In-situ frost heave loads in ground artificially frozen for tunnelling

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ABSTRACT: In the Netherlands the Westerscheldetunnel will be constructed with two tunnel bore machines and 26 connecting cross passages. The cross passages are built between the two main bored tunnels by using artificial ground freezing. During the construction phase of the cross passages an extensive monitoring program is carried out: (1) ground stress conditions and ground deformation and (2) deformation of the tunnel constructions – by measuring temperatures, frost heave loads and deformation. Measurements are taken in three different orientations: perpendicular, parallel and vertical to the cross passage with stress-monitoring stations and extensometers. Significant differences have been observed in loads and deformations, which are explained by different soil conditions. A significant variation in ground stresses has been monitored between the perpendicular and parallel direction with respect to the orientation of the freezing tubes. All separate phases of the construction activities of the cross passages (freezing, excavation, lining, thawing) are recognised clearly in the data.

1 INTRODUCTION AND OBJECTIVES

For the construction of the Westerscheldetunnel in the Netherlands 26 cross passages (or connections) are built between two parallel bored tunnels with artificial ground freezing (AGF) techniques (Hass et al., 2000). Special in-situ measurements of stress and deformation are carried out during the freezing period for the first two cross passages (DV1 and DV2) in order to evaluate the effects of the ground freezing technique. Due to the increase of volume of water to ice with 9% and ice lens formation frost heave loads are expected. The monitoring program includes measurements of stress and deformation development in the soil and the tunnel constructions. The objectives of this study are: (1) in-situ soil monitoring during the period of artificial soil freezing, (2) evaluation of the frost heave loads to the tunnel due to soil freezing and (3) increase of knowledge on use of ground freezing techniques in soft soils for tunnelling.

2 WESTERSCHELDE TUNNEL

2.1 Artificial ground freezing

The Westerscheldetunnel is located in the southwestern part of the Netherlands below the estuarine river that connects to the port of Antwerpen (Belgium). In order to increase safety at the Westerscheldetunnel the two lanes in each direction are designed as separate tunnel tubes. These two parallel tunnel tubes are bored with inner diameters of 10.10m. The total length of each bored tunnel tube is 6.6km. To increase safety in the tunnel two bored tubes are connected with 26 cross connections at every 250m. The cross passages are built from the inner side of the main tunnel tubes by using artificial ground freezing in order to make underground excavations possible. Artificial ground freezing techniques are used for both sandy soils and Boom clay because stiffness and permeability of all soil units in the trajectory is not considered to be sufficient for open excavation.

2.2 Construction of cross passages

The 26 cross passages will be constructed in different ground conditions. 9 cross passages are made in sand, 11 in the stiff clay and 6 in mixed (sandy) soils. The cross passage has an elliptical reinforced concrete lining with an thickness of 400mm, height of 2.75m and width of 2.5m. Outside these lining there is temporary shotcrete of 300mm thickness.

In the main tunnel tubes five special rings have been designed with a fixed sequence. In the middle ring steel segments replace two concrete segments. In these special enlarged steel segment the safety
doors to the cross connection are constructed (Fig. 1). 22 freezing pipes are bored through special watertight connections in the specially prepared ring segments (Fig. 2). After closing the period of soil freezing (a minimum thickness of 2m frozen ground) the excavation is started by the NATM-method. During the last phase the concrete lining of the cross passage is built and the freezing machine is stopped eventually for thawing the soil. During the excavation period the emergency door can be closed when an unsafe situation occurs.

after closure of the frozen body (cylindrical formed), a draining possibility was installed near the axis of the cross passage. With this draining system water pressure in the ice-enclosed area was released in a controlled manner (an increase of water pressure was measured in sandy soil at DV1). In order to safeguard a water-tight connection between the ice body and the tubbings, approx. 26 temperature sensors, distributed in a plane, are distributed in the western tube. With these sensors it was possible to monitor the ice connection by temperature of the frozen soil. In addition, a 10cm thick insulation is installed at the interior side of the tunnel tube. This insulation provided a faster connection of the ice body and warrants a longer remaining ice connection in case of a failure of the freezing system.

Figure 1. Situation of the western tunnel tube with the connection of frozen soil. The emergency door is opened after evaluation of the (frozen) ground temperature.

2.3 Ground freezing machine

Ground freezing is performed as brine glaciation (NaCl2) since a nitrogen glaciation in the tunnel was excluded for safety reasons. A freezing unit was developed for soil freezing at temperatures of the freezing tubes at approx. –38°C. This unit has a min. ammonia content of approx. 80kg for a performance of 95kW and numerous safety features. In a circular pattern 22 freezing tubes are bored with a distance of about 1.0m to each other and parallel to the cross way axis (Fig. 2).

The freezing tubes are connected to the main pipe by means of a flexible pipe. The complete piping system includes approx. 3m² brine. When starting the soil freezing system, the brine temperature at every brine head on the starting side is measured in order to check whether the freezing tube works properly and whether the freezing capacity is distributed evenly.

2.4 Safety valve

The spatial and temporal development of the complete freezing body was checked permanently. The freezing body has a target thickness of > 2.00m which was reached after approx. 26 days in sand at DV1 and after approx. 46 days in clay at DV2. Two diagonally arranged temperature monitoring devices controlled the formation of the frozen soil body. In order to avoid high pressure within the frozen body after closure of the frozen body (cylindrical formed), a draining possibility was installed near the axis of the cross passage. With this draining system water pressure in the ice-enclosed area was released in a controlled manner (an increase of water pressure was measured in sandy soil at DV1). In order to safeguard a water-tight connection between the ice body and the tubbings, approx. 26 temperature sensors, distributed in a plane, are distributed in the western tube. With these sensors it was possible to monitor the ice connection by temperature of the frozen soil. In addition, a 10cm thick insulation is installed at the interior side of the tunnel tube. This insulation provided a faster connection of the ice body and warrants a longer remaining ice connection in case of a failure of the freezing system.

Figure 2. Cross section of a cross passage with the position of freezing tubes, frozen soil body, shotcrete (outer lining) and concrete cast in-situ (inner lining).

3 SOIL CONDITIONS

3.1 Soil profile and geotechnical characteristics

Soil freezing was carried out at cross passages DV1 and DV2 in sand and clay (Table 1). Figure 3 gives a soil profile based on several geological borings, cone penetration tests and geophysical bore hole measurements. Soils of three geotechnical units will be frozen for the first two cross passages: Z1, BK1 and BK2. Boom clay units (BK1 and BK2) dip slightly towards the north and are covered with young coastal sands (Z1). Note the large difference
in plasticity and permeability between soil types at DVI and DV2 (Table 2). The top of the Boom clay varies strongly due to erosion (Figure 3). The axis off cross passage DVI is at 17.5m-NAP and is constructed in loose sand of Holocene age (Z1 unit). The axis of cross passage DV2 is located at a depth of 28.7m-NAP and is constructed in overconsolidated clay of Oligocene age (BK1 and BK2 unit). Both the top of the BK2 unit and the Z1 unit are characterised with a salt contents of 2000-13000 Cl mg/l. At both cross passages no significant ground water flow was recognised.

3.2 Frost susceptibility

Accordingly to the standards of the ISSMFE frost heave susceptibility of unit H1 (sand) is classified as 'negligible' and the overconsolidated clay of unit BK1 and BK2 as 'medium to strong'. Frost heave susceptibility is the tendency of the soil to expand during soil freezing - due to the volumetrical expansion of water to ice with 9% - and the growth of ice lenses. Note the difference in BK1 and BK2 in clay contents.

3.3 Natural stress conditions

In-situ stress conditions of frozen soil have been measured with stress monitoring stations and have been compared with calculated theoretically soil stresses (Table 3). Calculated stress conditions are based on soil columns and volumetrical unit weights. Measured and calculated horizontal stresses match very well (Table 3). The measured vertical stresses are significantly lower than calculated, probably due to the installation procedure of the monitoring equipment and remoulding of the soil.

| Table 1. Geotechnical units at the cross passages DVI and DV2 |
|---|---|---|---|
| location | geotechnical unit | lithology | geological formation |
| cross passage DVI | Z1 | SAND | Naaldwijk Formation |
| cross passage DV2 | BK1, BK2 | CLAY, silty | Rupel Formation (Boom clay) |

| Table 2. Soil properties of geotechnical units Z1, BK1 and BK2 |
|---|---|---|---|
| soil properties | Z1 | BK1 | BK2 |
| lithology | NEN510 fine SAND | 4 | 81.2 | 62.2 |
| clay contents (%) | 0 | 100 | 18.8 | 37.8 |
| sand contents (%) | (mean) | (mean) | (mean) | water contents w |
| γ wet | kN/m² | 15.5 | 19.4 | 19.3 |
| γ dry | kN/m² | 19.4 | 23.8 |

<table>
<thead>
<tr>
<th>cross passage</th>
<th>σ vertical [N/m²]</th>
<th>σ horizontal// cross passage [N/m²]</th>
<th>σ horizontal cross passage [N/m²]</th>
<th>σ vertical water pressure [N/m²]</th>
</tr>
</thead>
<tbody>
<tr>
<td>DVI (SM 1)</td>
<td>204 (309)</td>
<td>175 (153)</td>
<td>170 (151)</td>
<td>172 (168)</td>
</tr>
<tr>
<td>DV2 (SM 4)</td>
<td>310 (607)</td>
<td>289 (299)</td>
<td>291 (296)</td>
<td>279 (278)</td>
</tr>
</tbody>
</table>

Figure 3. Soil profile with the position and depth of the cross passage DVI and DV2. Soils of units Z1, BK1 and BK2 have been frozen during the construction of cross passages DVI and DV2.
In sand the water contents shall freeze almost instantaneously due to the high percentage of free water. Clay shows a different behaviour in the frozen state due to the strong bonding forces between water and clay minerals. In clay a percentage of water will be unfrozen at temperatures of \(-35^\circ C\). The grain size distribution also controls consequent differences in frost heave and strength development.

4.1 Frost heave and ice lenses

In freezing soil the formation water can segregate into ice lenses before it is frozen (Konrad & Morgenstern, 1981). As the result of growing ice lenses, a water pressure gradient will develop that is dependant on the grain size and permeability of the soil (Penner, 1986). The thickness of ice lenses can range from < 10\(\mu\)m to several cm’s. The freezing expansion of clay is partly caused by ice lenses. The growth of the ice lenses is enhanced by the (slow) migration of water at this frozen front. That is the reason why soils with low permeability such as silts and clays are frost heave susceptible and (drained) sand is not. The process of migration of water due to freezing temperatures is referred as cryosuction.

Konrad & Morgenstem (1981) have defined experimentally a linear relation between the frost heave rate \(h\) of a soil, the segregation potential \(SP_0\) and the temperature gradient \(T\):

\[
h = SP_0 \cdot \text{grad } T
\]

This linear relation between temperature gradient and frost heave rate has experimentally been established with samples of the Boom clay from the tunnel trajectory (Fig. 4). Resulting \(SP_0\)-values smaller than 0.5\(m^2^\circ C\) resulted in a low category of frost susceptibility. Following experiences in civil engineering with AGF works, frost heaves were expected in the direction perpendicular on the freezing front or isotherms and not in the direction parallel to the axis the freezing tubes.

Three-dimensional frost heave behaviour of Boom clay has been experimentally investigated and reported by Rijkers et al. (2000).

5 MONITORING PROGRAM

5.1 Monitoring equipment

In order to observe changes in stress, temperature and deformation a special monitoring program was designed. This program consists of water pressure recorders, stress monitoring stations, spade cells, temperature recorders, extensometers and inclinometers. The stress monitoring stations consist of three flat plates that record water pressure and temperature. The three plates can measure stresses directed parallel, perpendicular and vertically with respect to the freezing tubes. The inclinometers have also been installed, but results are not discussed in this paper. Extensometers are located at the top of each cross passage and consist of four packers at different depth levels. The packers are fixed in the soil and connected to the ground surface with bars. At the surface the shortening of the bars (distance between the packer and the surface) is measured. Water pressure recorders are situated on most of the instruments. During the entire period of ground freezing and thawing at cross passage DV1 and DV2 data was recorded.

Due to the large amount of recorded data and different data types, it is not in the scope of this paper to address all measurements. We restrict this paper to the highlights of stress monitoring stations (SM1 and SM4; Figs 6-7) and extensometers (EX2 and EX3; Figs 8-9) at cross connections DV1 and DV2. In table 4 the reported data types are summarised.
Table 4. Characteristics and orientation of monitoring data of cross passage DV1 and DV2

<table>
<thead>
<tr>
<th>data type</th>
<th>orientation</th>
<th>station</th>
<th>data record</th>
<th>soil unit</th>
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<tbody>
<tr>
<td>cross passage DV1</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>tangential stress</td>
<td>vertical</td>
<td>SM1</td>
<td>SM1-1</td>
<td>Z1</td>
</tr>
<tr>
<td>axial stress</td>
<td>parallel to freezing tubes</td>
<td>SM1</td>
<td>SM1-2</td>
<td>Z1</td>
</tr>
<tr>
<td>radial stress</td>
<td>perpendicular to freezing tubes</td>
<td>SM1</td>
<td>SM1-3</td>
<td>Z1</td>
</tr>
<tr>
<td>water pressure</td>
<td></td>
<td>SM1</td>
<td>SM1-4</td>
<td>Z1</td>
</tr>
<tr>
<td>deformation of soil above cross connection</td>
<td>vertical</td>
<td>EX2</td>
<td>EX2-1 to EX2-4</td>
<td>Z1</td>
</tr>
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<td></td>
<td></td>
</tr>
<tr>
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<td>SM4</td>
<td>SM4-1</td>
<td>BK2</td>
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<tr>
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<td>parallel to freezing tubes</td>
<td>SM4</td>
<td>SM4-2</td>
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<td>EX3-1 to EX3-4</td>
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</tr>
<tr>
<td>temperature</td>
<td></td>
<td>TE2</td>
<td>TE2-2</td>
<td>BK1</td>
</tr>
</tbody>
</table>

6 STRESS AND DEFORMATION

6.1 Monitoring results

Data records of stress monitoring, extensometers and temperature data are given as time series in Figures 6-9. Stress is given in these graphs as absolute values. In the upper parts of the graphs main construction phases of the cross connections are given, such as freezing, excavation and covering.

6.2 Stress deviations from SM1 at DV1

When the freezing process starts the frozen front has not immediately reached the monitoring instruments (Fig. 6). Radial and axial stresses in the surrounding unfrozen soil (sand) decrease. From the moment where the freezing front reaches the monitoring instrument (temperature < 0 °C), the instrument is part of the frozen soil. The frozen soil is expanding due to the continuing freezing process. Inside the frozen soil radial, axial and tangential stresses increase. Consequently the cylindrical frozen soil body is tightening.

The first construction phase is to excavate the heart of the cylindrical frozen soil body. Support to the frozen soil from inside is then removed. Axial and tangential stresses increase, while radial stress decreases. Shortly after excavations the frozen soil rises slightly in temperature, because the excavated area is in contact with relatively hot open air and hydration heat of the shotcrete. Temperature alternations cause deviations in the stiffness of the frozen soil and other equilibrium of stress distribution over the total cylindrical frozen soil. It leads to axial and tangential stress increase and radial stress decrease at the point of the stress monitoring stations.

The freezing process is continuing and the temperature in the total cylindrical frozen soil body decreases further. These actions provide radial and axial support from the inside. Radial soil stress increases, the frozen soil reacts against the shotcrete. Axial and tangential stresses decrease.

The freezing apparatus starts working with a lower capacity (33%). Temperature is increasing all over the frozen soil in a fast rate. Stress deviations also occur at a fast rate and stresses are redistributed over the frozen soil. The decrease in stiffness (due to temperature rise) leads to a decrease in radial and axial stresses. An overall radial stress release leads to an overall tangential stress increase. After the fast increase the temperature stabilises, causing a stabilisation of all stresses.

Finally, ground freezing stops and temperatures start to increase in the direction of the freezing point. The same process as freezing with a lower capacity recurs. There is one difference. Radial stress increases immediately after shutting the freezing apparatus. This reaction is probably caused by the redistribution of stresses over the frozen soil, with this difference that the total distribution of stiffness over the frozen soil is different as it was during the previous period of temperature increase. The final tangential stress increased with respect to the starting value. This is a result of the installation procedure of stress monitoring stations. The vertical stress after installation is less than the original vertical in-situ stress (Table 3), probably because soil has been excavated via a pulse boring to install this type of instrument.

6.3 Water pressure deviations from SM1 at DV1

There are no large water pressure deviations while freezing in sand. The maximum amount of water pressure decrease is approximately -15kPa. The sand is well drained and the decrease is a result of the drive out of water during volume increase of frozen water.

6.4 Soil deformations from EX2 at DV1

In order to make installation of the instruments possible, soil is improved at both cross passages at ground level. Besides, an extra cinder level is deposited. The extra load causes settlement to occur. In de beginning before the period of freezing this settlement is visible as a shortening of the packer bars.
After a while when the soil around the freezing tubes freezes and expands, packer EX2-1 reacts and shortening takes place (Fig. 7). Packer EX2-2 to EX2-4 react in a later stadium. The frozen front progresses slowly. In the period of excavation the shortening drops (most for packer EX2-1 and respectively less for packer EX2-2 to EX2-4). The excavation causes a radial release and therefore a contraction of the cylindrical frozen soil body.

After this period deviations in deformation are low. Packer EX2-1 to EX2-4 show similar fluctuations conformable with radial stress observed at SM1. Further decrease in temperature causes a small expansion.

The freezing apparatus starts working with a lower capacity. Packer EX2-1 shows a small decrease in shortening followed by an increase. Packers EX2-2 to EX2-4 stabilise and increase. The freezing process continued although at a lower level. The frozen soil is still expanding.

Thawing leads to an overall decrease in shortening. The residual shortening is the result of the surface settlement for packer EX2-2 to EX2-4. Packer EX2-1 has a larger residual settlement. In fact this packer was the only one of EX2 packers inside the frozen soil.

### 6.5 Stress deviations from SM4 at DV2

All stresses increase strongly from the beginning after a frozen soil body (clay) starts forming (Fig. 8). Also, tangential stress increases. The reason for this strong frost heave is the typical undrained character of clay. Water pressure initially increases with the same amount as the tangential stress. Radial and axial stresses increase conformable with expectations, as the freezing soil expands, causing a stress increase in the surrounding unfrozen soil due to freezing of the water contents of the clay and due to ice lens formation.

From the moment the temperature drops under the freezing point, water pressure decreases in a fast rate to just above the vacuum pressure. Radial, axial and tangential stresses react and show a slight decrease. The water pressure is not as dominant any more as most of the water starts to freeze. Radial stress increases again to about a level where all three stresses start to decrease. This decrease in stress is due to creep or the volumetric contraction of ice at lower temperatures. Creep occurs at a certain level of temperature and stresses. The crystalline structure of the frozen water is not stable anymore and the soil more or less liquefies. At cross passage DV1 stresses in all principal directions increase in this period. The stress rate is much lower. Excavation leads to a radial release. Axial stress increases as the support from the centre is removed. The cylindrical frozen soil body contracts and tangential stress increases.

Just after excavation the periods of cover and freezing with lower capacity begins. Temperature increases for a period of time and decreases again after a while. Radial stress increases slightly while the freezing process is continuing, only at a lower rate. Tangential stress increases fast. These phenomena take also place at cross passage 1. Temperature increase leads to a fast tangential stress increase, caused by the redistribution of stresses and stiffness deviations in the cylindrical frozen soil body. The axial stress decreases also due to the redistribution of stresses over the covered cross connection in the meantime. Finally, thawing does the same to the tangential stress as freezing with lower capacity. Tangential stress increases rapidly and due to a combination of creep and thawing the tangential stress is reduced to its original starting level. The axial and vertical stresses decrease by thawing. The total expansion stops and stresses decrease. The water pressure finds its original level at a temperature just below the freezing point.

### 6.6 Soil deformations from EX3 at DV2

The same soil improvement and cinder installation takes place at cross passage DV2. The same amount of settlement is visible in the beginning before the freezing period (Fig. 9). From the moment the freezing starts, expansion of the frozen soil body takes place. The difference between cross passage DV1 and cross passage DV2 is clear. The maximum shortening at DV1 is approximately 10 mm at packer EX3-1. The maximum shortening at DV2 is approximately 50 mm at packer EX3-1. Excavation leads to a decrease in shortening. The cylindrical frozen soil body shrinks. From this moment temperature increases and freezing continues at a lower level. A smooth small expansion is the result. Thawing leads to the final decrease in shortening and all packers return to their original position including the surface settlement. The residual shortening is large compared to DV1.

### 6.7 Drainage of the frozen soil and insulation measures

The average temperature evolution was the same as calculated for both cross passages. However, at the connection between the main tunnel tube and the frozen ring the temperature drops slower than expected. At the steel segments it is clear that the conduction of heat is high and that proper insulation was necessary. At cross connection DV1 dry-ice was eventually used to maintain low temperatures at the connection between the steel segments and the frozen ring.
Figure 6. Measurements of stress monitoring station SM1 at cross passage DV1 (sand).

Figure 7. Measurements of extensometer EX2 above cross passage DV1 (sand)

Figure 8. Measurements of stress monitoring station SM4 at cross passage DV2 (clay).

Figure 9. Measurements of extensometer EX3 above cross passage DV2 (clay).

Figure 10. Development of the water pressure in the unfrozen cylinder at DV1 (sand).

For drainage during soil freezing a borehole was used to control the water pressure inside the frozen body. Water expands when it is frozen, so in an undrained situation water pressure rises when the frozen ring is watertight (Fig. 10, point B). In the first part the increasing water pressure (Fig. 10, point A) gives an indication that the ring of frozen soil is watertight. In the case of DV1 the monitored temperatures through the tunnel segments show that soil was not frozen directly behind the segments. Another test was conducted with opening and closing the drainage tap (Fig. 10). Because the water did not stop we concluded that the frozen ring was not watertight. After one day using ‘dry-ice’ a significant rise of the water pressure occurs that proved the water tightness of the frozen ring.

At the cross passage DV2 it was not possible to drain the inner part of the frozen ring because of Boom clay with very low permeability. However, the drainage pipe was kept open to reduce ground pressure on the main tunnel segments.
7 FROST HEAVE LOADS AND TUNNEL DEFORMATION

During freezing activities tunnel convergence and offset of tunnel segments was measured (in the bored tunnel tube). Deformation and convergence of the main tunnel construction have been monitored during the freezing period. At cross passage DV1 (in sand) a maximum offset of tunnel segments of 4mm was observed. The deformation of the western tunnel at DV1 has been caused by the increase of ground pressure due to frost heave of 50kPa measured in the direction parallel to the freezing tubes (SM1 SM2 ⊥ //; Fig. 11).

Frost heave pressure is defined as the increase of ground pressure due to soil freezing. The absolute measurements of frost heave pressures at DV1 and DV2 are higher (see Figs. 6-7).

At cross passage DV2 (in overconsolidated clay) the maximum offset of 20mm was registered. Also the corresponding measured frost heave values are significantly higher. In the direction parallel to the freezing tubes 680kPa of frost heave stress has been observed that eventually caused 20mm offset of the tunnel segments (SM3 SM4 ⊥ //; Fig. 11).

8 CONCLUSIONS

Frost heave loads and soil deformations have successfully been measured in (frozen) soil bodies of sand and clay during an artificial ground freezing activities. Frost heave pressures differ significantly between frozen sand and frozen clay bodies. All phases of construction of the cross passage are recognized clearly in the data.

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LITERATURE


