

# The design of unlined hydropower tunnels and shafts: 100 years of Norwegian experience

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The main criteria for the successful design of the unlined pressure tunnels described here are based on 100 years of experience and development of more than 80 unlined pressure waterways in Norway, with maximum heads up to 1000 m. International experience of collapses in special, challenging ground conditions from other countries are also included. Recommendations that have emerged from this review are to locate tunnels in suitable rocks with sufficient confinement to avoid hydro-jacking, which could lead to water leakage failures, and, to detect and provide sufficient support to local sections with unstable rockmass conditions, including swelling and/or friable materials.

The term unlined tunnel is used to describe hydropower tunnels where water is in direct contact with the rock, that means, only limited parts of the tunnel are lined with concrete or shotcrete to protect against local tunnel collapses or major rock falls.

The benefit of using unlined tunnels and/or shafts is a significant reduction in construction costs and construction time. Unlined tunnels and shafts with heads of more than 1000 m function satisfactorily. This has been made possible by 100 years of development in design, construction and operation, and Norwegian experience has made an important contribution. Today's design, construction and operation of unlined pressure tunnels and shafts are discussed in this paper.

## 1 Norwegian experience in the application of unlined waterways

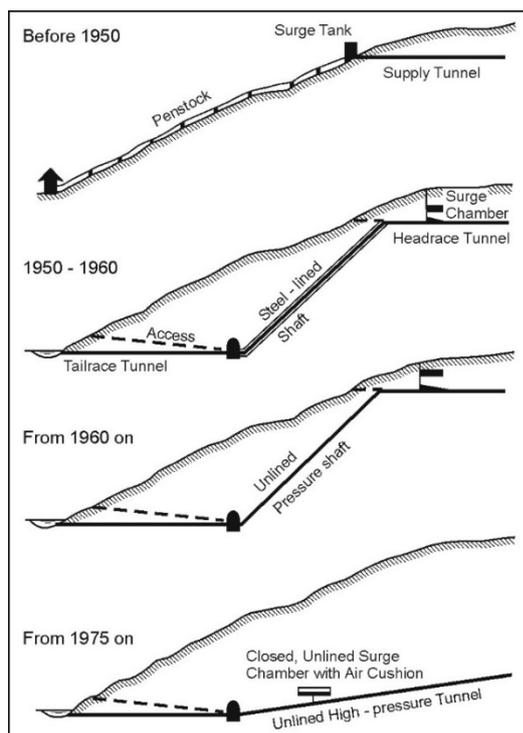


Fig. 1. Trends in the developments of the general layout of hydropower plants in Norway [Broch, 1982<sup>1</sup>].

### 1.1 Development of hydro plant layouts

Before 1950, a surface powerhouse with a penstock was the conventional arrangement for hydropower plants, as shown in Fig. 1. Early powerplants used either a steel penstock from the reservoir, a river intake or a short, unlined headrace tunnel to the power station. Depending on the topography, the penstock was installed at the surface or in a tunnel.

During and shortly after the First World War (1914-18) there was a shortage of steel, which led to uncertain delivery and very high prices. As a result of this, four Norwegian hydropower stations with unlined pressure shafts were put into operation during the years 1919 to 1921. The maximum heads varied from 72 to 152 m. Although three unlined pressure shafts, constructed around 1920, were operating without problems after some initial problems had been solved, it took almost 40 years for the record of 152 m of water head in unlined rock to be beaten. By 1958, nine more unlined pressure shafts had been constructed, but all with water heads of less than 100 m.

As Fig. 2 shows, new unlined pressure shafts were constructed in the early 1960s, and since 1965 unlined pressure shafts and tunnels have been the conventional solution. Today, more than 80 unlined high-pressure shafts or tunnels with heads of more than 150 m are operating successfully in Norway. The highest 'unlined' water head is more than 1000 m.

### 1.2 Development of design

The first useful experience of unlined pressure tunnels was the construction of the Herlandsfoss hydro plant, see Fig. 3, where a 150 m-long unlined pressure tunnel was built with a water head of 123 m. During the first filling of the shaft and the tunnel, increasing leakage through a mica-schist layer was observed<sup>A</sup>. The tunnel was then dewatered and the penstock was extended through the tunnel to the foot of the shaft, as the Figure shows. No leakage

<sup>A</sup> This can easily be explained, as the shortest distance from the cone to the surface is only 17 m, which compared with the required 55 m, is far too short (see Fig. 3).

from the shaft was observed subsequently, and the powerplant has now been in operation for almost 100 years [Selmer-Olsen, 1970<sup>3</sup>; Broch, 1982<sup>4</sup> and 1984<sup>5</sup>].

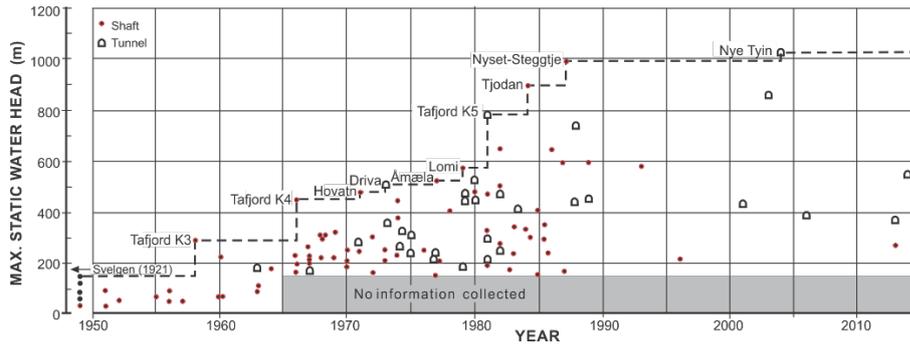


Fig. 2. The development of unlined pressure shafts and tunnels in Norway [updated from Broch, 1984<sup>5</sup>, 2010<sup>2</sup>].

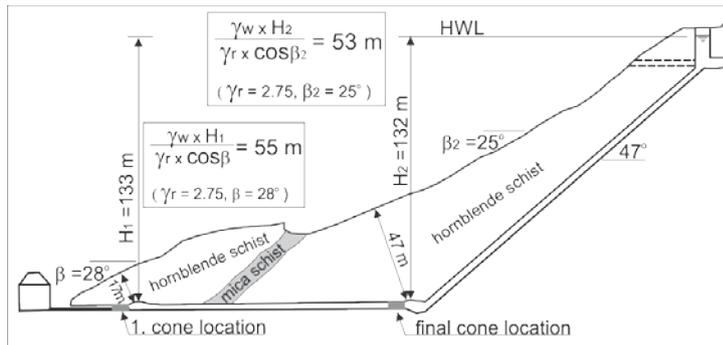


Fig. 3. Herlandsfoss hydropower plant, in operation since 1919. [revised from Vogt, 1922<sup>9</sup>].

### 1.2.1 Design by 'rule-of-thumb'

No calculations were used to locate the unlined tunnels before 1970. Then a simple 'rule-of-thumb' was introduced in the planning of unlined pressure shafts. It was associated with the general layout for hydropower plants used at that time (Fig. 1). The rule-of-thumb was based on the condition that the tunnel should be located deep enough, so that the internal water pressure was balanced by the weight of the overlying rock. This simplified approach was generally adequate where the overburden surfaces were relatively horizontal, and the rock stress situation normal.

In 1968, the unlined pressure shaft with a 300 m water head at the Byrte hydropower plant failed and flooded the underground powerhouse, [Brekke *et al.*, 1970<sup>6</sup>]. Two years later another less serious failure occurred at the Askora hydropower plant, where an unlined tunnel in quartzitic rocks with a head of approximately 200 m was hydraulically split, see Fig. 4. The tunnel split with leakage at Askora followed sand-filled, steeply dipping joints with a strike parallel to the very steep valley side (55°) and normal to the tunnel. The failure is described in detail by Bergh-Christensen [1975<sup>7</sup>]. The remedial measure was to move the penstock cone 320 m upstream. The plant has been operating successfully since then. Panthi and Basnet [2016<sup>8</sup>] present more details about failures at Norwegian hydropower projects.

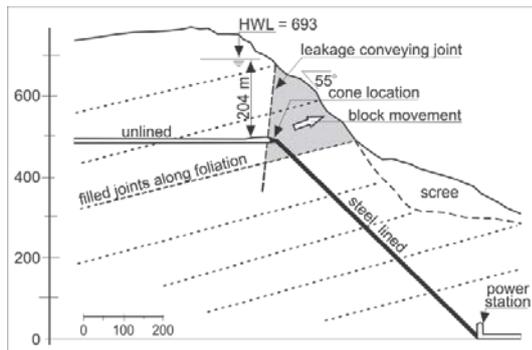


Fig. 4. The conditions at Askora hydropower plant. [revised from Berg-Christensen, 1975<sup>7</sup>].

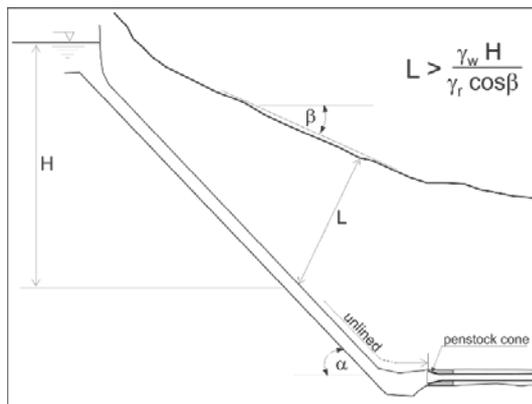


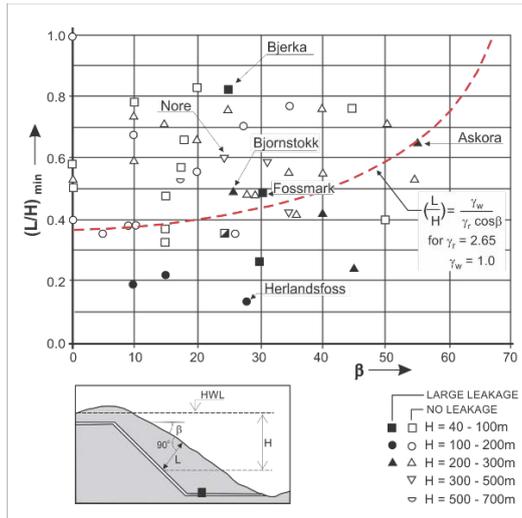
Fig. 5. Definitions for the new rule of thumb for unlined, high-pressure tunnels or shafts [Broch, 1982<sup>4</sup>].

### 1.2.2 Updated rule-of-thumb

After these failures, a revision of the rule-of-thumb was introduced by Bergh-Christensen and Dannevig [1971<sup>12</sup>], whereby the inclination of the valley side was taken directly into account, as well as the density of the water and the rock, see Fig. 5.

Based on this formula, a diagram showing existing unlined pressure shafts with or without leakage was presented. This was supplemented with unpublished information from the Norwegian Geotechnical Institute (NGI, 1972<sup>10</sup>) and is shown in a slightly revised version in Fig. 6, with later unlined tunnels and shafts.

Fig. 6. Unlined pressure shafts in valley sides with various valley slopes [revised from NGI, 1972<sup>10</sup> and Broch, 1982<sup>4</sup>]



### 1.2.3 Evaluation of the topography in the design

Careful evaluation of the topography in the vicinity of the pressure tunnel or shaft is necessary to assess the distance to the terrain surface. This is particularly important in areas where streams and creeks have eroded deep and irregular gullies and ravines in the valley sides. The remaining ridges, or so-called noses, between such deep ravines will, to a large extent be stress-relieved. They should therefore not be taken into account when the necessary overburden for unlined pressure shafts or tunnels is measured. This does not mean that pressure tunnels should not be running under ridges or noses, but only that the extra overburden this may give, should not be accounted for in the design unless the stress field is verified through in-situ measurements. An example is described in some detail by Broch [1984<sup>13</sup>] in relation to a project in Colombia. Fig. 7 shows how the ridge/nose is cut away by changing the contour lines in the map and subsequently revising the vertical profile along the tunnels and shaft. See also Fig. 12.

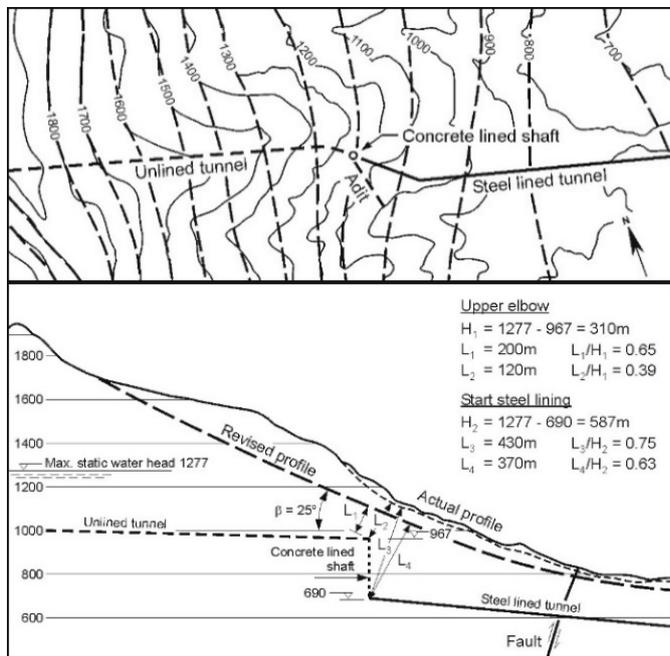


Fig. 7. The Chivor hydropower plant in Colombia, which has been in operation since 1982 [Broch, 1984<sup>8</sup>].

### 1.2.4 Unexpected low rock stresses

It is worth noting that the majority of unlined pressure shafts, at that time where leakage has occurred, are plotting below the curves defined by the new rule of thumb in Fig. 6. The exceptions are the Bjerka (1971) and Fossmark (1986) powerplants. At Bjerka, the penstock cone was located in a mica schist containing minor beds of karstic marble. The large leakage after watering up occurred along these karstic channels and was not caused by insufficient confinement. The cone (the unlined end of the tunnel) was therefore moved 90 m upstream. Since then the Bjerka plant has been working satisfactorily.

The Fossmark plant (see Fig. 8) had been modernized with a new unlined vertical shaft and a pressure tunnel. The rule-of-thumb showed that the chosen design was applicable according to the rule of thumb, see Fig. 6

However, the tunnel crosses a number of vertical joints parallel to the valley side from which many water inflows occurred during excavation of the tunnel and shaft. Before watering up, they were sealed by grouting. During watering, however, hydro-jacking caused the opening of some joints, leading to unacceptable water losses. Therefore, after some tentative trials, the pressure tunnel and the shaft were steel lined. The cause of failure was a combination of unfavourable geological features and unexpected low stresses.

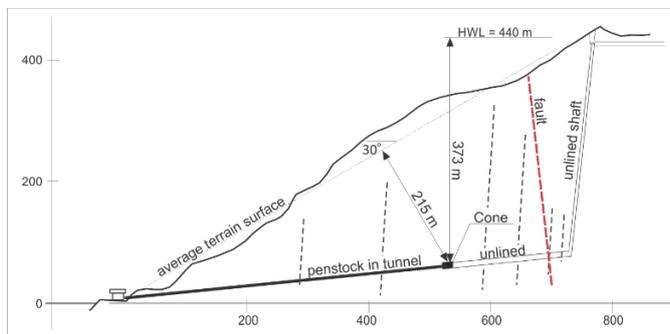


Fig. 8. The conditions at the Fossmark hydropower plant [revised from Garshol and Blindheim, 1988<sup>11</sup>]

The failures at Fossmark, and to some degree, also at Askora, showed that the rule-of-thumb was not sufficient for the design. The lowest principal stresses can sometimes be different from the estimated one. In subsequent years, the common practice has been to take stress measurements close to the location of the penstock cone, to ensure that the rock stresses are sufficient. If not, a new location for the cones is found after stress measurements.

The failure at Bjornstokk plant in 2016 took place as a result of unexpected, low vertical stresses (see Fig. 9). This might have been avoided, had stress measurements been taken. The failure caused two landslides of 5000 m<sup>3</sup>

and 130 000 m<sup>3</sup>, at distances of 400 m and 600 m from the tunnel.

Both the rule-of-thumb (in Fig. 5) and the diagram in Fig. 6 are still being used for quick, preliminary estimations.

**1.2.5 Design charts based on finite element models**

In parallel with the revisions to the rule-of-thumb, the search for better and more general design criteria was intensified at the Department of Geology at the Norwegian University of Science and Technology.

A new design tool was adopted in 1971-72. It is described in detail by Brekke *et al.* 1970<sup>6</sup>] and by Broch [1982<sup>4</sup>]. It is based on the use of computerized finite element models (FEM) and the concept that nowhere along an unlined pressure shaft or tunnel should the internal water pressure exceed the minor principal stress in the surrounding rock mass. The charts have been of great help in early phases of planning.

Later, modern commercial and more advanced computer programs were developed and used.

**1.3 Controlled watering**

A further development in the design and use of unlined pressure shafts and tunnels was to conduct controlled initial watering of the tunnel, and to carry out leakage measurements to detect any unforeseen leakage. With this procedure, the unlined tunnel or shaft is watered in stages. After infilling over a period of 10 to 30 hours, the water level in the shaft or tunnel is continuously and accurately monitored during a pause of 10 to 20 hours by an extra-sensitive manometer before the next step in the watering up. Any serious leakage can be discovered early by this, and necessary actions taken in due time. If not arrested, such seepage could lead to landslides like those at the Bjornstokk hydro plant (see Fig. 9).

Table 1 shows the infilling and dewatering rates used or Norwegian unlined tunnels.

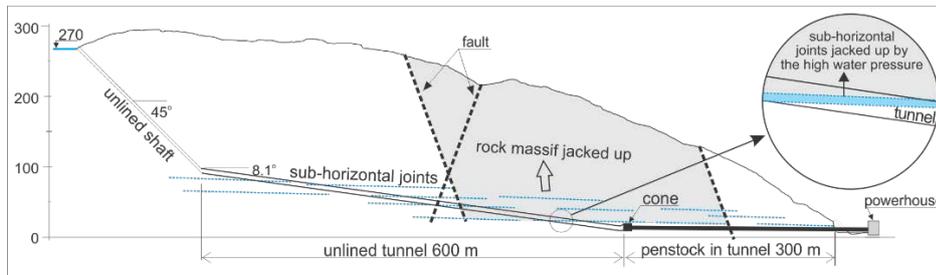


Fig. 9. Layout and section of the Bjornstokk hydropower plant, showing the probable failure principle. The leakage water moved more than 500 m and initiated two landslides. [Palmström and Buen, 2017<sup>14</sup>].

**Table 1: Rates of tunnel watering up and dewatering used in many Norwegian unlined power tunnels and shafts**

Operation	Increase/decrease in head	
	per hour	per day
First watering-up	10 m <sup>(a)</sup>	max. 200 m
Later watering-up	15 m <sup>(b)</sup>	max. 300 m
Dewatering	10–15 m <sup>(c)</sup>	max. 250 m

<sup>(a)</sup> Stop infilling, min. 2 hours for observations and measurements per 100 m head increase (10kg/cm<sup>2</sup>).  
<sup>(b)</sup> Stop infilling, min. 2 hours per 150 m head increase.  
<sup>(c)</sup> Some stops during dewatering to be evaluated for each tunnel.

**1.4 Tunnel support**

The main purpose of rock support is to reinforce unstable ground conditions in the tunnel during excavation, to obtain safe working conditions for the crew (often referred to as initial or temporary support) and a stable tunnel during the tunnel’s operating life (called final or permanent support). Appropriate rock support in a tunnel may be chosen and tailored according to the ground conditions encountered during construction. This requires follow-up of the excavation works by experienced engineering geologists.

In Norwegian unlined tunnels, the underlying design philosophy for operation of the plant is that minor rockfalls can be tolerated. As long as rockfalls in local parts of the tunnel do not develop significantly and increase the head loss, a reasonable number of minor rock falls spread out along the tunnel will not harm it or disturb operation of the hydro station. If necessary, rockfalls may be removed during later inspection and maintenance.

Normally a rock trap is located at the downstream end of the headrace tunnel to prevent rock fragments entering the turbine.

**2 Modern design of unlined hydro waterways**

A detailed, initial geological survey is essential in the planning of hydropower tunnels. In addition to assessing the geological features of the site, the survey should aim to identify possible difficult rockmass conditions, such as major faults and unsuitable rocks, or other features of importance likely to intersect the tunnel (see section 2.2).

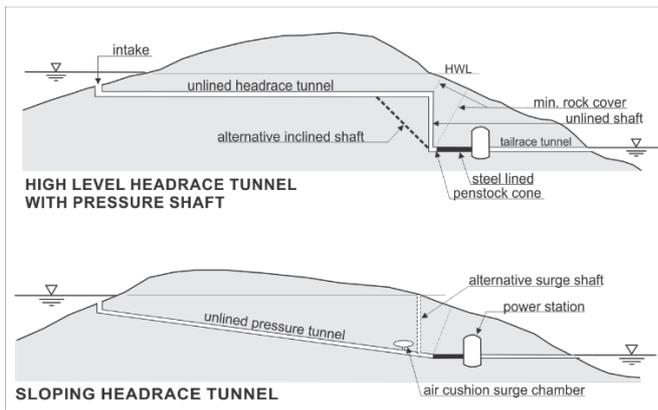


Fig. 10. An unlined headrace tunnel with an unlined pressure shaft and unlined pressure tunnel.

Fig. 11. Example of a plan and cross section of an underground hydro powerhouse with an unlined tunnel and shaft [Broch, 1982<sup>4</sup>]

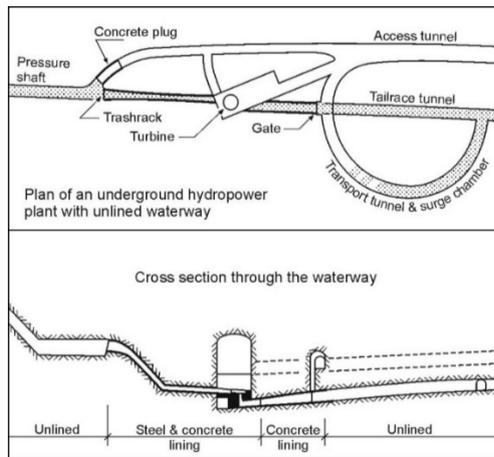
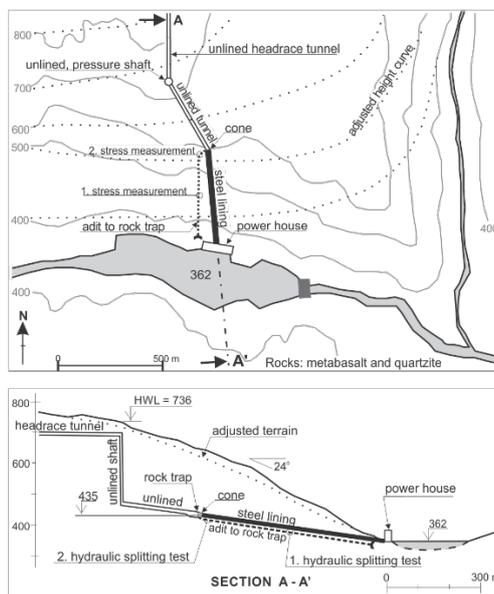


Fig. 12. The Nore hydropower plant, refurbished in 1995 [revised from Hope et al., 1997<sup>19</sup>].



maximum hydraulic gradient of 25. Around the concrete plugs, thorough high-pressure grouting of the rocks as well as contact grouting are carried out. This avoids leakage into the powerhouse and the access tunnel. Further details about the design of high-pressure concrete plugs are given in Dahloe *et al.* [1992<sup>16</sup>] and in Broch [1999<sup>17</sup>].

Normally the unlined headrace tunnel is designed with a water flow of approximately 1 m/s. The various types of permanent support will have different roughnesses. The head loss also varies with the tunnel size. The Manning’s formula or similar is used to calculate the head loss, with input for the roughness of the tunnel’s surface. The Manning’s formula can be used to compare head loss in tunnels with different support, given as:

$$A_1/A_2 = (M_2/M_1)^{0.75}$$

where  $A$  = area (cross section); and,  $M$  = Manning’s number, (see Table 2).

The two main challenges in the design of unlined pressure tunnels and shafts are to:

- avoid hydro-jacking resulting in major leakage, which means locating the tunnel with sufficient rock stresses (confinement); and,
- avoid a collapse blocking the water flow in the tunnel during power production.

The aim of the design described next is to prevent those two types of accidents, and based on this and other requirements, to arrive at a successful result.

### 2.1 Modern layout

Fig. 10 shows simplified examples of the application of modern, unlined waterways in two vertical sections. Fig. 11 presents the plan and cross section of a modern underground powerhouse equipped with one turbine. Similar layouts can be found at Norwegian plants with heads in the range of 200 to 600 m. The Tjodan plant, described by Palmström and Schanche [1987<sup>15</sup>] is an example of a design for a 950 m water head on unlined rock.

The layout should include future access ways to the tunnel and shaft for inspections, which are large enough for equipment to handle maintenance and repairs if a collapse should take place.

The critical point to avoid potential leakage failure is to locate the tunnel or shaft with sufficient confinement, that means, ensure that the minor rock stresses are higher than the water head at any point in the tunnel or shaft. This is normally where the unlined pressure shaft or tunnel ends, and the steel lining or penstock starts (at the penstock cone). The length of the steel lining to the underground powerhouse is commonly in the range of 30 to 80 m, depending on the head and the rockmass conditions. The access tunnel to the penstock cone, that is, to the start of the unlined pressure tunnel or shaft, is plugged with a concrete plug with an access gate. The steel tube (cone) with a hatch cover is in another concrete plug, see Fig. 11. The length of each of the two plugs is normally 10 to 40 m, depending on the head and the geological conditions. As a general rule, the length of the concrete plug is 4 per cent of the water head, which theoretically gives a

Type of tunnel surface	Manning's number
Unlined rock (drill and blast)	25 - 40
Shotcrete lining	45 - 55
Concrete lining	62 - 83
Steel lining	71 - 100

As an example: For an unlined tunnel excavated by drill and blast with Manning's number  $M_1 = 35$ , and another concrete lined tunnel with  $M_2 = 65$ , the unlined tunnel must be 1.6 times larger than the concrete lined tunnel to have the same head loss. Shotcrete will level out some of the irregularities in a drill-and-blast excavated tunnel, increasing the Manning's number. Using  $M_1 = 50$  (and  $M_2 = 65$ ), the shotcrete-lined tunnel must be 1.2 times larger than the concrete-lined tunnel to have the same head loss.

## 2.2 Location of the unlined pressure tunnel or shaft

### 2.2.1 Suitable rocks

Rocks suitable for an unlined water tunnel are:

- Crystalline rocks;
- unaltered igneous rocks;
- volcanic rocks; and,
- metamorphic rocks, as well as many sedimentary rocks.

Less suitable rocks are:

- some loose, friable and/or porous rocks;
- rocks containing swelling and/or slaking minerals<sup>B</sup>; and,
- some karstic rocks.

The unlined solution should not be used where such conditions occur along a significant part of the tunnel length. Crossing weakness zones/faults with slaking or swelling materials should be avoided. If this is not possible, careful sealing and grouting should be carried out, in addition to appropriate rock support. Special treatment of swelling and slaking rocks is a prerequisite. This has been described by Palmström and Stille [2015<sup>18</sup>].

### 2.2.2 Influence of rock stresses

The entire tunnel, including the shaft and the surge shaft, must be set deeply enough within the rock mass to ensure that sufficient in-situ compressive rock stress is available to prevent hydraulic splitting or jacking.

A first evaluation of the location of the unlined pressure shaft or tunnel normally involves a careful examination of the topography based on a general rule that the assumed vertical stresses calculated from the overburden are higher than the water head in the tunnel or shaft (see section 1.2.2 and Fig. 7). Ridges, or so-called 'noses' between deep ravines or depressions will to a large extent be stress-relieved. They should therefore not be taken into account in the evaluation of overburden for unlined pressure shafts or tunnels [Broch, 1984<sup>13</sup>]. The Nore hydropower plant is an example, shown in Fig. 12, describing how the ridge/nose is cut away on the map by changing the contour lines in the map and subsequently revising the vertical profile along the tunnels and shaft. Based on this, a preliminary location of the unlined tunnel and penstock cone was chosen. Further details about the Nore plant are given in Hope *et al.* [1997<sup>19</sup>]. This topographical exercise is also useful in the input to numerical analyses.

In many cases, reliable stress measurements are difficult to take before construction begins, because the critical point, that is, the penstock cone, is located deep below the surface. This was the case for the Nore plant. Therefore, the construction contract had some flexibility to locate the penstock cone according to the results of stress measurements performed during excavation of the access tunnel (see Fig. 12). The first stress measurements were taken after half of the access tunnel had been excavated. They showed stress magnitudes as expected. A second measurement was taken 25 m before the access tunnel had been driven to the planned rock trap. As this also showed acceptable stress magnitudes, the cone could be located as planned.

## 2.3 Rock stress measurements

Rock stresses can be found either by stress measurements or by hydraulic jacking tests. This is of particular importance in cases when the stress situation is not well known. The tests are normally carried out during the excavation of the access tunnel, as have been described for the Nore hydropower plant (Fig. 12). To make sure that all possible joint sets are tested in the jacking test, the test holes are normally drilled in three directions. During testing, the water pressure in the holes is raised to a level which is 20 to 30 per cent higher than the water head just at the penstock cone. If the testing shows that jacking of the joints may occur, the unlined part of the waterway will have to be located deeper into the rock. This will normally mean that the whole powerhouse complex is moved further in. A flexible contract which allows for such changes, is therefore vital when unlined high pressure shafts and tunnels are planned. Locating the powerhouse complex deeper into the rock adds length to the access tunnel, but not to the total waterway.

<sup>B</sup> As described in the examples in Section 3, some tuffs have been shown to have swelling properties; swelling that is not easily discovered by normal testing procedures.

## 2.4 Tunnel mapping

For tunnel support designs based on observations, a prerequisite is to understand the geological conditions, the rock mass characteristics and the ground behaviour, so as to identify appropriately and select the correct rock mass parameter values. It is very important to be aware of, or to detect, the occurrence of some special rocks described in Section 2.2.1, to be able to select the appropriate design and rock support. The quality of the rock supporting works installed is essential for a tunnel to function properly during the lifetime of the powerplant.

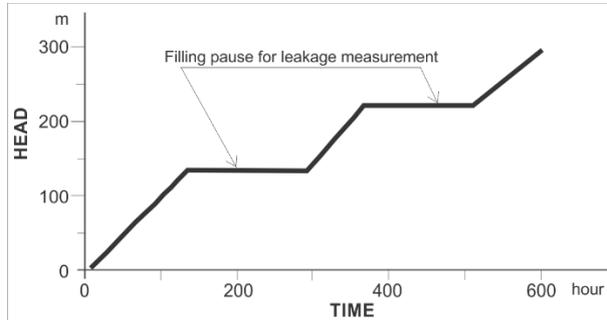


Fig. 13. An example of controlled, slow filling of a pressure conduit [Buen and Palmström, 1982<sup>20</sup>].

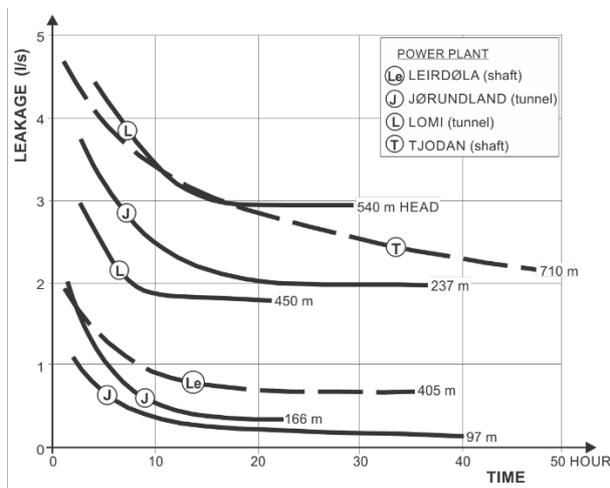


Fig. 14. Measured net water leakage from various unlined high pressure shafts and tunnels [Palmström, 1987<sup>21</sup>].

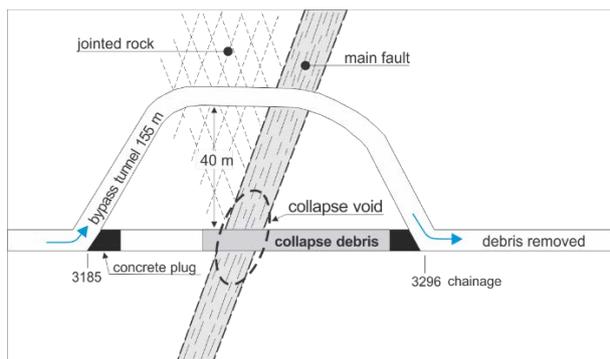


Fig. 15. The situation at the collapse in the unlined headrace tunnel at the Nye Vinstra hydro plant.

after this check can the tunnel be considered to be complete from a contractual perspective.

With respect to power generation, the dewatering should be carried out after approximately one year, according to a pre-decided schedule. If the measurement/control of water head indicates even small stability problems, the tunnel should be dewatered as soon as possible, regardless of the pre-decided schedule, to prevent the problems from developing further. It is much cheaper and faster to solve a small problem than a full scale tunnel collapse.

Later tunnel inspections, after dewatering, may be carried out two to five years after the first inspection, the time for inspection depending on the conditions observed during the previous inspection.

## 2.5 Watering and dewatering

During and after the excavation of the tunnel, the surrounding rock mass will have gradually been drained. The first watering of a pressure shaft or tunnel should therefore be done in a controlled way, with leakage being measured during the process to detect any unforeseen leakage, so that appropriate action can be taken in time.

A normal procedure is to fill the unlined tunnel or shaft in steps or intervals of 10 to 30 hours, as shown in Fig. 13. During these intervals, the water level in the shaft is continuously and accurately monitored by an extra-sensitive manometer. By deducting for the inflow of natural groundwater and the measured leakage through the concrete plug, the net leakage out of the unlined pressure tunnel or shaft to the surrounding rock mass can be calculated. Some typical leakage curves are shown in Fig. 14. Based on the measurements, an average permeability coefficient of  $1 - 10 \times 10^{-9}$  m/s has been calculated, giving a leakage of 0.5 - 5 l/s per km [Palmström, 1987<sup>21</sup>].

If serious hydrofracturing or hydro jacking takes place in the tunnel during the watering process, loose materials on the surface above can become more saturated, and if not arrested, this leakage could lead to landslides. An example of this is described in Section 1.2.3 and Fig. 9.

Table 1 shows some watering and dewatering rates used in Norway.

## 2.6 Follow up during power production

During the first months of operation, it is very important to keep a continuous watch on the potential loss of water head in the headrace tunnel and shaft. Even small head loss increases of a few centimetres could indicate serious fallouts of rock masses. A head loss of as much as 1 m may indicate a serious collapse. Then it is even more important to dewater as early as possible.

For all unlined tunnels, the most critical test for potential stability problems is the first dewatering. This is an important part of the whole construction process and should therefore not be delayed. Only

### 3 Examples of collapses in unlined headrace tunnels during power production

#### 3.1 The Vinstra hydro plant

The unlined pressure tunnel, in phyllite, at this Norwegian plant, was watered in 1999. Two years after a collapse occurred, involving a volume of 20 000 m<sup>3</sup> which extended for a length of 2 km downstream of the collapse (see Fig. 15). Adequate rock support of a sub-vertical weakness zone had not been provided in the invert. The collapse would have been avoided if the rock support works had been closely followed up, or the problem might have been discovered if inspection had been done earlier. The remedial works took only four months, thanks to good planning and an effective contractor. Easy access to the tunnel with modern equipment during the remedial works was important for efficient execution.

#### 3.2 La Higuera hydro plant

At this plant, in Chile, a collapse of the headrace tunnel occurred in 2011. An increase of head loss was registered after some months of power production, but it took one year before the tunnel was dewatered and inspection could take place to find a collapse of 12 000 m<sup>3</sup>, with debris extending 500 m downstream. The drill-and-blast excavation in volcanoclastic rocks (tuffs) had passed the potential collapse area without it being observed that the fault there had swelling potential. One reason for this was that the contractor applied shotcrete on the tunnel periphery immediately after the blast. This made it very difficult for the engineering geologist to study how the rockmass behaved over time. Later investigations found that the slow swelling of minerals was associated with frequent zeolite veins in the fault. If the tunnel had been dewatered earlier, the extent of the collapse could have been reduced considerably. Had preliminary watering been done, as was the case for the tunnel at La Confluencia hydro plant, the behaviour of the fault could have been discovered. Better access to the headrace tunnel to carry out repair works could have reduced cost and time for the remedial works, which took two years and required a 238 m-long bypass tunnel.

#### 3.3 La Confluencia hydro plant

This is located upstream of La Higuera plant in similar rock conditions. It was constructed at the same time as the collapse occurred at La Higuera. By taking advantage of the experience with swelling rocks at La Higuera, a test for rapid checking of the swelling using ethylene glycol was developed [Carter *et al.*, 2010<sup>22</sup>]. To check the site conditions further, the tunnel was watered, and after approximately one month, dewatered to inspect the ground behaviour. Additional support was installed where swelling had been occurring, and the powerplant was successfully put into operation, and has been working without problems.

#### 3.4 Glendoe hydro plant

This plant, in Scotland, is located in phyllites. The headrace tunnel was excavated by TBM. After only six months of power production, the first minor increase in head loss was registered. The head loss systematically increased, but it took four months before the tunnel was dewatered for inspection. A huge collapse of 20 000 m<sup>3</sup> was discovered. The contractor was fully aware of a potential fault zone in the area of the collapse. In spite of careful observation during the TBM excavation, no clear indication of a potential collapse of the zone was observed by the two teams of engineering geologists who mapped the tunnel. If the tunnel had been dewatered and inspected soon after the first head loss was registered, the extent of the collapse could have been significantly reduced. The remedial works, involving a 606 m-long bypass tunnel, took three years. This time could have been reduced, if the powerplant had been designed with a permanent access to the headrace tunnel.

### References

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