AP

Development of Unlined Pressure Shafts And Tunnels in Norway

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Topographical conditions in Norway are especially favorable for the development of hydroelectric energy. More than 99% of a total annual production of 90 TWh of electric energy is generated from hydro power. Figure 1 shows the installed production capacity of Norwegian hydroelectric power stations. It is interesting to note that, since 1950, underground powerhouses are predominant. In fact, of the world's 300–500 underground powerhouses almost one-half are located in Norway. Another proof that the Norwegian electricity industry is an "underground industry" is that it has approximately 2,500 km of tunnels.

Through the design, construction, and operation of all these tunnels and underground powerhouses, valuable experience has been gained. Also, special techniques and design concepts have been developed over the years. One such Norwegian specialty is the unlined, high-pressure tunnels and shafts which this paper describes.

It should be mentioned as a preliminary matter that the rock of Norway is of Precambrian and Paleozoic age. Although there is a wide variety of rock types, highly metamorphic rocks predominate. From an engineering point of view they may in general be classified as typical hard rock.

Most of our hydro power tunnels have only 2–4% concrete or shotcrete lining. Only in a few cases has it been necessary to increase this to 30–60%. The low percentage of lining is due not only to favorable tunneling conditions; it is also the consequence of a support philosophy which accepts some falling rock during the operation period of a water tunnel. As long as rockfalls in certain parts of the tunnel don't develop considerably and increase the head loss, a reasonable number of small blockages spread out along the tunnel will not harm the tunnel or disturb the operation of the hydro power station. If necessary, they may be removed during later inspection and maintenance.

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Early Unlined Pressure Shafts

During and shortly after the First World War there was a shortage of steel leading to uncertain delivery and very high prices. As a result, four Norwegian hydro power stations with unlined pressure shafts were put into operation (Table 1).

As early as 1922, three pressure shafts were described in detail in a publication from the Norwegian Geological Survey (Vogt 1922). The Herlandsfoss, Svelgen, and Toklev shafts were later investigated and described by Broch and Christensen (1962), and Herlandsfoss was discussed by Selmer-Olsen (1970).

The pressure shaft at Herlandsfoss is shown in Figure 2. According to the original design the penstock and the concrete plug were placed only 50 m from the turbine, leaving 150 m of the

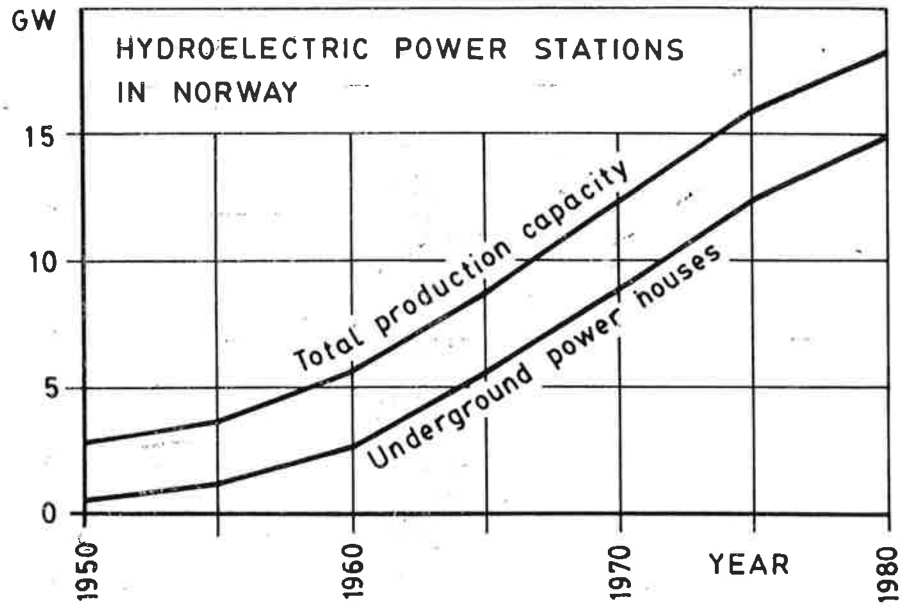

Figure 1. The development of hydroelectric power production in Norway (Myrset 1980).

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Table 1. The first unlined pressure shafts in Norway.

Name	Year	Water Head (m)	Diameter (m)	Rock Type	Experience
Herlandsfoss	1919	136	3.20	Mica-schist	Partly failed
Skar	1920	129		Gneiss-granite	Completely failed
Svelgen	1921	152	2.40	Sandstone	Minor leakage
Toklev	1921	72	2.50	Monzonite	No leakage

high-pressure tunnel unlined. During the first filling of the tunnel and the shaft, increasing leakage through the mica-schist layer was observed. The tunnel was then emptied and a 60-m-long reinforced concrete lining was placed inside the penstock. After two months of operation, rapidly increasing leakage was again observed. Inspection of the emptied tunnel revealed open cracks in the concrete on both sides of the tunnel near the springline. After this failure the penstock was extended through the whole tunnel to the foot of the shaft (Fig. 2). No leakage from the shaft has been observed since, and the power station has operated without unplanned stops for 60 years.

The pressure shaft at Skar was, to make a long and miserable story short, a complete failure and was replaced by an ordinary penstock (except for the upper part of the tunnel). The primary reason for the unacceptable leakage was the low overburden of rock, only 22 m where the water head was 116 m.

At Svelgen, leakage of 3–5 l/sec was observed as two small polluted streams during the first filling of the pressure shaft. A short section of the shaft was lined with concrete and grouted with cement. Since then the shaft has operated without problems.

The Toklev pressure shaft has functioned without any reported problems since it was put into operation.

Development of the General Plant Layout

Although three out of four pressure shafts constructed around 1920 were operating without problems after some initial problems had been solved, it took almost 40 years for the world record of 152 m of water head in unlined rock at Svelgen to be beaten. Through 1958, nine more unlined pressure shafts were constructed, but all had water heads below 100 m. Before 1950 the above-ground powerhouse with penstock was the conventional layout for hydropower plants (Fig. 3).

When the hydro power industry went underground in the early 1950's, they brought steel pipes with them. Thus, for a decade or so most pressure shafts were steel-lined. During the period 1950–65, a total of 36 steel-lined shafts with heads varying from 50 to 967 m (with an average of 310 m) were constructed.

The new record shaft of 286 m at Tafjord K3, which was put into operation successfully in 1958, gave the industry new confidence in unlined shafts. As Figure 4 shows, new unlined shafts were constructed in the early 1960's. Since 1965, unlined pressure shafts have been the conventional solution for heads up to 600 m. In 1981, the Tafjord Kraftselskap set its third world record by putting into operation an unlined pressure shaft with a water head of 780 m. However, plans are already completed for a new unlined pressure shaft with a water head of 1,000 m (Bergh-Christensen and Kjolberg 1982). Altogether, by the end of 1982, 64 unlined pressure shafts with water heads between 150 and 780 m (with an average of 314 m) were in operation in Norway. Figure 4 clearly demonstrates that increasing water heads reflect an increasing confidence in unlined pressure shafts over the last 20 years.

This confidence in the tightness of the unlined rock mass increased in 1973 when the first closed, unlined surge chamber with an air cushion was successfully put into service at the Driva hydroelectric power plant. This innovation in surge chamber design is described in detail by L. Rathe (1975). The bottom sketch in Figure 3 shows how the new design influences the general layout of a hydro power plant. The steeply inclined pressure shaft, normally at 45°, is replaced by a slightly inclined tunnel, 1:10–1:15. Instead of the conventional open surge chamber near the top of the pressure shaft, a

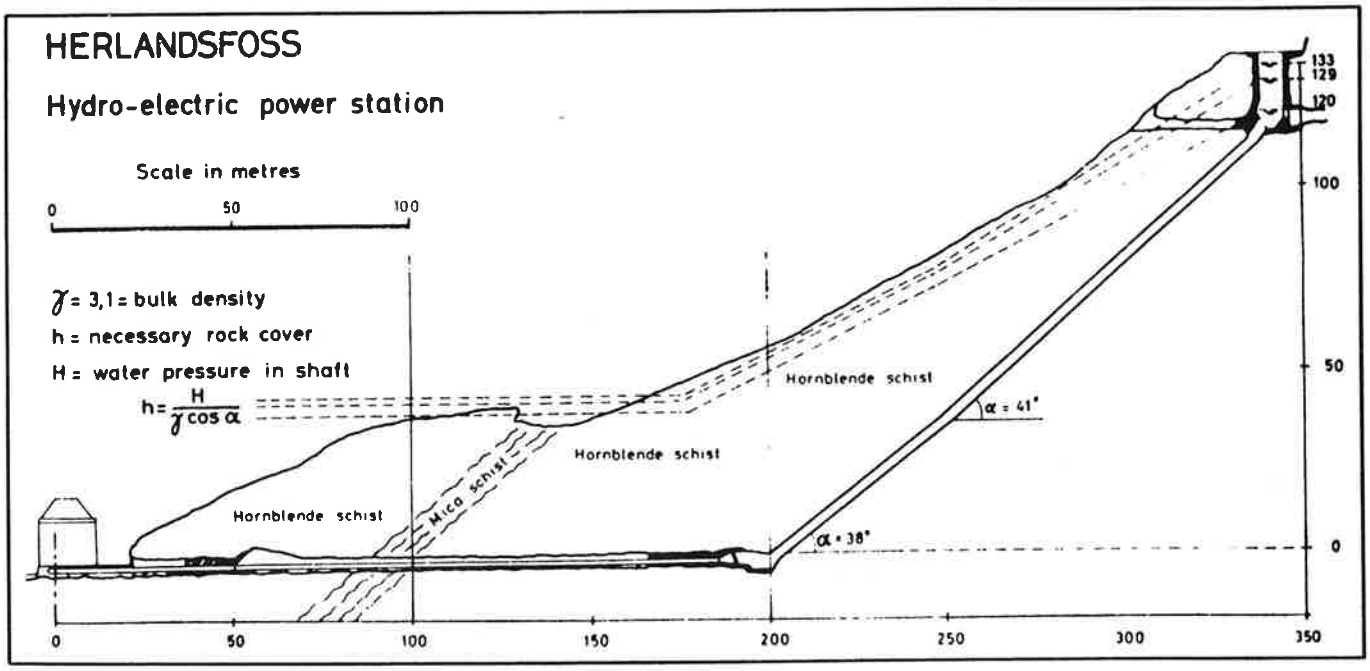

Figure 2. The Herlandsfoss hydroelectric power station, in operation since 1919 (Selmer-Olsen 1970).

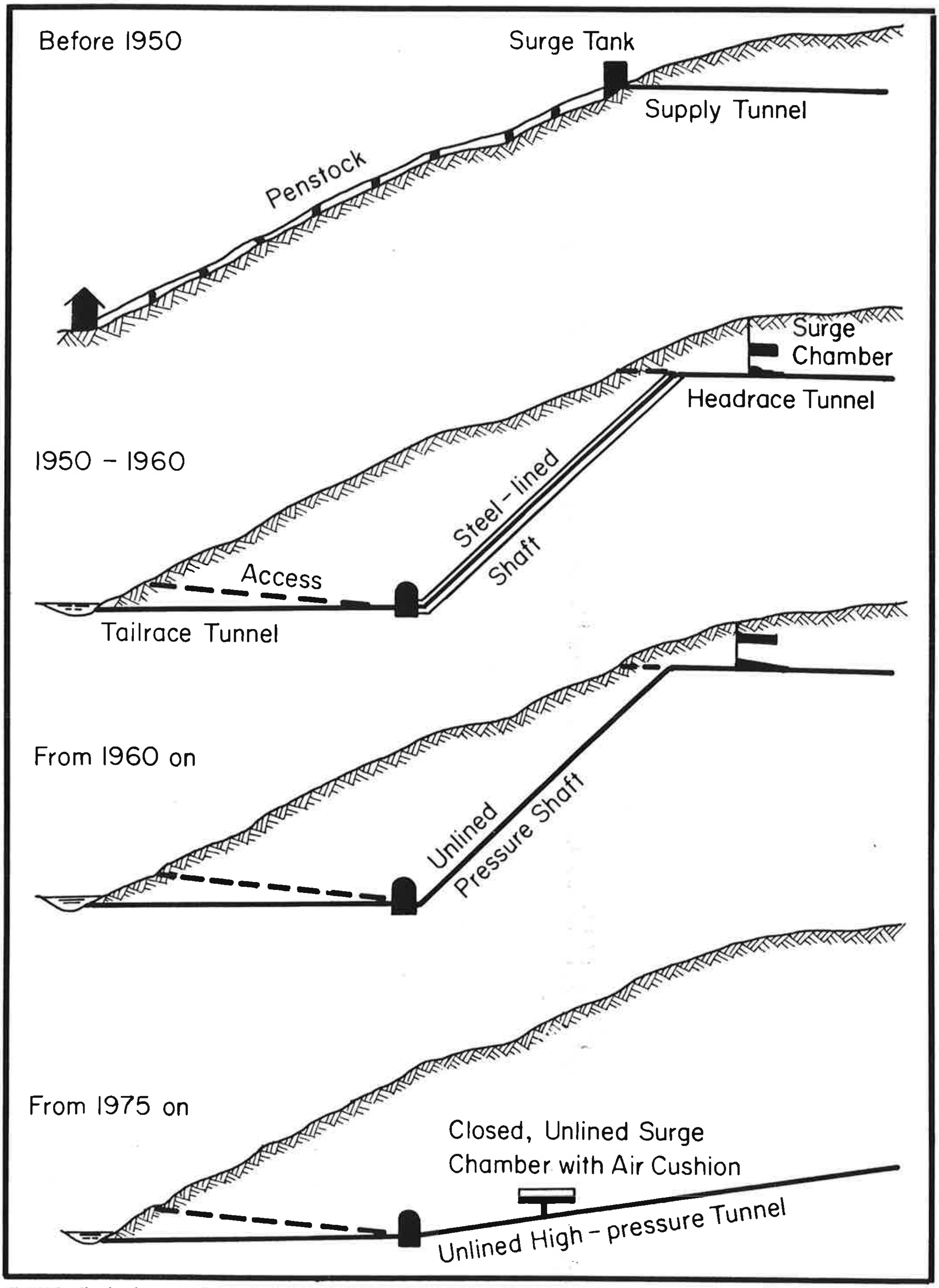

Figure 3. The development of the general layout of hydroelectric plants in Norway.

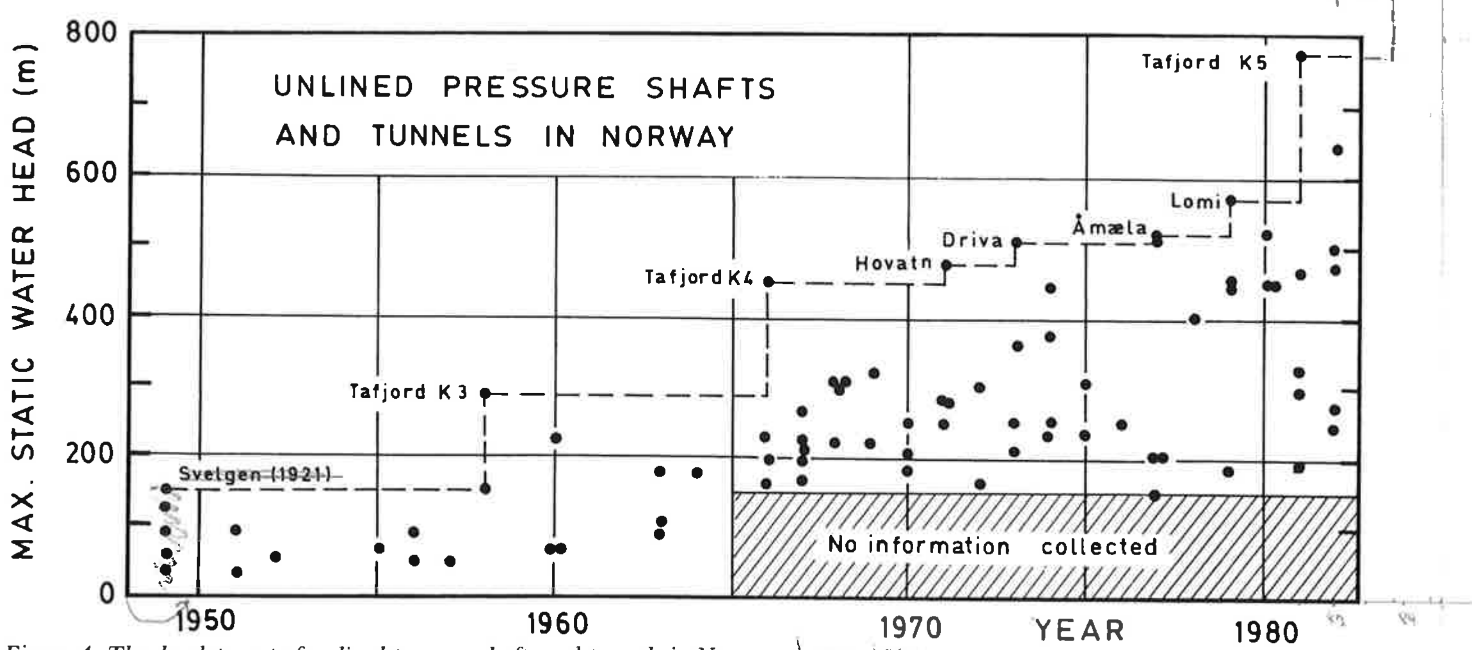

Figure 4. The development of unlined pressure shafts and tunnels in Norway. from 1950

closed chamber is excavated somewhere along the high-pressure tunnel, preferably not too far from the powerhouse.

After the tunnel system is filled with water, compressed air is pumped into the surge chamber. At Driva, where the water head at the chamber is 425 m, the total volume of the chamber is 6,000 cu m. Of this, 3,000 m is filled with compressed air. This compressed air acts as a cushion to reduce the water hammer effect on the hydraulic machinery and the waterways, and also ensures the stability of the hydraulic system. Table 2 describes the air cushion surge chambers in service.

The First Design Criteria

In summarizing the experience from early unlined pressure shafts, Vogt (1922) states that the first and foremost requirement for a pressure shaft is that leakage be avoided. He disagrees with those who at that time claimed that an unlined pressure tunnel is safe when the weight of the rock overburden is greater than the water pressure. (He even says that to get this idea out of the profession is one of his main reasons for writing the report.) According to Vogt, the main risk for unlined pressure tunnels and shafts is bad rock masses with weathered zones, joints, etc. Hence, the best way of avoiding leakage is to place the tunnel as deep into the rock mass as possible.

In the years before 1968 the rule of thumb for planning unlined pressure shafts in Norway was connected with the general layout for hydro power plants used at that time (Fig. 3). For construction reasons the inclination of the unlined shafts varied between 31° and 47°, with 45° as the most common. The rule was expressed as follows (Fig. 5):

$$h > c \cdot H$$

for every point of the tunnel, where

- h = vertical depth of the point studied (in m),
- H = static water head (in m) at the point studied, and
- c = a constant, which was 0.6 for valley sides with inclinations up to 35° and increased 1.0 for valley sides of 60°.

High valley sides steeper than 60° are rather uncommon in Norway. This simple rule was, of course, to be used with care under special geological conditions.

In 1968, the unlined pressure shaft at Byrte, with a maximum static water head of 300 m, failed. The shaft had the uncommon inclination of 60°. A revised rule of thumb which would also cover shafts steeper than the commonly-used 45° was presented by Selmer-Olsen (1970). It was expressed in a more general way as follows (Fig. 5):

$$h > \frac{\gamma_w \cdot H}{\gamma_r \cdot \cos\alpha},$$

where

- γ_w = density of water,
- γ_r = density of the rock mass, and
- α = the inclination of the shaft.

The failure of the Byrte shaft was also analyzed for the first time with a finite element model (Brekke et al. 1970).

In the fall of 1970 another failure occurred at Åskora, where an unlined tunnel in sandstone with a water head of approximately 200 m was hydraulically split. The split 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).

Table 2. Closed, unlined surge chambers with air cushions in Norway.

Name	Year Completed	Air pressure (bar)	Volume of Chamber (m^3)
Driva	1973	42.5	6,000
Jukla	1974	24	6,200
Oksla	1980	46	17,300
Sima	1980	50	7,100
Kvilldal	1981	43	100,000
Nye Osa	1981	18	12,000
Tafjord K5	1981	75	2,000
Brattset	1982	26.5	3,000

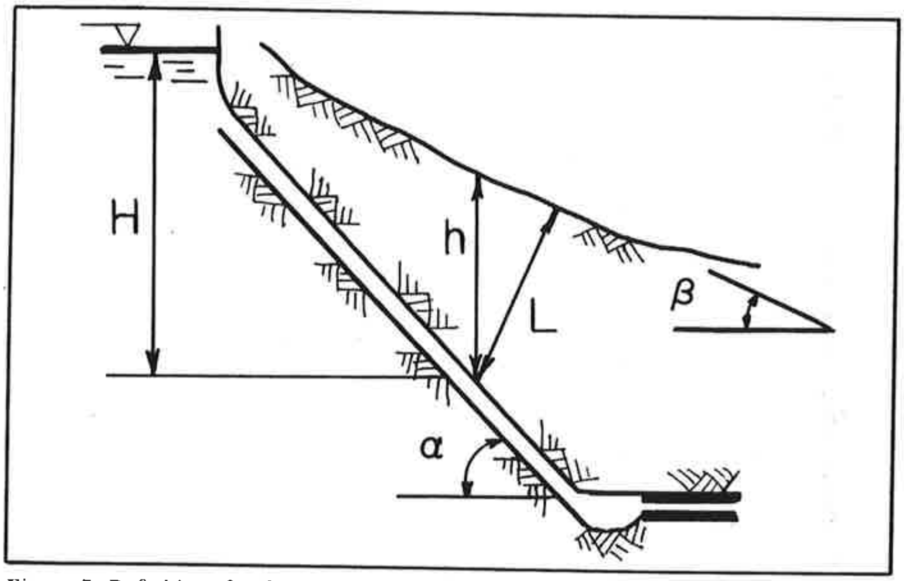

Figure 5. Definitions for the rule of thumb for tunnel design.

After this failure a new rule of thumb was introduced by Bergh-Christensen and Dannevig (1971), where the inclination of the valley side was taken directly into account (Fig. 5):

$$L > \frac{\gamma_w \cdot H}{\gamma_r \cdot \cos\beta},$$

where

- L = shortest distance between the surface and the point studied (in m), and
- β = average inclination of the valley side.

Based on this formula, a diagram showing existing unlined pressure shafts with or without leakage was presented. This was further supplied with information by the Norwegian Geotechnical Institute (1972) and is shown in a slightly revised version in Figure 6. It is worth noticing that the unlined pressure shafts where leakage is observed are, with the exception of Bjerka, plotted below the curves defined by the rule of thumb. At Bjerka unfavorable geological conditions with steeply dipping, permeable joints with strike parallel to the valley side caused leakage at a distance up to 1 km from the pressure tunnel.

Design Charts Based on Finite Element Models

Parallel with the revisions of the rule of thumb, the search for better and more general design criteria was intensified at the Department of Geology of the University of Trondheim. These criteria were to be valid for unlined pressure shafts and tunnels, and also for unlined surge chambers with compressed air cushions. The first hydro power plant with this new design, Driva, was already under construction.

The new design tool was put into use in 1971–72 and is described in detail by Selmer-Olsen (1974). 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.

Very briefly, the FEM models are based on plain strain analysis. Horizontal stresses (tectonic plus gravitational) increasing linearly with depth are applied. Bending forces in the model are avoided by making the valley small in relation to the whole model. If required, clay gouges (crushed zones containing clay) may be introduced.

In addition to real cases, a number of idealized but typical valley sides have been analyzed. One example of an idealized model is shown in Figure 7. In this case, the inclination of the valley side β = 40°, the bulk density of the rock mass γ_r = 2.75, Poisson's ratio υ = 0.2, and the $\frac{\sigma hor}{\sigma vert}$ ratio at a distance of 5 d from the valley is 0.5 (d is the depth of the valley and H the maximum static head; as Young's modulus E is kept constant, it will not influence the results). To make the model dimensionless, the static water pressure is expressed as the ratio H/d, where the water head is expressed as a height in the same units as the valley depth (e.g., in meters). The curved lines run through points where the internal water pressure in a shaft equals the minor principal stresses in the surrounding rock mass (σ_3 = H).

The use of the design charts can be illustrated by an example. Let the bottom of the valley, where the power station is located, be situated 100 m.a.s.l. and the top of the valley side 600 m.a.s.l. This makes d = 500 m. The maximum water level in the intake reservoir is 390 m.a.s.l. This makes H = 290 and the H/d ratio = 0.58. At all points inside or below the 0.58 line the minor principal stress in the rock mass exceeds the water pressure in an unlined shaft; hence, no hydraulic splitting should occur. If a factor of safety of 0.2 is introduced, the critical line will be the 1.2 × 0.58 = 0.7 line. As a demonstration, a 45° inclined shaft is placed in this position in Figure 7.

Design charts for a number of valley side inclinations, β, are available. To fit

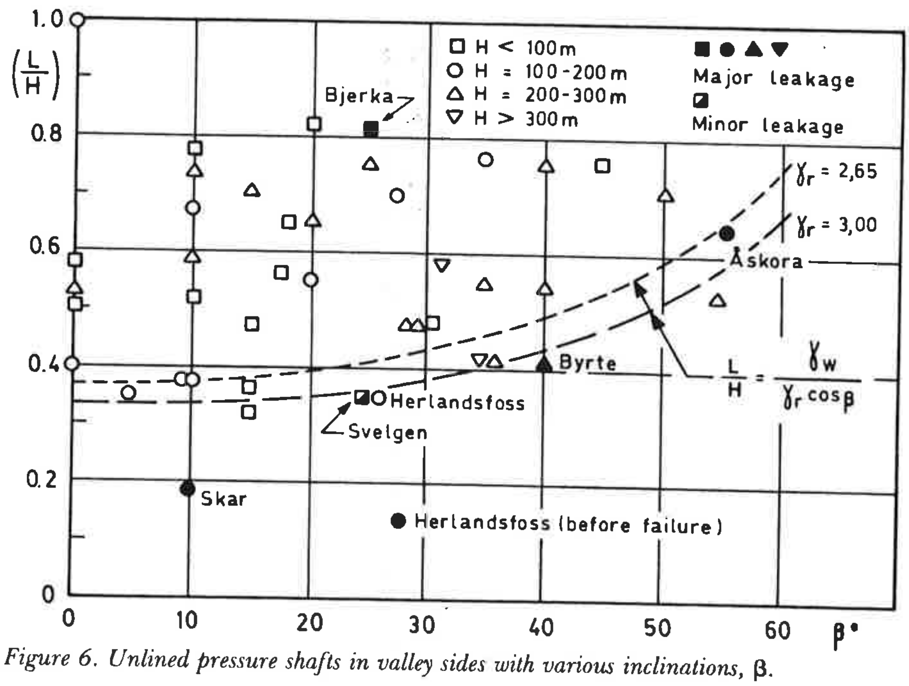

Figure 6. Unlined pressure shafts in valley sides with various inclinations, β.

the actual valley side to one of these is normally possible. In this process it is necessary to simplify and idealize the valley side by smoothing out the actual profile and ignoring protruding parts. It is also possible to make interpolations between two standard design charts. It is important that the profiles be made at a right angle to the contour lines of the map. In cases where the pressure shaft is placed in a part of the valley side which is protruding in the horizontal section, a series of profiles through the protruding part should be studied.

Through a number of analyses, all the factors influencing the results have been carefully evaluated within natural limits. For example, if a measured bulk density, γ_r, varies from the standard 2.75, this can be compensated by a correction of the overburden by the ratio $2.75/\gamma_r$. Also, an upper layer of topsoil or weathered rock masses can be compensated for by reducing the thickness of the overburden in accordance with the bulk density of these masses.

Measurements as well as observations indicate that the $\sigma_{hor}/\sigma_{vert}$ ratio in topographically undisturbed areas in Norway normally varies between 0.5 and 1.3, and very seldom exceeds 1.5. As a conservative solution, a ratio of 0.5 is used in the standard charts. An increase in Poisson's ratio, υ, will give σ_3 = H lines that go deeper in the valley side. For the standard charts, υ = 0.2 is used.

The shape and width of the valley have a major influence on the stress distribution near and under the bottom of the valley. The analyses, however, have shown that the σ_3 = H lines on levels above the bottom and at actual distances from the valley side are not influenced much. On the standard charts the width of the bottom of the valley is normally ⅓ d.

So far only two-dimensional models have been used, and the stress perpendicular to the model plane has been assumed to be the intermediate principal stress, σ_2. Stress measurements are sometimes carried out as a control, mainly where there is reason to believe that the tectonic stresses are not normal. Also, hydraulic splitting tests are sometimes used as a control of the σ_3 = H lines. For such tests it is important that the boreholes intersect natural joints with unfavorable directions with regard to the possibility of leakage from the unlined shaft. The splitting pressure is raised to 20% above the estimated minor principal stress at the actual location.

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Geological Restrictions

The FEM-developed design charts are based on the assumption that the rock mass is homogeneous and continuous, an assumption which cannot be absolutely correct even for massive Precambrian granites and gneisses. However, observations and investigations of stress-induced stability problems such as rock bursts, popping rock, and spalling rock in a large number of tunnels in fjord and valley sides clearly indicate that the natural jointing of rock masses has only minor influence on the distribution of the virgin stresses. The jointing may, however, have a strong influence on the final development of a stability problem as well as on leakage through the rock mass.

As the permeability of rock normally is negligible, it is the jointing and the faulting of the rock mass, and in particular the type and amount of joint infilling material, that is of importance when an area is being evaluated. Calcite is easily dissolved by cold, acid water, and gouge material like silt and swelling clay are easily eroded. Crossing crushed zones or faults containing these materials should preferably be avoided. If this is not possible, a careful sealing and grouting should be carried out. The

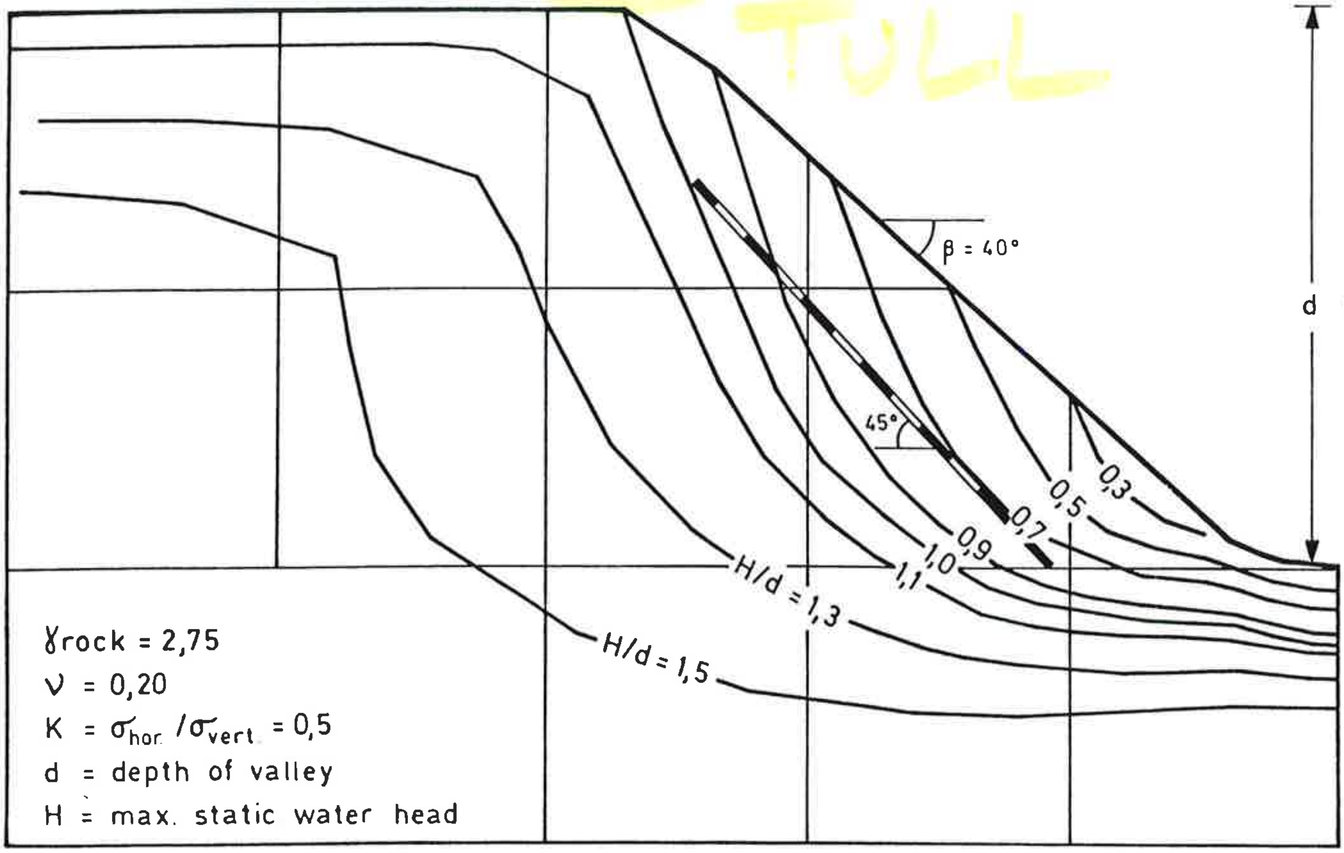

Figure 7. Design chart for unlined pressure shafts based on a finite element model. The curves run through points where the internal water pressure in the shaft equals the minor principal stress in the surrounding rock mass, $H = \sigma_3$.

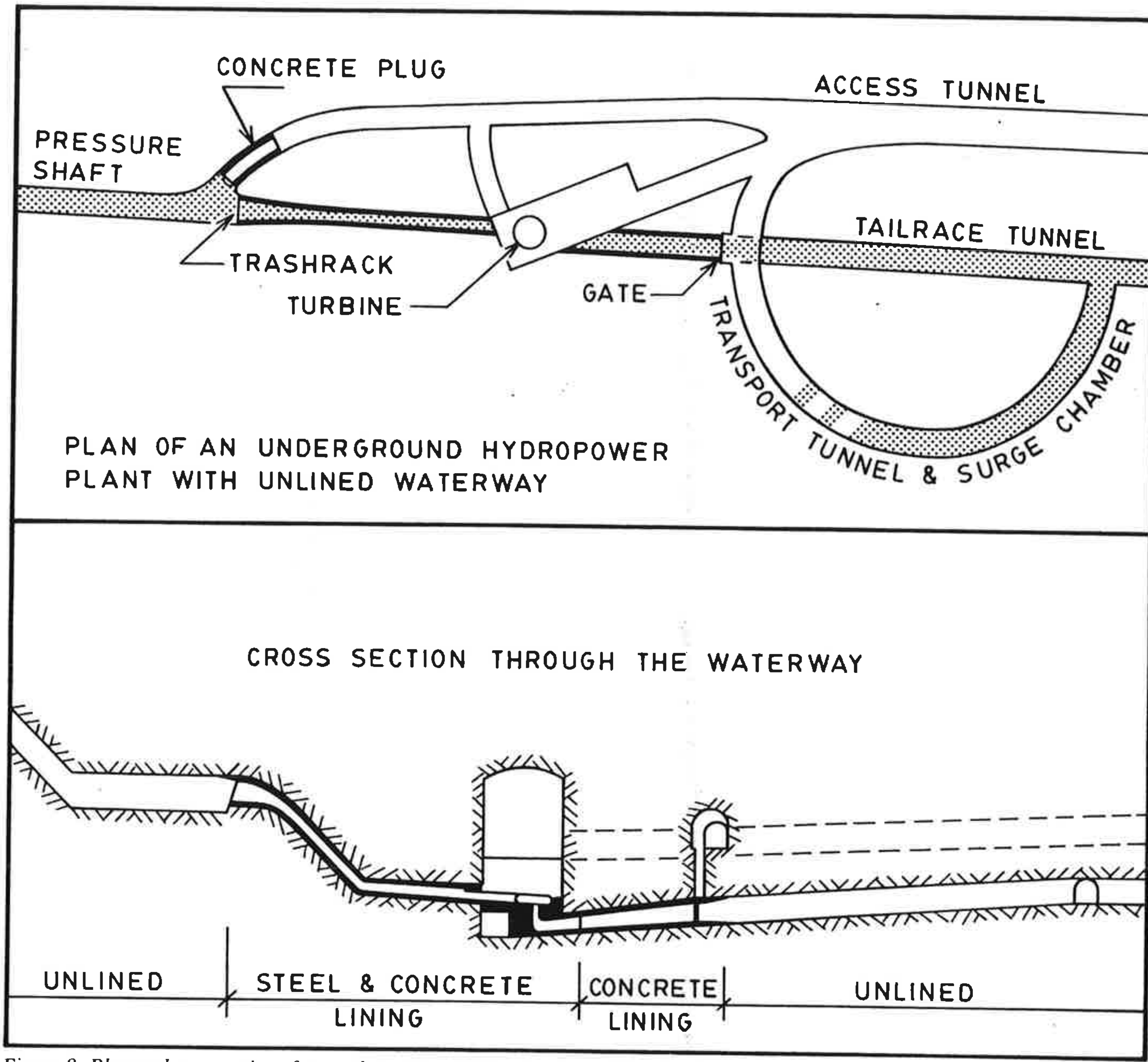

Figure 8. Plan and cross section of an underground hydropower plant with unlined waterways.

grouting is the more important the closer leaking joints are to the powerhouse and access tunnels and the more their directions point toward these. The same is also valid for zones or layers of porous rock or rock that is heavily jointed or broken.

For pressure shafts in valley sides, one should be warned against clay-filled, crushed zones in the area, as they may have an unfavorable influence on the stress distribution. If they have a strike nearly parallel to the valley side with a steep or medium-steep dip towards the valley, they are especially dangerous. Not only may they change the stress distribution; they may also often cause leakage during construction as well as during operation. The hydraulic splitting of the pressure tunnels at Åskora and Bjerka was caused by such unfavorably oriented joints and faults. A careful mapping of all types of discontinuities in the rock mass is therefore an important part of the planning and design of pressure shafts.

During the construction period it is important that changes in all leakage into the shaft or tunnel, even minor dripping or seeping leakage, be observed. For the selection of the places for unlined, closed surge chambers these observations are of crucial value, as these points of water leakage are openings where a loss of compressed air may take place. Core drilling and water-pressing tests in the drill holes are thus normally a part of the investigation for the location of these chambers.

Underground Hydropower Plants With Unlined Waterways

As a final point an example of an underground hydro power plant will be shown and briefly described. Figure 8 shows the simplified plan and cross section of a small hydro power plant with only one turbine. No dimensions are given, as the intention is to show a system rather than give details. Similar layouts can be found for Norwegian plants with water heads in the range of 200–600 m.

The figure is to some extent self-explanatory. It should be pointed out, however, that when the design charts are used the dimensioning or critical point will normally be where the unlined pressure shaft ends and the steel lining starts. This is where the selected

σ_3 = H line should intersect the waterway. The elevation of this point and the length of the steel-lined section will vary with the water head, the size and orientation of the powerhouse, and the geological conditions, in particular the character and orientation of joints and fissures. Lengths in the range of 30–80 m are fairly common.

The access tunnel to the foot of the unlined pressure shaft is finally plugged with concrete and a steel tube with a hatch cover. The length of this plug is normally 10–25 m, depending on the water head and geological conditions. Around the concrete plug and the upper part of the steel-lined shaft a thorough high-pressure grouting is carried out. This avoids leakage into the powerhouse and the access tunnels.

Concluding Remarks

Experience from a considerable number of pressure tunnels and shafts, as well as so-called air cushion surge chambers, have been gathered over a long period of time in Norway. These show that, providing certain design rules are followed and certain geological and topographical conditions avoided, unlined rock masses are able to contain water and air under pressures up to at least 75 bars, equaling 750 m water head. In the future this experience may also be important outside the hydro power industry, for instance in the construction of cheap, unlined storage facilities for different types of gas or liquid under pressure. □

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