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Analysis of Potential Cave-in from Fault Zones in Hard Rock Subsea Tunnels

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Summary

As a part of a research program on the rock engineering aspects of hard rock subsea tunnelling, analyses of potential cave-in from fault zones have been carried out at the Norwegian Institute of Technology. This is a topic of great importance for the planning of future subsea tunnels, and particularly for the selection of the minimum rock cover of such projects. The paper is divided into three main parts: a) review of cases of instability in Norwegian subsea tunnels, b) evaluation of theoretical maximum sliding, and c) discussion of cases of cave-in in tunnels under land. In theory, a cave-in during subsea tunnelling may propagate far higher than the normal minimum rock cover. Taking into consideration the comprehensive geo-investigations that are always carried out for subsea tunnel projects today, it would, however, be unrealistic to base the dimensioning of rock cover for future projects on worst-case scenarios. Consequently, the main result of this study is to emphasize the importance of comprehensive geo-investigations, detailed tunnel mapping, a high degree of readiness during tunnelling and a thorough quality control.

1. Introduction

The last 10–15 years, there has been a rapidly growing interest in many countries for tunnels under fjords and straits. In Norway this is a logical consequence of the country's long shore line with a large number of fjords and islands, and the fact that the majority of the population lives on the coast.

Since the late 1970s, more than 20 subsea tunnels with a total undersea length of about 50 km have been successfully completed in Norway. The majority of the projects are road tunnels with cross sectional areas between 50 and 70 m^2. The rest are mainly oil- and gas pipeline tunnels with cross sectional areas around 25 m^2.

The maximum sea depth is between 50 and 100 m for most of the Norwegian fjord and strait crossings. Often there is also a considerable soil cover on the sea floor (up to several tens of meters). Hence, the deepest parts of the tunnels are normally located between 100 and 200 m below the sea level (the Hitra road tunnel very soon will have the record with a maximum depth of 257 m below the sea level).

Ordinary drill and blast techniques have been used for all Norwegian subsea projects so far.

Generally, the rock conditions in Scandinavia are good. For the actual tunnels, however, this is not necessarily the case, as the locations of fjords and straits are often defined by major weakness zones in the bedrock. The deepest part of the fjord, and hence the most critical part of the tunnel, often coincides with particularly significant zones. The zones may have widths of 20–30 m or more, and mainly consist of crushed and altered rock. The gouge material often is of a swelling type (smectite). Swelling pressures of more than 2 MPa have been experienced.

Several new subsea tunnels (mainly road tunnels) are today under planning or consideration in Norway and are likely to be constructed within the next 4–5 years. The plans include several very challenging projects with difficult rock conditions and maximum depths of several hundred meters below the sea level.

Because of the characteristic profile of fjord crossing tunnels, see Fig. 1, the minimum rock cover will influence to a great extent the economy of such projects. For instance, for a typical Norwegian subsea road tunnel (cross sectional area 50 m^2, declination 8%), a reduction of the minimum rock cover of only 1 m may represent a reduction of about NOK 1 million (USD 150.000) in construction cost due to shorter tunnel length. In addition, considerable savings will be represented by the shorter construction time, lower operating costs (pumping, ventilation, etc.) and, not to forget, reduced fuel costs and reduced pollution, when traffic is running through the tunnel. In many cases, the savings during operation will exceed by far the savings in construction costs.

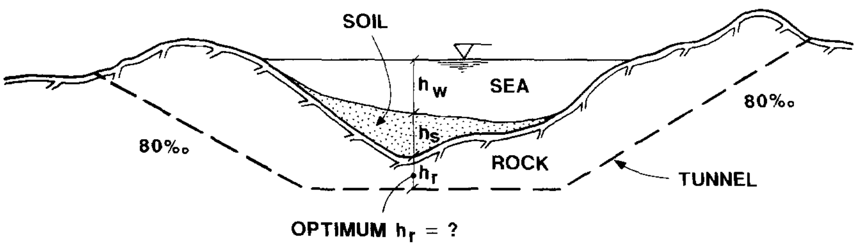


Fig. 1. Key problem in the planning of fjord- and strait crossing road tunnels

On the other hand, if the rock cover is too small, severe stability problems and large water inflow may be the result. Hence, the optimization of rock cover is a very important part of the planning of a subsea rock tunnel.

At the Norwegian Institute of Technology (NTH), research on the rock engineering aspects of subsea tunnelling has been going on continuously since 1986. Initially, a state of the art review was carried out to summarize and evaluate the experience which may be gained from the completed Norwegian subsea tunnels. Some key results of this study have been reported previously by Nilsen et al. (1988). More recently, the main emphasis has been placed on analyzing the stability and optimum rock cover. This research has covered several approaches, such as:

— empirical analyses
— rock stress analyses
— hydrological analyses
— calculation of theoretical maximum cave-in
— analyses of actual cases of instability.

The major problem so far in Norwegian subsea hard rock tunnelling has been represented by major weakness zones, and not, as might be expected, by concentrated, large water inflows. Therefore, this paper will focus mainly on the analyses of theoretical maximum sliding and cases of instability, which are topics of great interest also for planning of future subsea tunnel projects. For more detailed presentations of the empirical, rock stress and hydrological analyses, see Dahlø and Nilsen (1990), Nilsen (1990) and Lu and Nilsen (1990).

2. Instability in Norwegian Subsea Tunnels

The results from the Norwegian projects clearly demonstrate that stability problems caused by zones of faulted and crushed rock may represent a real threat for the safe completion of subsea rock tunnels. In five cases in Norway, serious instability has occurred during subsea tunnelling, and in one case (a water supply tunnel) a total collapse occurred after water filled the completed tunnel.

Thanks mainly to comprehensive geological investigation and control, and effective rock support procedures, cave-in with disastrous consequence has never occurred during tunnelling. In some cases, however, the situation has been very difficult. Probably the most difficult until today was the situation that occurred at the working face of the Ellingsøy road tunnel in 1986 (see Fig. 2).

The 68 m^2 Ellingsøy tunnel had a rock cover of about 45 m in the difficult section, and the sea depth was about 70 m. Several significant weakness zones had been identified by refraction seismic preinvestigations in this area. Hence, the rock conditions were realized to be difficult, and just before the incident, a 2.5 m blast round was carried out to shape up the tunnel face before casting. Almost immediately after blasting, however, rock fall activity started developing at the face. Shotcreting was attempted, but with no success due to seeping water in combination with clay, and consequently poor attachment to the rock. The weakness zone was beyond reach for spiling. Within 6 hours, the cave-in reached a height of about 7 m above the tunnel roof. Finally, it was decided to seal the working face with a concrete plug from the inner part of the previously concrete lined section. The resulting plug had a length of 7 m, and a total volume of about 700 m^3. The 20 m section, through the concrete plug and the continuation of the weakness zone, was carefully excavated by drilling and blasting during the following 5 weeks (Olsen and Blindheim, 1989).

At the Vardø tunnel, a similar cave-in with propagation about 7 m above the tunnel roof occurred in a section with rock cover 45 m. Here, as well as in the other cases of instability during tunnelling, stabilizing of the situation has been possible, mainly with concrete structures against the face, before development of cave-in that

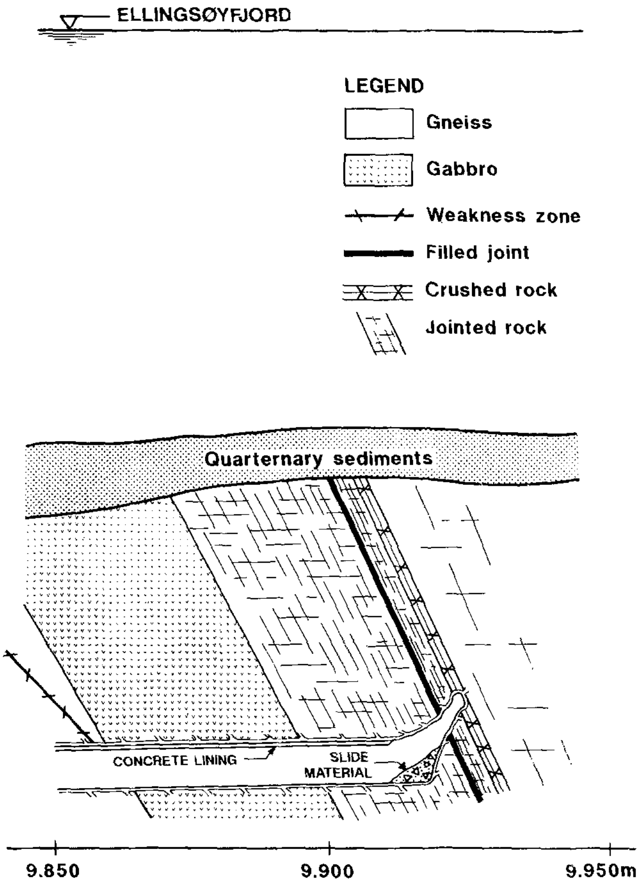


Fig. 2. Cave-in at the working face of the Ellingsøy road tunnel (based on Olsen and Blindheim, 1989)

potentially could be difficult to handle. In all cases, the stabilizing measures have made possible continued tunnelling.

Based on this part of the study, the following important common features of the experienced cases of cave-in and instability in subsea tunnels should be particularly emphasized:

— In all cases, refraction seismic preinvestigations have indicated low seismic velocities (<3500 m/sec), and hence potential stability problems. Percussive probe drilling ahead of the tunnel face has been carried out in all cases, and in one case (Vardø), also core drilling ahead of the tunnel.
— The unstable zones normally have not been very wide (width less than 10–15 m in most cases).
— The unstable zones have consisted mainly of a mixture of crushed rock and clay minerals. Swelling clay (smectite) has been a main cause of the problem in almost all cases, and in most cases a content of calcite and/or chlorite has also contributed to the problem.

— In all cases with difficult problems, there has been some water seepage through the unstable zone.
— The stand-up time in most cases has been very limited (for instance less than 0.5 hour at Vardø and less than 2 hours at Ellingsøy).

For a more comprehensive discussion of the cases of instability in Norwegian subsea tunnels, and of the special tunnelling and rock support procedures that are being used, see Dahlø and Nilsen (1990).

3. Theoretical Maximum Sliding

A potential slide during subsea tunnelling may have disastrous consequences. Comprehensive geological preinvestigations and detailed mapping during tunnelling are therefore always carried out for subsea tunnels, and special procedures for tunnelling and rock support are used in difficult rock conditions. If all directives and procedures are thoroughly followed, major cave-in in theory should not be possible.

However, to obtain a wider basis for planning of future projects, the NTH-study also has included the "impossible" incident, i.e. the theoretical maximum propagation above the tunnel roof of a potential, out of control, cave-in. The basic principle of the analysis is shown in Fig. 3.

The slide in Fig. 3 is assumed to occur at the working face, and the weakness zone is assumed to be vertical as this is the most critical situation for a potential propagation of the slide to the sea floor. The material of the zone will expand when a slide occurs, and a basic assumption of this analysis is that the sliding continues until the weakness zone is supported by slide material. In practice the internal cohesion of the zone material or other effects will, in many cases, cause the sliding to stop before this stage is reached. Thus, the analysis described here may be characterized as a "worst-case" study.

A coefficient of expansion of $u = 1.3$ has mainly been used in these analyses, representing a typical value of well consolidated clay or moraine and believed to be representative also of most fault zones. In addition, u-values of 1.2 and 1.5 have been applied, representing typical values of loosely compacted and well consolidated zone materials, respectively.

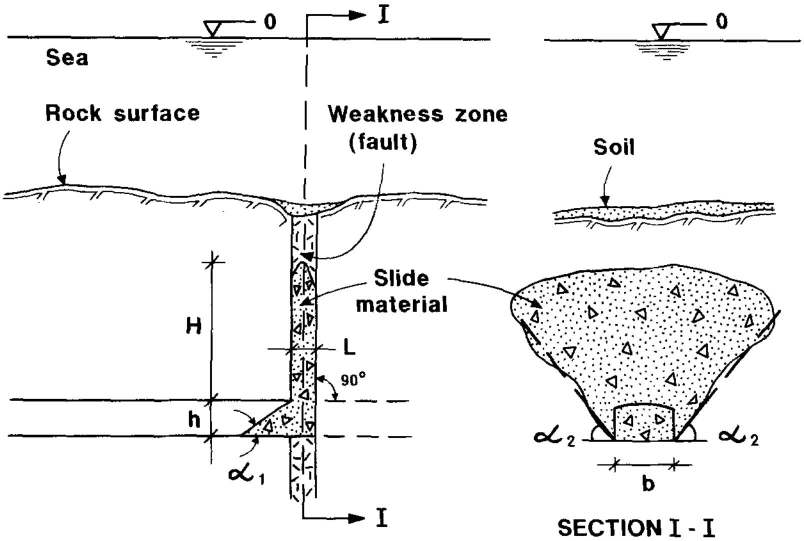
Depending on the character of the material, the scree-angle α_1 on Fig. 3 normally will vary between 25° and 40°. A high water content will reduce the angle, and also a high content of low frictional minerals like smectite, talc and graphite. In extreme cases scree angles down to about 10° have been experienced.

The calculation of the theoretical maximum height of a slide is based on the fact that the mass of slide material is the same before and after sliding:

$$V_1 \cdot \rho_1 = V_2 \cdot \rho_2 \tag{1}$$

$$V_2 = (\rho_1/\rho_2) \cdot V_1 = u \cdot V_1 . \tag{2}$$

Figure 4 shows the main results of analyses carried out for a typical, three-lane road tunnel (11.5 · 7.0 m) and a situation as shown in Fig. 3. The analyses include L-values



h = tunnel height

b = tunnel width

A = cross sectional area of the tunnel

L = length of cave-in

H = height of cave-in

α_1 = scree angle

α_2 = slide angle of weakness zone = 40°

$v_1 = L(H + h)\cdot\left(\frac{H + h}{\mathrm{tg}\alpha_2} + b\right) \div A \cdot L$ = volume of zone material prior to cave-in

$v_2 = L \cdot H\left(\frac{2h + H}{\mathrm{tg}\alpha_2} + b\right) + \left(L \cdot h + \frac{h^2}{2\mathrm{tg}\alpha_1}\right)\cdot\left(\frac{h}{\mathrm{tg}\alpha_2} + b\right) =$
volume of zone material after cave-in

$u = \frac{\rho_1}{\rho_2}$ = ratio of densities before and after cave-in = coefficient of expansion

Fig. 3. Geometry and key parameters of a potential "wedge-shaped", self-stabilizing slide in a subsea tunnel

of 2, 4 and 6 m. The former two are fairly representative of the drilled length and advance per round in poor and good quality rock mass, respectively. Various u- and α_1-values are included in the analyses, while a constant value of $\alpha_2 = 40°$ has been used for the slide-angle of the weakness zone. Based on registration of a great number of cases of cave-in in water tunnels (Thidemann, 1981), $\alpha_2 = 40°$ is believed to be representative of sliding in zones containing smectite of medium activity.

As both diagrams illustrate, the maximum height of a potential slide (H) evidently increases when the length of the slide (L) is reduced. This apparently contradictory result is explained by the fact, that for a low L-value, a relatively larger portion of the slide material will be needed to fill the tunnel. The height H is very clearly influenced by the value of the coefficient of expansion (u). As shown by

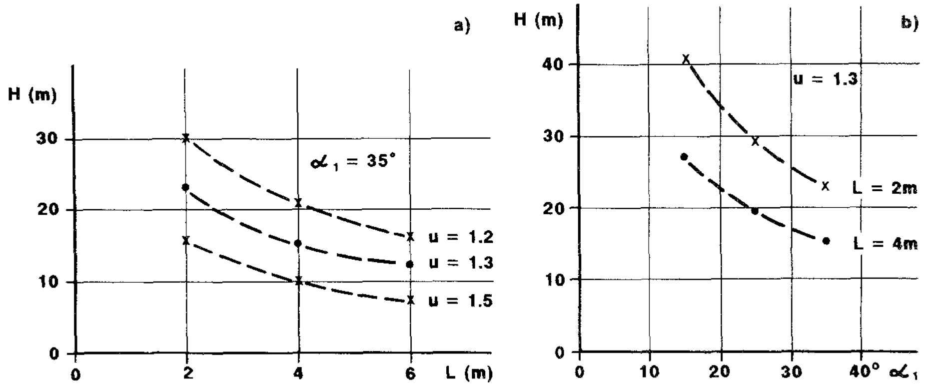


Fig. 4. Maximum height (H) of a potential slide in a three-lane road tunnel ($A = 68 \text{ m}^2$) versus **a** length of slide, and **b** scree-angle (α_1) based on the model in Fig. 5

Fig. 4a, a loosely compacted zone (low u-value) generally will have a much higher H-value than a well consolidated zone. The scree-angle (α_1) also has a strong influence on the H-value, and as shown by Fig. 4b, the relative importance of variations of this parameter is greatest for low α_1-values.

The main result in this connection is, however, the great influence which the length of the slide has on the height. Correspondingly, the width of the slide also has a great influence, and a piping situation therefore represents the most critical mode of instability. As shown by the diagrams in Fig. 4, a wedge-shaped slide may in theory reach heights of 30–40 m for a L-value of 2 m. For a piping situation the theoretical height of propagation is several times greater.

4. Cases of Cave-in in "Conventional" Tunnels

Because of the relatively limited length of subsea rock tunnels, and hence the limited basis of experience, the NTH-research on subsea tunnels also has taken advantage of the experience from the great length of "conventional" tunnels in Norway. The approximately 3500 km of hydropower tunnels and 1500 km of railroad and road tunnels under land also include a few cases of instability and cave-in, which are very well described and documented, and which are absolutely relevant also for a discussion of potential instability in subsea tunnels.

The cases of instability included in this research project all belong to one of the following main categories:

a. Cave-in at the working face during tunnelling.
b. Cave in after filling of tunnels with water.
c. Cave-in after completion in "dry" tunnels.

Category b is not very relevant for instance for subsea road tunnels, but absolutely relevant for subsea tunnels which are later to be filled with water. This is the case for instance for certain oil- and gas pipeline tunnels.

Table 1 summarizes some main data from the study of instabilities in conventional tunnels. The actual tunnels of category a are a road tunnel (Rørvikskaret, 45 m^2) and a hydropower headrace tunnel (Tonstad, about 70 m^2). The category b tunnels are both hydropower tunnels (Hemsil I, 11 m^2 and Kvenangen, 10 m^2), and the category c tunnel is a railroad tunnel (Kvineshei, 70 m^2).

Table 1. Key data for cases of cave-in in tunnels under land. Main reference for Rørvikskaret: Grønhaug (1972), for Tonstad: Thidemann (1981), for Hemsil I and Kvineshei: Brekke and Selmer-Olsen (1965)

Time of incident	a) During tunnelling		b) After water filling		c) After completion
Tunnel	Rørvikskaret	Tonstad	Hemsil I	Kvenangen	Kvineshei
Unstable zone					
Thickness (m)	5–10	7–8	2.5	<10	~ 2
Dip angle (deg.)	75	steep	80	steep	>60
Water leakage[2]	s	[1]	s	[1]	s – 1
Minerals[3]	1, 3, 4, (2)	1, 2	1, 2	5, 6, (1)	1, 3, 5
Stand-up time	<1 h	4–5 min	<7 mths[4]	~ 20 yrs[4]	~ 8 years
Rock support	none	none	shotcrete	concrete	concrete
Cave-in					
Rate of propagation	>5 m/day	high	[1]	[1]	high
Volume (m^3)	[1]	>1200	~ 400	~ 750	~ 500
Height above roof (m)	~ 100	[1]	10–15	[1]	~ 25
Probable cause[5]	a	b, c	a	c	c, (a)
Continued tunnelling					
Probe drilling	yes	yes	[1]	[1]	(tunnel)
By-pass	yes[6]	yes	—	yes	—
Freezing	—	yes	—	—	—
Forepoling	yes[6]	—	—	—	—
Grouting	yes[6]	—	—	—	—
Shotcreting	—	yes	—	—	—
Concrete lining	yes	yes	yes	yes	yes

[1] No data
[2] *s*: small leakage (seeping water), *l*: large (flowing water)
[3] 1: smectite, 2: chlorite, 3: calcite, 4: mica, 5: in-situ altered rock, 6: composite sheet minerals
[4] Stand-up time for supported zone
[5] *a*: swelling of smectite, *b*: squeezing due to high rock stresses, *c*: gouge minerals other than smectite/crushed or in-situ altered rock
[6] Attempted, but not part of the final solution

Principle sketches of two of the incidents, Rørvikskaret and Kvineshei, are shown in Figs. 5 and 6, respectively.

At Rørvikskaret, the cave-in occurred after 200 m of tunnelling, and, according to Grønhaug (1972), with practically no warning shortly after mucking and hauling of the previous round. Attempts to remove the slide material only caused further development of the instability in the Southern side as shown in Fig. 5. Forepoling and grouting from a by-pass was attempted, but with no success. Very briefly, the final solution at Rørvikskaret involved mucking and hauling of all slide material, casting of a concrete plug (as for the Ellingsøy tunnel) and, finally, drill and blast tunnelling through the plug.

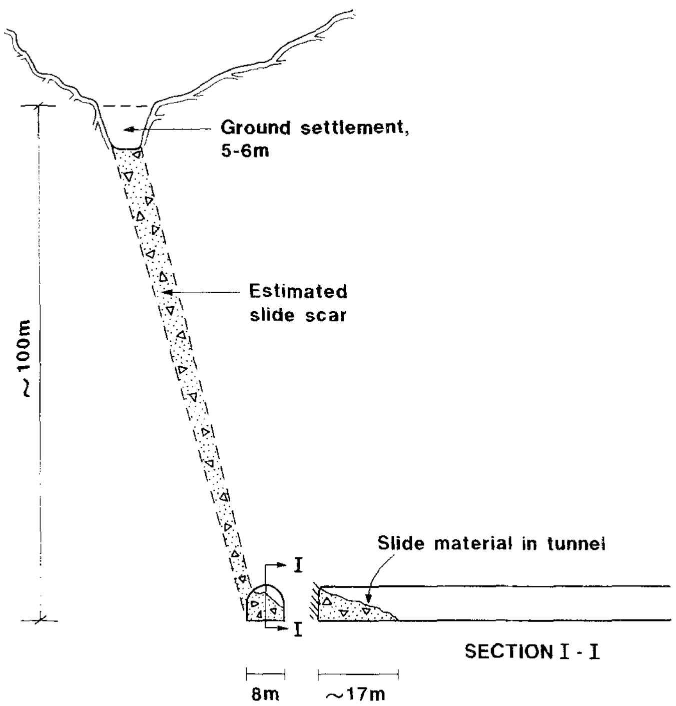


Fig. 5. Situation at the Rørvikskaret road tunnel about 2.5 weeks after initial cave-in (after Grønhaug, 1972)

At Kvineshei, the cave-in occurred as much as 8 years after completion of the tunnel, and it is also remarkable in this case that the incident occurred in a concrete lined section. The slide scar, according to Brekke and Selmer-Olsen (1965), had the geometry of a diameter 4–6 m pipe, and reached about 35 m above the tunnel floor, see Fig. 6. Dissolution of calcite, high groundwater pressure and swelling of smectite are described as the probable main causes of the incident.

Poor concrete quality, insufficient thickness of the concrete lining and rock falls on the framework, however, are also likely causes for the Kvineshei cave-in. In the more recent Kvenangen case, these factors are believed almost certainly to be the main causes.

Based on the study of conventional tunnels, the following factors, which are all relevant also for subsea tunnels, should be particularly emphasized:

- A cave-in may propagate several tens of meters above the tunnel roof (cf. Rørvikskaret and Kvineshei), and much higher than what is the normal minimum rock cover for subsea tunnels.
- Instability at the working face may develop very quickly (stand-up time only 4–5 minutes for the Tonstad slide, for instance). On the other hand, cave-in may also occur several years after completion of the tunnel (thus, after 8 years at Kvineshei, and after about 20 years at Kvenangen).
- Swelling of smectite seems to be the main cause of instability, and in particular

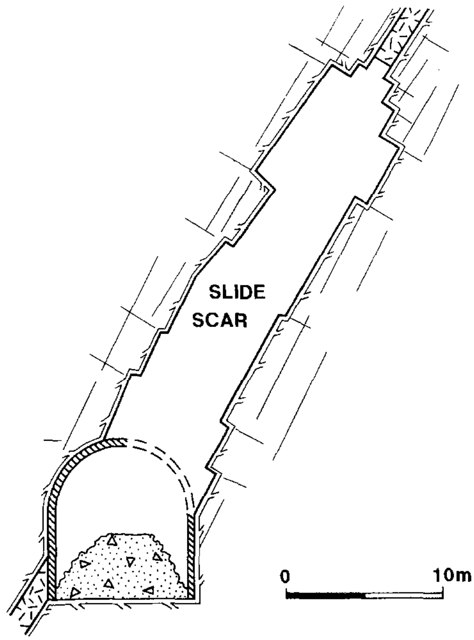


Fig. 6. Cave-in at the Kvineshei railroad tunnel (after Brekke and Selmer-Olsen, 1965)

the situation has been difficult when smectite occurred in combination with other problem-minerals like calcite (solvable) and chlorite (low frictional). Shotcreting of smectite-bearing weakness zones has turned out in several cases to be insufficient as rock support.

— Cave-in may occur also in fully concrete lined sections of a tunnel (cf. Kvenangen and Kvineshei). If cave-in in such sections occurs, it is very likely, however, that poor quality concrete lining is the main reason.

— Attempts to remove the slide material may cause the cave-in to propagate further without control (cf. Rørvikskaret and Tonstad).

5. Discussion

As shown by this study, with respect to instability and slide mechanisms, there is, in principle, no big difference between subsea tunnels and tunnels under land. Thus, for both categories of tunnels, the main stability problems have been associated with steep and relatively narrow weakness zones (width in most cases less than 10 m); the characters of the zones have been very similar with smectite, and often calcite and chlorite as the main problem minerals, and in most cases there has been water seepage through the zone. A common feature also is the fact that in most cases of cave-in during tunnelling, the stand-up time has been very limited.

Theoretical analyses as well as practical experience presented here clearly indicate that cave-in during subsea tunnelling potentially may propagate far higher than what until today has been the case, and also higher than the normal minimum rock cover of such tunnels, see Fig. 7.

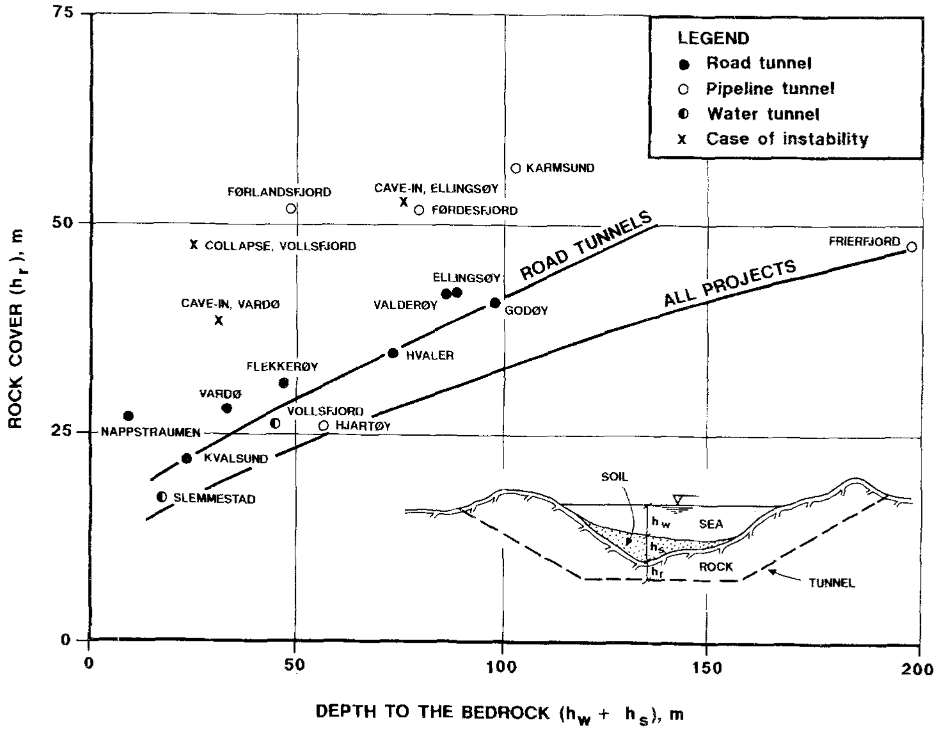


Fig. 7. Minimum rock cover under sea for Norwegian subsea tunnels as function of the depth to the bedrock

Thus, a "worst-case" cave-in has never occurred in a Norwegian subsea tunnel. In theory, this could mean that extremely difficult rock conditions so far have not been encountered. It is much more likely, however, that the main reasons are the comprehensive geological preinvestigations and the extensive mapping during tunnelling that are carried out for this category of tunnels.

A second factor which very likely is a part of the explanation why more critical situations have not occurred during tunnelling, is the fact that the few cases of instability and cave-in generally have not occurred where the rock cover is at a minimum. For the most dramatic incidents until today in Norwegian subsea tunnels (Ellingsøy and Vardø), this is illustrated by Fig. 7.

Hence, it would be unrealistic to base the final decision concerning optimizing of the minimum subsea rock cover on calculation of theoretical maximum sliding or worst-case situations from conventional tunnels. The curves in Fig. 7 probably also represent relatively high factors of safety. Thus, the great number of underwater piercings ("lake taps") in Norway, for example, include several cases of subsea tunnelling with rock cover less than 10 m at water depths up to 100 m, and in Japan the Kanmon rail tunnel was excavated with a minimum rock cover of only 10 m at 10 m water depth as early as 1944, (Miyaguchi, 1986).

6. Conclusions

In theory, a cave-in during subsea tunnelling may propagate far higher than what is today the normal, minimum rock cover under the sea, and it may have a stand-up time so limited that sufficient rock support is very difficult to install in time.

However, because of the potentially disastrous consequences of a major slide, particular comprehensive geo-investigations prior to, as well as during, excavation are always carried out for subsea tunnels. In Norway, for instance, the costs of preinvestigation for subsea tunnels are normally 5–10% of the total tunnel costs, while for conventional tunnels they are often less than 1%. Based on the results of the comprehensive preinvestigations and the tunnel mapping, all zones of poor quality rock are identified and supported at the working face.

Provided that investigations, planning, excavation and rock support are carried out according to all good intentions, no cave-in should be possible. Therefore, the main result of this study is to emphasize the importance of the following factors:

1. Comprehensive, high-quality geological and geophysical preinvestigations to eliminate all uncertainties concerning the ground conditions.
2. Detailed mapping during tunnelling, continuous probe drilling ahead of the tunnel, and geophysical investigations if required, for control and potential revision of the preinvestigation results.
3. A high degree of readiness for all types of immediate rock support and a continuous quality control of all work.

For the cases of instability in Norwegian subsea tunnels, there is reason to believe that the major weak point has been the performance and interpretation of the control described in point 2.

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References

Brekke, T., Selmer-Olsen, R. (1965): Stability problems in underground constructions caused by montmorillonite-carrying joints and faults. Eng. Geol. 1 (1), 3–19.
Dahlø, T. S., Nilsen, B. (1990): Stability and rock cover for subsea tunnels. In: Krokeborg,

J. (ed.), Strait crossings, Proc., 2nd Int. Symposium on Strait Crossings, Trondheim; Balkema, Rotterdam, 193–201.
Grønhaug, A. (1972): Investigations of cave-in at Rørvikskaret road tunnel. (In Norwegian). Proc., Annual Swedish National Conference on Rock Mechanics, Stockholm; IVA, 227–237.
Lu, M., Nilsen, B. (1990): Analytical study of the minimum rock cover for subsea tunnels. In: Krokeborg, J. (ed.): Strait crossings, Proc., 2nd Int. Symposium on Strait Crossings, Trondheim; Balkema, Rotterdam, 201–208.
Miyaguchi, K. (1986): Maintenance of the Kanmon railroad tunnels. Tunnelling Underground Space Technol. 1 (3/4), 307–314.
Nilsen, B. (1990): The optimum rock cover for subsea tunnels. In: Hustrulid, W. A. Johnson, G. A. (eds.): Rock mechanical contributions and challenges, Proc., 31st U.S. Symposium on Rock Mechanics, Golden; Balkema, Rotterdam, 1005–1012.
Nilsen, B., Maage, M., Dahlø, T. S., Hammer, T. A., Smeplass, S. (1988): Undersea tunnels in Norway: a state-of-the-art review. Tunnels Tunnelling 20 (9), 18–22.
Olsen, A., Blindheim, O. T. (1989): Prevention is better than cure. Tunnels Tunnelling 21 (3), 41–44.
Thidemann, A. (1981): Long term stability in water tunnels. (In Norwegian) Dr.ing. thesis, Norwegian Institute of Technology, Trondheim.

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