THE USE OF GEO-ENGINEERING KNOWHOW GAVE LARGE SAVINGS FOR THE ONE-LANE LYSE ROAD TUNNEL

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ABSTRACT

The Lyse one-lane road tunnel constructed in 1982 is 1,1 km long and of 27 m2 cross section. It is part of a construction road for the Tjodan Power Plant, later to become permanent. The main reason for constructing a tunnel was to avoid an area of difficult topographical ground conditions subjected to snow avalanches and rock slides. The tunnel has been excavated in granitic gneiss having compressive strength of 150 - 250 MPa, and subjected to rock bursts.

Tunnel alignment was mostly chosen based on engineering geological evaluations. The construction experience has shown that the tunnel has a favourable alignment both with respect to the crossing of weakness zones and reduced rock burst problems. More than 0.2 mill. USD was saved in this way.

Modern tunnelling excavation technique and a basically unlined tunnel reduced the costs to 15,000 (1982) NOK/m (2,000 USD/m) tunnel all included. This paper shows that the tunnel solution in addition to shorter construction time also resulted in a cost saving of the order 1.6-1.8 mill. USD (1982) compared to an open road with slide protections.

1. Introduction

The Lyse road tunnel is located in Lysebotn east of Stavanger, at the inner end of the 50 km long and narrow fjord Lysefjorden. See Fig. 1. Up to date Lysebotn has been without road connection and the only way to arrive there has been by boat.

As a part of the Tjodan power plant now under construction, it was necessary to build a construction road from Lysebotn up the valley side to the dams and tunnel sites at elevations 800-1000 m above sea level.

The topography and ground conditions in this steep valley side makes road construction extremely difficult. Only a limited area with more gentle slopes is possible for surface road construction. As can be seen on the map it was necessary to make 21 curves for the road having a slope as steep as 1 in 9.

On the lower part of the valley side frequent snow avalanches and rock slides indicate that the road ought to be protected. A tunnel solution was therefore chosen. The alternative was to construct 800 metres of slide protection as well as special construction, part on unstable scree area.

The tunnel excavation started in 1981 and was finished one year later. The 1100 m long tunnel has a theoretical cross section of 27 m3 and grade 1:9 (11 %). The one-lane tunnel has six bypasses 15-25 m long. In the curve the road width is enlarged from 5.0 to 6.5 m (Fig.2).

In autumn 1984, the road will be connected to the main road in Sirdal on other side of the mountain area.

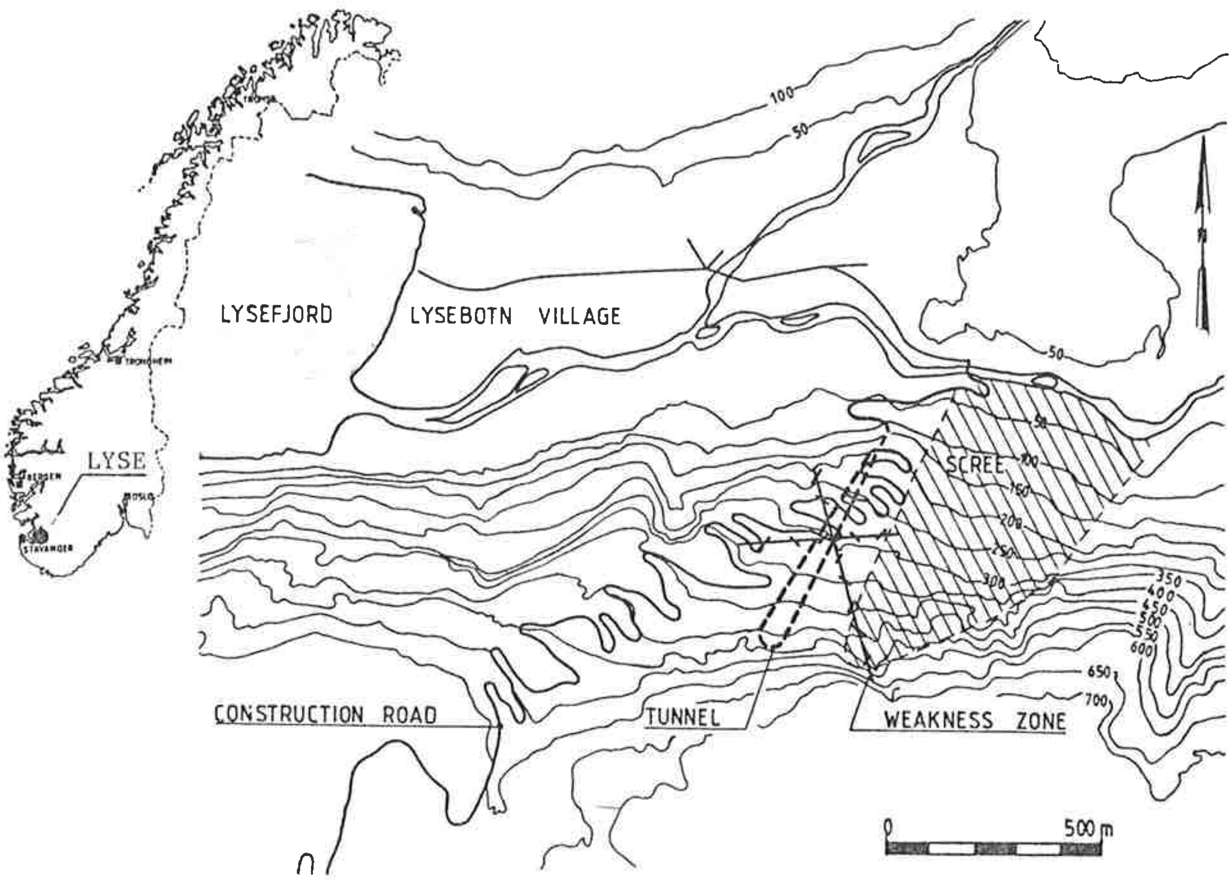

Fig. 1 Location of Lyse road tunnel

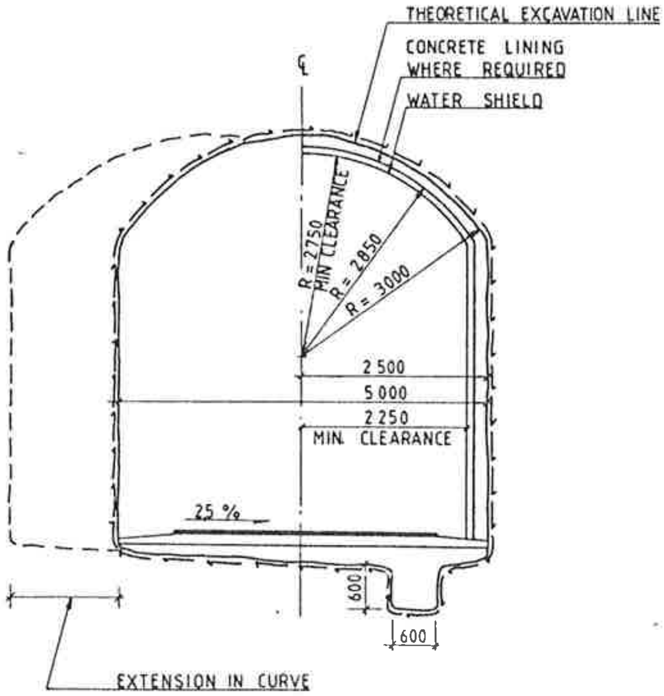

Fig. 2 Cross section of the tunnel

2. Geology

The bedrocks in the area consist of Precambrian gneisses, granites and migmatites with compressive strengths between 130 - 280 MPa. The jointing is moderate with 3 - 6 joints per m3 (Ref.3). The most pronounced joint direction is NW-55 or 50^g with regard to the tunnel axis. Flat-laying foliation joints occur along more mica-rich layers in the gneisses.

From the geological field investigations it was found that the tunnel had to pass through two weakness zones. Each of them would have to cross the tunnel twice with the alignment chosen as is shown later. These weakness zones were expected to be 1 - 5 m wide.

It was also expected that the rock mass conditions in the portals would be of poorer quality than elsewhere due to frost activity near the rock surface.

3. Rock mass evaluations and probable rock support.

Geological field observations were carried out to estimate the possible rock support. For this purpose the international Q-system for design of rock support was applied, Ref. (1). By combining different rock parameters the

$$Q \text{ value} = \frac{RQD}{Jn} \times \frac{JR}{Ja} \times \frac{Jw}{SRF}$$

gives an expression of the rock mass quality.

a) Outside weakness zones the following mean input data were used:

$$Q_1 = \frac{90}{4} \times \frac{2}{2} \times \frac{1}{2} = 11.3 \qquad \text{(good quality)}$$

In the curve where the tunnel axis was expected to have a more unfavourable direction with respect to rock pressure the SRF value was estimated to 7.5 (instead of 2) which gives:

$$Q_2 = \frac{90}{4} \times \frac{2}{2} \times \frac{1}{7.5} = 3.0 \qquad \text{(poor)}$$

b) There were two weakness zones expected to cross the tunnel. The rock mass quality in these was estimated to be between

$$Q_3 = \frac{40}{6} \times \frac{1}{3} \times \frac{1}{2.5} = 0.9 \qquad \text{(very poor)}$$

and

$$Q_4 = \frac{25}{9} \times \frac{1}{4} \times \frac{1}{5} = 0.15 \qquad \text{(very poor)}$$

Using Ref. (1) this road tunnel with a span of 5,0 m has an

$$\text{Equivalent Dimension} = \frac{\text{span}}{\text{ESR}} = \frac{5.0}{1.3} = 3.8$$

(where ESR is the Excavation Support Ratio. The value of this figure depends on the use and requirement for the tunnel).

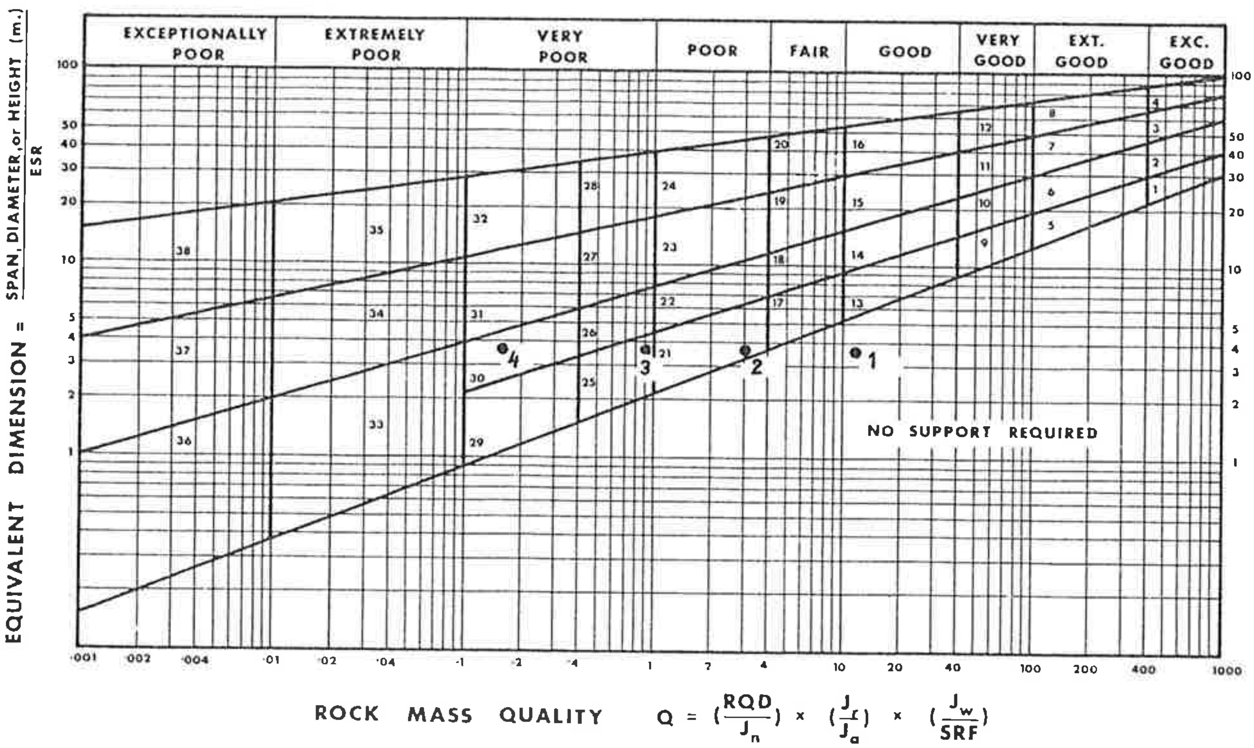

Fig. 3 Rock support categories (From Ref.1)

Based on the Q-value and the Equivalent Dimension found, the rock support categories have been found as shown in Fig.1. The probable rock support according to Ref.(1) is shown in Table 1.

Q-value	Support category no.	Probable rock support
Q_1 = 11.3	0	No support
Q_2 = 3	21	Rock bolts spaced 1 m
Q_3 = 0.9	25	Rock bolts spaced 1 m + shotcrete, mesh reinforced
Q_4 = 0.15	30	Cast concrete arch

Table 1 Estimated rock support in different rock mass conditions.

4. Tunnel alignment and construction cost savings

Norwegian tunnelling experience indicate that large construction costs are possible to save with favourable tunnel alignment based on geological evaluation of the rock mass conditions. Both the expected anistropic high rock stresses and probable weakness zones were found important in this case. In order to reduce rock burst problems as much as possible the tunnel was therefore orientated along the expected maximum principal stress i.e. across the valley side. At the same time the tunnel was located so that two weakness zones and the most pronounced joint direction, was cut at right angles.

With the chosen alignment of the tunnel it was found that only limited rock burst problems occurred. Only in the middle of the curve where it had unfavourable direction to the rock stresses more pronounced spalling phenomena (mild rock burst) were recorded. The rock support here consisted of systematic rock bolts and steel straps after some scaling works had been executed.

The two anticipated weakness zones were indeed found both in the lower and upper part of the tunnel. In the upper crossing it was necessary to carry out concrete lining because the rock mass quality was more or less according to the Q_4-value. An additional clay-containing fracture zone with parallel joints 5-10° to tunnel axis resulted in a longer concrete lining than anticipated. See Fig.4. In the lower crossing the weakness zones had a rock quality simmilar to the Q_3-value. The rock support here was carried out by reinforced shotcrete 15 cm thick as estimated in Table 1.

It became evident during construction that the direction of the tunnel with regard to rock stresses had great impact on the amount of rock support. The Q-value evaluations turned out to be a resonable correct indication of support requirements.

Based on the rock support used in the curve it is estimated that 3-5 rock bolts have been saved by choosing the alignment across instead of along the valley side. This amounts to 1500 NOK/m or 1.5 mill. NOK (200,000 USD) for the tunnel. Also allowing the weakness zones to cross the tunnel almost perpendicularly, the costs were reduced to a minimum.

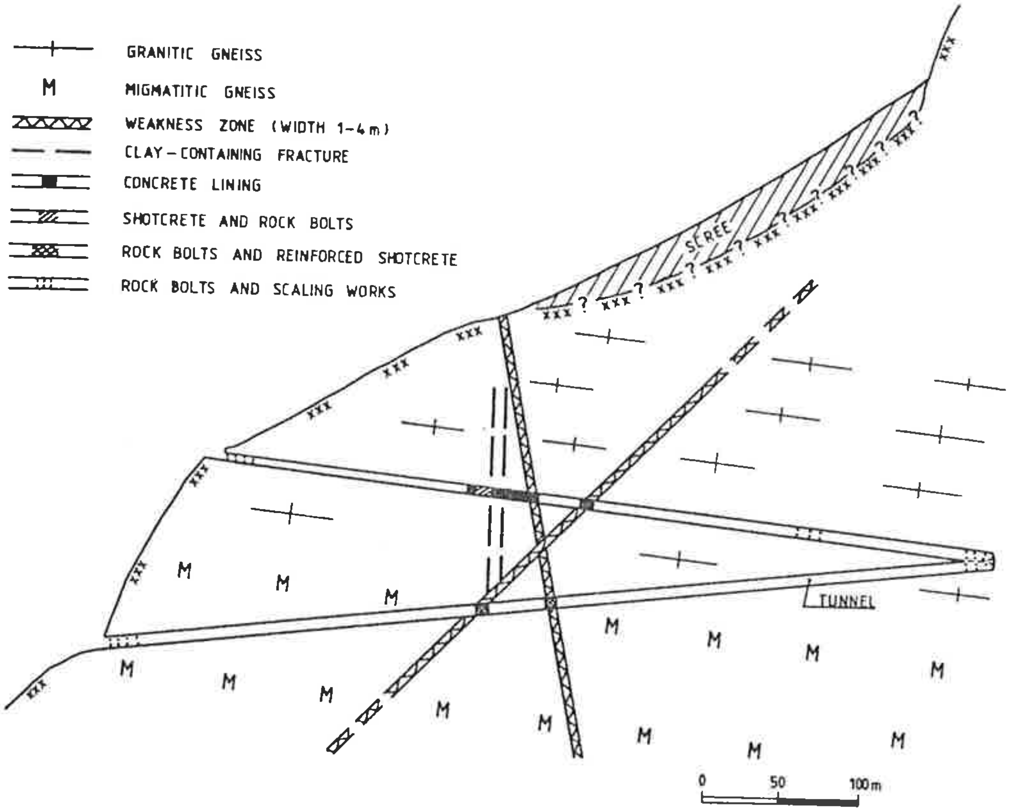

Fig. 4 Cross section along tunnel showing geology and rock support

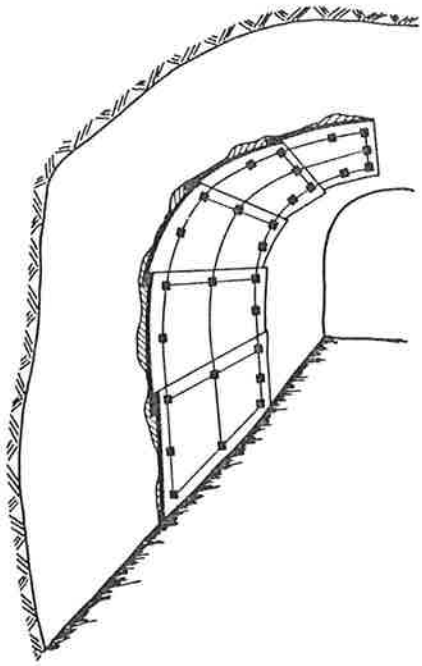

Fig. 5 Sketch showing principles for water protection by poly-ethylene foam plates, rock bolts and steel rods

Comparable costs for a surface road is difficult to evaluate. However, rough estimates for ca. 1.1 km surface road including difficult access, relatively large volumes of rock excavations and protection works for approx. 800 m, indicate that savings of the order 12-13.5 mill.NOK. (1.6-1.8 mill.USD) had been achieved.
The excavation costs included rock support of the tunnel were of the order 11,500 NOK/m. Difficult access to the tunnel portals during construction caused that this is somewhat higher than experienced in normal tunnelling works in Norway. The rock support works amounted to 1,350 NOK/m including costs for scaling works. This is only 13% of the cost for blasting and mucking out, which is lower than normal for Norwegian road tunnels.

Water protection was used only in limited areas and consisted of applying poly-ethylene foam (Alveolux) as shown in Fig.5. A total of 325 m2 was installed along approx. 50 m of tunnel. The cost of this was 200 NOK/m2. During the 2 following winter seasons this protection has been effective and quite satisfactorily.

5. References

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