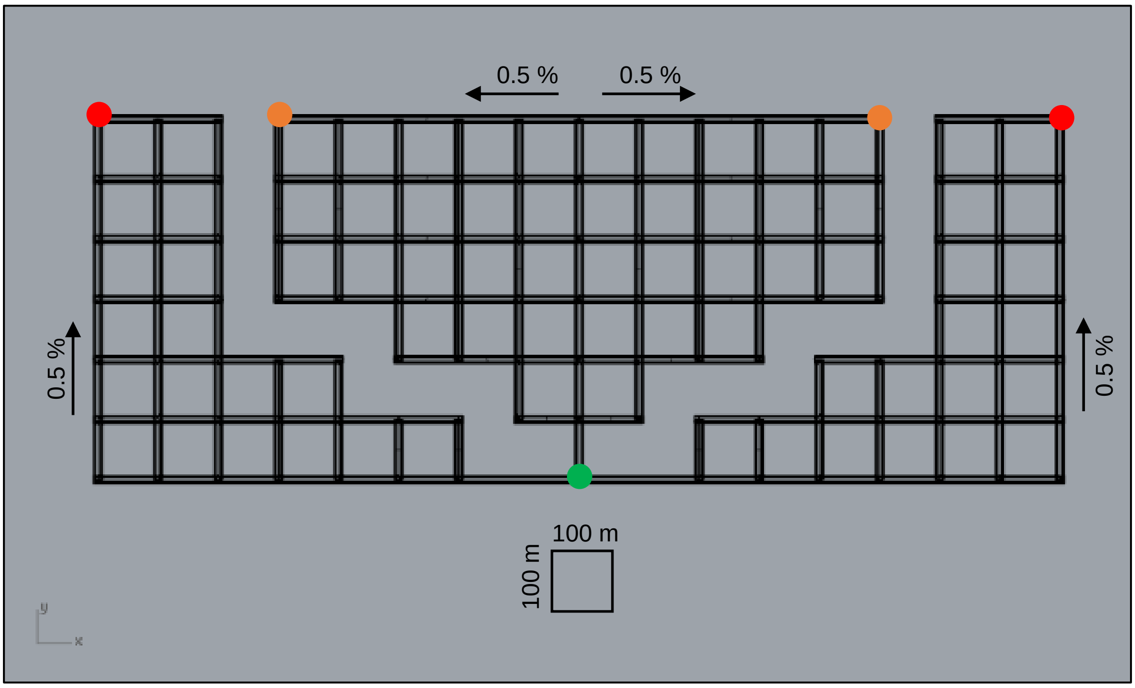


Figure 21: proposed final layout of the cavern system for the upper reservoir

By excavating the cavern system at approximately 375 m underneath the summit of the Plabutsch, the lowest point would be at an elevation of 372 m a.s.l., around 20 m higher than the city. This is an advantage for the future usage of the power plant as seasonal heat storage. Water can flow by gravity toward the district heating network of the city and has then to be pumped up again.

The lateral spacing between the upper reservoir and the eastern Plabutschtunnel is around 130 m, which is enough to avoid any influences of the existing structures during the construction and the operation of the UPH. Additional investigation measures have to be taken in consideration before starting the excavation to confirm the previous statement. The minimal overburden is 100 m, which can be considered as sufficient if the surrounding rock is stable. Therefore, only a few additional supporting measures might be necessary to assure the integrity of the reservoir.

The last positive point of this layout is the extensibility. As it can be seen on Figure 22, it would be possible to extend the part on the right toward north and the part on the left toward south, as the overburden is still very important in these areas. Nevertheless, the cavern system cannot be extended over its entire width. Step by step, the width of the interconnecting caverns have to be reduced to fit with the topography. Figure 22 shows the 3D model of the cavern system underneath the Plabutsch.

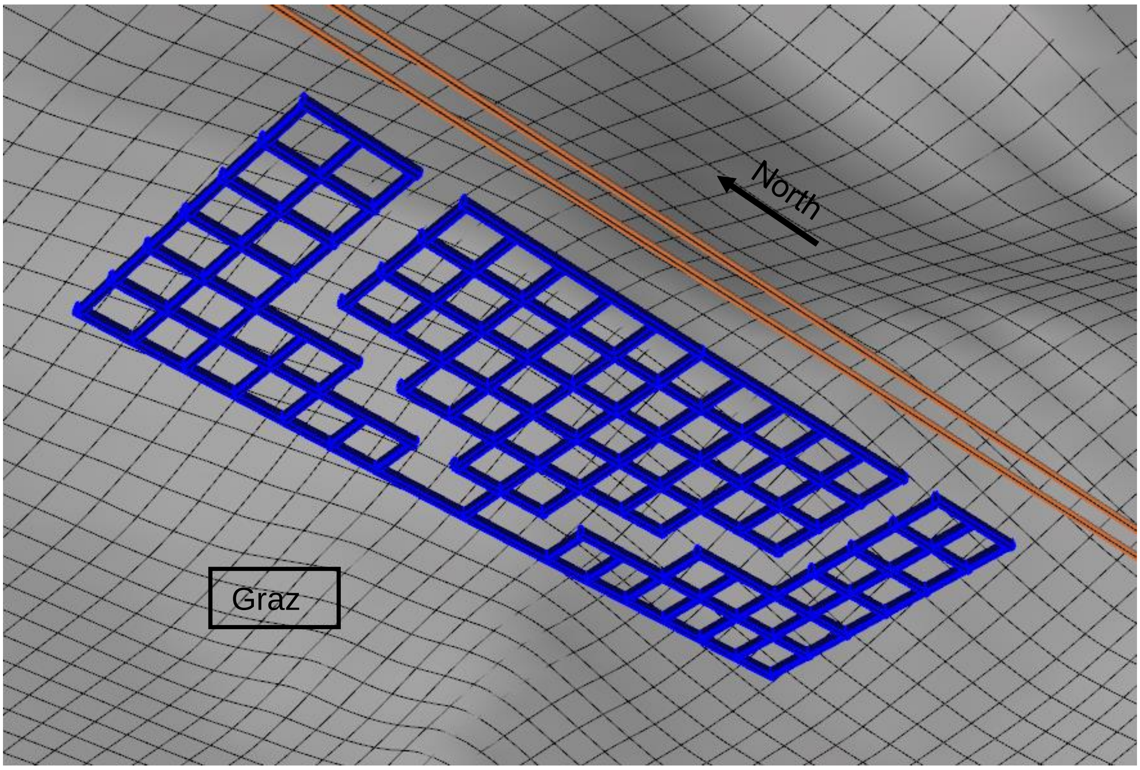


Figure 22: 3D representation of the final cavern system directly underneath the Plabutsch

The red points on Figure 21 are the highest points of the cavern system. The orange points are the highest points of the central part of the cavern system. The green point is the lowest point of the reservoir. The water stored in each part of the system will naturally flow from the highest points toward the lowest point where the vertical power water way is constructed. The height difference between the red points and the green one due to the inclination of 0.5 % in each direction is 7 m; 4 m in the x-direction and 3 m in the y-direction. The height difference between the orange points and the green one is 5.5 m; 2.5 m in the x-direction and 3 m in the y-direction.

4.4 Intake Structures

4.4.1 Boundary conditions

The intake structure is the constructional transition between an open basin or water stream to a closed pipe, in other words the change from an open water surface flow to a pressurised flow. In such a hydraulic scenario, air trapped or spinning flow could occur at the intake and could cause operating problems to the machinery. such problems occur most of the time in “Low-Head-Structures”. Bottom outlets of dams normally operate under a very high water head; therefore, such problems should not occur. However, it might occur if the water level in the reservoir is very low. As the water level in the reservoir only 25-30 m higher than the intake point is, this problem has to be considered.

Withdrawal direction, constructional feature and distance to the basin walls or bottom are the parameters influencing the air drag into the pressurised section. In general, a vertical downward oriented intake will tend to drag air inside of the pressure shaft easier than a horizontal or vertical upward oriented one. This is shown on Figure 23.

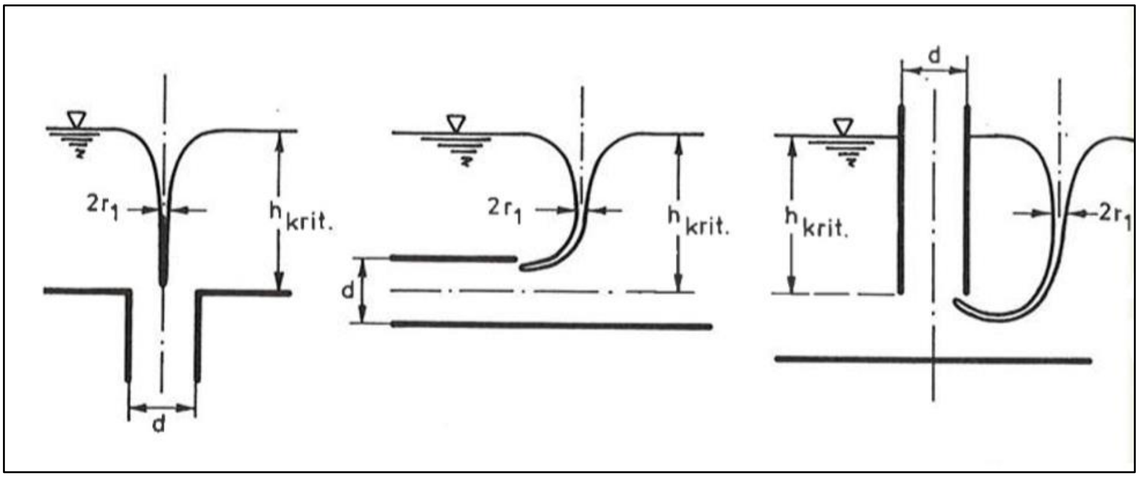


Figure 23: Air drag into a pressure pipe for different pipe layouts [21]

The most important question which has to be clarified during the design of the intake structure is the determination of the required coverage of the intake as the most effective measure to prevent air intrusion and to reduce the rotational flow in the pressure shaft [21]. Gordon´s work in 1970 brought good results regarding the required covering of the intake for the practical design. Figure 24 shows the functional relationship between critical coverage and geometrical parameters of the intake structure. It is specified that within the relevant area of Froude number corresponding to a moderate vortex buildup, a linear critical line can be assumed.

$$(h/D)_{crit} = K \cdot Fr \qquad (2)$$

The slope of the curve can vary as a function of the approaching flow characteristics and is defined by the parameter K.

K = 1.7 for symmetrical approaching flow

K = 2.3 for asymmetrical approaching flow

As general solution is to keep in mind that by increasing flow circulation, the value of K increases as well, resulting in a steeper slope.

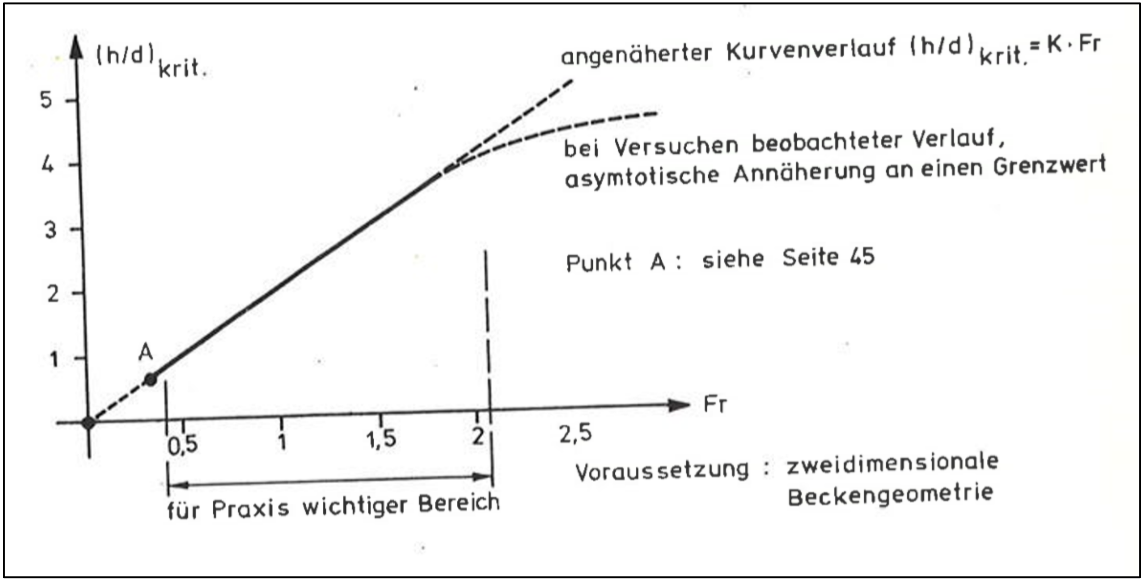


Figure 24: Relationship between Froude number and critical water depth for air drag into a pipe [21]

To estimate the required water coverage above the intake structure, the approaching flow velocity as well as the geometry of the intake has to be known. If the inlet would be a simple vertical overhanging shaft with 3.88 m diameter with a water velocity of 6 m/s in the shaft, the corresponding Froude number is approximately 1.0. The corresponding ratio $(h/D)_{crit}$ from Figure 24 is 2.0. The coverage depth has to be around 8 m, which would be uneconomic for the power plant. In fact, the cavern system has a height of 25 m and a very small ground inclination, which means that almost 1/3 of the storage volume would be underneath the intake level, so-called dead storage.

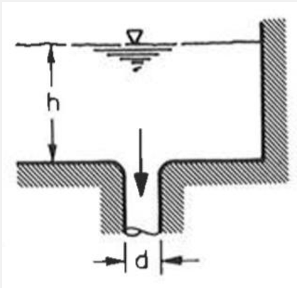
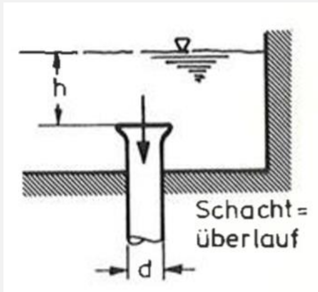
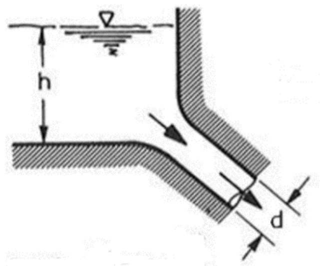
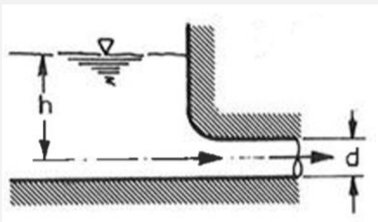
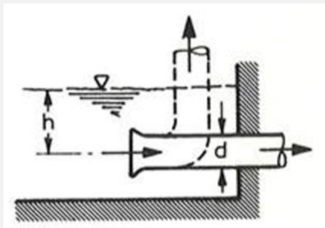
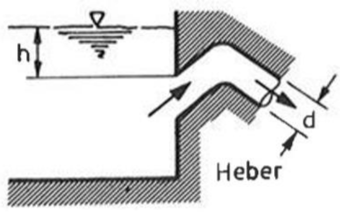
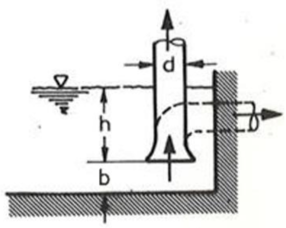
The geometry of the intake structure has to be somehow adapted to reduce the coverage height. One solution would be to design a horizontal overhanging intake structure withdrawing water from each three caverns separately. The detailed geometry of the intake structure will be shown in chapter 4.4.2. The approaching flow velocity has to be kept underneath 1 m/s in each cross section. As the water is flowing from three different tunnels toward the intake structure, a cross section of 23 m^2 is required in each part. This area corresponds to the discharge of 70 m^3/s (chapter 3.7.2) divided by the flow velocity of 1 m/s and distributed between the three cross sections.

For the calculations, an equivalent diameter of 5.4 m has been used, corresponding to an area of 23 m^2. With this parameters combination, the corresponding Froude number is approximately 0.14. The corresponding ratio $(h/D)_{crit}$ on Figure 24 is 0.3. The coverage depth has to be around 1.62 m by full load, which would be much more viable for the power plant. The three tunnel parts connecting respectively each part of the reservoir to the centre point can be excavated with a greater base inclination to reduce the dead storage very efficiently and enable a full emptying of the reservoir if needed. An inclination of 8 % will be taken over a length of 100 m. The elevation difference will therefore be 8 m. It will enable to keep the dead storage only in the first part of the reservoir. This part of the reservoir constituted of the 3 connection tunnels with a greater base inclination and the intake structure will be called intake.

4.4.2 Geometry of the intake structure

An intake structure is needed to guide the water from the reservoir into the power water way. It is located at the lowest point of the reservoir, where the three parts converge, so that the water can flow naturally toward it. Usually made out of concrete, it provides relatively clean water, free from floating material and rock pieces. As the reservoir is planned to be built underground, it does not communicate with any natural water body, and therefore floating material and sediments transport are not a large problem. Further, as the storage volume is very big, the water velocity in the reservoir will be very small. As an effect, the entire cavern system will act like a sedimentation basin avoiding sediments to go into the power water way. Nevertheless, a rock trap will be integrated in the design of the intake structure to assure that no sediment will be brought into the power water way. Table 4 shows several basic layouts for the upper and lower intake structure. As explained in chapter 4.4.1, the intake structure can be a mixed solution inspired from the table below.

Table 4: Different possible intake structure geometries [21]

Direction of Water Intake	**Intake succinct with basin wall or bottom**	**Intake overhanging in the basin**
Vertical downward		
Inclined downward		
horizontal		
Inclined upward / Vertical upward		

4.4.3 Layout upper intake

As the pressure shaft connecting the upper reservoir with the power cavern is vertical, the first type “vertical downward” seems to be well suited. However, this type of intake would lead to problems regarding rotating flow and air trapped in the pressure shaft. Further, during pumping operation, the water jet would flow in a very turbulent way in the cavern system, especially if high water temperatures are achieved. The flow has to be somehow guided toward each part of the cavern system to make the transition between pressurised flow to free surface flow as smooth as possible. A Further aspect is the rock trap effect. An overhanging intake would provide a natural rock trap in front of the intake structure. As the cavern system must remain unlined, it could happen that small rocks fall on the ground.

Therefore, the intake structure has to be a vertical overhanging intake, combined with three horizontal confusors constructed toward each cavern part and directly connected with the vertical pressure shaft. It would ensure low flow velocities at the intake structure entrance and a smooth transition to the pressure shaft.

The first idea was to design a simple vertical overhanging intake and connect it with 3 rectangular confusors. Figure 25 shows the first draft.

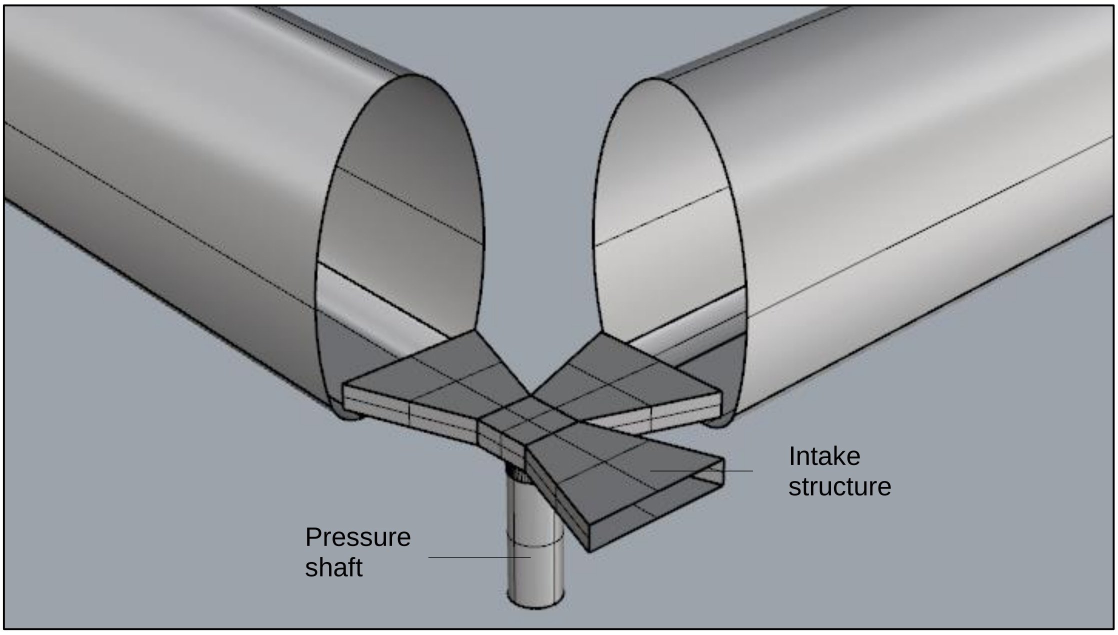


Figure 25: First draft of the upper intake

During turbine operation, the flow will be guided toward the pressure shaft with increasing velocity. During pumping operation, the water jet will come upward, hit the concrete structure, and go toward the caverns. The transition between pressure shaft and intake structure would be mushroom-shaped to obtain a smooth hydraulic transition.

Nevertheless, this structure has some drawbacks. First, it would be difficult to access to the pressure shaft for revision works. Then, the function of sand trap would be fulfilled, but the sediments would accumulate underneath the structure and it would not be possible to remove them easily. Finally, the geometry has to be revised in order to reduce as much as possible the hydraulic losses.

For the second draft, the central part of the intake was fully integrated to the caverns. Sand traps, closing devices, surge chambers, and aeration/access tunnel were designed for each arriving tunnel. Figure 26 shows the second draft. Each part of the system will be explained in the further sub-sections. Detailed 2D drawings of the intake structure can be found in Annexe 1: Detail drawings of the upper intake structure.

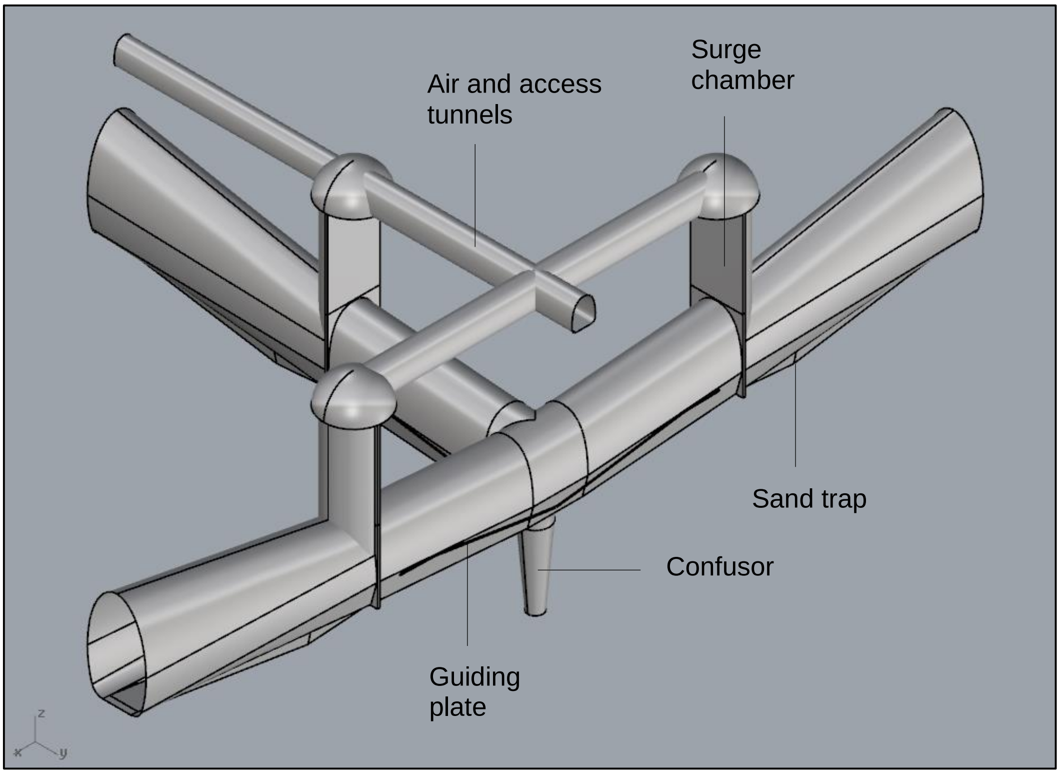


Figure 26: Final draft of the upper intake

4.4.3.1 Central Part of the intake

The central part is the intake structure composed of the pressure shaft coming vertically upward, a concrete guiding plate, and guiding pillars. The intake is fully integrated to the geometry of the cavern and the base has a trapezoidal form. The layout can be seen on Figure 27.

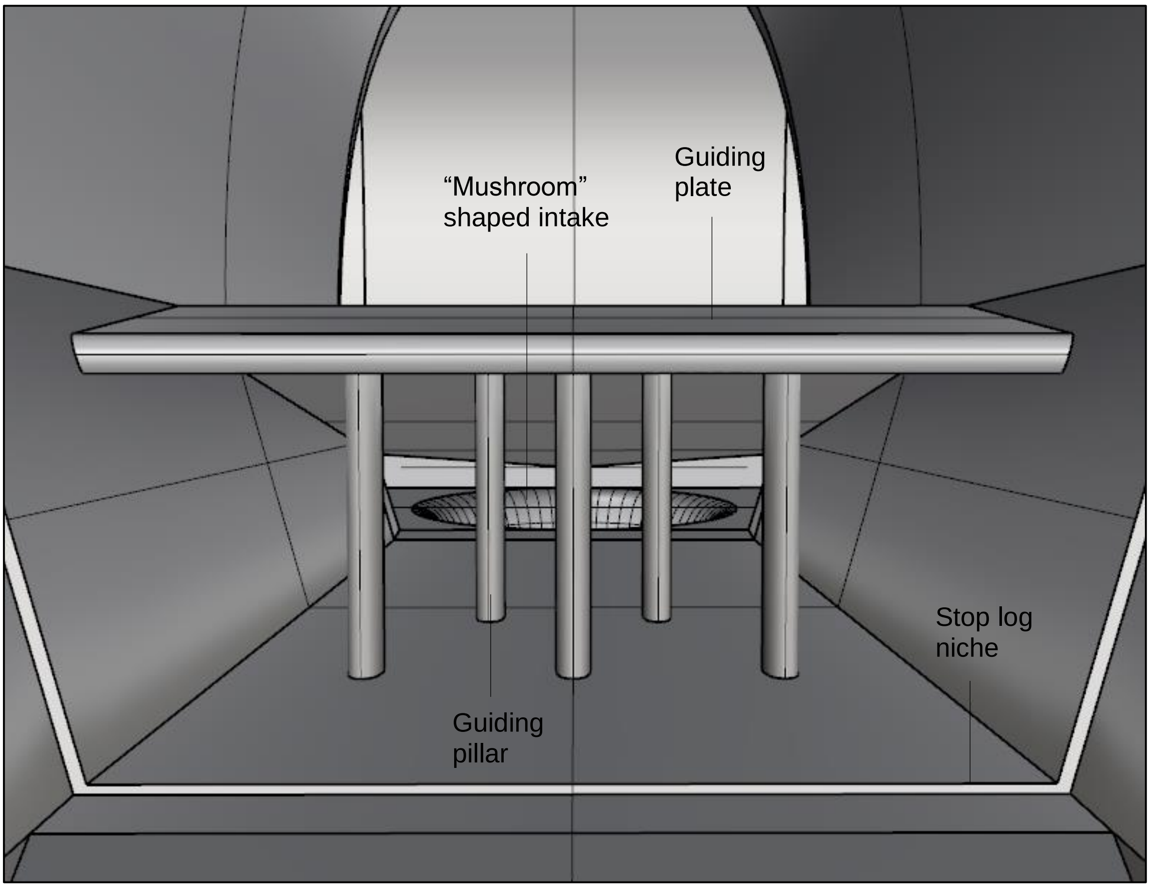
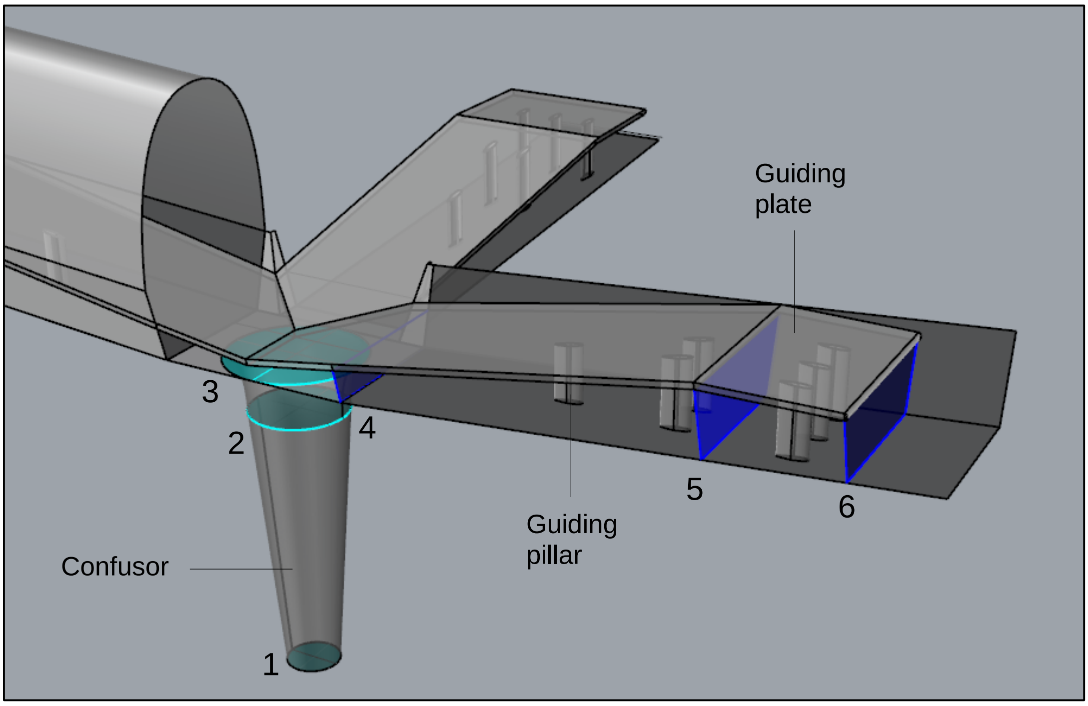


Figure 27: Layout of the upper intake structure

The full adjustment of the intake to the cavern boundaries has several advantages. First, the usage of the entire cross section enables the construction of a very efficient confusor. In this case, it has been designed with an initial height above ground of 2.5 m, an ending height of 4.0 m and a total length of 33.75 m. The concrete structure is split in two parts; the first one has an inclination of 18 % over 25 m, and the second one is horizontal over a length of 8.75 m. The inclined part ensures the diffusor function. The flat part at the end is guiding the water horizontally toward the caverns in a very smooth way.

Ahead of the intake structure, a circular confusor has been designed over 20 m with a mushroom-shaped transition to the bottom of the cavern over a length of 4 m. This provides a smooth velocity transition from the pressure shaft to the end of the intake structure. Table 5 indicates the different cross sections and the theoretical occurring flow velocities.

Table 5: Velocity profiles in the upper intake



Section number	1	2	3	4	5	6
Discharge [m³s]	70.0	70.0	70.0	23.4	23.4	23.4
Area [m²]	11.82	39.37	78.54	26.70	56.82	48.04
Velocity [m/s]	5.92	1.78	0.89	0.88	0.41	0.49

As it can be seen in the table, the velocity in the pressure shaft is reducing slowly from 5.9 m/s to 0.89 m/s. Then, the velocity at the beginning of each cavern section (4) is almost equal to the one at the end of the circular section (3). This allows to reduce the shock loss and therefore increase the hydraulic efficiency. Then, the velocity decreases slowly up to 0.49 m/s at the end of the intake structures.

The second advantage of this design concerns the load bearing. First, the guiding pillars which are built to guide the flow will also have a bearing function. They will transfer the load of the overlaying guiding plate to the ground. Further, the inclination of the concrete slab will not allow the water above it to flow away, even if the reservoir is fully emptied. This water load must balance the dynamic load from the water coming upward from the pressure shaft during pumping operation.

The dynamic force produced by the water flow can be calculated with the following formula:

$$F_w = \rho \cdot \mathrm{Q} \cdot \mathrm{v} \tag{3}$$

With:	F_w … Dynamic water force	[kN]
	ρ: … Water density	[kg/m^3]
	Q: … Discharge	[m^3/s]
	v: … Hydraulic head	[m/s]

In this case, with a discharge of 70 m^3/s, a flow velocity of 1.7 m/s and a specific ddensity of 1000 kg/m^3, the resulting dynamic water force on the structure would be 119 kN. The water impounded above the pressure shaft is at least 2.7 m deep, which corresponds to a water pressure of 27 kPa on the slab. If one assumes that the load-carrying area is only the guiding plate of 7 x 7 m above the pressure shaft, the resulting water force would be 1323 kN. Therefore, the stability of the intake structure is not endangered. Further, the intake structure has to be designed to bear the water load above it when the reservoir is completely filled with water. A reinforced concrete slab of 0.5 m is appropriate.

4.4.3.2 Sand trap

A further advantage of this design is the sand trap system which can be seen on Figure 28.

As it can be seen, the sand traps can be constructed before the intake structure, so that no overhanging intake is needed. The deepest point of the sand trap is located 3 m lower than the base of the cavern, which is enough to avoid sediments entering into the pressure shaft. When the minimal operating level is reached, the flow section at the deepest point of the sand trap is 74 m^2, which would result in a flow velocity of 0.32 m/s by full load. As it is very unlikely that such a load case occurs, the flow velocity in the sand trap will always remain lower than 0.32 m/s. This flow velocity is small enough to enable most sediments to settle

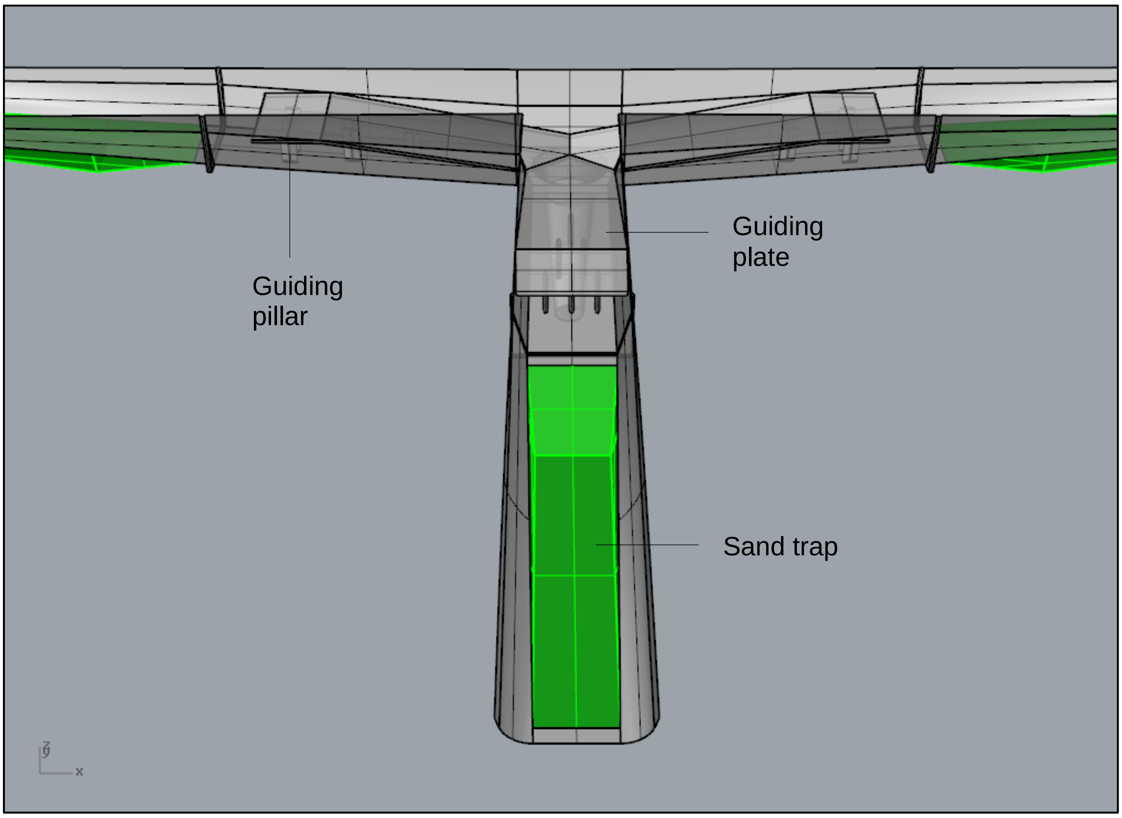


Figure 28: Layout of the sand trap

In order to obtain a drivable slope, the base inclination of the sand trap has been designed with 15 % inclination. This is necessary to ensure that excavators are able to access the emptying of the sand trap and during maintenance works of the intake structure.

4.4.3.3 Closing device and surge chamber

The closing device will be located between the sand trap and the intake structure. It will assure the water tightness of each part of the cavern system in case of revision or expansion of the reservoir. Directly before the closing device, a surge chamber has been designed in case of a sudden rises of pressure. The closing device and surge chamber can be seen on Figure 29.

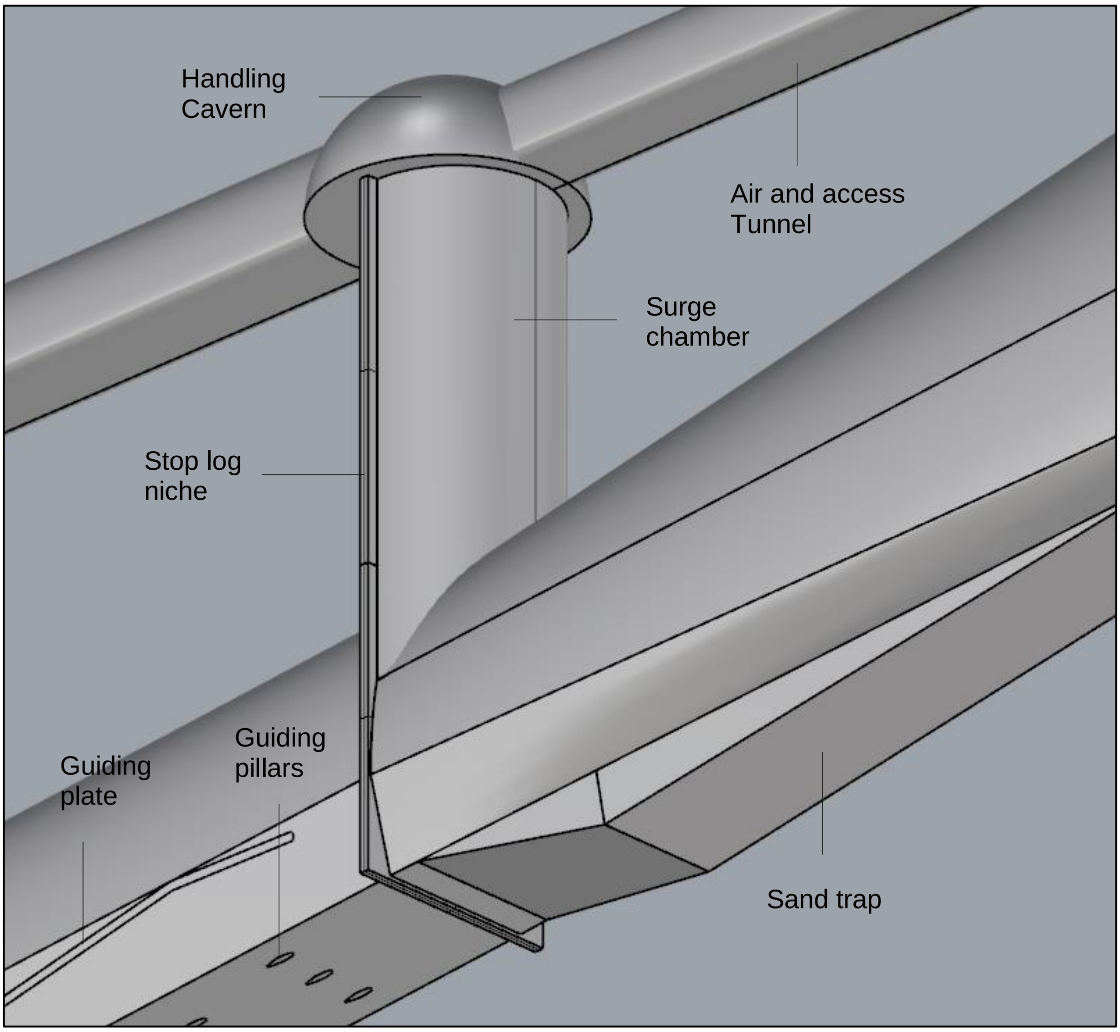


Figure 29: Layout of the stop log and surge chamber

The most appropriate closing device for this layout would be a stop log in several pieces. The guiding slot has to be at least 20 cm wider than the caver edges in order to achieve complete water tightness. At the top, a cavern linked to an access tunnel must be excavated to bring the elements. A crane can be used to let the elements down and pull them back up. The surge chamber is a half circular shaft. The top edge of the chamber has been modelled 5 m higher than the highest possible water level in the entire cavern system. Depending on the expected fluctuation in water pressure, this elevation can be adapted. The surge chamber will as well be used as air vent shaft. It will be linked with the air pressure compensation system through a horizontal gallery. This will be explained more in detail in chapter 4.5.

4.4.4 Layout lower intake structure

The intake of the lower reservoir has to be suited for pumping operations and provide a smooth water discharge during electricity production. Therefore, the circular pipe coming from the power cavern has to be integrated almost horizontally to the cavern system. The 3D model of the lower intake structure can be seen on Figure 30.

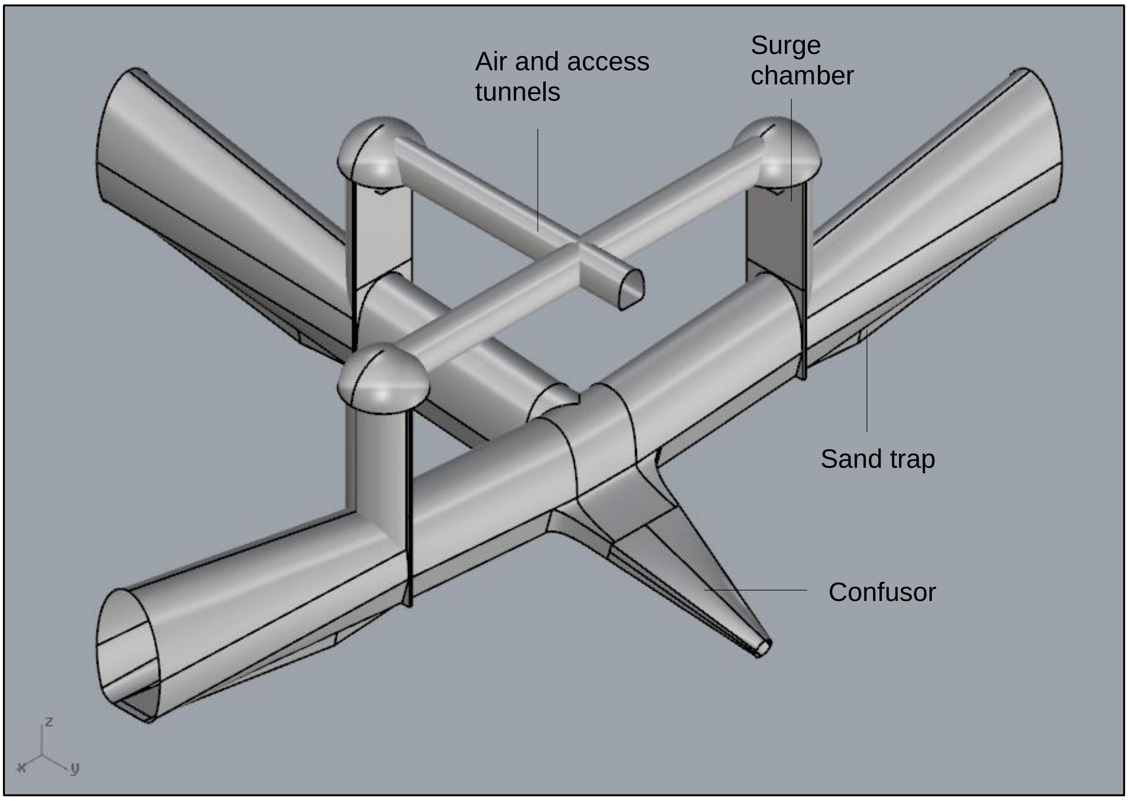
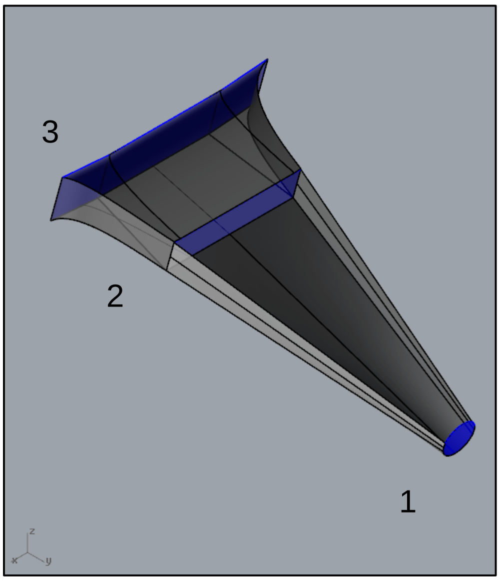


Figure 30: Layout of the lower intake

The geometry is similar to the upper intake design, with the only exception of the connection between circular pressure pipe and intake. The opening was designed rectangular and the transition zone to the circular pipe is round about 30 m long. The cross section of the opening has to be wide enough so that the developing flow velocities remains lower than 0.5 m/s in the entrance area. As the maximal operation discharge is 70 m^3/s the area of the opening must be equal or higher than 140 m^2. If this condition is met, no further intake structure is needed to control and guide the flow toward the different cavern parts. If higher flow velocities are expected, measures like guiding walls have to be taken to distribute the flow equally toward each part of the reservoir. Apart of the intake structure, the rest of the cavern system directly proximate to the intake (stop log, sand trap, surge chamber, inclination and geometry) stays the same as for the upper reservoir. Table 6 indicates the different cross sections and the theoretical flow velocities.

Table 6: Velocity profiles in the lower intake



Section Number	Discharge $[m^3s]$	Area $[m^2]$	Velocity [m/s]
1	70	11.82	5.92
2	70	59.21	1.18
3	70	148.02	0.47

As it can be seen in the Table 6, the velocity in the transition zone reduces from 5.9 m/s to 0.47 m/s over a length of approximately 40 m. The minimal operating level is corresponding to the elevation of the highest point of the intake, therefore 6m above the lowest ground elevation.

4.4.5 Layout intermediate intake structure

The intermediate intake structure must provide a smooth hydraulic transition between the two stages. To achieve this, the hydraulic losses have to be reduced as much as possible.

4.4.5.1 Excavation volume

According to chapter 3.6.2., all units of one plant (intermediate or lower) must be able to discharge during about 15 minutes, without compensating operation in the other. To gain flexibility, the design was made with a full discharge in the intermediate reservoir during 30 minutes. The required active storage can be calculated by multiplying the full discharge of 70 m^3/s with the time span, which results in an active storage of 126,000 m^3. According to chapter 3.6.1., the global safety margin is 21.16 % of the calculated active storage, which results in an excavation volume of 152,662 m^3. A tunnel length of 500 m with the cross section showed in chapter 4.3.1. is sufficient to obtain the required volume.

4.4.5.2 Central part of the intake

The central part of the intermediate intake structure can be designed in several depending of the utilisation of the power plant. If the power plant is planned to be used as a standard UPH, no special measure to control the water flow in the intake is needed. The layout is shown on Figure 31.

A transition from the circular pipe coming from the upper powerhouse to a rectangular cross section will be needed. The pressure shaft bringing the water to the lower power cavern will be situated right at the end of the transition zone. A Sand trap will as well be designed, but no stop log is needed. Only one tunnel with a total length of 500 m will be excavated. The inclination of the caverns will be 8 % in the first part and then 0.5 % for the rest of the reservoir. For this layout, the flow velocity will be higher and most likely turbulent. If the UPH is planned to be used as well as seasonal heat storage, some additional measures have to be taken in the intake to control and guide the flow, in order to avoid turbulent flow conditions. The layout is shown in Figure 32.

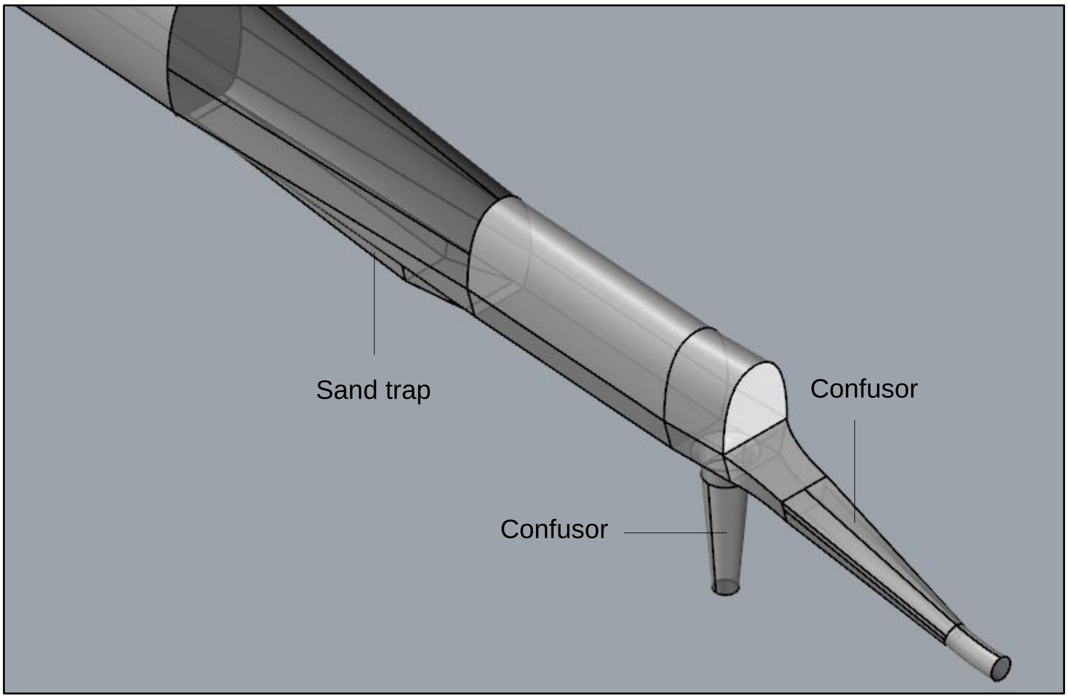


Figure 31: Layout of the intermediate intake for a conventional Underground Pumped Hydro Strorage

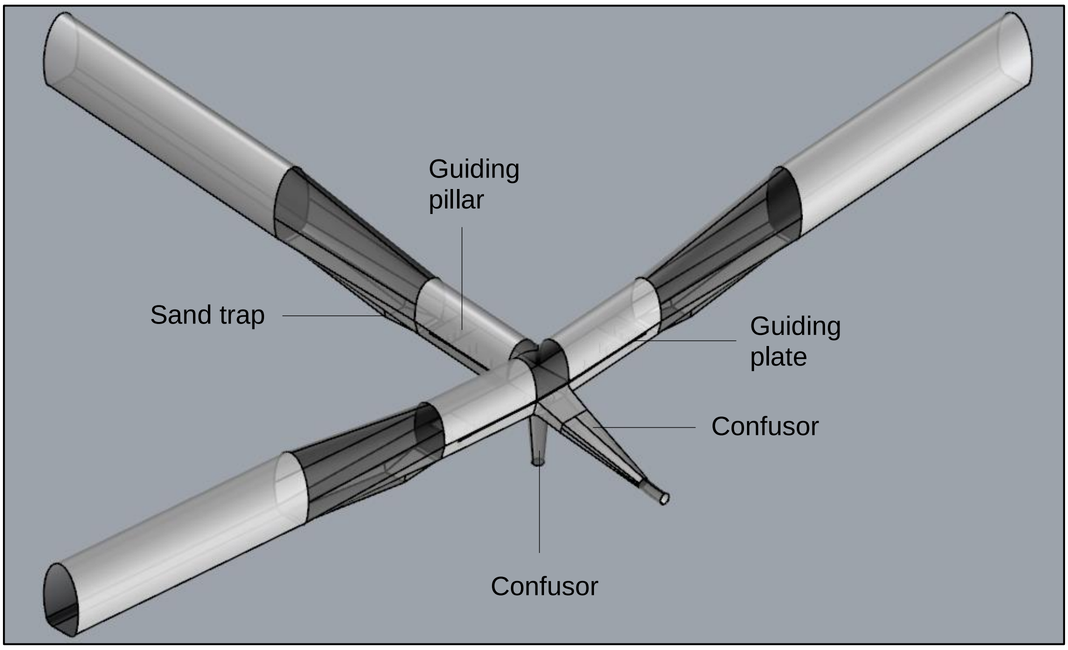


Figure 32: Layout of the intermediate intake for an underground Pumped Hydro Storage combined with heat storage

The design of the central part is in this case a combination of the upper and lower intake. The transition zone toward the upper power cavern and the design of the vertical pressure shaft remain unchanged. The guiding plate and guiding pillars will be built to guide the flow toward the three directions, however higher, in order to match with the height of the rectangular opening in the cavern wall toward the upper power cavern. Laminar flow must occur at this place to avoid high hydraulic losses and assure a smooth transition between the power waterways. The slope inclination is 8 % in the central parts and then again 0.5 %. The total tunnel length is approximately 600 m. A sand trap will be built as well, and again, no stop log is needed.

The interesting point about the intermediate reservoir is the fact that it can work as a flexible interface between the upper and lower reservoir. First, it can be used as surge tank for the lower power cavern. If a sudden closing occurs, the pressure wave will be reflected in the intermediate reservoir, which has enough volume to absorb the upcoming surge wave. Then, it has a regulating function at the begin of the operation. During production, water can first be discharged in the intermediate reservoir to assure that no air will be drawn in the second vertical pressure pipe. When a sufficient water level has been reached, the lower turbine can slowly be started. The same concept is valid during pumping operations.

4.5 Air pressure compensation system

As the power plant is operating in a closed-loop, an air pressure compensation system is required between the different reservoirs. During electricity production, the water mass flowing from the upper to the lower reservoir will compress the air from the free space of the lower reservoir. This air has to flow somewhere or it would produce enormous pressures against the cavern walls and endanger the integrity of the structure. The same principle applies during pumping operation. To avoid it, it is usual to build air shafts between the reservoirs and the outside to keep the air at atmospheric pressure. In the case of a UPH used as seasonal heat storage, this must be avoided in order to keep the heat into the system. Therefore, air galleries and air shafts have to connect the different reservoirs.

Regarding the constructional features, the galleries are integrated at the highest points of the various parts of the reservoir. During the filling of the reservoir, the air will be pushed naturally toward the highest points of the reservoir. During emptying, the air has to enter in the system at a place where there is no water. A horizontal gallery will connect the highest points of each part of the cavern system. The horizontal galleries will then be connected with a vertical shaft toward the intermediate and lower stage. The entire air pressure compensation system can as well be used as additional access to the different parts of the power plant.

4.6 Power waterways

4.6.1 Vertical pressure shaft

The vertical pressure shaft has the function of conveying the water from the reservoirs to the power caverns. It has to be as short as possible to reduce the hydraulic losses due to friction. In the case of the investigated double-stage UPH, two pressure shafts with a length of about 800 m are designed.

The lining, is planned to be made out of concrete, and grouting will be used to fill the annular gap between the lining and the rock excavation. If it is necessary, grouting can as well be used to pre-stress the concrete or prevent eventual high water pressure from a natural groundwater level. Figure 33 shows a typical cross section from a grouted pressure shaft in a fractured rock.

First, it can be seen that the grout is penetrating in the fractures all around the pressure pipe until a certain distance. the circular influence of the grout will be depending on the size of the openings and the characteristics of the chosen grout material [22]. Then, it is possible to see that the grout assures the contact between the rock mass and the lining by filling the annular gap. This is an essential point in order to get a shared load bearing system between the lining and the rock mass.

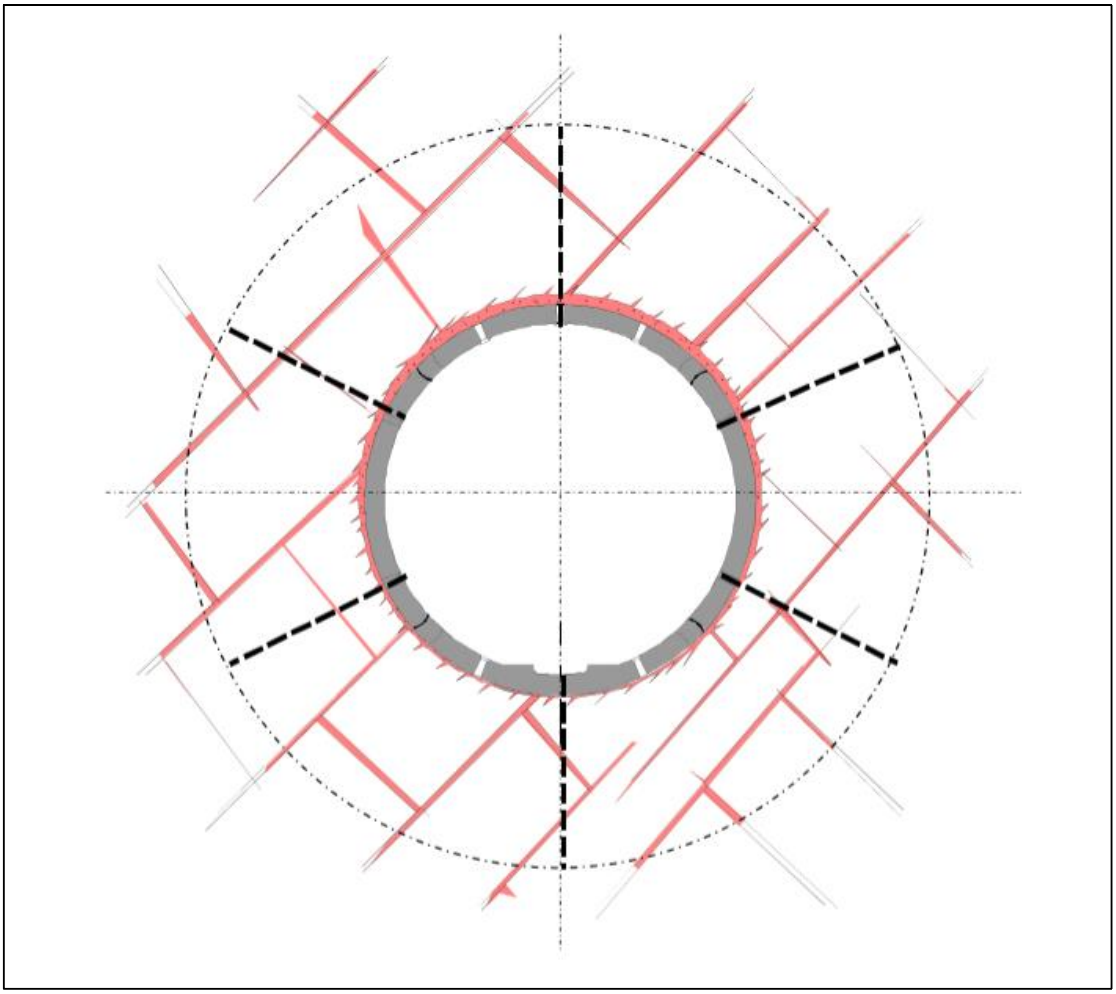


Figure 33: Cross section of a grouted pressure shaft [22]

4.6.2 Horizontal pressure pipe

At the end of the vertical pressure shaft, a horizontal pressure pipe is needed to bring the water to the power cavern. A 90° bending assures the connection between both pipes. On the other side of the power cavern, another horizontal pressure pipe is needed toward the next reservoir. Both horizontal pressure pipes are slightly inclined toward the power cavern. Thereby, the water will flow naturally toward the power cavern, which enables an emptying of the pipes in case of revision. A branching is needed before the power cavern in order to bring an equal amount of water toward each turbine. The branching can be seen on Figure 36. The total cross section after the branching must be the same as before due to continuity. With two pipes of 2.74 m diameter, the total cross section would remain the same.

4.7 Power cavern

The power cavern is the place where the pump turbines, motor generators are located. A lot of space is needed to host these machines, therefore is the required excavation section significant. To come to an economical solution, several criteria have to be met according to Norwegian state of the art regarding underground excavation [23]. Obstacles, related to rock stress and support measures in large underground openings, had to be overcome in a smart way in order to assure project viability. Engineering geologists met this challenge and developed new technologies in the past like bolting, shotcrete, etc., as well as further considerations, which are nowadays a precondition for the constructability and economy of caverns. First, the shape of the cavern is of prior importance. The ratio between height and width of the cavern must be increased as much as possible, combined with a favourable geomechanical shape. The reduced width often induces a considerable saving in the cost of support measures for the ceiling, however depending on the quality of the surrounding rock mass. Further, the location of the transformer gallery has to be chosen in a smart way. Among all possible options, one arrangement is preferred because of safety considerations. The transformer should be in a separate gallery, if possible downstream from the powerhouse and combined with a draft tube gate chamber connected to the downstream penstock, as shown on Figure 36. Then, since the cost of excavating works for an underground facility is more or less directly proportional with the excavated volume, great emphasis has to be placed on solutions to reduce the space requirements. The three following consideration have been taken from the book "Norwegian Tunneling Today" from 1988 [23].

- For Francis-type turbines with closure valves, to position the pressure conduit´s centreline non-perpendicularly to the powerhouse axis, in order to improve accessibility to the valves and reduce the cavern width.
- To reduce to a bare minimum the workshop and repair facilities in the caverns. Such space is now largely wasted, since modern power units, although extremely reliable, are also very complex and are better shipped back to the manufacturer for any revision work to be done.

- To keep an eye on the possibility of a permanent use of any of the auxiliary adits that are frequently excavated to gain temporary access to various points of attack. An adit from the access tunnel down to the tailrace may for instance be converted later into a tailwater surge chamber. Likewise may an adit from the access tunnel up to the top heading for the powerhouse cavern be used later as a permanent reservoir for cooling water.

Other concepts can serve to cut costs either directly through reduced construction time or by saving downtime during maintenance operations.

- Ceiling support system with systematic bolting immediately after excavation of the top heading followed by placing of reinforced shotcrete.
- The rockbolted crane girders ensures early erection of the permanent overhead crane, which will then be available also for the initial concrete works, the erection of the spiral casings etc., reducing the construction time.

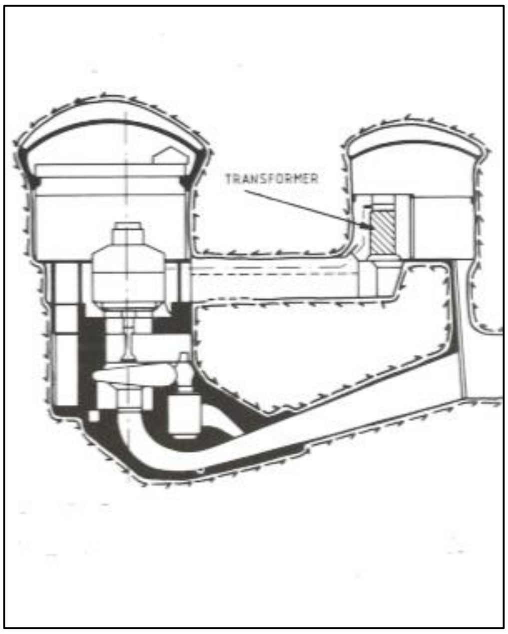


Figure 34: Transformers in cavern with draft tube gate hoists [22]

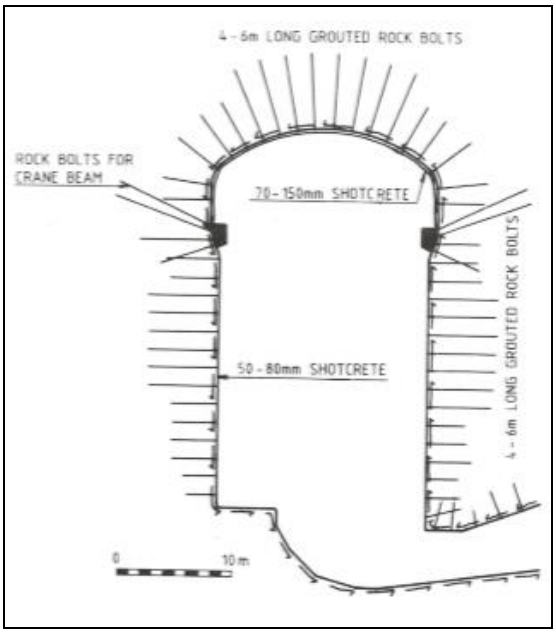


Figure 35: Rock support of cavern roof and walls, and rockbolted crane girders [22]

The chosen layout was inspired from the Power Plant Limberg II in Austria [24]. The Power cavern is there 200 m underneath the surface and hosts two Francis pump turbine with a nominal output of 240 MW. The Power cavern can be seen on Figure 36. The transformer gallery has been constructed downstream of the powerhouse, as recommended above.

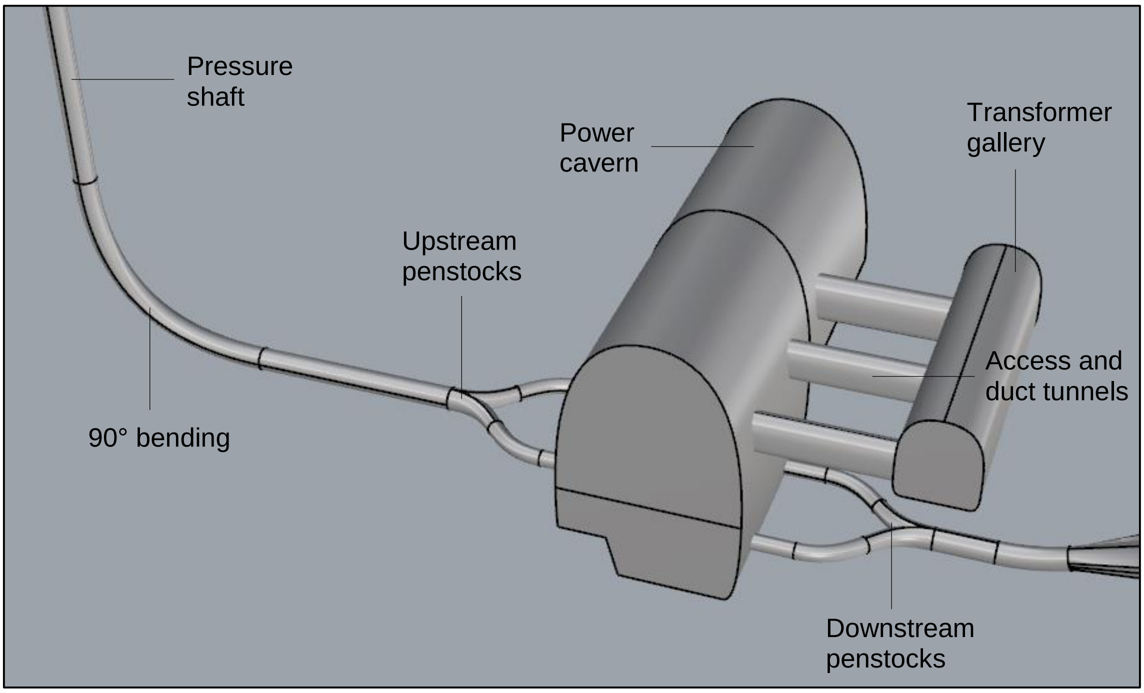


Figure 36: Layout power cavern, upstream and downstream penstock

First, on the left side, the end of the vertical pressure shaft and the 90° bend can be seen. After that, the branching splits the 3.88 m diameter shat into two 2.74 m diameter pipes running separately to the power cavern. The upstream penstocks are located higher than the downstream penstocks to fit with the layout of the pump turbine with vertical wave. Then, in the central part, the power cavern can be seen. It has a maximal length of 60 m, a width of 30 m and a height of 45 m. The section has a horseshoe form which is a favourable geomechanic profile. Finally, on the right side, two bus duct tunnels and an access tunnel are leading to the transformer gallery. Another branching on the downstream side is needed to connect again the two penstocks into one.

One last important aspect to consider is the hydraulic gradient. The smallest distance between the vertical pressure shaft and the power cavern is defined by the maximal admissible gradient of the seepage flow, developing from the vertical pressure shaft toward the power cavern. in very god rock mass conditions, gradient from 8-10 are admissible. In weaker rock mass conditions and especially if joint fillings which are sensitive to wash out are present, the hydraulic gradient should remain underneath 5 [25]. The connection between the bend at the bottom of the vertical pressure shaft and the branching has to be lined with steel to make sure that the hydraulic gradient mentioned before is not exceeded. In this case, the static water pressure at the bottom of the vertical pressure shaft is 800 m and the distance to the power cavern 95 m. The corresponding gradient is 8.4, which is comprise between 8 and 10. Depending on the in situ conditions, the horizontal penstock length can be adapted.

4.8 Closing device

Right below the intake structure, a gallery with clothing device is required. The main reason is to stop the flow in case of maintenance of the intermediate or lower reservoir and the pressure shaft. An air gallery downstream of the closing device between the pipe and the upper reservoir is needed in case the vertical pressure shaft needs to be emptied. The gallery must be accessible easily from outside. However, this air gallery must not be linked with the access gallery, so that no heat dissipation can occur. It must be insulated from outside and connected to the air pressure compensation system. A typical design for such an installation can be seen on Figure 37. The design shows a “mushroom” shaped intake, a confusor and a butterfly valve as closing device. A small gallery with access tunnel has been build round about the closing device [26]. The air gallery is connected to the vertical pressure shaft and isolated from the access gallery. This would be necessary as well for an UPH used as a seasonal heat storage in order to avoid energy losses.

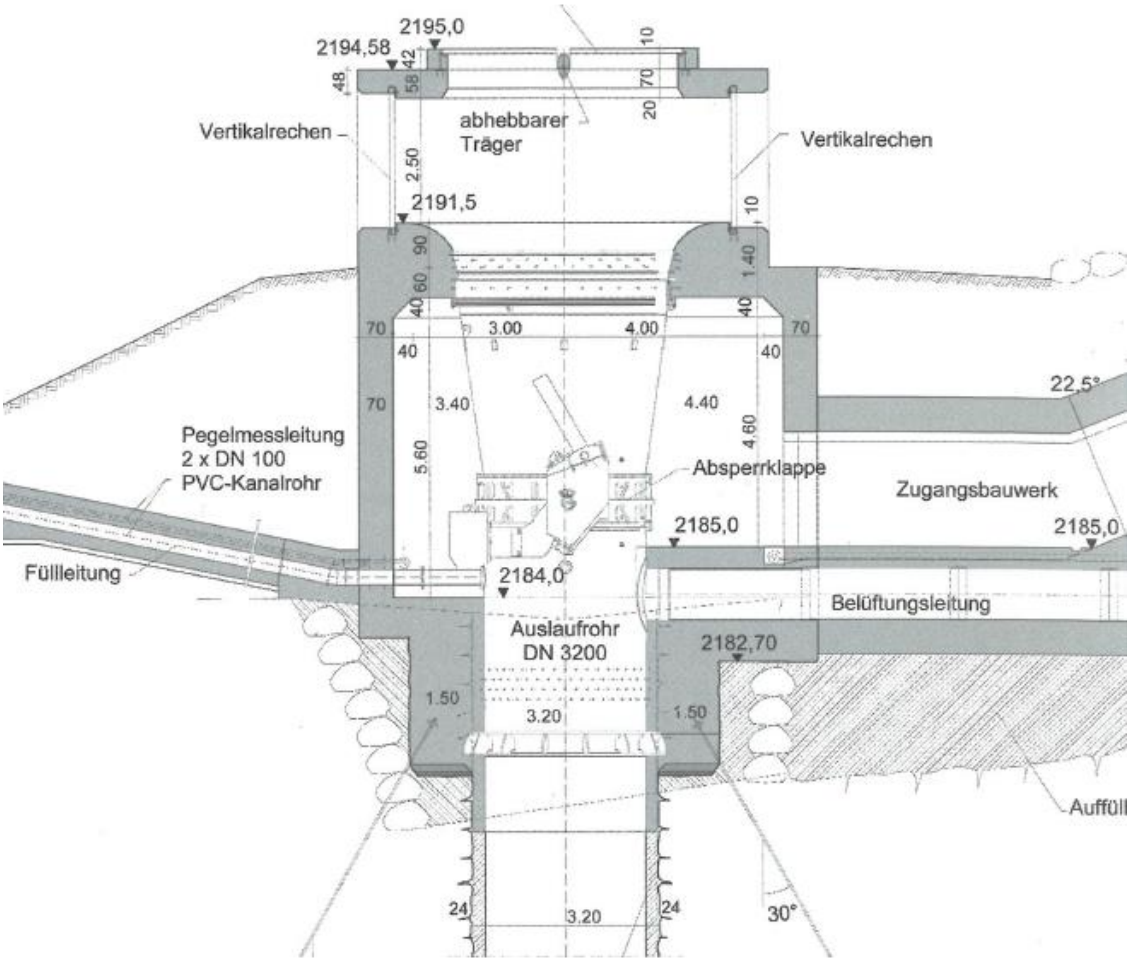


Figure 37: Intake structure PSH Feldsee in Austria [26]

4.9 Heavy hoist shaft

The heavy hoist shaft is the largest shaft of the power plant. It provides a fast connection between the different underground parts of the plant and the surface. It has to be suited for personal as well as for heavy machines.

The diameter of the shaft can be between chosen between 10 and 14 m. Its location has to be chosen so that it is very close to each main part of the plant (reservoirs, power cavern, air pressure compensation system, and access tunnels). At the top of the heavy hoist shaft, a cavern (50 x 40 x 60 m) has to be excavated in order to host the installations needed for the excavation machine (see chapter 7).

4.10 Entire Underground Pumped Hydro System

Finally, it is possible to link each part of the power plant described in the previous chapters to get an idea of how it looks like in 3D view. The hydraulic head between each stage is 800 m. The air pressure compensation system is represented in blue, the heavy hoist shaft in brown and the power caverns in red. The model is shown on Figure 38.

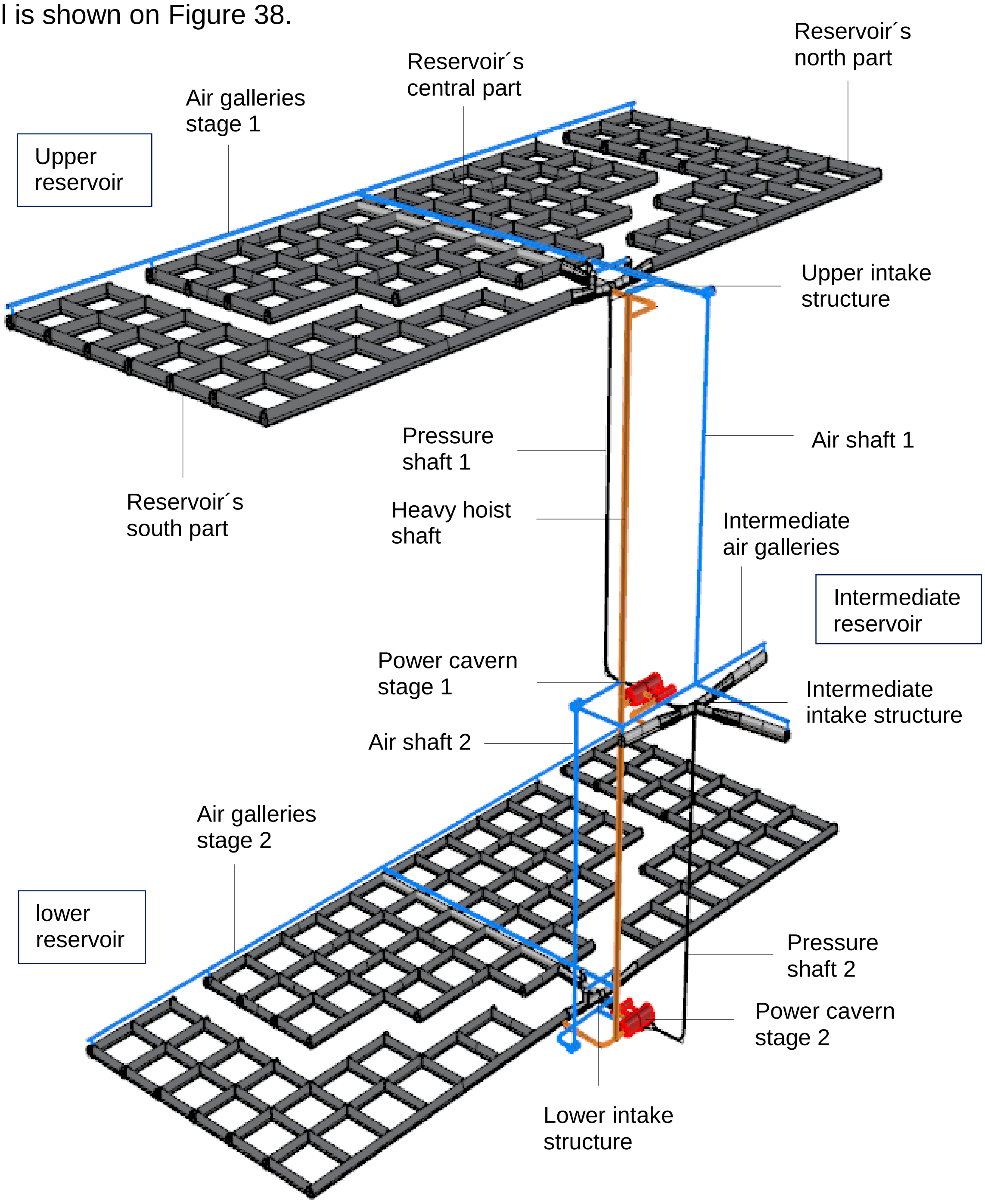


Figure 38: 3D Model of the entire Underground Pumped Storage Hydro system

5. Hydraulic losses

The assessment of the hydraulic losses along the system is of prior importance in the feasibility study. It significantly influences the power plant´s efficiency. The usual approach when considering reservoirs connected by piping system is to write the Bernoulli equation between two points, connected by a streamline, where the conditions are known. The total head between the two chosen points must match. Any increase or decrease of the head due to pumps, friction losses or local losses have to be taken into consideration. The following parts will explain which approaches were used to estimate the losses along the entire system.

5.1 Friction losses

The friction losses are head losses in a hydraulic system due to the effect of the fluid´s viscosity against the pipe surface, creating turbulences in the flow itself. They are the most significant hydraulic losses in the pipes. As it is a significant economic aspect in project, the power water ways are kept as short as possible in order to keep the friction losses as small as possible. Strictly speaking, the energy is not "lost" but converted in heat which dissipates in the environment once the water flows back into natural streams or reservoirs. In the case of a UPH combined which a seasonal heat storage, this energy remains in the system, increasing at the same time the overall efficiency.

There are different approaches to calculate the friction losses like Darcy-Weisbach or Manning-Strickler. Both are taking into account the roughness of the pipe material, the length and diameter as well as the occurring flow velocities. For the further calculations, the Manning-Strickler equation as shown below has been used.

$$Q = k_{st} \cdot {R_{hy}}^{2/3} \cdot I^{1/2} \cdot A \tag{4}$$

With:

Q	… Discharge	[m³/s]
k_{st}	… Strickler coefficient	[$m^{1/3}$/s]
R_{hy}	… Hydraulic radius	[m]
I	… Flow gradient	[-]
A	… Cross section	[m²]

As the flow gradient I is correspond to the head difference over the length, the equation can be re-write in the following form to calculate the friction losses.

$$\Delta_h = \left(\frac{Q \cdot \sqrt{l}}{k_{st} \cdot R_{hy}{}^{2/3} \cdot A}\right)^2 \tag{5}$$

Concerning the Strickler roughness, a value of 85 $m^{1/3}/s$ has been taken for the concrete lining and a value of 110 $m^{1/3}/s$ for the steel lining. Concrete lining will be used in the intake and along the pressure shaft. Steel lining will be used from the 90° bending until end of the pipe junction downstream of the power caverns.

5.2 Local losses

Local losses occur because of a discontinuity like for example a change of direction, dissolution or contraction of the flow. The involved change in velocity lead to an increase of the kinetic energy of the water and subsequent to high turbulent swirling. A part of this energy is converted in potential energy and stays therefore in the system. Another part of this energy is converted into heat which dissipates to the environment [27]. Again, In the case of a UPH combined which a seasonal heat storage, this energy remains in the system, increasing at the same time the overall efficiency.

The approach to estimate local losses is rather simple. The energy height has to be multiplied by a certain local loss coefficient.

$$h_l = \zeta_l \cdot \frac{v^2}{2g} \tag{6}$$

With:		
	h_l … Energy loss	[m]
	ζ_l … local loss coefficient	[kg/m³]
	v … Flow velocity	[m³/s]
	g … Gravitation constant	[m/s²]

The local loss coefficient has to be estimate for each place where a discontinuity can be found in the system. Two books where used to estimate the local losses: “Technische Hydromechanik 1” [28] and “Handbuch der Hydraulik” [27].

5.2.1 Entry loss

The mushroom shaped intake, the local loss coefficient ζ has to be taken between 0.04 and 0.1. An optimal intake is rather hard to design, therefore a value of 0.07 will be taken for the further calculations.

5.2.2 Trash rack loss

The estimation of the local loss due to the trash rack depends on several physical and geometrical factors. It basically constitutes a cross section reduction inducing an acceleration of the water mass. Due to the turbulences occurring while the water flows through the trash rack, a flow separation occurs, who mostly responsible for the losses. The following formula can be used to calculate the local loss coefficient.

$$h_v = k_s \cdot \left(\frac{d}{s}\right)^{4/3} \cdot \frac{v^2}{2g} \cdot sin\alpha \tag{7}$$

With:

k_s	… Profile coefficient	[-]
d	… Rod thickness	[m]
s	… Rod spacing	[m]
v	… Flow velocity	[m/s]
α	… Vert. flow angle	[°]
g	… Gravitation constant	[m/s²]

First, it can be seen that the energy loss can directly be calculated from the different parameters. If one wants to back calculate the local loss coefficient, the following formula can be used. The Formula is derived from *Kirschmer* and the profile coefficient can be taken from Figure 39.

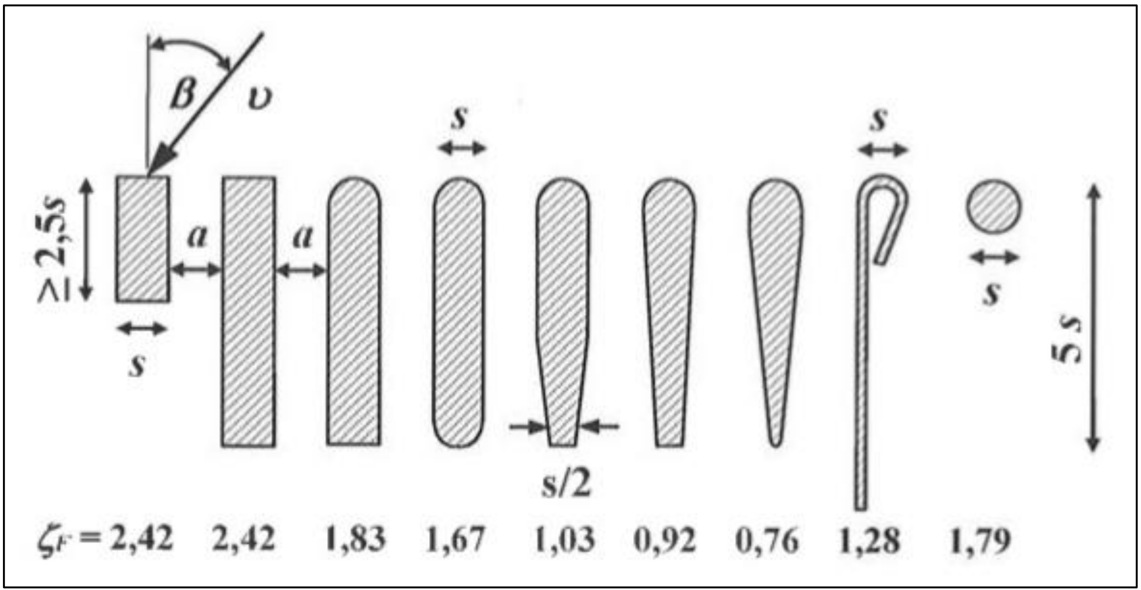


Figure 39: Shape coefficient after Kirschmer (1926) [27]

As the reservoir is not exposed to sediments coming from the outside, the rod spacing can be 10 cm and the rod thickness 3 cm. For a standard rod, the profil coefficient would be 1.67. In the end, the local loss coefficient ζ can be is 0.335.

5.2.3 Confusor loss

Each confusor of the system is designed with an angle α smaller than 15°, therefore, the formula shown on Figure 40 can be used to calculate the local loss coefficient.

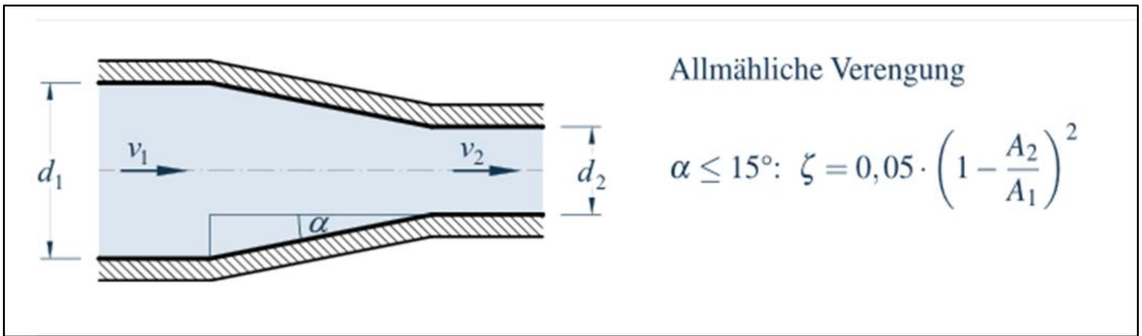
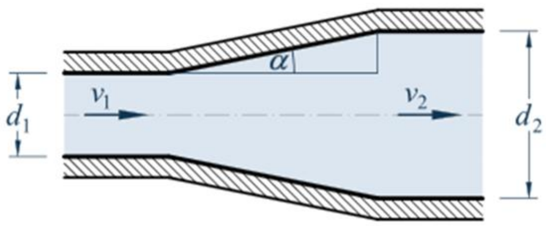


Figure 40: Local loss coefficient for a confusor [29]

The local loss coefficient for each confusor is round about 0.01. The first one is located in the upper intake, and the second one in the intermediate intake.

5.2.4 Diffusor loss

To calculate the local loss due to a diffusor, the formula shown on Figure 41 can be used.



Allmähliche Erweiterung

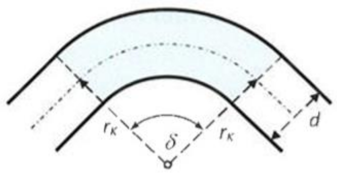
$$\zeta = 3{,}2 \cdot \left(\tan\frac{\alpha}{2}\right)^{1{,}25} \cdot \left(1 - \frac{A_1}{A_2}\right)^2$$

Figure 41: Local loss coefficient for a diffusor [29]

However, if the angle α is smaller than 8°, the local loss due to the diffusor is so small that it can be neglected [27]. Each diffusor has been designed with angles from 4-8°, therefore the local losses due to diffusors can be neglected in the calculations.

5.2.5 Bending loss

The local loss due to a bending in a pipe can be read from the table on Figure 42. It is influenced by the ration between the radius of the bending and diameter of the pipe, and by the bending angle α.



δ	30°	45°	60°	90°	120°	150°	180°
k_δ	0,45	0,63	0,78	1,0	1,17	1,29	1,4

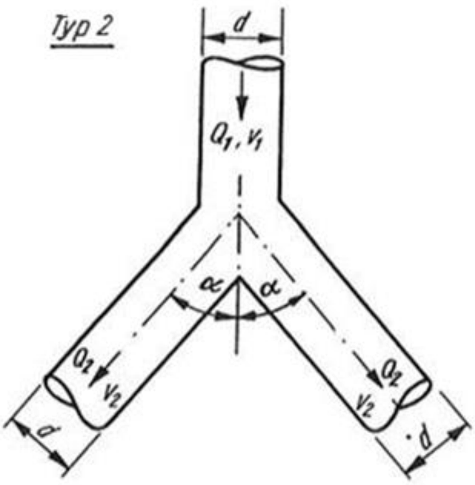
k/d	0,0004	0,0006	0,0008	≥ 0,0010
r_K/d	Verlustbeiwert ζ_{Kr} bei $\delta = 90°$			
1,0	0,29	0,34	0,38	0,42
1,5	0,24	0,27	0,31	0,34
3	0,17	0,19	0,22	0,24
4	0,15	0,17	0,19	0,21

Figure 42: Local loss for a bending in a pipe

As the roughness of the bending is influencing the direction change, the local loss coefficient is already taking friction into account. An equivalent sand roughness of 0.5 mm can be assumed for steel. The bendings in the system are designed with 90° and a radius of 30 m. Therefore, the ratio r/d is equal to 7.7. The ratio k/d is 0.000013, therefore the value is out of the table. With these values as input and an interpolation from the values in the table, a local loss coefficient of 0.11 can assumed.

5.2.6 Junction loss

The local loss due to a sudden symmetrical pipe junction can be read from the table on Figure 43. It is mostly influenced by the angle α which is the angle between the axis of the first pipe and each other pipe.

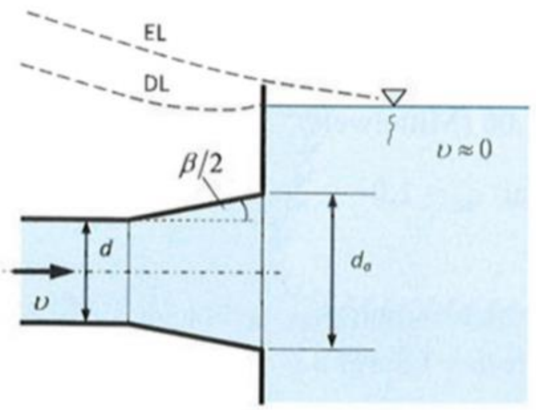


Typ 2		
α	ζ_{12}	ζ'_{12}
10°	0,1	0,4
30°	0,3	1,2
45°	0,7	2,8
60°	1,0	4,0
90°	1,4	5,6

Figure 43: Local loss due to sudden junction in a pipe

5.2.7 Discharge loss

The discharge loss is strongly dependent of the occurring flow velocity in the last section of the pipe directly before the reservoir. If the geometry of the pipe stays unchanged until the end, the local loss coefficient is 1.0 or 1.1 for a turbulent flow and 2.0 for a laminar flow. However, diffusors have been designed before the reservoir over a length of 40 m with an angle β between 5° and 8°. Therefore, the ζ-values can be taken from Figure 44.



$$h_a = \zeta_a \cdot \frac{\upsilon^2}{2g}$$

$\zeta_a = 0{,}15$ bis $0{,}3$ bei $\beta = 8°$

$\zeta_a = 0{,}22$ bis $0{,}33$ bei $\beta = 14°$

$\zeta_a \approx 0{,}97$ bis $1{,}06$ bei $\beta \geq 60°$

Figure 44: local loss coefficient for discharge in a reservoir

The value of the local loss coefficient can be assumed with 0.23 for both diffusors in the system.

5.2.8 Valve loss

Valves have to be installed on the upstream and downstream penstocks connected with the powerhouse. Rotary valves are needed on the upstream side where the water pressure is significant, and simple slide valves can be used for the downstream side.

A rotary valve which is fully opened has a local loss coefficient of 0.1. A slide valve which is fully opened, has a local loss coefficient of 0.12.

5.3 Calculation of the total head losses

The calculation of the hydraulic losses over the entire system was done with an excel table. The results can be seen in Table 7. The total head loss is approximately 14.06 m, which is only 1 % of the total hydraulic head for a power plant with a gross head of 1600 m,

5.4 Comparison with conventional Pumped Storage Hydro schemes

It is interesting to compare the result of chapter 5.3 with the hydraulic losses that would occur for a conventional PSH with the same installed output. conventional PHS with the same output have on average a power waterway which is 11.8 times longer than the one from an UPH [30]. As a matter of simplification, it can be assumed that the local losses stay unchanged because the different parts of the power plant remain the same. Only the power water way is assumed to be longer, as it is usually when natural reservoirs have to be connected together. In this part, the length of each vertical pressure shaft will simply be multiplied by 11.8, resulting in a total length of 19,343 m instead of 2,063 m. The total head loss of such a conventional PSH would 101.36 m, round about 7.2 times more than an UPH.

It is obvious that most of the hydraulic losses come from the friction between the water mass and the lining of the power water way. If one is able to reduce drastically the total length of the power water way, the project is much more viable. UPH seems to be a very good solution to achieve a given output with the maximal efficiency.

Table 7: Total head loss for the Underground Pumped Hydro Storage with two stages of 800 m

Section	local hydraulic losses	Friction losses	Material	Diameter	A [m²]	L [m]	R_{hy} [m/s]	ζ	k_{st} [$m^{1/3}/s$]	Q [m³/s]	V [m/s]	v²/g	h_v (λ)[m]
	trash rack 3x			-	48.04			0.3350		23.3	0.49	0.01	0.012
		intake struct.	1	-	41.76	40.00	1.42		85.0	23.3	0.56	0.02	0.003
	Intake loss			7.00	38.48			0.0700		70.0	1.82	0.17	0.012
Upper reservoir		"mushroom"	1	8.50	56.75	4.00	2.13		85.0	70.0	1.23	0.08	0.000
	confusor			3.88	11.82			0.0125		70.0	5.92	1.79	0.022
		confusor	1	5.44	23.24	20.00	1.36		85.0	70.0	3.01	0.46	0.017
	stop log niche			3.88	11.82			0.0550		70.0	5.92	1.79	0.098
		vertical part	1	3.88	11.82	800.00	0.97		85.0	70.0	5.92	1.79	4.042
pressure shaft 1	90° bending			3.88	11.82			0.1100		70.0	5.92	1.79	0.197
		90° bending	2	3.88	11.82	48.70	0.97		110.0	70.0	5.92	1.79	0.147
		horizontal part	2	3.88	11.82	35.00	0.97		110.0	70.0	5.92	1.79	0.106
	Junction			3.88	11.82			0.3000		70.0	5.92	1.79	0.536
		Upstream pennstock 2x	2	2.74	5.90	22.00	0.69		110.0	35.0	5.94	1.80	0.212
	rotary valve 2x			2.74	5.90			0.1000		35.0	5.94	1.80	0.359
Power House 1	slide valve 2x			2.74	5.90			0.1200		35.0	5.94	1.80	0.431
		Downstream pennstock 2x	2	2.74	5.90	22.00	0.69		110.0	35.0	5.94	1.80	0.212
	branching			3.88	11.82			0.3000		70.0	5.92	1.79	0.536
		connection	2	3.88	11.82	20.00	0.97		110.0	70.0	5.92	1.79	0.121
	diffusor			3.88	11.82			0.0000		70.0	5.92	1.79	0.000
		diffusor	1	6.94	37.83	40.00	1.74		85.0	70.0	1.85	0.17	0.018
	Shock loss			-	78.54			0.0200		70.0	0.89	0.04	0.001
Intermediate reservoir	Intake loss			7.00	38.48			0.0700		70.0	1.82	0.17	0.012
		"mushroom"	1	8.50	56.75	4.00	2.13		85.0	70.0	1.23	0.08	0.000
	confusor			3.88	11.82			0.0100		70.0	5.92	1.79	0.018
		confusor	1	5.44	23.24	20.00	1.36		85.0	70.0	3.01	0.46	0.017
		vertical part	1	3.88	11.82	800.00	0.97		85.0	70.0	5.92	1.79	4.042
pressure shaft 2	90° bending			3.88	11.82			0.1100		70.0	5.92	1.79	0.197
		90° bending	2	3.88	11.82	48.70	0.97		110.0	70.0	5.92	1.79	0.147
		horizontal part	2	3.88	11.82	35.00	0.97		110.0	70.0	5.92	1.79	0.106
	Junction			3.88	11.82			0.3000		70.0	5.92	1.79	0.536
		Upstream pennstock 2x	2	2.74	5.90	22.00	0.69		110.0	35.0	5.94	1.80	0.212
	rotary valve 2x			2.74	5.90			0.1000		35.0	5.94	1.80	0.359
Power House 2	slide valve 2x			2.74	5.90			0.1200		35.0	5.94	1.80	0.431
		Downstream pennstock 2x	2	2.74	5.90	22.00	0.69		110.0	35.0	5.94	1.80	0.212
	branching			3.88	11.82			0.3000		70.0	5.92	1.79	0.536
		connection	2	3.88	11.82	20.00	0.97		110.0	70.0	5.92	1.79	0.121
	diffusor			3.88	11.82			0.0000		70.0	5.92	1.79	0.000
Lower reservoir		diffusor	1	6.94	37.83	40.00	1.74		85.0	70.0	1.85	0.17	0.018
	discharge loss			-	140.00			1.0000		70.0	0.50	0.01	0.013
Total						**2063.40**							**14.057**

6. Variant study

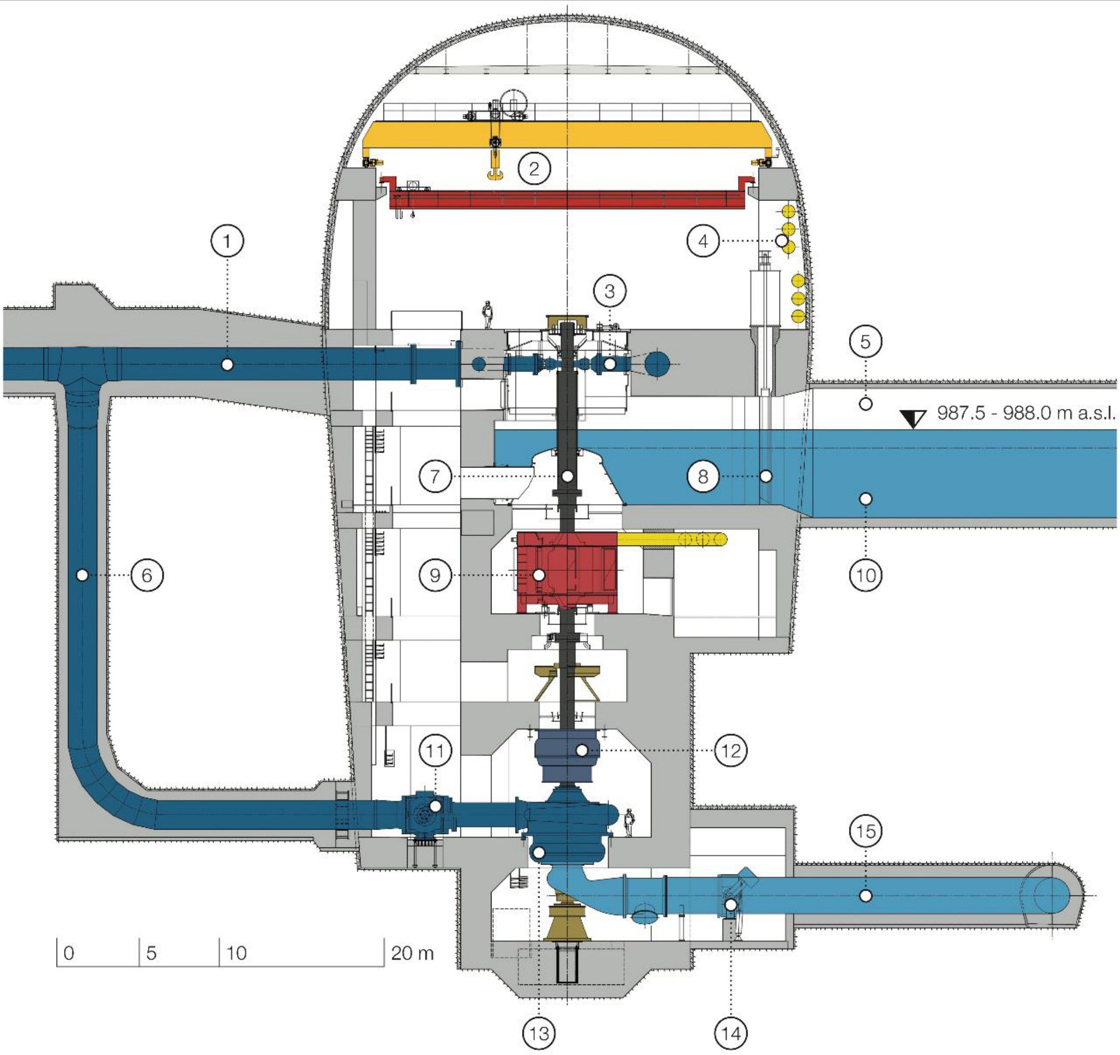
The aim of this part is to make a variant of the model used for the feasibility study by changing some input parameters. The water volume and the layout of the reservoirs stay unchanged. However, the hydraulic head between each stage of the power plant is increased to 1200 m. Therefore, the pump and turbine type has to change in order to be able to fit the new layout. A reversible pump turbine is not able to capture a pumping head of 1200 m. Therefore, a ternary machinery unit will be designed with separate pump and turbine. The design of the power cavern was inspired from an existing PSH in Austria, Kopswerk II, which can be seen on Figure 45. Nevertheless, the geometry of the proposed variant is much simpler, as there is no spatial constraint in the case of an UPH. The size of the penstocks will remain unchanged, and the number of machinery unit will be increased from three to four.

6.1 Hydraulic boundary conditions

First, the water flow will be increased to 100 m^3/s in order to bring about 25 m^3/s to each turbine for energy production. As the reservoir volume remains unchanged, this increased discharge directly involves a reduction of the possible production time by full load. With 4,000,000 m^3 of storage, it would be possible to produce during 11 hours. The corresponding output per stage is 1070 MW, so 2140 MW in total. A total of 8 turbines (4 per stage) with a capacity of 267.5 MW are required.

The dimensions of the power cavern will be almost the same than the one from Kopswerk II. A cross section of this power cavern can be seen on Figure 45. As the cavern has to host one machinery unit more, the length has to be increased to 120 m. The height remains by 61 m and the width by 30 m. The spacing between the penstocks is 22.7 m. The tailrace canal has a width of 7 m and a height of 8 m.

In order to keep the mean velocity in the vertical pressure shaft to 6 m/s, the diameter has to be increased to 4.6 m. The layout of the air pressure compensation system and the heavy hoist shaft have to be changed, but all in all the design idea remains unchanged.



1. Turbine manifold, diameter 1.90 m
2. Two cranes, load capacity each 130 tons
3. Pelton turbine and distribution pipe
4. SF6-generator main leads
5. Compressed-air chamber
6. Upstream pump riser, diameter 1.70 m
7. Turbine shaft
8. Two tailrace turbine gates
9. Motor-generator
10. Tailrace canal, Width 7 m x height 8 m
11. Spherical valve
12. Hydraulic torque converter and coupling
13. Storage pump
14. Valve
15. Pump suction pipe, diameter 2.20 m

Figure 45: Cross section of the power cavern Kopswerk II [31]

6.2 Power Cavern

Figure 46 and Figure 47 show the final 3D geometry of the power cavern for the preliminary design of the variant study in this work.

As it can be seen, the direction changes and junctions are very rounded, reducing as much as possible the hydraulic losses. The pump suction pump run horizontally after the power cavern and then vertically up to be incorporated to the tailrace canal. A tunnel with closing valves has been designed upstream of the junction between the turbine manifold and the upstream pump riser. This enables to regulate the incoming flow.

As mentioned in chapter 4.7, the hydraulic gradient has to be considered. In this case, the static water pressure at the bottom of the vertical pressure shaft is 1200 m and the distance to the power cavern is 145 m. The resulting gradient is 8.3, which is comprise between 8 and 10. No water seepage problems should occur if the rock mass conditions are good enough.

6.3 Hydraulic losses

The Hydraulic losses along the system have been calculate in order to compare them with the one from the variant from the feasibility study. Again, local and friction losses have been taken into account. For the sake of simplicity, only the results will be shown in this part. A detailed explanation of the different losses which have to be considered was done already in chapter 5. The total head losses for this system is around 37.1 m. Compared to the first system, the losses are almost three times greater. However, a head loss of 37.1 m in a power plant with 2400 m of hydraulic head is very small, only 1.5 % of the total head.

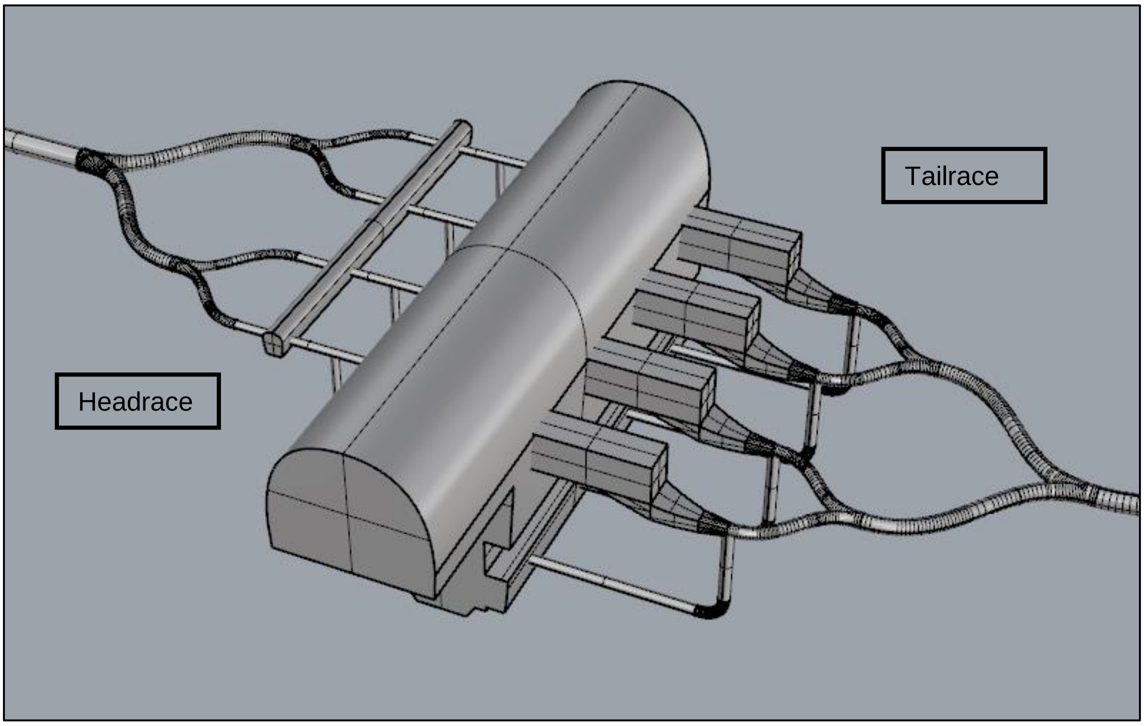


Figure 46: Top view of the power cavern with separate pump and turbine

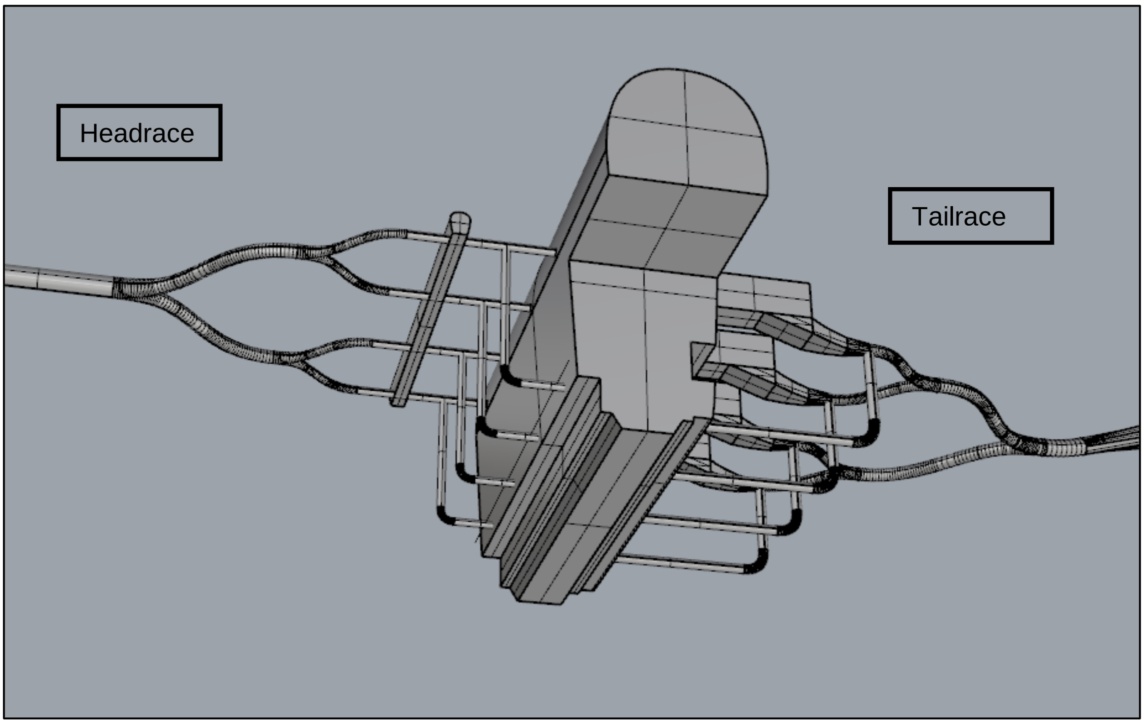


Figure 47: Bottom view of the power cavern with separate pump and turbine

7. Constructional feasibility study

One important part of a feasibility study is to assess the constructional measures that are needed for each part of the power plant. These measures have to be suited to the in situ underground conditions and be as efficient as possible in order to speed up the construction process. In this chapter, technical solutions will be suggested for each part of the UPH.

7.1 Access Tunnel and reservoir system

The first part of the power plant which has to be excavated is the access tunnel. As it is a rather short tunnel, it can be excavated with a traditional drill and blast method combined with the New Austrian Tunnelling Method (NATM). Basically, this approach aims the utilisation of few support measures by allowing displacements of the rock mass towards the excavated space until it stabilises by itself. The behaviour of the rock mass has to be monitored in order to adapt the lining if needed. At the entrance, a pipe umbrella will be needed to provide additional stability to the rock mass that might be weathered. Figure 48 and Figure 49 show respectively the installation of a pipe umbrella, and the excavation, installation of the shotcrete lining and the monitoring process according to the NATM .

*Figure 48: Construction of a pipe Umbrella **[32]***

*Figure 49: Excavation, shotcreting and monitoring according to the NATM **[33]***

The pipe umbrella is necessary as the daylighting rock mass from the quarry looks slightly weathered at the surface. It has the advantage to prevent overbreak and reduce ground displacement ahead of the face [34]. After the entrance part is excavated and supported, the drill and blast process can start. After each blasting sequence, a ductile lining like shotcrete (reinforced by mesh or fibres) has to be installed to prevent overbreak and at the same time allow displacements of the rock mass. This is of prior importance in order to use the inherent strength in the surrounding rock mass to stabilize the tunnel.

Additionally, active support measures like rock bolts or grouted anchors can be installed to increase locally the stability of the tunnel lining [34]. For example, if a weak zone has to be crossed, these measures are unavoidable.

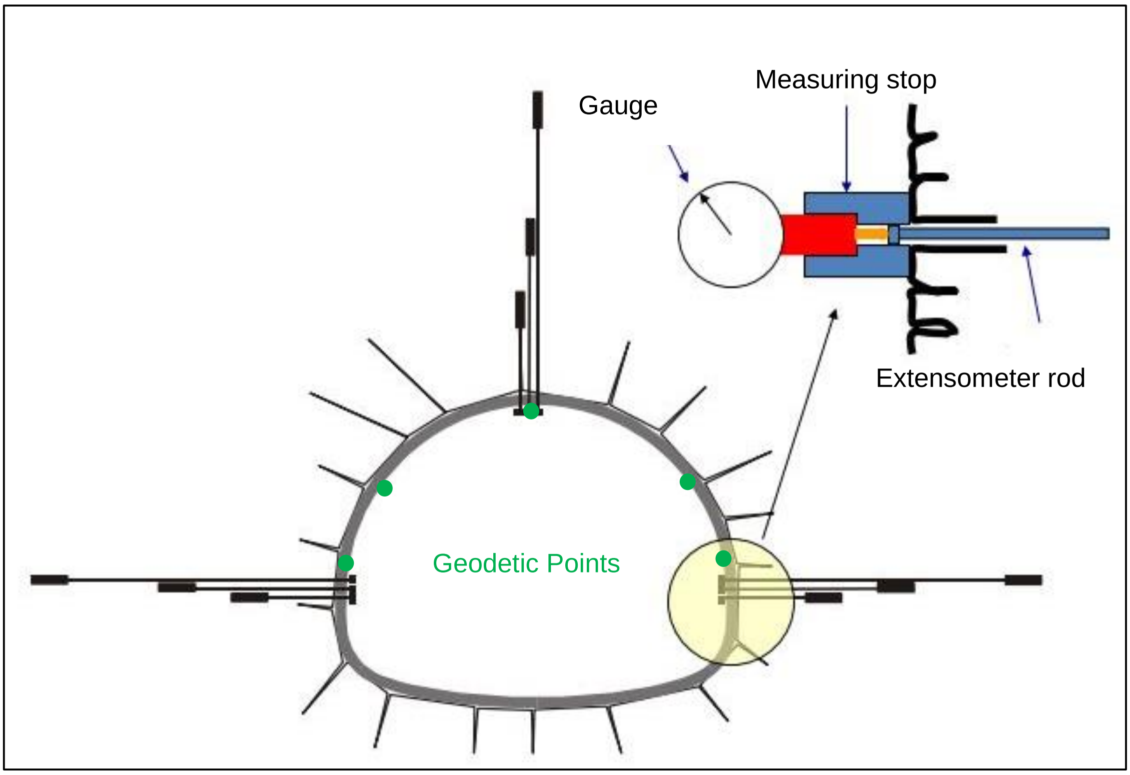


Figure 50: Monitoring installations in an tunnel with NATM method [35]

A very important aspect of such an excavation sequence is monitoring. First, absolute displacements have to be monitored with geodetic points along the tunnel lining. These points have to be controlled daily in order to assess the displacement of the rock mass. Then, relative displacements have to be monitored by mean of extensometers and inclinometers. They have to be drilled with several length and several positions to capture accurately the behaviour of the rock mass. Figure 50 shows a typical monitoring layout of a tunnel cross-section.

Regarding the reservoir system, the same excavation method will be used. The grid layout of the reservoirs would not enable a mechanical excavation method like a TBM, as it is not a straight line but several secant tunnels. A conventional drill and blast method is as well much cheaper.

However, it is clear that any major requirement for roof support in the reservoirs, either in form of ribs or rock bolts, would be unacceptable, both from the standpoint of overall cost and in the caused delay on the construction schedule. "Similarly, any other form of large-scale rock treatment such as grouting would equally be unacceptable. Local treatment of minor problem areas could be accommodated [7].

7.2 Shaft excavation and support

7.2.1 Heavy hoist shaft

The heavy hoist shaft is the largest and deepest shaft of the entire power plant. It is around 10-12 m diameter and has to be 1600 m deep. For such a structure, a conventional drill and blast method would probably not be efficient enough. Typical advance rates using this method are 2-3 m per day. Therefore, a mechanical excavation method should be privileged.

The company Herrenknecht AG is nowadays the leader on the market regarding machines for mechanical excavation of tunnels and shafts. The SBM (Shaft Boring Machine) technology, on the market since 2010, is well suited for such kind of excavation. A representation of this machine can be seen on Figure 51.

The SBM uses conventional disc cutting technology, however in a new configuration. It is capable of excavating hard rock formations with a continuous dry mucking system similar to to a rock tunnel boring machine up to a depth of 2000 m and for shaft diameters in the range of 10-12 m.

The semi-full-face sequential excavation process is based on the use of a rotating cutting wheel excavating the full shaft diameter in two stages for one complete stroke. The excavation process is divided in two steps:

- Excavation of the trench to the depth of one stroke with the cutting wheel rotating around its horizontal axis and being pushed downward in the shaft direction.

- Excavation of the entire face are by slewing the rotating cutting wheel 180° around the shaft vertical axis of the machine.

The machine has three major areas of equipment and operation which are stated below, starting from the bottom:

- Cutting wheel, muck transport and gripping system, as well as equipment for the primary rock support and investigation drilling.

- Primary platform for the supply infrastructure and the powering of the machine.

Secondary platform for final lining installation, muck transport and services extension

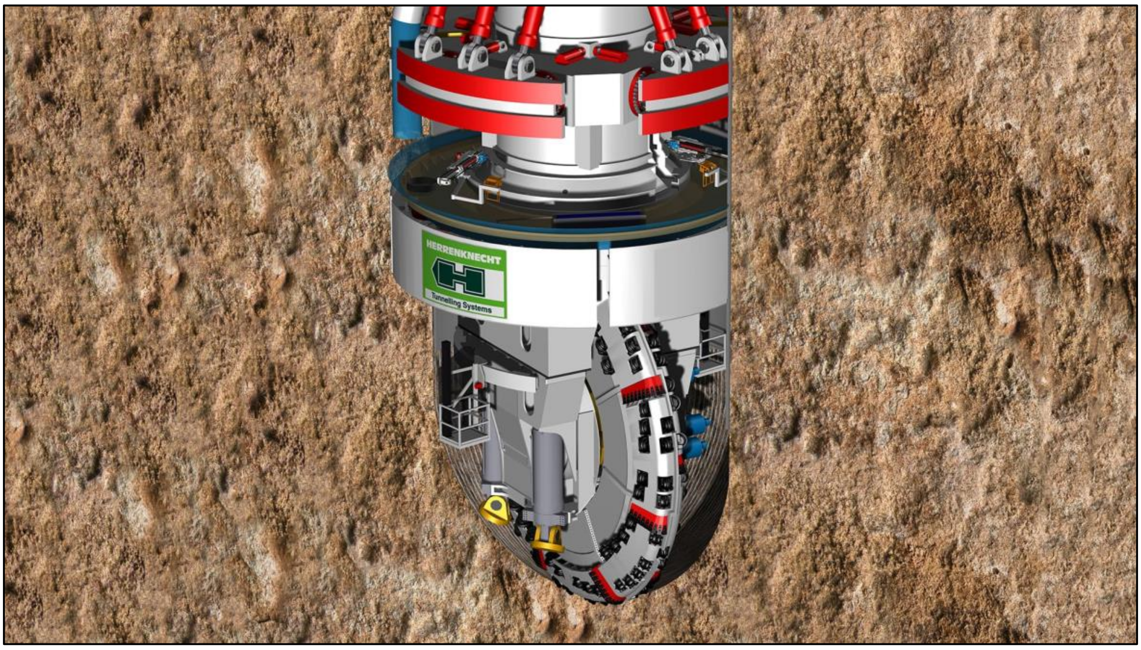


Figure 51: Shaft Boring Machine [36]

In order to excavate the hard rock and remove the cuttings simultaneously, the cutting wheel circumference and each side are equipped with strong cutting tools and muck buckets. Thereby, the excavation process stays uninterrupted. The reaction forces from the excavation process are transferred into the shaft walls by the grippers. Finally, an alignment control and readjustment of the vertical axis of the machine of the machine can be done after each excavation cycle ensuring a high drilling accuracy.

7.2.2 Pressure shaft and air gallery

The diameters and lengths of the pressure shafts and air shafts are smaller than the one from the heavy hoist shaft. Their diameters are around 4 m and their lengths around 750 m. Therefore, a big machine like a SBM would not be adequate for the excavation. Further, these shafts have to be excavated at several different locations, unlike the heavy hoist shaft which is excavated in one stroke of 1500 m. Therefore, a raise boring technology is more viable for the excavation. The company Herrenknecht was once again solicited to provide information about the latest available technology, the RBR (Raise Boring Rig). A representation of this machine can be seen on Figure 52.

The RBR is designed for the excavation of shafts in hard rock down to 2000 m depth. Its design is very compact, therefore only a small cavern is required at the starting point of each shaft. Connection shafts for mining or penstocks for pump storage power plants can be constructed quickly and safely.

First, the drilling rig is installed above the starting point by mean of a transport system. The so-called “pilot drill” is sunk vertically downward with the drill bit. Depending on the drilling depth, further drilling rods are installed progressively, until the target depth is reached. At the end point, an already existing cavern is reached.

Subsequently, the pilot drill bit is removed in the cavern and a reamer with the desired diameter is fixed to the drill rod. After the assembly work has been done, the drill rod and the reamer are pulled upwards. At the same time, the road is rotating. This rotation enables the cutter head to excavate the rock around the drill hole. The muck falls down and can be easily removed from the cavern by mean of machines. As the heavy hoist shaft is excavated first, it can be used to bring the muck to the surface. In this way, the entire shaft is extended upwards from the bottom with the diameter of the reamer. The lining of the shaft, if needed, has to be installed in a further step. Figure 53 shows a reamer installed in the cavern which is ready to start the clearing works.

Figure 52: Raise Boring Rig [37]

Subsequently, the pilot drill bit is removed in the cavern and a reamer with the desired diameter is fixed to the drill rod. After the assembly work has been done, the drill rod and the reamer are pulled upwards. At the same time, the road is rotating. This rotation enables the cutter head to excavate the rock around the drill hole. The muck falls down and can be easily removed from the cavern by mean of machines. As the heavy hoist shaft is excavated first, it can be used to bring the muck to the surface. In this way, the entire shaft is extended upwards from the bottom with the diameter of the reamer. The lining of the shaft, if needed, has to be installed in a further step. Figure 53 is showing a reamer installed in the cavern which is ready to start the clearing works.

Figure 53: Reamer of a Raise Boring Machine [38]

After some exchanges with Herrenknecht AG [39], some important information specific to this feasibility study have been gathered. These information are listed below:

- Machine type: RBR400.
- Required cavern dimensions to host the machine: 9 m length, 7 m width and 8 m height.
- Drilling accuracy: 0.1 % usually less, which means maximal 0.8 m deviation for an 800 m deep shaft. As example, a ventilation shaft at the

Gotthard Base tunnel was excavated with raise boring. The target precision for this 800 m deep shaft was 0.125 % deviation, and precision actually achieved was 0.03 % [40].

- Advance rate: around 30 m/day for the pilot drilling and 5-10 m/day for the clearance.
- Price: 2.6 M€ for the machine, 4.1 M€ for the drilling rod and 0.6 M€ for the reamer. In total 7.3 M€. depending on the number of shafts, it can be cheaper to employ a sub-contractor for these works.

Usually, a thin shotcrete lining is installed as a primary support after the shaft has been excavated. If necessary, anchors or rock bolts can as well be installed to increase locally the stability of the shaft. This process is done from the top to the bottom. Further, a sliding formwork installed on a hanging stage can be driven from the bottom to the top to install the final concrete lining. The new rings first transferred their dead load to the previous ring before being able to transfere it by friction and mechanically to the inner walls of the shaft. This procedure in three steps is standard in hydro power plants [39].

Another interesting example of shaft lining can be found in the project of the Gotthard Base Tunnel. During the project, several ventilation shaft of roundabout 800 m had to be sunk. They also have been used as access and supply shaft during the construction of the 57 km long main tunnel [40]. The sinking method used was a traditional drill and blast method. For the inner lining, a system with hanging formwork was used to concrete the last ring. The inner ring consists of 6 m high rings constructed in three stages. Starting from down to top, the first ring was 1 m, the second 2 m, and the third 2.7 m. A gap was left in order to assure the drainage of mountain water. The formwork for the first and second stages was hung from the concreted shaft ring above with chains. After a timespan of about 8 h, the concrete reached the required early strength of 5 MPa. The hanger chains were then released and the load could be transferred by friction and mechanically to the surrounding rock formation. This example shows, that it would as well be possible to construct the final lining from top to down

7.2.3 Alternative shaft excavation method

An alternative excavation method which would be applicable for the heavy hoist shaft as well as for the smaller shafts is the Vertical Shaft Sinking Machine (VSM). This system basically enables the installation of the final lining simultaneously to the excavation works. Figure 54 shows a VSM.

This machine is suited for excavations in soft soils to hard rock up to a 80 MPa and below groundwater for shaft diameters of 4.5-18 m. The advance rate of up to 5 m per shift can be achieved [41].

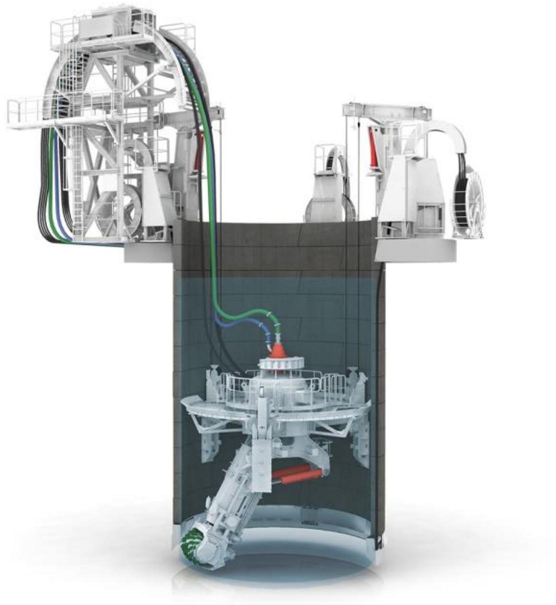


Figure 54: Vertical Shaft Boring Machine

It is composed of two main components; the shaft boring machine and the lowering units. First, a launch shaft structure is built to fix the three arms of the boring machine. A rotating cutting drum is attached to a telescopic arm and fixed to the structure. This roadheader excavates the soil at the base of the shaft. The excavation material is removed hydraulically with a pump. When the entire cross section has been excavated with an overcut, an additional concrete ring is built at the top of the shaft and the entire structure slides downward. This sliding is possible because of the bentonite which is lubricating the annular gap, reducing the friction between the shaft and the surrounding soil. Lowering units at the surface are linked with the lowest ring with steel cables, allowing a controlled lowering of the structure. Monitoring system has to be used in order to control the position of the shaft during the entire process.

7.3 Caverns

Several caverns with different sizes are needed for an UPH. At the end of the access tunnel, a cavern is needed in order to host the installations for the SBM, as described in chapter 7.2.1. Then, at the starting and end point of each raise boring, a smaller cavern is needed respectively to host the machine and to enable the muck removal.

Regarding the excavation methods, a traditional drill and blast method has to be used from top to down. The top to down excavation of caverns shows a higher economic viability (Figure 55). From the left to the right, the letters A, B and C are showing respectively the excavation cost (including mucking and dumping), the rock support cost and the pumping cost. It is obvious that slender caverns are more economical than wide ones, as the excavation price in $/m^3$ is decreasing with depth.

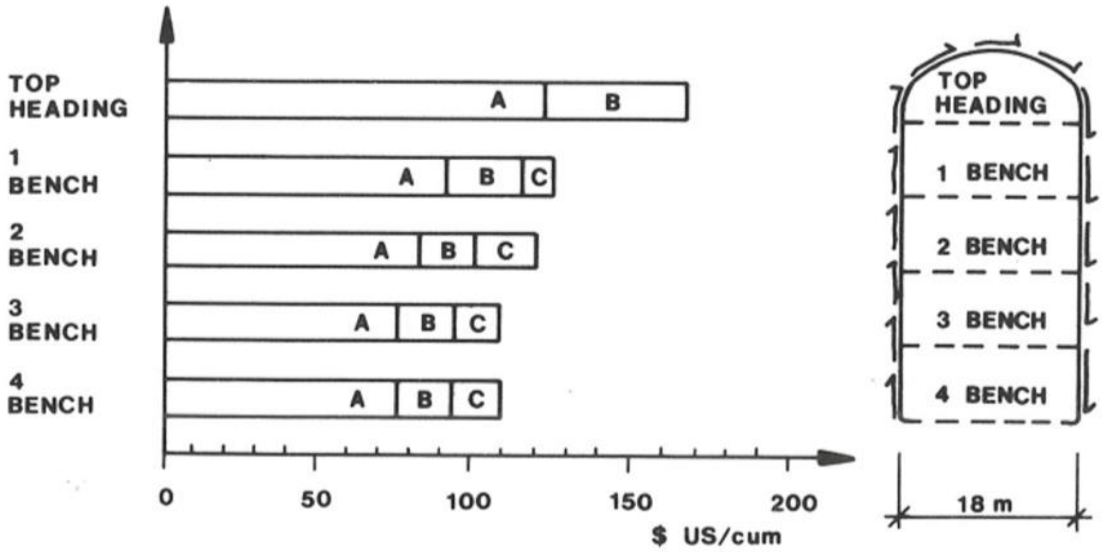


Figure 55: Cavern depth/cost relationship [23]

Support measures like rock bolts and shotcrete has to be installed immediately after excavation, especially in the roof area. Displacements monitoring is unavoidable in order to control the movement of the rock mass after the in situ stresses have been redistributed.

8. *Conclusion and outlook*

Based on the literature review, it was possible to show how important it is, to construct underground pumped hydro storages combined with thermal energy storage in a near future. It could enable a very efficient exploitation and management of intermittent renewable energy sources, like wind and solar energy, and provide more stability and reliability for the grid. The concepts of hydraulically based electrical energy generation and energy storage as well as thermal energy storage are already well defined since several decades, and the technical know-how required to build such a facility is available nowadays. Furthermore, the city of Graz shows favourable features to build an underground pumped hydro. Favourable geology, geography and geotechnical conditions can be found on the west side of the city, directly near the electrical energy load point and transportation facilities. The chosen site would enable minimal environmental impacts on the city during construction and operation of the power plant.

The conceptual feasibility study showed that the project´s spatial requirements are fulfilled. There is enough space underneath the Plabutsch mountainto build a cavern system with a total storage capacity of 4,000,000 m^3 without creating spatial conflicts with existing infrastructures. Further, this work proposes a detailed design of each part of the power plant, as well as an arrangement of the entire system. High emphasis was placed on a smart hydraulic design of the different parts of the system (intakes, sediment traps, pressure shafts, reservoirs, etc.). The final design is a two-stage fully underground thermal pumped hydro scheme with a total gross head of 1600 m, a capacity of 1000 MW, and a full-load production time of 16 h. It is composed of two large underground reservoirs with an active hydraulic storage capacity of 4,000,000 m^3 each, an intermediate balancing reservoir, two underground power caverns, two pump-turbines per stage, an air pressure compensation system, and accesses to the various levels.

The variant study shows that a high flexibility is possible regarding the total gross head, the energy output, and the geometry of the various parts of the system. Spatial constraint is in the case of underground pumped hydro not a problem at all. The variant study is a two-stage fully underground power plant with a total gross head of 2400 m, a capacity of 2140 MW, and a full-load production time of 11 h. Each part of the system remains unchanged with regard to the pilot study, except for the machine units which have to be upgraded to four ternary machinery unit per stage.

Based on hydraulic and geomechanical considerations, this study proved that underground pumped hydro is much more economically viable over time than conventional pumped storage plants or any other energy storage. Short connections between the reservoirs combined with a smart overall design enable the significant reduction of the hydraulic losses compared to conventional pumped storage plants, making the project more viable.

Furthermore, some recommendations regarding state of the art excavation technics for tunnels, caverns and deep shafts were done. The utilisation of the New Austrian Tunnelling Method for the excavation of tunnels and caverns combined with high-tech excavation machines for the sinking and lining of different shafts enables a safe, rapid, and cost-effective construction process.

In future works, several aspects should be studied more in detail. First, geological field investigations as well as hydrogeological models of the proposed site have to be done. In addition, a more detailed 3D Model of the Plabutsch mountain, including any existing underground structures (pipes, cables, etc.), could be done. Then, CFD simulations and hydraulic model tests of a full underground storage cavern system could be useful to optimise the proposed geometry of the several parts of the power plant. Finally, a detailed cost and time estimation would be helpful to get a more precise idea of the project scale.

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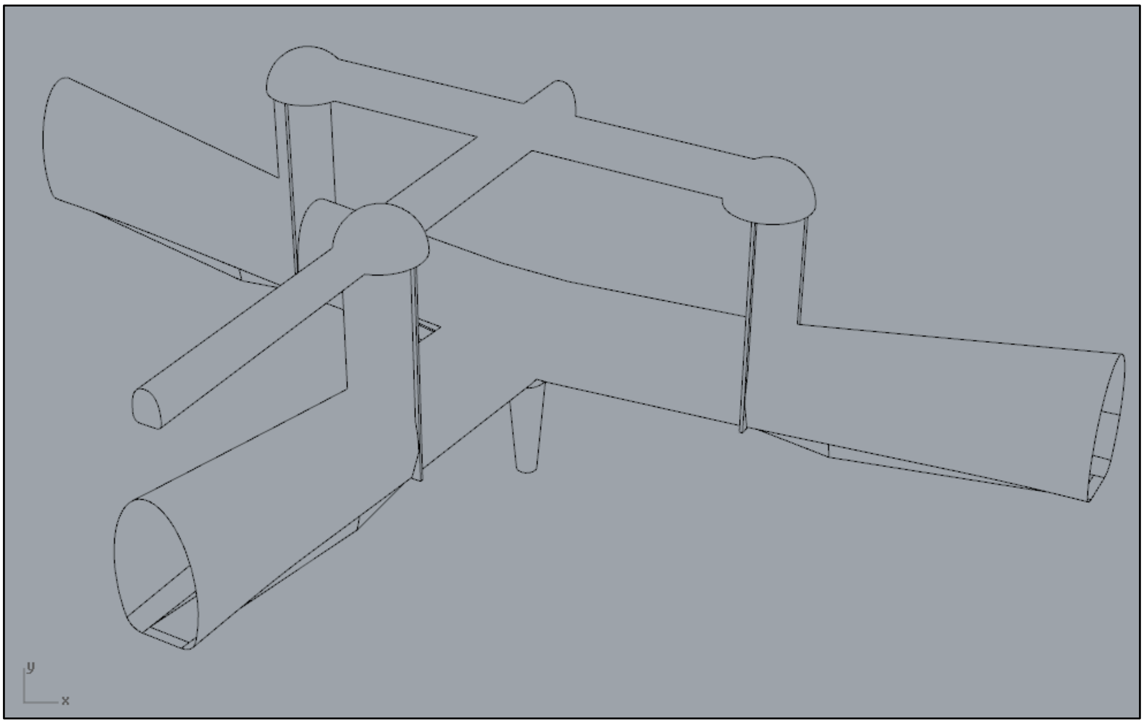
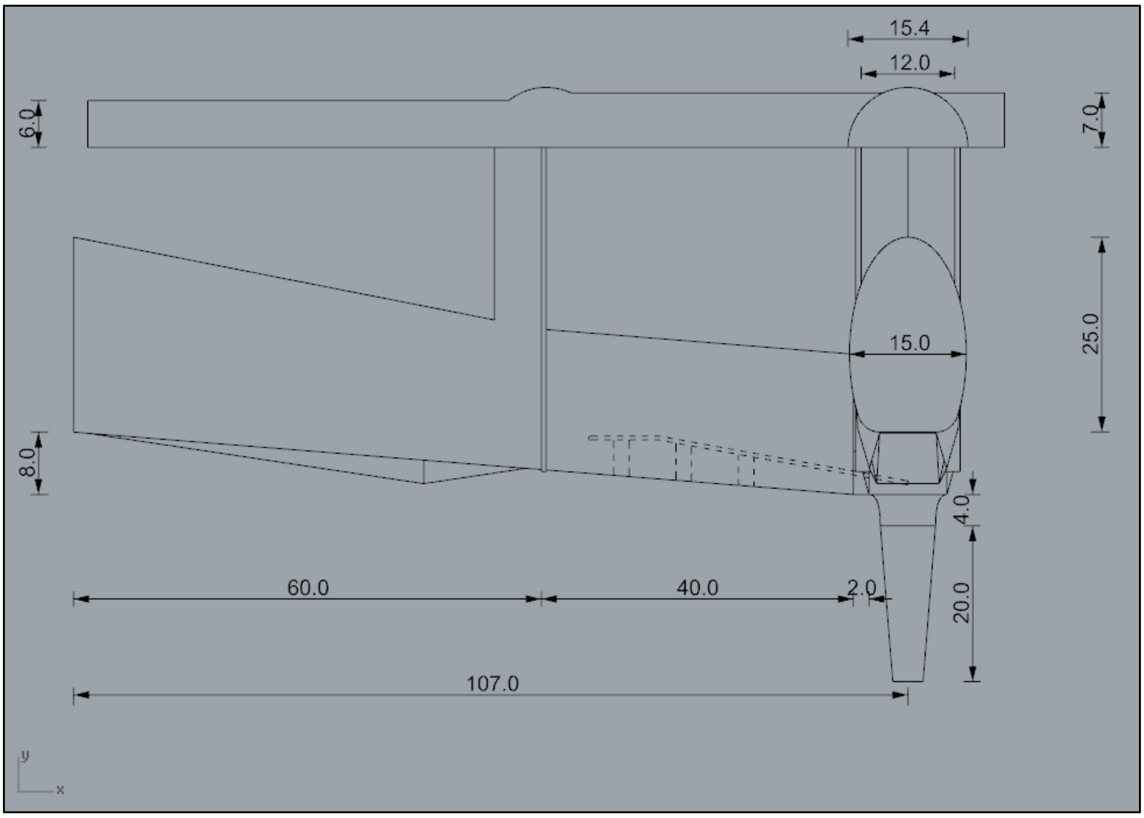
Annexe 1: Detail drawings of the upper intake structure

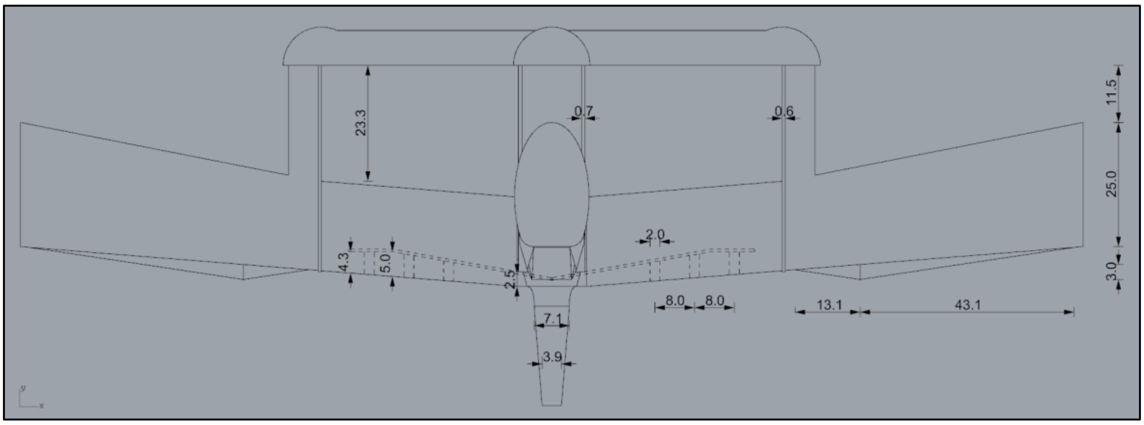
Isometric view:



Side view:



Front view:



Top view: