TUNNELS AS ELEMENTS OF THE ROAD SYSTEM

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ABSTRACT
Tunnels can eliminate conflicts arising from the need for roads and the desire to preserve areas of high user value and aesthetic appeal. Tunnels designed according to modest standards, and which are thus comparatively cheap (low-cost tunnels) can therefore be the key to a wider extension of the road network. The designation "low-cost" can be applied to those tunnels in which the choice of standards is financially balanced against the standard of the network it serves, where the support work is carefully matched to the purpose for which it is intended, and where the equipment requirements have been relaxed, provided this only has a minor effect on safety in comparison to other measures used in the road network.
A study of traffic safety in tunnels was undertaken at the Norwegian Technical University in Trondheim in 1981. The standards of the tunnels investigated must be described as very modest. It was concluded that tunnels are generally slightly safer than uncovered roads.
In Scandinavia alone, some 2,200 people are killed annually in traffic. We lack the resources to construct completely safe roads. Thus it would seem somewhat unreasonable to demand a standard for some elements in the road system which can be expected to yield only marginally greater safety than a slightly lower but considerably cheaper standard.

l. GENERAL
The development of a road system must take into account the need for:

- Efficient transport,
- Modest construction and maintenance costs,
- High safety standards,
- Preservation of the environment.

It is difficult to establish a road system that can satisfy all these requirements. Roads located with a view to efficient transport and low construction and maintenance costs often tend to come into conflict with the desire to preserve areas of high user value or aesthetic appeal.
Tunnels can eliminate some of these conflicts. In areas of difficult topography, tunnels can help to shorten road routes and permit development without undue injury to the landscape. In urban areas use of tunnels can spare existing buildings and valuable sites whilst permitting roads to be built along the most convenient routes.
A large number of tunnels have been constructed in Norway. The great majority of them have been intended to improve transport conditions in rural districts. These are tunnels carrying comparatively light traffic loads, and designed according to modest standards in order to reduce construction costs. On roads in heavily built-up areas, tunnels are less common.
This is primarily because they used to be costly, both to construct and to run. Tunnels were regarded as traffic risks and as inconvenient elements of the road system. In the past, consequently, those responsible for the roads had reservations about road plans that necessitated extensive tunnelling. This attitude is gradually changing, for one thing because present cost trends make rock tunnels, in particular, an increasingly interesting alternative.
As a rule, the traditional reaction to building new main roads is strong opposition. Such work normally entails severe encroachments upon established environments.
New roads laid in tunnels, on the other hand, are usually accepted, but the design that has customarily been demanded of such tunnels in urban areas has entailed such heavy costs that in

many cases they are prohibitively high. If we were able to find technical solutions that made the construction of tunnels less costly, while at the same time providing acceptable standards of safety and traffic flow, it would be possible in many cases to obtain both political consent to a further extension of the road network, and grants of funds for the purpose.
In this context it is important to regard the tunnel as an element of the road system, and to choose standards that are in conformity with those of open roads. At the same time, standards must be based on analyses which illustrate costs and effectiveness in relation to variation in the quality of the standard components.

2. COSTS

2.1 Construction costs

As Fig. 1. shows, the costs of different types of tunnels depend to a considerable extent on the support work and equipment required.
Simple tunnels can be constructed in sound rock at a cost corresponding to that of open road under difficult conditions. On the other hand, tunnels can cost more than bridges if extensive support work is required, and a complicated ventilation system is chosen. In order to bring the price down, it is therefore important to make a careful choice of standards, both for equipment and geometry

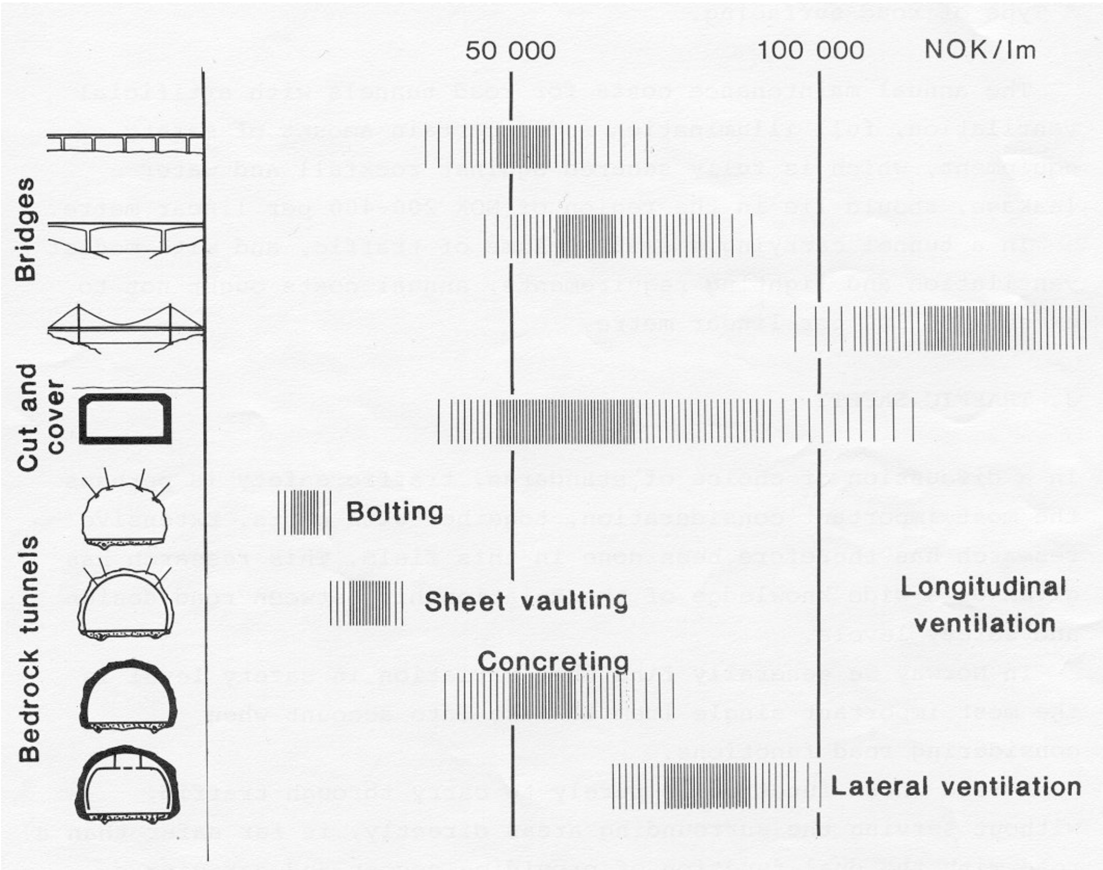


Fig. 1. Cost ranges for various two-lane road elements, in terms of 1984 NOK.

2.2 Operating and maintenance costs

Some of the factors which influence operating and maintenance costs are:

- Traffic volume and safety requirements
- Leakage and state of stability, type and ex tent of support work

- Drainage system
- Electricity
- Extent of technical installations
- Type of road surfacing.

The annual maintenance costs for road tunnels with artificial ventilation, full illumination and a certain amount of safety equipment, which is fully secured against rock fall and water leakage, should lie in the region of NOK 200-400 per linear metre.
In a tunnel carrying a small volume of traffic, and with modest ventilation and lighting requirements, annual costs ought not to exceed NOK 100 per linear metre.

3. TRAFFIC SAFETY

In a discussion of choice of standards, traffic safety is perhaps the most important consideration, together with costs. Extensive research has therefore been done in this field. This research has given us a wide knowledge of the relationship between road design and safety levels.
In Norway we generally find that variation in safety level is the most important single item to take into account when considering road functions.
A road whose function is purely to carry through traffic, without serving the surrounding areas directly, is far safer than a road with the dual function of providing access and carrying a considerable volume of traffic.
A multi-lane motorway has an accident rate (number of accidents per million vehicle kilometres) of less than 5% of that which may be expected for town centre streets with heavy through traffic and many shops. Accident rates for different types of road are shown in Fig. 2.
This also includes the accident rates for tunnels. The statistics for tunnel accidents have been taken from a study carried out for the Norwegian Public Roads Administration by the Foundation of Scientific and Industrial Research at the University of Trondheim. (SINTEF)

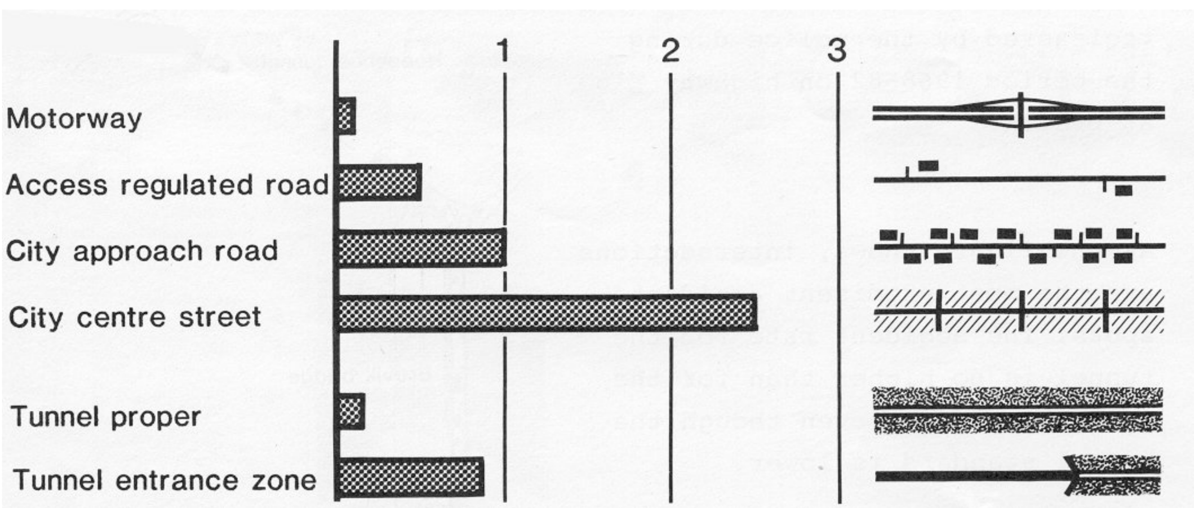


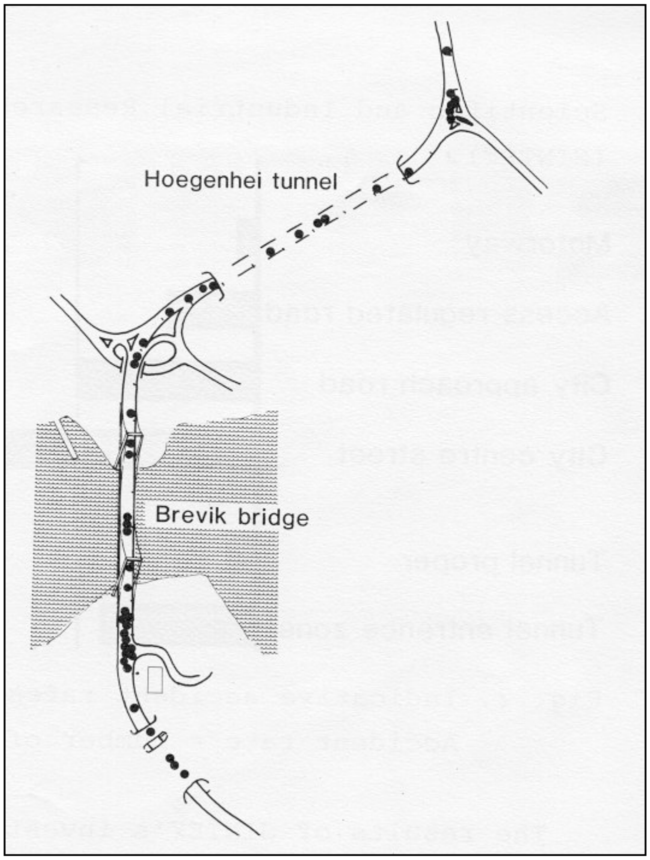
Fig. 2. Indicative accident rates for different types of road. Accident rate = number of accidents per million vehicle-km

The results of SINTEF's investigation show general agreement with international experience. The following conclusions can be drawn from the fairly extensive material:

- In general, tunnels are slightly safer than uncovered roads.
- Urban tunnels have a higher accident rate than tunnels in rural areas, but not higher than uncovered streets in towns.

- The approach zone is far more dangerous than the interior of the tunnel.
- There are wide variations in accident rates.

The comparatively high safety level found in tunnels can be partly explained by the assumption that the tunnels are of a high technical standard and that they are very well equipped. This is not true of the Norwegian tunnels included in the above study. The standards of these must be described as very modest, both in geometry and equipment. The accident history of a stretch of road on which a tunnel is an important feature is shown in Fig. 3. This section is located on the E18 highway in Telemark County. It is approximately 2.5 km long, the tunnel accounting for 0.6 km.



As the figure shows, intersections are the most prominent accident spots. The accident rate for the tunnel is no higher than for the tributary roads, even though the tunnel standard is lower.

Road users probably experience alignment and road width as the most significant factors for safety. A wide road with gentle curves is considered to be much safer than a narrow road with sharp bends.

And indeed, we do find a distinct relationship between safety levels and geometrical configurations, under conditions of more or less unrestricted speed. If the speed level is reduced to about 50 kph, the geometry of the road is of minor importance for safety.

Vehicles which accidentally run off the road at various types of bends exemplify the interrelationship between road geometry and accident rate, as shown in Fig. 4.

Fig. 3. Accidents (marked by dots registered by the police during the period 1968-82 on highway E18 at Brevik.

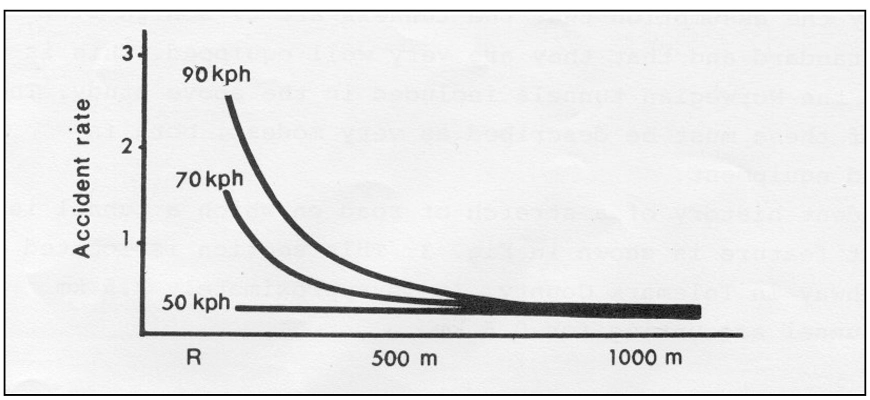


Fig. 4. Relationship between accidents due to vehicles running off the road and curvatures designed for various speeds.

5. CHOICE OF STANDARD

5.1 General

In view of the great influence it exercises on the cost level of tunnels, choice of standards deserves close attention. In many cases the adoption of a "low-cost" tunnel standard will make the tunnel a reasonable alternative in urban areas and in terrain with difficult topography.
The designation "Low-cost" tunnel can be applied to those in which:

- the choice of standard is financially and functionally balanced against the standard of the road network it serves,
- the support work is carefully matched to the purpose for which it is intended,
- requirements relating to geometry, support work and equipment have been relaxed if they have only a minor effect on safety in comparison with other measures applying to the road network.

Low-cost tunnels are not to be confused with tunnels with very low standards. The standards of the former are simply low in relation to a high class specification in which the choice of support work and equipment may perhaps rest only on marginal safety considerations. The concept also implies a willingness to accept a minor restriction of the degree of freedom of traffic flow (speed) in the tunnel in cases where this yields high economic benefits. In Scandinavia alone, some 2,000 people are killed in the traffic every year, partly because we lack the resources to construct completely safe roads. Thus it would seem inconsistent to demand a standard for some elements of the road system which can be expected to yield only marginally greater safety than a slightly lower, but considerably cheaper standard. This applies especially to tunnels. As already mentioned, the cost of tunnels varies widely with design and equipment, yet safety levels in tunnels do not vary appreciably with design. It may be claimed that the desire to keep operating expenses as low as possible justifies the choice of a high tunnel standard. But this consideration cannot justify ex pensive concrete support work in places where satisfactory stability can be achieved much more cheaply by other means. If we disregard the difference in cost of maintenance for a tunnel with and without concreting, we find with only a few exceptions that the difference in maintenance cost is insignificant compared with the difference in capital expenditure.

5.2 Width

The width of rock tunnels can have a decisive effect on the cost level. Narrow tunnels have greater stability than wide ones. In Norway it has been accepted in recent years that even main roads may be made comparatively narrow. For instance, the widest dual carriage roads have a carriage width of 7 m. Tunnel sections are adapted to this, and it is recommended that the cross section of tunnels on main roads should be sufficient to take a floor width of 9 m. If three lanes are required, a cross section giving a maximum floor width of 11 m is recommended.

5.3 Alignment

Of all components, alignment is perhaps the one that cap yield the greatest saving. A gentle alignment of both the horizontal and vertical planes can make it possible to choose a location that minimizes construction costs. The primary need in urban areas is not a high speed level, but a smooth flow of traffic. In other words, if we succeed in promoting a main arterial road with an anticipated speed limit of e.g. 50 or 60 kph, its capacity will be only marginally lower than that of a main arterial road with an anticipated speed limit of 80 kph. There will be a certain time loss between these two solutions, but this loss is insignificant compared with the difference in travelling time on a high capacity road with an anticipated speed limit of 60 kph and the present situation, where capacity breakdowns may occur during large parts of the day. If the choice lies between an alignment with a high speed level, which will take a long time to realize, and a scheme with a modest alignment standard which can be achieved in the short term, there is little doubt as to which will confer the greater benefit.

6. CONCLUSIONS

The most serious problems with regard to safety, environment and traffic flow are encountered in towns and built-up areas which lack a differentiated road network. Here large volumes of traffic are directed into roads and streets that were never intended to carry dense motor traffic. This problem can be reduced by further development of the road network. The bulk of the motor traffic can then be transferred to an adequate network of main roads, and the foundation laid for rationalization of traffic in predominantly residential and service areas.

Tunnels designed according to modest standards, and thus comparatively cheaply, can be the key to a wider extension of the road network in built-up areas. It may be argued that the tunnel standard which we define as low-cost does not provide the "best" solution for road users. Nevertheless, it provides "good" conditions, conditions which are very much better than those offered by an old road network when there is neither the political will nor the money to replace it with a high-class system.

STANDARD AND COSTS OF NORWEGIAN ROAD TUNNELS

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ABSTRACT
About 15-20 km of road tunnel are constructed annually in rock in Norway. Some of the tunnels are located in areas with heavy traffic, but most of them are to be found on sections of road with limited traffic.
Tunnel costs vary widely, depending on their design and the technical equipment employed. The Norwegian Road Specifications classify two types of tunnel cross-section for two-lane roads, and one type for single-lane roads. Tunnel standards are selected accordingly, with the aim of keeping costly concrete work to an absolute minimum. Rock support is provided primarily by means of rock bolting, and water leakage is contained by means of sheet vaulting (insulated in the frost zone). Lighting and ventilation systems may have a considerable influence on tunnel costs. The Road Specifications define two main types of illumination, and describe three different ventilation systems. However, it is unlikely that tunnels which cannot be longitudinally ventilated will be built. Very long tunnels carrying large volumes of traffic have to be divided into sections by means of a shaft or shafts. Other installations, such as fire extinguishing equipment, emergency alarm systems, emergency power plants, signal systems etc., have played a subordinate part in the context of costs. The total costs of Norwegian road tunnels in rock, depending on design, will vary from NOK 15,000 to NOK 60,000 per linear metre.

STANDARDS AND COSTS OF NORWEGIAN ROAD TUNNELS

General

In Norway we have nearly 450 rock tunnels on the main road system. About 15-20 km of tunnel is constructed each year. Some of the tunnels are located in areas of heavy traffic, but most of them are on sections of road with sparser traffic. Today nearly all road tunnels in rock are built by conventional methods, and will probably continue to be in the foreseeable future. However there is an increasing number of road tunnels in towns and villages, where complicated rock conditions and nearby built-up areas call for more unconventional constriction methods.
With the completion of the Vardø Tunnel (2890 m), subsea tunnels have become a reality. Similar solutions may be used in several places along Norway's lengthy coastline.

Road width

Width requirements have an influence on tunnelling costs. Narrow tunnels provide greater stability than wider ones. This is particularly true in bad rock or of tunnels with limited rock cover (which often applies to urban tunnels) . The Road Specifications classify two types of tunnel cross-section for two-lane roads, and one type for single-lane roads (Fig. l).

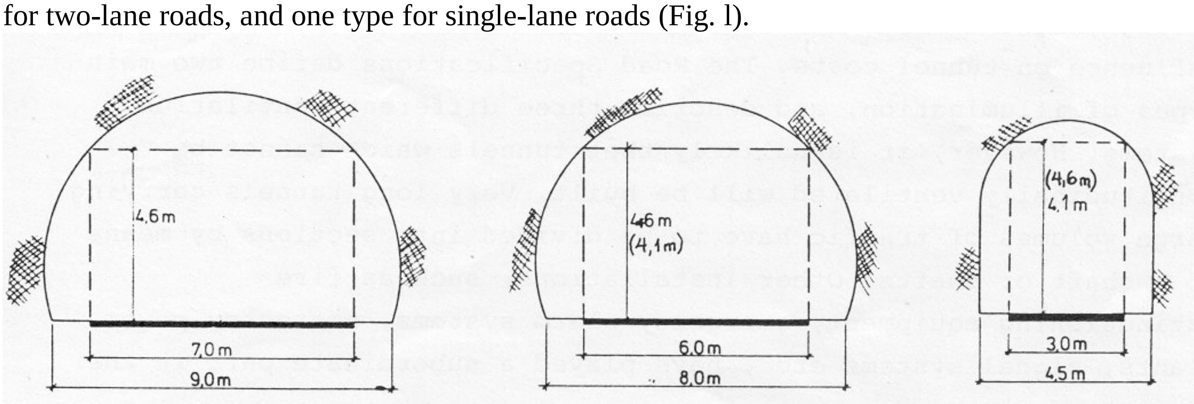

Fig. 1 Cross-section standards in the Road Specifications

Relatively narrow major roads are becoming acceptable. No two-lane roads are currently being built with roadway widths exceeding 7 m. The recommended corresponding ground-level tunnel width is 9 m. With three lanes this should be increased to 11 m, which was previously required for two-lane motorway tunnels. On roads with limited traffic, we use two lane roads with widths exceeding 6m.
On roads with very little traffic we use single-lane tunnels. The ground-level tunnel width is then reduced to 4.5 m, but there is a bay every 200 metres (Fig. 2) .

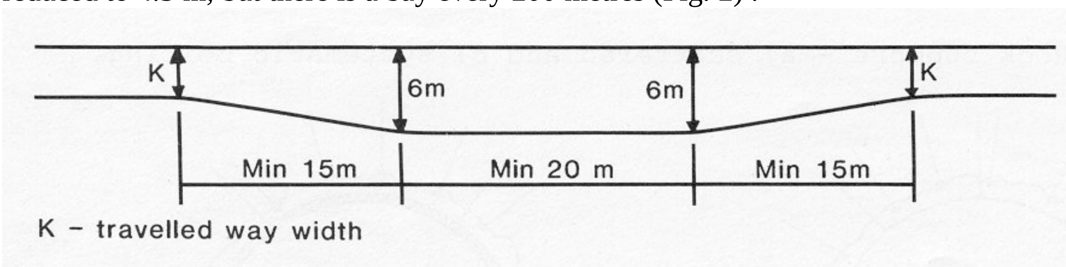

Fig. 2 Meeting area in single-lane tunnel

The major argument against narrow tunnels on heavily trafficked routes is the problem of the congestion caused by vehicles breaking down inside tunnels. Some people claim that tunnels should be sufficiently wide to allow the passage of one heavy vehicle in either direction past a broken down heavy vehicle. Such a requirement can hardly be justified in terms of either safety or economy. Heavy vehicles break down so rarely even on heavily trafficked routes that the investment required to maintain unrestricted flow under such circumstances is prohibitively high. For smaller vehicles emergency stopping bays may be justified.

Tunnel-support methods.

Tunnels standards are selected with a view to keeping costly concrete work to an absolute minimum. Rock support is provided primarily through bolting, and water leakage is contained by means of vaulting, and sometimes by application of PE-foam. Support and protective methods are illustrated in Figs. 3 through 6. The diagrams show rock support consisting of scattered and systematic bolting. Nowadays bolting is the method of support most commonly used in road tunnels. Bolting, often combined with steel straps and netting, can provide practically 100% safety against rock fall, even in heavily fractured rock. The Norwegian Public Roads Administration currently uses about 30,000 bolts annually.

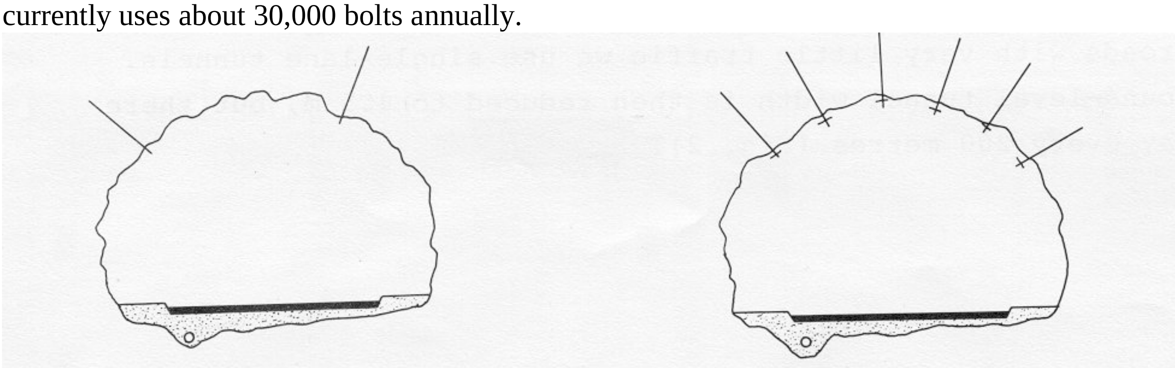

Fig. 3 Rock support – left: scattered and right: systematic bolting.

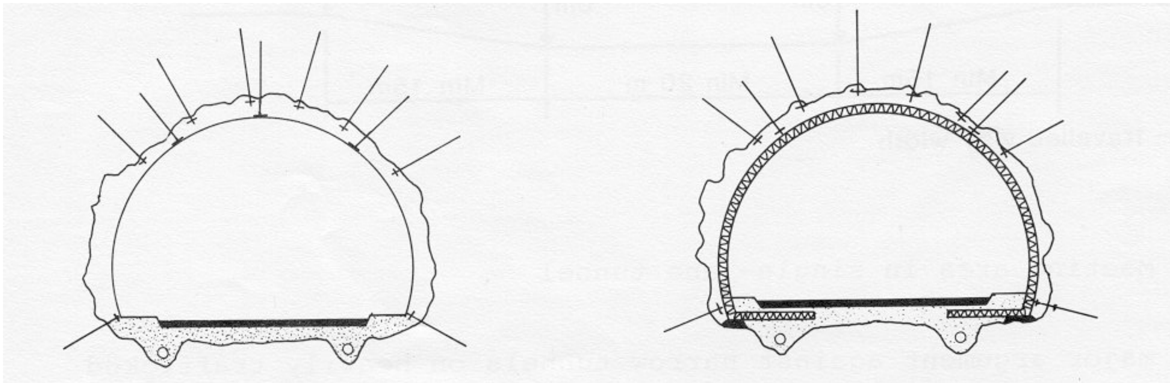

Fig. 4 Sheet vaulting used to screen off water leakage.

The diagrams show sheet vaulting being used to screen off water leakage. The vaulting is insulated in the frost zone. Note that this is only a water screen: the rock behind is supported by bolts
Sheet vaulting is the most commonly used anti-leak device. Each year, 2000- 3000 linear metres of single or double sheet vaulting are installed by the Public Roads Administration. However, PE-foam sheeting is an acceptable alternative to sheet vaulting. Being flexible, PE-foam provides a reasonably smooth exterior, even when mounted on uneven rock surfaces.

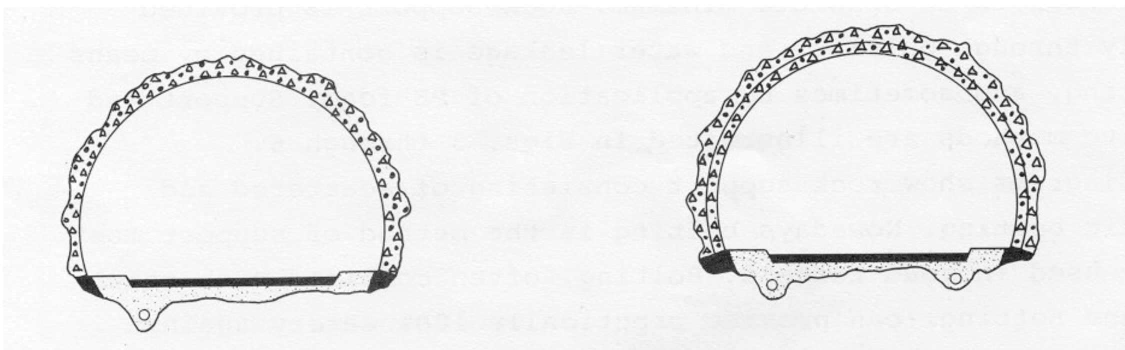

Fig. 5 Support consisting of plain concrete in dry rock and double diaphragm-insulated concrete in rock with water seepage.

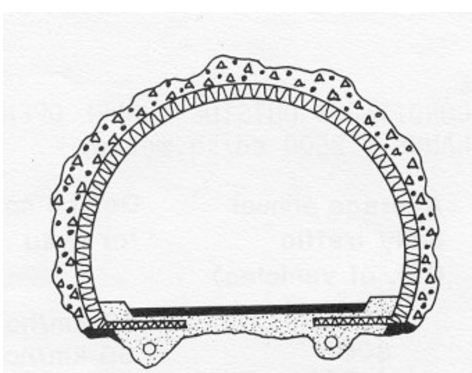

Fig. 6 Alternative to diaphragm-insulated concrete if initial concrete provides satisfactory stability.

The diagrams show support consisting of plain concrete in dry rock, and double, diaphragm-insulated concrete in rock with water seepage. These methods are very costly and should only be used for particularly difficult rock conditions. Except for around portals, the Public Roads Administration uses relatively little concrete in rock tunnels each year (less than 500 linear metres)
Large sums (NOK 10-15,000 per linear metre) can be saved if plain concrete is accepted as an alternative to diaphragm-insulated concrete. Any leaks at formwork joints are eliminated by means of PE-foam sheet grouting. If water leaks also occur at places other than the framework joints, it may be necessary to use continuous sheet vaulting (insulated in the frost zone) .
In addition to the methods sketched above, a great deal of shotcrete is used -particularly as temporary support. The method is much more expensive than bolting, so it should

preferably be used only where the rock is so poor that bolt anchorages are unsafe. Reinforced shotcrete (normal or fibre reinforcement) is also often used in combination with bolting.

Technical installations

Lighting and ventilation systems may have a considerable effect on tunnel costs. As yet, other installations such as fire extinguishing equipment, emergency warning systems and power plants, signal systems etc., have played a minor part for costs. This section therefore deals only with illumination and ventilation.

Illumination

55% of our road tunnels have illumination of sorts. The Norwegian Road Specifications classify two main types of illumination: reduced and full. Reduced illumination (35 watt sodium lamps) is used in tunnels of moderate length. A lighting system of this kind costs NOK 250-350 per linear metre (1984 prices) . A full illumination system costs much more. The average price per linear metre is greatly dependent on tunnel length (the relationship between the lengths of the approach zone/entry zone/tunnel proper) and traffic volumes. For tunnels longer than approx. 300 metres, the average price per linear metre could vary between NOK 1000 and NOK 3000. (This price includes fittings, cable trays, installation of transformer and mains connection) .

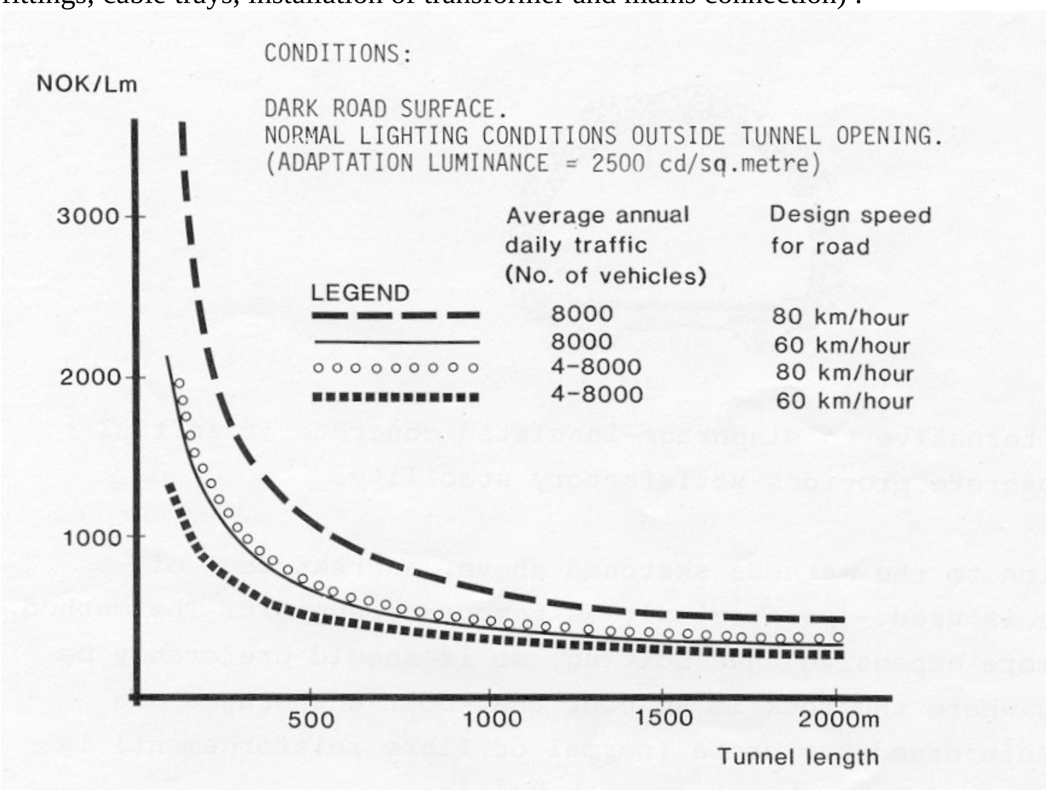

Fig. 7

Price per linear metre for lighting in relation to tunnel length (excluding transformer and power supply lines)

Ventilation

Only 5% of our tunnels are ventilated (20% of the total length of tunnels}. The Norwegian Road Specifications define 3 different ventilation systems:

Axial (or longitudinal) ventilation

Semi-lateral ventilation

Lateral ventilation

Tunnels that cannot be longitudinally ventilated are unlikely to be built in Norway. Very long tunnels carrying a large volume of traffic have to be divided into sections by means of one or more shafts. A ventilation system with artificial longitudinal ventilation will seldom cost more than NOK 1,500 per linear metre.

Total costs

Tunnel costs vary widely as a function of design and amount of technical equipment employed. Fig. 8 illustrates the varying total costs of road tunnels according to a survey of the various items of expenditure. Total costs of Norwegian road tunnels in rock will be between NOK 15,000 and NOK 60,000 per linear metre.

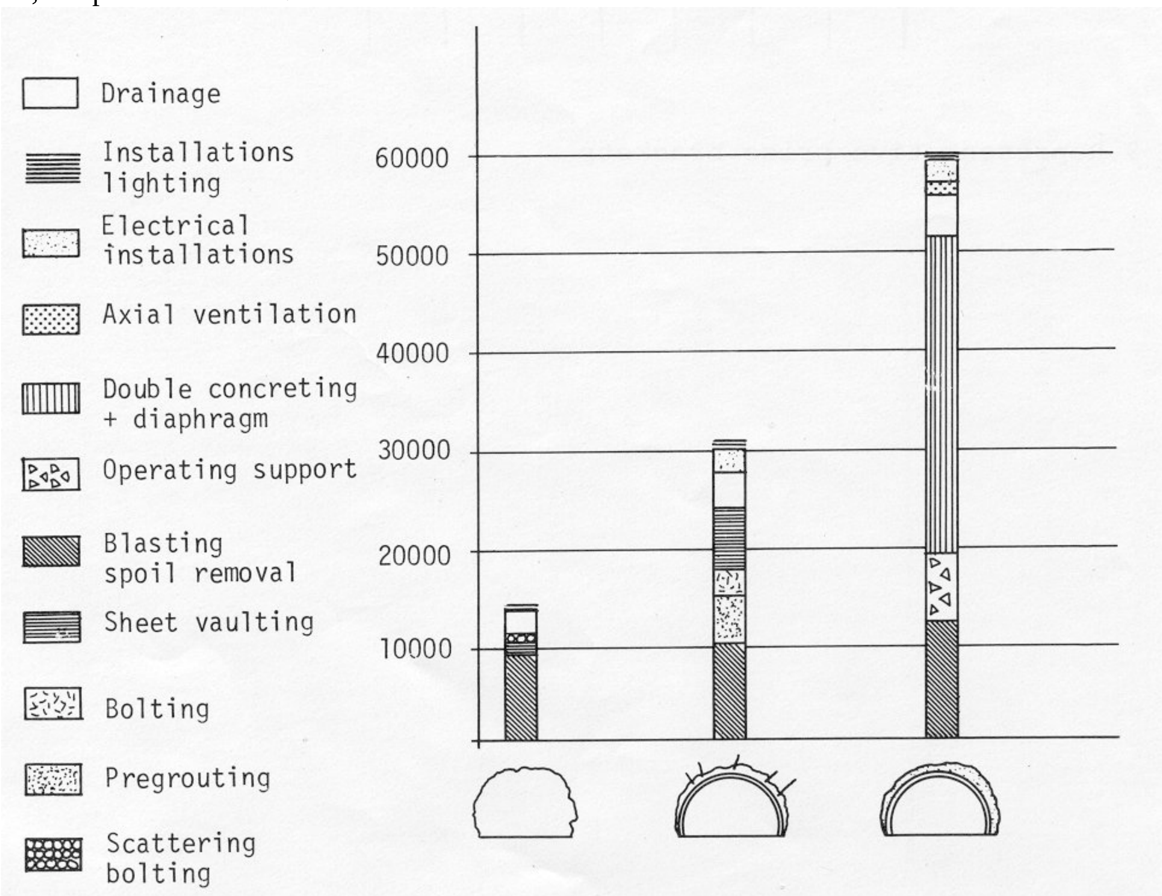

Fig. 8 Total costs of road tunnels, depending on design prices) 1984

Representative price brackets for various elements in road construction are shown in Figure 9.

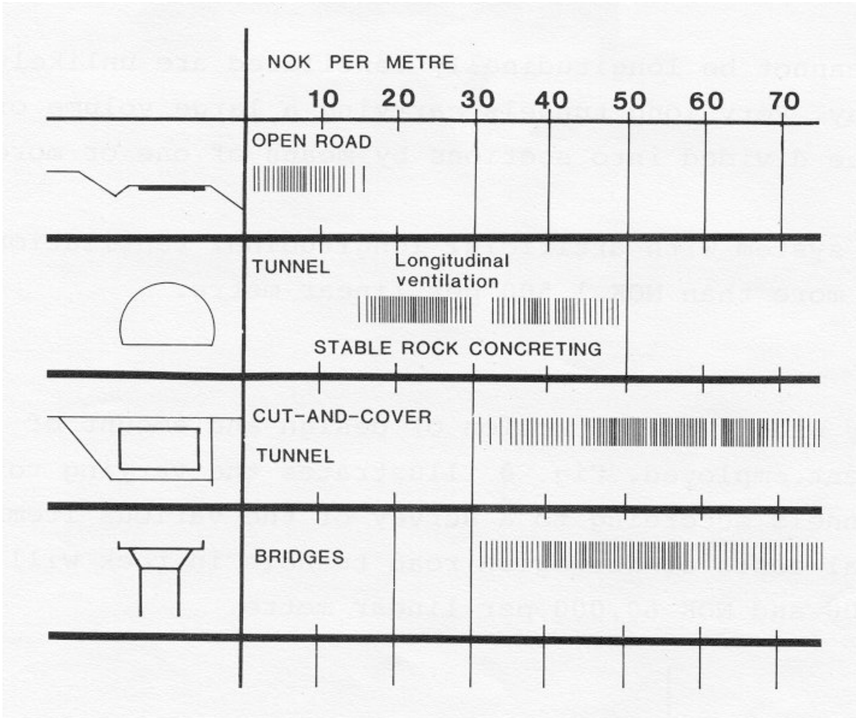

Fig. 9 Representative price brackets

MOST ACCIDENTS AT TUNNEL ENTRANCES

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ABSTRACT
Accidents associated with road tunnels seem to occur more frequently at the entrances to tunnels, despite risky driving inside tunnels. A recent study undertaken by the Foundation of Scientific and Industrial Research (SINTEF) at the Norwegian Institute of Technology (NTH) shows that 45% of all accidents occur in transition zones, while 22 per cent occurred in the middle of tunnels. The remainder took place outside tunnels. The aim of this report is to focus attention on the possible risks involved in driving through tunnels. It is based on traffic accidents recorded and technical measurements in tunnels. A regression analysis shows the relationship between several tunnel parameters and driving behaviour. Finally, some proposals for safety measures are reported.

FEW SPECIFIC STUDIES ON TUNNEL ACCIDENTS
Few specific studies have been made of traffic accidents in road tunnels. The reason for this is probably that the total number of kilometres of tunnel is low, and consequently so is the number of accidents recorded. Clear, relevant breakdowns of accident statistics are also lacking. Special analyses would therefore be required.
Our knowledge is based on a dissertation by Magne Mo at NTR:
"Tunnel Accidents Associated with Road Tunnels" (in Norwegian).
He studied 221 traffic accidents occurring over a ten-year period, from 1970 through 1979. In addition, literature on traffic accidents in road tunnels has been studied.

TUNNELS AS SAFE AS OPEN ROADS
The two most comprehensive data compilations were made in connection with the PIARC Road Congresses in Prague, 1971, and Mexico, 1975. Some interesting conclusions were drawn from the latter:

- Tunnels are as safe as open roads.
- Most accidents occur at tunnel openings.
- Good illumination influences safety.
- Accidents occur more often in curved tunnels than otherwise
- Roadway width only has a slight effect on safety.
-

The last conclusion probably applies to tunnel widths comparable to those of our new tunnels for wide two-lane roads. In a Swiss study, accidents are examined in relation to lighting. Although there is much to indicate that brighter lighting results in fewer accidents, it is difficult to substantiate this with facts. In this connection, the accidents in 36 tunnels were investigated. The tunnels had a considerably lower accident frequency than stretches of open road (0.48 versus 1.29 accidents per million vehicle-kilometres). This applied to tunnels without double carriageways. For roads with a double carriageway, tunnels and open roads had approximately the same accident frequency, about 0.30 accidents per million vehicle-kilometres.

Furthermore, it was found that accident frequency in the middle of tunnels was about one third of that near the openings. The most common types of accidents were hitting from the rear {about 35 per cent), single vehicle accidents (about 32 per cent), and overtaking accidents (about 22 per cent) Only about 10 per cent were head-on collisions. The small number of head-on collisions may be partly explained by the fact that some tunnels had double carriageways.

HITTING THE TUNNEL WALL THE MOST COMMON TYPE OF ACCIDENT

Norwegian road tunnels are typically short and seldom illuminated. When they are, it is mostly only by a row of light fixtures along the crown of the tunnels, which serve as a visual guide.
In view of the standard of Norwegian road tunnels, one would be inclined to expect a relatively high accident frequency in tunnels compared to that on stretches of open road.
The Norwegian study covered 361 tunnels on national roads. 141 were less than 100 metres long, and 32 more than 1 kilometre long. 26 of them had reduced lighting and 42 full illumination.
In addition to accidents inside the tunnels, those taking place up to 100 metres outside the openings were included. 72 (32.6 per cent) of the accidents took place outside the tunnels, 100 (45.2 per cent) in the transition zones, and 49 (22.2 per cent) inside tunnels. 51.6 per cent of the accidents were due to a single vehicle hitting the tunnel wall, 20.4 per cent to head-on collisions, and 12.7 per cent to collisions with a vehicle driving in the same direction.
The tendencies among these three types of accident were the following: hitting from the rear increased with rising AADT (Average Annual Daily Traffic) and augmenting roadway widths. The number of head-on collisions decreased with augmenting roadway widths, while there was a slight increase with rising AADT. Most accidents due to vehicles leaving the road occurred in connection with tunnels on roads with low traffic densities, whereas the proportion of such accidents on open roads rose in pace with AADT.

MORE SINGLE VEHICLE ACCIDENTS IN TUNNELS

It is the single-vehicle accidents which distinguish themselves amongst the accidents occurring on the national roads in general. The proportion of these which are connected with road tunnels is twice that on the road network as a whole. Furthermore, the accident frequency has been calculated and compared with that on national roads with and without road exits:

Road/type/tunnel zone	Accidents per million vehicle-km
Tunnel transition zone	0.53
Middle of tunnels	0.15
Tunnels (overall)	0.31
2-lane motorway	0.08
2-lane road without exits	0.15- 0.30
2-lane road with some exits	0.30- 0.50
2-lane motorway through. peripheral. urban areas	0.40- 0.70

ACCIDENT FREQUENCY IN TUNNELS AND ON OPEN ROADS

Whereas the middle of tunnels corresponds to 2-lane roads without road exits, tunnels on the whole appear to correspond to 2-lane roads with some road exits. Furthermore, the results seem to indicate that accident frequency decreases with increasing tunnel length, road width, and amount of traffic. It also appeared as though illuminated tunnels were safer than those without lighting.
Some of these factors are normally interrelated, making it difficult to determine which really influence the number of accidents. In spite of poor data, it can be concluded that tunnel entrances are three to four times more dangerous than the middle of tunnels. The middle zone seems to be safer than 2-lane roads without exits. An investigation of a number of stretches of road with many

tunnels, totalling 816 kilometres of open road and 42 kilometres of tunnel, showed that the tunnels were safer than the roads between the tunnels. The accident frequency of the tunnels was 0.29 compared with 0.43 accidents per million vehicle-kilometres for the open roads between the tunnels. On the basis of these figures, one can therefore conclude that the investigated tunnels are safer than the adjoining open roads.

POOR UTILIZATION OF ROADWAY

In this connection, the distance of vehicles from the edge line, and in some cases their speed, were used as a measure of driving behaviour. Measurements were made in Il tunnels with widths varying from 6.7 to 13.7 metres and lengths from 95 to 2770 metres.

Measurements were taken manually with an accuracy of +/- 10 centimetres by determining the distance from the edge line. These distances were recorded separately for heavy and light vehicles, and broken down according to whether or not meetings with other vehicles had taken place in the tunnels. For light vehicles, the distance from the sideline varied from 0.7 to 1.7 metres without meetings in the tunnel, and from 0.5 to 1.3 metres with meetings. For heavy vehicles the figures were 0.5 to 1.3 metres without, and 0.3 to 1.1 metres with meetings. The relationships between the distance from the edge line and different tunnel parameters were studied by means of regression analysis. The following factors seem to influence this distance:

- Increased tunnel width results in driving further away from the tunnel wall
- Higher AADT results in driving nearer the edge line
- Longer tunnels result in driving further away from the edge line
- Reduced speed limit results in driving nearer the edge line

The distance from the edge line was measured from the vehicle's right front wheel. The results show that in practice the left front wheel lies approximately on the centre line in narrow tunnels. The risk of accidents when vehicles meet is therefore great, and drivers seem to utilize the roadway poorly. To encourage drivers to drive closer to the tunnel wall, it is proposed that reflectors (RPM) be installed along the centre line. This has been tried, and seems to have the desired effect. Reflectors on the edge lines would have the opposite result, and would rapidly become soiled.

TECHNICAL MEASURES FOR MAKING ROAD TUNNELS SAFE

Safety problems with road tunnels seem to be associated with the entrances. By far the majority of accidents take place there. One reason may be that drivers need a considerably longer adjustment period when driving from daylight into darkness. Drivers coming from daylight will only with difficulty discern ice on the roadway at the tunnel entrance. This is a common phenomenon, because the road warms up faster than the tunnel. A vital part of safety work should therefore be making the road 100 metres before and after tunnel openings safe. The following measures should be adequate:

- Frequent sanding/salting increased illumination level in entrance zones
- Painting the entrance dark (to reduce the contrast)
- Improving the curvature

In the middle of tunnels, reflectors along the centre line and fixed reflectors on the tunnel walls will often provide adequate safety. It is important, of course, that reflectors are cleaned regularly, so that they always serve their purpose. Lighting providing visual guidance, or full illumination when there is heavy traffic or pedestrians in the tunnel, is to be preferred.

NORWEGIAN TUNNELLING EXPERIENCE AS A BACKGROUND TO THE LOW-COST CONCEPT OF ROAD TUNNELS

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ABSTRACT
This paper deals with current construction practice and methods in Norwegian tunnelling, and is based on a contractor's experience and viewpoints. Subjects like geological problems, risk sharing, contract models and current techniques and equipment, primarily in Norwegian tunnelling, are covered briefly. Some recently developed equipment, mainly microprocessor-assisted drilling rigs, is mentioned with references to other publications. The aim is to trace the relationship between the extensive experience gained mainly through the Norwegian hydroelectric programme, and the low-cost road tunnel concept. The author's views on finances are based on construction costs alone, and exclusive of items such as lighting and ventilation.

BACKGROUND
The part played by contractors in building road tunnels in Norway is small. Furuholmen's experience is limited to 8 projects, the most significant being the 2,700 m long undersea connection to the island of Vardø in Finnmark, which is described in other papers at this symposium.

Nevertheless, the low-cost concept for road tunnels owes much to the experience of contracting firms, gained mainly through the extensive Norwegian hydroelectric programme in which they have had by far the dominant role. This programme has been in progress since 1945, and comprises a total of about 2,500 kilometres of tunnels and more than 150 rock caverns.

The author's company, Furuholmen, has completed about 700 kilometres of tunnel and some 50 major rock caverns for various purposes in Norway, mostly in connection with hydroelectric plants.

The majority of the tunnels have been for water conduction, but there are also a considerable number of tunnel kilometres for permanent access and traffic purposes. Of these, some 12 kilometres of tunnel have been included in the public road system at a later stage. Furuholmen's background also includes tunnelling for some 10 hydroelectric plants in South America and one in Greece, and a number of unsuccessful international tenders. Only a fraction of the overall length of these tunnels, perhaps 5%, has been completely lined with concrete. The rest has been secured by means of rock anchors, shotcrete, wire mesh, ties, and to some extent pregrouting. Steel arches went out of use about 20 years ago. In this way construction costs have been significantly reduced and a term like "low-cost water tunnel" would also have been appropriate, although to the best of the author's knowledge it has not yet been introduced.

GEOLOGICAL PROBLEMS AND RISK SHARING
This short summary, and the well known fact that hard, solid rock predominates in Norway, might well lead one to suppose that problems like major faults, high rock stresses (often combined with spalling (rock burst) and squeezing), swelling clay, high groundwater pressure and infiltration rate, which are the nightmare and sometimes ruin of tunnellers around the world, would be unknown in Norway. This is far from being the case. In planning most major projects, such hazards have had to be expected, anticipated and dealt with to varying degrees. It goes without saying that thorough geological surveys and qualified interpretation of the results are important prerequisites for

designing a tunnel. It is also obvious that the most economical tunnelling is achieved if the three parties normally involved, the client, his consultants and the contractor, share the same, or at least a comparable background of experience. This is particularly the case with the low-cost concept, where so much is left to be decided on the spot during construction. With international contracts the situation, quite understandably, is different. In our tunnelling in Norway, however, we usually have common experience, and this is reflected in the way our contracts are drawn up. The basic philosophy is expressed in the following two important principles:
Firstly, contractors are not invited nor pressed to accept un-assessable risks, because an experienced client knows this may raise the bids, lead to arbitration or court, to the ruin of the contractor, or even a combination of the three. Certainly it would force all parties to spend time, energy and money in dealing with claims, instead of joining forces to solve technical problems. If the risks involved in a project cannot be carried by the clients alone, they certainly cannot be covered by the contractor's meagre profit and risk margin. Several of the international contracts we have seen were out of balance on this point.

Secondly, assuming that the contractors invited to tender are known to be capable of carrying out the job, and the contract specifications and terms drawn up by an equally experienced consultant or client cover the different operations and techniques that may be used, most of the instant on-site decisions should be left to the contractor. If the different unit prices are well balanced, which is up to the client or consultants to judge before awarding the contract, this procedure will lead to the lowest possible bids and most economical execution. The reason is that in tunnelling, as in most heavy construction work, the time-dependent elements of the costs, such as wages, capital costs and all other running expenditure, constitute the major part of the total tender.

Hence the contractor's economic result depends heavily on the speed of execution, which in turn, and particularly in tunnelling, is a question of attaining and maintaining a steady rhythm. Obviously the client, who is expected to pay the bill, should make the major decisions. Any direct interference on site, i.e. at the tunnel heading, or, worse, delaying the work by failure to make decisions, will disrupt the rhythm and result in loss for the contractor. This loss will be equal to the idle time multiplied by the total time-dependent costs, which means more than half of the total costs for the phase of the job in question. If that phase is on the critical path, the loss and claim figures will become really ugly.

CONTRACT MODEL
As a rule, Norwegian contracts are unit price contracts with lump sums for initial costs (transport, outfitting etc.) and for running costs that are largely independent of the volumes produced.

Water infiltration problems and the cost of pumping, for example, are covered by lump sums which vary stepwise with the infiltration rate measured at the end of each tunnel branch. Depending on local conditions, the combined amount for these lump sums may be from 10 to 20% of the total contract figure. Thus unit prices are reasonably suitable for dealing with variable quantities. The lump sums for running costs will in most cases be kept constant if the total unit price bill does not deviate more than 15-20% from the corresponding contract figure. If it exceeds that limit, it will be adjusted in proportion to the additional bill. The lump sums may also be subject to claims, but only if major changes beyond the contractor's control are introduced. Recent examples are number of working hours being changed by law, improved living quarters through central trade union agreements, and additional work calling for extra plant to be brought in.

It has been emphasised above that time is essential to the contractor's economy. Variations in the volume of work that has to be done immediately at the tunnel heading have a direct effect on the advance rate. This has led to the introduction of a so-called "equivalent construction time system" that still varies somewhat from one contract to the next, where the figures may be stipulated either by the consultant or filled in by the contractors as part of the bid. The following approximate coefficients, which refer to work at the heading and thus delay the drilling of a new round, will give an idea of how it is built up:

Bolting	0.25	shift for 10 bolts
Wire mesh	0.2	shift for 1 m^2
Steel ties	0.12	shift for 10 m
Shotcrete, reinforced	0.5	shift for l0 m^3
Welded wire netting	0.3	shift for l0 m^2
Rigging for shotcreting at new location	0.15	shift per location
Concrete lining (excluding floor)	8	shifts for a length of 10 m of tunnel
Concrete with a thickness in excess of 40 cm	0.2	shift for 10 m
Concrete lining behind the heading, temporarily holding up drilling	5	shifts for 10 m

Scaling in excess of l hour per blast is paid for separately and adds to the total construction time in the same way as the "equivalent shifts" arrived at from the figures indicated above. Applying these coefficients to the amounts of each type of work deviating from the contractual quantity adds up to a revised -mostly prolonged -construction period for the contractor, with an effect for penalty or premium as well as for the running cost lump sums mentioned initially. All major contracts during the last few decades have included escalation clauses, which have become more or less standardised, of the following type:

Prices of consumable materials like diesel oil, explosives, drill steel, timber, building steel and cement are dealt with by wholesale price index clauses and invoices.

The percentage of the bid representing wages, including social expenses, has partly been determined by the client, but sometimes been up to the bidders to fill in. Escalation has mainly followed indexes produced by the Associated General Contractors of Norway, based on their own statistics, and has thus had a certain self-propelling effect. It has be en maintained by clients that this system has served to divert the contractors' attention from controlling wages. They may have a point, even though the wage increases for labour in general and tunnellers in particular have really been a consequence of scarcity of qualified labour under the earlier full employment policy.

Costs of equipment and of other materials, assumed to represent a certain percentage of the bid, have largely escalated together, according to an official materials index.

The total escalated part of the contracts has varied between 80 and 90%. During recent decades, with inflation rates varying from 7 to 14% per year and contract periods of around 4 years, there is no doubt that such escalation clauses have been necessary. In periods with increasing efficiency, they have overcompensated the contractors. This fact has, however, been realised by most of them, and been taken into consideration in the last minute bid reduction aimed at winning the contract.

It may appear that the contract model described above favours the contractor at the client's expense. In a construction market where fierce competition reigns, which has been the case in Norway this will not be so. q n the contrary, it will bring out the lowest possible bids and keep the claims that still invariably accompany underground work within reasonable limits, and help to solve them amicably.

ORGANISATION OF WORK

Norwegian tunnelling for traffic purposes has so far been based entirely up on drilling and blasting. The normal routine is as follows:

At the heading, 2-3 men on each shift do the drilling, charging detonating, scaling, mucking out, placing of bolts, nets and ties, and shotcreting and pregrouting.If there is are two headings within about 4 km of each other which will normally be the case when starting out from a new adit, one 3-man gang per shift will do the same work on both faces, except scaling and mucking out, which will be dealt with by a different gang. When distances between headings grow to about 1.5 km, there will be a drilling rig at each heading.

The day-shift at the heading will consist of one mechanic, one electrician, one operator to install ventilation, piping etc., in addition to one foreman, one engineer/surveyor and if necessary, one operator on the concrete and crushing plant. Working hours in underground works in Norway have gradually been reduced to 36 per week, and the ordinary night shift was abolished in 1977. This leaves 10 ordinary shifts of 72 working hours per week. Advance rates vary from 1.5 rounds per shift, or about 60 metres per week in stable rock, down to about 20 metres per week where a full concrete lining has to be applied after blasting. When the time factor is critical, working hours can be extended up to about 100 per week by bringing in a third gang, which generally means more ex pensive tunnelling. Lodging expenses increase by about 50%, and the longer shifts tend to be less productive than the traditional 7 ½ hours.

Professional tunnellers are thus specialised, but also very versatile people, invariably paid in proportion to production. This leads to high productivity, but also to earnings up to twice the normal wages for outdoor work.

Nor is there any doubt that this payment system, traditional in Norwegian tunnelling, produces the highest possible advance rate, which has been vital to hydroelectric construction, from whence it originates. The extra kilowatt-hours gained through early completion appear to outweigh extra construction costs due to careless blasting and mistreatment of equipment. Equipment costs are mainly the contractors concern; drilling and blasting however, may be of great and immediate concern to the client, who pays for support work. The modern tunnel-drilling jumbo is a very efficient tool, but to date has lacked precision, and done far more damage to the rock surface, entailing more support work in its wake, than the old hand-held small-hole hammers. So drilling and blasting techniques still need to be improved. As far as drilling is concerned, microprocessor technology is expected to represent a major step forward.

METHODS AND EQUIPMENT

Turning to methods and equipment, Furuholmen has drilled about 30 kilometres with full face TBMs of the total of around 100 km drilled in this way in Norway so far, but these have only had smallish cross-sections, since they were destined for water conduction. TBM drilling of road tunnels is now being introduced by the Public Roads Administration. As the bottom, corners still have to be drilled and blasted out conventionally afterwards, the financial and other advantages still remain to be proved. One thing that is already clear, however, is that it is now possible for TBMs to drill through very hard rock, like gneisses and granites. Furuholmen is at present drilling 80-100 m per week in a gneissic granite with a compressive strength of about 200 MPa. The economy, however, lies entirely in time saving on a critical part of the job. Roadheader excavation is being used by Furuholmen on a hydro-electric project in Greece. This technique has also been tried in the Oslo underground system, but without success. The soft rocks which have been its domain so far, are rare in Norway. Future improvements, like aid from high-pressure water jets, might change this.

The roadheaders have also up to now lacked a guidance system for precise excavation. Such systems are, however, being developed by several manufacturers. Furuholmen, together with its British licensee, Perard Torque Tension Ltd., is installing a microprocessor- assisted automatic guidance system on a Dosco roadheader under a development contract for the British National Coal Board. Thus drilling and blasting is still the dominant excavation method, and the only one used so far in Norwegian road tunnels. For cross sections ranging from 20 m^2 right up to large caverns, the equipment and methods used are very much the same.

Drilling:

Furuholmen has standardised, 3-boom jumbos with hydraulic hammers. From about 50 m^2 on, two jumbos drill side by side. For the narrowest sections, only one of them drills, with two drilling booms. Bigger jumbos lack the versatility required by a contractor. Contractors will also quite often make special demands regarding tunnel equipment, that are not met by the manufacturing firms. This applies particularly to drilling jumbos, but also to shotcrete robots and ammonium-handling rigs. Furuholmen prefers to design and build the major part of such equipment itself.

The introduction of microprocessor guidance systems for drilling jumbos has taken longer than was anticipated w hen development started in 1972, or even w hen the first prototype was put to work by Furuholmen in 1978. There are many reasons for this: General conservatism; doubts as to what could be achieved at w hat additional equipment cost, and as to the reliability of electronic equipment in tunnels; and reduced investment in the mining industry, as well as in construction, has also affected development. The present situation is that the mining industry in general appears to be ready to go for the new technology, and some orders for guidance systems have been received by Furuholmen through its licensees in Norway and Britain, Andersens Mekaniske Verksted, Flekkefjord, and Perard Torque Tension Ltd., Notts. respectively.

The construction industry is somewhat slower, which is understandable. Whereas a mining company can plan with some confidence what to do and how to do it in the future, a contractor does not know the details and time of his next tunnel contract before it is too late to think about equipment that is not to be found either in his back yard or in the market place.
This will soon be changed by the big manufacturing firms, such as Tamrock of Finland, which has acquired the technology from Furuholmen.

For details of microprocessor-operated drilling jumbos, see the author's article in "Tunnels and Tunnelling" of May 1981. Since that article was written, new experience has been gained through use of the prototypes mentioned in the article. A real, planned test, performed exclusively for testing purposes, providing comparable results for drilling with and without the microprocessor guidance system, has however been lacking up to now. Such a test is now being planned, and should take place in a short road tunnel near Oslo around the time of this symposium.
As mentioned before, the microprocessor-based guidance system is being adapted to roadheaders to be used in the British coal mines. One would expect that such guidance systems would very soon also be found on roadheaders in traffic tunnel excavation.

Charging:
Ammonium nitrate is now used in practically all tunnels, and represents about 2/3 of our total consumption of explosives. In most cases it is mixed at the site, and it represents a considerable saving. Special equipment for mixing and charging has been developed, as mentioned previously. One of the jumbos is, however, usually fitted with a hydraulically operated basket for drilling, charging of dynamite and scaling etc.

Bolting:
Bolting at the heading is done with ordinary jumbos, where the cross section permits it. For bolting behind the heading, and in small cross sections, special bolting jumbos have been developed. Many types of bolts are used, but the common, cement-grouted 25-32 mm reinforcement bar is still preferred by us, except where immediate support is needed.

Scaling:
Scaling after each round, working from the top of the blasted rock, is still done manually, as indeed is much of the after scaling. This may seem strange, as various types of mechanical and hydraulic equipment already exist and are being used in mines and underground caverns where space is not a problem. For narrow tunnel headings, a self-propelled vehicle that could extend a 10-15 m long robot arm carrying a shotcrete nozzle, hydraulic hammer and water jets, might be the answer, but to the best of the author's knowledge it is still lacking.

Shotcreting:
Furuholmen uses only wet mix, and builds its own robots. With the concrete technology and additives now available, the quality of wet mortar is no problem. Properties like less rebound (less than 10%), overall economy and working environment all speak in its favour. hence no reason can be seen for reverting to dry mix. A steel-fibre content of about 75 kg/m) represents a very significant quality improvement, as well as being a time saver. Plastic fibres have not yet been used in Norway to the author's knowledge. However, shotcrete is covered in other papers at this symposium, 50 no more will be said about it here.

Mucking out:
The common set-up is a wheel loader, mainly the Cat 980 C, and 25-35 tonne dumpers, or 15-20 tonne bogie trucks, depending on whether it is a question of off- or on-highway transportation. The reason for the dominant position held by the wheel loader is its versatility. In sufficiently large cross-sections, there is little doubt that an excavator like Brøyt X40 or X50 loads rock at less cost, and that with an electric motor, without producing toxic gases.

FUTURE TRENDS
It seems clear that in the future tunnels will play an increasing role in road building in many countries. For a great number of fiord crossings, for example, a tunnel will now come out cheaper than the bridge alternative, and cheaper than the capitalised cost of ferry operation. Norway is one of the countries where this seems clear after the successful connection of Vardø to the mainland, described in other papers at this symposium. Such undersea tunnels may well have to dip down to 500 metres below sea level, which is the case with a tunnel now to be constructed for the purpose of carrying an oil pipeline from the North Sea to Mongstad, near Bergen. As contractors, we see no extraordinary problems in this. Lake-piercing has been carried out routinely for many years, and at depths down to 100 metres. Fjord crossings have been done by Furuholmen at depths of up to 250 m so far.

As regards new techniques and equipment, some have been mentioned above, and others may be on the way. One obvious aspect which has been receiving too little attention from contractors, is their own quality assurance and control. If sensibly adapted to the purpose in each case, this can do much to improve quality and reduce the costs of both client and contractor simultaneously.

CONSTRUCTION OF ROAD TUNNELS IN NORDLAND COUNTY, NORTHERN NORWAY, REDUCTION OF COSTS

Edward Pegg
The Norwegian Roads Authority

ABSTRACT
Since the last world war, the pattern of communications has been radically altered in Nordland. From this county, where the steam-ship used to be the vital link between centres with a concentrated population, there are now roads to most places of importance. The local geography has necessitated the construction of numerous tunnels, partly to reduce road-length and partly to reduce winter problems such as avalanches. The seventies saw a revolution in tunnelling techniques that brought about rapid drilling, better working conditions and cheaper tunnels. In Nordland, where much of the tunnelling work is done by direct labour, efforts to reduce costs even further are continuing.

INTRODUCTION
Nordland County is the long, thin part of Norway located on either side of the Arctic Circle. The E6, or Arctic Highway, as it is known, traverses the country from end to end, a distance of some 650 km.
The coast is broken by several long fjords that penetrate deep into the mountains. At several points the distance from the sea to the Swedish border is less than 20 km.
The mountains and fjords create barriers to land transport which block the natural communication corridors. Continuity can only be achieved by building ex pensive bridges, time-consuming ferries, or tunnels.
At the end of the last war, communications in Nordland were all oriented towards the sea. In fact the sea was highway no. l. Roads were built chiefly to cover the need for communication between the many small harbours and their immediate hinterlands. The first fumbling strides were taken towards trunk road construction, a few roads being built from fjord to fjord. However post war reconstruction work was well in hand before serious thought was given to a road-building strategy for transferring traffic from sea to land. The central areas of Eastern Norway were given priority during the reconstruction period, so that it was not until the sixties that road construction gathered momentum in the North. Since then, we in Nordland have never looked back.

GEOLOGY
The earliest native rock in Nordland is of Precambrian origin. In fact some of the mountains in Lofoten and Vesterålen are over 3000 million years old. The rocks here are heavily metamorphosed, and faults and weak zones yield nasty surprises to the tunneller in the form of swelling clay.
In the northern half of the county there are several areas of younger Precambrian granite and gneiss. Where these rock massifs are high, there are invariably problems with rock burst. Rock burst can also occur in these areas as a result of tectonic stresses or valley-side stresses.
During the Caledonian period Nordland came under great pressure from Greenland. This resulted in a new generation of gneisses, large areas of mica schists and veins of limestone. The latter can give rise to leakage and drainage problems. Younger rock is virtually non-existent in Nordland. The hard rock of Nordland is, in fact, most conducive to low-cost road tunnelling, but conceals enough problems -rock burst, swelling clay and drainage -to make life interesting for the tunneller.

HISTORY

Road tunnelling in Nordland began after the war with a few short tunnels driven by hand-held rock drills with pneumatic feeds. The tunnels were driven sporadically, often as a measure against unemployment in the difficult winter months.

When the work volume increased in the early sixties, the same methods were used. A typical road tunnel, with a cross-section of 45 square metres, would be driven by two or three 8-hour shifts of 7 men + 1 mechanic, at the rate of 3 m/shift, i.e. a steady rhythm of one round per shift. Muck shifting was carried out by small tractor-dumpers with a capacity of 5 m^3.

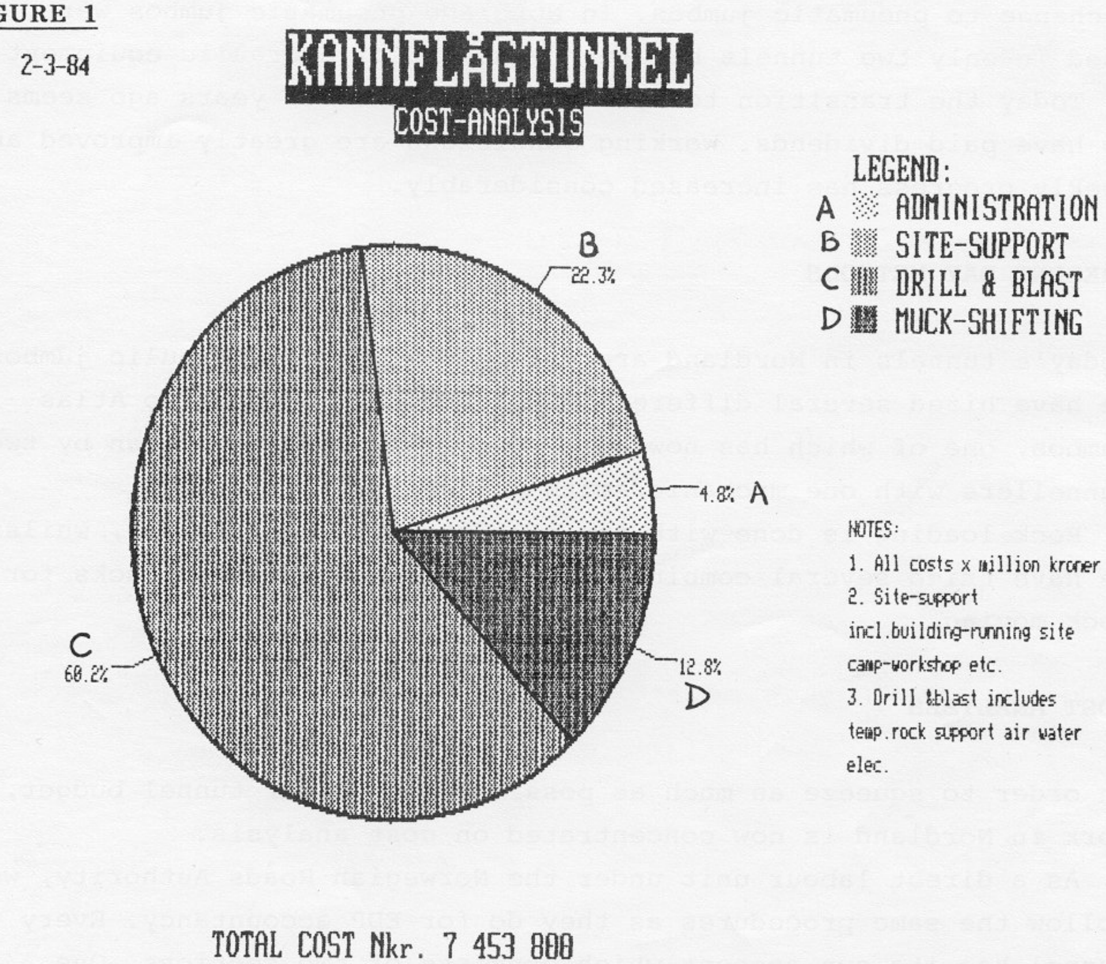
These methods continued to be used far into the 19705, but a need to improve working conditions and increase tempo brought about a change to pneumatic jumbos. In Nordland pneumatic jumbos were used in only two tunnels before the advent of hydraulic equipment.

Today the transition to hydraulic jumbos seven years ago seems to have paid dividends. Working conditions are greatly improved and weekly progress has increased considerably.

PRESENT DAY METHODS

Today's tunnels in Nordland are drilled by 3-boom hydraulic jumbos We have hired several different makes and have bought two Atlas jumbos, one of which has now been sold. The jumbo is driven by two tunnellers with one mechanic/smith per shift. Rock loading is done with our own Cat 980C wheel loader, whilst we have tried several combinations of lorries and dump trucks for rock moving.

FIGURE 1



COST ANALYSES

In order to squeeze as much as possible out of our tunnel budget, work in Nordland is now concentrated on cost analysis. As a direct labour unit under the Norwegian Roads Authority, we follow the same procedures as they do for EDP accountancy. Every tunnel has its own account which consists of two sections. One section shows how money is used on wages, machine hire and consumables, the other shows how much money is used in a particular work process, e.g. drilling and blasting, rock support, muck shifting, administration etc. We decide locally how detailed the analysis of work procedures shall be. In addition we have a more detailed study of commodity consumption, especially explosives, drill bits and steel.

The cost analysis of Kannflåg Tunnel will serve as an illustration of our methods of cost analysis for raw tunnels (i.e. tunnels without any permanent rock support or any form of permanent installation). The Kannflåg Tunnel is situated on the new E6 round Leirfjord, which will eliminate the penultimate ferry on the Arctic Highway. The tunnel is 755 m long, and was holed through in October 1983. The first diagram, Fig. l, shows how tunnel costs are divided into their individual components. Three of the components depend on tunnel length -administration, drilling and blasting and muck shifting. The fourth factor, called site support, is more complex. It includes the investment necessary for building and maintaining temporary roads, building power lines, transporting equipment to the site, transporting and housing workers, and safety equipment.

This cost component depends on site accessibility and the total time taken to do the job.

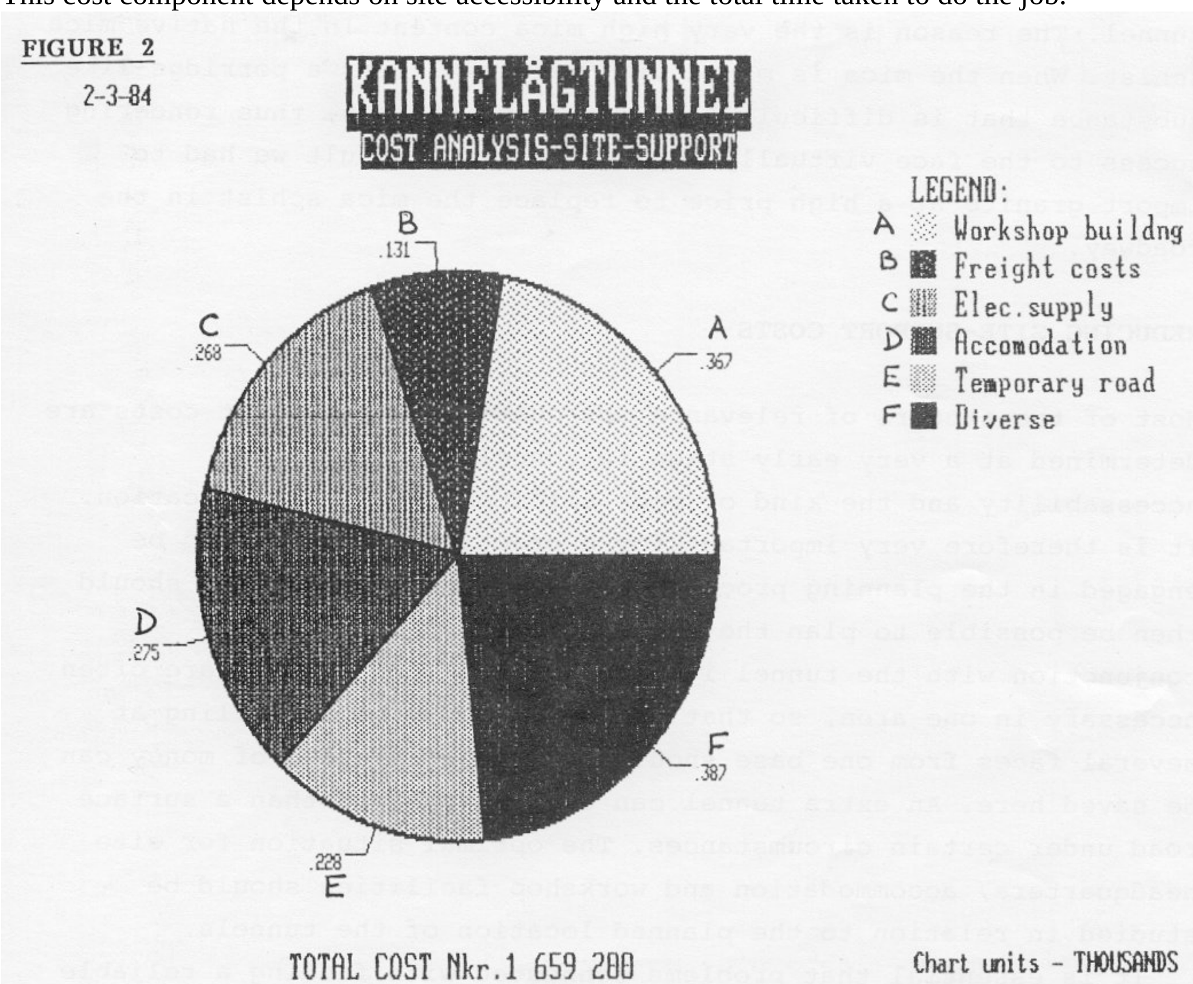


Fig. 2. gives a further breakdown of site support costs. In the case under discussion the most important single factor is the construction of the site workshop, sanitary facilities and the expansion of an existing site camp.

Norwegian legislation sets very strict regulations for the standard of temporary installations. For example, a workshop to be used for more than six months has to have a concrete floor.
Freight costs reflect, to some extent, the remoteness of the site and the distance to the previous site from which most of the equipment has been fetched. The item "Electricity supply" includes the building of power lines and the installation of the necessary transformers. This can be a very expensive item: the cost of a 22 kV power line is about NOK 150,000 per km -but, ironically, when this has been paid, the line remains the property of the local electricity board.
The item for accommodation shows w hat it costs to have a site camp in existence for 7 months. It includes wages for 2 cooks and a cleaner, rent for the camp buildings, electricity, soap etc. etc.
A special feature of Kannflåg is the high cost of temporary roads. In this case the road in question is the haul road in the tunnel. The reason is the very high mica content in the native mica schist. When the mica is mixed with water it forms a porridge-like substance that is difficult to drive on or through, thus rendering access to the face virtually impossible. As a result we had to import granite at a high price to replace the mica schist in the roadway.

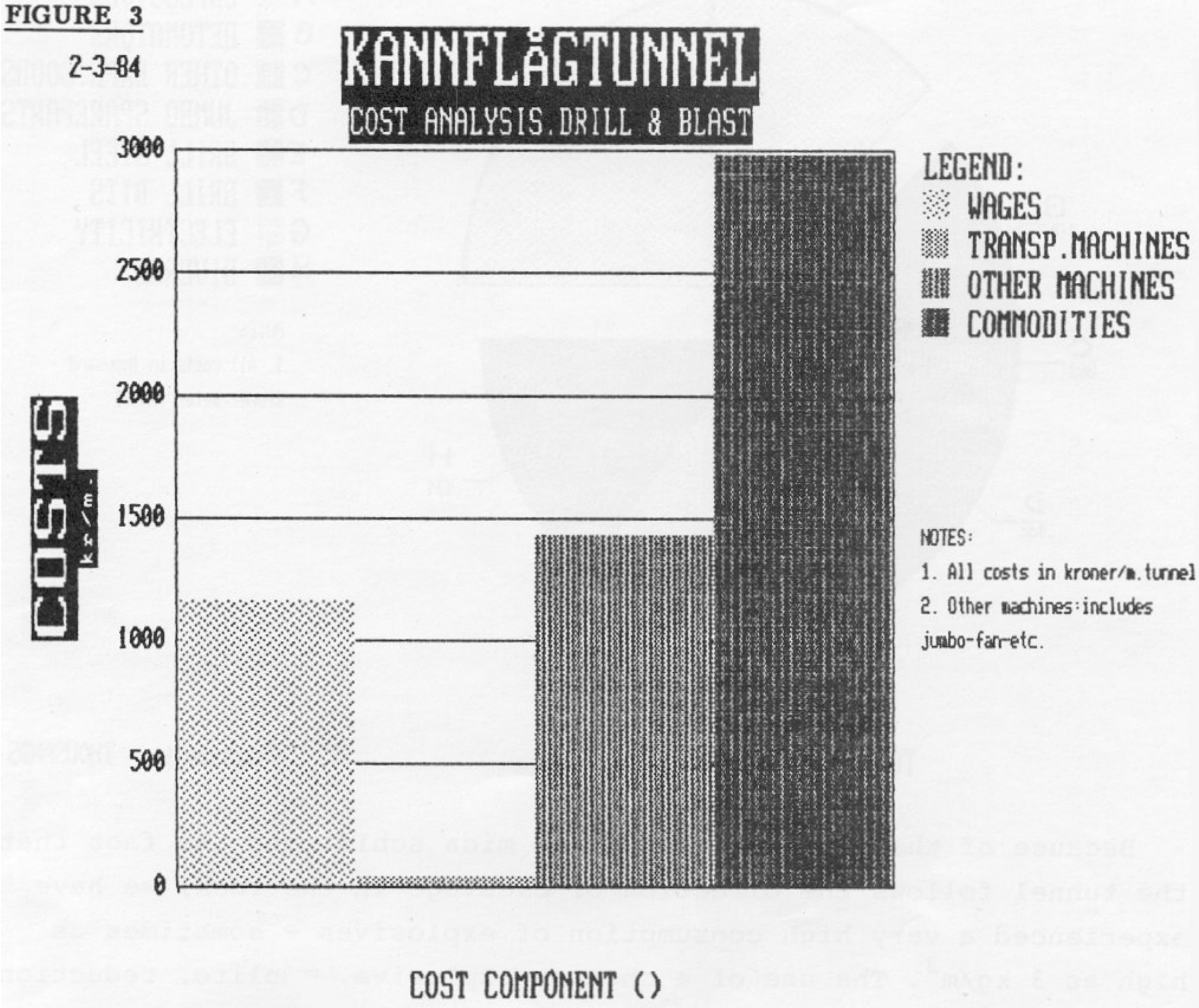
REDUCING SITE-SUPPORT COSTS

Most of the factors of relevance for cutting site-support costs are determined at a very early stage in tunnel planning. Site accessibility and the kind of rock all depend on tunnel location.
It is therefore very important for a practical tunneller to be engaged in the planning procedure as early as possible. It should then be possible to plan the necessary site facilities in conjunction with the tunnel itself. Several road tunnels are often necessary in one area, so that the possibility of tunnelling at several faces from one base should be studied -a lot of money can be saved here. An extra tunnel can even be cheaper than a surface road under certain circumstances. The optimal situation for site headquarters, accommodation and workshop facilities should be studied in relation to the planned location of the tunnels. It is essential that problems connected with finding a reliable water supply and the building of power lines contra the use of a generator are solved in the planning phase. The most important cost-saving factor is re-use of equipment. Most of our site workshops are either of plastic or aluminium, and can easily be moved from site to site and used several times. Much time can be saved in rigging a site if the container method is used. Investment may be somewhat higher, but it is more than compensated for by the amount of time saved in rigging the site. It is of the utmost importance to know w hat equipment is available, and where to find it. During the start phase of a tunnel, everything is geared towards getting rock over ones head as soon as possible. This may be detrimental to the establishment of the site base, which may be left half finished until tunnelling is under way. Our experience is that it is essential to have a good plan of what has to be done before tunnelling starts, and a strong leader team to implement the plan.
If necessary, delay the start of tunnelling work to make sure that site support functions correctly. A well established site base pays dividends throughout the tunnelling phase.

DRILLING AND BLASTING

In Fig. 3, the most important process in tunnelling is broken down into its constituent cost components. It can be seen that wages account for about 21% of the total, transportation for under 1%, tunnelling equipment for 25%, and consumables for 53%. Consumables cost the same per metre of tunnel virtually regardless of tunnel length. Wages and machine costs are time-dependent. The cost will depend on how effectively they are utilized. Fig. 4 gives a further breakdown of the costs of materials. By far the most important item used is explosives -together with the detonating system it accounts for over 50% of all the materials and equipment that are bought for tunnelling. Because of this the use of explosives is followed up separately. An ideal curve for explosive consumption shows that we should use somewhere between 2.3- 2.5 kg explosive per m^3 of rock. Because of the tough nature of the mica schist and the fact that the tunnel follows the direction of cleavage in the rock, we have experienced a very high consumption of explosives -sometimes as

FIGURE 3

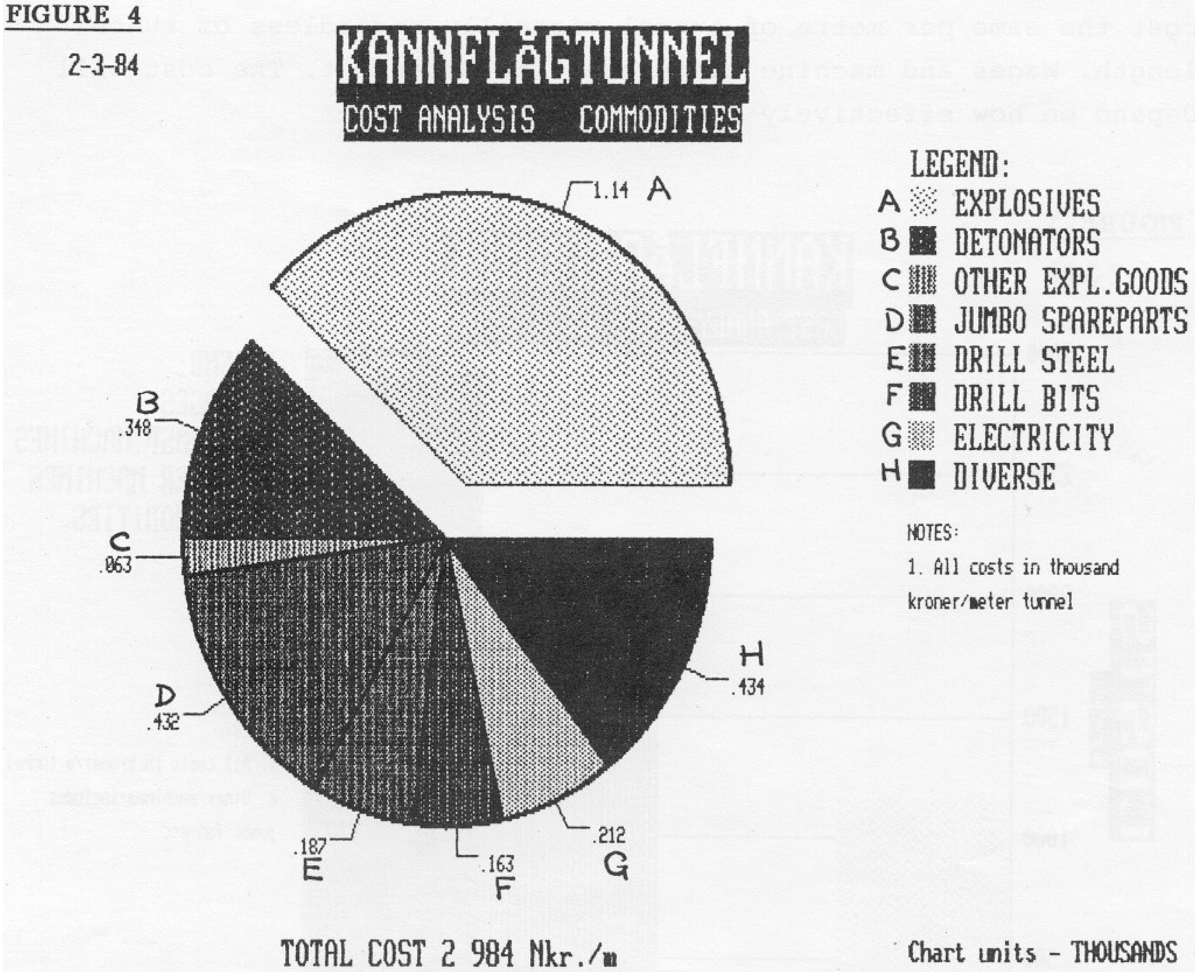


high as 3 kg/m^3. The use of a cheaper explosive, anolite, reduction of the number of holes, and strict control with the filling of the holes are the factors that have to be paid close attention in order to reduce costs.

Spare parts for a jumbo are no cheap matter. Prices vary from NOK 2-4 per metre of drill hole, depending on machine type and age. Correct use of equipment and a capable mechanic are the best guarantee for low costs, but attention must be paid to the proper stocking of spare parts. In recent years it has become more usual to negotiate a guaranteed price ceiling for spare parts. This invariably cuts costs and ensures better service from the manufacturer. The same applies to drill steel and rock bits. A guaranteed price ceiling guarantees not only price but the manufacturer's interest. Electricity in Norway is cheap by world standards, but a monthly bill of NOK 25,000 is enough. The local electricity boards have a monopoly and dictate prices. The only possibility for saving a few kroner is to switch off the ventilation fans at the end of the day.

There remain various miscellaneous costs. These include ventilation equipment, cables, piping and the thousand and one small things that are essential for tunnelling. Though many of them are consumables, there still remains a good deal of equipment that can be passed on from tunnel to tunnel. Wages account for 21% of tunnelling costs. The average wages of a tunneller are about NOK 100 per hour, and in addition the employer has to pay 43% to social security. The prime saver is a bonus system which sets a target of a-s high production as possible without undermining safety The size of the work force should be dimensioned to gain optimal results. A small change in crew

FIGURE 4



size - either adding or taking away a man from the face crew –frequently results in much faster advance or more efficient production. Machines account for 26% of the total costs. All the Roads Authority's machines are owned by a machine holding group that hire the machines out to the sites. Investments, taxes, maintenance, capital and interest costs are calculated into the hire cost.

The largest single machine we hire is the jumbo. In Kannflåg we have been debited with a monthly hire rate of over NOK 170,000, a price that is a long way above normal. A more realistic price is NOK 75,000 per metre of tunnel. In addition to the jumbo we have used a wheel loader with a platform in the shovel, to assist in installing ventilation and in the daily scaling of the tunnel.

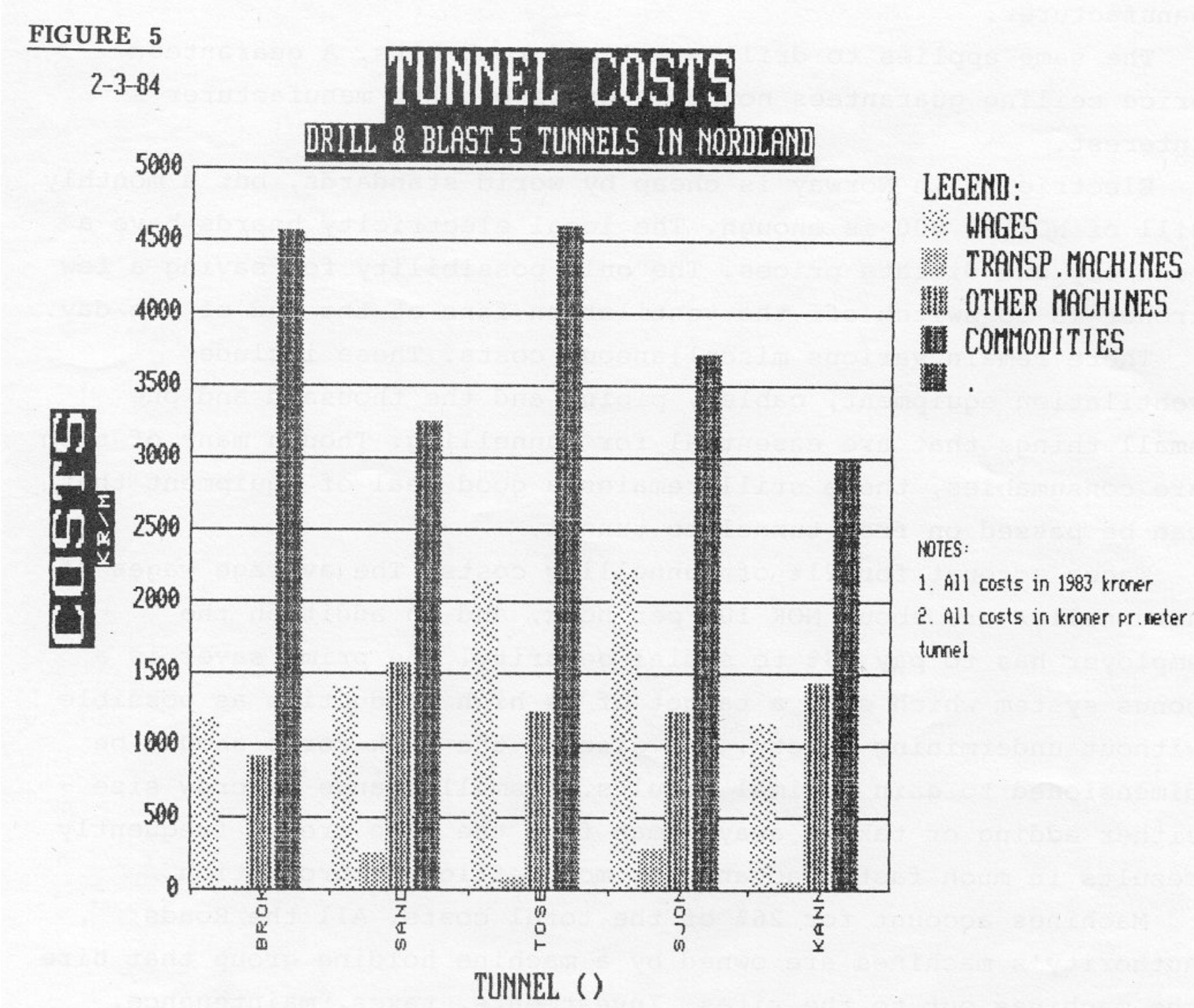
In addition we have a quantity of smaller items of equipment - pumps, fans, hand-held rock drills etc. etc. that we have to pay rent for.

The monthly cost of machines appears to be the same however much or little is produced. Cost saving must therefore depend on increasing the rate of tunnelling.

Fig. 5, a comparison of drilling and blasting costs from 5 different tunnel sites in Nordland, shows that there has been a substantial saving made in the Kannflåg Tunnel. This has been achieved by reductions in the consumables sector, and reflects the fact that equipment bought new for the first "hydraulic" tunnels is now being re-used, greatly lowering costs. There are still other areas where savings can be made, and this is now receiving our attention.

MUCK SHIFTING

In all the tunnels shown in Fig. 6 we have used a Cat 980C wheel loader for rock loading. It is our experience that this machine is the best suited for loading in tunnels of about 45 m^2, in terms of



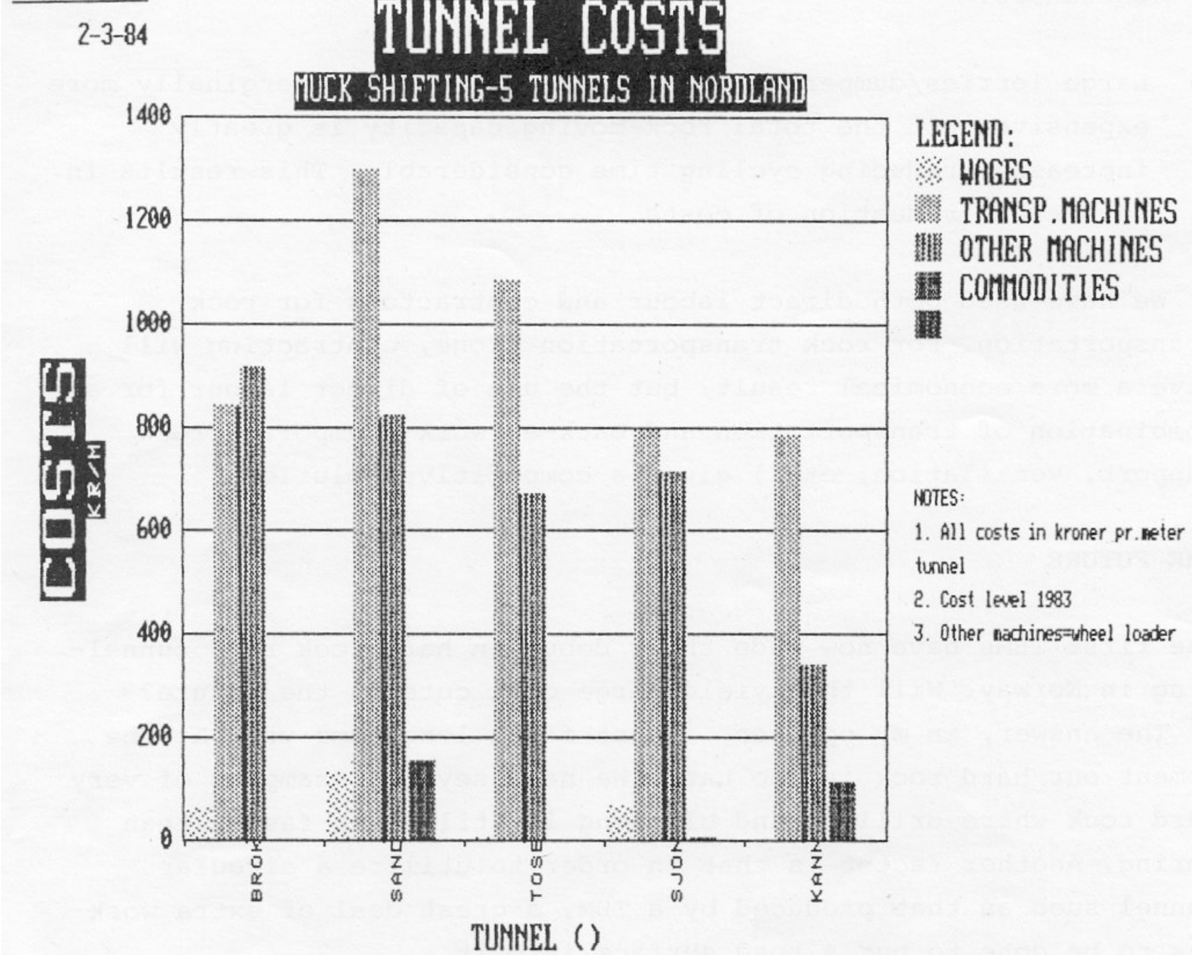
both working conditions and machine capacity. If conditions permit, we fill tunnel rock directly onto the road. Otherwise we have to use a tip, which increases muck-shifting costs. As a tipping machine we use a smaller wheel loader that can be used as a reserve loader in the tunnel in case of breakdown. Faster tunnel advance is the only sure way of reducing tunnel costs during loading. We tried several combinations of lorries and dump trucks for rock moving.

l) Ordinary lorries -which are cheap, but have a low overall capacity because of congestion in the loading area.

2) Articulated dumpers, e.g. Volvo 860- cheap, but slow over long distances.

3) Large lorries/dumpers e.g. Merc. 3232, Kockum -marginally more expensive, but the total rock-moving capacity is greatly increased, reducing cycling time considerably. This results in an overall reduction of costs. We have used both direct labour and contractors for rock transportation. For rock transportation alone, contracting will give a more economical result, but the use of direct labour for a combination of transportation and back-up work (temporary rock support, ventilation, etc.) gives a competitive solution.

FIGURE 6



THE FUTURE

The first TBMs have now made their debut in hard-rock road tunnelling in Norway. Will they yield large cost cuts in the future? The answer, in my opinion, is not for a long time yet. At the moment our hard rock is too hard. We have several examples of very hard rock where drilling and blasting is still much faster than boring. Another factor is that in order to utilize a circular tunnel such as that produced by a TBM, a great deal of extra work has to be done to put a road surface into it. Drilling and blasting still has a great future in Norway. There is still room for saving and faster advance.

ROCK-BURST PROBLEMS IN ROAD TUNNELS

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ABSTRACT
During the last decade a lot of long, deep-lying road tunnels have been driven through the mountain ranges between the Norwegian fjords. This has in many cases created rock-burst problems due to horizontal rock stresses. Other tunnels often have rock-burst problems due to gravitational "valley side" stresses. In many of the tunnels, stress measurements have been made.
Approximately horizontal rock stresses, measured along the major principal stress direction, are 25-35 MPa. In most cases the rock masses consist of different types of gneiss, with lenses of other rock types. The majority of the tunnels are stabilized by extensive use of rock bolts.
In the 7.5 km Høyanger Tunnel, 80,000 rockbolts were installed to prevent spalling. Most of the rockbolts were inserted close behind the face after each blasting round, which reduces tunnelling progress. After breakthrough supplementary rockbolts and wire mesh were installed.
At the sites of two tunnels, one of which is under construction, shotcrete reinforced with steel fibre has been used in combination with rockbolts. This method has given somewhat greater driving rates.
The cost of the two methods seems to be rather similar. The total cost per metre of tunnel is NOK 13-18,000 (1982) when there is intensive spalling.
The long-term effect of the rock stresses acting on the fibercrete is under observation.

INTRODUCTION
The rough Norwegian topography, with fjords and mountains, has led to the construction of a large number of road tunnels. Traditionally, most of the tunnels have been driven along the fjords and mountainsides, which are often very steep and high (1000-1500 m.a.s.l.). This has often created lateral gravitational "valley side" rock stresses with rock-burst and spalling from the tunnel wall facing the fjord or the bottom of the valley. During the last decade it has become common to construct road tunnels straight through the mountain ranges between fjords and valleys, instead of going around with a series of short tunnels. This has resulted in longer, more deep-lying road tunnels than previously. The tunnels are mainly driven through hard rock, and are usually unlined, except in occasional zones of weakness where some kind of lining has been used.
Many of the tunnels have been subject to rock-burst problems due to high horizontal rock stresses. Most of the tunnels are situated in Western Norway in Precambrian gneisses, with a great depth of rock cover (Fig. l).

STRESS MEASUREMENTS
When the Oppljos Tunnel was driven, rock-burst problems were really not expected, because of the moderate overburden. However, near the middle of the tunnel, where the overburden was small, extensive spalling occurred in the crown. Stress measurements were made, and confirmed the presence of high horizontal stresses, nearly 20 times as high as the theoretical value (Table l).
Later, w hen the Høyanger Tunnel scheme was started, any horizontal stresses of the same order as those in the Oppljos Tunnel (1 = 20.4 MPa) were expected to be compensated for, or exceeded by the thick overburden (900 to 1200 m) through most of the length of the tunnel, to give an approximately isotropic stress field. Gravitationally induced "valley side" stresses were expected to give rock-burst problems near the ends of the tunnels. Rock-burst problems were estimated to extend for a total distance of 3 km.

A few hundred metres from the entrances, extensive spalling and rock-burst occurred in the crown and face. Triaxial stress measurements were made in situ at three different sites in the tunnel (Table l). The major principal stress was approximately horizontal and perpendicular to the tunnel axis /2/.

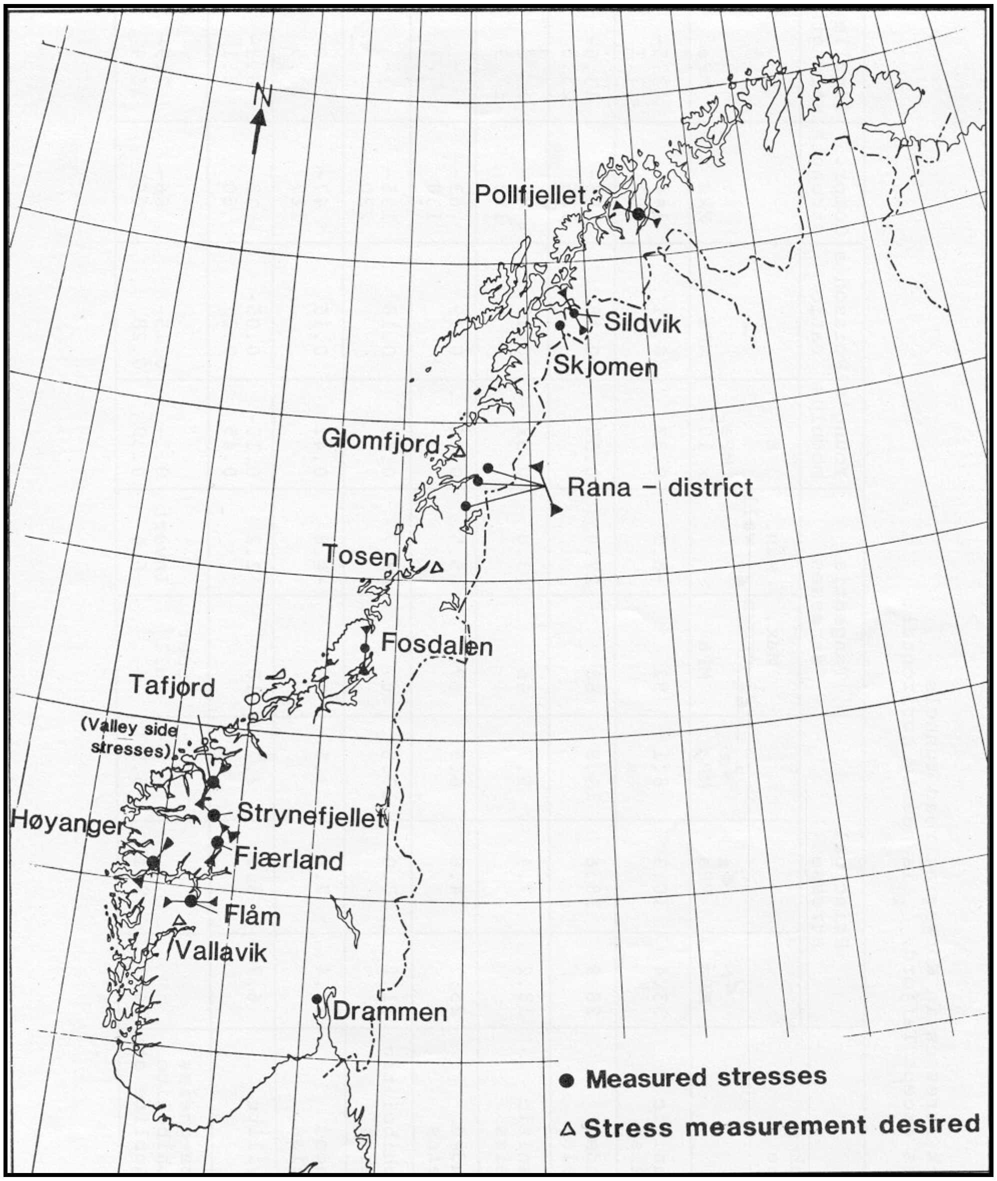


Fig. l: Stress measurements in Norway

The highest major principal stress was about four times as great as the vertical minor principal stress at the same place. The vertical stress was about half of the theoretical value as calculated with reference to the overlying rock masses. The two other measurements verified the high horizontal stress perpendicular to the tunnel axis (Fig. 2).

The calculated maximum tangential stress in the crown was 92 MPa while the side wall was under tension -9 MPa. The compressive strength varies from 55 to over 200 MPa according to laboratory tests. Due to fissures in the tunnel, the real strength is considerably less. This explains the heavy spalling and rock-burst in the crown during tunnel driving until breakthrough (Fig. 3) .

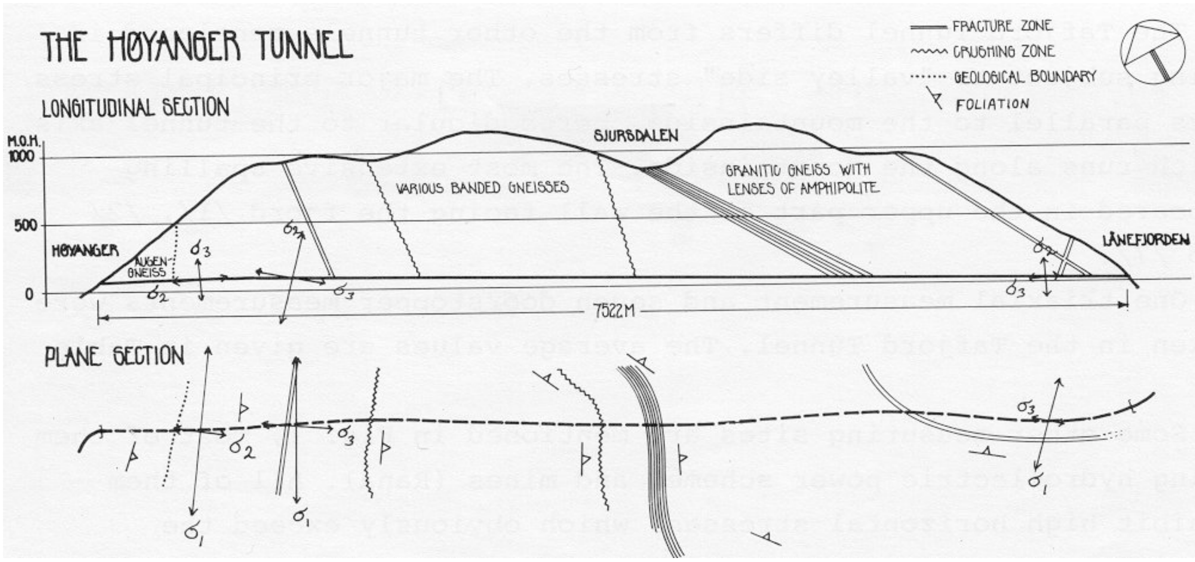


Fig. 2 Longitudal profile of the Høyanger tunnel.

Similar conditions, though less extensive, were observed and verified by means of stress measurements in the Fjærland and Flåm Tunnels in the same district. In the Pollfjellet Tunnel Scheme in Northern Norway, measurements were taken before blasting, to test the stress conditions at the bottom of the approximately 1200 m high Poll Mountain. The measurements, which were taken in a roughly 20 m deep borehole, revealed the major principal stress to be horizontal. When the tunnel was driven, there was moderate to no spalling due to the nearly isotropic stress pattern. The Tafjord Tunnel differs from the other tunnels mentioned in being subject to "valley side" stresses. The major principal stress acts parallel to the mountainside, perpendicular to the tunnel axis which runs along the mountainside. The most extensive spalling appeared in the upper part of the wall facing the fjord /l/, /2/ and /7/.

One triaxial measurement and seven doorstopper measurements were taken in the Tafjord Tunnel. The average values are given in Table l.

Some other measuring sites are mentioned in Fig. l, most of them being hydroelectric power schemes and mines (Rana). All of them exhibit high horizontal stresses, which obviously exceed the vertical stresses. The stress measurements were carried out by Dr. Eng. A. Myrvang
at the Norwegian Institute of Technology.

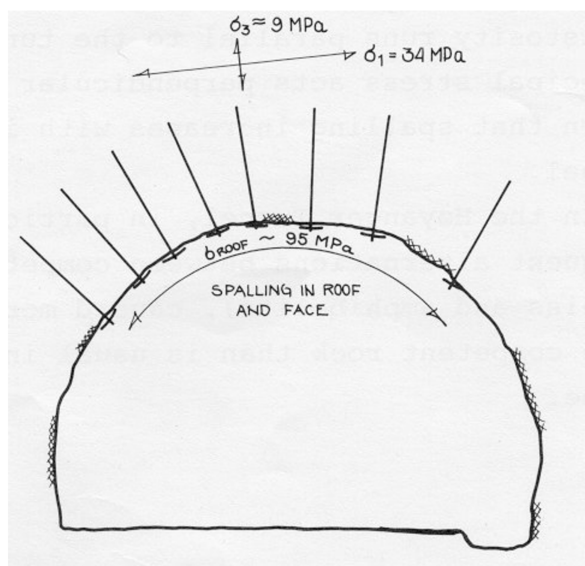


Fig. 3. Cross section of the Høyanger Tunnel

Table1: Measured rock stresses in Norwegian road tunnels
For all sites except Tafjord, l is nearly horizontal.

Measuring site/ Tunnel	Rock type	Principal stresses $\sigma 1$ MPa	$\sigma 2$ MPa	$\sigma 3$ MPa	Tangential stresses σt Max crown MPa	σt Min. wall MPa	Young's modulus E. MPa·10^5	Poisson's ratio MPa	Comparative strenght MPa	Tensile strenght MPa
Høyanger I	Granittic gneiss	33,4	10,3	8,1	92	- 8,9	0,32	0,11	144 - 207	13,5- 17,1
Høyanger II	Banded gneiss	28,9	18,6	13,8	68	27,0	0,58	0,18	55 - 126	13,6 - 23,6
Høyanger III	Granittic gneiss	18,2	7,4	5,7	48	3,8	0,32	0,12	89 - 152	13,8- 17,9
Fjærland	Banded gneiss	25,7	14,6	6,5	67	5,5	0,46	0,19	63- 138	-----
Flåm	Ampi-bolitic gneiss	31,2	27,0	17,3	67	49,7	0,48	0,18	135 - 220	-----
Oppljos (Strynefjellet)	Banded gneiss	20,4	7,6	3,5	56	- 4,8	0,42	0,15	47 - 127	5,0 - 12,5
Pollfjellet (meas. from. surf)	Phyllite	6,7	3,3	2,7	18	2,2	0,15 - 0,45	0,05 - 0,2	32 - 60	6,3 - 8,1
Tafjord	Mica-gneiss Ampibolitic granules etc..	24,3	9,1	6,5	70 (upper wall)	- 6,4 (invert)	0,1 - 0,39	0,15 - 0,28	66 - 206	2,9 - 16,4

MAIN FACTORS INFLUENCING ROCK BURST

In recent years definite relationships have been observed between rock-burst problems in tunnels under construction, and a range of rock properties and other factors.

Stress measurements have indicated that anisotropy and tangential stress, rather than the magnitude of the major principal stress, determine rock-burst intensity.

Structural features, like schistosity and jointing, and the orientation of the structures, are of very great importance for spalling and rock-burst problems in general. The compressive strength of the rock mass in situ is found from experience to be greatly reduced when the schistosity runs parallel to the stress faces. Spalling is observed to increase dramatically when the schistosity runs parallel to the tunnel surface, while the major principal stress acts perpendicular to the tunnel axis. It is well known that spalling increases with increasing cross-section of the tunnel.

In the Høyanger Tunnel, in particular, it was noticed that frequent alternations between competent and incompetent rock types (gneiss and amphibolite), caused more extensive spalling in the more competent rock than is usual in the homogeneous competent rock alone.

When the tunnel is driven towards a zone of weakness which dips in the opposite direction from the line of advance of the rounds, spalling has often ceased for some rounds. After crossing the zone of weakness, spalling has returned with great intensity. Sometimes, however, spalling behaves in quite the opposite manner when crossing a zone of weakness.

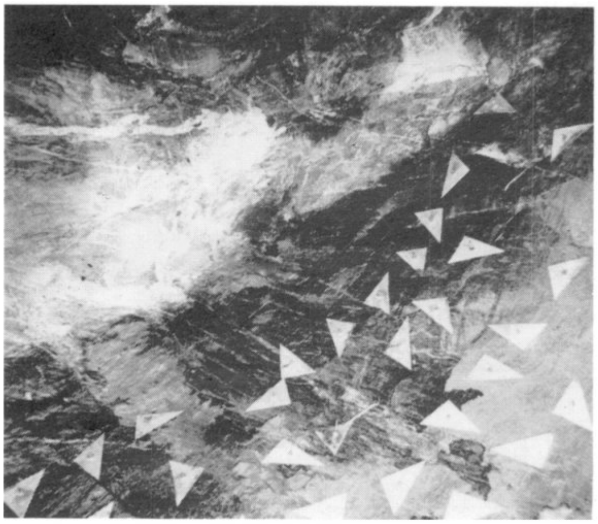
The zones of weakness may evidently give the rock local relief from the general stress field.

The evenness of the tunnel contour is of the utmost important to spalling intensity and the quantity of overbreak due to spalling. Hence there is much to be gained from drilling the perimeter holes with great accuracy.

WORKING PROCEDURE AND DESIGN OF ROCK SUPPORT AND LINING

Rock-burst problems have been well known to Norwegian tunnellers for many decades. The traditional remedial action has been extensive, systematic rock bolting, and sometimes blasting of an asymmetric cross-section to reduce the surface parallel to the stress force.

In all the above-mentioned road tunnels, hot galvanized rebar bolts, 20 mm in diameter and 2.4 m long, have been used. The bolts are point-anchored with resin cartridges. Triangular plates with dimensions 400 x 500 mm x 8 mm are used. When installing the rock bolts, the bolts are not tensioned (Figs. 4 and 5).



Figs. 4 and 5. Rock bolts with plates against spalling.

The deformation of the rock will eventually develop the necessary tension in the bolt. Pull-out tests showed that the threads were torn off at a jacking force of about 18 tonnes.
A few rock bolts have been torn off due to rock pressure. In the Høyanger Tunnel a broad sheet of rock parallel to the foliation broke away and tore off several rock bolts many weeks after blasting at that point /5/.

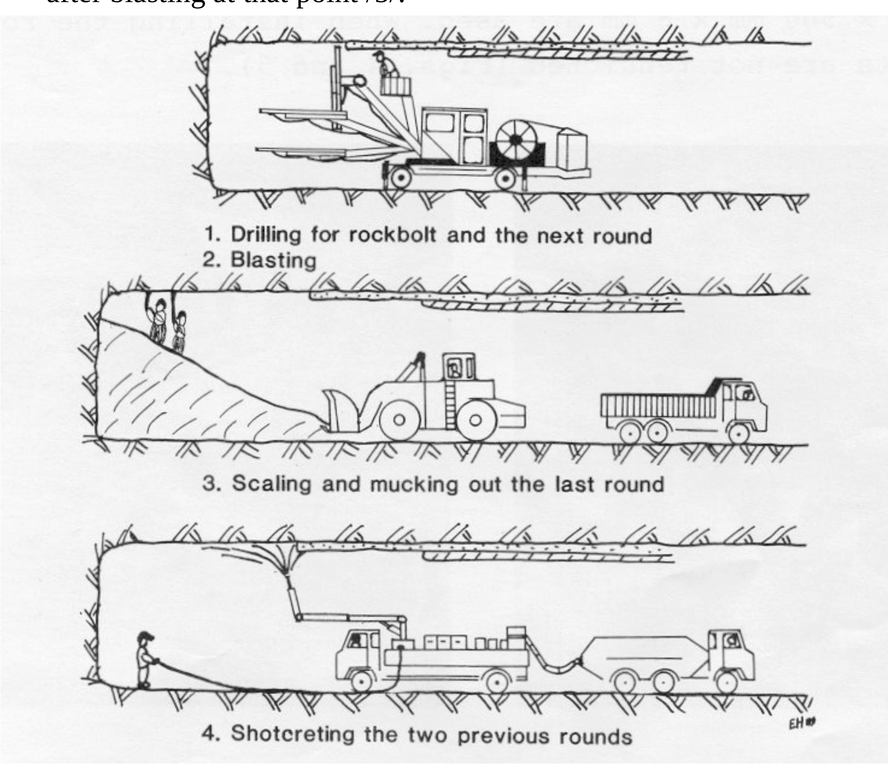


Fig. 6. Working procedure under conditions of rock stress

Working procedure has changed somewhat since the Oppljos Tunnel at Strynefjellet was driven /6/.
In the Oppljos Tunnel and the western face of the Høyanger Tunnel, bolt holes were drilled with hand-held jacklegs operated from a hydraulic platform. In areas with heavy spalling and rock burst, scaling was not done while mucking out the first half of a round after blasting. Then scaling and rock bolting were carried out as far as one could reach. After that the rest of the mucking was completed. Finally, scaling and rock bolting were

completed up to the face. Under the worst conditions wire mesh was mounted on the crown, and sometimes the face had to be bolted with l m long bolts, on rare occasions combined with wire mesh /5/.
In sections with more moderate spalling, the whole round length was scaled during mucking out (Point 3 in Fig. 6). Afterwards the bolting was carried out in one operation.
On the eastern face of the Høyanger Tunnel, the drilling of the bolt holes was done with a pneumatic drill jumbo with 2 booms specially mounted for bolt-hole drilling, with a third boom carrying an access platform. In the tunnels which were driven later, drilling for rockbolt holes was carried out with a three-boom hydraulic jumbo, with a charging platform mounted on the fourth boom. Rock-bolt holes are drilled along with the blast holes for the next round. Insertion of the bolts is carried out in the same operation from the charging platform (Fig. 6, point l). One or two of the booms, usually equipped with Cop 1038 rock drills, are used for drilling the bolt holes, depending on the number of rock bolts provided. The diameter of the drill holes is 45 mm, as is usual for blast holes. This has led to the use of the "Ørstad bolt". The bolt is an ordinary hot-galvanized 20 mm diameter rebar bolt, equipped with a steel spring for rotating and mixing the 40 mm diameter polyester cartridges which function as an anchoring mass. The insertion and rotation is done by means of hand-operated pneumatic jack drills operated from the charging platform.
The use of a jumbo for drilling bolt holes increases safety and reduces time consumption. This procedure has be en used in the following tunnels: Tosen /4/, Vallavik /3/, Flåm, Fjærland, Tafjord /1,2,7/.

Table 2: Rock support due to rock stress

Tunnel suite	Area	Lenght of tunnel	Lenght of stress-affected section	Average progress	Number of rock bolts		Steel-fibre reinforced shotcrete		Wire mesh		Max tang.	Max over burden
	m^2	A)m	B)m	m/week	total no.	3) no./m	total	m^3 B)m	total	m^2 B)m	MPa	m
Oppljos	50	4380	1900	20 [1)]	30000	10	0	0	1500	0.5	56	600
Høyanger	50	7460	6800	22	75000	11	0	0	26000	3.8	92	1200
Tafjord	39	5266	3900	38 [2)]	20700	5	2650	0,7 [4)]	0	0	70	700
Vallavik	52	4500	2500	32	13680	5	2350	1,2 [4)]	900 [5)]	[5)]	---	800
Tosen	50	2550	2300	30	9000	4	0	0	200 [5)]	[5)]	---	600

1) Converted from 3 to 2 shifts/day, 5 days/week
2) Converted from 9 to 8 hours/shift, 2 shifts/day, 5 days/week
3) Number of rock bolts in stress-affected section of tunnel
4) In Tafjord: Shotcrete in ½ profile.In Vallavik: Shotcrete in full profile over a distance of 2000 m
5) Not finished

Table 2 shows that the tunnels mentioned had made better average progress per week than the Oppljos /6/ and Høyanger /5 tunnels, where hand-operated equipment was used.
After breakthrough the rock bolts have to be supplemented, due to intense spalling and loose rock fragments hanging on the plates. Scaling is not necessary in areas which are rock bolted. Removal of rock fragments, or loosening of rock bolts, will result in new spalling. The total number of rock bolts used per metre of tunnel varies from four to seventeen, depending on spalling intensity (Table 2). The point-anchored hot-galvanized rebar rock bolts are used both during driving and as permanent support. In the Tafjord and Vallavik Tunnels, rock bolts were combined with steel-fibre reinforced shotcrete to prevent spalling.

In both of the tunnels rock bolting was carried out before shotcreting.
The plates used in this connection were circular with a diameter of 150 mm. Usually shotcreting was carried out one round behind the face, but it varied. Shotcreting was carried out at the face when extensive spalling occurred, and four or five rounds behind the face w hen spalling was limited. The steel-fibre reinforced shotcrete is added in two instalments, each with a thickness of about 5 cm (Fig. 6 point 4) The shotcrete acts as a permanent support, but has to be strengthened with supplementary rock bolts to prevent sheets of it loosening.
In the Tafjord Tunnel about 25% of the total number of rock bolts were installed after shotcreting /7/. Experience indicates that as high a percentage of rock bolts as possible should be installed after shotcreting. It is difficult to predict the thickness of shotcrete necessary. However experience has shown that a thickness much less than 10 cm should be avoided. With increasing spalling intensity, more rock bolts have to be used to support the shotcrete, and to resist deformation.
Measurement has indicated that the shotcrete layer is stress-less, while the rock behind the shotcrete layer may be subject to high stresses. These forces act on the rock bolts to some extent.
The Tafjord Tunnel was constructed by the Norwegian contractor Høyer Ellefsen A/S. The other tunnels were constructed by the various county highway departments.

COSTS AND DRIVING RATES

It is impossible to make an exact comparison of the intensity of rock-burst problems in the different tunnels. Hence it is not correct to compare the total costs of the tunnels. However the cost of the various working procedures can be analysed, and the cost of employing them in different tunnels estimated. In Tafjord the total cost per metre of tunnel was remarkably low in spite of the use of ex pensive fibre-reinforced shotcrete. This is mainly due to the high driving rate compared to that of the other tunnels considered (Tables 2 and 3). Since the driving of the Oppljos and

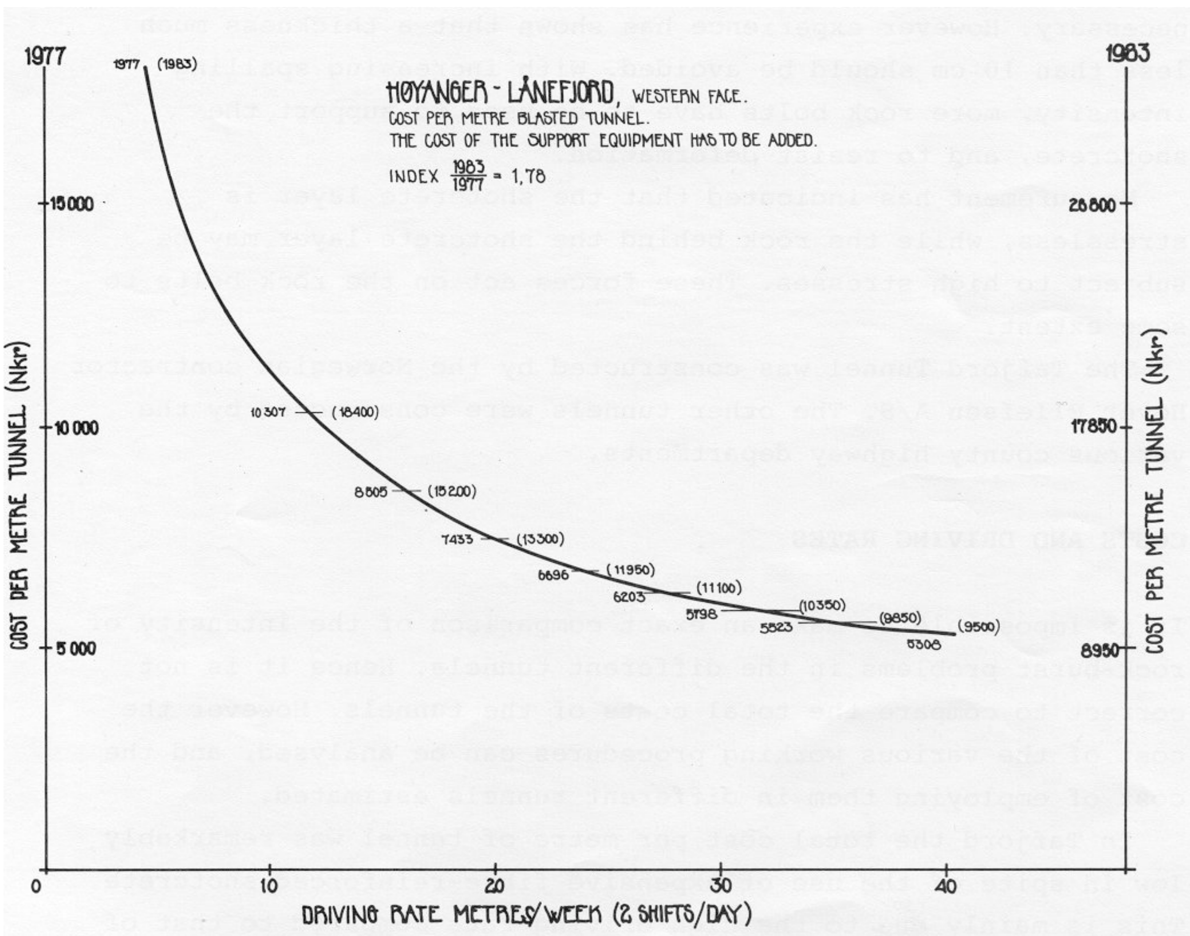


Fig. 7. Tunnel cost related to driving rate.

Høyanger Tunnels, there has been great development in productivity (compare with the working procedures mentioned).
There are strong reasons for assuming that the average progress per week would have been approximately 30-35 m/week in the Høyanger Tunnel today, if modern equipment and working procedures had been employed. This would have distributed the fixed expenses over more metres per week, and saved about NOK 3000 (1983) per metre in comparison to the 22 m/week actually driven (Fig. 7, after 15/). Recently (February 1984) an average driving rate of about 40 m/week has been reported in four tunnels under construction. This rate includes the installation of 15 to 25 rock bolts per round, or 4 to 6 bolts per metre due to moderate rock-burst problems. These tunnels are Fjærland, Flåm, Vallavik and the newly started Søreide-Vamråk, all working with 108-hour shifts per week. In Vallavik the progress per week has increased from 32 m with heavy to moderate spalling, and the use of rock bolts and shotcrete, to 40 m with moderate to little spalling, and the use of rock bolts without shotcrete.

Fig. 8. Spalling in an unsupported area

In most of the tunnels mentioned, scaling has been very time consuming. In the Høyanger Tunnel, where nearly half of the rounds were lost (22 m/week in progress), 2-3 hours per round were consumed in rock bolting. About 5 hours were employed in scaling per round. This was necessary for reasons of security, w hen rock bolting with hand-operated equipment.
Rock-bolting with the jumbo has reduced the scaling time to less than half. The use of steel-fibre reinforced shotcrete has also reduced the necessity of thorough scaling, and to some extent the number of rock bolts needed. These factors have reduced the time used per round to such an extent that the reduction in costs will compensate for the use of the expensive shotcrete.
If 1 m^3 steel fibre reinforced shotcrete (NOK 3000) is used, the driving rate has to be increased by nearly 10 m/week to compensate for the extra cost of the shotcrete (see Fig. 7 and Table 3).
When tunnels are driven in regions with high rock stresses it is better to insert too many rock bolt than too few. Both in Vallavik and in the Tosen Tunnel, rock-burst problems were not tackled with an adequate number of rock bolts at the commencement of tunnel construction. This resulted in spalling and loosening of rock fragments between the rock bolts (Fig. 8), which meant more scaling, and so on ad infinitum. The low number of rock bolts per metre in the Tosen Tunnel (Table 2) is due to the sparse rock bolting in the first 1000 m, which will have to be supplemented later. Mechanized scaling will probably further reduce the time spent on scaling.

CONCLUSION
In the previous section working procedures and costs have been discussed. As a general conclusion it seems clear that the driving rate is the most important factor with respect to costs.
The use of the jumbo to drill bolt holes is a great advantage in cutting time and costs, and increasing the safety of the workmen. The employment of steel-fibre reinforced shotcrete is slightly more expensive than bolting with use of wire mesh, despite the time it saves.

The long-term effect of the different supporting equipment is being followed at Høyanger and Tafjord. Some spalling has occurred in the shotcrete layer, especially shortly after spraying. Spalling' has occurred particularly in places with thin layers of shotcrete, and in places with an insufficient number of rock bolts.
In Høyanger very few rock fragments have loosened and been caught by the wire mesh, due to the close rock bolting. However in the lower part of the walls some spalling has taken place.
This part of the profile is not rock bolted. Hence, sufficiently close rock bolting, if necessary combined with mesh, seems to be the most economical and reliable long-term form of rock support in rock which is under stress. Shotcrete seems to work well, and has to be further developed with respect to thickness and number of rock bolts necessary.

Table 3: Cost for tunnel driving and support works under high stress conditions

Tunnel	Høyanger	Oppljos	Tafjord	Vallavik	Tosen
Lenght subject to stress/ lenght driven	0.91	0.43	0.74	0.55	0.90
Total cost. Tunnel blasting with temporary support. NOK/m	13,700	12,000	12,700	12,000	13,000
Of the above: temporary support NOK/m in sections subjected to stress	3,100	3,900	4,900	4,100	2,600
Final support. NOK/m in sections subjected to stress	1,100	1,800	500	1,200 [1) 2)]	[2)]

All costs in 1983 prices
1) In sections with shotcrete
2) The tunnel is not completed

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MECHANICAL SCALING

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ABSTRACT
One condition for constructing low-cost road tunnels is that most of the rock must be sound enough for scaling and fastening of loose blocks with anchor bolts to provide adequate support. Rock-scaling is still done manually in part, but supplemented by a rock-scaling machine. The hard, dangerous manual work can often be avoided in an efficient and economical way.
The Bergrensk machine has a boom with 360°C rotation and a telescopic extension of 2.4 m. The maximum reach is approximately 12 m horizontally and vertically. On the boom-end there is a replaceable jackhammer with 120° knuckle action. The hammer can easily be fitted with a high-pressure water lance, or replaced by a hydraulic drilling unit. The machine has proved both practical and versatile.
We have had 3 years (6,000 hours) of experience with this machine in Norwegian road tunnels, and it forms the subject of this report.

Blast face scaling
Workers stay out of areas that are not cleared. The scaling time and idle times for driving equipment are reduced. The average advance per blasting operation is 15-30 cm.
Capacity: 1-6 hours between blasting, averaging 2.6 hours when driving 50 m/month.

Final and maintenance scaling
The Bergrensk machine is already paying its way by allowing use of anchor bolts to be reduced by 2-3 per hour. The scaling capacity is a lot higher than that achieved by traditional methods, and costs are lower. Used in combination with high pressure water jets (500 - 1,000 bars} the machine is efficient in soft, slaty and highly fractured rock.
Scaling capacity: 3-20 m per hour.

Drilling for anchor bolts.
Operated entirely from the protected cabin, it can drill 28 mm holes to a depth of 3.2 m, 38 mm holes to a depth of 3.9 m and 5l mm hales to depths of about 15 m.
Capacity: 28 mm, 2.4 m deep: 16 hales per hour.

Knocking out ditches
Capacity: 6- 15 m per hour.

l. INTRODUCTION
A considerable part of the total costs of a tunnel go into scaling, anchoring and/or lining. Assuming that the same safety requirements are made in all cases, it would lower construction costs if the most expensive operations could be partly or entirely rationalised by using new methods and machines. The use and extent of scaling, anchoring and lining has until recently been based on manual scaling alone. Long experience of tunnelling is necessary for assessing how much should be scaled, anchored or lined. Ever more exacting requirements regarding safety, working environment and economising have created a greater need for mechanical scaling equipment.
The last few years have seen the development of such machinery from the primitive first generation to reliable units which have had a revolutionary effect on costs and safety. Depending on the circumstances, the following advantages can be achieved:

- Improved safety and reduced costs for securing crowns.
- Reduction in the number of anchor bolts used, and higher quality in those fitted.
- Increasing areas where mechanical scaling combined with anchor bolts can be used instead of concrete lining. This alone can save considerable amounts of time and money.

2. DIFFERENT TYPES OF MACHINE

Many types of machine have been developed, from the ordinary broken tooth excavator, to the most advanced and versatile specialized machines. Circumstances determine which machine is best for a job. It can generally be said that simple problems are solved at lowest cost by using a simple machine, while more complex and difficult circumstances call for a more versatile machine. The essential thing to know is how to choose and operate the machine which gives the best overall results in the particular circumstances. Space does not permit a description of the various kinds of machine here, and I shall confine myself to a description of the machine I know best: the Bergrensk machine.

3. GENERAL DESCRIPTION

The Bergrensk machine is based on an 18.5 ton rubber-wheeled excavator, fitted with a boom with 360° rotation and a telescopic extension of 2.4 m. A hydraulic jackhammer with a knuckle action of 120° is mounted on the working end of the boom. The percussion power can be switched from 3,300 Nm to 1,500 Nm with 450 or 900 strokes per minute respectively. Theoretical output in both positions is 24,270 Nm/sec or approximately 24.3 kW. Light scaling is achieved by scraping or light percussion strokes. If the chisel is not pressed against the rock, the effect of the impact is reduced. In this way we have removed 2.5 cm of loose concrete from reinforced structures without damaging the superstructure or reinforcement. The modern two-lever servo controls provide a sensitive and precise control of the percussion effect.

The maximum impact is comparable to 1,800 men with 4-kilo hammers hitting at a rate of lo strokes per minute. Maximum impact is applied for chiselling off lumps or trenching etc.

Mounted on the front of the machine is a bulldozer blade for clearing rubble, and two stabilizers for use with the 2 m boom extension. Two 1,500 W floodlights give good working light. The cabin is specially protected against falling rocks and has air conditioning to protect the operator from dust. All tyres are puncture-proof.

The hammer can be replaced by a drilling unit for drilling anchor bolt holes. The rig is equipped with an automatic feed mechanism to avoid wedging in bad rock, and a hydraulic drill for extension drilling. Special filters are fitted to prevent dust entering the engine and hydraulic circuit. The exhaust from the engine is very clean.

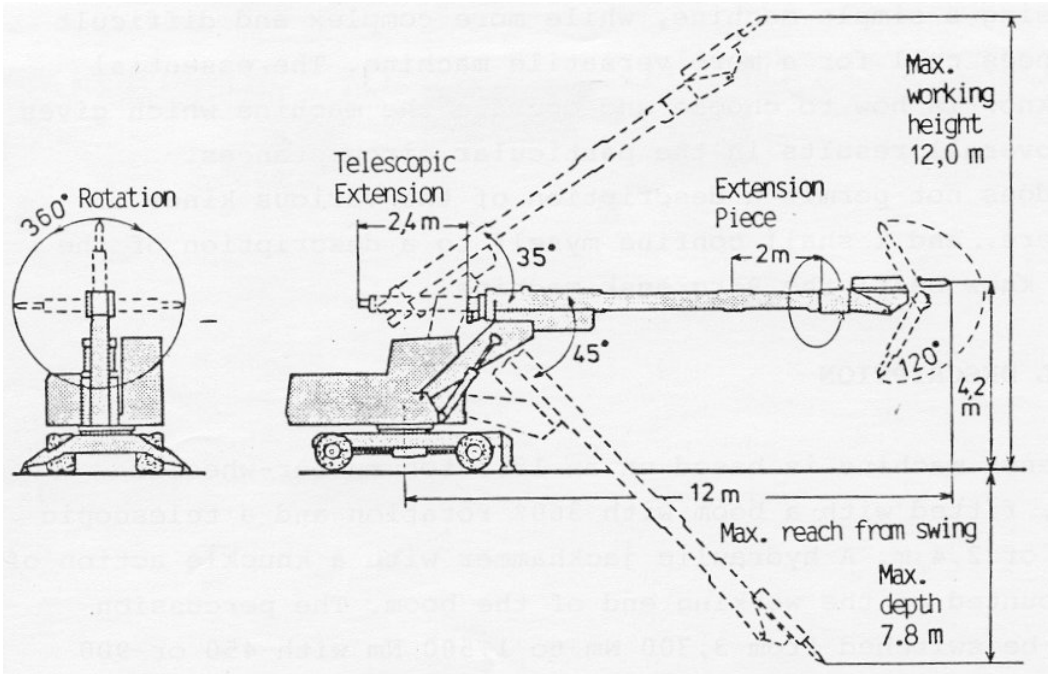

Fig. 4.1 Hydraulic scaling machine with rotating boom

4. TECHNICAL DATA

Basic machine	LIEBHERR A 922
Motor	Deutz 78 kW (106 hp)
Max. hydraulic feed	2 x 155 l/minute
Max, hydraulic pressure	300 bars, variable
Rotating boom.	SHAND ROTEL BOOM
Hydraulic jackhammer	Krupp HM 800
Drilling unit.	Atlas Copco BMH 612
Drilling hammers	COP 1028 or COP 1038
220 V A/C generator.	5 .5 kW
Total width	2,590 mm
Transport height	3,230 mm
With jackhammer and 2 m extension:	
Max. working height	12.0 m
Max. reach from slewing centre	12.0 m
With drilling unit and extension:	
Max. working height horizontal	11.5 m
Max. working height vertical	14.5 m
Max. reach from slewing centre horizontally	14.5 m
Max. reach from slewing centre vertically	11.5 m

5. SAFETY AND WORKING ENVIRONMENT

Use of a scaling machine allows unprotected workers to remain at a safe distance from dangerous crown sections and relieves them of the heavy scaling procedure. Under bad rock conditions the machine grants great safety than manual scaling. The machine operator directs all actions and movements via modern ergo-dynamic controls from the safety of a specially protected cabin. The cab is supplied with pressurized filtered air to reduce the danger of dust or gas entering. The official stamp of approval has been awarded to this type of cabin.

6. MECHANICAL SCALING GENERALLY

Good cooperation between experienced miners and an able machine operator is of paramount importance for obtaining optimal results. Whether and to what extent different working methods should be employed depends on the rock structures encountered. Compact and tightly cracked rock can very well be mechanically scaled, but if the rock is too loose, extensive manual scaling will be necessary subsequently. W hen blasting causes cracks, or w hen the rock surface does not coincide with the tunnel profile, mechanical scaling is best.

Bursting (popping) rock or rock with clay-filled cracks should normally not be scaled mechanically!

A machine with a rotating boom will have access from all possible angles. This is important for loosening a block with a minimum of force, in order not to disturb the surrounding areas. It saves clearing work and time.

It is often necessary to vary the working angle of the hammer, in order to chisel from new angles or levels. Here scaling is best done by prying out the loose rock, shifting the force from side to side, like a dentist extracting a tooth. Consequently it is very important that the angle of approach of the chisel can be easily and quickly changed. A good scaling machine should be like a fencer: swift, flexible and ready to attack from all angles.

Choosing personnel for this type of work, particularly scaling machine operators, cannot be done too carefully. A bad operator can spoil a scaling operation and damage ex pensive equipment, and also reduce safety and reliability.

7. BLAST FACE SCALING

Where manual scaling alone is easy and adequate, it is preferable. Use of the scaling machine should also be restricted in very loose strata or clay mixtures. Overuse of a mechanical scaler can loosen locking blocks and necessitate extensive securing work further up in the crown.

The most critical and risky phase is shortly after blasting, before solid rock is stabilised. Where the strata are poor, scaling may be necessary during mucking out following the blast. Without a scaling machine, this work has to be done by hand from on top of the blast rubble, using a wheel-loader bucket. This is a very slow and highly dangerous operation. In the Holmestrand Tunnel, for example, the Bergrensk machine has scaled about 240 m^3 of loose rock from one blast face.

Economic gain due to use of mechanical blast face scaling varies.

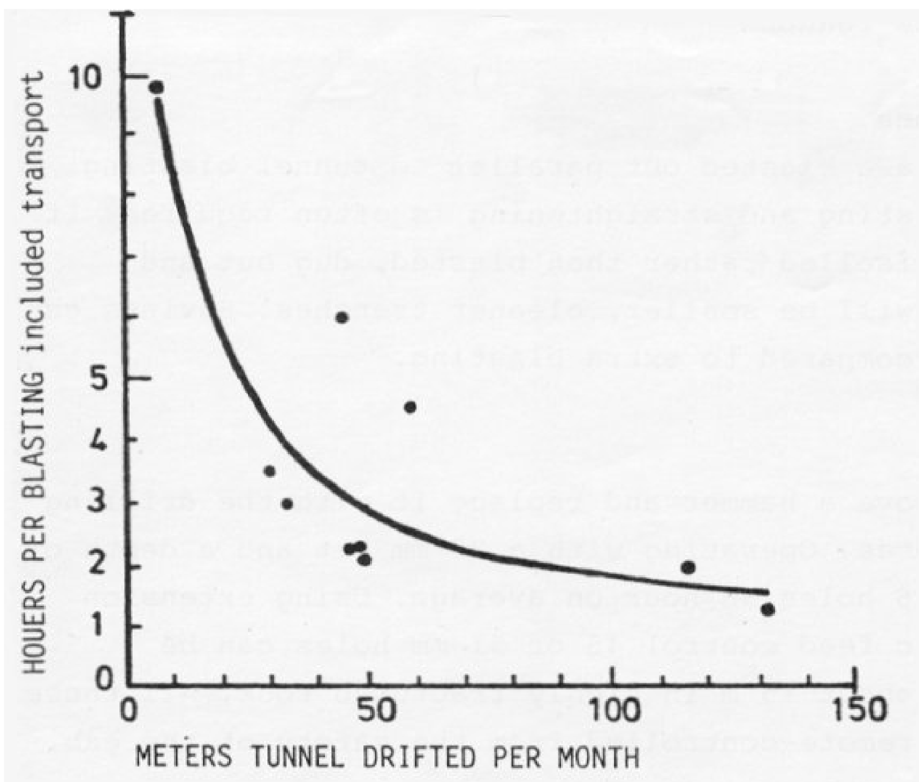

Fig. 7.1 Mechanical blast face scaling in the Holmestrand tunnel

The following advantages may be achieved:

- Increased blast lengths
- Shorter scaling time
- Less anchor bolts needed
- Less lining necessary
- Faster drilling and blast charging
- Increased driving speed and less overheads.

Conditions will be safer and delays avoided. Traditional placing of explosives may, for example, take 6 to 7 hours because of stone fall.

The capabilities of a scaling machine depend on rock quality. Fig. 7.1 shows scaling time in the Holmestrand tunnel, averaging 2.6 hours per blast including transportation of the machine.

8. FINAL AND MAINTENANCE SCALING

Before final scaling and anchoring take place, the tunnel must be inspected and marked with paint to show cracks and unstable sections. Correct marking, and cooperation between the inspection crew and the machine operator, will ensure that scaling and bolting are carried out only where needed, saving both time and money. Also included in the scaling operation are the removal of protruding rock from the tunnel profile, and cleaning out of 2-3 m of transverse fracture zones to enable vault construction. This can be achieved without removal of cables etc.

It is difficult to evaluate savings on the final scaling, as the cost of manual scaling under the same circumstances is not easy to estimate. Cooperation between inspectors and the scaling operator will greatly influence the efficiency and cost of the operation. As a rough guide: time saved will be in the region of 50 to 75%, and cost reduction will vary between 4 and 30%, the looser the strata, the less the amount saved.

A reduction of about two anchor bolts per hour will suffice to cover the expense of the machine.

9. OTHER OPERATIONS IN TUNNELS

Chiselling out trenches

If drainage trenches are blasted out parallel to tunnel blasting operations, extra blasting and straightening is often required. If these trenches are chiselled rather than blasted, dug out and refilled, the result will be smaller, cleaner trenches. Savings can be as high as 50-70% compared to extra blasting.

Anchor bolt drilling

The time taken to remove a hammer and replace it with the drilling unit is about 30 minutes. Operating with a 28 mm bit and a depth of 2.4 m, one can bore 16 holes an hour on average. Using extension drilling and automatic feed control 45 or 51 mm holes can be drilled to depths of about 15 m in highly fractured rock. All these operations are fully remote-controlled from the safety of the cab.

Increasing the crown height

The Bergrensk machine was used in three tunnels totalling 200 m, to increase the crown height from 3.5 to 4.0 metres. The rock consisted of gneissic granite with loose sections. The job, including scaling, was completed in 65.5 hours.

Fig. 9.1. The Bergrensk machine

10. CONCLUDING REMARKS

I consider flexibility to be the key word for future machines. For each type of tunnel, we need a machine to suit the situation. For front-end scaling after blasting, the choice is between a cheap scaling machine to meet simple demands, and a more sophisticated and more adaptable machine in a higher price range which can be utilized for other work while drilling and blasting are in progress. The right choice of machine for final scaling can only be made by people with a complete knowledge of existing machines and their limitations. Whether a single machine can accomplish all operations, such as anchoring, use of high pressure water jets, shotcreting or lining, depends on the conditions in the individual tunnel. The combination possibilities are numerous, and only when combined with thorough inspection will the best results be obtained.

My prediction is that inspection and evaluation of scaling work in each individual case will be of paramount importance for many years to come.

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USE OF WET-PROCESS STEEL-FIBRE REINFORCED SHOTCRETE IN TUNNEL LININGS

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ABSTRACT
Wet-process shotcrete is superior to dry-process with respect to capacity, rebound and working conditions. Use of various concrete additives, micro-silica and fibre reinforcement, together with general improvement of the method, have made it possible to produce high-strength concrete. Steel-fibre reinforced wet-process shotcrete was introduced in Norway about 4 years ago, and up to the present has taken over more than 50% of the shotcrete market. Both test results and a great deal of experience show that steel fibres (1% by volume is most common) can successfully replace wire mesh in linings of thickness of up to 100-150 mm, giving linings which are at least equally strong and more ductile. Use of fibre-reinforced concrete with a high early strength, applied by remote-controlled shotcreting rigs, has considerably reduced the delay caused by shotcreting. Applying fibre-reinforced concrete is 3 to 6 times faster than applying linings of the same thickness. Although the cost of the materials for fibre-reinforced concrete is about 70-80% higher than the price of concrete and wire mesh, the final product may be 10-20% cheaper. In addition you gain time. Use of fibre-reinforced shotcrete in combination with rock bolts is very suitable for combining temporary and final support.

1. INTRODUCTION
According to the American Concrete Institute, shotcrete is defined as mortar or concrete which is pneumatically projected at high velocity onto a surface (ACI 506.2- 77) .
Shotcrete was introduced into Norway in 1971. Until the middle of the 705 the dry-process dominated. Improvements in rock tunnelling methods now made support work the critical activity, which limited the total excavation capacity. The wet method was introduced primarily because it was faster and more economical.

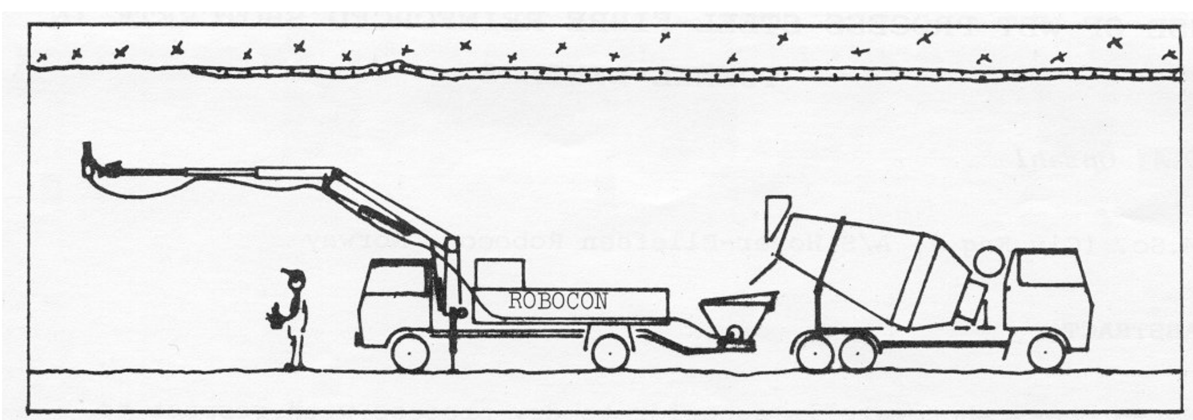

Fig. l. Shotcreting by means of a robot

The wet method was improved by using concrete additives, steel fibre and micro silica (pozzolanic additive). When shotcreting by robot became common in the late seventies, this method took over most underground applications in a short space of time. The situation in Norway today (1983) is special in two ways:

- Practically all shotcrete is "wet"
- About 55% of the total volume is fibre-reinforced shotcrete.

The big market for shotcrete, more than 90% of the total volume, is for use in connection with rock excavation as temporary linings, permanent linings or a combination of the two.

2. FIBRE-REINFORCED SHOTCRETE IN TEMPORARY LININGS

In bad rock rapid support is required as part of the cycle, and support is generally the critical activity. The basic advantages of wet-process fibre-reinforced shotcrete used for temporary support are:

- High rate of application (10-14 m^3 per hour with a robot)
- The reinforcement is applied immediately, producing a rapid reinforcing effect
- No delay due to putting up wire mesh (levelling out the rock surface prior to meshing is avoided, reducing the volume of shotcrete needed)
- Problems with encasing reinforcing bars in shotcrete are avoided (sand pockets behind bars, delamination)
- The fresh concrete including fibre is fully mixed before pumping (i.e. W/C-ratio and fibre content are fixed)
- Safer and cleaner working conditions through use of remote-controlled robot
- Little rebound (5-10%).

One important aspect of rock tunnelling is keeping the tunnel stable, i.e. avoiding stone release and collapse. In addition, rapid support with fibre-reinforced shotcrete will reduce the volume of scaling needed, making it easier to keep to the theoretical cross-section in bad ground.

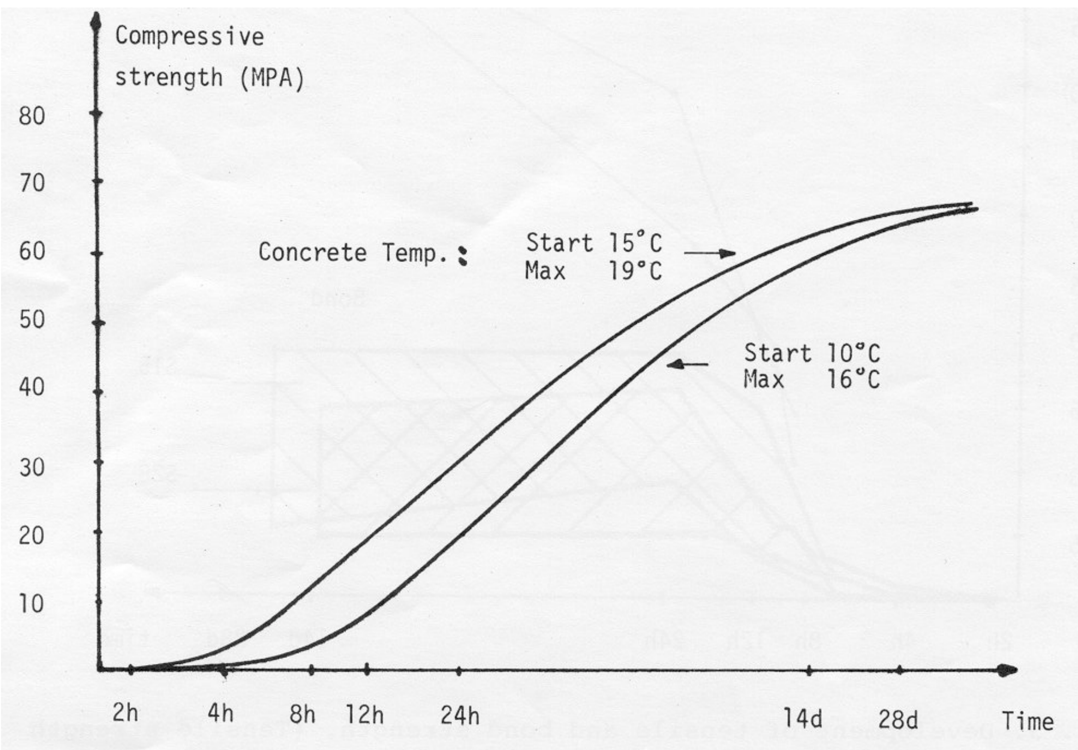

Fig. 2. Strength development in wet-process fibre-reinforced shotcrete (drilled out cores: Ø 60 mm, height = 60 mm tested in compression) Cem:: Agg: Micro silica = 1:3:0.1, by vol. of 18 mm EE-steel fibre, W/C = 0.5, 201 shotcreting accelerator per m^3.

Where rapid support is required, a mix resulting in high early strength should be specified. Use of quick-hardening Portland Cement, super-plasticizers (non-retarding), shotcreting accelerator and concrete temperatures of 20-300 are recommended (Fig. 2).

One important advantage of fibre-reinforced concrete is its rapid development of tensile strength, described in Fig. 3 in terms of the increase in tensile properties. As early as 24 hours after shotcreting, 70-80% of the final bond strength of a hard-rock underlay is developed (temperature of concrete 15°C).

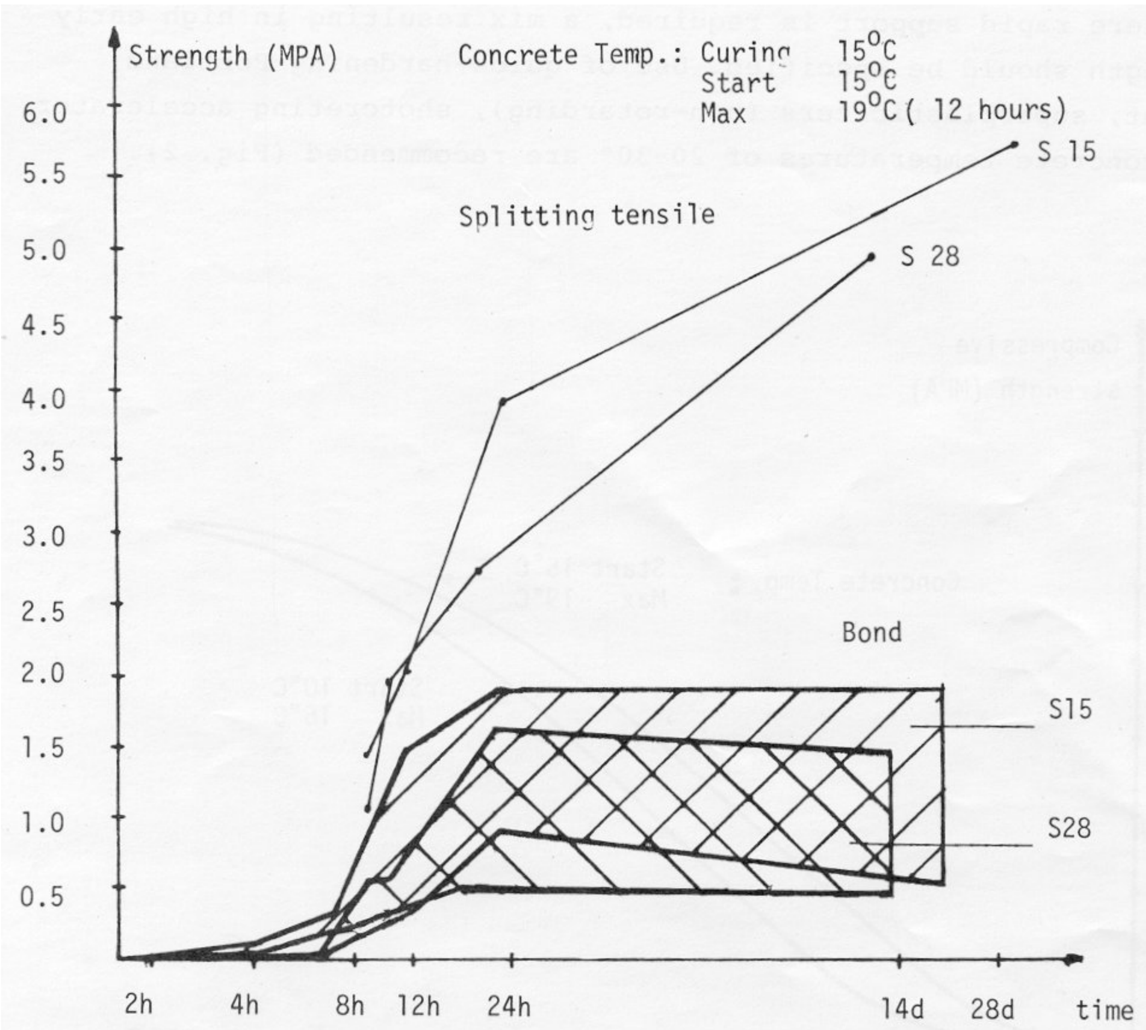

Fig. 3. Development of tensile and bond strength. (Tensile strength tested on drilled-out cores Ø = 60 mm L = 60 mm, bond strength measured by pulling shotcrete cores from the rock underlay). The cross-hatched areas indicate the scattering of results for bond strength (all observed values lie inside these areas). 15 and 28 litres of shotcreting accelerator per m^3 *were added. Same mix as in Fig. 2.*

Shotcreting accelerator gives a flash set and has great influence on the properties at the early stage, i.e. the first 4-6 hours, depending on temperature. However, it is the hydration of the cement after final setting that makes the major contribution to the development of strength /l/.

Fibre-reinforced shotcrete should be combined with rockbolts, usually through systematic bolting carried out after shotcreting. The function of shotcrete is then to prevent loosening, especially in zones between bolts. Since bond strength cannot be expected between shotcrete and bad ground, rock bolts are needed to set up an interaction between the rock mass and the shotcrete (see Fig. 5 -"arch theory"). In very bad ground, where the tunnel face is in danger of immediate disintegration, fibre-reinforced shotcrete should be applied only minutes after each round is fired /2/. Rock-bolting

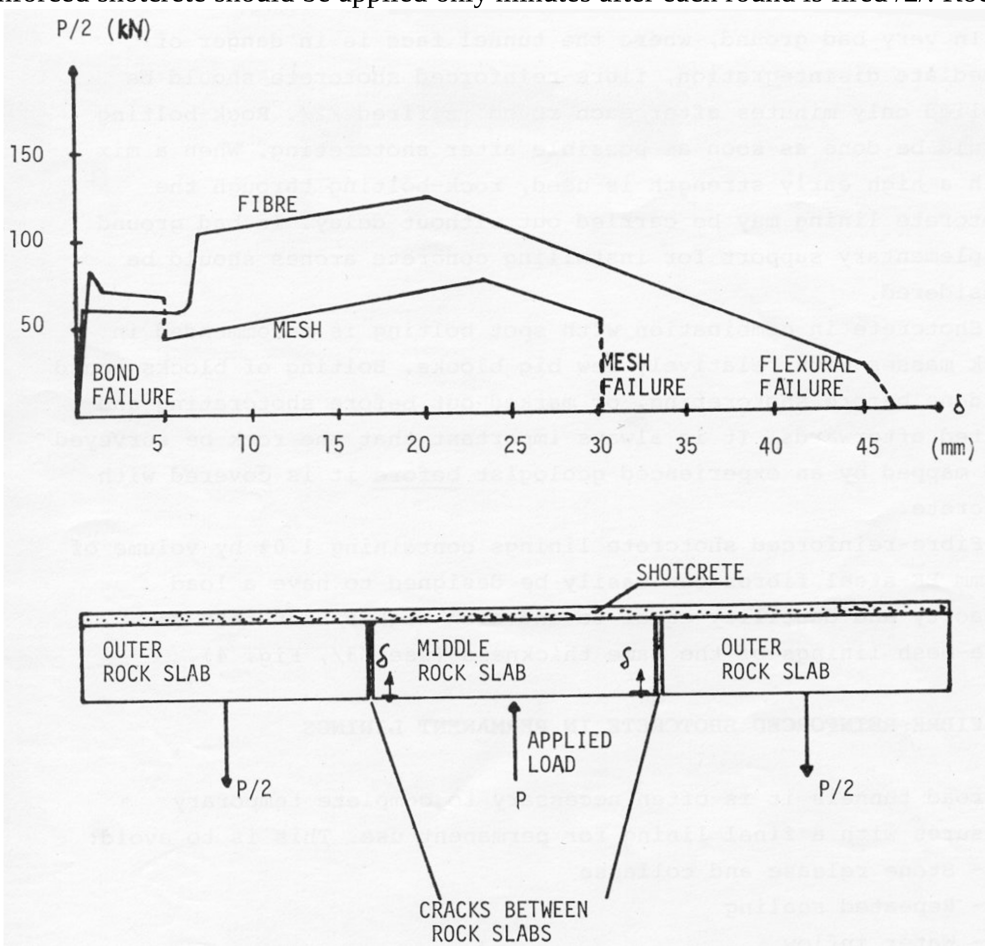

Fig. 4. Load-deflection curves from large-scale "falling block tests" and principal view of main test specimen.

should be done as soon as possible after shotcreting. When a mix with a high early strength is used, rock-bolting through the shotcrete lining may be carried out without delay. In bad ground supplementary support for installing concrete arches should be considered.

Shotcrete in combination with spot bolting is recommended in rock masses with relatively few big blocks. Bolting of blocks could be done before shotcreting, or marked out before shotcreting and bolted afterwards. It is always important that the rock be surveyed and mapped by an experienced geologist before it is covered with concrete.

Fibre-reinforced shotcrete linings containing 1.0% by volume of 18-mm EE-steel fibres can easily be designed to have a load capacity and ductility equal to and even higher than that of wire-mesh linings of the same thickness (see /3/, Fig. 4).

3. FIBRE-REINFORCED SHOTCRETE IN PERMANENT LININGS

In road tunnels it is often necessary to complete temporary measures with a final lining for permanent use. This is to avoid:

- Stone release and collapse
- Repeated scaling
- Water inflow

It is good economy to have the permanent support in mind when planning and installing the temporary support. High quality temporary fibre-reinforced shotcrete support may also serve as final support, or at least part of it.

For dealing with inflow of water, it is important that shotcrete be combined with drainage and insulation. Shotcrete is suitable for concentrating and deflecting leaks, not stopping them. The water permeability of shotcrete and concrete in general is more a function of age and maturing conditions than of their strength. Shotcrete, and concrete in general, must mature at least 3-4 days before it can resist water pressure. Once penetrated by water at an early age, a shotcrete lining will have permanent leak channels in the concrete structure.

Problems with moisture and moderate water inflow may be solved with shotcrete. Heavy water inflow or high water pressure should be dealt with by pregrouting before excavation or grouting before shotcreting.

Shotcrete linings with water pressure behind them will be saturated with water. Frost will cause destructive expansion in the ground behind the lining, and at the same time the shotcrete itself will be subjected to freezing and thawing.

In permanent linings the final strength and durability are essential, unlike temporary support where early strength is most important.

In general, the durability of concrete is improved by lowering the water/cement ratio. Resistance to water and liquids is improved by using special cements, for instance sulphate-resisting cement. Another way of greatly increasing durability is use of micro silica, an ultra fine pozzolanic material containing 85-95% amorphous SiO_2 14/. W hen producing high strength shotcrete, micro silica is required. Today, however, micro silica is used in almost all wet-process fibre-reinforced shotcrete.

In order to obtain durable shotcrete in linings, the following requirements should be satisfied:

- Compressive strength : 35 MPa or more
- Coefficient of permeability : 10-12m/s or less
(D'Arcy's law)
- Air void characteristics : Required if frost resistance is important. Generally accepted values (see 15/) should be specified.
- Corrosive liquids/water : Micro silica
W/C < 0.45
Special cement.

4. DESIGN OF FIBRE-REINFORCED SHOTCRETE

In spite of many publications on shotcrete for rock support, clear principles for dimensioning are lacking. Design will not be discussed here. It should just briefly be mentioned that we have designs based on:

- Case histories
- Engineering classification of rock masses /6/
- Measurements of deformation (NATM)

At least two of the design theories should be mentioned:

"The falling block model" (see Figs. 4 & 5)

"The arch theory" (see Fig. 5)

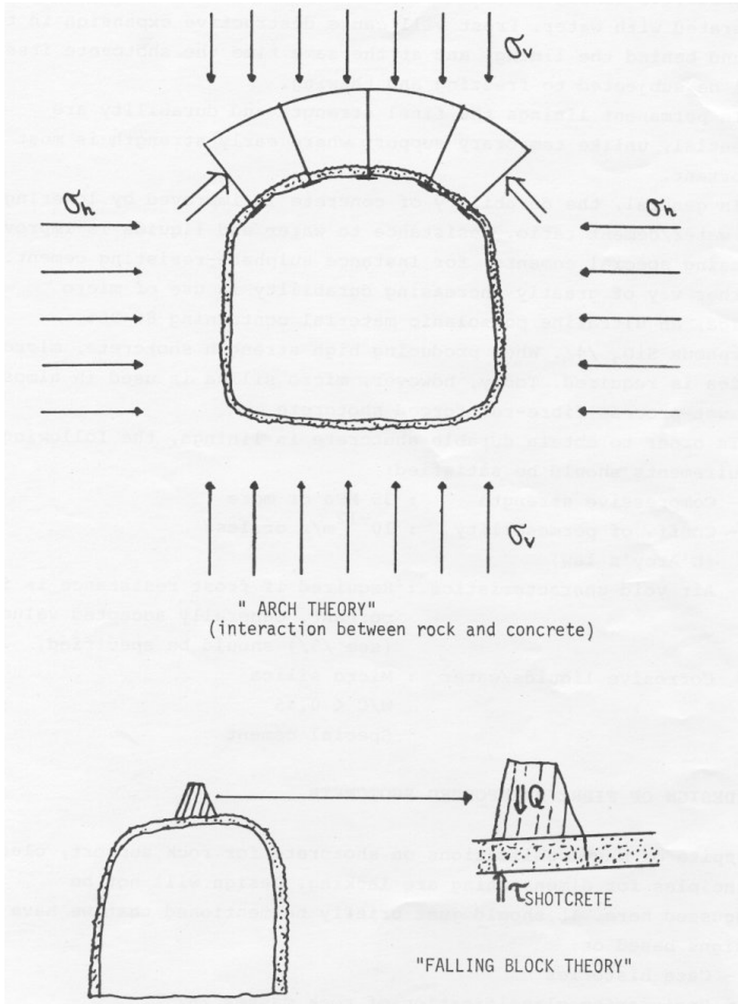

Fig.5. The arch and falling block theories

Documented characteristics of fibre-reinforced shotcrete are

• Compressive strength:	20 - 100	MPa
• Uniaxial tensile strength:	1.5 - 6	MPa
• Flexural strength	3 - 12	MPa
• Shear strength	8 – 12	MPa
• Ductility (fracture energy)	2000 – 15000	Nm/m^2
• Bond strength to hard rock	0.5 – 2.0	MPa

The big difference between plain and fibre-reinforced shotcrete is the crack-distributing and crack-arresting effect resulting in ductile or quasi-ductile behaviour. This very ductility increases the load-bearing capacity, but this in turn depends very much on the loading situation and boundary conditions. Yield line theory, non-linear material models etc., should be used rather than the linear theory of elasticity when looking at the load capacities of fibrous linings.

5. WHY WET-PROCESS FIBRE SHOTCRETE IN LOW COST ROAD TUNNELS?

The price of wet fibre shotcrete, with a compressive strength, C35, of 35 MPa, containing 1.0% by volume of steel fibres is:

Shotcreting at face:	NOK 2000-2500 per m^3
Shotcreting behind face:	NOK 1800-2200 per m^3

Comparing costs of fibrous and wire-mesh linings of equal capacity, both 120 mm thick, one arrives at a price difference of 5-15% in favour of fibrous linings. Since levelling off the rock surface is usually necessary before putting up wire mesh, the actual volume of shotcrete in wire-mesh linings is 20-50% greater than in fibrous linings. This will make fibrous linings even more economical by comparison. As mentioned earlier, support is often the critical activity in tunnelling. In a road tunnel with a cross-section of 50-60 m^2, the standstill costs, including capital cost and wages, will be NOK 2500- 3500 per hour. Use of fibre instead of mesh will typically reduce the support activity from 8 to 4 hours per cycle (shotcreting and rock-bolting). This gain of time will result in a saving of NOK 12,500- 17,500 per cycle.
There are many examples in Scandinavia of jobs where mesh has been replaced by fibre, resulting in reduced cycle time, which in turn resulted in greatly improved progress.

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THERMAL DESIGN IN NORWEGIAN ROAD TUNNELS

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ABSTRACT
A large number of Norwegian road tunnels are located in regions with severe frost.
Problems invariably arise in the part of the tunnel's frost zone where leaks do not freeze solid. The accumulation of ice along walls and ditches often clogs up drainage paths, causing water to spill onto the road, where it freezes, making driving hazardous and increasing maintenance difficulties.
Frost progression in road tunnels is governed by:

- Wind-induced forces
- Temperature-induced forces
- Forces induced by pressure differences
- Mechanical ventilation

In some places meteorological conditions are complex, and calculating the contributions of the individual variables is difficult. An empirical method is proposed for mapping a section of the freezing index through a road tunnel. Statistical data have provided a basis for dimensioning anti-frost measures.

INTRODUCTION
There are already more than 700 road tunnels in Norway. About 450 tunnels are on the primary national road network, and the remainder are on country or secondary roads. Less than 3% of the total tunnel length has zones of in-situ concrete lining. In many tunnels the cost of permanent support amounts to 40-50% of the total costs, which include water and frost protection, some scaling, plus construction of the drainage system and the road itself. Normally water and frost protection constitute the greater part of the
initial expenses, and they are sometimes as high as the cost of drilling and blasting. In recent years, time and money have been spent in an endeavour to map frost penetration in road tunnels, and to work out rules for economical dimensioning adapted to local conditions and traffic safety.

FROST EFFECTS IN ROAD TUNNELS
In dry, smooth rock, frost rarely causes any damage. Near tunnel openings, where there may be considerable variations in temperature, freezing and thawing processes occur, but in Norway this does not cause major difficulties.
When the rock forming the roof of a tunnel is jointed, there will almost invariably be some leakage, and this effect is compounded because the water drips freeze in the tunnels, making it almost impossible to drive vehicles through them. When water freezes, it expands about 9%, and this in turn may expand joints and cracks. With time, water flow causes erosion and loosens small pieces of rock from the crown of the tunnel. In addition to the direct results of expansion due to frost, a lot of other problems accompany an unlined road tunnel:

- Accumulated ice in the crown loosens and falls in periods of mild weather and in the spring.
- The drains skirting the road freeze, making the road slippery.
- Water dripping from the crown produces ice humps on the road.
- The weight of ice causes damage to the tunnel lighting.
- Small ice wedges in the pavement structure result in rough roads.

FROST PENETRATION IN TUNNELS

Frost penetration in tunnels is governed by the heat balance. When cold air streams in, heat conduction and convection cool the rock. In the open, radiation is an important factor for the heat balance at the pavement surface. Effects such as surface icing due to long-wave heat radiation do not occur in tunnels. The following meteorological forces are of significance for frost penetration in road tunnels:

- Wind-induced forces
- Temperature-induced forces ("chimney effect")
- Forces induced by pressure differences

In tunnels with mechanical ventilation, this ventilation will be the dominant factor causing frost penetration. Most tunnels in Norway have a low traffic density, which means that the "piston effect" is usually of minor importance On calm days the air flow in horizontal tunnels will be zero when the temperatures outside and inside are fairly similar If the surrounding rock has a higher temperature than the air in the tunnel, it heats the air masses and warm air flows along the ceiling and out of both tunnel entrances During the winter, however, frost will penetrate this type of tunnel, except the longest ones, which have a frost-proof section in the middle. Frost penetration in horizontal tunnels depends on the tunnel length, the freezing index, and any forces resulting from differences in the pressures at the two tunnel entrances.

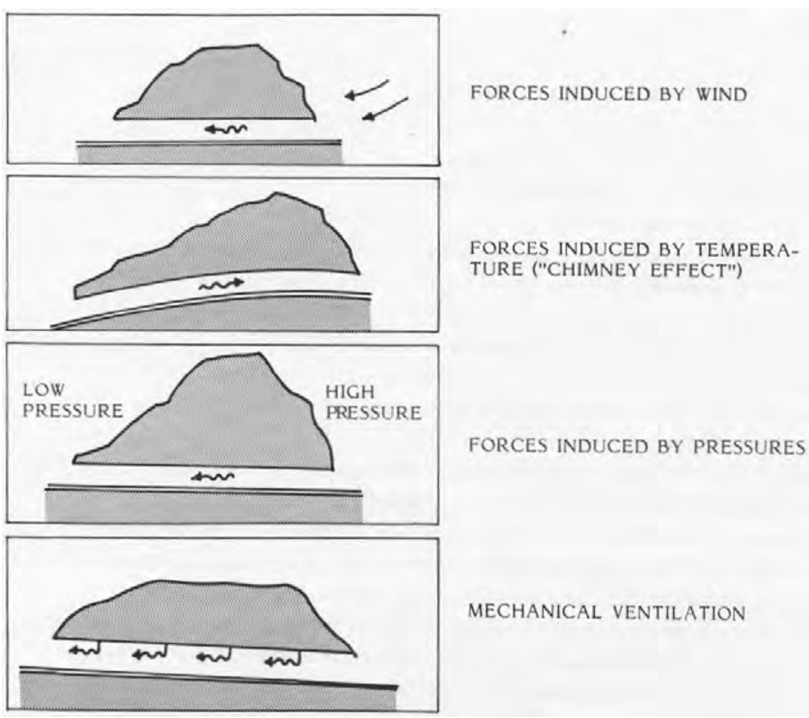

Fig. l. Effects of meteorological forces and mechanical ventilation on frost progress in road tunnels.

In short tunnels (length < 500 m) the predominant wind direction during winter will be a factor which contributes to frost penetration. Such effects are particularly noticeable in tunnels along the sides of valleys and fjords. In tunnels with a gradient, forces generated by temperature differences will cause air flow. This is also known as the "chimney effect".

The temperature in the walls of long tunnels (> 1000 m) is about 3-6°C throughout the year (Norway) .In the mountains the rock temperature in tunnels will be lower, especially in relatively shallow ones. Hard freezing periods may cause the air temperature 2-3 km inside the tunnel to fall below zero.

Warm air is less dense than cold air, and warm air rises following the gradient of the tunnel. Temperature-induced forces create a pressure gradient which can be illustrated as follows:

$$\Delta p = \gamma \cdot \Delta T/T \cdot \Delta H \text{(units are N/m}^2\text{)} \qquad (1)$$

T = average air temperature in tunnel, ºK
γ = air density (N/m^3)
ΔT = difference in air temperature inside and outside the tunnel
ΔH = difference between heights of tunnel entrances

On cold, calm days, this effect can be observed in tunnels with lengths less than 300 m and gradients of about 1%.

In summer the situation is reversed, and the air flows in the opposite direction.

In the mountains, some tunnels are situated such that they drain cold air from mountain plateaus. Experience has shown that the "chimney effect" may be outweighed by forces induced by such pressure differences, so that the draught direction is downwards even in tunnels with gradients of up to 6%

In some places meteorological conditions are complex, and calculating the contributions of the individual variables is difficult. In road tunnels with heavy traffic, mechanical ventilation has to be installed. This will contribute to frost penetration and increase the freezing index inside the tunnel, unless the ventilation air is heated.

HEAT BALANCE IN TUNNELS

The temperature inside a tunnel is governed by the amount of solar radiation, resulting primarily in convection and conduction between the air and the rock surface. The interaction of all these factors is complex, and a state of equilibrium is seldom attained. The result is constant changes in temperature and draught direction. In an unlined tunnel, the rock has the effect of reducing fluctuations in air temperature, whilst the temperature in tunnels with insulating panels will have larger variations because heat exchange between air and rock is prevented. The heat balance in an unlined tunnel and in a tunnel with insulating panels can be expressed by the equations:

$$Q_{Al} + Q_{A2} = Q_R + Q_I + Q_w + Q_p \qquad \text{Unlined} \qquad (2)$$

$$Q_{Al} + Q_{A2} \approx Q_p \qquad \text{Lined with insulating panels} \qquad (3)$$

Where

Q_{Al} , Q_{A2} = heat removed from the tunnel by air flow
Q_R = heat emitted by the rock
Q_I = heat released through ice formation and cooling
Q_W = heat emitted by water over 0c
Q_P= heat emitted by pavement structure
Q= heat removed/emitted in lined tunnel

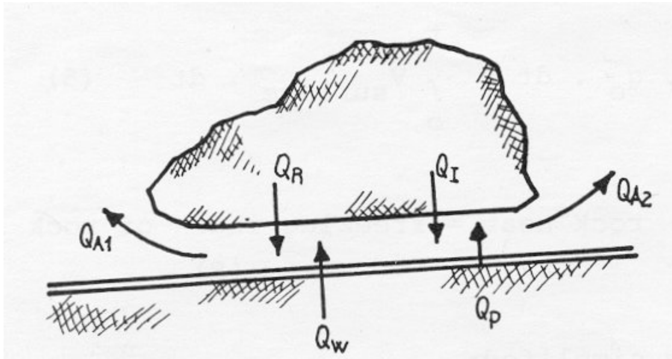

Fig. 2. Heat balance in an unlined rock tunnel

In tunnels which are partly lined with insulating panels this insulation will cause changes in the heat balance and an extension of the frost zone.
If the depth of frost penetration into the rock increases a little the heat balance according to /2/ can be expressed:

$$L \cdot d_z + q_o \cdot dt = \lambda_f \cdot G_f \cdot dt \qquad (4)$$

Latent heat + rock heat = heat conducted away from the freezing front.

$L = W \cdot C_d \cdot 1$ = latent heat (by volume, in Wh/m^3)

Where

W = water content (by weight)
C_d = dry density (in kg/m^3)
1 = latent heat of water (= 93 Wh/kg)
q_o = heat given off by unfrozen rock (W/m^2)
λ_f = thermal conductivity of frozen rock (3-5 W/mK)
G_f = temperature gradient above freezing point (ºC/m)
t = time (hours)

If the heat capacity of the frozen rock is ignored, the temperature gradient, G_f can be replaced by:

$$G_f = V_{surface} / z$$

Where $V_{surface}$= temperature of rock surface (ºC)
z = frost depth (m)
t_F = duration of frost period

$$[L \cdot z/\lambda_f \cdot dz]_0^{zmax} + [z/\lambda_f \cdot q_o \cdot dt]^{tf}_0 = [V_{surface} \cdot dt]_0^{tf} \qquad (5)$$

Latent heat of rock + rock heat = freezing index of rock surface
(Ω) (E) (F)

The expression can be simplified:

$$(\Omega) + E = F_{surface} | \qquad (6)$$

Rock in Norway generally has a low porosity. Consequently the latent heat is essentially associated with the water in discontinuities. In hard rock this latent heat contribution will be zero, and equation (6) can be simplified to the expression:

$$E = F_{surface} \quad (7)$$

In practice the freezing index can be calculated by measuring the temperature of the air in the tunnel. Such registrations have demonstrated that the following relationship exists between the freezing indexes of the rock surface and the air in the tunnel:

$$F_{surface} \sim (F_{tnnel\ air} - 3000)/1.7 \quad (8)$$

The equation is only valid a fair distance inside the tunnel (> 100 m), because the difference between the freezing indexes tunnel air/rock surface is small near the tunnel entrance.

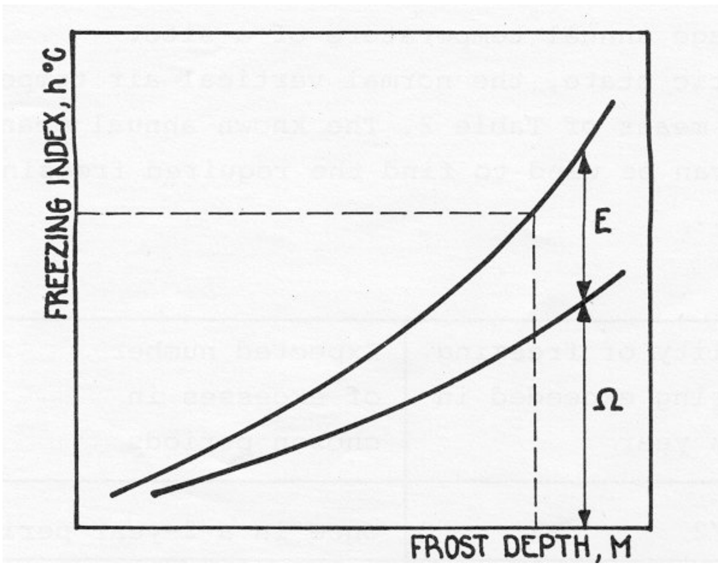

Fig. 3 Equation 6 illustrated graphically /2/

CALCULATING THE FREEZING INDEX AT THE SIDES

The method of calculation presented here is based upon temperature measurements made by the county authorities in several tunnels. The aim has been to include measurements made in tunnels situated in diverse climatic zones. The data is therefore incorporated in the design diagrams given here. For practical purposes it is assumed that temperature zones and freezing indexes are related. A relationship between freezing index, average annual temperature and location has also been arrived at. This relationship can be used for calculating the freezing index at the site.

Using accumulated frost data measured over a period of several years, it is possible to determine the probability of a definite amount of frost being exceeded. This opens the possibility of basing the dimensioning of tunnel linings on the conditions the structure is meant to withstand.

Accumulated frost, referred to here as the "freezing index", is denoted by a number, which is defined as the time integral of temperatures below zero throughout the winter.

Maps exist which give the average freezing index (F2) and the maximum freezing index (F100) in Norway (see Table l) .However, F10 has been chosen as the general freezing index for road design. This same F10 has also been chosen as the basis for frost calculations in Norwegian road tunnels. F10 can be found in tables relating to each centre of population. It can, however, also be calculated on the basis of the average annual temperature of a site. Assuming an adiabatic state, the normal vertical air temperature can be ascertained by means of Table 2. The known annual mean temperature, Fig. 4, can be used to find the required freezing index, F10 of the air.

Table l. The probability of any freezing index being exceeded, based on accumulated frost data. (See also explanation of freezing index at the end of the paper "Pavement Design" by R. Eirum, appearing in this volume.)

Freezing Index	F_2	F_{10}	F_{100}
Probability of Freezing Index being exceeded in a single year	50% (1/2)	10% (1/10)	1% (1/100)
Length of period in which one excess is expected	2 years	10 years	100 years

If an adiabatic state is assumed to exist, the normal vertical air temperature can be ascertained by means of Table 2. The known annual mean temperature (Fig. 4) can be used to find the required freezing index, F10 of the air.

Table 2. Assuming adiabatic conditions, the change in normal air temperature with height can be estimated with the aid of this table.

Area	°C per 100 m rise
Steep coastal areas, all height differences	-0.70
2. Inland areas, all height differences	-0.60
3. Narrow inland valleys, small height differences	-0.30
4. Open inland valleys, small height differences	-0.14

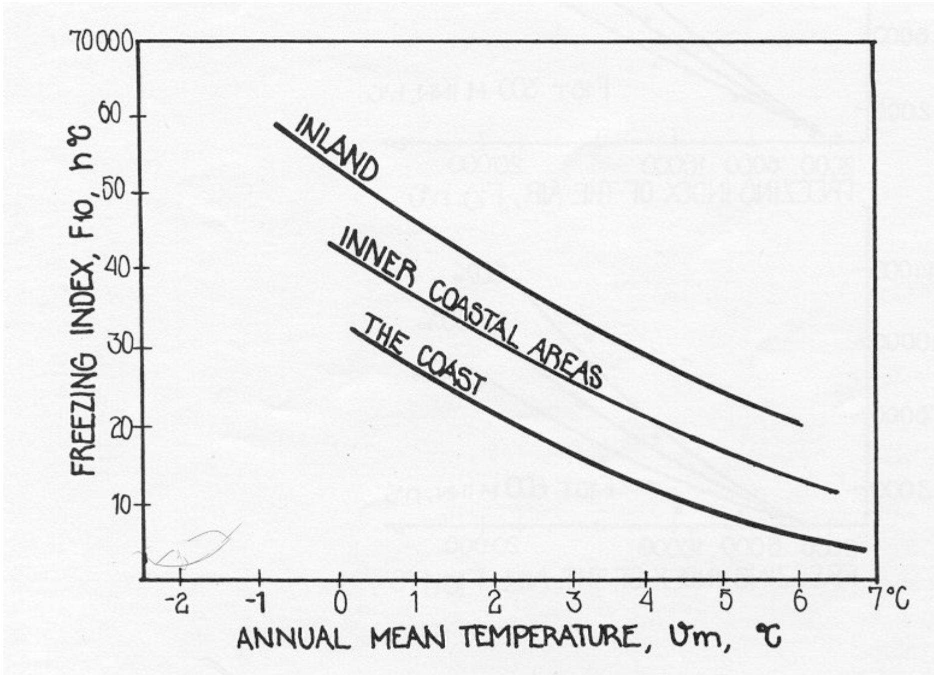

Fig. 4. When the annual mean temperature of a site is known, the desired freezing index can be obtained from the graph.

ESTIMATION OF THE FREEZING INDEX IN TUNNELS

There are several physical reasons for frost penetrating into road tunnels, as discussed earlier in this paper. Fig. 5 can be used to estimate frost incursion into tunnels with a cross section of 50 m^2 without mechanical ventilation.

To estimate the accumulated frost deeper inside the tunnel (more than 600 m from the tunnel opening), one can apply a rule of thumb according to which the freezing index can be reduced by 600 h°C per 100 metres. In long, rising tunnels frost seldom penetrates further than 200-300 m from the upper openings.

Many will wonder just how far tunnel linings or PE foam for water and frost insulation should be extended beyond the wet section of the tunnel. A good indication, which has been verified by measurements and calculations, is that frost will penetrate l m into homogeneous dry rock where the freezing index of the tunnel air equals 10,000 h°C.

In order to prevent frost from penetrating deeply into the roadway, with ensuing danger of icing and frost heaving, it is advisable to insulate the sole plate in the wet sections of the tunnel if the freezing index of the air in the tunnel exceeds 8000 h°C. The same problem arises if the drainage trenches freeze up. These can also be insulated, for example by means of extruded closed-pored polystyrene. Most places in Norway are so

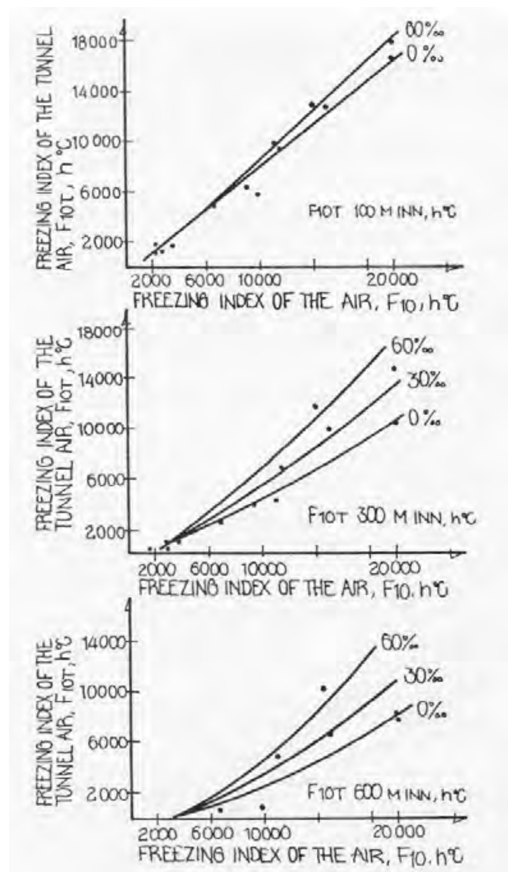

Fig. 5 Curves which can be used to estimate frost incursion in road tunnels without mechanical ventilation.

cold that high freezing indexes may be measured far inside tunnels. As a result insulating linings will sometimes be needed to prevent icicles forming on the tunnel crown. An metal or plastic lining that is not insulated cannot withstand much ice loading. Therefore such structures should not be used where the freezing index exceeds 3,000 hºC.

Table 3. Proposed criteria for frost prevention.

TYPE OF FROST PREVENTION	FREEZING INDEX OF TUNNEL AIR
Need for pavement insulation, wet areas	$F_{10T} > 8{,}000$ hºC
Drainage trench insulation it water flow in trench exceeds 1 litre/sec.	$F_{10T} > 6{,}000$ hºC
Drainage trench insulation if water flow in trench is less than 1 litre/sec.	$F_{10T} > 4{,}000$ hºC
Insulating panels	$F_{10T} > 3{,}000$ hºC

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LOW-COST PROTECTION AGAINST WATER AND FROST IN TUNNELS

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ABSTRACT
A large number of Norwegian road tunnels are situated in areas that are subject to severe frost. Problems invariably arise in the part of the frost zone of the tunnel in which leaks do not freeze solid. The accumulation of ice along walls and ditches often blocks drainage paths, causing water to spill onto the road, where it freezes, making driving hazardous and increasing maintenance difficulties. The problem of water leakage has been solved by the Norwegian Road Research Laboratory by means of shielding. Since the first shielding sections were developed in the early 1970s, more than 25 km of shielding has been mounted in Norwegian road tunnels.
Various types of shielding have been tested in experimental tunnels, and the most efficient types have proved to be insulated panels of aluminium, and a sandwich structure of fibre-reinforced polyester. Another method that has proved suitable, especially in limited areas of leakage and in old, narrow tunnels, is the use of polyethylene (PE) foam. Sheets of foam are impaled on rockbolts and pressed directly against the rock crown.
The protective systems used, and the experience of the past ten years, are described in this paper.

INTRODUCTION
Of the more than 700 road tunnels in Norway, some 450 are on the primary national road network, and the remainder on country and secondary roads. Less than 3% of the total tunnel length has zones with in-situ concrete lining. In many tunnels the cost of permanent support amounts to 40-50% of the total costs, which cover water and frost support.
Various forms of shielding are used, first and foremost in tunnels that are difficult and ex pensive to insulate by means of shotcrete injection, and where considerable demands are made of insulation. One prerequisite is that the rock be sufficiently stable for it not to be necessary to line it with concrete and a membrane. It is therefore of great importance that unstable rock be reinforced with bolts, bands, and if necessary, netting.
Reinforcement of the rock must be thorough: once the shielding has been erected it is no longer possible to survey the rock for purposes of supplementary reinforcement. To ensure that the structure is stable and its appearance acceptable, the shielding should cover the whole tunnel profile, from the base of one drainage ditch to the other. Installation is simplest and most secure in tunnels with a circular cross-section, but the method can be applied to any cross-section. The sheets or elements used are uniformly curved, which simplifies mounting, ordering and storage.

MATERIALS
Shielding materials should be corrosion resistant and fire-proof. Aluminium has proved to be suitable and is the material most widely used in Norwegian road tunnels, but some steels and plastics can also be employed. The thickness of the sheets ought not to be less than 0.5 mm. The aluminium sheets used in Norway have a thickness of 0.71 mm. The aluminium has a relatively high magnesium content, to make it resistant to salt water. The sheets are also treated with aludin and coated with baked enamel for additional protection against corrosion from copper-containing water. The aluminium sheets used have sine corrugation -a principle that should also be adopted if steels or plastics are used. They divert leakage, and the form also lends itself to geometrical

adjustments during installation, especially on bends. Hot-galvanized steel and aluminium can be used for bearing rails. The latter is the more commonly used material because it is easier to handle and mount. If different types of material are used in combination, the possibility of galvanic tension developing has to be taken into account.

The materials that are most widely used as sandwich elements are glass fibre-reinforced polyester, with polyurethane as the insulating core. With these structures it is particularly important that fire-resistant additives be used. The polyester should display satisfactory resistance to fire and to the spread of flames{aluminium trihydrate is also favourable in this respect). In general, it is important not to use halogenized polyesters in any situation in which fibres would produce poisonous chlorine and bromine compounds. Polyurethane insulation must not drop during fires, and materials must qualify as being self-extinguishing. A third method of shielding involves the use of polyethylene {PE) foam. Extruded PE foam has several properties that make it suitable for water and frost protection purposes, the most important being its good insulating capacity and low moisture absorptivity. The head conductivities of the materials are in the region of 0.033-0.055 W/m^{2}K.

TYPES OF STRUCTURE

Aluminium shielding

Two types of aluminium shielding have been developed: one is single sheeting without insulation for use in frost zones; the other, the double sheeting, is made of parallel pairs of corrugated sheets with 10 cm of insulating material between them. The outer sheet is held rigid by means of bow-shaped steel-pipes. The bearing section of the shielding consists of bearing rails of aluminium with a "C"-shaped profile, which are fixed by means of key-headed bolts (dowels} 20 mm in diameter. The location of the bearing rails is indicated in Fig. l. The rails must be mounted very precisely, the location being determined by means of measuring equipment, including a specially designed template.

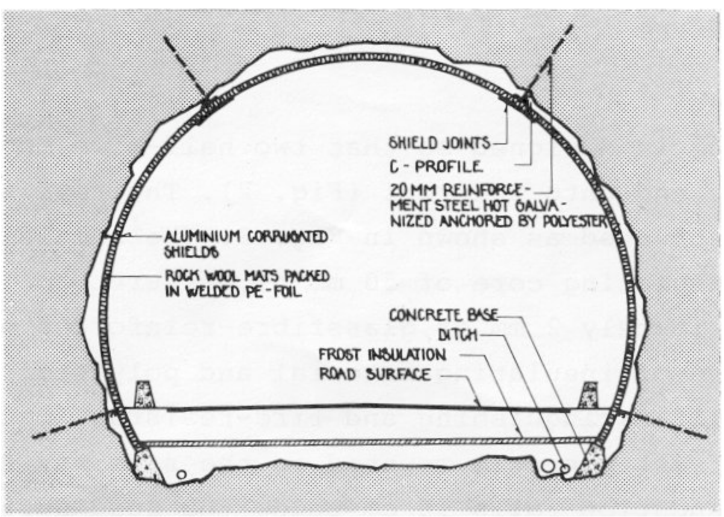

Fig. 1. Double lining of aluminium with frost insulation in between.

Each sheet is mounted on seven bearing rails. Four bearing rails are used for the double shielding, plus two foundation rails installed 50 cm below the road surface. The foundation rails must be grouted in place every 3 m.

Rivets are used to join the corrugated aluminium sheets together and to the bearing rails. To prevent leakage around the rivet holes, sealed rivets with steel nails are used. The automatic machines that are now available simplify riveting work and enable greatly increased productivity. Thread-forming (Drilkvik) screws have been tested, but they proved to be unsuitable for fastening sheets onto the bearing rails. The sheets used in Norway are 871 mm wide and about 6000 mm long, and are profiled to the radius of the individual tunnel profile. The sheets are laid so that two corrugations overlap and are riveted together. In mounting the double sheets, the inner sheet is erected first. Then the insulation and outer sheet are added. Up to the present, mineral wool mats (Glava, Rockwool) have been used for insulation. The mats are lo cm thick, and to protect the material against damp it should be lined with 0.15-mm thick polyethylene foil.

The outer sheet is fastened in such a way that it is self-bearing in principle. To make the shielding rigid, bow-shaped pipes are mounted every 3 m: they are fastened to the foundation rail and can be tightened so that they follow the radius of the shielding snugly. Lighting equipment can also be suspended from the curved pipes.

Sandwich shielding

Sandwich shielding is designed so that two half-bows meet and are fastened together and onto the roof (Fig. 2) .The roof suspension mechanism is constructed as shown in Fig. 3. The shielding itself consists of an insulating core of 50 mm of polyurethane foam, fully lined with approximately 2 mm of glass fibre-reinforced polyester. Both in the choice of insulating material and polyester, the emphasis is on self-extinguishing and fire-resistant materials. Shielding in frost zones is mounted in the same way as aluminium sheeting -on foundation rails 50 cm below the road surface.

Foundation rails must be grouted in place every 3 m, but it is also possible to use "foundation feet" on the jacking principle (Fig. 4) .

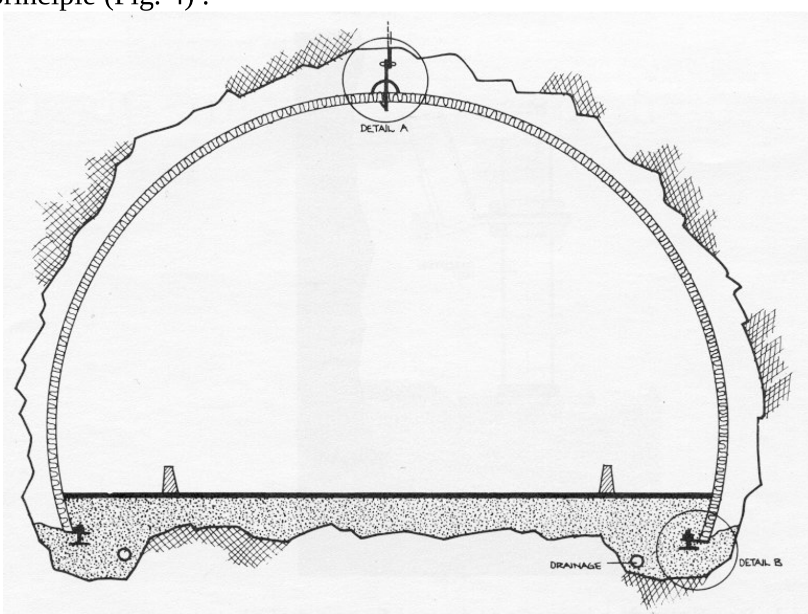

Fig. 2 Sandwich structure -basic principles

The sheeting is fastened to a "jack", which enables the height to be adjusted during foundation laying. After the sheeting has been installed the jack must be grouted in place or fastened to a grouted foundation with bolts

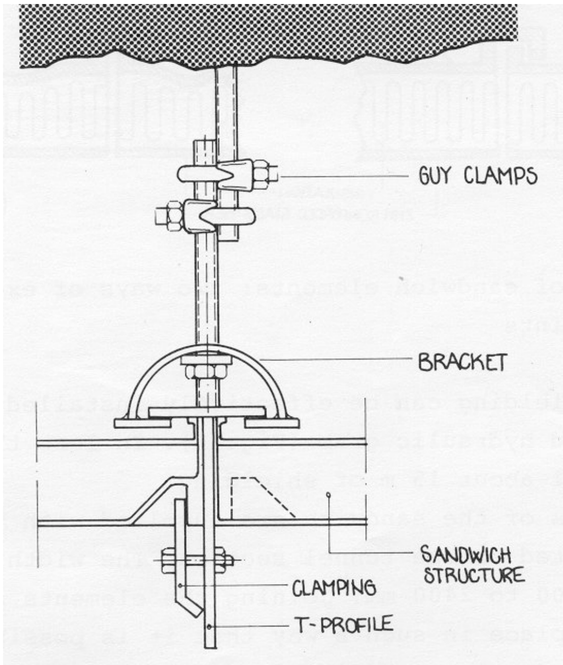

Fig. 3. Detail A: Suspension from roof of sandwich structure

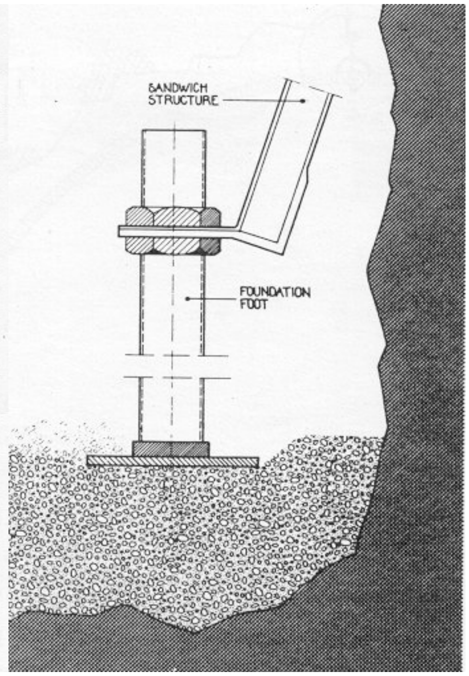

Fig. 4. Detail B: Foundation footing of sandwich structure

Joints between the elements are watertight, and to prevent frost penetration they are sprayed with polyurethane foam (Fig. 5)

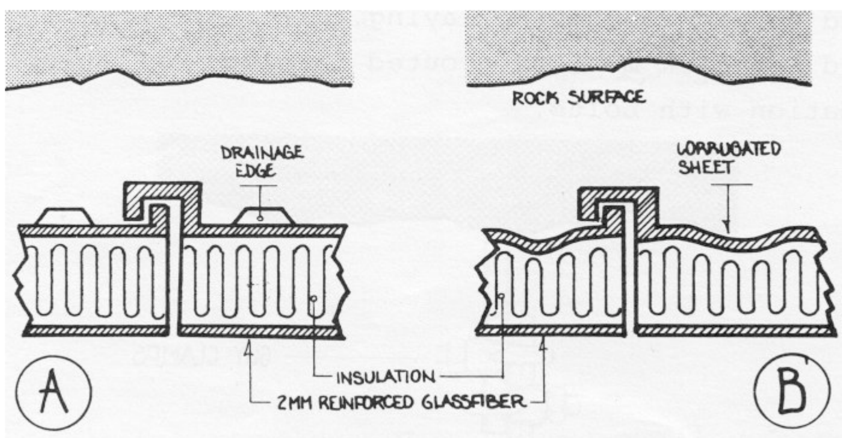

Fig. 5. Joints of sandwich elements: two ways of excluding water from joints.

Sandwich shielding can be effectively installed with the aid of a truck-mounted hydraulic grab (Fig. 6) .In less than 6 hours three men can install about 15 m of shielding. The elements of the sandwich are supplied with various cross sections, adapted to the tunnel section. The width of the elements varies from 1500 to 2400 mm. Joining the elements along their lengths takes place in such a way that it is possible to mount the elements on bends with a radius as small as 300 m.

Fig. 6. Mounting of sandwich elements

Plugging the ends
With both types of shielding, the gap between the rock and the sheeting must be plugged at the ends so that cold air does not enter behind the sheeting. The basic principle is that the heat of the rock must be retained so that water leakage is diverted into the drainage system without freezing. Two methods are currently employed to plug the ends of the shielding. The first involves packing them with plastic-covered rock-wool mats, which are fastened in place with rockbolts and steel wire. The second method employs mats of FE foam, which are bolted to the rock about 500 mm in front of the edge of the shielding. The mats are then bent down against the shielding and forced into place with L-profiled aluminium brackets (Fig. 7) .The

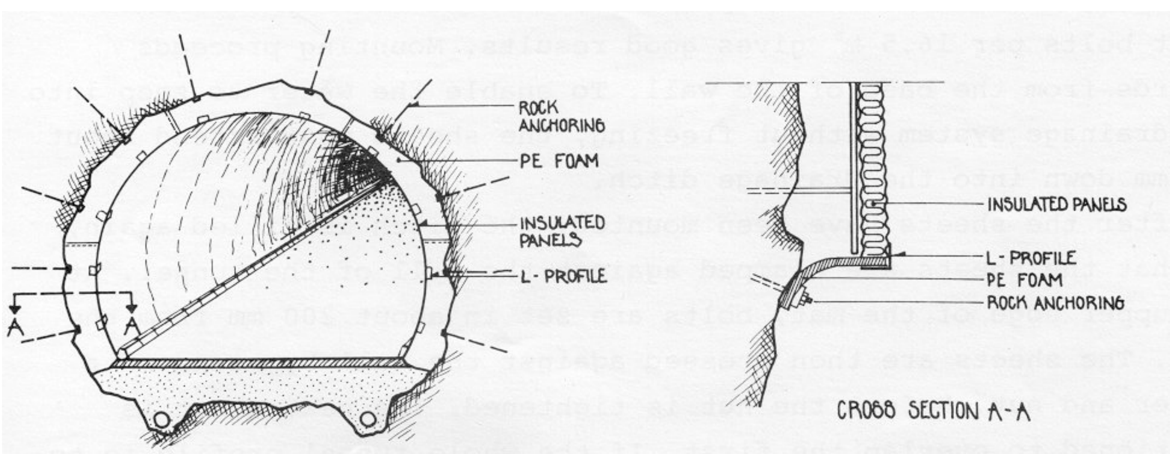

Fig. 7. Plugging ends by means of PE foam

mats are cut into shape so that they follow the profile of the shielding, and are laid overlapping each other so as to cover the whole section.
Experience to date indicates that the latter method is superior in terms of both function and appearance.

PE foam

The two previously described methods give total protection against water and frost w hen the materials are correctly dimensioned and mounted. As a supplement to these methods, a simple technique that does not offer complete watertightness has been developed, for use in old, narrow tunnels in particular, and for diverting water in limited areas of leakage.

Extruded PE foam has several properties that make it suitable for water and frost protection purposes. It is essential that the material exhibit low moisture absorptivity, and flexibility and tensile strength are also very important. The various PE-foam manufacturers supply the material in sheets of somewhat varying size, but it is a simple matter to glue or fuse sheets together to achieve the desired size. For ease of transportation and installation, experience has shown that sheets of about 15-20 m^2 are suitable. They can be glued or fused together at a separate site or can be supplied ready for use by the manufacturer.

The foam sheets can be mounted directly onto the tunnel lining by impaling them on bolts set in the rock, or by piercing the material and installing the bolt after it has been positioned. The number of bolts depends on the evenness of the rock contours and the size of the sheets, but it is important that the sheets fit snugly against the rock. In practice, it is found that five to eight bolts per 16.5 m^2 gives good results. Mounting proceeds upwards from the base of the wall. To enable the water to seep into the drainage system without freezing, the sheets must extend about 500 mm down into the drainage ditch.

After the sheets have been mounted, the ditch is filled again, so that the sheets are clamped against the wall of the tunnel. At the upper edge of the mat, bolts are set in about 200 mm from the edge. The sheets are then pressed against the wall by means of a washer and nut. Before the nut is tightened, the next sheet is positioned to overlap the first. If the whole tunnel profile is to be covered, one can built up the sheets from both sides, taking the roof last. To make the construction even more rigid, steel bands are added between the bolts (Fig. 8), This is particularly important on the outer edges, where the space between rock and sheet must be filled either with rock wool or with pieces of PE foam.

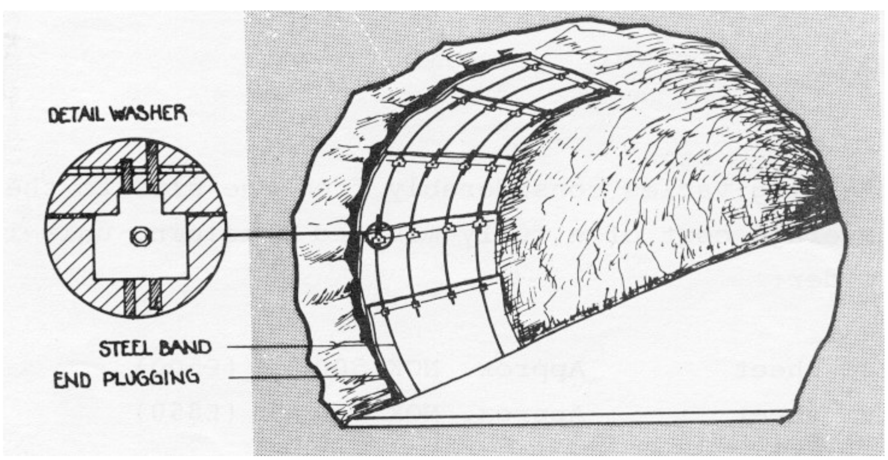

Fig. 8. PE foam - basic principles.

When covering areas that are longer than the length of the sheets they must also be overlapped lengthwise. If the tunnel has a slope the overlap principle must be applied along the downward slope.

The greatest weakness of the method lies in the insertion of bolts through the sheets: to date no good method has been discovered for piercing them without causing leakage. Work is being done on this problem, and exploitation of the properties of laminated sheets offers hope of better results.

There is also hope of finding materials that can be used for plugging around the bolts, to eliminate or reduce the leakage problem.

Estimation of fire hazard

Polyethylene foam is a plastic material, and as such inflammable. The risk of fire has been assessed on the basis of a literature survey, our own experience, and statistics relating to car fires in Norwegian road tunnels. In 1982 the Norwegian Road Research Laboratory cooperated with the Norwegian State Railways in a realistic fire experiment in a disused railway tunnel. It was noted that the element of risk associated with the development of temperature, gases and smoke was very little greater than that which develops with an ordinary car fire. Most of the materials can be supplied in so-called flame-reducing or self-extinguishing varieties. If halogens are used as additives, bromine and chlorine, in particular, will be released by fire. We have concluded that these heavy poisonous gases may be an added hazard. Consequently, we have not recommended the use of any of these materials.

COSTS

The cost of shielding varies considerably from one site to the next, but the average cost of a fully-mounted shielding unit is of the following order:

Single aluminium sheet	Approx. NOK 5000/m (E500)
Double aluminium sheet (insulation included)	Approx. NOK 8500/m (E850)
Sandwich sheet	Approx. NOK 8500/m^2 (E,850)
PE foam	Approx. NOK 250/m^2 (E25)

For the aluminium and sandwich sheets, the cost of plugging the ends must be taken into account (about £500-3800 for each end that is plugged) .

Labour and materials are approximately as follows:

Aluminium	-materials, 35%; labour 65%;
Sandwich	-materials, 90%; labour 10%;
PE foam	-materials, 35%; labour 65%.

EXPERIENCE TO DATE

During the past decade the number of motor vehicles has doubled, and the transportation of goods on Norwegian roads has increased by an average of 5% per annum. Thus the proportion of large and heavy vehicles has increased steady. A larger percentage of new tunnels are therefore being constructed with the maximum cross-section specified in the Norwegian Road Tunnels Standards. This, combined with the additional traffic, has resulted in increased dynamic strain on the shielding, and in re cent years it has been necessary to reinforce structures and also revise design specifications to meet present-day requirements.

Three or four years ago, damage to aluminium shielding, especially the single, un-insulated type, were reported. Five causes for this damage emerged from a detailed investigation:

l) Damage caused by cars and lorries
2) Damage caused by ice {un-insulated shielding mounted in the frost zone)
3) Incorrectly mounted shielding
4) Insufficiently strong rivets
5) Insufficient stability

As mentioned earlier, points 2-5 can be solved through reinforcement and revision of detail. Point l) is not so easily dealt with, but the best way of protecting shielding is to use a solid guard rail to prevent vehicles from leaving the carriageway.

Experience with standard structures to date is very good, despite the relatively short observation period, and we expect the use of sandwich shielding to increase. With regard to PE foam, it may be said that a successful result is very dependent on precise, prudent mounting. The method is most suitable for leaks in limited areas, but it has also been used successfully over larger areas.

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EXPERIENCES FROM EMPLOYING AUTOMATIC GATES: TEMPERATURE CONTROL IN TUNNELS ON THE STRYNEFJELL ROAD

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In most road tunnels there is a greater or lesser amount of water leakage. With the sort of temperatures we have in Norway, measures for draining off water normally have to provide insulation against frost as well. We have learned from experience that frost can make itself felt even in the middle of long road tunnels. One example of this is the year-round Stryn-Ottadalen road, commonly known as the Strynefjell Road.
There are three tunnels on the Strynefjell Road:

	Length	Height above sea level
Ospeli Tunnel	2605 m	approx. 475-555 m
Grasdal Tunnel	3660 m	approx. 660-830 m
Oppljos Tunnel	4506 m	approx. 880-940-930 m

The heights given here are from west to east.
Shielding was placed and implemented as protection against water and frost. From the mouth of the tunnel, and as far in as frost was expected to penetrate, double-insulated shielding was used. Further in single, non-insulated shielding was used.
The road was opened to traffic in autumn 1978, and already the first winter the shielding and electrical installations suffered considerable damage from frost. This was particularly great in the Oppljos Tunnel, but was also extensive in the Grasdal Tunnel.
It turned out that there was far greater penetration of cold than had been anticipated. Right in the middle of the Oppljos Tunnel air temperatures down to -15°C were measured.
After the first winter, then, it was clear that measures would have to be taken to prevent damage of this nature. Through the Public Roads Administration we learned that in a couple of tunnel systems in other parts of the country, gates had been tried out, and appeared to be effective.
One of these tunnels was on the Suldal road system in Rogaland. In this tunnel {which was only used by access traffic) there had been major problems due to icing up. Before the winter of 1978-79 an automatic gate had been installed, and the ice problem appeared to have disappeared completely. It should be mentioned here that the flow of air through the tunnel placed far greater strain on the gate than expected. The gate originally installed broke down very soon, and had to be replaced by a far stronger one.
The road over the Stryn Mountains does not have much traffic, and since its opening it has been closed at night in winter. Consequently a gate would be very suitable. Despite positive experience from Rogaland, we were still uncertain whether a gate was the right solution. This is because there are completely different climatic conditions in the Stryn Mountains from around the tunnel on the Suldal road.
Nevertheless, it was decided, on the advice of the Public Roads Administration, that we should try a gate in the Oppljos Tunnel. We applied to the same firm which had supplied the gate for the Suldal road, but in view of previous experience, a major point was made of getting a stronger gate. The electrically operated automatic raise-and-lower gate, with a breadth of 6000 mm and a height of 4500 mm, was installed in November 1979.
It was placed at approximately the highest point in the Oppljos Tunnel, i.e. about 1 km from the eastern portal. This location was selected mainly due to practical considerations. The rock was good, and it was close to the transformer kiosk, so that the excavation necessary for the gate, and

the power supply were relatively cheap. The impulse for opening the gate was generated by means of magnetic loops situated in the road near the gate. There were two such loops on either side. Their distances from the gate were 2.0 and 1.5 m respectively. The gate can also be manually operated if the automatic mechanism should fail. Between the magnetic loops, a stop-line was painted a cross the right-hand lane of the road, and right beside it a sign was erected with instructions to drivers in English and Norwegian to drive right up to the stop line.

In the gate area the speed limit is reduced to 50 km. The approach area and the gate itself are well lit up and the gate is equipped with plenty of reflecting material. When the gate is closed, it is additionally marked with a blinking red light.

It soon turned out that the gate had a good effect on the temperature in the tunnel as long as everything was functioning as it should. During the first winter, however, we had a number of problems in getting the automatic mechanism to work. Faults developed in various components. It turned out that cars with a high chassis did not set off the impulses necessary for opening the gate. In such cases it was natural to operate the gate manually.

The only problem was that after passing through the gateway drivers had to stop and lower the gate again. As a result the gate could remain standing open for long periods. Over-careful drivers who stopped well before the stop line were a similar problem. They failed to pass the loops, and activate the opening mechanism. Despite these difficulties, the gate functioned so well that it was determined to install one in the Grasdal Tunnel as well. By then the gate dealer had developed an even stronger gate, which was installed in the Grasdal Tunnel in autumn, 1980. This gate was also operated by magnetic loops in the road. In autumn 1982 the magnetic loops were replaced by a photo-cell drive in both gates. This has turned out to function far more reliably than the magnetic loops. The problem of over-cautious drivers was also solved through employing photo-cells.

Since there is little traffic on this road during the winter, the use of gates during this period has not caused any particular problems on account of CO gas or poor visibility in tunnels. Air circulation can be regulated to a certain extent by means of a small door in the side of the gate. In certain circumstances it has been necessary to keep the gate open for “airing".

Systematic measurement of the temperature has not been carried out since the gate was opened. Nevertheless there is no doubt that the gates have had a great effect on the temperature in the tunnels, and that money has been saved by comparison with other frost protection measures which might have been used instead. To support this statement we carried out some measurements last winter with the gate both open and closed, in the Oppljos Tunnel.

The measuring points were located as follows:

Measuring point	l: about 100 m from the western entrance
" "	2: about 250 m west of the gate (about 1250 m from the eastern entrance)
" "	3: about 200 m east of the gate (about 800 m from the eastern entrance)

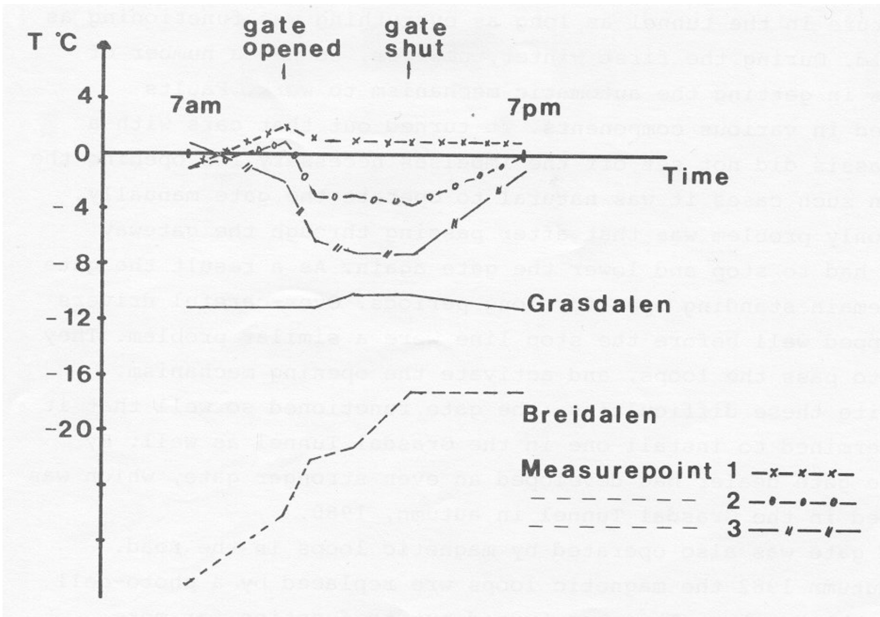

Figure l.
Temperature development in the tunnel January, 27, 1984.

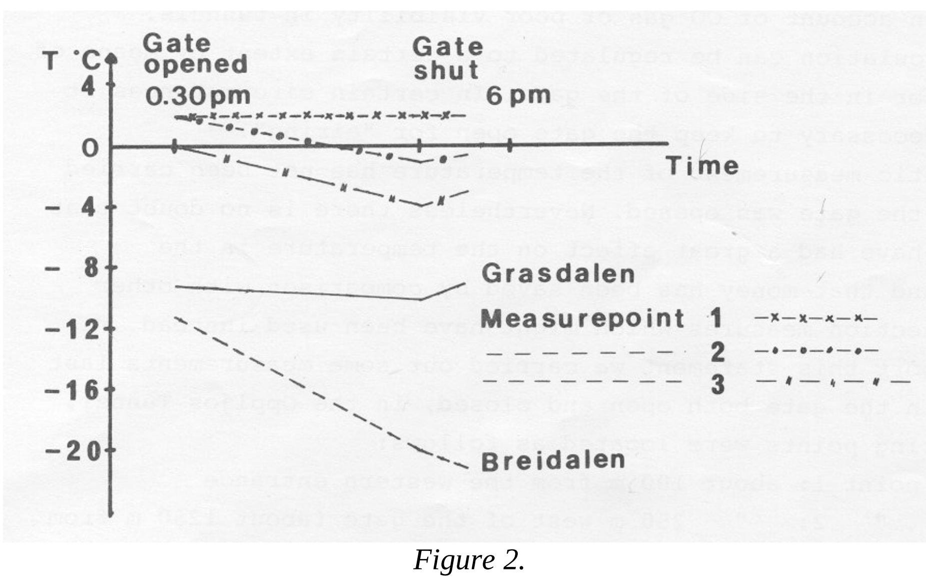

Figure 2.
Temperature development in the tunnel February, 8, 1984.

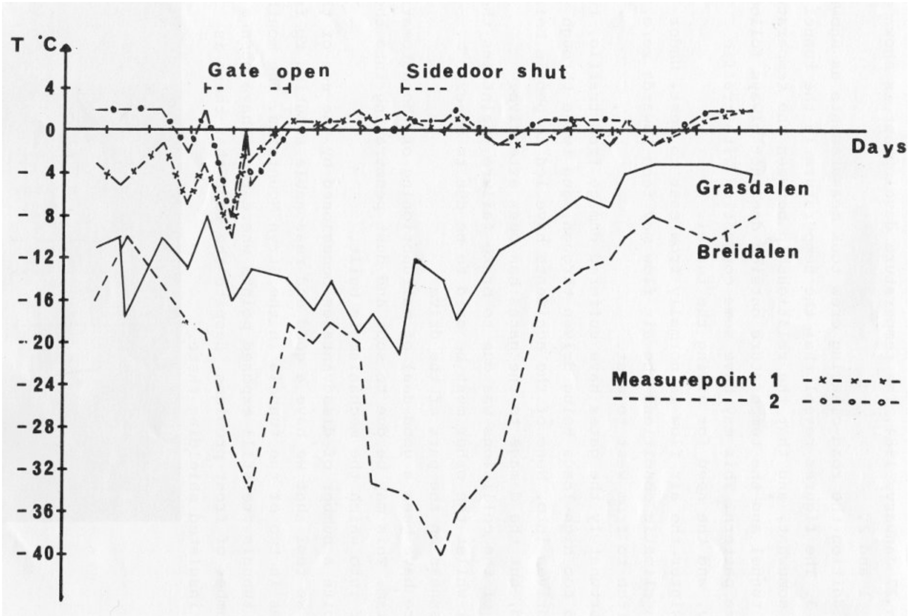

Figure 3.
Temperature measurements in the Oppljos tunnel and in Breidalen (east of the tunnel) and Grasdalen (west of the tunnel. From January 17 to February 2, 1984.

After a relatively long cold period, we opened the gate at 10.30 a.m. on 27 January, 1984. The temperature development is shown in Figures 1 and 2.

In addition the road-clearing crew took measurements as shown on Figure 3. The figures reveal that the temperature in the tunnel varies somewhat, and that the relationship between the temperature in the tunnel and the temperature outside does not always follow the same pattern. This may have some connection with traffic density, and the need for airing the tunnel. In winter the air flow is normally from east to west. Under certain climatic conditions the air flow may tend towards zero, or may switch to from west to east.

Unfortunately the gates have suffered damage from traffic, twice through too high loads being driven through, and twice through being driven into. None of the accidents have led to people being injured, but the damage to the gates has been extensive. One of the collisions was due to brake failure {maintains the driver) whilst the other must be said to be due to sheer carelessness on the part of the driver.

There have been a good deal of malfunctioning of the automatic mechanism. This may be due to soot and dust penetrating into the cabinet into which the mechanism is built.

Despite a number of disadvantages occasioned by the use of the gates, we feel that we have a good and reasonable solution to frost problems in two of the tunnels in the Stryn Mountains. The mouths of the tunnels are still exposed points where there have been a fair number of frost problems, despite the fact that there is double insulated shielding there.

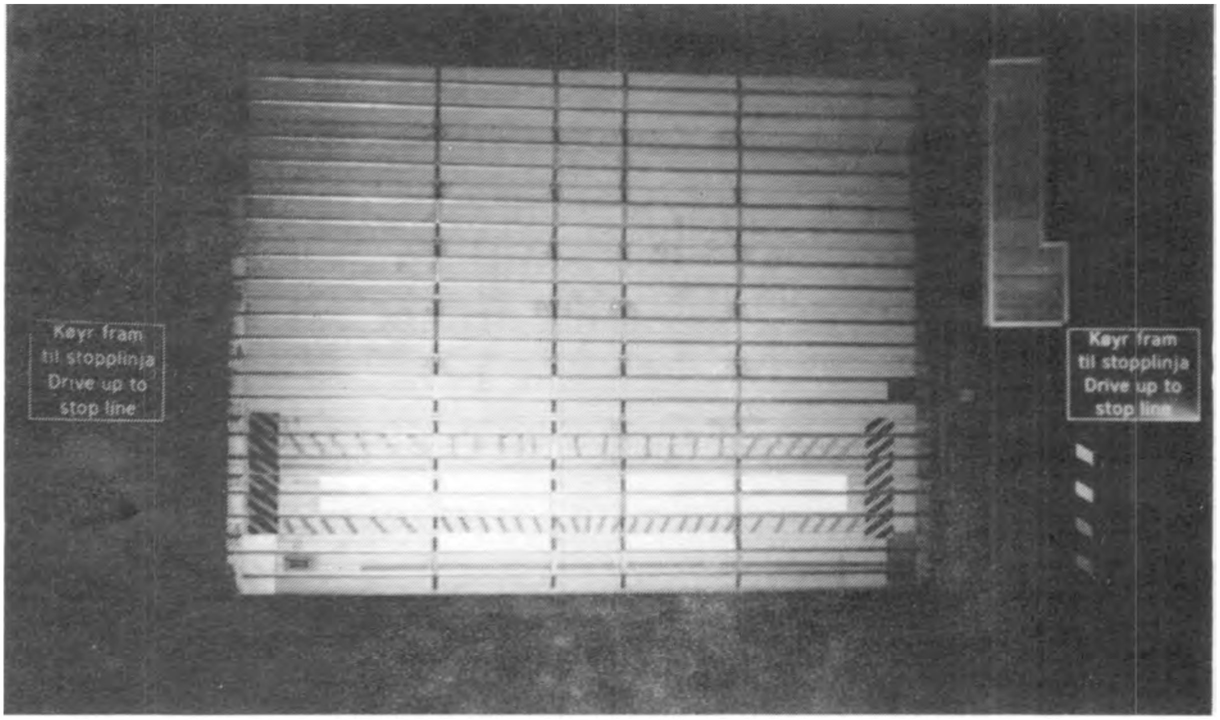

Photo l. From the Oppljos Tunnel

Photo 2. From the Oppljos Tunnel

Photo 3. From the Oppljos Tunnel

PAVEMENT DESIGN FOR ROAD TUNNELS

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ABSTRACT
The design of pavements for road tunnels, as for road pavements in general, is based on traffic loads and the quality of the materials used. Due to the special water and frost problems encountered in road tunnels in Norway, the design procedure is rather special, however.
This paper deals with practice for pavement design in road tunnels with a low traffic density, which is the case for most Norwegian road tunnels. A distinction is made between "dry" and "wet" tunnels, as well as between tunnels (or parts of tunnels) with more or less frost, expressed as freezing index, Fx, measured in h°C.
Extruded boards of closed-pored plastic are frequently used for insulating both pavements and drainage systems in tunnels. Concrete pavements are mainly used for tunnels with an AADT (Average Annual Daily Traffic) greater than 1000.

BASIS FOR TUNNEL PAVEMENT DESIGN
In Norway the index method is generally used for pavement design. This method is based on a division of sub-grade materials into six different groups, ranging from soft clay to rock, and the use of material coefficients to express the load-spreading ability of a specific paving material.
The sum of the products of the material coefficients and thickness of all pavement layers is called the index value of the pavement. The higher the index value, the higher the pavement strength of a given sub-grade.
Traffic loading is expressed as "equivalent 10-ton axles" for the entire expected pavement lifetime, usually ten years.

TUNNEL SUB-GRADE CONDITIONS
The sub-grade conditions in a tunnel will normally be more homogeneous than those of roads in general. The tunnel floor consists of muck overlying solid rock. The formation of fines as a result of traffic during the construction period often causes these masses to be more or less water susceptible, i.e. to become unstable with high water content. Under dry conditions, however, they may have a high stability, and in tunnels without water leakage the pavement may therefore be rather thin. Where water is a problem, it may be necessary to replace the water-susceptible materials with stable, non-frost susceptible materials. However this work is very costly, and will not prevent the formation of ice in the pavement. If water leakage cannot be stopped, the normal procedure is to use plastic boards to prevent frost penetrating beneath the pavement. Extruded polystyrene is normally used, and this is often cheaper than replacing the water-susceptible masses or stopping or reducing water leakage.

FROST IN TUNNELS
In Norway most tunnels will have frost along their entire length in winter. In the lowlands the frost index in the middle of tunnels more than approx. 1000 m long will be low, however, and frost proofing of the pavement will not be required.

Although the temperature variation in a tunnel will only be learnt from experience when the tunnel is completed, it is possible to make an estimate of the future thermal conditions of the tunnel at the planning stage.
In (2) and in the paper "Thermal design in Norwegian road tunnels" published in this volume, a method has been shown by which the frost index at different places in a road tunnel can be estimated (see also explanation at the end of this paper). The procedure is valid only for Norwegian conditions. In order to plan the frost proofing, leakage prevention and drainage systems necessary, the variation of the frost index along the length of the tunnel must be estimated.
The graphs presented in (2) can be used to arrive at design frost indices at various points along the tunnel, provided mechanical ventilation is not installed.
As mentioned earlier, insulation, mainly in the form of boards, is used for frost-proofing tunnels. Table 1 shows where insulation is required on the basis of the frost index in the tunnel.

Frost index, F10 in tunnel {air), h°C	**Frost protection (insulation) requirement**
0-4000	Not required
4000-6000	Required for drainage systems if water runoff is less than l litre/sec.
6000-8000	Required for drainage systems if water runoff is greater than 1 litre/sec.
8000<	Required both for drainage and for pavement

Table l. Insulation requirement depending on frost conditions. Definition of frost index, see end of paper) .

INSULATING MATERIAL
In practice, only insulating boards of extruded, close-pore polystyrene (often Styrofoam HI 50) are used for frost-proofing tunnel pavements and drainage systems. This material is used because it represents a good combination of compressive strength and (very important) moisture absorbing characteristics for its present price. Extruded polystyrene boards less than 50 mm thick are not used because of the increased risk of water accumulation. The compressive strength is specified as min. 350 kN/m^2 (3.5 kg/cm^2), but for certain applications even higher compressive strengths may be required.
The price of extruded polystyrene will vary with compressive strength requirements (i.e. density), and is at present approx. US$ l00/m^3 (excluding VAT), for delivery at the site. Expanded polystyrene is much cheaper, but fails to meet the moisture absorption requirements, unless the thickness is considerably increased. The lower compressive strength would also require a thicker pavement design.

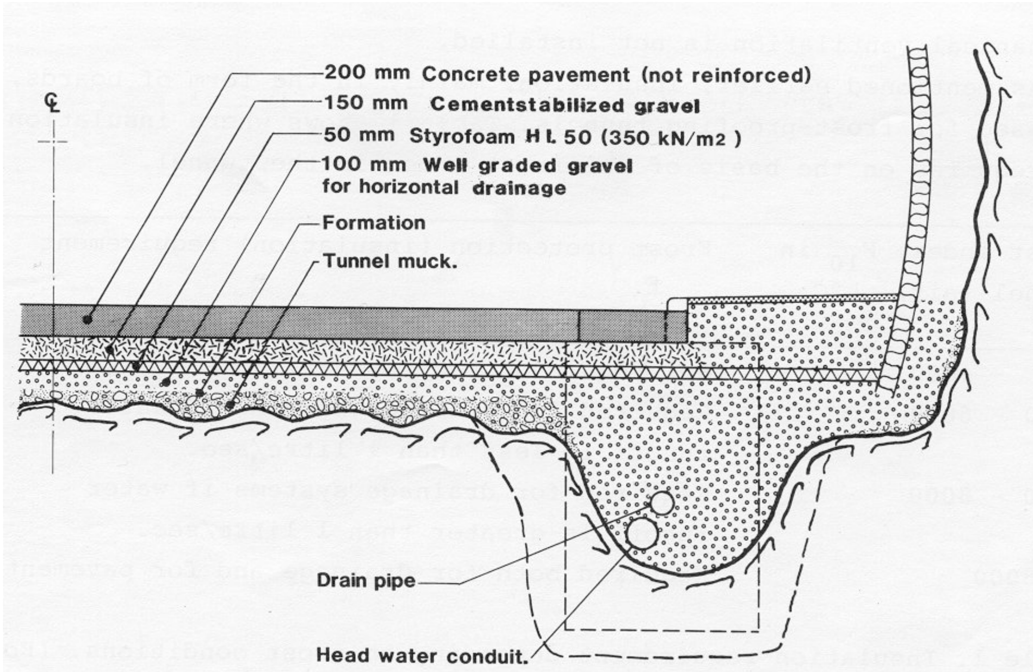

Fig. l. Example of tunnel pavement with insulation boards over the tunnel muck and the drainage system in a tunnel with leakage problems in parts.

During the construction period great care must be exercised to avoid damage to the boards The gravel or cement gravel layer over the boards should have a minimum thickness of 20 cm in order to spread the load sufficiently for boards with 350 kN/m^2 compressive strength Only then may construction traffic be allowed over the boards It may sometimes be necessary to place wooden formwork or something similar over this layer to prevent damage to the boards

PAVEMENT DESIGN

One major question in the design of tunnel pavements is whether asphalt or concrete surfacing should be chosen. Norwegian specifications state that concrete pavements should generally be preferred for tunnels. However, for tunnels with low traffic, asphalt surfacing is normally used. Although the tunnel climate does not favour such surfaces, it appears that the service life of an asphalt pavement is only slightly shorter than that of an asphalt surface outside the tunnel. W hen asphalt surfacing is used, the binder content is normally increased somewhat. Figure 2 shows examples of tunnel pavements for tunnels with an AADT of heavy vehicles of less than 500. The designs are based on the assumption that the frost-susceptible muck is not replaced. When concrete pavements are used, the minimum thickness shown in Fig. 2 for the higher traffic volumes are normally increased, in order to be able to plane down the pavement once or twice following wear due to studded tyres.

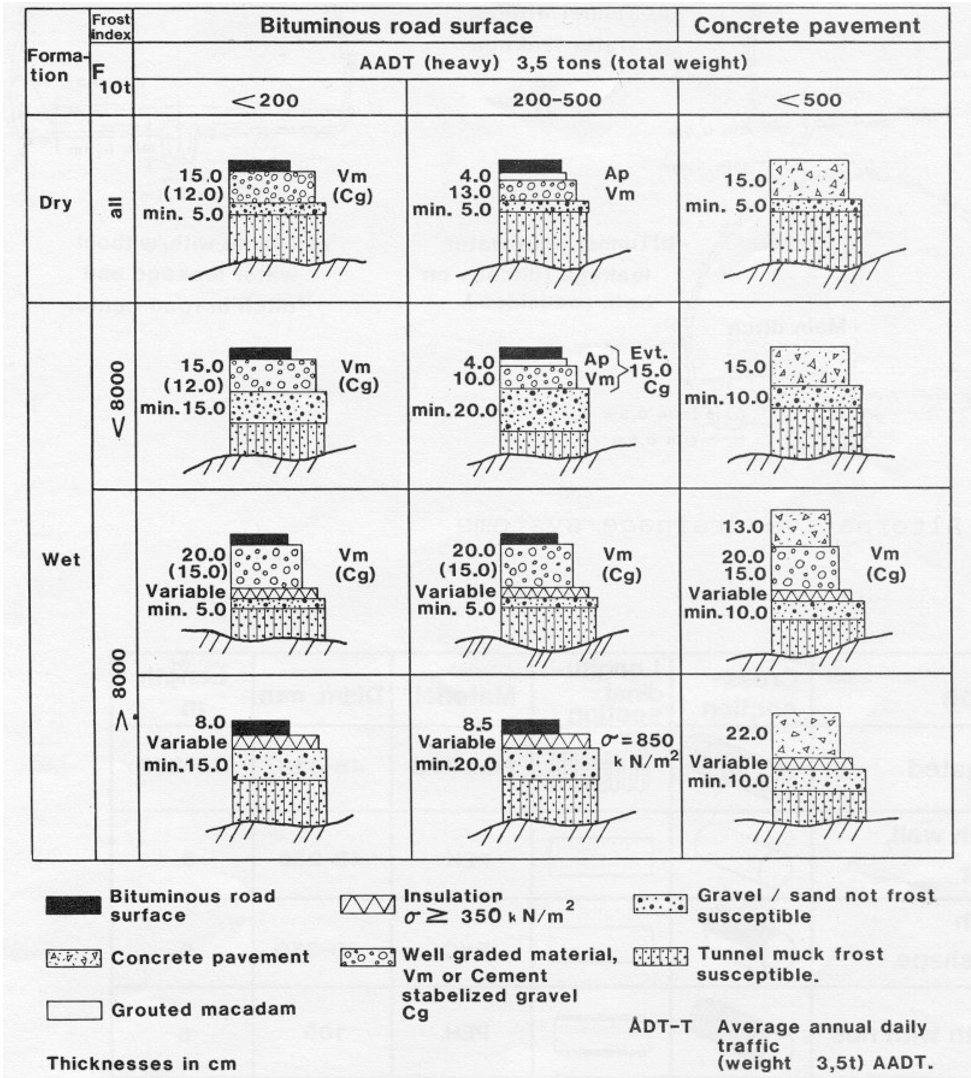

Fig. 2. Pavement designs for road tunnels.
Some examples for AADT (heavy) below 500.

DRAINAGE

Figure 3 shows alternative drainage systems, and Figure 4 various types of plastic drainage pipes.

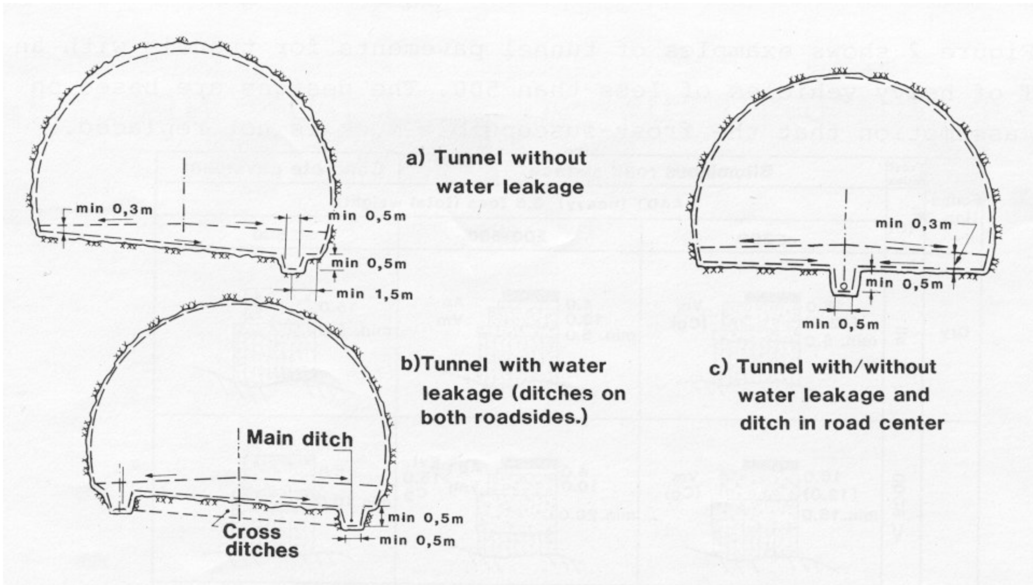

Fig. 3. Alternative Drainage systems.

Shape	Cross-section	Longitudinal section	Material	Diam. mm	Length m
Corrugated			PEH/PVC	48-113	25-250
Smooth wall, circular			PEH	48-235	6
Smooth tunnelshape			PVC	80-350	5
Smooth with ribs			PEH	105	6
Corrugated top . Smooth bottom. Tunnelshaped			PVC	65-150	5

Fig. 4. Various types of plastic drainage pipes.

EXPLANATION OF FROST INDEX

Accumulated frost, calculation:

Accumulated frost is defined as the time integral of temperatures below 0°C during the winter. The calculation method is based on summation of monthly mean temperatures:

$F = 730 \cdot \Sigma| \nu_{month} |$: h°C

Condition: $\nu_{month} = 0$°C

F = accumulated frost : h°C

ν_{month} = monthly mean temperature : °C

Accumulated frost is expressed in hour-degrees centigrade, or h°C, based on the average number of hours per month = 24.365/12 = 730. The consequence of a calculation which is based on monthly mean temperatures is that only months with temperatures predominantly below 0°C contribute to the accumulated frost, whilst months with short freezing periods (spring and autumn) will not contribute.

STATISTICAL TREATMENT OF ACCUMULATED FROST

Frost data observed over a period of several years is used for determining the probability that a certain amount of accumulated frost will be exceeded. Anti-frost measures are based on a figure for accumulated frost which will only be exceeded once in a chosen number of years, often connected with the expected lifetime of the construction. The accumulated frost used for the design is determined by the risk of excess frost in a single year being 50%, 20%, 10% or 1%. For instance, a 20% risk of excess in any one year corresponds to the accumulated frost which is exceeded in 20% of a long series of years. In practice this is expressed as one excess per 5 years. It may, of course, happen that in a certain 5-year period the F5 may be exceeded once or twice, whilst in the next period the F5-winter may not occur at all.

Accumulated frost	**Probability of Frost excess in a single year**	**Expected number of excesses during chosen periods**
F2	50% 1/2	Once in 2 years
F5	20% 1/5	Once in 5 years
F10	10% l/l0	Once in 10 years
F100	1% 1/100	Once in 100 years

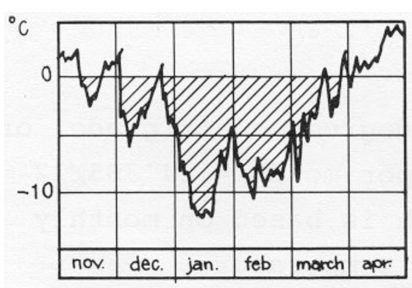

Fig. 5. True air temperature and accumulated frost.

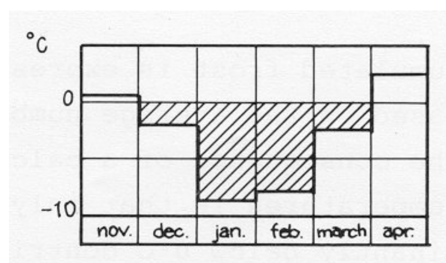

Fig. 6. Calculated accumulated frost based on monthly mean temperatures

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TUNNEL VENTILATION LONGITUDINAL VENTILATION OF ROAD TUNNELS

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SUMMARY
Longitudinal ventilation of road tunnels is by far the most commonly used method in Norway. The main objectives of a ventilation system are to ensure that the concentration of poisonous gases is kept down to an acceptable level, and that the concentration of dust and soot does not cause poor visibility in the tunnel. There are hardly any tunnels which cannot be ventilated longitudinally as a means of accomplishing these objectives. Long tunnels with heavy traffic can be divided into separate ventilation sections by means of shafts, and CO concentration and air flow controlled in each section. In most cases longitudinal ventilation can be accomplished at a lower cost than other ventilation systems.
When considering safety measures, one must distinguish between road tunnels with traffic in both directions, and tunnels with one-way traffic. The choice of safety devices depends on traffic volume and air-flow velocity.

GENERAL
The most usual means of<ventilating road tunnels in Norway is longitudinal (axial) ventilation, with impulse-controlled fans mounted on the crown of the tunnel. Fig. 1 shows the various systems for longitudinal ventilation of road tunnels.
The main objective of any ventilation system is to ensure a safe level of poisonous gases, and also to keep unpleasant smells down to an acceptable level. Another important aspect is to ensure acceptable visibility as a general traffic safety measure.

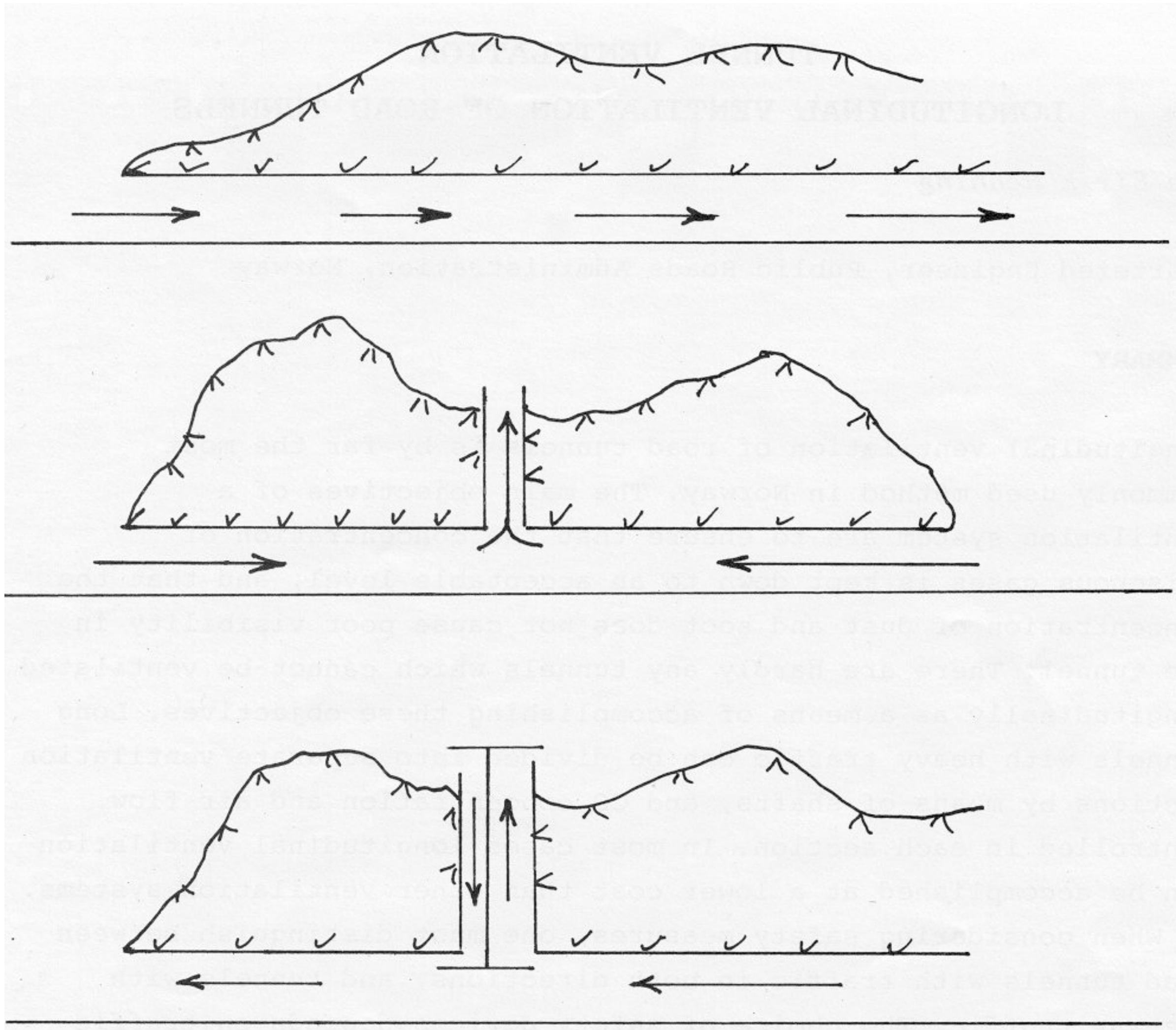

Fig. 1. The longitudinal ventilation system. Powered fans are usually reversible.

When dealing with exhaust gases from engines, it is necessary to determine acceptable upper limits for concentrations of carbon monoxide (CO) and nitrogen dioxide (NO_2). Other poisonous gases hardly ever present a health risk if sufficient dilution of the CO and NO_2 is ensured. Accordingly it may be concluded that all road tunnels can be longitudinally ventilated. Very long road tunnels with a high traffic density can be divided into sections by means of vertical and horizontal ventilation shafts. These shafts should be located so as to ensure acceptable gas concentrations and visibility. Longitudinal ventilation systems can be installed at a much lower cost than other ventilation systems, and in most countries there is an increasing tendency to use them. The Høyanger Tunnel with its length of 7500 m is the longest longitudinally ventilated tunnel in Norway. Its design was based on a traffic density of 250 vehicles per hour, whilst the Holmestrand Tunnel on the E18 highway was based on 1500 vehicles per hour. This latter tunnel is 1850 m long and has no vertical shafts.

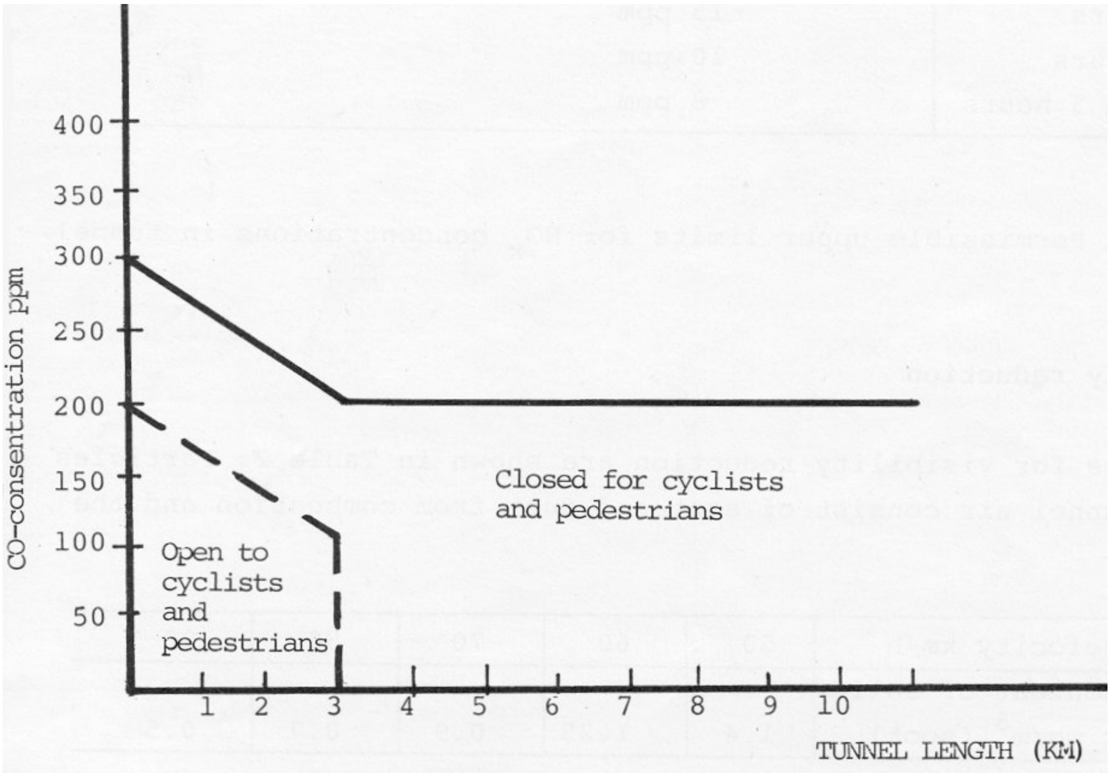

Fig. 2 shows the permissible concentrations of CO

ACCEPTABLE GAS CONCENTRATIONS

The following assumptions are made

- Concentrations of 200 to 300 ppm CO are exceptional, and should never be exceeded, even under very unfavourable traffic conditions.
- Under normal traffic conditions the CO concentration should be substantially lower.

NITROGENOUS GASES

This group consists mainly of nitric oxide (NO) and nitrogen dioxide (NO_2).
Limits for the concentrations of nitrogenous gases are shown in Table l.

Time	**Permissible Concentrations (C_{NOX})**
Less than 20 min	30 ppm
20- 30 min	25 ppm
20 min -1 hour	20 ppm
1- 2 hours	15 ppm
2- 5 hours	10 ppm
More than 5 hours	8 ppm

Table l: Permissible upper limits for NOx concentrations in tunnel air.

Visibility reduction

The limits for visibility reduction are shown in Table 2. Particles in the tunnel air consist of soot and dust from combustion and the road.

Traffic velocity km/h	**50**	**60**	**70**	**80**	**90**
Max. content of solid particles mg/m^3 (soot)	**1.4**	**1.25**	**0.9**	**0.7**	**0.5**

Table 2: Limiting values for reduction of visibility.

REQUIRED VOLUME OF FRESH AIR

The volume of fresh air necessary to uphold the above-mentioned limits can be calculated from the actual amounts of CO, nitrogenous gases and visibility-reducing particles. These in turn depend on the density, composition and speed of the traffic, and the gradient and length of the tunnel.

CALCULATION OF TOTAL CO PRODUCTION

$Q_{oCO} = q_{oCO} \cdot N \cdot k_{hh} \cdot k_s \cdot k_f \cdot L$

Q_{oCO} = Total CO produced (m /h)

q_{oCO} = CO produced per vehicle (0.013 m^3/vehicle/km)

N = number of vehicles per hour

k_a = correction factor for altitude (see Table 3)

k_g = correction factor for road gradient (see Table 4)

k_s = correction factor for reduced speed (see Table 5)

L = length of tunnel (km)

Altitude	**500**	**600**	**700**	**800**	**900**	**1000**	**1100**	**1200**
Correction factor	**1.06**	**1.12**	**1.18**	**1.24**	**1.29**	**1.35**	**1.41**	**1.47**

Table 3: CO production -correction factor for altitude (K_a).

Tunnel gradient

	down%					**up%**		
8	**6**	**4**	**2**	**0**	**2**	**4**	**6**	**8**
0.25	**0.45**	**0.65**	**0.80**	**1.00**	**1.15**	**1.25**	**1.30**	**1.50**

Table 4: Correction factor for road gradient -k_g

Traffic speed, km/h	**5**	**10**	**20**	**30**	**40**	**50**	**60**	**70**	**80**
K_{rs}	**6.3**	**3.5**	**2.0**	**1.5**	**1.2**	**1.1**	**1.0**	**0.9**	**0.8**

Table 5: Correction factor for reduced speed, k_{rs}

The permissible CO production of idling engines is stipulated to be 0.5 m^3/h per vehicle. This quantity corresponds to a fuel consumption of about 1 litre/h.

The necessary volume of fresh air can be computed from the amount of CO (QoCO) produced and the permissible CO concentration (ppm) in the tunnel, which can be found from Fig. 2.

$Qo\ air = QoCO \cdot 10^6 / ppm\ CO = [m^3/h]$

This is the necessary volume of fresh air at atmospheric pressure (760 mm Hg) and a temperature of 0°C. This volume can then be calculated for other atmospheric conditions from the formula:

$\mathbf{Q_{air} = Q_{oair}\ P_Q/P\ .\ T_O/T_1}$

P_0 = Atmospheric pressure (760 mm Hg)
P = Actual pressure (mm Hg)
T_1 = Normale temperature (273 K)
T_0 = Average air temperature in tunnel (K)

NO_x production (calculation model)

$QNO_x = qNO_x\,[N_L + 10N_T]k_g \cdot L$

QNO_x = Total volume nitrogenous gases produced in tunnel (m^3/h)
qNO_x = Volume produced per vehicle: 0.5 .10-3 (m^3/vehicle/km)
N_L = No. of light vehicles per hour
N_H = No. of heavy vehicles per hour
k_g = Correction factor for road gradient (see Table 6)
L = Length of tunnel (km)

The concentration of nitrogenous gases is

$CNO_x = QNO_x/Q_{air}$

Q_{air} is the volume of fresh air necessary to dilute CO gas and visibility-reducing particles in the tunnel. From this volume of fresh air the time (t) for complete air renewal can be calculated. Maximum NO concentrations can be found from Table l.

	Down%					**Up%**	
Correction factor	**6**	**4**	**2**	**0**	**2**	**4**	**6**
	0.5	**0.5**	**0.5**	**1**	**1.7**	**2.2**	**2.8**

Table 6: Correction factor for road gradient (k_g)

Reduction of visibility

$P_{soot} = p_{soot}\,(N_H + 0.08\,N_L)k_a \cdot k_g \cdot L$

P_{soot} = Total amount of soot produced in tunnel (mg/h)
p_{soot} = Soot production per heavy vehicle (750 mg/vehicle/hr)
N_H = No. of heavy vehicles per hour
N_L = No. of light vehicles per hour
k_a = Correction factor for altitude (see Table 7)
k_g = Correction factor for tunnel gradient (see Table 8)
L = Length of tunnel

Altitude	**500**	**600**	**700**	**800**	**900**	**1000**	**1100**	**1200**
Correction factor	**1.12**	**1.24**	**1.35**	**1.47**	**1.58**	**1.69**	**1.81**	**1.93**

Table 7: Correction factor for altitude

	Down %					Up %			
	8	**6**	**4**	**2**	**0**	**2**	**4**	**6**	**8**
kg	**0.5**	**0.5**	**0.7**	**0.8**	**1**	**1.8**	**2.7**	**3.6**	**4.5**

Table 8: Correction factor for road gradient (kg)

$Q_{air} = P_{soot}/\rho_{soot}$ [m^3/h]
P_{soot} = Total production of soot in tunnel (mg/h)

ρ_{soot}= Limiting value for visibility reduction (mg/m^3 -see Table 2)
The pollution component (CO, NOx or visibility reduction) which requires the greatest volume of fresh air forms the basis of ventilation design.

TYPES OF VENTILATION
Types of tunnel ventilation can be divided into three groups:

- Natural ventilation
- Piston effect
- Mechanical ventilation

Natural ventilation
This type of ventilation occurs as a result of pressure differences caused by climatic conditions. Differences in temperature at the ends of the tunnel may be caused by the heat of the sun. In areas where the air becomes warmer, the air will rise and the air pressure will fall, which may cause a difference between air pressures at opposite ends of the tunnel. Depending on the time of the year, temperatures will be different inside and outside the tunnel.
During winter the temperature is higher inside than outside the tunnel, and if the tunnel is not horizontal, a chimney effect will occur, and cause an appreciable flow of air through the tunnel.
In summer the air in the tunnel is colder than that outside, and this causes a flow of air towards the lowest point. Local meteorological conditions, such as force and direction of prevailing winds, will also have an effect on the flow of air through a tunnel.
The piston effect
As a vehicle moves along a tunnel with a velocity different from the tunnel air, a pressure difference will be produced. The shape and velocity of the vehicle is important in this respect. Depending on whether the tunnel has one-way or two-way traffic, and the traffic density, the net self-ventilating effect may be appreciable. A large number of tunnels with one-way traffic are in fact self-ventilating, even at very low traffic velocities. Tunnels with traffic in both directions may in some cases also be self-ventilated, especially if the traffic has rush-hour characteristics.
Mechanical ventilation
Longitudinal mechanical ventilation is based mainly on the use of impulse-governed fans. In long tunnels or where the traffic density is high, or if there are certain limiting pollution concentrations around the ends of the tunnel, ventilation by means of shafts may be used. Even if shafts are used there will be a need for impulse-controlled fans in most cases. This will enable development of a sufficiently high air speed and also control of the air in the tunnel.
The fans are normally mounted on the ceiling of the tunnel, one or more together if the necessary space is available. The distance between these fans or groups of fans should not be less than 80 to 90 metres. It is then possible to establish a stable and even velocity profile between the ventilators. In tunnels with two-way traffic, fans should in principle be reversible, so they can be run in the same direction as the natural ventilation forces. In practice, however, this has proved not to function as well as might be expected, largely due to the complicated sensor system necessary. The reason for this inadequacy is that the natural ventilation current may change direction after the fans

have started running. This would in time necessitate the choice of a longitudinal ventilation system to run against the natural ventilation forces.
In tunnels with two-way traffic and a typical rush hour, ventilation may be made time-dependent, so as to follow the direction of the traffic. By separating a tunnel into sections, using shafts, it is possible to renew the air along the tunnel. In tunnels with one-way traffic the ventilation direction will usually be the same. Combined effects of natural and mechanical ventilation forces.
When designing a mechanical longitudinal ventilation system, attention should be paid to the natural ventilation forces. This may be done in several ways:
The ventilation direction may be automatically determined by the direction of natural ventilation. The ventilation direction may be made independent of the natural ventilation direction. The ventilation direction may be changed manually (or by means of a time switch) according to the periodical or seasonal variations of the natural ventilation forces.

CONTROL SYSTEMS

In order to gain efficient control of air currents, equipment for regulating their volume and direction is necessary. The air volume is usually controlled by varying the number of fans operating. Automatic control of the fans is usually achieved by using instruments for measuring the concentration of CO gas present in the tunnel. Fans previously used to be connected to instruments for measuring visibility, but in practice these instruments proved to be of little use. At present most ventilation systems are connected exclusively to CO meters, and this has been working satisfactorily.

Control of ventilation systems

An example will serve to illustrate that using visibility measurements to control the fans may in fact sometimes make conditions worse. In summer the air in a tunnel is cooler than that outside, and this often causes condensation in the tunnel. This reduces visibility, and the fans start, bringing into the tunnel even more warm air which further reduces visibility.
During winter cars normally have snow chains or studded tyres which cause considerable wear on the road surface, resulting in dust and reduced visibility. Experience shows that this dust accumulates in tunnels during humid conditions and is released when it gets drier. W hen this happens the fans will start and conditions rapidly become even drier, and more dust is released into the air in the tunnel. Good results have been achieved in Norway by adjusting the starting time of the fans, according to CO measurements which depend on the time of the year.

SAFETY CONSIDERATIONS

In Norway, where there are a large number of tunnels with a relatively low traffic density, longitudinal ventilation will be quite safe even in long tunnels. This also applies to short tunnels with high traffic densities.
Tunnels with one-way traffic and two-way traffic are considered separately in relation to safety. With traffic in both directions, an accident resulting in a fire will create traffic queues in both directions. Traffic approaching the accident site against the ventilation direction will be met by smoke and gases from the fire.
An accident in a tunnel with one-way traffic will only cause a queue to form behind the accident spot. In most cases the fresh air current will follow the direction of the traffic, towards the accident spot, and there will be no smoke or gas problems. The risk of fire in a tunnel is not very high, but the possibility has to be considered, and also that high temperatures and smoke will result. Longitudinal ventilation is acceptable w hen safety requirements for categories a) to d) are complied with, and the air velocity does not exceed 7 m/sec.
In two-way traffic tunnels the limiting AADT values selected are 1500, 4000 and 12000 in accordance with the Road Class System. The AADT values are those measured the year the tunnel was opened.

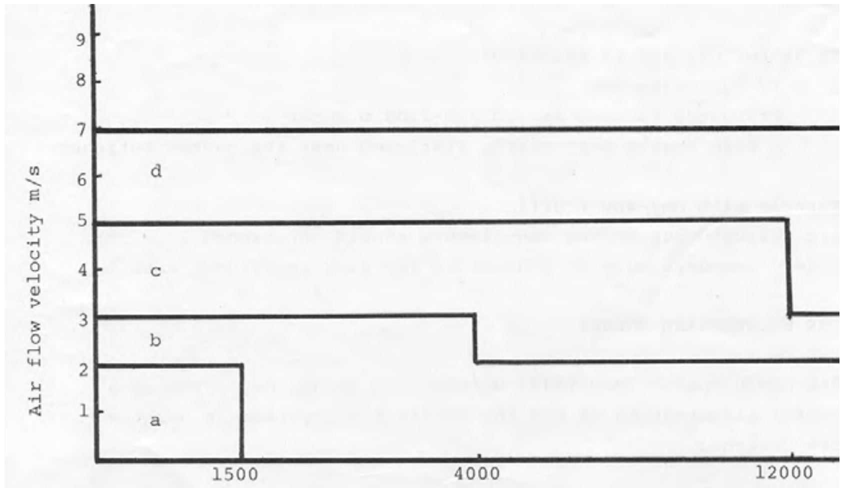

Fig. 3 Safety categories (a,b,c,d) in longitudinally ventilated tunnels with two-way traffic, depending on traffic densities and air speeds.

Examples of safety measures.
An AADT figure greater than 4000 vehicles/day will require an estimated air velocity of 3.0 m/sec., and safety measures as described for category c) below.

Tunnel categories:
a) Tunnels more than 1000 m long must be lighted with a minimum of 0.2 cd/m^2 .

b) Every 500 m an emergency telephone must be installed, together with a fire extinguisher and fire detectors connected to traffic lights and information signs at the tunnel entrances.

c) As for b), but with intervals of 250 m. In addition the fire alarm should govern the fans and lower the air velocity to 2 m/sec. Space for vehicles to turn should be provided at intervals of 750-1000 m, and should be clearly marked. These areas should be combined with space for the transformers. Traffic signs may be installed at the turning areas after taking account of the specific problems in practice.

d) As for c), and in addition:
- -TV surveillance
- -Emergency escapes spaced 500-1000 m apart
- -A fire engine permanently stationed near the tunnel entrance.

Tunnels with one-way traffic
Air velocity due to the ventilators should not exceed 10 m/sec.
Safety measures must be considered for each individual case.

THE HOLMESTRAND TUNNEL
The newly opened (May 1983) Holmestrand Tunnel may serve as a useful illustration of how the ventilation systems in our tunnels are designed.
Ventilation system
The tunnel is longitudinally ventilated, by means of fans mounted on the ceiling.
Because of the built-up area at the north-end of the tunnel, the ventilation direction is southwards. The fans can be reversed if necessary, however.

Control and safety systems

The 18 fans are divided into three groups, forming a three-stage system. The operational phases of these three stages are governed by the CO content of the tunnel air.

Stage l: 6 fans, stage 2: 12 fans, and stage 3: 18 fans.

As a safety precaution, emergency telephones are installed at 250 m intervals, and connected to the Holmestrand Fire Station. The following additional information is supplied to the fire station:

- Predetermined CO level exceeded
- Lighting system out of order
- Fans out of order
- A first aid kit is being opened

The CO concentration can be viewed on monitors in the fire station and at the mid-tunnel drift entrance. The ventilation system and air speed and direction can also be operated from these two positions.

COST OF TUNNEL VENTILATION

Fig. 4 shows the approximate costs of ventilating road tunnels. The following assumptions are made:

- Traffic speed 60 km/h
- Altitude 0-400 m
- Tunnel gradient +/- 0
- Peak hour traffic:
    - A: 250 vehicles/h
    - B: 500 vehicles/h
    - C: 1000 vehicles/h
- Percentage of heavy vehicles 7%
- Two-way traffic with 2/3 against the ventilation direction and 1/3 in the ventilation direction.
- Cross section of tunnel: 45 m^2
- Coefficient of friction loss 0.05
- Hydraulic diameter OH = 7.0 m
- AADT:
    - Curve A: 1500 vehicles
    - Curve B: 4000 vehicles
    - Curve C: 12000 vehicles
- Number of fans per transformer: 6-8
- Control system based on CO measurement
- The costs of electrical equipment and work as well as the fans are based on 1984 prices.

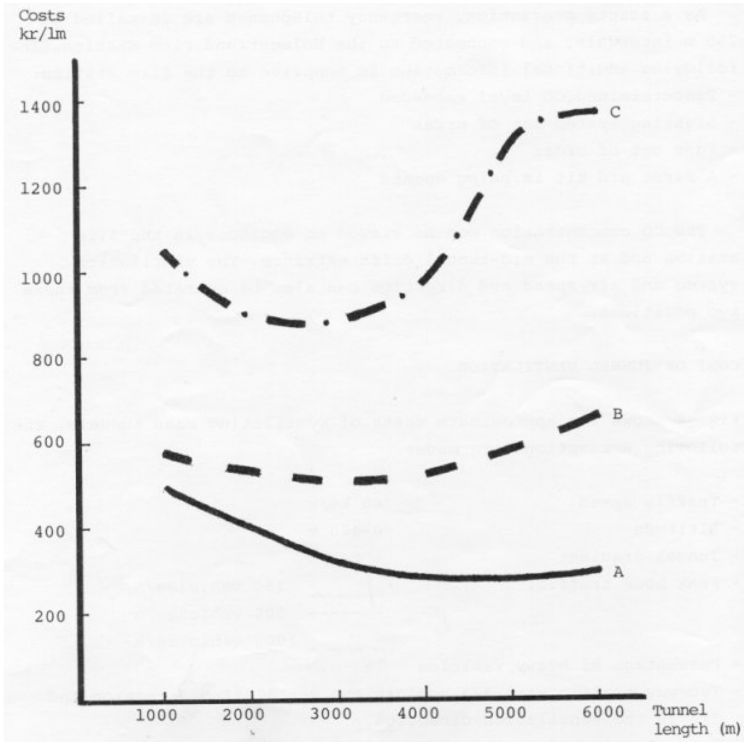

Fig. 4. Cost of ventilation per metre as a function of tunnel length (excluding transformers and power supply lines).

RUNNING AND MAINTENANCE OF THE LØVSTAKK TUNNEL

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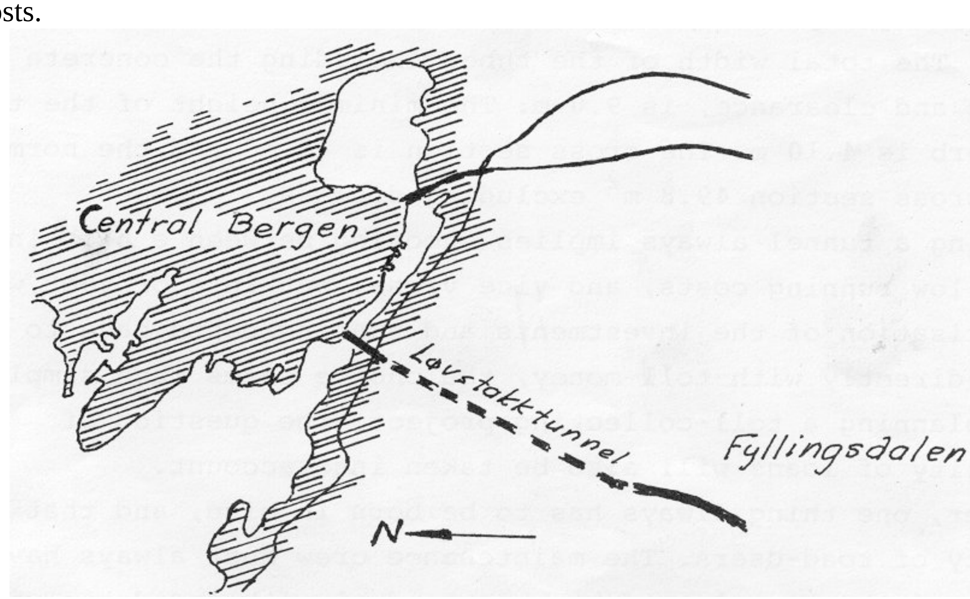
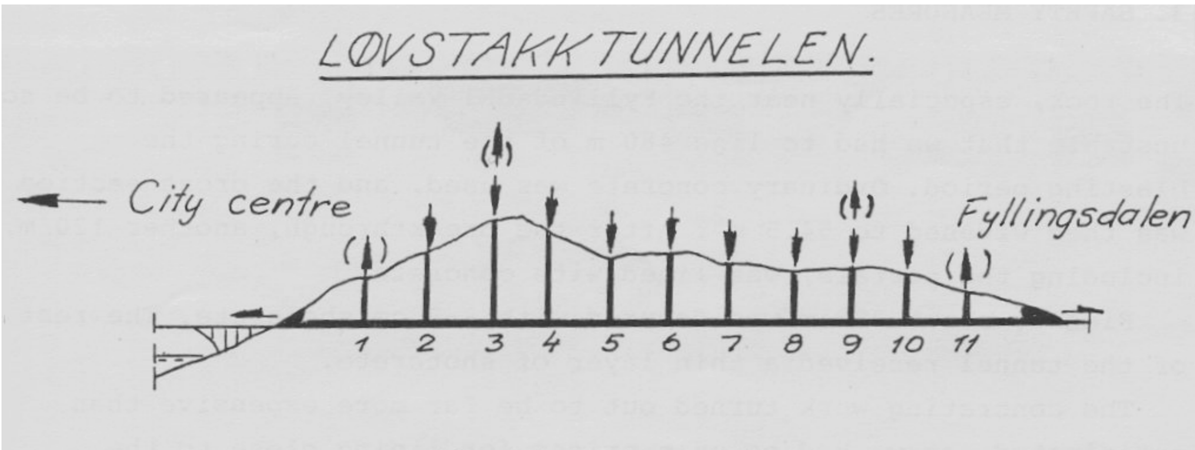
ABSTRACT
Bro- og Tunnelselskapet A/S, of Bergen (Norway) was founded in 1953. The company was originally more or less a private one. Today Hordaland County and the municipality of Bergen are also part owners. The company has built two tunnels financed by toll money. The first of these, the Eidsvåg Tunnel, is 830 m long. It has now been taken over by the Norwegian State. The company is still collecting toll at the second, the 2032 m long Løvstakk Tunnel. This paper deals only with the Løvstakk Tunnel. The paper describes the technical features of the tunnel, with special emphasis on safety aspects. Maintenance of the tunnel and periodic operations are also taken up. The increase in traffic is shown, and also the annual numbers of incidents in connection with the tunnel. Running and maintenance costs are also given. Finally, the construction costs, with percentage cost per item, are shown.

l. INTRODUCTION
The company I represent is called Bro- og Tunnelselskapet, Bergen. It was founded in 1953 for planning, financing, constructing and running communication projects based on toll money. The company's first tunnel project was the Eidsvåg Tunnel. It was opened for traffic in 1956, and was the first artificially ventilated vehicular tunnel in Norway. This tunnel was fully paid off by January 1st, 1975, and is now owned by the State. The company's next tunnel, the Løvstakk Tunnel, was opened in 1968. We still collect toll at this tunnel. The toll is supposed to cover all expenses such as instalments and interest, as well as running and maintenance costs. The only exception is the tunnel lighting. The Bergen Electricity Board deals with lighting maintenance, and it also bears the power costs.

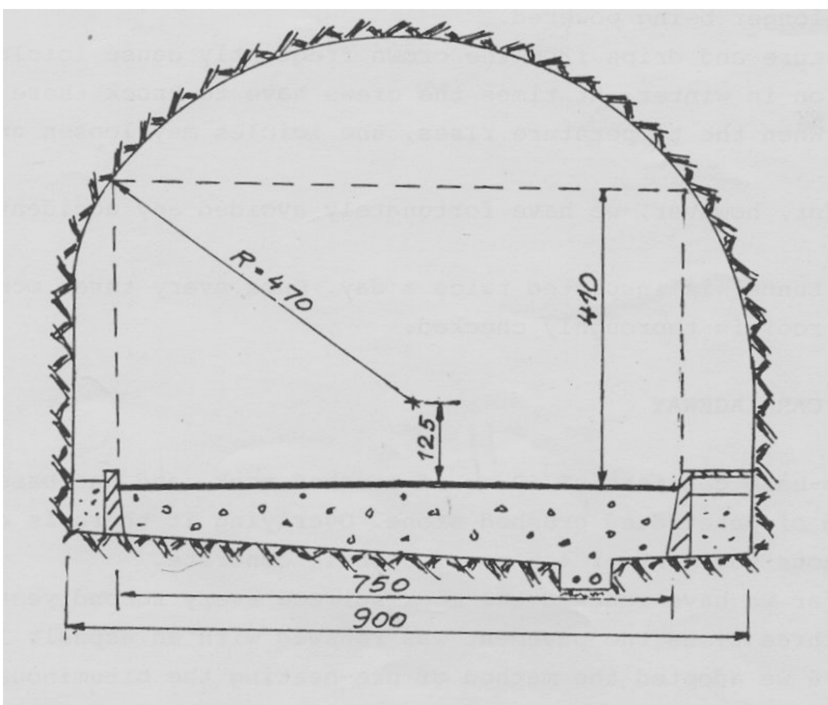


2. TECHNICAL DATA
The tunnel, with its two-way traffic, runs from the outskirts of the city through Mount Løvstakken, south to the Fyllingsdal Valley, one of the new suburbs. The tunnel is 2032 m long. It is straight, except for a gentle curve near the entrance in Fyllingsdal Valley. The gradient is 2,7 % from the city end. The carriageway is 7.50 m wide. On one side there is a 0.9 m sidewalk. The total width of

the tube, including the concrete curb stone and clearance, is 9.0 m. The minimum height of the tunnel at the curb is 4.10 m. The cross section is 45 m^2 and the normal blasted cross section 49.8 m^2 excluding ditches. Planning a tunnel always implies choosing between a high investment and low running costs, and vice versa, but in a project where the amortisation of the investments and running costs have-to be paid off directly with toll money, the choice seems much simpler. W hen planning a toll-collecting project, the question of availability of loans will also be taken into account. However, one thing always has to be born in mind, and that is the safety of road-users. The maintenance crew must always have a full view of the tunnel, and be able to deal with incidents without unnecessary risks to the users. In this connection I shall take a closer look at the safety level in the Løvstakk Tunnel.



Ventilation system.



Normal cross section.

3. SAFETY MEASURES

The rock, especially near the Fyllingsdal Valley, appeared to be so unstable that we had to line 480 m of the tunnel during the blasting period. Ordinary concrete was used, and the cross section was then widened to 54.5 m^2. After the breakthrough, another 120 m, including the portals, was lined with concrete. Finally about 280 m was secured with 3-5 cm shotcrete. The rest of the tunnel received a thin layer of shotcrete. The concreting work turned out to be far more ex pensive than anticipated, as we had no unit prices for lining close to the tunnel face. In order to reduce expenses, we installed arches at spots with water leakage, were the rock was otherwise stable. To avoid water leakage onto the carriageway, we used a longitudinally halved plastic pipe with the open side to the tunnel wall, to drain the water aside to the walls. To prevent freezing in winter, we installed heating cables along the pipes. In the long run this solution has not proved satisfactory. The heating did not help as anticipated, and consequently the cables are no longer being powered. Moisture and drips from the crown frequently cause icicle formation in winter. At times the crews have to knock these down daily. When the temperature rises, the icicles may loosen and fall down. So far, however, we have fortunately avoided any accidents due to ice. The tunnel is inspected twice a day. Once every three months the tunnel roof is thoroughly checked.

4. THE CARRIAGEWAY

The sub-base consists of 60 cm of crushed rock, and the base course of 6 cm of penetrated crushed stone. Overlying it there is a bituminous pavement of 4 .5 cm of asphalt concrete. So far we have renewed the rock surface every second year. The first three times the pavement was renewed with an asphalt layer In 1976 we adopted the method of pre-heating the bituminous surface, which resulted in a considerable reduction of asphalt consumption. A new milling machine was introduced in 1981, and we used it in the Løvstakk Tunnel the following spring. The thickness of the asphalt layer had increased so much that measurements indicate that we can mill the asphalt this year as well. To illustrate this development, I have included graphs showing annual costs. The graphs are based on nominal NOK, and the development of costs for paving work during the same period is shown in a separate diagram. The curves show that despite the substantial rise in prices, we still haven't reached the sums in nominal NOK that we had to pay during the first few years, w hen we merely renewed the asphalt. If we look at the asphalt consumption of those early years and extrapolate according to increasing prices, we could anticipate the development of costs and accumulation of maintenance costs of the carriageway indicated in the diagram. The accumulated saving up to 1984 is estimated to be about NOK 1.3 million. We engage special contractors for the bituminous work and milling. The work proceeds only during the night, between 10 p.m. and 6 a.m., w hen the tunnel is closed for all traffic except police, ambulances, fire-engines and buses.

5. DRAINAGE

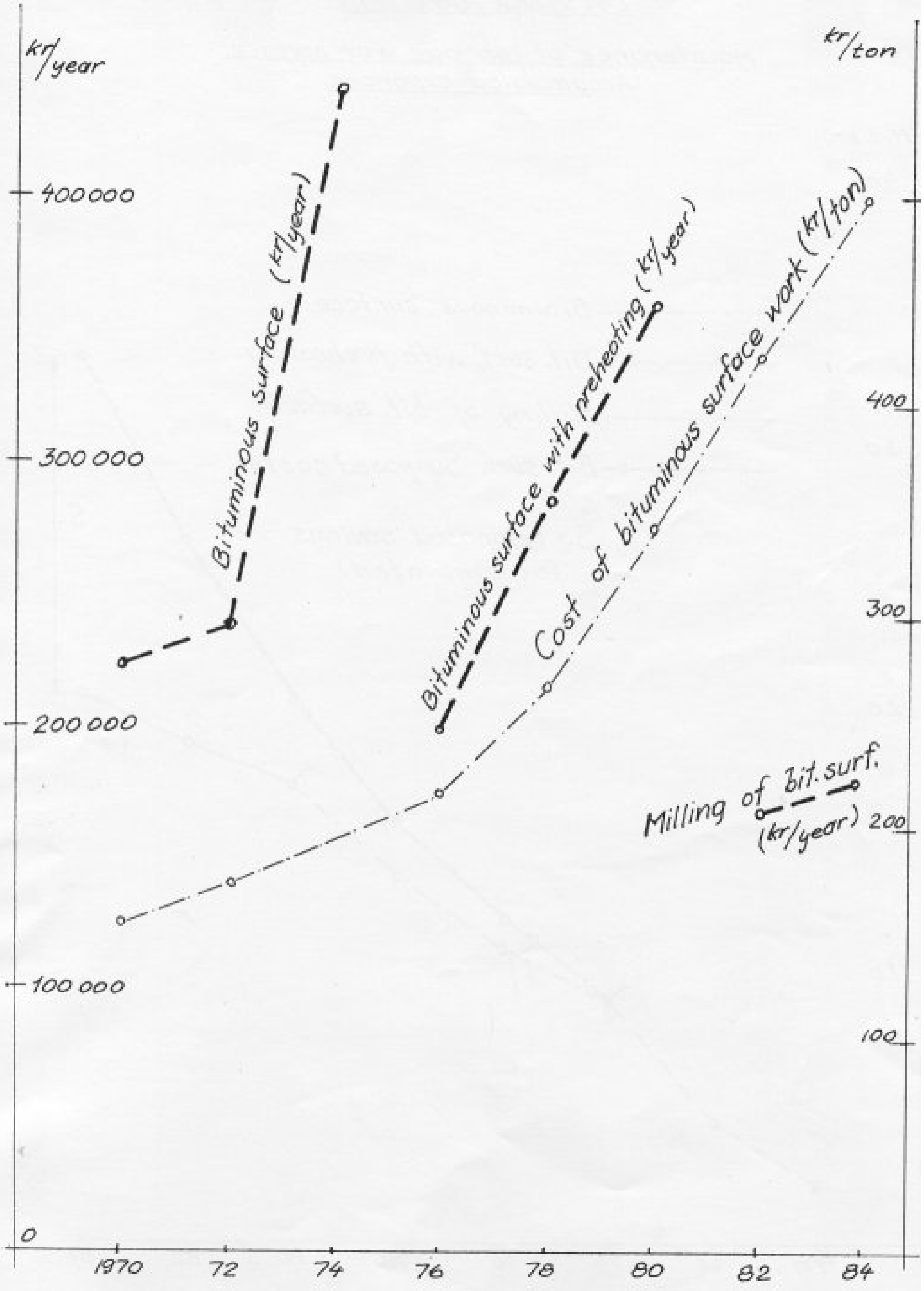
Special emphasis is placed on solving the drainage problem. This is because of the amount of cold air drawn into the tunnel through the shafts during the winter. Along one side of the tunnel there are 9" concrete pipes with socket sleeves and open connections. In addition there are 4" crossing pipes laid every 30 m. Parallel to the drainage pipe there is a 6" pipe dealing with water from the road surface. And lastly there is a drain from the foot of the shafts and connected to the 6" pipe. Gutters have been installed every 160 m. All sand traps are checked and emptied once a month.

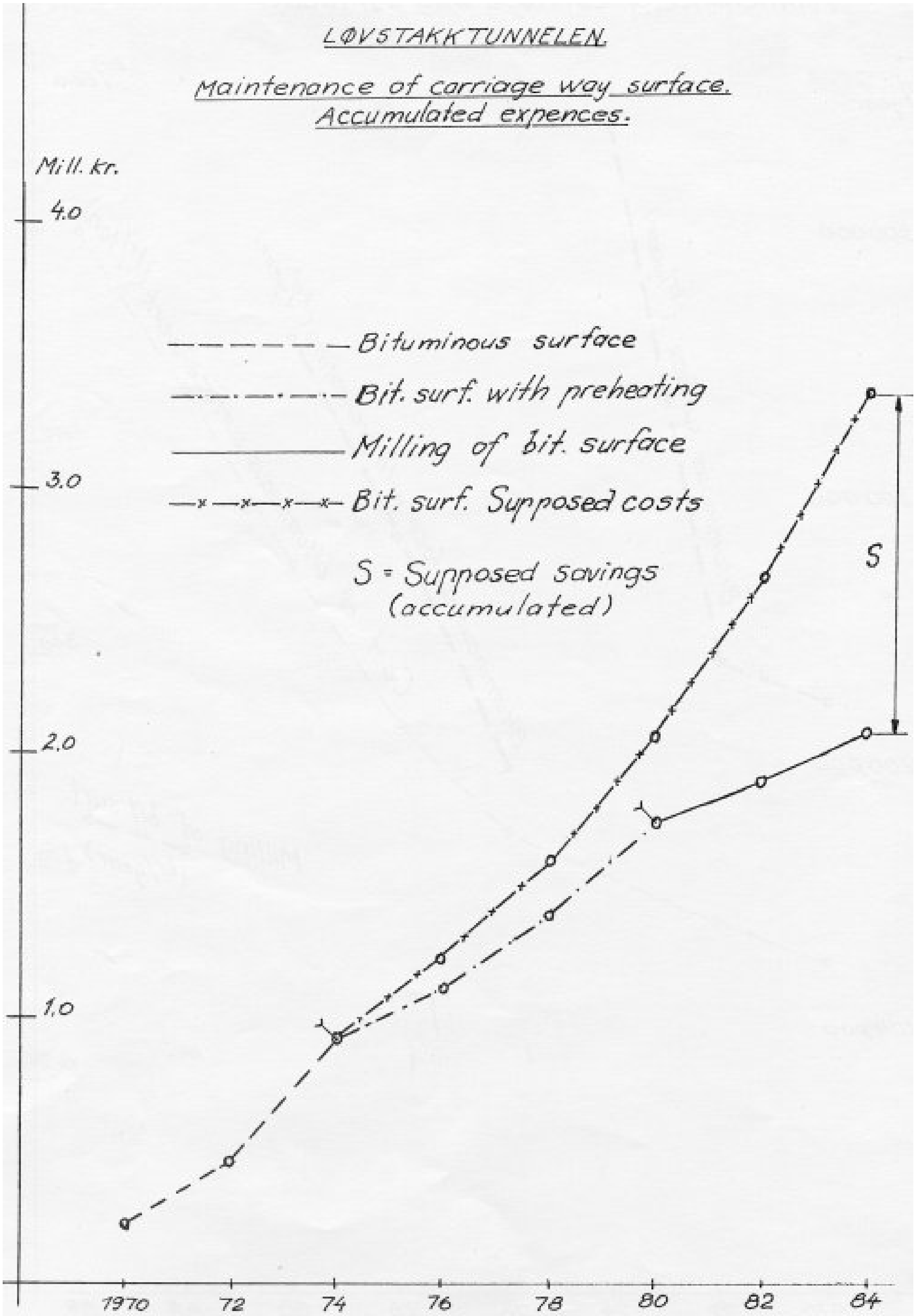
6. VENTILATION SYSTEM

The tunnel is ventilated longitudinally, except for the central section, where a 420 m fresh air distribution duct provides semi-transverse ventilation. The shafts were dimensioned to serve a future second tunnel tube as well. However, it is very uncertain whether another tube will ever be built. It should come, the ventilation system will be purely longitudinal, and just one, or possibly

LØVSTAKKTUNNELEN.

Maintenance of carriage way surface.




LØVSTAKKTUNNELEN.
Maintenance of carriage way surface.
Accumulated expences.
Mill. kr.
4.0
3.0
2.0
1.0
Bituminous surface
Bit. surf. with preheating
Milling of bit. surface
Bit. surf. Supposed costs
S = Supposed savings
(accumulated)
S
1970
72
74
76
78
80
82
84

two, of the shafts will come into service for the second tunnel tube. 15 fans have been installed, each with a power of 50 kW and a capacity of 55 m^3/s. We found it very important to have reliable fans, to avoid extra maintenance and break-downs. Consequently quite an ex pensive type of fan was bought. Today we realise that this was the right thing to do. We do wish, however, that the dampers had been of a different type, as we have had quite a lot of trouble with the present ones. The expenses of making changes and maintenance have been far too high. The dampers were not supplied by the same manufacturer as the fans, but they were part of a package delivery. They have an overall aperture of 2.75 x 2.75 m, and are vaned. The air stream through the dampers is uneven, and quite high velocities develop. Both the bars and the vanes have proved to be too weak. Vibrations have caused considerable wear and tear. We have tried to reinforce the dampers several times, but without success. We subsequently tried out different types of dampers, and are now successively replacing them all. The new dampers were designed by our works manager and are produced by a Bergen firm. The new ones have turned out to be far more reliable than the original ones. The ventilation system was originally intended to function by blowing exhaust air out through shafts l, 3, 9 and 11, and drawing fresh air in through the other shafts. Unfortunately the air stream was not so easy to direct. The effect of wind in the outer tunnel sections in particular often caused unsatisfactory ventilation conditions in this part of the tunnel. We have therefore installed reversible fans in shafts 3 and 9, to supplement the original two fans. Now we normally use shafts nos. 3 and 9 as fresh air shafts. They can thus be said to serve as reserve exhaust shafts in case of fire. The fans are programmed to run day and night, seven days a week.

7 FAN MAINTENANCE

As already mentioned, we find the fans very satisfactory service routine is as follows:
Once a month a complete inspection is made of all fans and associated equipment. Twice a year we grease the bearings following the manufacturer's recommendations. Once every third month the fans are cleaned, and every year we do the usual patching and painting work. During the initial stage in particular, the driving belts are under very great strain, and we have consequently had to change them several times. This applies only to those fans which are most frequently in use, however. We anticipate that a bearing on one of the fans will have to be changed soon -the first time on any fan in the tunnel in sixteen years.

8. NOISE

The fans are slow-running: 330 rpm. This choice was designed to maintain a low noise level and avoid complaints from the neighbourhood. Nevertheless, we have had to line three of the shafts with noise-reducing materials. The noise-absorbing layer is made of perforated stones with a lining of rock wool against the rock.

9. THE DUST PROBLEM

The greatest problem for ventilation in the Løvstakk Tunnel is the dust. Although the fans are run so much that the CO concentration is only 20-30 ppm, visibility has frequently been unsatisfactory because of dust. In winter in particular, the dust problem is considerable, because studded tyres abrade the asphalt surface, producing mobile particles. Wet dust, asphalt, oil, dirty snow and water are brought in by cars from outside. In mild, wet periods, however, the dust is deposited more easily on the surface of the road and tunnel. In dry periods the dust is stirred up, causing visibility problems. The most effective way of fighting the dust problem in the tunnel has been sweeping and cleaning. The ditches along the sides are regularly swept manually. In addition the road surface is cleaned three times a year. But the main cleaning must be done every spring, when the season for studded tyres is over. Studs and chains grind large quantities of asphalt into dust during the winter. In spring the walls, crown and carriageway are thoroughly washed. At the same time we remove the heavier bituminous particles that cannot be blown out with the ventilation current. This major cleaning every spring is undertaken by various athletic clubs, which do the job for a fixed price. This spring it will cost about NOK 25,000. They work only at night, and normally take three weeks. The traffic is not stopped; the clubs use their own people to direct it. Once the washing has been completed, the centre line, edge of the carriageway and portals are painted.

10. VENTILATION SHAFTS

A system with eleven shafts was found to be economical because of the fairly thin overburden overlying the tunnel. The shafts vary from 32 m to 110 m in length. Two of them are inclined, the others vertical. The cross-section of shafts 3 and 9 is 22 m^2, and of the other nine 8.5 m^2. Shaft 3 is lined with concrete. Short sections of three of the other shafts are shotcrete-lined. The other seven are unlined. The chimneys protecting the top of the shafts are circular and made of concrete. They have steel inspection doors, and are covered with galvanized wire. During the sixteen years of operation we have had no problems due to rock stability. Very little rock fall has occurred, and ice lumps and small stones which fall down are caught in a pit at the bottom of each shaft. For this reason it has not been required that we inspect the shafts from top to bottom. For inspection purposes, a hoist can be installed inside the chimneys. More sophisticated methods have to be used in the inclined shafts.

11. ELECTRIC POWER

Electric power reaches the tunnel through an 11 kV cable located in a ditch under the sidewalk. The signal and monitoring cables are also located in this ditch. Transformers, from which the 230 V electric power is distributed to the lighting and ventilation systems, are installed in special cells connected with five of the fan stations. According to the concession, Bergen Electricity Board is technically and economically responsible for the power supply.

12. TUNNEL LIGHTING

The illumination in the tunnel is gradually reduced in four steps from 1000 lux at the portal to 40 lux in the middle of the tunnel The lighting is automatically monitored by a photocell located outside the south portal. The lamp fittings are connected to a cable running along the centre of the crown. The system is divided and anchored every 108 m. Once a year lamps and fittings are cleaned and inspected. For the last few years Bergen Electricity Board has had a contract with the firm which now carries out this work. In the middle of the tunnel there is a lamp every 9 m. Some years ago bus drivers complained of experiencing headaches and dizziness when driving through the tunnel. They felt that it was the distances between the lamps that were wrong and causing problems. We wrote to the bus company and showed that the lamps were located according to internationally specified standards. Since then we have had no more complaints.

13. MEASURING CARBON MONOXIDE

Four CO monitors which operate on the "infra red" principle have been installed in the tunnel to check the carbon monoxide concentration. When the concentration becomes too high, a warning signal is released, and the crew then start up more fans. The specified upper CO limit is 200 ppm. The registered maximum concentration during the day is only about 20-30 ppm, however. Only occasionally in rush hours has the concentration been as high as 50 ppm Another reason that we normally maintain such a low carbon monoxide concentration is that there are frequently maintenance crews in the tunnel. Up to the present there have been no instances when it has been necessary to bring in more fresh air because of a high CO concentration. The CO monitors have been in operation for sixteen years, and are now nearly worn out. The first twelve-fourteen years they worked satisfactorily, and regular inspection and calibration was sufficient. Now there are increasing maintenance problems, and we shall therefore soon be buying new monitors. This time they will be a newer type, with lower purchase price and running costs.

14. VISIBILITY

There are two monitors in the tunnel to monitor visibility. Some years ago we changed the original monitors for new ones, but now we are having problems with these as well, and are planning to have them repaired if possible. The visibility monitors are adjusted in such a way that when visibility is reduced to 40% of clean air, the fans will automatically start up one by one, if necessary all 8 of them. There is a built-in delay before the fans start automatically, to prevent too frequent starting and stopping.

15. OTHER EQUIPMENT

The tunnel is equipped with meters to register the number of vehicles per lane passing in each direction. The meters are connected up to induction loops. They monitor traffic density in relation to time, and also check toll collected. There are traffic lights in the tunnel to guide the traffic in case of fire or accidents. At each fan station, every 170 m, there are alarm buttons and a fire extinguisher. There are also alarm telephones connected to the attendant guard room, but not the public telephone network.

Running along the tunnel there is a l 1/2" pipe for pressurised water, with a tap every 50 m. 12 V batteries are used in the control circuits. These were changed in 1983, as they were fifteen years old. We are now planning to install an antenna wire for radio transmission in the tunnel, in cooperation with the police and the warning and information service. Then radio announcements can be made directly to motorists.

16 .TRAFFIC

It is apparent from the diagram that traffic has increased from 1.35 mill. vehicles in 1969 to 4.88 mill. in 1983. Heavy vehicles constitute about 10% of the total traffic.

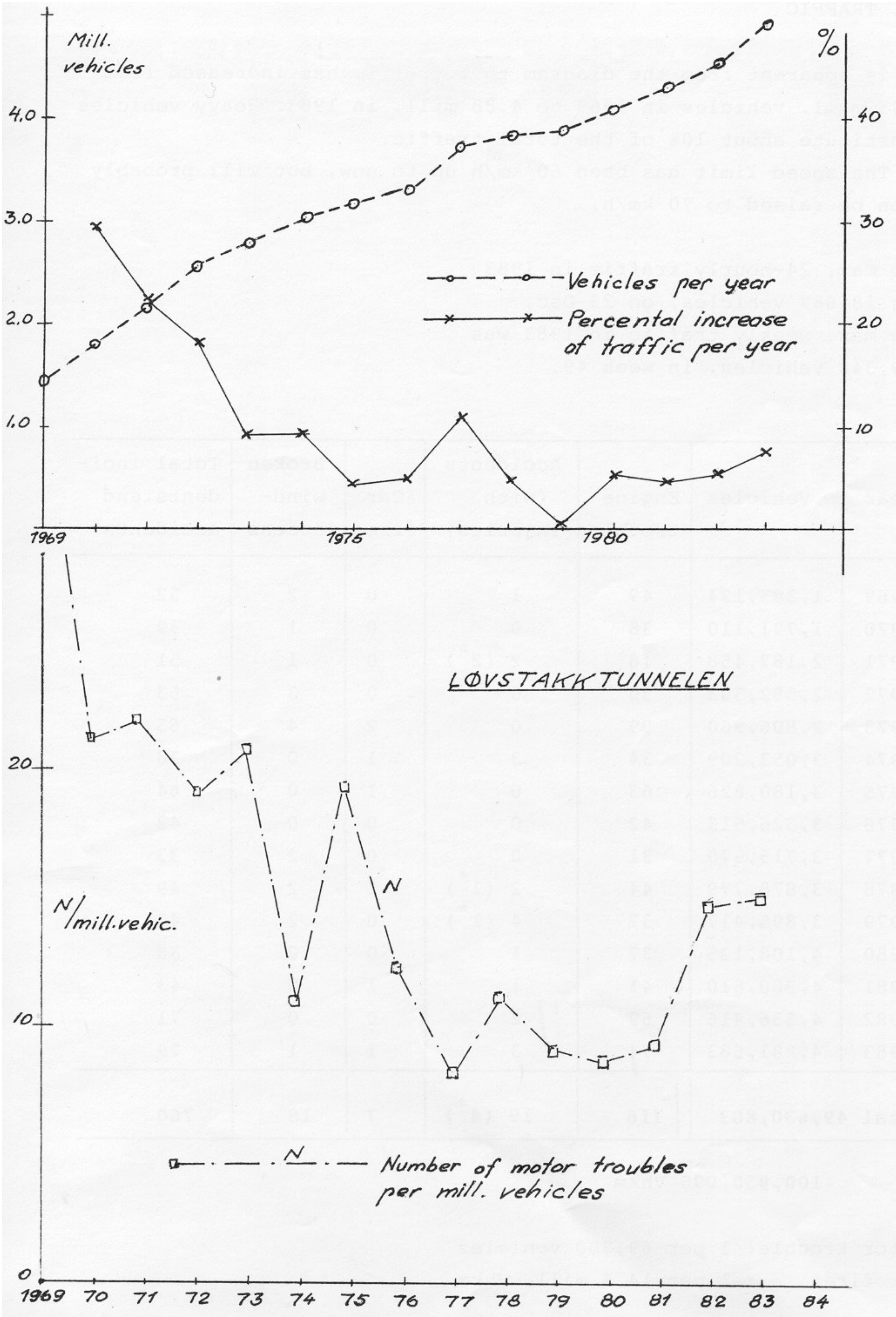
The speed limit has been 60 km/h up to now, but will probably soon be raised to 70 km/h.

The max. 24-hourly traffic in 1983 was 18,689 vehicles, on 23 Dec.

The max. weekly traffic in 1983 was 109,348 vehicles, in week 49.

Year	**Vehicles**	**Engine trouble**	**Accidents (with injuries)**	**Car fires**	**Broken wind-screens**	**Total incidents &accidents**
1969	1,385,124	49	1	0	2	52
1970	1,791,110	38	0	0	1	39
1971	2,187,458	48	2 (2*)	0	1	51
1972	2,582,503	50	0	0	3	53
1973	2,806,960	59	0	2	4	65
1974	3,053,209	34	3	1	0	38
1975	3,180,826	63	0	1	0	64
1976	3,326,513	42	0	0	0	42
1977	3,715,970	31	0	0	2	33
1978	3,878,779	44	2 (1*)	1	2	49
1979	3,895,417	37	4 (1*)	0	2	43
1980	4,108,125	37	1	0	0	38
1981	4,300,810	41	1	1	0	43
1982	4,536,416	69	2	0	0	71
1983	4,881,583	74	3	1	1	79
Total	**49,630,803**	**716**	**19 (4*)**	**7**	**18**	**760**

Motor trouble: 1 per 69.300 vehicles, Car fires: 1 per 14.4 mill Vhkm.


LØVSTAKKTUNNELEN
Mill. vehicles
%
4.0
3.0
2.0
1.0
40
30
20
10
1969
1975
1980
Vehicles per year
Percental increase of traffic per year
N/mill.vehic.
N
20
10
0
1969 70 71 72 73 74 75 76 77 78 79 80 81 82 83 84
N
Number of motor troubles per mill. vehicles

17. COMMENTS TO THE TABLE

In case of engine trouble in the tunnel, the company's service truck tows the car out. For this service motorists have to pay NOK 60. It is prohibited to fill up petrol or change tyres in the tunnel. Numbers of incidents of engine trouble per l mill. vehicles are shown in the table and in the diagram. I should think the development may be explained by the economic situation, the prices of maintenance and petrol, and the average standard of the cars. I make an exception for the first year, when motorists still weren't aware of the towing rate. The few accidents so far all occurred inside the tunnel, except for one in 1979 which took place at the approach, just outside the northern portal. Three of our crew were badly injured by a bus whose driver was blinded by the sun. Despite claims by drivers that windscreens have been broken by rock fall or ice, the cause has never been proved. We imagine that windscreens are broken through vibrations in the air in the tunnel affecting windscreens which already have a weakness. Only small fires have occurred in the tunnel, and the statistics show l fire per 14,400,000 vhkm. The last few years show a favourable development, in contrast to the numbers of engine troubles.

Vehicles with loads which represent a fire or explosion hazard, such as petrol and gas-carrying vehicles, are prohibited in the tunnel. As long as we have the toll-collecting crew, this is easy to check. And we believe it is important, because of the two-way traffic, the longitudinal ventilation, and because there are no emergency exits, only the tunnel portals.

18. WORKING CREW

There is a tunnel supervisor, who is responsible for the daily running of the tunnel, and also directs the toll-collecting crew. In addition to the supervisor, we estimate 3.5 man-years for operating and maintenance. We hire outside firms for special jobs such as asphalting and milling, washing and cleaning the tunnel in the spring, and special services for the technical equipment.

19 RUNNING AND MAINTENANCE COSTS 1983

(for 2032 m tunnel and 264 m of approach road)	NOK
Wages for working crew, including social costs	664,000
Elec. power for fans and lighting, except in tunnel	545,000
Snow-clearing, sanding, cleaning, painting	38,000
Repairs and maintenance	284,000
Equipment, running vehicles	67,000
Insurance	69,000
Telephone	16,000

As mentioned previously, Bergen Electricity Board pays for the electric power for tunnel lighting, and also for the maintenance and changing of lamps and fittings. In 1983 these costs were:

Electric power	140,000
Maintenance, replacing lamps	110,000

These final items are taken from a collective account, and are therefore estimates

20. CONSTRUCTION COSTS

The following list shows the total investments related to the various items.
Construction occurred during the period March 1965- June 1968
The percentage constituted by each item is also shown.

	NOK	%
Preliminary work	175,000	0,6
Tunnel, cuttings, approaches (2032 m & 265 m)	14,890,000	56,0
Ventilation system (construction, machinery, electro technical work)	5,450,000	20,5
Lighting	730,000	2,7
Safety items	550,000	2,0
Control house, toll collecting	850,000	3,2
Fees, management, insurance	1,685,000	6,2
Interest	2,370,000	8,8
Total	**26,700,000**	**100,0**

Estimated costs for 1984 for a similar tunnel and the same construction methods are NOK 80 million. March 1st 1984: US$ 1 = NOK 7.50; GB£ 1 = NOK 11.20

MAINTENANCE AND ECONOMIC CONSEQUENCES OF WATER LEAKAGE IN THE BRYN TUNNEL

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ABSTRACT
The Bryn tunnel consists of 2 separate road tunnels, each with 2 lanes, which pass through approx. 200 m of rock. They were opened to traffic in 1970. The support system consists of a single in situ concrete lining with no membrane. This paper describes the repairs and maintenance required to deal with the leakage which occurred during the first 8 years. The repair and maintenance costs have all been converted to price equivalents during the opening year, and compared to the construction costs. With a calculated horizon, or depreciation period, of 40 years, and interests of 0% and 7%, the figures indicate that the construction costs could have been 29% and 14% respectively higher, and still been economical, assuming the result would have been a watertight tunnel. The costs to road users caused by interruption of traffic flow during repair and maintenance periods have not been included, nor the risk of an increased number of traffic accidents. The repair and maintenance costs were assumed to remain constant throughout the remainder of the 40-year period. Finally, some general aspects of computing the present value of future costs and life-cycle costs are presented.

l. INTRODUCTION

The task of investigating the economic consequences of water leakage in the Bryn Tunnel in Oslo, part of the Norwegian Public Roads Administration's research program on low-cost road tunnels, was assigned to my firm. The investigation was made possible by the positive response of the owner, the Oslo Road Authority. The accounting system of the Oslo Road Authority was also an important contribution. Throughout the period, all expenses for repairs and maintenance of the Bryn Tunnel were entered in a special account.

2. DESIGN AND CONSTRUCTION OF THE BRYN TUNNEL

Fig. l. Bryn Tunnel, south entrance.

The Bryn tunnel is part of Oslo's main ring road (Store Ringvei), located about 3 km from the city centre. The tunnel consists of twin road tunnels, each with two lanes (Fig. l).
The tunnel is cut through solid rock. The rock-cover is relatively thin, approx. 5-20 m, and the tunnel is about 200 m. long

Drainage is accomplished by gravity flow. The support system of the tunnel consists of a single in situ concrete lining with no supplementary waterproofing.

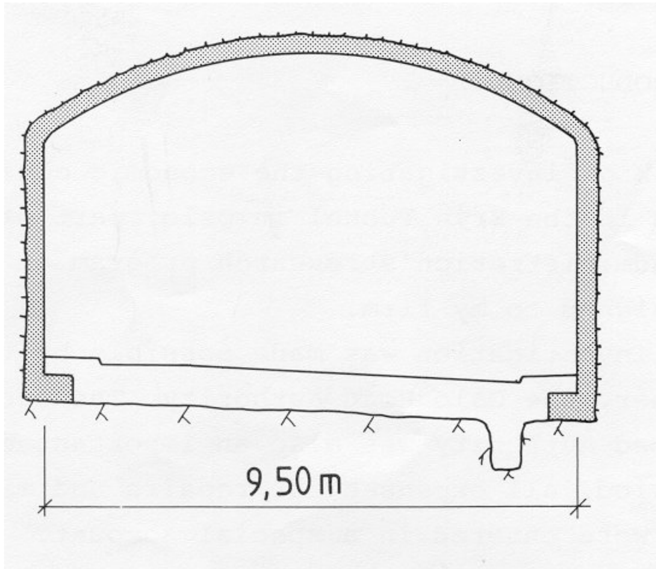

Fig. 2. Bryn Tunnel, Cross section.

The lining was put up in 6 m sections with the aid of movable scaffolding. The lining joints were furnished with rubber water stops. The tunnel was opened to traffic in 1970. The total construction cost amounted to NOK 9.1 mill.

3 .LEAKAGE, MAINTENANCE AND REPAIRS

The watertightness of the lining soon turned out to be unsatisfactory. Leakage occurred at cracks and construction joints, especially after periods of heavy rainfall. In the winter this resulted in icicles on the crown and humps of ice on the walls and roadway.

As a result tunnel inspection and maintenance routines were necessary to remove the ice. Ice removal activities were most intensive after a thaw or a rainy period followed by frost.

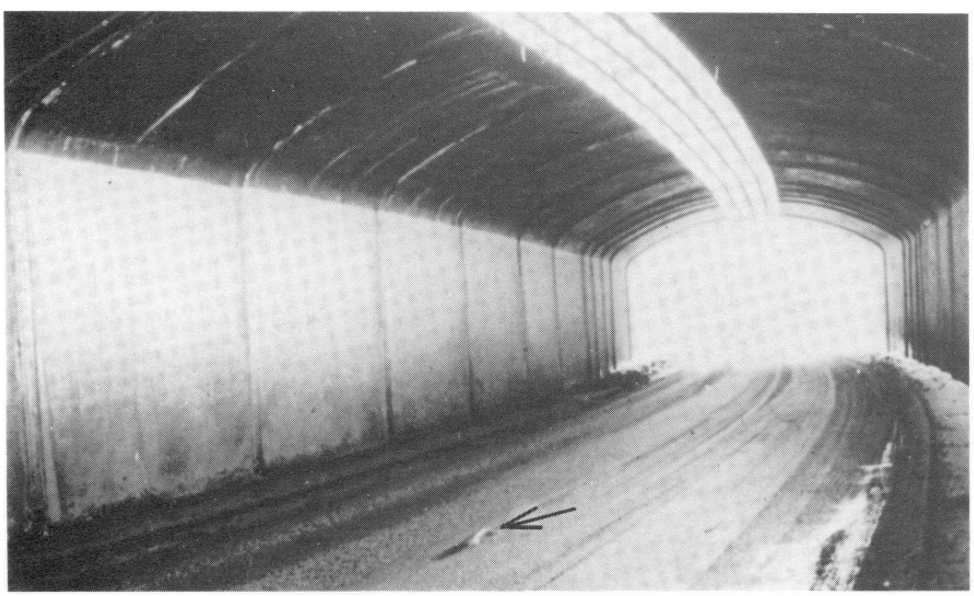

Fig. 3. Bryn tunnel. Drains have been installed. Note hump of ice (arrow) due to leakage from the crown.

The amount of maintenance led to repairs in 1974 for the one 2-lane tunnel, and in 1976 for the other. In order to divert water into the drainage system, pipes were installed along leaking cracks and joints. To avoid

freezing, they were equipped with a thermostatically controlled electric heating wire.
The results of the repairs were fairly good, but leakage still makes ice removal necessary during the winter.

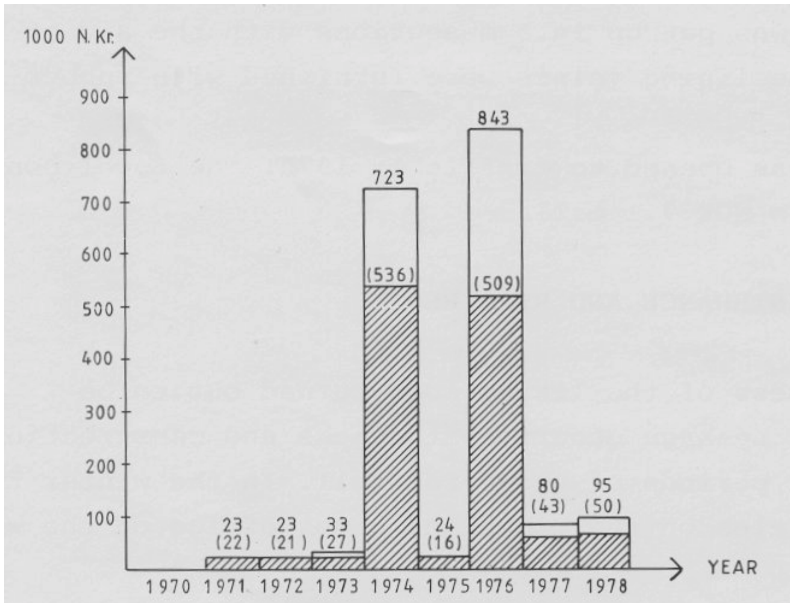

Fig. 4. Annual repair and maintenance costs due to leakage. The cross-hatched columns show the costs converted to 1970 NOK.

Fig. 4 shows the annual cost of repairs and maintenance during the period 1971-78. The costs for 1977 and 1978 also include the costs of heating the piping system electrically, which amounted to NOK 48,000 and NOK 57,000 respectively.

4. PRESENT EXTENT OF OPERATION, MAINTENANCE AND REPAIR COSTS

Compared to the original investment, the costs of operation and maintenance of roads and tunnels are considerable. Both should therefore be considered together in the form of life-cycle costs (LCC). To achieve this, the costs should be converted to equivalent values at the same point of time, usually the time of investment. Conversion of future costs to the present value is the key problem here.
general formula is:

$$LCC= I + \Sigma x\ Wx = I + \Sigma x \sum_{t=0}^{t=n} cxt \cdot \beta xt$$

where

I	=	investment cost, including cost of raising the capital
W_x	=	he present value of the operation and maintenance cost for the component x from
	=	time = 0 (time of investment) to the horizon (end of life-cycle), t = n.
C_{xt}	=	cost of operation and maintenance of the component x in the year t.
β_{xt}	=	conversion factor for converting Cxt to present value.

The calculation of life-cycle cost is of great importance to the concept of low-cost road tunnels. Life-cycle costs of a tunnel can be presented in a matrix, as shown in Table l, where the components may be:

X_l: Costs of operation, maintenance and repair of support and waterproofing systems.
X_2: Ditto for roadway pavement.

X_3: Ditto for drainage systems.
X_4: Ditto for cleaning and painting.
X_5: Ditto for lighting and ventilation.
X_6: Ditto for signalling and traffic information systems.
X_7: Other costs relevant to an investment decision, e.g. construction phases necessary at a later date to adapt the traffic system to predicted traffic volume.
X_8: Users' transportation costs.
X_9: Traffic accident costs.

Component	YEAR NO												
	1	2	3	4	5	6				n-3	n-2	n-1	n
X_1													
X_2													
X_3													
X_4													
X_5													
X_6													
X_7													
X_8													
X_9													

Table l. Matrix of life-cycle costs

Each cost in the matrix must be multiplied by the appropriate conversion factor. When processing alternatives, only those costs which are specific to the alternative in question need be considered. The costs of operation, maintenance and repair of the support and waterproofing system necessary due to leakage in the Bryn tunnel are indicated in Table 2.

	YEAR									
	1970	71	72	73	74	75	76	77	78	1979-2010
Investment	9100									
Maint. & repair		22	21	27	536	16	509	43	50	43 Annually

Table 2. Matrix for the life-cycle cost of operation, maintenance and repair of waterproofing systems in the Bryn Tunnel. All costs in NOK 1000, converted to 1970 values.

The present value for a horizon (end of life-cycle of 40 years with continuous payment of interest is:
A) at an interest of 7%:
Wl
$= 22 \cdot 1.07^{1} + 21.1 \cdot 07^{2} + 27 .1.07^{-3} + 536 \cdot 1.07^{4}$
$+ 16 \cdot 1.07^{5} + 509 \cdot 1.07-6 + 43 \cdot 1.07^{7} + 50 \cdot 1.07^{8}$

$+ \sum_{t=9}^{t=40} 43 \cdot 1.07^{-t} = 1230$ (NOK 1000)

W1 = 0.14 · I

B) Wl is similarly calculated for a rate of interest of 3%, Wl = 0.20 · I

C) Ditto for 0% interest: Wl = 0.29 · I

The calculation shows that the present operation, maintenance and repair costs are considerable, and that waterproofing is one of the major problems of low cost road tunnels.

5. LIFE-CYCLE COST

The life-cycle cost (LCC) consists of the original investment plus operation and maintenance costs. The "life-cycle cost method" is a result of efforts made in many countries to develop a design criterion which takes into account initial investment and future costs in a balanced form. The LCC method is a useful tool in the search for the most favourable design from the point of view of life-cycle. It is this author's opinion that the life-cycle cost methods and experience gained in the building industry may usefully be applied to the design of low cost road tunnels. In low cost road tunnels operation and maintenance costs should be converted to present values by means of a decision model which takes into account all relevant factors e.g.:

- Availability of capital
- Standard of the components of the whole road including the tunnel
- Reliability of the technical solutions
- Horizon (end of life-cycle)
- Conversion factors

The decision model should be designed to meet the requirements of the client.
The influence of the horizon and interest rates on the conversion factors are indicated in Table 3.

YEAR No.	INTEREST RATE 0 %	3 %	7 %
0	1.0	1.0	1.0
5	1.0	0.86	0.71
10	1.0	0.74	0.51
15	1.0	0.64	0.36
20	1.0	0.55	0.26
40	1.0	0.31	0.07
100	1.0	0.05	0.001

Table 3. Conversion factors for different numbers of years and interest rates.
Conversion factors for continuous payment of future annual costs are given in Table 4.

YEAR No.	INTEREST RATE 0 %	3 %	7 %
10	10	8.53	7.02
25	25	17.41	11.65
50	50	25.73	13.80
100	100	31.60	14.27
∞	∞	33.33	14.29

Table 4. Transformation factor γ for continuous payment of annual equivalent cost for different numbers of years and rates of interest.

Tables 3 and 4 show that the influence of the horizon on life-cycle costs is reduced w hen the interest rate is high. The figures also show that there is less advantage in increasing investment to save maintenance costs when the interest rate is high.

The task of setting the parameters should be left to the client.

I. Oefverholm (l) proposes an interim decision model with two alternative rates for constructions (see Fig. 5 and Tables 5 and 6).

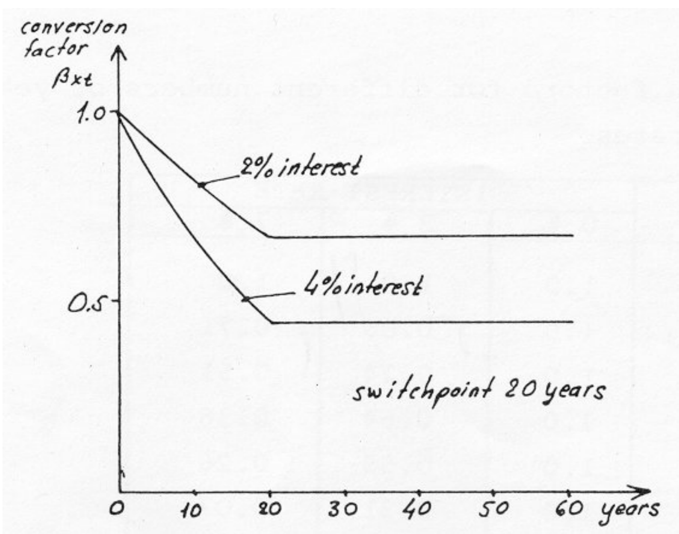

Fig. 5. Conversion factor as a function of time according to (1)

YEAR	INTEREST RATE 2 %	4 %
5	0.905	0.891
10	0.819	0.670
15	0.741	0.549
20	0.670	0.449
25 etc.	0.670	0.449

Table 5. Conversion factor xt when switch point is 20 years.

YEAR	INTEREST RATE	
	2 %	4 %
5	4.8	4.5
10	9.1	8.2
15	13.0	11.3
20	16.5	13.8
Annual increase of assumed horizon	0.67	0.45

Table 6. Transformation factor(present value factor for annual cost) when switch point is 20 years. (Switch point -the point in time after which the investment is not profitable)

Calculating the life-cycle cost is a new approach. It requires data, particularly on operation maintenance activities and their costs. The process of collecting and processing data for this purpose should be given priority.

References:

1 Oefverholm, I., Life-cycle Cost of Buildings. IABSE Journal J-2l/83.