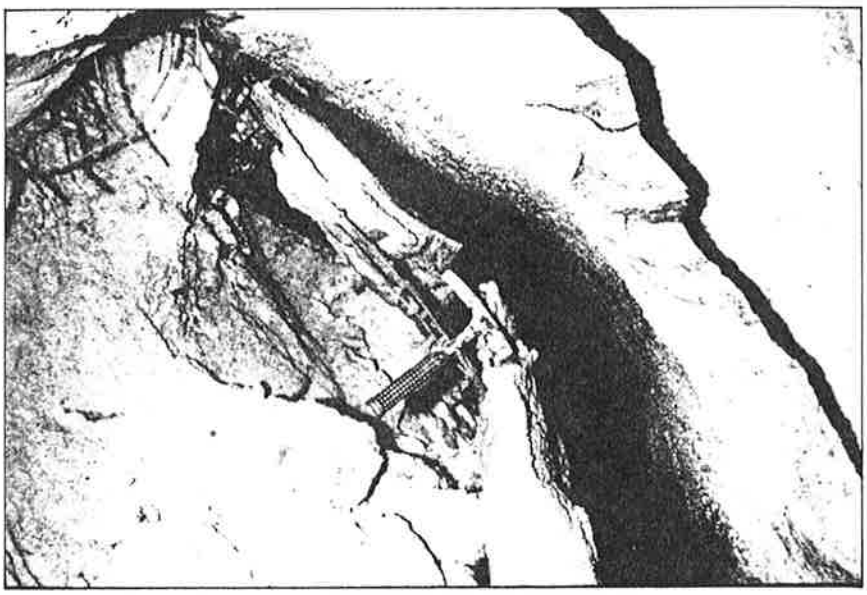
Tunnel collapses in swelling clay zones

Professor Rolf Selmer-Olsen, Geoteam and Arild Palmstrom, Berdal Strømme

Swelling rocks are often a special challenge in tunnelling, since they may cause great excavation problems. In several cases swelling zones or rocks containing swelling minerals have caused tunnel collapse which has resulted in considerable additional costs and delays for the project. Further financial loss arises if the problems caused by swelling rocks result in closure of a facility, e.g. a road tunnel or the reduction or cessation of production, e.g. a hydroelectric plant. The fact that swelling rocks can be difficult to detect during tunnel excavation makes this geological feature of special interest to tunnellers. The phenomenon can be caused by the following minerals:

1. Smectite (montmorillonite, vermiculite, etc);
2. Anhydrite;
3. Some pyrrhotites in calcareous shales.



Failure due to swelling clay in a shotcreted water supply tunnel.

This paper mainly concentrates on experience from the first group listed, where swelling minerals are associated with weakness zones or gouges, that are faults, seams, feather joints, crushed rocks or other discontinuities. The experience of the authors is mainly from Norwegian tunnel projects where such geological features have been found in most types of rocks of Precambrian and Palaeozoic age. It is, however, believed that swelling clay minerals may be present in many old and young rocks throughout the world as a constituent of gouges and as rock-forming minerals.

About 75% of the cost of extra reinforcement for tunnels after they have been put into operation, has in Norway been associated with swelling clays. The problems that have arisen have often been caused by the lack of experience on the part of the people involved concerning the swelling mechanism, combined with an underestimation of the high pressure, existing from swelling clays.

The second part of this article deals with laboratory tests, excavation methods and assessments of rock support in swelling zones.

What are swelling clays?

The swelling of clays is mainly caused by smectites: a group of minerals where montmorillonites, vermiculites and mixed-layer swelling minerals are most common. These are secondary minerals formed from the alteration either of in-situ rock forming minerals in shales, or of solution deposits. They are most often very small sheet minerals and differ from other sheet minerals (such as mica and chlorite) in their ability to take up and release water in accordance with the external pressure to which they are subjected.

Swelling clays associated with weakness zones occur in two different ways:

1. As fillings strictly associated with joints, veins, fractures or faults;
2. As rock-forming minerals in altered rocks mostly associated with the first group. This type occurs less frequently.

Swelling materials may be found only in some of the faults or joint systems in a geological province; younger or older systems crossing the same area may not contain swelling clays. Usually rock powder and fragments are present together with the swelling minerals; also other secondary minerals like calcite, quartz, chlorite, talc, zeolites, kaolinite and hydromica/illite may be associated with the swelling clay minerals in the fillings. The type of cation present affects the degree of swelling to a great extent. Na^+, for example, will cause a high degree of swelling, while CA^+ will cause a lower one.

The most important factors affecting the degree of swelling and softening in a zone are:

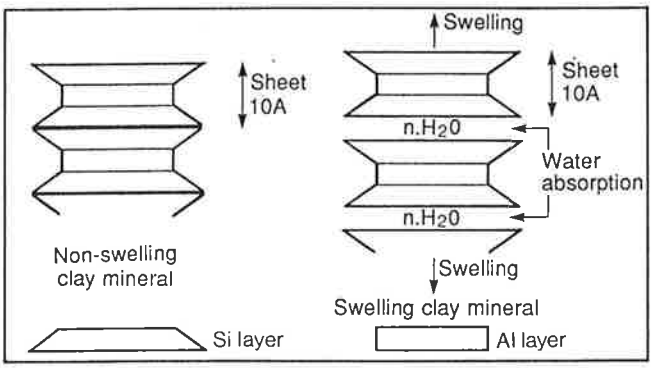


Fig 1. Principal difference in structure between swelling and non-swelling clay mineral structure. The length is given in Angstrom, $1A = 10^{-7}mm$.

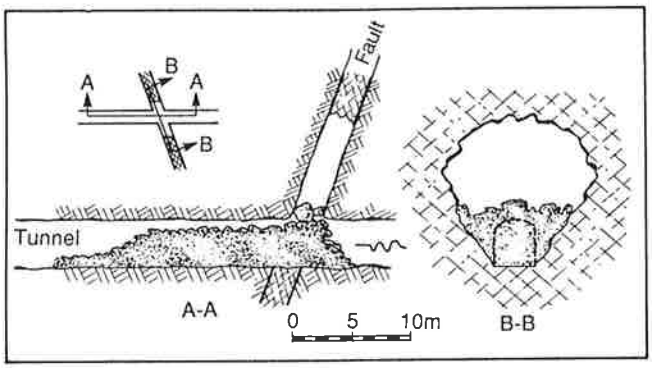


Fig 2. Sketch of the collapse blocking the $12m^2$ headrace tunnel at Hemsil Power Plant (1960). It was caused by a 2.5m-thick, brecciated zone with thin veins containing swelling material.

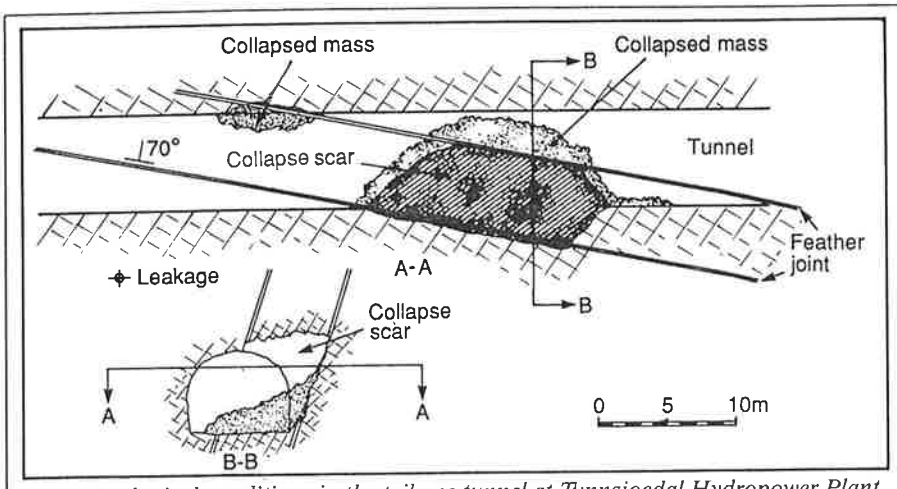


Fig 3. Geological conditions in the tailrace tunnel at Tunnsjoedal Hydropower Plant (1962). Fast installation of the concrete lining prevented a larger slide which could have resulted in much higher costs for rock support.

The amount and type of swelling minerals;
The amount and type of mobile cations;
The degree of consolidation of the material in the zone;
The access to water;
The degree of unloading after excavation.

In addition to the factors listed above, the amounts, types and shear properties of the other fine-grained, loose materials in the zone will influence the swelling properties and the behaviour of the zone. Calcite and other minerals that can be dissolved and so make space for the softened filling to be washed out are important in this connection.

No swelling will take place under dry tunnelling conditions. Here any water in the swelling zone and groundwater from the surrounding rock masses entering the zone can evaporate. In such situations it is almost impossible to distinguish by inspection between occurrence of non-swelling and swelling materials. It may be easier to detect the presence of swelling materials by washing the tunnel walls, especially if the washing allows the swelling materials to absorb water.

Under wet tunnelling conditions it is easier to detect swelling materials. Here the swelling squeezes the gouge material some millimetres out. This feature is often best seen near the lower part of the tunnel wall where the clay can more easily absorb water from the invert.

In all the examples given below the occurrence of clay containing veins of material had been detected during tunnel excavation. But insufficient rock support had been installed due to underestimation of the large forces that the swelling of clays can produce. It is, however, often hard to imagine that the apparently strong rock observed in the tunnel wall can disintegrate and that formidable swelling pressures on the rock support can build up. The examples also show that other materials in and on the side walls of the swelling zone may play a great role in triggering a collapse.

The first example, which is from the Hemsil I Hydropower Plant, relates to a fault zone containing swelling clays with unaltered surrounding rocks. The collapse occurred in the 14km-long 12m^2 headrace tunnel located in gneisses and granites of Precambrian age. A dominating system of steep dipping faults crossed the tunnel approximately at right angles. The fault zones were up to 8m thick mostly without complete clay fillings, but with brecciated rocks containing many veins of swelling clay. Horseshoe concrete lining was mostly used as rock support, though some smaller zones were shotcreted.

Due to increasing head loss in the power plant, the tunnel was emptied for inspection after eight years of operation. It was then discovered that a 200m^3 collapse had taken place in an area where the tunnel had been supported by shotcrete only. The collapse was caused by a 2.5m-thick, coarse-brecciated zone having many thin veins with swelling material (Fig 2). All shotcrete applied on the side walls of the zone had been scaled off.

While the collapse in the first example started after the tunnel had been put into operation, the second example shows that a collapse can start shortly after excavation if the swelling material is in contact with water. The collapse occurred in a tailrace tunnel, this time in the Tunnsjoedal Hydropower Plant. Here the swelling clays were not restricted to the filling material only as in the first example, but mainly associated with the altered, adjacent rocks where the feldspar had turned into montmorillonite.

The 35m^2 tailrace tunnel at Tunnsjoedal Hydropower Plant passes through Palaeozoic granitic gneiss. During excavation a coarse system of feather joints or small, pinnate faults was encountered in the tunnel. Apparently they did not represent any stability problems as the first of them was dry. In one area, however, water leakages came from a brecciated zone in the adjacent rocks between two of these feather joints, 6m apart (Fig 3). Two weeks after excavation, collapses started to occur in a 30m length. A rather complicated and time-consuming period of rock support started. The support consisted mainly of cast-in-place concrete lining and had to be installed quickly to prevent further progress of the collapse. During this work it was surprising to find that the apparently strong rocks which looked like granite disintegrated into a clayish mud when coming into contact with water.

The relatively long stand-up time of the rock masses, in spite of ample access of water, is explained by a slow water absorption rate caused by low internal permeability of the altered granite. Where smectites only are present as joint or gouge fillings, the stand-up time in similar conditions would have been as little as one or two days.

The third example is from a railway tunnel in southern Norway. Here a collapse was caused by swelling and softening of gouge material. The tunnel is located in granitic rocks of Precambrian age intersected by steep dipping, heavily leaking calcite zones. Unfortunately, the two types of zones crossed each other at a point in the tunnel where water leakage came from holes eroded in the zone.. A cast-in-place, horseshoe lining was installed along 25m in the tunnel.

One day, some eight years after the tunnel was completed, cracks were observed in the lining, and at the same time, the drainage water dramatically decreased. The following day the whole lining collapsed, and a cavern approxiately 30m high developed. The repair work was very dangerous, time-consuming and expensive.

It is believed that this collapse was a result of the drainage effect of the tunnel. The calcite zone and the tunnel drained a large bog located where the zone outcropped. The leaking water dissolved the calcite and washed out the material in the zone, leaving space for swelling and softening of the adjacent, altered rock. This caused the water-saturated material to collapse suddenly, the lining cracked and the drainage clogged. The high water pressure that rapidly built up then caused the lining to fail.

The last example is from the Rafnes 16m^2 water supply tunnel. The tunnel, which is partly beneath the sea, is located in Precambrian gneisses and amphibolites intersected by several zones up to 5m thick containing clay. Also, the rocks on both sides of the zones were often altered, partly to swelling clay minerals. Because the zones were dry, only a few of them caused minor stability problems during tunnel

Breakdown of a 0.25m-thick mesh reinforced shotcrete lining in the Rafnes Tunnel.

excavation, but they were all given initial support by shotcrete. Later, additional shotcrete, often reinforced, was applied as the final rock support.

A few months after the tunnel had been filled with water, but before going into service, it was blocked by several collapses. During inspection of the emptied tunnel it was found that up to 30cm-thick reinforced shotcrete had been destroyed in about 30 locations and that larger collapses had blocked the tunnel in four places. The reason for this was the occurrence of swelling clay partly combined with altered rocks, and the fact that shotcrete had been applied shortly after excavation. The rapid spraying of shotcrete after excavation did not allow the clay to swell, resulting in the build-up of high swelling pressures. In this connection it is interesting to observe that in a parallel tunnel 15 years older, also used for water supply, where cast-in-place concrete linings had been used, no collapses have been reported. Because some time elapses before concrete lining is installed, the clay is allowed some radial deformation and swell before the concrete is hardened. The gap between the concrete and the roof will also give some extra space for the initial swelling of the clay.

As the tunnel collapses occurred before the industrial plant had been placed in service, the mostly fine-grained collapsed masses were not transported away, which limited the size of the collapses.

In one of the larger collapses that led to water leakage, grouted rock bolts were installed through the zone before mucking out of the slide masses in an attempt to stabilise the tunnel roof. However, this rock support was not strong enough to prevent the collapse developing further. The result was a much larger collapse where the water-saturated collapse debris was transported a long distance in the inclined tunnel. The collapse could therefore develop up to the surface about 50m above the roof without being supported against collapse material that was built up in the tunnel, as had been the case in the first slide. □

Part 2 of this article will be published in a forthcoming issue of *Tunnels & Tunnelling.*

It is with great regret that we announce the death of the author, Professor Selmer-Olsen.

TAILRACE

No light at the end of the tunnel . . .

'We have had an unfortunate experience, but it will not stop our excursions to Norway.' This was the comment made by the two pensioners who spent the weekend in an unfinished Norwegian road tunnel.

Last Saturday afternoon, the couple took a wrong turning at a crossroads, and ended up, despite all the warning signs, in a tunnel being built between Underdal and Gudvangen. After driving almost six kilometers in the dark tunnel, the couple's car stopped in a large pool of water.

'We thought it was just a puddle in the road, but suddenly the water was all around us and the car started to float. When we got out, the water was chest deep,' said Karl Kallin, aged 76, for the Norwegian News Bureau. In the pitch black, Karl and his companion, Inger Loefgren, 73, made their way out of the pool. They tried to find a torch in the car, without success.

'Instead, we had to feel our way along the walls to try to get out. After four or five hours, we came to a place that seemed to be the end of the tunnel. As we could not get any further, we had to turn round, and we continued all Saturday night, all Sunday and Sunday night' said Karl Kallin.

It was cold in the tunnel and the couple's clothes had been wet from the start. 'My companion was wearing wooden clogs, and lost them several times on the stoney road, so that we had to crawl round in the dark looking for them.' said Karl.

The two pensioners, who come from Farsta outside Stockholm, became tired, and while resting, the man fell asleep. 'When I woke up, Inger had gone, and I was convinced that I would never see her again,' he said. A short while later, he saw vehicle lights coming towards him, but the driver did not notice the incapacitated Swede. Another vehicle soon came along, and this time the driver stopped, and they together found the lady asleep a little further down the tunnel.

'Now we are resting in the site offices. We have a few cuts and bruises, but otherwise we are alright,' concluded Karl Kallin. □

Our thanks are due to Ged Pakes, who found this in the Swedish daily newspaper *Sydsvenska Dagbladet,* and who furnished the English translation.

. . . and no tunnel if you follow the light!

According to a report in the London *Daily Telegraph,* the religious group known as the Moonies, founded by the Rev Sun Myung Moon, have begun burrowing their tunnel from Fukuoka in Japan to Masan in South Korea (*T&T,* July '87, p5). Geological surveys have been carried out, land acquired and shafts bored in the first moves towards excavating a 140 mile-long tunnel by 2009.

The fact that the Tokyo government, for strategic and political reasons, is unlikely to approve the project in the near future fails to deter the promoters. The Japan-Korea Tunnel Research Institute is a part of Dr Moon's wealthy Unification Church, which is putting up most of the money.

First proposed in 1981, the road and rail tunnels would run under the seabed from the Japanese island of Kyushu to the tip of the South Korean peninsula via some islands where ventilation shafts could be placed. The cost of the 15-year construction work is estimated at £64bn (making the Channel Tunnel look fairly cheap at £7bn).

The Moonies' scheme further envisages Bullet Trains going under the Korea Strait, up the peninsula and into China, with an international highway running from Tokyo to London. So far, only a 400m experimental drift (decline) has been excavated on Koje Island on the Korean side, and a 480m boring on the Japanese shore, both on private land. There is no word yet on whether the tunnels are to be TBM driven, blasted or shield-excavated, or from how many headings work will be undertaken. However, separate road and rail tunnels are envisaged.

Ever since stormy seas in the strait saved Japan from invasion by Mongol fleets in the 13th century, most Japanese have been quite happy to know that there are 120 miles separating them from mainland Asia! □

No tunnel at all

Channel Tunnel Investments, a company formed more than a century ago to drive tunnels between Britain and France, but completely unconnected with the present work being carried out for Eurotunnel, is seeking an injection of trading or commercial activities. It has been bypassed by the present cross-Channel link as its previous attempts all ended in failure for one reason or another, but its shares are still quoted.

HASWELL – There will be a Service of Thanksgiving for the life and work of Charles Kenneth Haswell at St Margaret's Church, Westminster, London, on Thursday 23 November at 12 noon. (Obituary, *T&T,* Oct '89, p43).

PART TWO

Tunnel collapses in swelling clay zones

Professor Rolf Selmer-Olsen, Geoteam and Arild Palmstrom, Berdal Strømme

Part 1 of this article (Nov '89) gave some examples of the collapse of tunnels in swelling clay gouges. Many of the collapses were caused by insufficient rock support as a result of underestimating the high loads exerted by the swelling process. When swelling zones are present in a tunnel it is essential that a method for detecting their occurrence, together with calculations of appropriate rock support, be established. Part 2 shows how careful site description and laboratory investigations of the material with swelling properties can be used to calculate the maximum swelling pressure to be accommodated in the design of the required rock support.

The aim of laboratory investigations of swelling clays is to measure the most important clay properties required for calculations of the swelling loads whereby methods of rock support can be determined. The presence and types of swelling clay minerals can be found from X-ray and Differential Thermal Analysis (DTA) analyses. However, neither of these methods gives an indication of the swelling pressure that can be exerted. They can therefore only be used in qualitative description of the gouge material.

Swelling properties are usually measured by oedometer tests. They used to be carried out as qualitative analysis based on results from earlier experience. Today, a method using a laboratory procedure that corresponds to the conditions in a tunnel is under development. It therefore reduces the need for estimating the various factors acting during the swelling process. The method requires, however, that the site is properly described with respect to structure, orientation and thickness of the weakness zone(s), together with total thickness of the clay-containing parts. As described later the method is of particular importance when shotcrete is used as support.

Clay zones are mostly composed of secondary minerals and of a mixture of rock fragments ranging in size down to clay particles. The zones encountered in Norway are believed to have been formed and developed under the special geological conditions of stresses and heat existing in the Earth's crust. The particles in such zones are therefore shaped and well adapted to each other under high stress conditions. Because of this the texture of these 'loose' materials is more like that of rock than soil.

Since undisturbed, in-situ samples of a clay zone can seldom be obtained, it is necessary to use a procedure which includes the main properties of the swelling material. The swelling tests are therefore carried out on elutriated and dried samples with particle size less than 20 microns. This grain fraction will contain almost all the swelling minerals because the larger grains will disintegrate during rough treatment of the sample material. Twenty grams of dry sample powder is loaded at 4MPa in the oedometer before it is given access to water through the lower filter (Fig 1). After all the material in the sample has finished swelling at this pressure, the thickness of the sample is recorded. A step by step release of the sample is then carried out to best imitate the in-situ conditions in a tunnel.

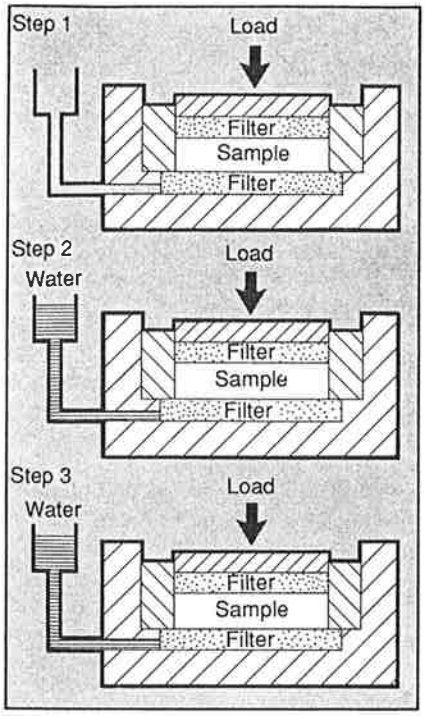


Fig 1. Principle of oedometer test for measuring the maximum swelling pressure of gouge materials.

Swelling curve

In spite of careful preparation of the sample, an artificially larger, elastic deformation than in undisturbed, in-situ samples will occur during the release of pressure. This pure elastic contribution can be compensated for by subtracting a 'standard' elastic deformation curve from the curve measured for the swelling clay. The 'standard' curve is found from equivalent tests on other gouge materials free from swelling minerals. They show variations down to a load step of 1MPa. Fig 2 shows an example of a swelling curve found in an oedometer test and the subtraction of a 'standard' curve to give the resulting swelling curve.

Both physiochemical (swelling) and elastic forces are accumulated in the clay sample before unloading. The elastic forces decrease rapidly during unloading, while the mobilisation of the swelling forces requires, in addition to water, an increasing volume during unloading. The swelling forces will therefore give the dominating contribution to the pressure curve after initial volume increase. This causes the resulting swelling curve to show a peak after a small volume increase, which is the maximum swelling pressure for the sample for the consolidation chosen.

Testing shows that maximum swelling forces increase linearly with the (pre) consolidation pressure, at least up to 8MPa. At higher consolidation, the swelling forces in samples with very high content of smectite increase somewhat less. As shown in Fig 2, the pressure is strongly reduced if a small deformation or volume increase is allowed in addition to time for release of elastic forces.

Most of the radial, elastic deformation in a swelling zone generally takes place before even quickly installed rock support can be applied at the tunnel face. The radial swelling pressure may, however, act on the support while the swelling decreases with increasing pressure depending on the shear strength of the material and the 'silo effects' in the zone.

One practical consequence of this is that the amount of rock support in a tunnel can be reduced if some space for

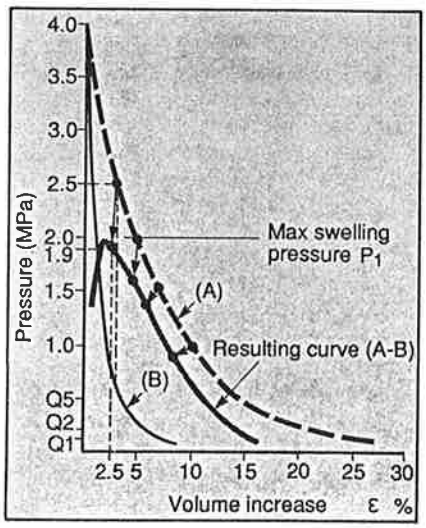


Fig 2. Swelling curves measured for samples of (a) swelling clay and (b) non-swelling materials. The representative swelling curve used in the calculations of support is found as the difference between the two curves.

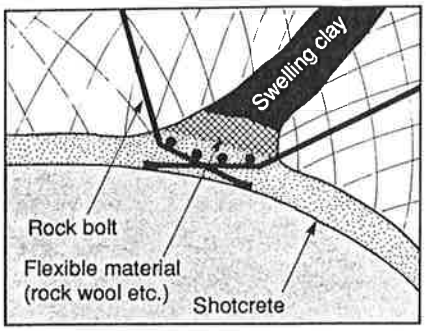


Fig 3. Method of sealing of smaller swelling zones using shotcrete.

volume increase is introduced between the support and the zone to be supported. This is in fact what takes place when cast in place concrete linings are applied as rock support. There will always be a gap in the roof between the concrete and the rock where an expansion of the swelling clay can take place. Shotcrete applied at the face gives no such space or time for initial swelling. A way of introducing the necessary space when shotcrete is applied is to cover the zone with some type of compressible material before shotcreting (Fig 3).

Another practical implementation of the swelling tests is the ability to predict the maximum swelling pressure on the support, as further described in the next section.

Rock support of swelling zones

Evaluation of support requirements needs a knowledge of both the loads acting on the support and the load carrying capacity of the support itself. The loads from swelling clay zones are often difficult to assess exactly because of the many factors involved. Evaluations must therefore be based on simplifications dealing with only the most important factors. Some preliminary results from a system under development show how calculations of the maximum swelling pressures can be made are given below.

The load from a swelling zone on the rock support is the sum of the swelling and the gravitational pressures and possible mechanical squeezing forces (Fig 4). Under equal external conditions the clays with the highest swelling properties will cause the highest swelling pressure. The consolidation of a zone is, however, an important external factor, which on the one hand can mobilise high swelling pressures, and on the other increase the gravitational pressure.

The principles of calculating the maximum swelling forces from the zone are as shown below.

The relationship between these parameters is given by the equation

$$P_s = P_1 \times \frac{c}{100} \times \frac{K}{k_1} \times B_1$$

where

P_1 = the maximum swelling pressure (corrected for elastic expansion) measured in the laboratory with a consolidation pressure of 4MPa (Fig 2);

c = content in per cent of material less than 20 microns (μm) in the sample taken from the zone;

K = actual consolidation (in MPa) of the material in the zone;

k_1 = consolidation applied in the laboratory (4MPa);

B_1 = the clay thickness, i.e. the sum of all clay fillings in a section across the zone (Fig 5). If the total width of the zone containing clay is much larger than the span or the height in the tunnel, the latter is used in the calculations.

The maximum consolidation (K) of a zone in relatively flat areas can be estimated roughly from the location of the zone in the rock masses, i.e. from the rock stresses caused by the overburden. The knowledge of possible high tectonic stresses occurring in the rock mass and the direction of these are not important in this rough assessment. This is because such zones are easily compressed and deformed by shear forces.

If pore water pressures and rock stresses have been measured, the in-situ effective stresses across the zone have to be more precisely considered. The maximum stresses acting across clay zones in steep valley sides may in some cases be particularly high. This is the case where the zone strikes parallel to and slopes inwards from the valley.

For flat topographical conditions, the correlation can roughly be expressed as:

$K = h \times (r_R - r_w)$

r_R and r_w = density of rock mass and water respectively

h = the overburden (in m)

Fig 5 gives an example of how the thickness of the clay fillings, B_1, are defined. If large differences in swelling properties exist between two present clays, B_1 should be measured for each of the two types, and the swelling load be calculated as the sum of the two.

At many larger swelling zones, gravitational loads will act on the rock support. This is particularly true if some swelling of the gouge material is allowed; it is caused by a reduction of the internal friction when that material increases in volume and softens (Fig 4).

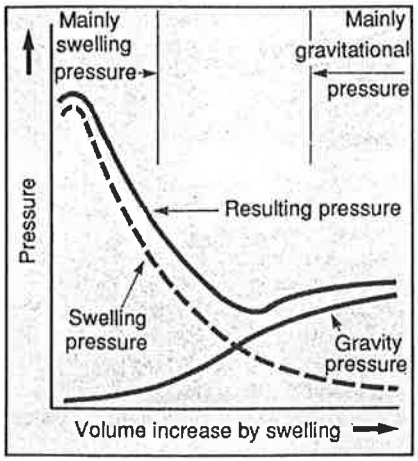


Fig 4. Stages of the pressures resulting from a large swelling zone on the rock support.

With time, may gravitational loads also occur in zones where water leakages have dissolved calcite veins, regardless if the zone containes swelling material or not.

Gravitational load

The gravitational load from a swelling clay zone is a function of many parameters. The main ones are:

- size and orientation of the zone in the tunnel;
- structure and content of soft materials with low shear strength (clay, chlorite and in particular talc);
- consolidation of the soft material;
- high tangential stresses across the zone caused by the tunnel;
- rock support method and time elapse between tunnel excavation and support installation;
- access of water after excavation and during operation of the tunnel;
- content of soluble minerals;
- joints in the adjacent rock masses, and their orientation with respect to the tunnel periphery. (If not supported by rock bolts, this could be the dominating load.)

The gravitational load will mostly be low if swelling results in no increases or a small volume of them. An example is the support with shotcrete shown in Fig 4. As high gravitational loads are mostly associated with concrete linings, it is recommended the gap between the roof and the concrete be partly grouted. The purpose of this grouting is to reduce these loads when found necessary. Water leakages must not be prevented however.

It might be of interest to know if the support chosen will collapse or not when deformation by overloading is observed. If wet conditions and no washing out or dissolving occurs at moderate depth in hard rock, the pressure in the zone will be maintained and the swelling will stop, which means that a future collapse will probably not take place.

Swelling forces from the zone	=	Max. swelling pressure measured in the lab	×	Correction for content of particles <20μm in the collected sample	×	Correction for consolidation pressure of the zone	×	Thickness of the clay fillings in the zone

Principles of calculating maximum swelling forces from the zone.

With regard to the load bearing capacity of support, refer to the studies made by Holmgren and Hahn, and by G Fernandez-Delgado et al[3,5,11].

Lining through swelling zones

Zones containing swelling clays will behave quite differently in dry and in wet tunnel conditions. Under dry conditions, no swelling will occur, which will cause stability problems. The main task here is to detect that swelling materials are present and to carry out investigations for the purpose of obtaining enough information to be able to dimension the final rock support for the future conditions the tunnel will be subjected to.

If the swelling materials obtain access to water during or after excavation, then swelling can be observed shortly afterwards. In rock masses with short stand-up time, where collapse may involve larger volumes, the speedy installation of rock support is important. The use of shotcrete seems at first most attractive as initial support. This method, however, requires a thorough description of the rock mass conditions and sampling of the clay and of the possible altered rock material *before* the shotcrete is applied and the rock masses covered, making later observations impossible. The information obtained will later be used for assessment of the final rock support.

While shotcrete has been destroyed several times by swelling forces in Norway, no collapses have occurred when unreinforced concrete horseshoe linings have been used as initial support provided the minimum thickness was 5% of the tunnel span (minimum 30cm). The whole profile in a section has to be lined continuously with good-quality concrete with larger thicknesses in high, planar walls. The reason for this positive experience is the arching effect of the lining and the fact that this method allows some initial swelling to take place, which highly reduces the mobilised swelling pressure on the construction.

Shotcrete is best used to increase the stand-up time in rock masses of very low stability, thus preventing collapses and rock falls during concrete placing.

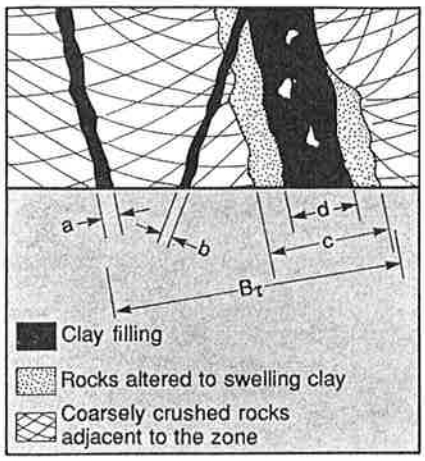


Fig 5. Field measurement of the thickness of clay fillings. With swelling clay only in the fillings, $B_t = a+b+d$. If two types of swelling clay occur, B_t will be $a+b+d$ and c-d respectively. B is the total thickness of the zone.

It is important in all tunnel constructions to prevent progressive collapse taking place and developing. In swelling zones with especially short stand-up time a pore pressure reduction and small step-by-step excavation and rock support by shotcrete followed by concrete lining, as mentioned above, has been found most useful in preventing collapses from starting.

The removal of collapses caused by swelling clays is often a hazardous job. When progressive, larger collapses occur in swelling zones, individual rock falls of various sizes will take place at unpredictable intervals. In cases where the collapse opening is covered by the collapse material, it is often impossible to know how the collapse has developed. Attempts to stablise such loose material by grouting and bolting have not been successful. Stabilising by freezing requires water-saturated, tightly packed material and low groundwater movement. The forepoling method requires a very low content of larger blocks, and a reasonable friction angle of the collapse material. Before the tunnelling method is chosen, it is important that the situation be carefully examined and considered in its entirety.

The best solution for tunnelling may be achieved with a by-pass tunnel which is excavated by a step-by-step excavation and rock support (Fig 6). This is best done in water tunnels where variation of the alignment is of no consequence. If the material in the zone has a very low consolidation, long drainage holes have to be drilled high above the roof in order to reduce the pore pressure before the by-pass tunnel advances into the zone.

Tunnel collapses caused by swelling clay zones have taken place in Norway for the following reasons:

- high swelling pressure acting on shotcrete supports;
- large gravity loads by the rock material caused by loss of friction and shear strength as a result of swelling;
- high water pressure build up.

As in most other fields of engineering the best understanding of the processes taking place is often gained from failures. The experience collected from tunnel failures in swelling zones has given valuable information used in the development of laboratory procedures as well as in the principles for calculating the pressure on the rock support.

The failures have also shown the consequences of underestimating the swelling rocks. Swelling zones must therefore be treated seriously both with regard to sampling and description of the rock mass as well as assessment of the loads to be applied in the design of the rock support. This is particularly important when shotcrete is used.

In spite of several collapses in swelling zones in water tunnels in hydroelectric power plants in Norway, most of the repair work was carried out without expensive loss of electricity production. This is due to the fact that the work was done at a time when the water could be collected in the reservoir(s). The cost of the repairs has, however, in some cases been considerable.

Acknowledgement

As in most other cases where geology is involved, a great variety exists in the

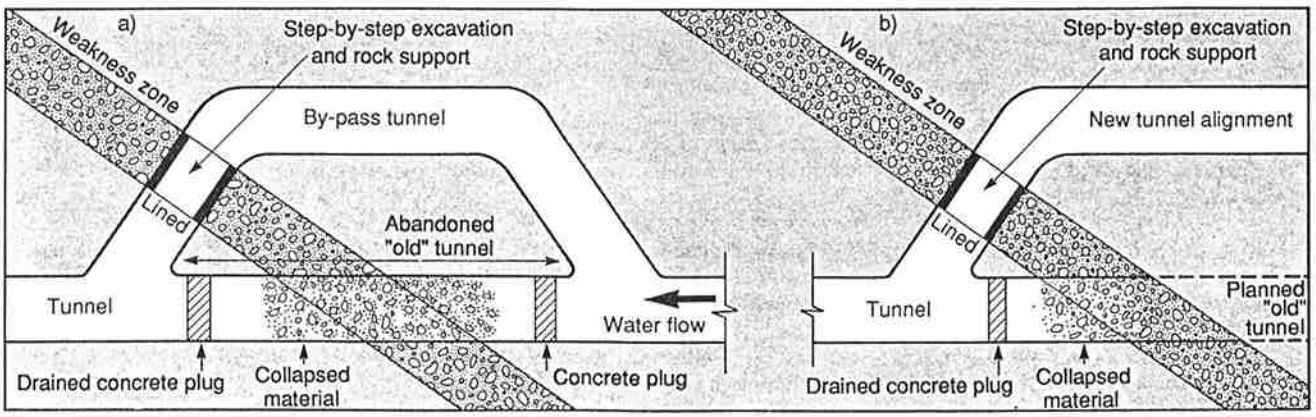


Fig 6. Examples of by-pass solutions if large, progressive collapses occur in tunnels conveying water: a) slide occurred after tunnel excavation was completed; and b) during tunnel excavation.

occurrence and behaviour of swelling clay zones. The difficulties in systemising all the factors involved to make this interesting subject accessible to readers is fully emphasised. Dr Syver Froise, Berdal Strømme AS, has kindly made valuable improvements in the text and has corrected the English. □

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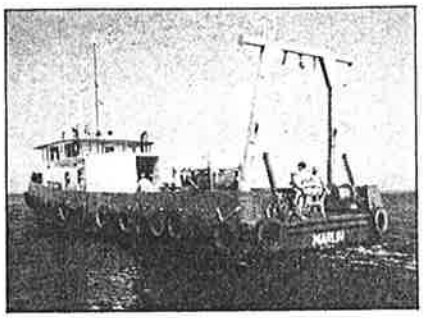
Going for gold

The British Department of Trade & Industry (DTI) Enterprise Initiative and the Quality Scheme for Ready Mixed Concrete (QSRMC) have joined forces to make a video, 'Going for Gold,' whose message is clear: quality is the key to competitiveness and will pay dividends after 1992. QSRMC itself received quality approval in January 1989 when it received a certificate of accreditation from the National Accreditation Council for Certification Bodies, a national body set up by the DTI in 1985 to uphold the standards of certification bodies. The standards set by QSRMC require certified companies to operate a quality management system in accordance with BS 5750 1987. **Express Enquiry 108**

Offshore oilfield technology aids Boston Harbour survey for clean-up

Marine geophysical surveying techniques adapted from offshore oilfield technology are being employed by Mott MacDonald on the $6bn Boston Harbour Clean-up project. As the harbour geology is complex and difficult to survey conventionally, geophysical pro-



filing with muti-channel digital surveying is being undertaken to expand geological data required for the 8km cross-harbour wastewater tunnel and 15km ocean outfall involved in the clean-up project (*T&T*, Oct '89, p35).

In a $450 000 contract including computer processing and geological reporting, the offshore surveying work around Nut Island and Deer Island has produced some 240km of digital data, which is currently being processed in Houston by Texseis. Favourable weather conditions enabled the contractor, Seattle-based Williamson and Company, to complete the survey in just eleven days. The contract was awarded by Mott Hay Inc, part of the Mott MacDonald Group. Mott Hay Inc is acting as subconsultant to Metcalf & Eddy, the lead design engineer, on the Boston Harbour Clean-up project for the Massachusetts Water Resources Authority.

Mott MacDonald and specialist engineering geophysical consultant J Arthur & Associates applied similar techniques on the Channel Tunnel to predict key strata levels. The two firms are currently working in association to develop further oilfield technologies for civil engineering, in particular the use of land-based vibroseis surveying.

Express Enquiry 105

Japanese jumbos

European hydraulic drilling jumbos are well known and widely used, but Japanese models have so far tended to be used only in Japan or by Japanese contractors working abroad. As with other Japanese equipment, it is only a matter of time before the scene will change.

Furukawa manufactures a successful all-hydraulic 3-boom wheel jumbo capable of drilling large tunnel sections up to about 11m high and with cross-sections of 25m^2 to 100m^2. The operator deck is movable up and down in accordance with lifting and lowering of the boom yoke.

The variable stroke drifters make drilling possible in all rock types from soft to very hard. The multifunction boom diminishes overbreak. The same jumbo can be used as a 2-boom machine since one boom may be demounted.

The Furukawa offers rapid boom positioning, fast travelling capability, a charging cage capable of covering a wide area, and safe drilling because of its special anti-jamming mechanism. The low noise level, good, mist-free visibility, reduced vibration and exhaust gas cleaning system give improved working conditions in the tunnel.

The electric cable and water hose reels are hydraulically driven. As you might expect on a Japanese machine, there is a wealth of automatic controls and safety devices. The automatic guide shell parallelism device is operable both horizontally and vertically. The booms are rotatable through 400° and extendable over 1.6m. The jumbo has a distinguished slope-climbing performance with 4-wheel drive.

Jumbos are equipped with the hydraulic drifter HD135, which has a built-in adjuster controlling the impact rate and impact force to suit the rock type of the tunnel face. Elongated pistons improve impact energy transmission efficiency which, it is claimed, gives better drilling performance even on very hard rocks and extends bit and rod life.

All kinds of advantages are usually claimed by equipment manufacturers, but the real test comes when the contractor actually uses the machine to drive a tunnel. Japanese equipment has been underestimated in the past, but has often turned out to be the world's best in the long run. Only time will tell when and whether Japan's drifters and jumbos will beat the best available in Europe. But we are sure to see more of them about outside Japan. **Express Enquiry 106**

Construction health

The Building Advisory Service (BAS) has combined with Environmental Management to provide more information on achieving a healthier environment for the construction industry. The newly formed team has the expertise to deal with: noise surveys; building surveys to locate hidden hazards; toxic substances reviews; monitoring dust, gas and chemicals; safety policy reviews; in-house and public training courses; site health checks and first-aid training.

New legislation places responsibility on employers to take more care over employees' health. **Express Enquiry 107**