Stabilisation of a glacifluvial zone in the Oslofjord subsea tunnel with ground freezing

Stabilisation d'une zone glacifluvial dans le tunnel sous-marin d'Oslofjord avec la congélation de la terre

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ABSTRACT: The Oslofjord sub sea tunnel runs through a rock depression zone filled with glacifluvial material. The zone is highly permeable with a hydrostatic water pressure of 120 m. Artificial ground freezing was undertaken to get through this zone in a controllable and safe manner. The design of the frozen structure was verified with 3D finite element analysis. New experiences were gained within mechanical behaviour of frozen soil with saline pore water, drilling, blasting, and casting procedure of the concrete lining.

RÉSUMÉ: Le tunnel d'Oslofjord passe a travers une dépression remplie de matériel glacio-fluvial. La zone était fortement perméable avec une pression hydrostatique de 120 mètres. Une congélation artificielle du sol a été entrepris pour obtenir une zone de manière sure et contrôlable. La conception de cette congélation a été vérifiée par une analyse en éléments finis en trois dimensions. Une nouvelle expérience a été acquise sur le comportement mécanique d'un sol gelé en présence d'eau saline, lors d'un forage, du souffle d'une explosion, et d'un procédé de moulage pour la doublure en béton..

1 THE OSLOFJORD TUNNEL

The Oslofjord-tunnel is part of a ring road project outside Oslo, Norway. The ring road covers a total distance of 26.5 km and includes 7 bridges and 6 tunnels. The longest tunnel is 7.2 km and crosses under the Oslo fjord and is the 19th subsea tunnel built in Norway. The tunnel is a three-lane highway with maximum inclination of 7%. A detailed description of the design and construction of the tunnel is given by Andreassen 1999 and Berggren 2000.

2 THE GEOLOGY IN THE OSLOFJORD REGION

The Oslofjord is situated in a major regional rift belt, the Oslo grabend, in which the total vertical displacement is about 2000 m. The tunnel crosses the rift belt within a gneiss region. The rift belt consists of several faults and weakness zones and some of the major ones follow the fjord over long distances. A seismic survey showed three wide channels in the threshold of the fjord, eroded by glaciation. In each of these channels, major weakness zones where confirmed. The "Hurum" weakness zone having the lowest seismic velocity of 2600 m/s was among other methods investigated with two penetrating core holes. One of the holes wAS made by directional core drilling. The core material consisted of crushed rock and some clay. The core drilling did not discover the weakness zone with high water pressure.

2.1 *Weakness zone*

About 80 m under the seabed and 120 m below sea level, the tunnel hit the "Hurum weakness zone". The weakness zone is at the bottom of a channel probably cut by a glacial melting river.

The weakness zone was first encountered from by a probe hole. As a routine, three 30 m long probe holes were drilled in the tunnel crown, having a 15 m overlap. Soon after the overlap with the previous probe, the new probe struck water at enormous pressure. The probe intersected with full hydrostatic pressure from the 120m head of the fjord above.

Further investigations of the area showed that the upper half of the tunnel cross section intercepted the loose glacial moraine at the bottom of the fjord. The zone consists of sand, gravel and blocks, and was highly permeable. In the lower half of the cross section hard but fractured rock prevailed.

It was decided to excavate a bypass tunnel under the main tunnel alignment, passing some 20 m under the glacial channel, and rising up to the main tunnel on the other side about 100m from the zone. From there the tunnel proceeded eastward under the fjord as well as backwards to the problem zone. The bypass tunnel is now used as drainage sump, replacing the one originally designed.

A 46 m long section was left between the two tunnel faces, including the approximately 15m wide zone of permeable glacifluvial and morainic deposits at the bottom of the deep channel, with good quality rock on each side.

It was attempted to stabilize the weakness zone by grouting, but after more than 700 tons of cemetiteous grout no reduction in water ingress and pressure was observed. Therefore ground freezing was chosen to get through the zone. The decision was based on assumed higher and more controllable safety for the freezing method compared to the uncertain results from continued grouting. At this time a bypass tunnel had been built, taking the freezing activities out of the critical time schedule.

3 MATERIAL MODEL FOR FROZEN SOIL

A creep based model for frozen soil, developed by Berggren 1983, was used in the design of the freezing zone. The model is intended for engineers in practical work and defines design strength as a function of temperature and load exposure time with active loading. The use of this creep model in design of the Oslofjord tunnel is also presented by Bergren 1999.

Classical creep theory defines three deformation phases for frozen soil: primary, secondary and tertiary creep, where the creep velocity is decreasing, constant and increasing with time respectively. A typical creep curve is shown in Figure 1.

The creep strength is defined as the maximum uniaxial stress that may be inflicted upon a material over a given time, with a decreasing strain rate.

This is at the transition between primary and secondary creep. At stresses above this limit the strain rate will increase with time.

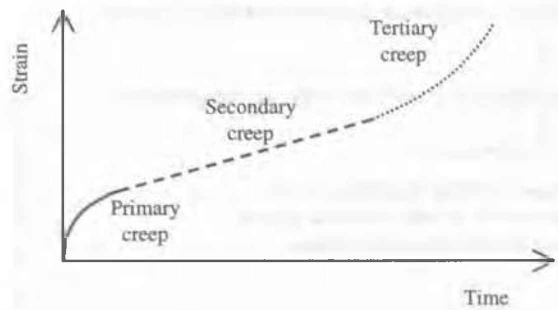

Figure 1 Typical creep curve for frozen soil

The design creep strength σ_d is expressed as:

$$\sigma_d = \frac{f_d \cdot \sigma_\theta}{\gamma_m} \tag{1}$$

Where: f_d = degree of mobilisation
σ_θ = reference strength at the temperature θ
γ_m = material coefficient

The degree of mobilisation f_d is defined as the ratio between the actual stress and the reference strength at the same temperature. The reference strength for a specific temperature σ_θ is defined as the maximum stress measured in constant strain rate uniaxial tests. The reference strength takes care of the temperature dependency. The degree of mobilisation takes care of the time dependency and is a function of the load duration. These relations can be found by laboratory tests on frozen samples.

3.1 *Laboratory tests*

Ordinary core drilling failed to give undisturbed samples. Soil flushing out of the drill holes was therefore gathered, compacted and saturated with seawater, frozen and tested. All tests on frozen samples were performed at the Geotechnical Department at the Norwegian University of Science and Technology (NTNU).

Unconfined compression tests were performed at –10, -20 and –28°C to investigate the temperature dependency of the material strength. The presence of saline water results in reduced strength of the frozen material. In Figure 2 the design strength for different frozen materials are compared.
The strength was remarkably low for temperatures higher than the eutectic point of natrium chloride at –21.3°C.Creep tests were performed at several stress levels to give the time dependency and interpreted according to the model. Due to the low strength at temperatures above -21°C, it was decided that the design temperature should be -28°C. The design reference strength

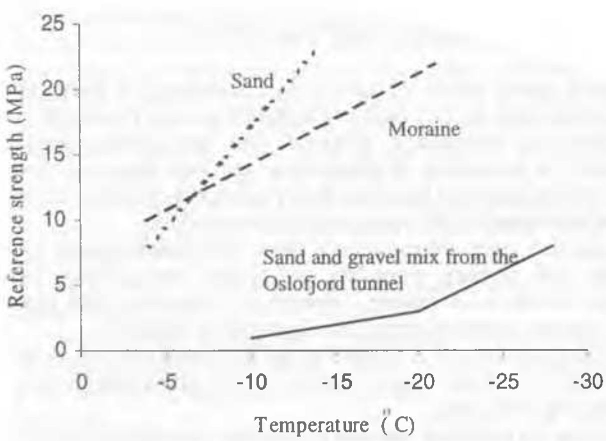

Figure 2 Reference creep strength for different frozen materials

at -28°C was found to be 8000 kPa. The maximum time period the frost structure had to carry load unsupported by the lining was two weeks. The design value of the degree of mobilization for two weeks loading was set to be $f_d = 0.25$. For the parts of the frost structure carrying load for only for one week the allowed mobilisation was set to 0.29.

4 LOAD AND MATERIAL FACTORS

4.1 *Material factors*

According to the Norwegian standard NS-ENV 1997-1 NAD case C a material factor for the strength of soil and rock of 1.4 can be used for this case. The standard does however recommend a higher factor when there is a danger for progressive failure. A material-factor less than 1.5 should therefore not be used for the frost structure of this type.

4.2 *Load factors*

In this case the most uncertain load is the effective stress from the unfrozen soil against the frost structure. Full effective overburden stress is 800 kPa, while arching effect may reduce this stress to less than 100 kPa. The water pressure is determined by the sea-level and therefore quite well known. One suggestion is to use 1.2 for the water pressure and 1.6 for the effective earth pressure. With a water pressure of 1200 kPa and an earth pressure estimated to be 200 kPa this gives a combined load factor:

$$\gamma_f = (1200 \cdot 1.2 + 200 \cdot 1.6)/1400 = 1.25 \tag{2}$$

4.3 *Lump factor of safety*

In problems involving soil structure interaction the use of factors on both loads and material is not always a straight forward procedure. In the design analysis presented in this paper we have therefore chosen to use a characteristic values for the loads and material factors for the frost structure equal to a lump factor of safety. A material factor of 1.6 combined with the load factors gives the following lump factor of safety $\gamma_{lump} = 1.6 \cdot 1.25 = 2.0$. The design creep strength is then:

$\sigma_{d,1\text{-week}} = 0.29 \cdot 8000$ kPa / 2.0 = 1160 kPa
$\sigma_{d,2\text{-week}} = 0.25 \cdot 8000$ kPa / 2.0 = 1000 kPa

In finite element analysis it is not practical to distinguish between frozen material staying unsupported for one and two weeks. The same strength of σ_d = 1000 kPa was therefore used for all the frozen material in the frost structure.

5 DESIGN OF THE FROZEN STRUCTURE

The design and construction of the frost structure is also presented by Berggren 2000. The external load on the frost structure consists of the hydrostatic water pressure (1200 kPa) and the effective stress from the soil in the weakness zone estimated to be 200 kPa.

5.1 *Thick wall pipe model*

For materials with high compression strength and low tension strength as frozen soil, it is advantageous to use the arching effect of a shell type structure. The arching effect of the frozen structure in the plane normal to the tunnel axis can be expressed by plasticity solution for a thick walled pipe with external pressure. The yield stress is equal to the design creep strength σ_d. The external pressure capacity is then:

$$\sigma_T = \sigma_d \ln \frac{R_i + T}{R_i} \tag{3}$$

Where σ_T is load capacity, $R_i = 8.2$ m is the internal radius under the tunnel crown and T is the frost structure thickness. By replacing the capacity σ_T with the load σ_n this equation can be solved in terms of T to find the required thickness.

$$T = R_i(e^{\frac{\sigma_n}{\sigma_d}} - 1) \tag{4}$$

Due to the high external load and the large radius the required thickness according to this equation becomes almost 20 m. This solution corresponds to excavation of the entire weakness zone before installing the supporting concrete liner. To reduce the required frost structure thickness it was therefore decided to excavate in sections of 2 -3 m and to install full concrete lining before advancing to the next section.

5.2 *Beam model*

With short excavation sections, the load carrying mechanism are similar to a beam supported by the frozen tunnel face at one end and the concrete lining at the other. The capacity of this "beam" must be checked both for shear capacity and compression strength.

The shear strength gives the following capacity:

$$\sigma_{b,s} = 2 \cdot \tau_d \cdot \frac{T}{L} \tag{5}$$

Where:

τ_v = ½ σ_d
L = unsupported length of the frost structure beam
$\sigma_{b,s}$ = load bearing capacity due to shear strength
T = thickness of the beam
σ_d = design strength of the frozen material

The "frost beam" may also carry the load as an arch from the frozen facing to the concrete lining as is shown on the figure. The thick walled pipe solution can be applied for this arch also.

$$\sigma_{b,a} = \sigma_d \ln \frac{R_{e,b}}{R_{i,b}} \tag{6}$$

The internal radius are $R_{i,b} = ½ L / cos(\alpha)$. An approximate expression for the external radius will be $R_{e,b} = T + ½ L \cdot tan(\alpha)$.

The shear stress at the supports are $\tau_v = \sigma_n \cdot L/(2T)$. The shear strength when $\sigma_{b,a}$ is the major principal stress with and angle α with the vertical are $\tau_{cap} = ½\sigma_d \cdot sin(2\alpha)$. This arch solution is only valid if $\tau_v < \tau_{cap}$. Using this requirement it can be shown that the angle α can be as low as 25° -27° degrees.

The frost structure will stay unsupported for two weeks at the concrete lining. The weakest support will determine the capacity. The two-week creep strength must therefore be used for the beam.

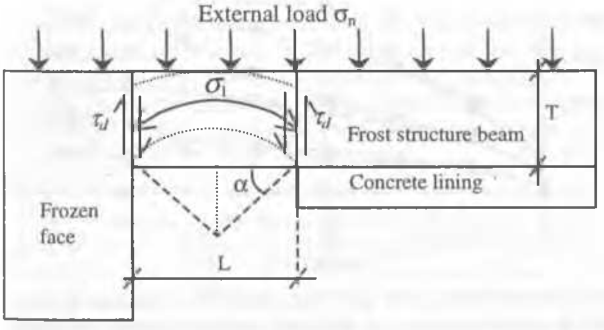

Figure 3 Beam model for design of the frost structure

The arching effect according to equation (3) will also give a contribution to the capacity. Due to the short open length this solution will only be valid for centre part of the span.

This contribution is therefore divided by two when calculating an average capacity for the open length. The centre part stayed unsupported for one week. The one-week strength is therefore used for the mechanism.

The smallest of $\sigma_{b,s}$ and $\sigma_{b,a}$ will define the capacity of the beam in the length direction. The total capacity is then:

$$\sigma_{cap} = ½\sigma_T + \sigma_b \tag{7}$$

With a material factor of 2.0 and a load factor of 1.0 and a 3 m thick frost structure the following capacities can be found:

Table 1 Capacity for the frost structure with 3 m frost thickness

L (m)	½σ_T (kPa)	$\sigma_{b,s}$ (kPa)	$\sigma_{b,a}$ (kPa)	σ_{cap} (kPa)
2.0	180	1500	1145	1325
2.4	180	1250	988	1168
3.0	180	1000	805	985

6 INTALLATION OF FREEZING PIPES

6.1 *Drilling*

Freezing pipes were installed in two rows from one side of the weakness zone. The drilling was extremely difficult due to high strength boulders in a matrix of sand and gravel. Due to the high water pressure a "down the hole" water driven hammer was used. A total of 115 holes where drilled of which 12 had to be abandoned.

6.2 *Freezing work installation*

Coaxial freezing pipes were installed inside the casing and grouted to achive good contact with the surrounding ground. Brine was used as cooling medium. The brine was cooled by an ammonia based freezing plant, placed in the tunnel. Heat from the plant was transported in steps; first to the cooling water running in a closed circuit, then to the tunnel air through a heat exchanger, and finally out of the tunnel through an extra set of tunnel vents placed by the heat exchanger.

7 TUNNEL EXCAVATION

Andreassen 1999 gives a more detailed description of experiences from the construction of the tunnel.

The Scandinavian Rock Group AS (SRG) excavated the Oslofjord tunnel. The full face of the tunnel was excavated from both sides of the fjord by conventional drill and blast methods, and with an average advance of 30 to 40 m a week at each tunnel face.

7.1 *Excavation of the weakness zone*

The full face of 130m² was planned to be excavated by means of short rounds and full concrete lining, before advancing to the next section.

The excavation was to start from the opposite end of the freezing work installations. Because the freezing pipe pattern was coned, the thickness of the frozen structure would decrease as tunneling proceeded. On the other hand, the freezing had lasted for a longer time period and hence the structure had grown somewhat thicker. To achieve as long rounds as possible, there was a meeting each week to decide the length of next round, based on the temperature and thickness of the frozen structure at the place to be excavated. Sections were planned to vary between 3.0 and 1.5m.

The time schedule for each round of drill and blast, one week sharp, was important to keep due to the design requirement. Starting with blasting the planned procedure was:

- Wednesday: load the holes for blasting, blast and remove the masses, shotcreeting
- Thursday: drill for the next round
- Friday/Saturday: place reinforcement for the bottom slab, concrete the slab, prepare concreting of the vault
- Sunday: spare time (reserve, if problems occur)
- Monday: reinforce and install formwork for the tunnel vault
- Tuesday: concrete the vault.

7.2 *Shotcreting*

The vault and face were shotcreted, with layers up to 20 cm, to avoid falling stones as the surface thawed. To give a high early strength, 30 kg/m^3 of cement was replaced with Rescon Cem-1. With Rescon AF 2000 accelerator the compressive strength reached approximately 1 MPa already after 15 minutes, and 5 MPa after 3 hours. There were no problems due to the cold surface.

7.3 *Concrete lining*

The thickness of the concrete lining was 1.2 m in the bottom and 1.0 m in the vault. The lining was designed to take full water pressure as long term condition. The strength requirements before next blast, was 40 MPa and was reached after approximately 20 hours. The concrete temperature at casting time was +25°C. During curing the temperature increased with more than 60° C.

Strain and temperature was measured at several places in the concrete. According to Berggren 2000, the concrete close to the rock/soil was still pertly frozen four months after dismantling the freezing plant, and in average 30 to 40% of the predicted earth- and water pressure was loading the structure.

8 FINITE ELEMENT CALCULATIONS

8.1 *Finite element mesh*

The 3D model include over 6000 twenty noded volume elements. The element mesh were some of the elements are removed for better visualisation is shown in Figure 4. To reduce the model size, the lower half of the tunnel and the relatively strong rock below the tunnel centre was not included. The model covered 15 m in the tunnel axis direction, equal to the length of the weak zone.

The effect of strong rock on each side of the weak zone modelled with fixed displacement boundary conditions. The mesh was 36 m in the vertical direction and 25 m to each side of the tunnel centre line.

The 2D model had 400 elements and was a plain strain calculation of a vertical cross section in the tunnel axis direction.

The hydrostatic water pressure was included as a distributed load outside the frozen structure varying from 1250 kPa at the lower boundary of mesh to 1170 kPa at the top of the frost structure. The material outside the frost structure was modelled with it's submerged unit weight. The submerged weight from the material above the mesh, approximately 50 m of gravel sand and clay, was included as a distributed load on the upper boundary of the mesh.

8.2 *Material parameters*

The frost structure material was modelled both with a creep model and as an elasto-plastic material. In the analysis including

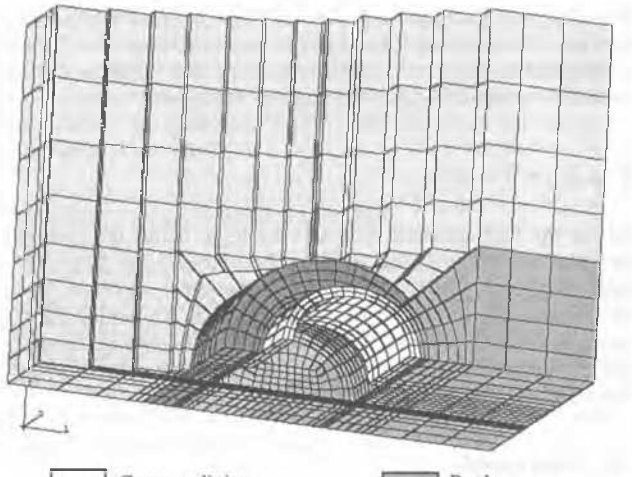

Figure 4 Finite element mesh, partly cut trough.

creep the power law creep model available in ABAQUS was used. In this model the creep behaviour is calculated according to:

$$\varepsilon^{cr} = Aq^n t^m \tag{8}$$

Where ε^{cr} is the uniaxial equivalent creep strain rate, q is the von- Mises stress, t is the total time and A, m and n are model parameters. The creep model was calibrated against four of the creep tests. Results from the calibration are shown in Figure 5.

The calibration resulted in the following creep model parameters: $A = 3 \cdot 10^{-17}$, n = 4 and m= -0.55. The elastic properties and short time strength was also specified, Youngs modulus E = 900 MPa and the uniaxial compression strength 8 MPa.The poison ratio (ν) for ice is close to 0.4 according to the book Geotechnical Engineering for Cold Regions. The poison ratio for the frost structure is believed to be governed by the ice, ν = 0.4 was therefore used for this material.

In the elasto-plastic analysis the E was reduced to 300 MPa to account for the creep deformations. The uniaxial strength was set equal to 1000 kPa corresponding to the two-week strength with a lump material factor of 2.0. A von-Mises model was employed.

For the unfrozen glacifluvial material the following material parameters was used, friction angle φ = 30°, attraction = 5 kPa, ν = 0.3, E = 150 MPa and a horizontal effective earth pressure coefficient K'_0 = 0.5. For the fractured rock and concrete the elastic parameters E = 10 GPa and ν =0.3 were used.

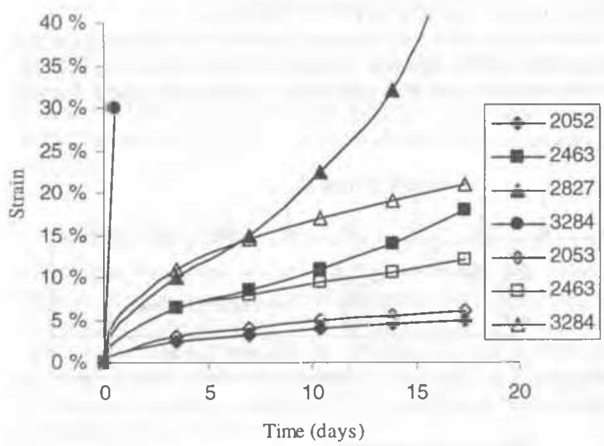

Figure 5 Results from creep parameter calibration. The legend corresponds to the uniaxial stress in kPa. Test results are plotted with filled markers and simulated results with open markers.

8.3 *Steps in the FEM analysis*

With in the time frame for the project, 3D analysis including creep of the frozen structure was not possible to complete due to long computation time. The creep analysis was therefore conducted in the 2D model.

The steps in both in the 2D and 3D the analysis was as follows:

1. The initial stress condition including the water pressure is established
2. The first 6 meters of tunnel is excavated and 4 m of concrete lining is added giving 2 m unsupported length. This gives deformations in the tunnel roof. For the creep analysis this step lasts 7 days.
3. The concrete lining with 1.5 m length is added stress free leaving 0.5 m is unsupported for 1 day.
4. The next blast is excavated. The length of this part was in the analysis varied from 1.5 m to 2.5 m giving 2 to 3 m unsupported length. For the creep analysis this step lasts 7 days.

8.4 *2D creep analysis*

The stress redistribution and creep deformations were analysed in a 2D model. Shortly after excavation the 2D analysis showed maximum von-Mises stresses in the frost structure close to 2500 kPa which is higher than the one week design strength with a material factor of 1.0. After seven days of creep the maximum von-Mises stresses in the frost structure was reduced to less than 1000 kPa. The stress redistribution due to creep does support the design assumptions of stress distribution in the frost structure according to plasticity theory.

The creep analysis showed that the majority of the stress redistribution and creep deformation was completed within the first 24 hours after blast and excavation, giving little load on the fresh concrete lining casted 6 days after the previous blast. The resulting calculated effectiv vertical stress against the top of the frost structure and vertical displacement of the tunnel roof is shown in Table 2. In the 2D analysis both the length of the unsupported zone and the thickness of the frost structure was varied.

8.5 *3D elasto-plastic analysis*

On basis of the results from the 2D analysis it was decided to continue with the 3D analysis using an elasto-plastic model for the frost structure with 1000 kPa uniaxial yield strength. The effective stress on the frost structure found in the 3D analysis varied from 40 to 100 kPa depending on the length of the unsupported zone. The effective vertical stress on the frost structure and vertical displacement of the tunnel roof is shown in Table 3. The 3D analysis gives larger vertical displacement of the tunnel roof than the 2D creep analysis. This is mainly due to the reduced elastic stiffness of the frost structure.

The stress distribution in the 3D analysis confirms that the frost structure carries the external load as an arch between the concrete lining and the frozen tunnel face. This mechanism can be seen in Figure 6 which shows contour plots of the von-Mises stress for a vertical cross section in the length direction of the tunnel. Only a small part of the mesh around the unsupported zone is shown. The results are from step 4 in the analysis.

The area with von-Mises stresses equal to 1000 kPa increases with length of the unsupported zone. With 3 m unsupported zone nearly all the elements in the frost structure above the unsupported zone are fully mobilised. This shows that with 3 m open length the lump factor of safety of 2.0 is fully utilised.

Table 2 Results from the 2D creep analysis

L (m)	T (m)	σ_v'	Displacement mm
2.0	2.0	110	35
2.4	2.0	110	36
3.0	2.0	110	38
2.0	3.0	125	24

Table 3 Results from the 3D elastoplastic analysis

L (m)	T (m)	σ_v' (kPa)	Displacement mm
2.0	3.0	100	37
2.4	3.0	70	40
3.0	3.0	40	58

Table 4 Material factor on basis of the stress from the 3D analysis

L (m)	γ_f	γ_m
2.0	1.23	1.70
2.4	1.22	1.54
3.0	1.21	1.35

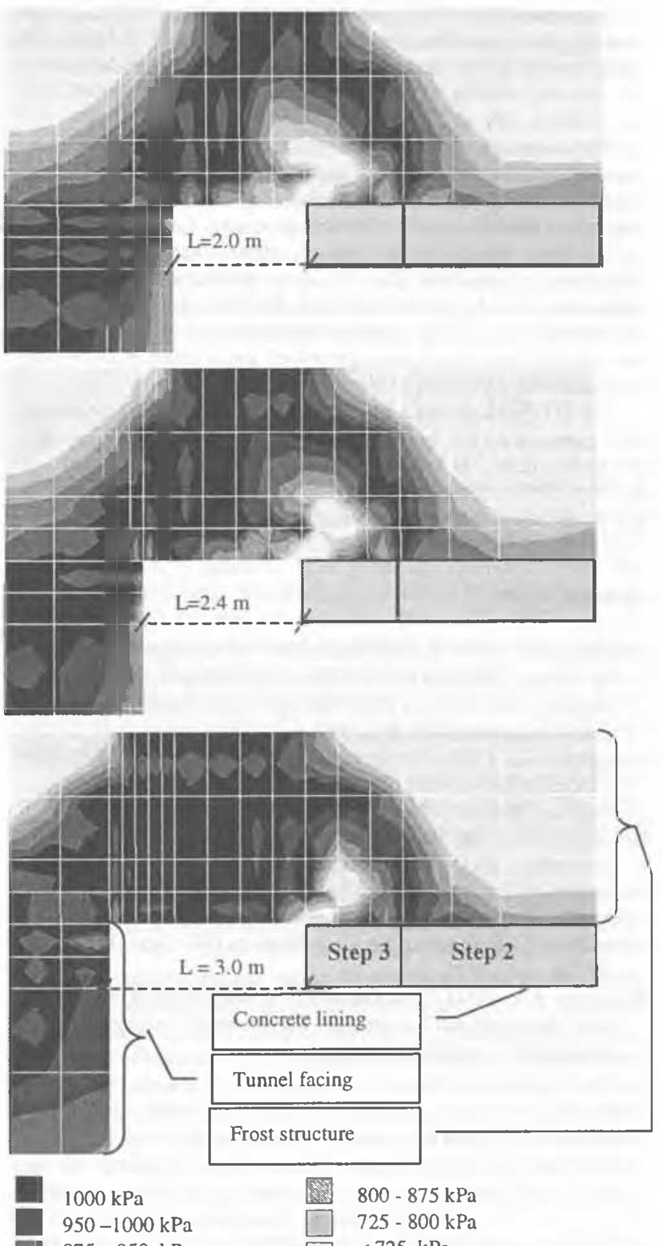

Figure 6 Contour plot of the von-Mises stresses with 3 m thick frost structure and 2.0, 2,4 and 3.0 m unsupported length.

8.6 *Material factors based stresses from the FEM analysis*

Using the effective vertical stress from the 3D finite element analysis and the analytical capacities from Table 1, more accurate material factors and combined load factors can be found. The resulting load and material factor are presented in Table 4. This shows that and open length of about 2.5 m meter combined with a thickness of the frozen zone of 3.0 m gives a material factor of 1.5 for the frost structure.

8.7 *Concrete lining stress analysis*

The 3D finite element model established for verification of the frost structure design was also used to find the stress condition in the concrete lining.

9 CONCLUSIONS

This case demonstrates that a very difficult situation was solved in a controlled manner by ground freezing. It was possible to plan, design and control all operations necessary. Drilling for the freezing pipes was the most challenging field task. There were neither stability problems nor any water leakage. Shotcreeting and concrete casting against the exceptionally cold ground surface worked very well.

The analytical model for design of the thickness of the frost structure and the length of the unsupported zone are verified by finite element analysis. The case also shows that creep model for frozen soil is well suited for practical design.

The finite element creep analysis shows that the stress concentrations redistribute due to creep deformations. A stable situation with only primary creep is therefore reached about one day after blasting. The stress redistribution due to creep does also support the design assumptions of stress distribution in the frost structure according to plasticity theory.

The 3D FEM analysis also shows that the effective vertical earth pressure on the frost structure are reduced from about 800 kPa to less than 100 kPa due to arching effects. The capacity of the frost structure estimated by analytical solutions is also verified by the finite element analysis.

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