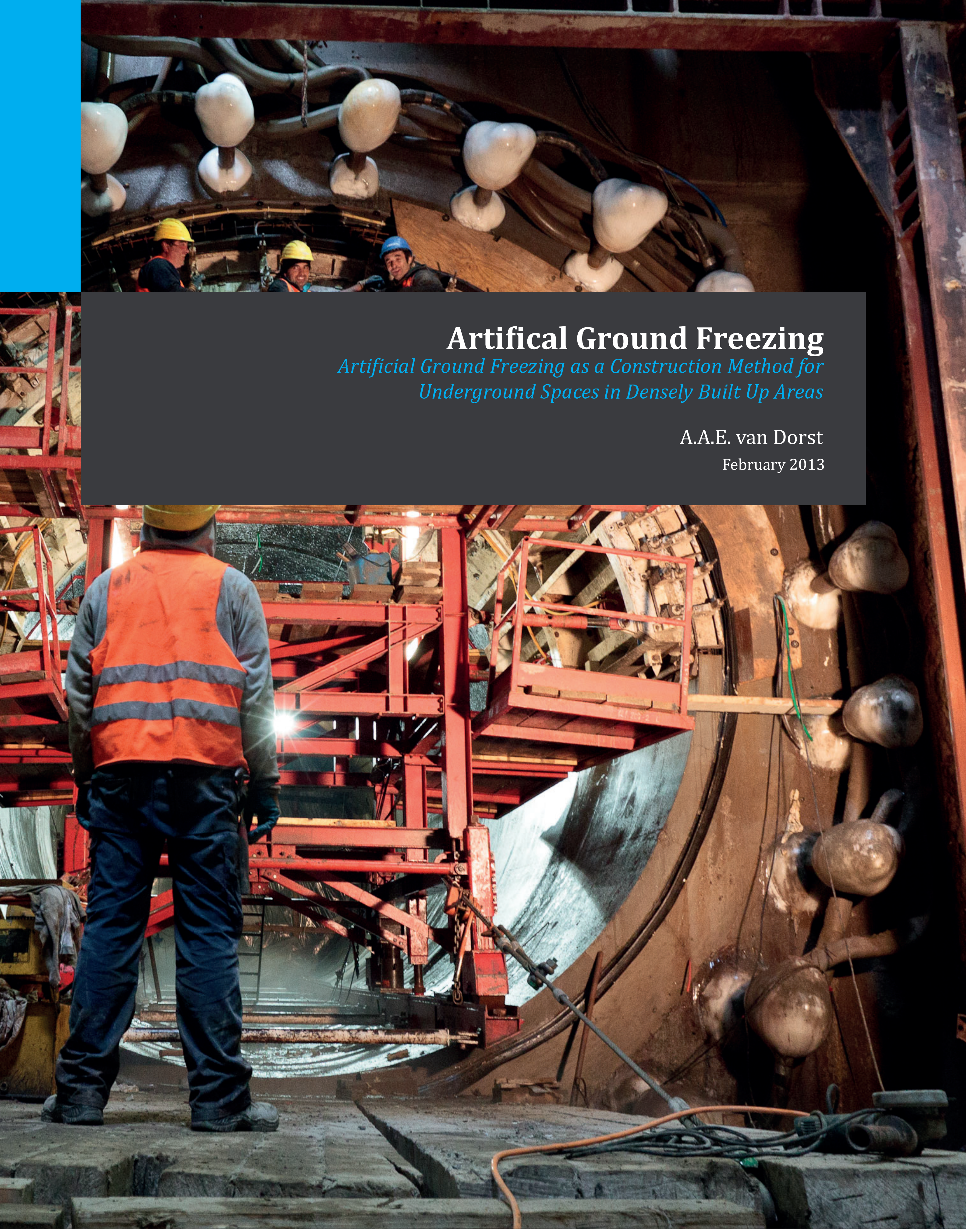

Artifical Ground Freezing

Artificial Ground Freezing as a Construction Method for Underground Spaces in Densely Built Up Areas

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February 2013

TU Delft Delft University of Technology

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Challenge the future

ARTIFICIAL GROUND FREEZING AS A CONSTRUCTION METHOD FOR UNDERGROUND SPACES IN DENSELY BUILT UP AREAS

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February 2013

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PREFACE

This master thesis concludes my research on the feasibility of an underground station constructed with artificial ground freezing. It is written within the framework of my graduation at Delft University of Technology and was conducted at the department of Geo-Engineering at the Faculty of Civil Engineering & Geosciences.

I worked with great pleasure on this thesis and the results would not have been achieved without the support and supervision of several people. This master thesis is supervised by the graduation committee consisting of prof. ir. J.W. Bosch, dr. ir. K.J. Bakker, ir. R.R.E. Vervoorn, ir. F.J. Kaalberg, and ir. V.M. Thumann. I would like to thank my graduation committee for their advice, critical comments and effort they put in to my work.

Renée van Dorst

Delft, February 2013

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SUMMARY

Artificial ground freezing is being applied more in more in underground construction. Still it is mainly used on a small scale but lately also freezing is used (or planned to be used) on a larger scale in projects. This study first looked into the current developments in the field of artificial ground freezing. It is found that in Germany, where the technique is already further developed, the most freezing projects already have been executed. In addition in Amsterdam recently directional drillings to install freeze pipes

To determine whether soil freezing is a viable option for underground construction in densely built up areas a case study has been performed for the North-South Line in Amsterdam. With the help of current developments found in the field of artificial ground freezing four different options of constructing a station were developed. The option most suitable to apply in Amsterdam was a construction phasing in which:

- two tunnel tubes are bored through diaphragm walls constructed for two entrance shafts;
- then the shafts are excavated and freeze pipes are installed from both shafts towards each other surrounding the circumference of the station to be built;
- the diaphragm walls are partly demolished at depth to achieve an entrance to the station and being able to excavate soil within the frozen soil body;
- in steps the soil is excavated, shotcrete is applied and eventually the final lining is made;
- after finalizing the shell of the station the frozen soil body is thawed.

The North-South Line in Amsterdam is being constructed at the moment and is planned to be ready for operation in 2017. The metro line consists of seven stations of which five are being built below ground level. Station Rokin is chosen to elaborate the alternative station design for as on beforehand the feasibility of the design at this location seemed the most promising. The choice of location is mainly made based on the alignment of the tunnel and local soil profile.

The design was modelled in Plaxis, software based on the finite element method and intended for geotechnical analysis. To obtain the correct input parameters the processes occurring as a result of a freeze-thaw cycle were studied. Most important processes are the creep behaviour of the frozen soil, heave as a result of formation of ice lenses in low permeable soils and changing soil parameters due to the freeze-thaw cycle. The German research centre CDM performed tests to determine the strength of frozen soils present in the subsoil of Amsterdam at certain points in time. These results are used to reduce the stiffness of the frozen soil stepwise in Plaxis. Related to heave less clear information was available. Based on both information found in literature and research performed by Deltares volume strain increments are applied in Plaxis to account for heave.

Results of the calculations in Plaxis have led to the conclusion intermediate freeze walls in the cross section are necessary to keep settlements within the required boundaries. With those freeze walls the settlements are reduced to less than 1/3 of the original settlements. The uncertain behaviour of the Allerod has led to a minimum and maximum boundary for the actual settlement value. It is shown with Plaxis the effects of the Allerod behaving as a clay after thawing (remoulded behaviour) would have large effects on the total settlements. Settlements would almost be doubled.

It is concluded an alternative design for station Rokin using artificial freezing would be technically feasible, but the behaviour of the Allerod after thawing should be further verified. However, if the worst case scenario occurs the maximum settlements are not exceeded largely. Mitigating measures could be taken to keep the settlements within boundaries. In general one can conclude in advantageous soil conditions artificial soil freezing is definitely a construction method to consider for underground construction in complex situations in densely built-up areas.

From an economical viewpoint the freeze design for station Rokin can compete with the currently executed design. The costs of the structural works are estimated at 85 million euro for the freeze design and 60 million euro for the current design. Although the freeze design is a more expensive design the costs are not two or three times as much with respect to conventional techniques as is often thought. Also it should be noted that in the cost estimation costs of rerouting cables and ducts and hindrance to the surroundings are not yet taken into account. This is in advantage of the freeze design.

CHAPTER 1

INTRODUCTION

The application of freezing techniques is currently limited to situations where more commonly used techniques cannot be used. In Germany on the other hand freezing is already used more extensively, not only to build ground and water retaining walls but also, for example, for constructing parts, or even complete, underground stations.

Freezing is a costly technique. However in special situations, for instance when a subway station has to be created at limited distance below or close to existing buildings, freezing could be an attractive solution. Freezing can have many advantages over conventional construction methods (e.g. no hindrance on ground level, less likely to have leakages), but there are also some obstacles to overcome (e.g. costs, influence on soil properties).

This master thesis aimed to do research on the applicability of freezing techniques as a construction method in densely built up areas. To determine whether freezing is a technical feasible solution and if it could be a more attractive solution than other construction methods, the focus was laid on a case study: 'An alternative design for one of the stations of the North South Line in Amsterdam'. For which station the design was to be made was determined based on the results of the first part of the thesis.

This has led to the following research question:

→ *Can soil freezing be a viable option as a construction method for constructing underground stations in densely built up areas?*

CHAPTER 2

ARTIFICIAL GROUND FREEZING IN GENERAL

The earliest documented use of freezing techniques was in 1862 in the mining industry. In the South of Wales a mineshaft was constructed using freezing techniques. After the first application in 1862 it was patented in 1883 by Poetsch. His technique is still used nowadays. For a long time soil freezing was only applied in the mining industry. Its share in the civil industry to construct below surface is still not large, but it is growing as a result of increasing complexity in underground construction.

2.1. Goals of the Ground Freezing Method

There are two main characteristics of freezing soil which makes it an interesting method to use in underground construction. Firstly, when groundwater is frozen an impermeable layer is formed due to the impervious nature of ice. Secondly the frozen soil-water mixture is very strong. As with cement in concrete the ice bonds the soil particles which gives the mixture strength. In addition, ground freezing is possible in all strata. The effects of the freeze-thaw cycle are however different in each strata.

To freeze soil a constant energy input is needed, therefore its application is limited to temporary structures. A structure will be made under the temporary protection of the surrounding frozen soil and when this structure is completed the frozen soil will be thawed. Freezing is a reversible process, thus after thawing no improper materials are left in the subsoil. The construction site will be left in its original state, although soil properties and volume can be changed.

In earlier days only vertical freezing was applied. Circular cofferdams, mainly for the purpose of mining, were the most well-known application of vertical freezing. Soils with low strength are stabilized and construction below groundwater level is possible due to the impermeable layer that is formed. Nowadays horizontal freezing is also possible which has led to a wider scale of applications, for instance in the field of tunnelling.

In mining the technique proved to be a safe and cost-effective method, but in civil engineering the technique is often treated as a last resort. It is viewed as an expensive solution, which it often is when it is used as a crisis solution. If, however, it is implemented early in the design phase of a project freezing, could be a cost-effective solution.

2.2. Methods to Freeze Soil: Execution Aspects

What kind of refrigerating plant will be installed in order to freeze the soil depends on the type of coolant used. Mostly a choice is between two main types of coolants when soil freezing is applied; a brine solution or liquid nitrogen. Each coolant has its own advantages and disadvantages and the application depends on the requirements of the construction works. With each coolant, however, a distribution system is required and freeze pipes will be installed.

2.2.1. Brine Freezing

Most common in the application of freezing techniques is the use of brine as a coolant. Poetsch, who patented soil freezing, used brine already in 1885. Over the years there is much experience has been gained with this coolant and it has proved to be a successful way of freezing soil.

Freezing using brine is also known as the indirect freezing method. Indirect due to the fact that a brine, usually a calcium chloride solution, is used as secondary refrigerant. A primary refrigerant is used to cool down the secondary refrigerant which is pumped around the freeze pipes in a closed circuit. The primary refrigerant is in most cases ammonia, but freon is used as well. Ammonia is toxic but seven times as cheap as freon in use and it does not contribute to the greenhouse effect. Freon is a greenhouse gas.

There are requirements on the refrigerant which cools down the soil and this is the reason it is not desirable to insert ammonia or freon directly into the freeze pipes. The requirements of a coolant are listed below.

- The volume of the coolant in gas is small.
- There are no high pressures needed in order to condense the coolant.
- The coolant is not toxic, explosive or flammable.
- The boiling point of the coolant is low.
- The coolant does not affect parts of the refrigerating system.

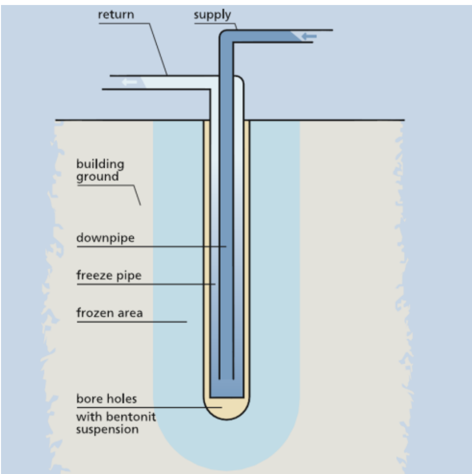


Figure 2.1- Schematic view of a freeze pipe

In some cases freon is used as single refrigerant and inserted directly into the freeze pipes. This can result in a gain of 15 [%] in thermal efficiency, but it also has a considerable disadvantage. Freon has no smell and therefore leak detection is difficult and expensive.

The brine is cooled down to a temperature between -30 [°C] and -40 [°C] before it is inserted in the soil through a downpipe into the freeze pipe. Via the annulus (space) between the downpipe and the freeze pipe wall and with the help of a pump the brine flows back into the return pipe. The temperature of the returning brine is 2 [°C] to 3 [°C] higher than the inserted brine. The heat is extracted from the soil surrounding the freeze pipe. The schematic layout of a freeze pipe is shown in Figure 2.1, a figure by Max Bögl. The refrigerant system is a closed pipework system, all brine is reused. After passing through the freeze pipes the brine is chilled again in the evaporator by the primary refrigerant.

Next to the freeze pipes also a compressor, cooling tower, evaporator, condenser, pumps and valves are needed for the total freeze system. There are three sub circuits in the system; (1) the cooling agent circuit, (2) the refrigerant circuit and (3) the cooling water circuit. The system is usually driven by an electrical motor, sometimes by a diesel motor. In Figure 2.2 (also a figure by Max Bögl) the layout of total freeze system is shown.

The refrigerant circuit contains the ammonia (or Freon). The compressor liquefies the ammonia gas due to which its temperature rises to 100 [°C]. A pump transfers the liquid ammonia into the condenser under high pressure. Here the ammonia passes several coils and is being cooled down by water from the cooling tower. The water extracts heat from the ammonia. Then the ammonia passes the expansion valve and is being sprayed with high pressure into the evaporator. In the evaporator the pressure is lower, which results in the ammonia transforming to gas and a temperature drop. The ammonia now has a temperature around -35 [°C].

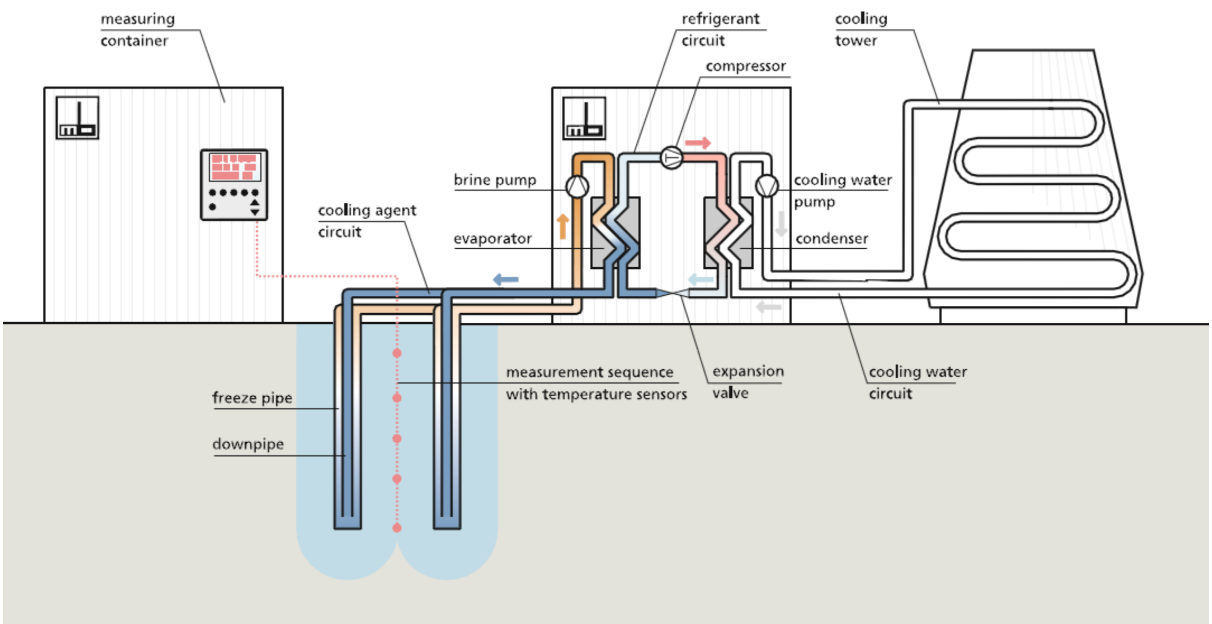


Figure 2.2- Schematic diagram of brine freezing method

The time from commencing of freezing to a contiguous ice-wall may take three weeks to three months depending on freeze pipe spacing, the plant capacity and the scale of the project. Freeze pipes usually have a diameter of about 10 [*cm*].

2.2.2. Liquid Nitrogen Freezing

No primary and secondary refrigerant are needed with this method. This method is also called the direct freezing method because liquid nitrogen serves as a single coolant. The liquid nitrogen has a temperature of -196 [°C] at a pressure of 1 bar in liquid form.

Liquid nitrogen is produced on a large scale at air liquefaction plants and transported by special trucks to the construction site. A refrigerating plant is not necessary, because the liquid is supplied into a vacuum storage tank or directly into the freeze pipes. In the storage tank the nitrogen wants to evaporate, but is not able to due to the vacuum pressure. This forces the liquid through the freezing circuit, without needing the help of a pump.

In the freeze pipes, due to their relatively high temperature, the nitrogen evaporates and warms up quickly. It exits the pipe into the atmosphere in gaseous form. Nitrogen is not toxic or flammable and constitutes about 80% of the air; therefore it is not a problem that it enters the atmosphere. However, nitrogen in large concentrations can be lethal. Retrieving the nitrogen and liquefying it again is not feasible, therefore the supplied nitrogen can only be used one cycle. A constant supply of liquid nitrogen is necessary with this method.

Freezing with nitrogen is fast due to the extremely low temperature of the coolant. The time from commencing of freezing to ice-wall closure may take two to six days depending on freeze pipe spacing, the plant capacity and the scale of the project. The freeze pipes usually have a diameter of about 5cm. In Figure 2.3 the layout of a construction site using nitrogen freezing is shown.

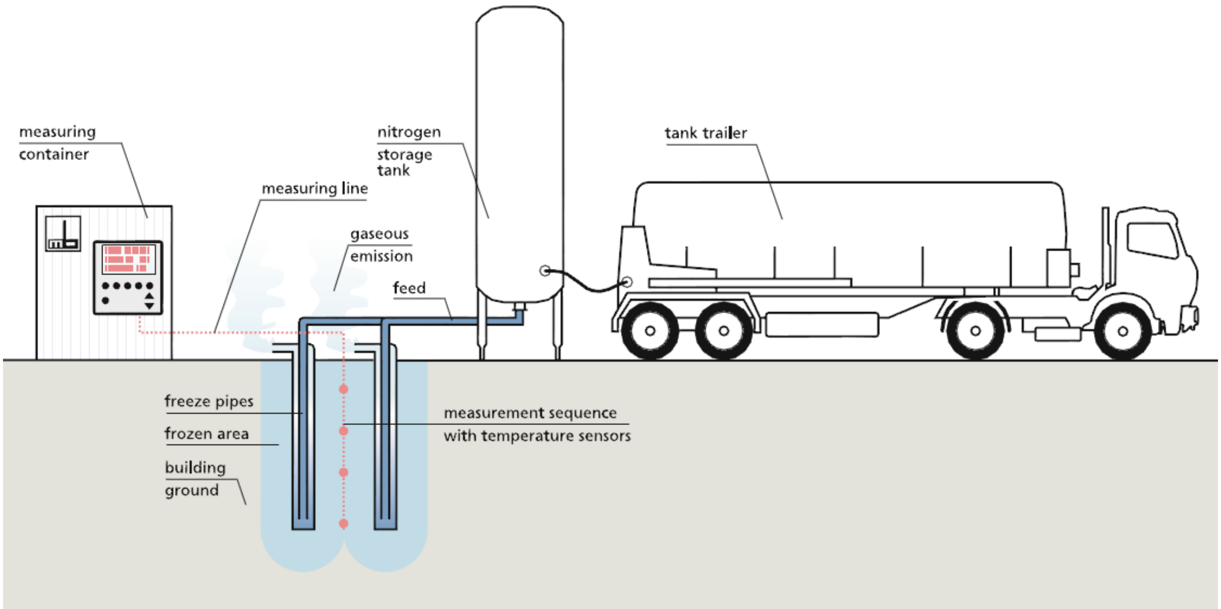


Figure 2.3 - Schematic diagram of liquid nitrogen freezing method (Figure from Max Bögl)

2.3 Advantages and Disadvantages of the Two Methods

Two refrigerants are necessary to freeze the soil when using brine and therefore a refrigerant plant needs to be installed. At the construction site there needs to be space for such a plant. During the freezing process the refrigerating plant produces noise, which could be a problem for the surroundings. For brine freezing a long freezing period is needed. The freezing time till wall-closure takes three weeks to three months. As a result of the long freezing period the soil attracts more water and the expansion is larger. Heave can be a result.

On the other hand brine is relatively cheap en the method is simple. Since brine is being used as a coolant for over 100 years it is certainly a reliable method to artificially freeze ground. Passing through one freeze pipe the brine does not warm up very much, only 2 to 3 degrees. As an effect of this the brine can be circulated through several freeze pipes instead of only one. A more or less uniform frozen soil body is then formed.

The biggest disadvantage of liquid nitrogen freezing is the not being able to reuse the coolant after one cycle. Therefore a constant supply of new coolant is necessary which makes this method relatively expensive. Good access roads are essential in this case. Nitrogen can be suffocating, in large quantities, therefore a good ventilation is of importance. As the nitrogen warms up quickly when entering the freeze pipes the frozen soil around the freeze pipes will have an irregular shape. The nitrogen comes in contact with the freeze pipe wall for the first time at the end of the freeze pipe. Therefore the frozen body grows faster at the bottom of the freeze pipe than at the top.

The process of freezing with liquid requires a space needed on ground level which is similar to the space needed for brine freezing. Nitrogen freezing uses a more simple construction site. The methods biggest advantage is that its freeze-up period is short compared to brine freezing. The freezing of a closed frozen soil body takes in general only two to six days, due to the low freezing temperatures that can be reached with nitrogen.

In general one can say brine freezing is the most suitable when construction takes a longer period and is on a big scale. Liquid nitrogen freezing becomes more interesting when a project has a short duration and a smaller scale. When constructing an underground station in densely built-up areas it will be a large scale project with a long construction time. Brine freezing is than the most suitable economically and logically seen. The constant needed supply of nitrogen could lead to problems in an inner city. Also, because freeze pipes will be installed from a building pit the risks of freezing with nitrogen are considered too large.

The advantages and disadvantages are listed in Table 2.1, after to Stoss and Valk (1979).

Table 2.1 – Comparing freezing with brine and freezing with liquid nitrogen

Site installations	*Brine*	*Liquid nitrogen*
Pumps and electric power	Required	Not required
Water for cooling	Required	Not required
Refrigeration plant	Required	Not required
Storage tank	Required	Required
Pipe system for distribution coolant	Supply and return	Supply only
Low temperature material for surface pipes, valves, etc.	Not required	Required
Low temperature material for freeze pipes	Not required	Not required

Execution of freezing	*Brine*	*Liquid nitrogen*
Physical condition coolant	Liquid	liquid/vapour
Minimal temperature achievable (theoretic)	-34 °C with $MgCl_2$ -55 °C with $CaCl_2$	-196 °C
Re-use of coolant	Standard	Impractical
Control of system	Easy	Difficult
Shape of frozen body	Regular	Often irregular
Temperature profile in frozen body	Small differences	Great differences
Frost penetration	Slow	Fast
Impact on frozen body in case of damage to freeze pipe	Thawing effect	None
Noise	Little	None

2.4 Applications of Freezing Techniques

Where in the beginning freezing was only used in mining engineering its applicability nowadays has been enlarged. Fields in which freezing is applied are tunnelling, junctioning, underpinning of buildings and creating the retaining walls for shafts and building pits. In this chapter several construction projects in which freezing has been applied are reviewed. They are listed based on the field in which they are applied and the year construction was or will be finished. Underpinning is often used in combination with one of the other three fields, so this one is left out in the list below.

Tunnelling

- Vienna Subway Section U 6/3 (1989)
- Vienna Subway Section U 3/10 (1991)
- Düsseldorf Subway Section 3.4H (1993)
- Boston Central Artery/Tunnel Project (2002)
- Nuremberg-Fürth Underground Section 3.1.1 (2004)
- Munich Subway Station Marienplatz (2006)
- Amsterdam North/South Line: Damrak (2011)
- Berlin Subway U5: Station Museumsinsel (2019)

Shafts and building pits

- Rotterdam Subway Station CS (2009)
- Amsterdam North/South Line (2017)

Junctioning

- The Netherlands Westerschelde Tunnel (2003)
- Bremen Weser Tunnel (2003)
- Antwerp Liefkenshoek Rail Link (2014)
- Cologne North South Urban Railway (2015)
- The Netherlands Botlek Railway Tunnel (1980)
- Rotterdam RandstadRail: Cross passages (2010)
- Amsterdam Hubertus Tunnel (2008)
- Amsterdam North/South Line: Cross passages (2017)

In the following sections information is given about the afore mentioned projects. At the end of each section a summary table is made of the important characteristics for each project. These characteristics, if they could be found in literature on the specific project, are:

→ Year of completion
When was the project completed, or is it still in progress?

→ Soil type
What is the geology of the soil? In which soil types did the construction take place? Was the groundwater table of influence on the construction works?

→ Freeze method
Was the freezing done using brine or liquid nitrogen? Were the freeze pipes installed vertically or horizontally? What was the time needed to complete the frozen body? How many freeze pipes were used? What was the capacity of the refrigeration system?

→ Length and shape
Over what distance was the freezing done? Which shape was frozen? Was the freezing done in different phases?

→ Heave and settlements
Did heave occur during the freezing process? After thawing of the soils did settlements occur? The settlements mentioned occurred at ground level. Was the strength of the soil reduced as a result of the freeze-thaw process?

Vienna Subway Section U 6/3

Tunnelling underneath a telecommunication building

For subway section U6/3 in Vienna two tunnel tubes will be excavated using compressed air, dewatering and the NATM method. The construction of the tubes takes mainly place in a silt layer and partly in a sand layer, shown in Figure 2.4. Compressed air and dewatering are necessary to keep groundwater out of the excavation. Part of the route of the two tunnels underpasses the foundation of a telecommunication building with only a 1.6 [m] cover at the minimum. To ensure stability of the soil a plate of 1 [m] thick and a plan area of 65x22 [m] was frozen above the tunnel crown.

Due to high costs and problems with nitrogen delivery in the city, brine was used to freeze the soil. An extensive testing program was performed beforehand to determine the best means of interrupted freezing and evaluate the in-situ behaviour of soil. The program consisted of two fields, one with dimensions of 30x9 [m] and one with dimensions of 12x7 [m]. In Figure 2.5 one can see the cross section of frozen soil body in the biggest field at 30 [m] from the start shaft. The frozen body has an irregular form due to inhomogeneities in the soil and deflection of the pipes, but it is 1m thick along the entire cross section.

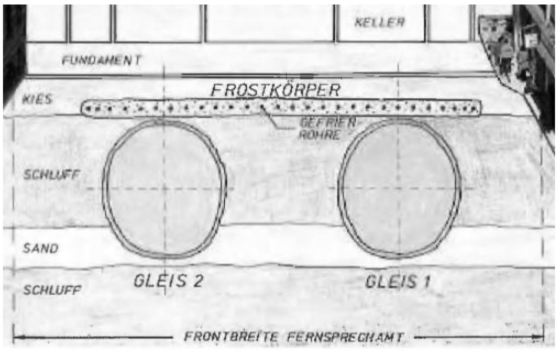


Figure 2.4 - Schematic view of two tunnel tubes and the frozen body (after Fisher, 1987)

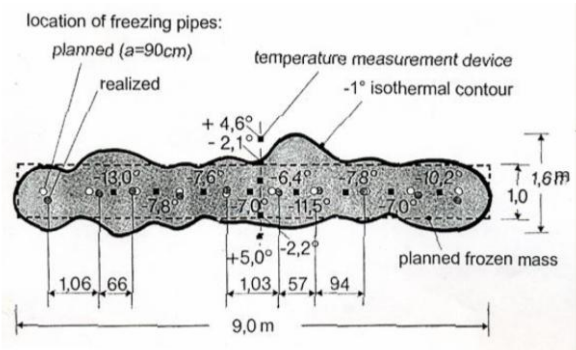


Figure 2.5 - Cross section of the frozen body at 30m from the starting shaft (after Arz et al, 1988)

The thickness of 1 [m] of the frozen body of 65x22 [m] was reached in eleven days. Freeze pipes were placed 0.9 [m] apart. After reaching the desired thickness the freezing was intermittent. The freezing continued for 2.5 to 7.5 hours a day, instead of 24 hours, to limit heave effects (Semprich, 2005). A frost heave of 13 [mm] was measured. The heave did not jeopardize the requirement on maximum skew of 1:1000. As a result of excavating the tunnels underneath the frozen body and creep of this body, the heave reduced to 3 [mm] to 8.5 [mm]. No further settlements occurred due to thawing.

Table 2.2 - Characteristics Vienna Subway Section U6/3

Year of completion	1989
Soil type	Mainly sandy gravel, but also silt Groundwater flow played no role
Freeze method	Horizontal brine freezing 11 days needed for freezing
Length and shape	65m Horizontal 1m thick plate, 65mx22m
Heave and settlements	13mm heave 3.5mm to 10mm settlements due to excavation tunnels No settlements due to thawing

Vienna Subway Section U 3/10

Nitrogen freezing underneath the Herzmansky Department Store

In section U3/10 of the Vienna subway the two tunnel tubes proceed in a double deck position. As a result of the transition the upper tube underpasses a department store with a cover of only 2 [*m*] up to the foundation of this store. Nitrogen freezing is used to stabilize the soil.

The frozen soil body is mainly made in sandy gravel, with a water content of only 4 [%] to 5 [%]. Underlying this sandy gravel layer are stiff clayish silts. Laboratory tests were done to see if freezing was possible in this subsoil. The conclusion was that with a minimum water content of 3 [%] the freezing will cause sufficient cementation of the soil. The free groundwater level is situated very deep in this area, several metres below the lowest excavation, thus groundwater was not an obstacle.

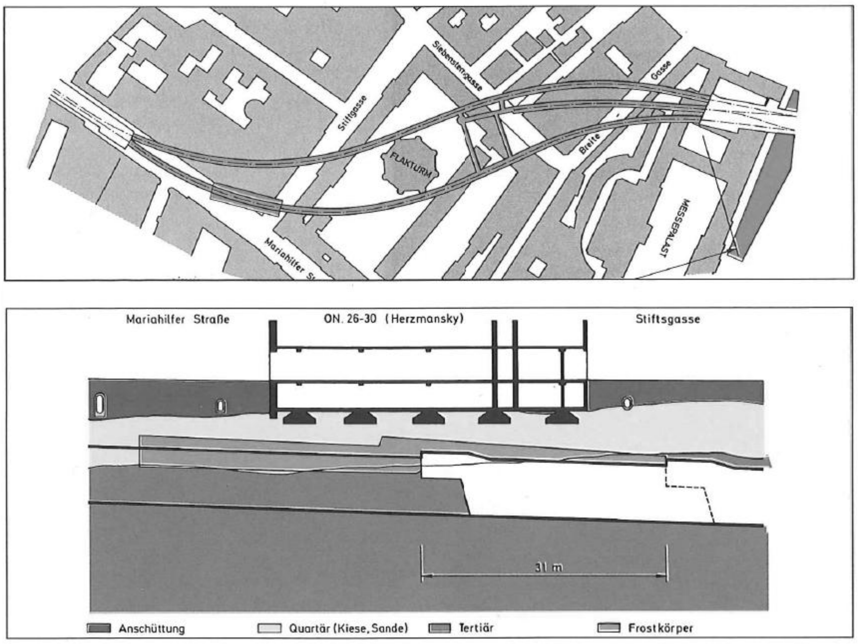


Figure 2.6 - Location and longitudinal section of the freezing segment (after Hinkel, 1991)

In two phases of 35 [*m*] long a cover with total length of 70 [*m*] was frozen above the tunnel to be constructed. In four days one of the two phases was completed. Only the roof of the tunnel was frozen in a vault-like shape. When the frozen body was at desired dimensions the freezing was intermittent. Six hours of freezing followed by 18 hours of tunnelling a day. This way heaving was kept to a minimum and the temperature was more or less constant. Measurements showed 1 [*mm*] to 2 [*mm*] heave and net settlements of 7 [*mm*] to 9 [*mm*]. The nitrogen consumption overall was 900 [l/m^3] of frozen soil (after Hinkel, 1991).

Table 2.3 - Characteristics Vienna Subway Section U3/10

Year of completion	1991
Soil type	Mainly sandy gravel, but also clayish silts Construction above groundwater
Freeze method	Horizontal nitrogen freezing Temperatures between -60 °C and -80 °C before entering soil Consumption of 900 l/m^3 of frozen soil
Length and shape	70m, frozen in two phases of 35m Frozen body in a vault-like shape above the tunnel crown
Heave and settlements	1mm to 2mm heave 7mm to 9mm net settlements

Düsseldorf Subway section 3.4H

Freezing three 40m long tunnels

In Düsseldorf in 1993 a mass transit expansion of the subway system took place. Three tunnels of about 40 [*m*] were to be constructed. There was very little space between the roofs of the tunnels and foundation of the buildings above. The smallest overburden had a thickness of 0.6 [*m*]. No settlements were allowed, therefore ground freezing was chosen as a construction method in combination with the NATM.

The subsoil consists of gravel and sand with a groundwater table about 15 [*m*] below ground level. Also unsaturated soil was frozen in this project. The soil located above the groundwater table water was injected so that the soil mass could be frozen. Vertical cut-off walls were grouted to prevent the water from running off. This measure ensured enough bearing capacity of the frozen soil mass. The freezing was done in four phases, which can be seen in Figure 2.7.

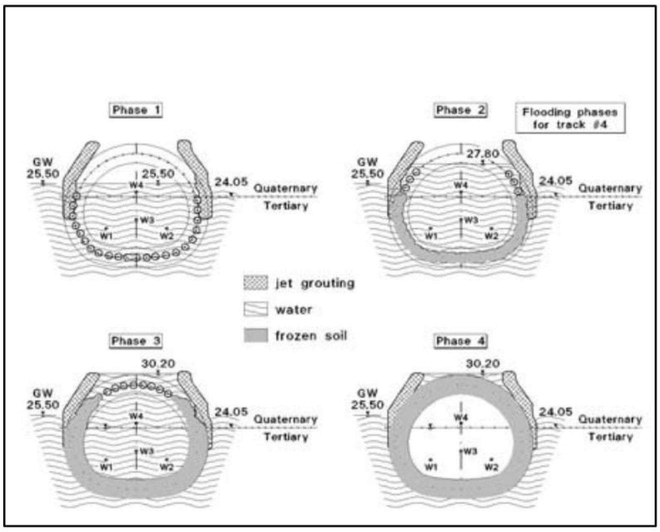


Figure 2.7 - Water injection and freezing phases (after Haß & Schläfer, 2005)

Brine was used as a coolant in this project. Two refrigeration units were used, each with a capacity of 330 [*kW*]. In total 7100 [m^3] soil was frozen for the three tunnels (Haß & Släfer, 2005). The construction site was located in an urban residential area, therefore refrigeration units were placed in an isolated hall. The total settlements measured had a maximum of 13 [*mm*], which is not significant. Part of the settlements which were caused by the tunnel progress was limited to only 0 [*mm*] to 2 [*mm*]. The residual settlements were caused by thawing of the soil. Heave due to the freezing was not identified at the three tunnel locations.

Table 2.4 - Characteristics Düsseldorf Subway section 3.4H

Year of completion	1993
Soil type	Gravel and sand Construction partly above the groundwater table, water was injected
Freeze method	Horizontal brine freezing Brine temperature of -35 °C Two freezing units with capacity of 330kW 7100m^3 of frozen soil
Length and shape	40m Circular frozen body, frozen with the help of grouting
Heave and settlements	Heave has not been indentified Max. 13mm settlement

Boston (U.S.A) Central Artery/Tunnel Project
Underpass of railway tracks with three tunnels[1]

In 1982 the planning of the 'Big Dig', an enormous project in the centre of Boston started. In 1991 the construction works started and in 2007 the project was completed. Since 1959 Boston had a six-lane highway running through the city centre over an overpass. With the 'Big Dig' this highway was brought below ground level in several tunnels of in total 5.6 [km] long.

Part of the project was passing nine railway tracks with three tunnels. The tracks had to remain active during the tunnelling and therefore settlements were allowed. For this part of the tunnel ground freezing was used to stabilize the soil, assure complete groundwater cut-off and assure the tunnel face stability. Circulating brine in hundreds of steel pipes froze the soil in several weeks. Then three tunnel boxes, of which the largest was 107 [m] long, were jacked through the frozen soil. The tunnel boxes were located only 2 [m] below the railway tracks. An extensive laboratory test program on the soil properties of frozen soil was performed for this project (after Jessberger et al, 2003).

Table 2.5 - Characteristics Boston (U.S.A) Central Artery/Tunnel Project

Year of completion	2002
Soil type	Clay
Freeze method	Vertical brine freezing Jacking tunnel boxes of 107m through frozen soil
Length and shape	2m Freezing a large soil body completely
Heave and settlements	Initially the settlements were limited, but in 2011 a giant sinkhole was detected near the tunnel. It is unsure what is the cause of this sinkhole, but thawing clay may by part of it.

[1] *Central Artery/Tunnel Project-The Big Dig.* (2012). Retrieved 2012-03-27, http://www.massdot.state.ma.us/highway/TheBigDig.aspx

Nuremberg-Fürth Subway Section 3
Freezing under historic buildings

The project concerned the extension of the Nuremberg subway with 1.3 [*km*] in the direction of the city of Fürth. Two tunnels were driven close to each other and for part of the tunnel construction brine freezing was used. This part, with a length of 60 [*m*], is located 5 [*m*] beneath the foundation of historic buildings. The subsoil consists of solid keuper sandstone overlaid by quaternary sands. The tubes are partially located in these sands, but mainly in the rising solid keuper, Figure 2.8 and 2.9 (both after Bayer, 2002). The groundwater table is located close to the ground level.

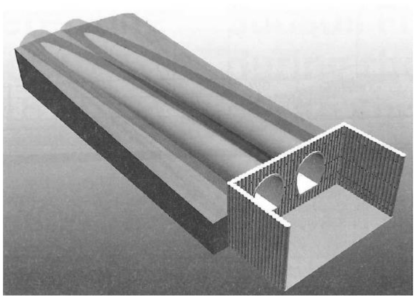


Figure 2.8 - Freezing bars above the tunnel tubes

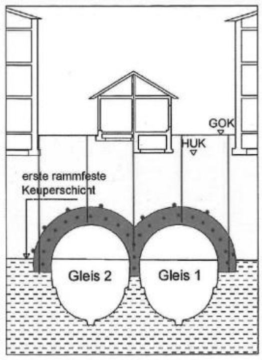


Figure 2.9 - Cross section of the tunnel tubes

Freezing was necessary to ensure no water would infiltrate during construction in the tunnels due to the high water level in the sand. Two watertight “bars” in a vault-like shape were frozen on top of the tunnel tubes and integrated into the solid keuper. The sand has a permeability varying between $2.5x10^{-3}$ [*m/s*] and $2x10^{-4}$, which is fine for using freezing techniques. The ground freezing also had a bearing function. The thickness of the frozen body will be at least 1.5 [*m*] on top and 1.0 [*m*] near the inclusion in the keuper. The route of the tunnel is not straight but in a curve, this meant the freeze pipes also had to be inserted in a curve. For the first time in history freezing in combination with horizontal directional drilling was applied. A requirement for the HDD was that the deviation of the boreholes could be maximum 25 [*cm*] over the total length of 60 [*m*]. In total 23 horizontal boreholes with a maximum length of 56 [*m*] were used to install the freeze pipes. 10 of the 23 drillings were successful on the requirement of maximum deviation.

Freezing was one of two possible solutions; jet grouting and freezing. With jet grouting the risk of creating a leak in the cover that was supposed to be watertight was considered too high. Therefore this technique would always be combined with working under overpressure. Due to this extra measure freezing seemed the best and most cost-effective solution. The volume of the frozen soil was approx. 3000 [m^3]. The energy needed to freeze the soil in total was ca. $6.5*10^5$ [*MJ*] (after Sieler, 2001). The freezing power needed to gain the needed dimensions of the frozen soil body was 355 [*kW*], the required freezing power to keep that body frozen was 175 [*kW*].

Table 2.6 - Characteristics Nuremberg-Fürth Underground Section 3

Year of completion	2004
Soil type	Keuper sandstone and sand with a fine grain share of 5% Groundwater is located slightly below the surface
Freeze method	Horizontal brine freezing 23 freeze pipes installed in a curve in combination with HDD Freezing unit with a capacity of 355kW 3000m^3 of frozen soil
Length and shape	56m Frozen body in a vault-like shape above the tunnel crown
Heave and settlements	max. 20mm heave and a max. 10mm settlements

Munich Subway Station Marienplatz

Enlargement of the subway platforms[2]

In the subway system of Munich in Germany the station Marienplatz is one of the busiest stations. After being at its capacity limit for a long time in 2003 it was decided to enlarge two platforms. A tunnel was made parallel to the current platform and by cross-cuts they were connected to each other. The construction has doubled the surface of the platform.

The station is located in the centre of the city near Munich's city hall. The enlargement of the platforms is located only 10m beneath the historic building of the city hall. Therefore a solution had to be found to construct the platform without causing settlements and with minimal risks. Soil freezing using brine was chosen to be the best solution to this problem. Above each tunnel a pilot heading was excavated of 100 [*m*] long. From these pilot headings boreholes were made for the purpose of freezing. The boreholes had a relatively short length, 3 to 10 [*m*]. Ice caps were frozen and underneath those caps the new tunnels were made using a mining technique with shotcrete. The ice caps act as a seal against the groundwater. Two freezing units of 275 [*kW*] were used. The circulating capacity was 200 [m^3] of brine per hour (Müller, 2005).

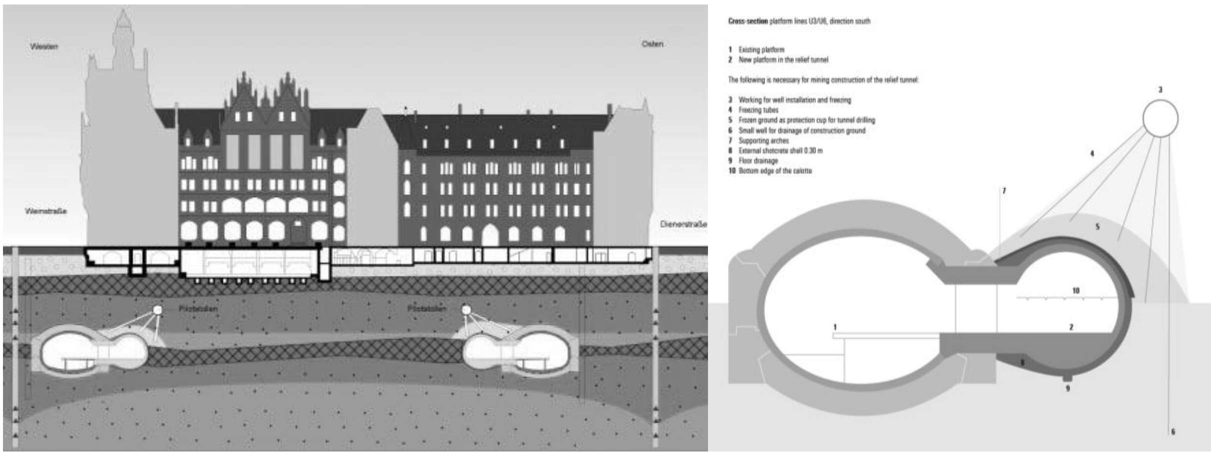


Figure 2.10 - Ground freezing underneath Munich's city hall

The subsoil in Munich consists of a sandy fine and coarse gravel top layer underlaid with clays and silt. In those clays and silt are very thick (7.5 to 12 [*m*]) layers of fine and medium dense sand. As a result the floors of the two tunnels are located in the sand layer. The tunnel roofs are mainly located in silt and clay layers but also partially in the sand. Only water infiltrating by rain was expected, a high water level was absent.

Table 2.7 - Characteristics Munich Subway Station Marienplatz	
Year of completion	2006
Soil type	Clay, silt and sand
Freeze method	Brine freezing Boreholes were drilled from a pilot heading, 350 freezing heads Two freezing units of 275 kW Capacity of 200m³ per hour
Length and shape	3 to 10m Frozen body in a vault-like shape above the tunnel crown
Heave and settlements	Extensive testing was done on beforehand to minimize heave and settlements; heave and settlements were minimal due to oscillating operations 12mm total deformations

[2] *Metro station Marienplatz, platform widening, Munich.* (2005). Retrieved 2012-03-26, http://www.ssf-ing.de/en/projects/tunnels/metro-station-marienplatz-platform-widening-munich.html

Amsterdam North/South Line: Damrak
Freezing canopy at start of tunnelling

At the location of the Damrak the tunnel tubes of the Amsterdam North/South line are located relatively shallow and in very soft silty soils. Extra difficulties are the wooden piles present that are part of a historic masonry quay wall. For the start of the tunnelling at the Damrak, during the launch of the TBMs, freezing is used for double security.

A canopy of frozen subsoil was formed above the tunnel tubes. This way the wooden piles were encapsulated by frozen soil so that they could be cut off by the TBM. The canopy also provided an airtight cover which allowed an entry to the working chamber, under air pressure.

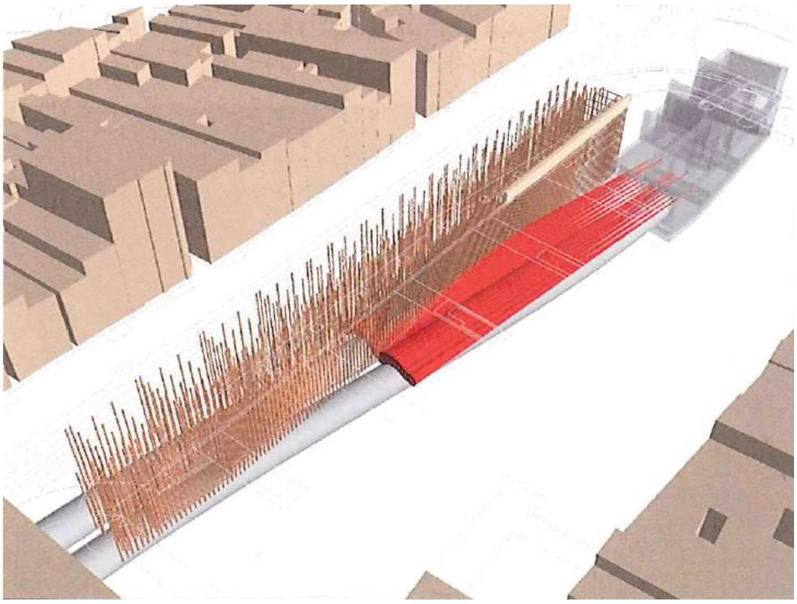


Figure 2.11 - Frozen canopy above the tunnel tubes at the Damrak

The soil was frozen using 32 pieces of 85 [*m*] long directional drilling pipes (Kaalberg, 2011). The freezing canopy was successful in stabilizing the soil and tunnel face during tunnelling. However during and after thawing of the soil larger settlements than expected occurred. As was suspected prior to construction, the quay wall was in poor condition because the wooden pile foundation was floating in a clay layer just above the freeze body. The 1st Sand layer historically used for founding piles was absent at this location. Although surface settlements were small, the pile tips (which were located directly in or above the freeze body) settled some 60 [*mm*], most likely due to inadequate pile tip bearing capacity.

Table 2.8 - Characteristics Amsterdam North/South Line: Damrak

Year of completion	2011
Soil type	Soft silty soils
Freeze method	Horizontal brine freezing 32 freeze pipes installed with directional drilling
Length and shape	85m Frozen body in a vault-like shape above the tunnel crown
Heave and settlements	60mm at ground level

Berlin Subway U5: Museumsinsel

The missing link between Alexanderplatz and Brandenburg Gate

In the end of April 2012 the construction of the extension of subway line U5 in Berlin has started. Between the stations Alexanderplatz and Brandenburger Tor 2.2 [*km*] of connecting subway lines with three new stations will be constructed. The project should be finished in 2019. Part of the project is the construction of station Museumsinsel. This station will be partly constructed at short distance below the Spree Channel and partly underneath the foundation of the Crown Prince Palace. In Berlin the soil consists mainly of sand and gravel deposits, but locally there can also be thick peat layers. This is the case near the Spree Channel.

Two shafts, with a maximum depth of 43 [*m*], will be constructed at the ends of the station to be built using diaphragm walls. In between, over a distance of 105 [*m*], the soil will be frozen and excavated using the NATM. The minimal cover of the frozen soil body up to the channel is 4.50 [*m*]. Freeze pipes from both shafts will be installed with lengths of about 25 [*m*] and 85 [*m*] to cover the 105 [*m*]. Part of the drillings will be performed over the whole length of 105 [*m*] (Erdmann, 2011).

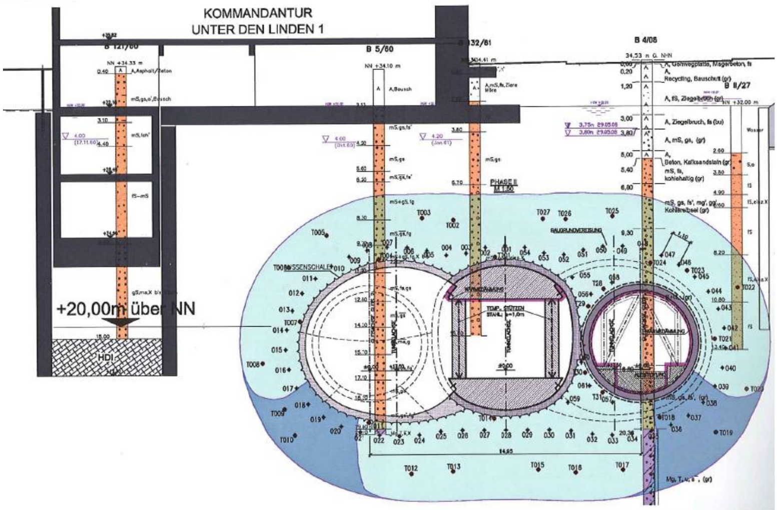


Figure 2.12 - Frozen body at Station Museumsinsel

The TBMs cross the two shafts at the two ends of the station. The cross sections where the TBM passes through the diaphragm walls are reinforced using glass fibre. To ensure a good connection and especially the water tightness of the connection between the tunnel and diaphragm walls a jet-grout body is used. The cross section of the station consists of three parts. One middle part and two side parts in which the tunnels are located.

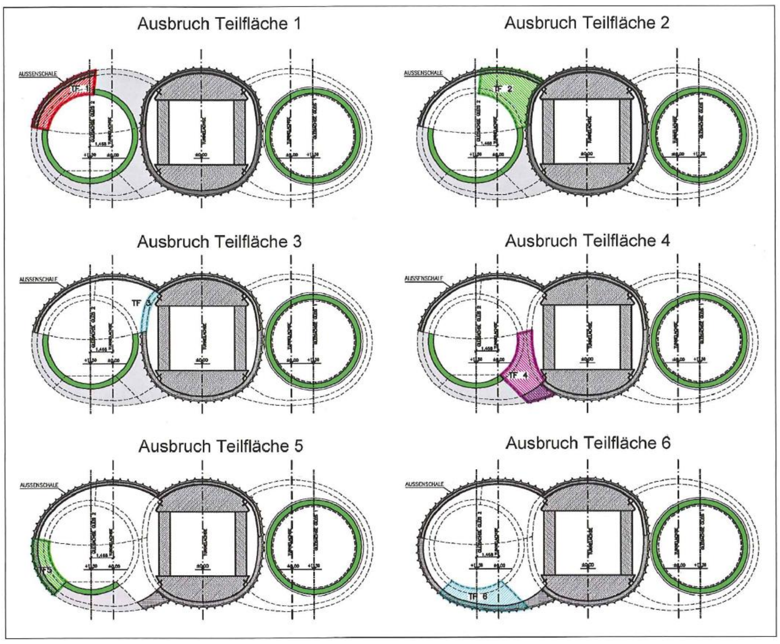


Figure 2.13 - Construction order sections station Museumsinsel

Figure 2.13 shows the construction sequence of the station. First the middle part will be excavated, followed by the side parts in which the tunnels tubes are located. The two side parts will be constructed by removing piece by piece the tunnel lining and excavating the soil with a router. Then shotcrete is applied to ensure the temporary stability. The frozen body surrounding the entire cross section of the station will have a thickness of at least 2 [m].

Table 2.9 - Characteristics Berlin Subway U5: Museumsinsel

Year of completion	2019
Soil type	Sand and gravel, but also peat Groundwater table located 3m below surface
Freeze method	Horizontal brine freezing Frozen body around the entire cross section of the station with a thickness of 2m HDD used for freezing
Length and shape	85m and 25m Frozen body around the entire cross section
Heave and settlements	Construction in progress

The Netherlands Westerschelde Tunnel
Ground freezing the cross passages[3]

The Westerschelde Tunnel consists of two tunnel tubes of 6.6 [*km*] long driven by a TBM. The tubes have an inner diameter of 10.10 [*m*]. Every 250 [*m*] there is a cross passage between the two tubes. In total there are 26 cross passages each with a length of 12 [*m*]. The TBM was still working when the cross passage construction works started, therefore only half of the space in the tunnel could be used as work area and there were very strict safety rules.

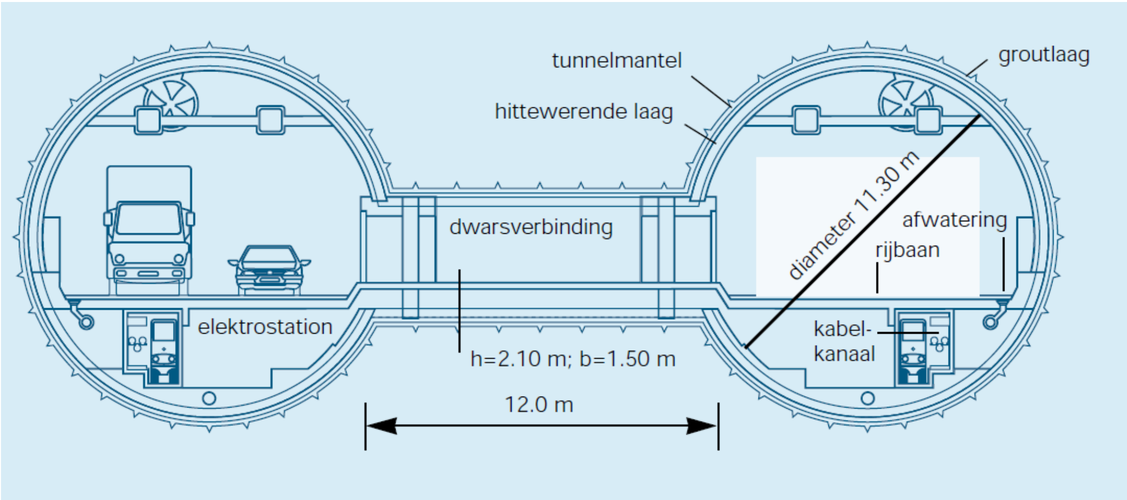


Figure 2.14 - Cross section of the two tunnel tubes and the cross passage in between

The cross passages were made using ground freezing with brine as a coolant. Nitrogen freezing was excluded for safety reasons. The freeze pipes were installed from the East tube towards the West tube. The freezing of a 2 [*m*] thick closed circular soil body takes place, depending on the soil type, in four to six weeks. When measurements indicate the frozen body is thick enough the water inside is drained. When there is no outflow of water anymore the frozen body is closed. 22 freeze pipes were used per cross passage. A maximum of six refrigeration systems was in operation and seven cross connections were constructed simultaneously (Thewes et al, 2001).

Excavation is done step by step and after each excavation a 30 [*cm*] thick shotcrete layer is applied on the walls of the cross passage. The cross passage is excavated from the West tube towards the East tube. Next a watertight sealing is applied and then a 40 [*cm*] thick concrete layer is used the finish the tunnel on the inside.

Table 2.10 - Characteristics The Netherlands Westerschelde Tunnel

Year of completion	2003
Soil type	Mainly Boom clay, but also Westerschelde sand
Freeze method	Brine freezing 2m thick frozen body 4 to 6 weeks to freeze complete body
Length and shape	12m Circular frozen body around the entire cross section
Heave	Both the sand and the clay have a low to negligible frost heave

[3] NV Westerscheldetunnel (2001). *De Westerscheldetunnel: Een megaproject met grensverleggende boortechniek.* Utrecht: Podium.

Bremen Weser Tunnel

Ground freezing the cross passages[4]

For the Weser Tunnel two tunnels are driven with a TBM over a length of 1636 [m]. The diameter of the both tunnel tubes is 11.67 [m]. Four cross passages with a length of 11.30 [m] were made in the Weser Tunnel. Ground freezing using brine is applied to construct these cross passages every 327 [m].

The subsoil is strongly inhomogeneous at the location of the Weser Tunnel. Layers of different sands, clays, silts and also peat alternate each other. This made the driving of the tunnels and also the construction of the cross passages difficult. The risk of failure of the frost block could not entirely be ruled out due to the inhomogeneous soil. To prevent the tunnel from flooding when such an event would occur a bulkhead was installed in the tunnel at the location of the cross passage.

The circular frozen soil body was calculated to have a minimum thickness of 1.75 [m]. Temperature sensors every 800 [m] made it possible to monitor the development of the frozen body closely. Soil studies performed beforehand showed that a soil temperature of -14 [°C] had to be reached to ensure a waterproof and stable construction (Lüesse et al, 2001).

After excavation first a primary 30 [cm] thick shotcrete layer was applied. The final lining of the cross passages consists of an in-situ concrete layer.

Table 2.11 - Characteristics Bremen Weser Tunnel

Year of completion	2003
Soil type	Very inhomogeneous soil, clays, silts, sands and peat
Freeze method	Brine freezing 22 freeze pipes
Length and shape	11.30m Circular frozen body around the entire cross section
Heave and settlements	Unknown

[4] *TBM development at the Weser Tunnel.* (2004). Retrieved 2012-05-02, http://www.convertingtoday.co.uk/story.asp?sectioncode=2&storycode=24608&featurecode=955&c=1

Antwerp Liefkenshoek Rail Link
Ground freezing the cross passages

The cross connections between the two tunnel tubes of the Liefkenshoek Rail Link in Antwerp will be constructed using brine freezing. The tunnels have a diameter of 8.4 [m], the cross connection will have a length of 7.2 [m]. The tunnels are mainly located in sands. A difficulty is the changing tides. The water level in the Schelde varies between +6.5 TAW and -0.1 [m] TAW ('Tweede Algemeene Wateraanpassing'), which means that the pressure on top of the tunnels will vary greatly. Especially for the tunnel face stability this brings some complications; it needs to be adjusted to the prevailing water pressures.

The freezing is done using brine. The freeze pipes are installed in a circular shape around the cross section of the cross passages. To control the growth of the frozen soil body and steering of the freeze pipes at different locations in the soil temperature measuring pipes were installed. These were installed in such a way that information about the length and thickness of the frozen soil body could be derived. The measurements showed that the frozen soil body grows faster on the inside body. In the first 20 days of freezing the body grows the fastest.

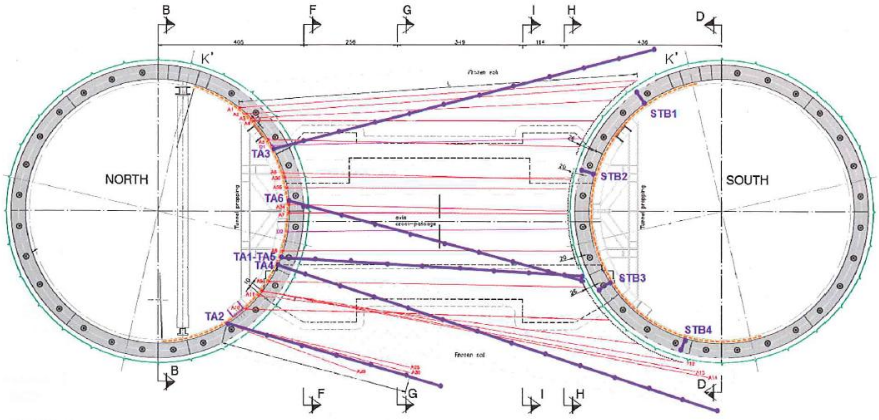


Figure 2.15 - Cross section of the two tunnel tubes with temperature sensors

Also measurements of the pore water pressures were of importance. When pore water pressures start to rise this is a sign the frozen soil body starts to close. Water expands when it is frozen. When the soil body starts to close pore water cannot dissipate to the surrounding soil and excessive pore pressures start to develop. For the cross passages of the Liefkenshoek Rail Link the soil bodies were closed after 6 to 7 weeks, then the pore water pressures stopped developing, see Figure 2.16 (after Boxheimer, 2011).

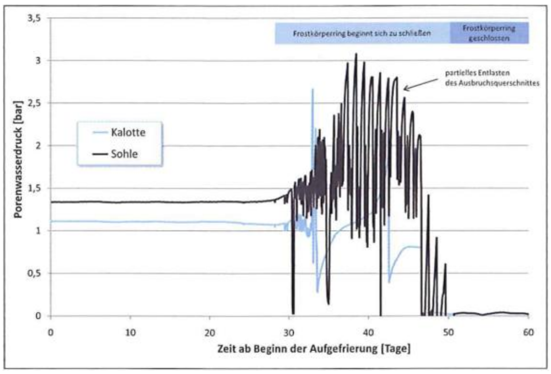


Figure 2.16 - Measurements in the first cross passage: pore pressure development in time

When the size of the designed frozen soil body is reached, part of the tunnel lining at the location of the cross passage is removed. Then the excavation can start. Shotcrete is used to ensure short term stability after which a watertight sealing is built in. Finally a permanent stability system is built in, which consists of in-situ reinforced concrete. The connection of the cross passage to the main tunnel is realized using a closing structure of bentonite-cement.

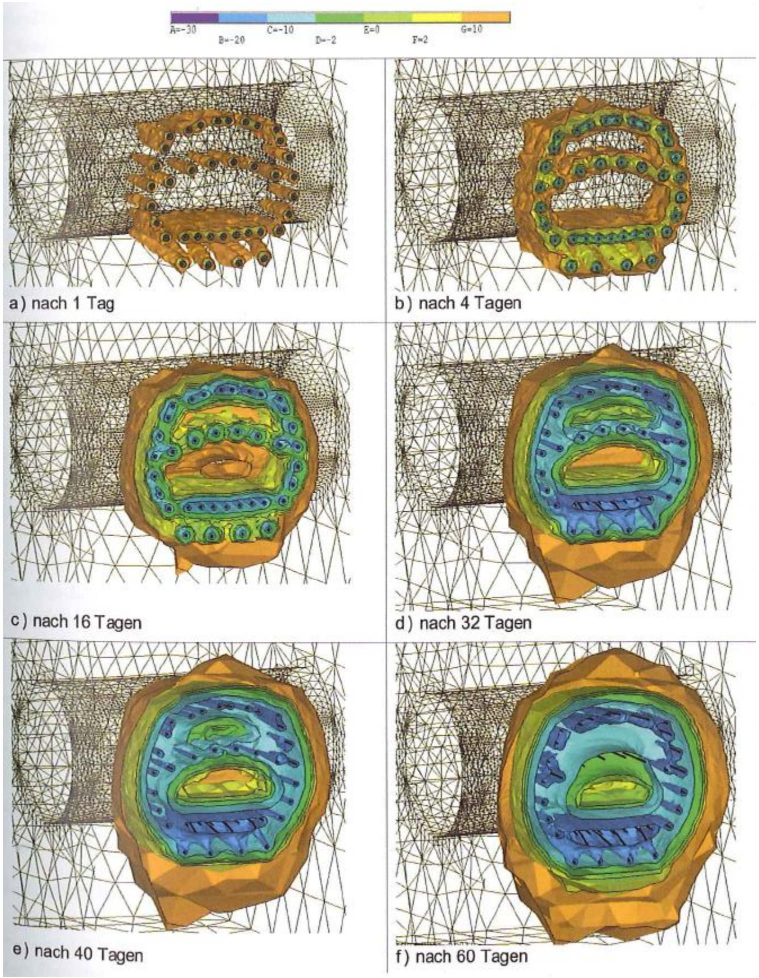


Figure 2.17 - The modelled development of the frozen soil body with ANSYS

The development of the frozen soil body is modelled with program ANSYS, see Figure 2.17. Soil properties are entered for a frozen and an unfrozen parameter. As a starting point of the modelled calculation the temperatures measured before the start of the freezing are used for the area near the tunnel. The temperatures a bit further from the tunnel are estimated. Input for the calculation are the initial temperatures in the soil, the tunnel air temperatures, the temperatures in the freeze pipes and the flow of brine.

Table 2.12 - Characteristics Antwerp Liefkenshoek Rail Link	
Year of completion	2014
Soil type	Sand
Freeze method	Brine freezing Thickness of 1.5 to 1.7 to ensure stability and water tightness Water differences in the Schelde of 7.5m
Length and shape	7.2m Circular frozen body around the entire cross section
Heave and settlements	Unknown

Cologne North-South Urban Railway
Artificial soil freezing to construct required special structures

A 4 [*km*] long urban railway is made in water-bearing quaternary gravel-sand formations. Two single track tunnels with a diameter of 7.30 [*m*] will be made using shield driving (Thon et al, 2001). For special structures, like stations, freezing will be applied. Three of the in total eight stations will be constructed using freezing; Rathaus, Kartäuserhof and Severinstraße. The entrances of the stations will be built with an open pit method. At the level of the railway tracks the connection between the two tracks will be made using artificial ground freezing and a mining method to excavate.

There are two types of stations made by freezing. The first design consists of separate platforms for each of the two tracks, connected by three cross passages. The second design is made for stations where the passenger flow is expected to be larger. There one large platform will be made connecting both tracks. Columns will be used to transfer vertical forces. A cross section and longitudinal section of the second design can be seen in Figure 2.18.

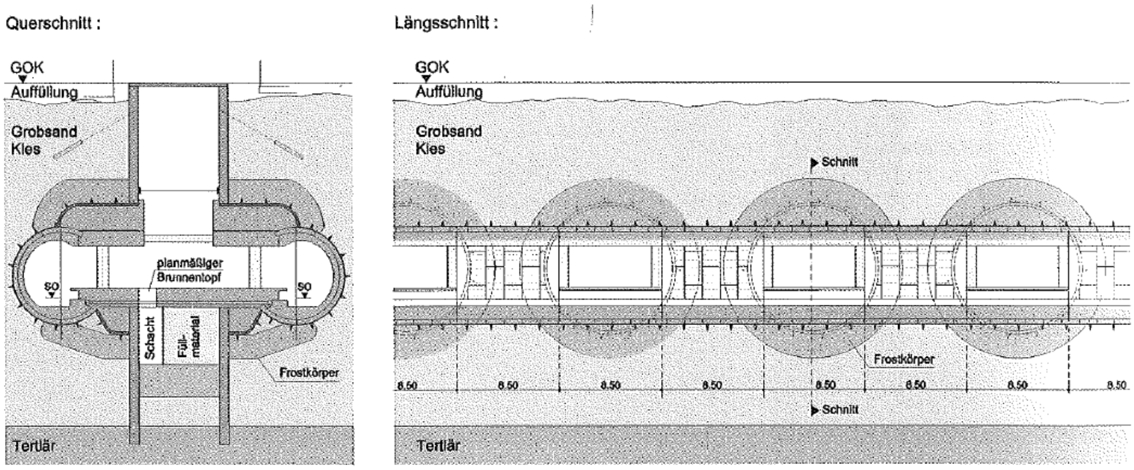


Figure 2.18 - Cross and longitudinal section of the optional station design with one large platform for two tracks

A narrow building pit will be constructed and along the sides the two tunnels will be bored. This will be done closely to the walls of the building pit.

Determining the dimensions of the frozen soil body and its strength, stiffness, stresses and water tightness is done with static and thermal calculations. In German construction regulations it is said that there are four design states to be considered:

1. *Soil is frozen but not yet excavated*
   In this phase the frozen body has only a sealing function. In the subsoil there will be a stress redistribution due to the freezing process and due to drainage which is applied before excavation.
2. *Soil is frozen, partly excavated and partly stabilized with shotcrete*
   In this phase the frozen body has a sealing function and it stabilizes the surrounding soil. The stresses will rise in the frozen body during excavation. An enlarged creep phenomena will occur which leads to a continually changing stress-strain diagram for the soil.
3. *Soil is excavated and shotcrete is applied to the walls shotcrete*
   Shotcrete is not water retaining, but the frozen soil body is still fulfilling this function. The stability of the structure is ensured by applying the shotcrete. Due to ongoing creep and softening processes in the frozen soil body stresses will rise in the shotcrete wall.
4. *Final concrete wall of the tunnel is made*
   The final concrete lining is constructed to withstand full soil and water pressures in the final situation. The freezing can stop as the final lining is applied.

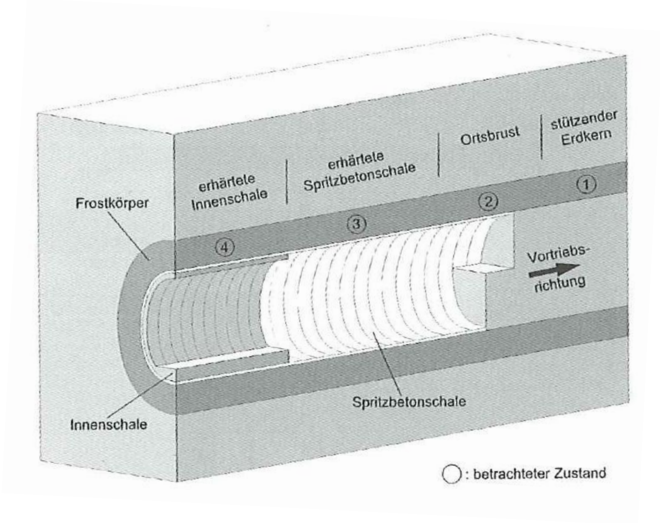


Figure 2.19 - Schematic view of the construction sequence in a freezing project

The normative phase is when the shotcrete is applied and it has to withstand stresses due to creep and softening of the frozen soil body. The equation used to model the development of strains is:

$$\varepsilon = \frac{\sigma}{E_0} + A \cdot \sigma_n \cdot t^c$$

In which E_0 is Elasticity modulus at start, A is a temperature dependant viscosity parameter, n is a softening parameter and c is a parameter which describes the primary creep part. A and c are determined by laboratory tests.

Table 2.13 - Characteristics Cologne North-South Urban Railway

Year of completion	2019
Soil type	Coarse sand Constructions are below groundwater level
Freeze method	Brine freezing Brine temperature of -35 °C 5 freeze units with a total capacity of 2000 kW 99 freeze pipes when 5 cross passages are made
Length and shape	9m Spherical frozen body
Heave and settlements	Construction has not yet started

CHAPTER 3

DESIGNING AN UNDERGROUND STATION

Starting the design phase several sketches were made to determine the most feasible design of the station. The sketches focussed on the lay-out of the construction site as well as the cross section of the station. Each design will be explained shortly in the next subchapters. The designs are not focussed on a specific location yet. At the end of this chapter the most feasible design will be chosen based on a comparison.

Starting points for the designs:

- A rough design will be made for each station having a platform surface of 1130 [m^2]. This is approximately the platform surface of station Vijzelgracht, with a length of 125 [m] and an average width of 9 [m]. The railway tracks have a width of about 3.70 [m]. The bottom of the station is located 28 [m] below surface level.
- Two entrances to the station are required as a minimum (written down in the 'DPvE 96'). These entrances have the same width as the platform. An entrance with no stairs, but only escalators and elevators is acceptable. Maximum angle of an escalator is 35 degrees.
- One of the requirements ('DPvE 96') to which the station must comply is the realisation of a central platform. This is also an important reason why two separate tunnels are being built for the North/South Line, instead of one bigger tunnel in which both tracks are located.
- An overview of the general soil profile assumed for the centre of Amsterdam is given in Table 3.1. Most of the historic buildings are founded in the second sand layer. New foundations are constructed in the third sand layer.
- The main part of the refrigeration plant, with an assumed surface of 120 [m^2], should be located mainly inside the building pit to keep it from causing extra hindrance on ground level. Part of the plant should always be located on ground level, about 30 [m^2].

Table 3.1 - Soil profile centre Amsterdam[5]	
+1.5m to -12m NAP	Holocene layers (clay and peat)
-12m to -14m NAP	1st sand layer
-14m to -17m NAP	Alleröd layers (clay and peat)
-17m to -26m NAP	2nd sand layer
-26m to -41m NAP	Eem clay
-41m to -43m NAP	Interlayer of sand
-43m to -50m NAP	Glacial clay
-50m to at least -100m NAP	3rd Sand layer

Using these starting points for each design several characteristics of the building pit can be determined, for example what is the needed surface of the diaphragm walls and the volume of the frozen soil. The most economic design can be chosen its most suitable location should then be clear.

3.1. Station Designs

Design 1 – One narrow building pit with tunnel tubes bored alongside

The station will be constructed parallel to existing structures. A narrow building pit is made along which two tunnel tubes will be bored closely. The freezing will be done from the building pit in the direction of the tunnel tubes to be able to excavate the soil in between and construct the station. NATM will be used to excavate the soil. Phasing of the station building process will contain in general the following steps:

1. Construction building pit
2. Boring two tunnel tubes
3. Freezing soil body
4. Excavation of soil
5. Construction of station

This site design can have various results in station design. There are two main options for the internal construction of the station and two main options for the global lay-out of the station.

1. General lay-out station
    a. Tunnel tubes located each on one side of the building pit on the same level
        A connection can be made between the building pit and the tunnel tubes to create two separate or one large platform connecting the two tracks.
    b. Two tunnel tubes located above each other on one side of the building pit
        When there are certain conditions, for example a narrow street profile, locating the tubes above each other might be a fit solution. Two separate platforms on different levels will be constructed. A result of placing the tubes above each other will be an increasing depth of the station.

Figure 3.1 - Overview design 1A

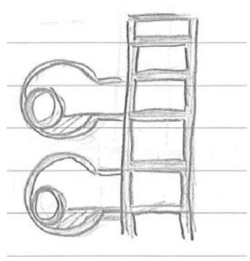


Figure 3.2 - Cross section tunnels above each other design 1B

[5] Adviesbureau Noord/Zuidlijn (2000) Algemene geologie Amsterdam. *Grondonderzoek Noord/Zuidlijn - Parameterset definitief ontwerp*

2. Internal lay-out station
   a. A number of cross passages (i.e. 5) between platforms constructed in the tunnel tubes
   b. Connecting the tunnel tubes over the whole length of the station, creating one large platform serving two subway tracks

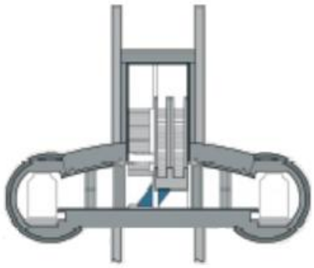


Figure 3.3 - Cross section cross passage (2a) or entire station (2b)

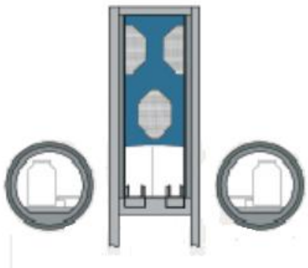


Figure 3.4 - Cross section platform in tunnel tube (2a)

The first option for the internal lay-out of the station is constructing two separate platforms using several cross passages to connect them. This solution is used for station Severinstraße of the North/South Line Cologne. In between the cross passages the platform will only be constructed inside the tunnel tube. Therefore a precondition for the application of this design is a large enough tunnel diameter to be able to construct a platform inside. In Amsterdam the inner tunnel diameter is 5.88 [m], which is not large enough. One can consider building a tunnel with a larger diameter to be able to construct a platform inside, but in between the stations this larger diameter has no function.

The second option is to construct one large platform connecting the two tracks. The diaphragm walls will be demolished over almost the entire length of the station at the level of the tracks. Columns are needed to carry the diaphragm walls and transfer loads. These columns will need a robust character. In Cologne where this design is being executed circular columns with a diameter of about 1.5 [m] are placed every 18 [m] in the length direction and every 5 [m] in the width direction of the platform, see Figure 3.3. The width of the platform in Cologne is 19 [m].

Starting points to define some characteristics of the station are the following:
- The building pit will have a length and width of 130 x 6 [m].
- The tunnel tubes will be bored at a distance of 1m to the building pit wall, which is assumed to have a thickness of 1 [m] for now.
- The diaphragm walls will be installed downto a depth of 30 [m].
- The platform will be constructed 1m into the tunnel tube and will then have a total width of 12[m]
- To define the amount of freezing the characteristic freeze surface is used. The thickness of the frozen body is still unknown therefore the amount of freezing is expressed in a surface instead of a volume.

Table 3.2 - Design 1: Tunnel tubes bored closely to one long narrow building pit

Working space on ground level	780m^2
Length freeze pipes	8m
Volume diaphragm walls	8160m^3
Freeze surface	3640m^2
Height of the station	4m
Continuous boring process	Yes, after construction of the building pit the tunnels can be bored continuously along side.
Support structure	Columns to take over the function of the diaphragm walls and to support the lining of the tunnel wall.

Some notes on this design:

- \+ Length of the freeze pipes will be limited.
- \+ Over the whole length of the station entrances can be made.
- \+ The station does not need to be finished before the tunnels can be bored.
- \+ Boring process is continuous.
- \+ The building pit does not need to cover the entire width of the station.
- 0 The main part of the tunnel lining stays in place.
- \- Columns need to be made to take over the function of the diaphragm walls. These columns will need quite large dimensions and might disturb the spacious character of the station.
- \- Over the whole length of the station hindrance on ground level will occur.

Design 2 - Two building pits through which the tunnels are bored

Two tunnel tubes will be made using a TBM. They will pass through two building pits, which are constructed at each end of the station and still filled with soil during this procedure. The station will be constructed perpendicular to existing structures. This means part of the construction could take place below existing structures or other obstacles on ground level that cannot be removed, like a busy traffic junction. Freeze pipes will be installed from both building pits parallel to the tunnels to create a frozen body around the two tunnels. When the frozen soil body is closed excavation of the station using NATM can start. This method (design 2a) will be used for the construction of station Museumsinsel in Berlin.

The phasing of the construction works consists of the following steps:

1. Construction of two building pits (i.e. caissons)
2. Boring two tunnel tubes, through the building pits
3. Excavating building pits
4. Freezing soil body in between the building pits and around the tunnel tubes
5. Excavation soil and construct middle part station
6. Excavation soil, remove tunnel tubes and construct side parts station

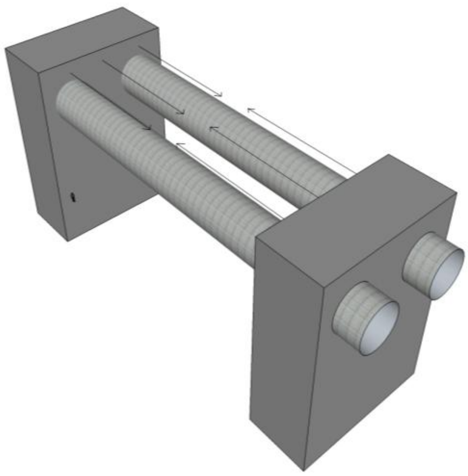


Figure 3.5 Overview of design 2

Instead of boring through the building pits the entire station could be constructed first followed by the boring of the tunnels. A variation (design 2b) on the previous phasing therefore is:

1. Construction two building pits
2. Freezing soil body in between the building pits
3. Excavation soil and construct middle part station
4. Excavation soil and construct side parts station
5. Boring two tunnel tubes, transport TBM through the station

The second phasing leads to a non-continuous boring process and tunnel sections instead of one tunnel. The station needs to be finished before the TBM can be transported through it. A stationary TBM is economically very unattractive, which is the main disadvantage of this phasing. The difficult connection between tunnel tube and building pit is however removed in this design.

Entrances to the station can be made in the two building pits. One downward movement for users is however not possible due to building pits being perpendicular to the tunnel tubes and their relatively restricted width in which also the tunnels are present.

The freezing direction in this design is from the building shafts parallel to the tunnel tubes towards the other building pit. The soil will not only be frozen in between but also around the tunnel tubes. In the final phase the lining segments of the tunnels are removed in between the building pits. The final lining of the station will consist of shotcrete and in-situ concrete in a more spacious cross section. A sketch of the cross section is shown in Figure 3.6. The station will be excavated in roughly three phases, first the middle part and followed by the two side parts which include the tunnel tubes. This figure shows the phase in which only the middle part is finished yet. The next phase is to excavate a side part and remove the tunnel tube part by part simultaneously. The dotted line shows the planned side walls of the station.

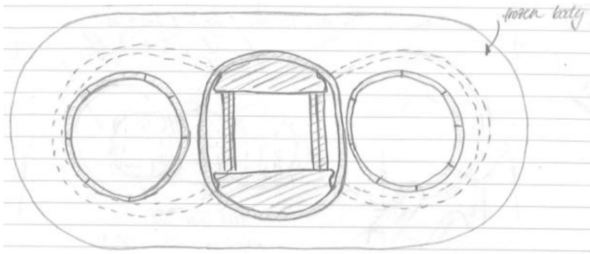


Figure 3.6 - Sketch of station design 2: Cross section in which the middle part is constructed

Starting points to define some characteristics of station design 2 are the following:

- The building pit will have a length and width of 10 x 28 [m]. A width of 28 [m] is enough to cover two times the outer diameter of the tunnels, a free space of 5 [m] in between them and 5 [m] on each side to freeze the soil body and extend the width of the station.
- The diaphragm walls will be installed downto a depth of 30 [m].
- The platform will have a width of 9 [m] and a length of 125 [m].
- The freezing will be done from two sides towards each other.
- To define the amount of freezing the characteristic freeze surface is used. The thickness of the frozen body is still unknown therefore the amount of freezing is expressed in a surface instead of a volume. The width of the frozen soil body will be 22 [m].

Table 3.3	**Design 2A - Two building pits through which the tunnels are bored**	**Design 2B - Two building pits up to which the tunnels are bored**
Working space on ground level	560m^2	560m^2
Length freeze pipes	Max. 65m	Max. 65m
Volume diaphragm walls	4560m^3	4560m^3
Freeze surface	7200m^2	7200m^2
Height of the station	4.5m	4.5m
Continuous boring process	Yes, after construction of the building pits/caissons the tunnels can be bored continuously through them.	No, the station needs to be finished before the TBM can pass it. The TBM will be transported through the building pit.
Support structure	The station can be thought of as a two- or three arch structure. A two arch structure uses one row of columns in the middle of the platform. Two rows of columns will support the lining of the tunnel wall with a three arch structure.	

Some notes on design 2A:

+ With the application of this design the station can be constructed without opening up all surface at ground level. Also construction below existing structures is possible.
+ The station does not need to be finished before the tunnels can be bored.
+ Boring process is continuous.

0 Entrances to the station need to be constructed at the beginning and end of the station.

0 The tunnel lining will be removed in the final situation.
- The length of the freeze pipes is quite long.
- Creating a watertight connection between building pits and tunnel tube leads to difficulties.
- Entering the station must be done in two 'actions'.

Most notes on design 2B are similar as on 2a, but the following three notes differ greatly:

+ No difficult connection is needed between the building pits and tunnels as a result of the TBM's not boring through them.
- The station needs to be finished before the tunnels can be bored on both sides.
- The boring process is not continuous and as a result tunnel sections will be made. The TBM needs to be transported through the station, therefore a standstill is inevitable.

Design 3 - Two building pits with tunnel tubes bored alongside

Design 3 is a combination of the first two designs. Two building pits will be constructed at the ends of the station. The tunnel tubes are bored closely along these pits, not through them. The freeze pipes will be installed from the two building pits towards each other and towards the tunnels.

Phasing

1. Construction two building pits (i.e. caissons)
2. Boring two tunnel tubes closely to the building pits
3. Freezing soil body in between the building pits and tunnel tubes
4. Excavation of soil
5. Construction of station

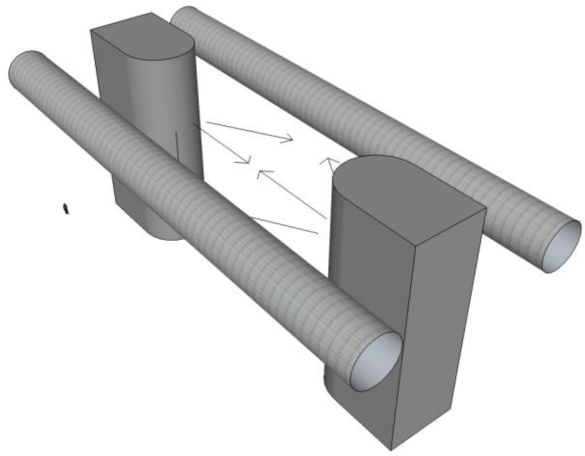


Figure 3.7 – Overview of station design 3

A problem to overcome is the difficulty to freeze from the building pits, which are located at some distance from the tunnels, towards the tunnels and create a closed frozen body. The freezing will be done from two circular building pits, possibly caissons, towards each other. The building pits are located between the tunnel tubes therefore freezing around the tubes is not possible. The main part of the tunnel lining segments will also be used in the final cross section of the station. Possible cross sections are shown in Figure 3.8 and 3.9. Excavation of these cross sections will be done in phases. Shotcrete will be used to ensure temporary stability of all parts.

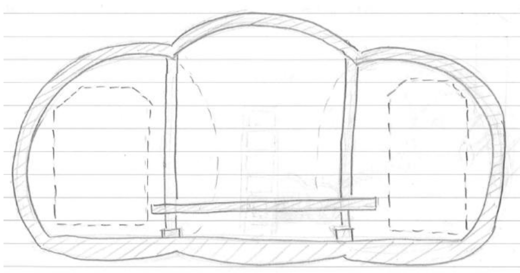


Figure 3.8 - Cross section with a three arch structure

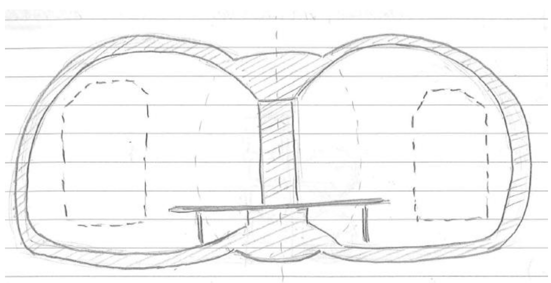


Figure 3.9 - Cross section with a two arch structure

The width of the building pits will be 6 [m], as in the first design. Its length will be 35m to ensure enough space inside and to be able to construct entrances in the pits.
Other starting points to define some characteristics of station design 3 are the following:

- The platform will have a width of 12 [m] and a length of 125 [m].
- The building pit will have a length and width of 35 [m] x 6 [m].
- The diaphragm walls will be installed until a depth of 30 [m].
- The tunnels will be bored at a distance of 1m from the building pit.
- The width of the frozen body will be 16 [m].
- To define the amount of freezing the characteristic freeze surface is used. The thickness of the frozen body is still unknown therefore the amount of freezing is expressed in a surface instead of a volume.

Table 3.4 - Design 3: Two building pits through which the tunnels are bored

Working space on ground level	420m^2
Length freeze pipes	Max. 65m
Volume diaphragm walls	4920m^3
Freeze surface	5280m^2
Height of the station	4.5m
Continuous boring process	Yes, after construction of the building pit the tunnels can be bored continuously alongside.
Support structure	The station can be thought of as a two- or three arch structure. A two arch structure uses one row of columns in the middle of the platform. Two rows of columns will support the lining of the tunnel wall with a three arch structure.

Some notes on this design:

+ With this design the station can be constructed below existing structures.
+ No building pit is needed that covers the whole width and length of the station.
+ The station does not need to be finished before the tunnels can be bored.
+ Boring process is continuous.

0 Part of the tunnel lining will be used in the final design of the station.
0 Entrances to the station need to be constructed at the beginning and end of the station.
0 Part of the building pit wall will be to make a curve. A hexagonal shape using diaphragm walls can be constructed.

- The length of the freeze pipes is quite long.
- Creating a closed frozen body might lead to some difficulties.
- Entrances need to be constructed in the two building pits. The pits therefore need to be of a certain length, which leads to quite large building pits in the longitudinal direction.

Design 4 - One building pit with freezing towards the tunnel tubes

A building pit will be constructed located in the middle of the station to be built. The tunnel tubes will be bored through the building pit. From the pit in parallel direction to the tubes the freeze pipes will be drilled and a frozen body will be created. The drilling length of the freeze pipes does not increase with this solution and only one building pit is required.

Phasing

1. Construction building pit
2. Freezing soil body towards tunnel tubes and freezing from the tunnel tubes
3. Excavation of soil
4. Construction of station

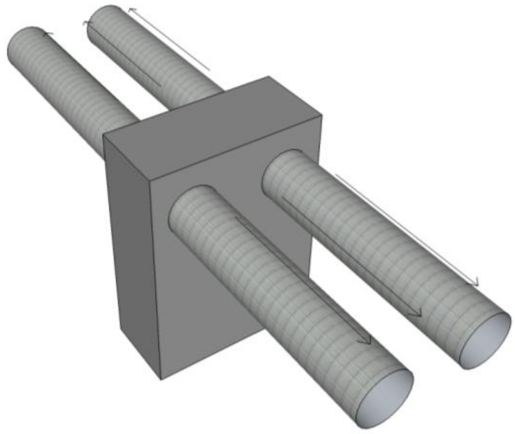


Figure 3.10 - Sketch of station design 4: Overview

A problem is the vertical sealing when freezing is not executed towards a building pit or other form of boundary. Freezing could be done from the tunnel tubes to create this sealing or a vertical freeze wall could be made.

Entrances at two different locations are a requirement for all stations of the North/South Line. Though this design saves a building pit, and thus costs and hindrance on ground level, a second pit still has to be made to construct a second entrance. Together with the problem of vertical sealing at the ends this contradicts the positive points of this design. Design 4 therefore is not considered further.

3.2. Selection of the Most Feasible Design

In the previous subchapters for each design some rough design characteristics are shown in a table. In Table 3.5 these characteristics are put together to make comparison. Making a comparison is difficult due to the "mouldability" of the characteristics, though some conclusions can be drawn on this table.

Table 3.5	**Designs**			
	1 *One narrow building pit with tunnel tubes bored alongside (Cologne)*	**2a** *Two building pits through which the tunnels are bored (Berlin)*	**2b** *Two building pits up to which the tunnels are bored (not applied yet)*	**3** *Two building pits with tunnel tubes bored alongside (not applied yet)*
Volume diaphragm walls [m³]	8160	4560	4560	4920
Surface frozen soil [m²]	3640	7200	7200	5280
Drilling length freeze pipes [m]	8	65	65	65
Work surface ground level [m²]	1040	560	560	420
Distance adjacent buildings	Large	Small	Small	Large
Height station [m]	4.0	4.5	4.5	4.0
Continuous boring process	Yes, after construction of the building pit the tunnels can be bored continuously along-side.	Yes, after construction of the building pits the tunnels can be bored continuously through them.	No, the station needs to be finished before the TBM can pass it. The TBM will be transported through the station.	Yes, after construction of the building pit the tunnels can be bored continuously along-side.
Support structure station	Columns to take over the function of the diaphragm walls and to support the lining of the tunnel wall.	This station can be thought of as a three arch structure with two rows of columns or as a two arch station with one row of columns.	This station can be thought of as a three arch structure with two rows of columns or as a two arch station with one row of columns.	This station can be thought of as a three arch structure with two rows of columns or as a two arch station with one row of columns.

Overall design 3 seems the most economic design based on this table. Though the choice is made to proceed with design 2a, for the following reasons:

→ For design 1a large working space on ground level is needed. This design seems to undo one of the main advantages of using the freezing technique, less hindrance on ground level. Further columns will need to take over the function of the diaphragm walls, which will disturb the desirable spacious character of the station.

→ A continuous boring process will save a lot of costs in relation to a non-continuous process where the TBM stands still for some time. Design 2b, with a non-continuous boring process is therefore less advantageous when compared to the other three designs.

→ In required quantities designs 2a and 3 do not differ a lot, design 3 is slightly favorable on most characteristics. Less frozen soil is needed due to freezing not around the tunnel tubes (as with design 2a) but freezing up to the tunnel tubes. Also less surface on ground level is needed. However creating a closed frozen body will be difficult for design 3. The building pit does not cover the entire width of the station; therefore freezing under an angle is needed.

→ A difficulty for design 2a is a watertight connection between the bored tunnel and the building pits as the tunnel will be bored through the diaphragm walls.

Creating a closed frozen body in design 3 is thought of to be a more difficult task than creating a watertight connection between the tunnel tube and building pit in design 2a. Freezing the soil body for design 2a is considered not to cause any large difficulties. Design 2a is therefore chosen to elaborate further in the next chapters.

The removal of the lining in design 2a has to be elaborated a bit further. In the design the tunnel will be bored also at the location of the future station. Lining of the tunnel will be removed during construction of the station, therefore two tunnel tubes are bored over 120 [m] with no purpose in the final design. Boring the tunnel costs time and money; do the advantages of a continuous boring process outweigh these aspects?

In the current design for the North South Line tunnel sections are bored, between the stations, and the tunnel boring machine is being transported through the station with the use of a Shield Transfer System (STS). The transportation of the TBM with the STS takes about five weeks. When drilling the TBM has a speed of about 10 [m/day]. Constructing the tunnel over the length of the station, 120 [m], would take 12 days.

The construction of the station has to be proceeded that far that STS can be used and transportation of the TBM trough the station is possible. If there are delays in the construction of the station and the TBM had already arrived at the station this will lead to delays in the drilling process. Assuming there will be no delays the costs for the two options are given in Table 4.6 and 4.7.

Table 3.6 - Shield Transfer System

Standstill TBM	5 weeks x 2	€	2.625.000,00
Development STS	33%	€	1.000.000,00
Total		€	3.625.000,00

Table 3.7- Continuous boring process with removal of lining

Drilling 120m x 2	12 days	€	1800.000,00
Reinforced concrete segments	500 €/m^3	€	750.000,00
Total		€	2.550.000,00

Notes on the cost estimation:

- The cost of a working TBM are estimated at €75.000 per day and for a TBM which stands still 50% of those costs is assumed, €37.500. With this estimation the standstill and drilling costs are determined for two bored tunnels.
- The Shield Transfer System is a newly developed technique, especially developed for the North South Line. Costs of development and construction are approximated at three million euros. The system is used for transferring the TBM through the station and creating a watertight connection between tunnel and station at three locations; Rokin, Vijzelgracht and Ceintuurbaan. Therefore 33% of the costs are charged in the above overview.
- The tunnel lining segments cannot be reused after removal; the costs of manufacturing are therefore also in the cost overview. The costs per cubic meter of lining are set at €500. With a cross section of about 6.25 [m^2] and a length of 120 [m] the volume becomes 750 [m^3] for one tunnel tube and 1500 [m^3] for two tunnel tubes.

Although it is a very rough estimation it can be concluded that a continuous boring process will shorten the planning and will lead to a reduction of costs.

3.3. Location to Implement the Alternative Design

When finished the North/South Line in Amsterdam will consist of eight stations.

- Central Station
  The length of the underpass to be made at this location is at least 130 [m]. Besides that there is an important existing building on top of the constructions works, the railway station Amsterdam Central Station. Its foundations will hinder the works and very little settlements will be allowed. Overall Central Station is thought of as a too complicated location to make an alternative design for.
- Rokin
  Above station Rokin a parking garage has been realized. However this garage was not part of the requirements for the station. The possibility of integrating a parking garage in the station design occurred during the design phase. The garage is not a necessity in the design. This makes Rokin also a suitable location for the chosen design.
- Vijzelgracht
  As with Rokin Vijzelgracht is a suitable location for an alternative design using freezing techniques. The station is located in a densely built up area, but with enough space in between the buildings adjacent to the street.
- Ceintuurbaan
  Due to the extremely narrow street profile at the location of station Ceintuurbaan this possibility is no option. Only 12 meters is available between the adjacent buildings. The chosen design cannot be realized at this location, a width of 22 [m] is needed.
- Europaplein
  At the location of station Europaplein is it possible to construct in an open building pit. The surroundings are not as densely built up as in the city centre so the need to keep the hindrance at ground level is less present. Therefore an open building pit is an optimum solution. Making an alternative design here using freezing techniques is not a logical option.

Stations Noorderpark and Noord are being built above ground level. This means Noord, Noorderpark, Central Station, Ceintuurbaan, Europaplein and Zuid are not an option as a location for the alternative design. Rokin and Vijzelgracht remain options.

Neighbours and surrounding buildings had and still have to endure a lot of hindrance as a result of the construction works at both locations. Works at ground level took more time and space than planned and still a large part of the road is unavailable due to these works. This hindrance is a reason to search for an alternative construction method in the future.

Figure 3.11 - Tracé North/South Line

At the location of Vijzelgracht the depth of the station needs to be at least NAP -26 [m]. Research has been done on the tunnel face stability. If the tunnel is bored less deep the face stability is at risk. At depth of NAP -26 [m] the station is located in the Eem Clay. The floor of station Rokin is at NAP -21.5 [m]. With this depth the station would mainly be constructed in the Alleröd or sand layers. Due to the known difficulties with freezing in clay (addressed in the next chapter) the choice is made to locate the alternative design at Rokin.

CHAPTER 4

OBSTACLES WHEN USING GROUND FREEZING

The use of artificial ground freezing in construction works has the main advantage of ensuring a stable excavation surface and watertight seal. High strength and low permeability are the most important properties of frozen soils. Disadvantages might occur however, especially when clayey soils are frozen. In this chapter the obstacles when using ground freezing are more thoroughly studied.

In general the strength and deformation parameters of soil are time- and temperature dependent. The strength of a soil, frozen and unfrozen, depends on the interparticle friction, particle interlocking and cohesion. Of main influence on the strength properties of frozen soils are (Jessberger et al, 2003):

- Time
  Frozen soil has a non-linear stress-strain relationship. The creep process causes a time-dependent compressive strength.
- Temperature
  Ice strengthens the linkage between particles. Due to a change in temperature ice can converse into water or the other way around. This greatly changes the linkage among particles, thus influences the soils mechanical properties.
- Salinity
  The freezing point of water is lowered by the addition of salt. It is one of the reasons why brine is used as a coolant. Saline pore water however is not desirable. Lowering the freezing point influences the strength properties of frozen soil. The higher the salinity the lower the compressive strength at the same temperature.
- Water content and degree of saturation
  The ice content and thereby strength of frozen soil is determined by the water content and temperature. An increasing water content in combination with a decreasing temperature leads to an increasing ice content. Above the ground water table the strength properties of frozen soil depend on the degree of saturation. A lower degree of saturation gives a lower compressive strength.
- Soil type
  The uniaxial compressive strength of frozen soil decreases with increasing fines content.

4.1. Soil Deformations due to Freezing and Thawing of the Soil

A frozen soil can heave, is susceptible to creep and its structure can change due to a freeze-thaw process. These aspects of freezing and thawing of soil are explained in this paragraph.

4.1.1. Frost Heave due to Freezing

When pore water freezes the pore water volume will expand with 9 [%] due to the phase change. A bigger contribution to the expansion however is the freezing of transported water, shown by Black and Hardenberg (1991). During the ground freezing process a water movement occurs towards the cooler areas. In frost susceptible soils pore water flows through unfrozen water films into the frozen zone. This is called cryosuction. The formation of ice lenses causes a volume increase in the direction of the temperature gradient. If movement in the upward direction is not possible, for example due to the foundation of a structure, a heaving pressure will develop. The application of surcharge reduces heave.

Three conditions are required for frost heave, namely a supply of water, soil temperatures sufficiently low to cause some of the soil water to freeze and a frost susceptible soil. Frost susceptible soil is soil that will encounter significant ice segregation when the essential moisture and freezing conditions are present (Beskow, 1935). In coarse grained materials with a low frost-susceptibility usually no significant heave will occur. Excess pore pressures will not develop as a result of drainage to unfrozen areas conditions of the material. In clayey soils heave occurs.

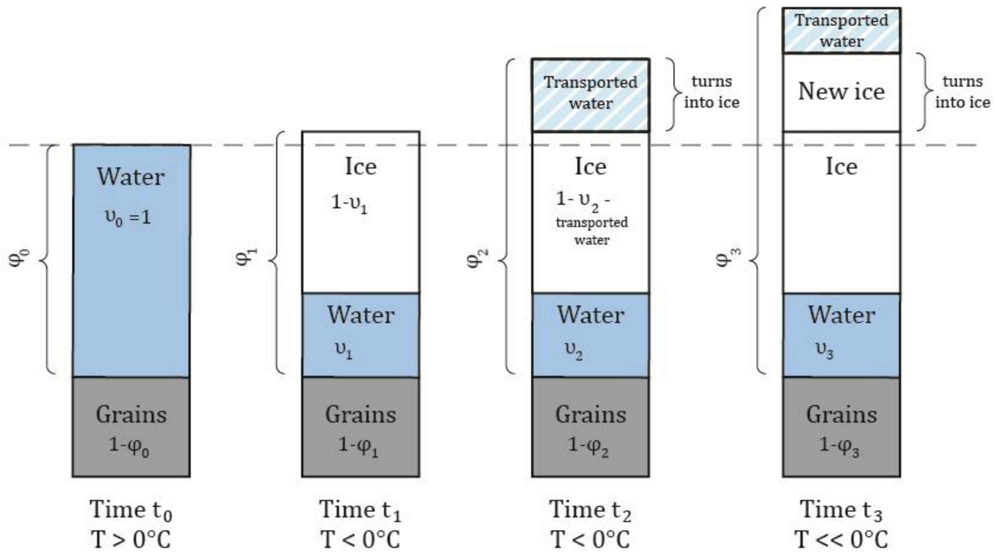


Figure 4.1 – Schematic view of phases during freezing in low permeable soils

For frost susceptible soils a schematic view of phases during freezing and the occurring frost heave is shown in Figure 4.1. In frozen soil not all pore water will be frozen directly if a temperature below zero is reached. The amount of unfrozen pore water in a soil depends on the type of soil (de Lange, 2001). For sands the amount of unfrozen water is around 5 [%] at a temperature of -1 [°C] and 2 [%] at -5 [°C]. For clays a larger amount of water stays unfrozen; 9 [%] at -1 [°C] and 5 [%] at -5 [°C].

With Figure 4.2 the frost susceptibility of soils can be determined. Frost susceptibility is one of the important conditions to have frost heave. Other necessary conditions are the availability of water and a low enough heat extraction rate to allow a water movement towards the frost front. Thus the main influences on the magnitude of heave are (1) soil characteristics, (2) availability of water, (3) heat extraction rate and (4) overburden pressure (Jessberger et al, 2003).

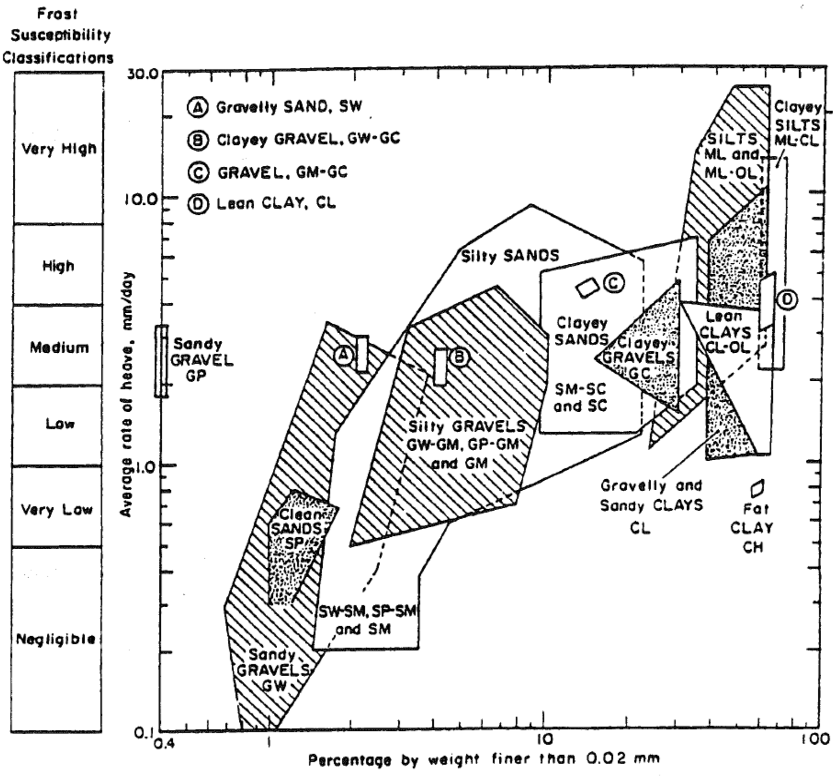


Figure 4.2 – Frost susceptibility of soils based on soil type and particle size (Landanyi 2006)

4.1.2. Effects of Thawing Process

After completion of construction the refrigeration plant will be turned off. The frozen soil will be subjected to an increase in temperature, which will start the thawing process. A phase change of ice into water and, when the soil has heaved, a flow of excess water out of the soil will lead to a volume change of the thawing soil. As a result of the thawing process settlements occur.

When thawing occurs at slow rate pore water has a chance to flow from the soil with approximately the same velocity as melting occurs. No excess pore pressures will develop. For faster thawing rates however, in fine grained soils with low permeability excess pore pressures will be generated (due to undrained conditions) resulting in a shear strength reduction. This strength reduction could lead to stability problems in the thawing soil. The soil will regain its strength as it consolidates.

Morgenstern and Nixon (1971) described a solution thaw consolidation in fine grained soils where the drainage of excess water is impeded by a low permeability. When soil consolidates under its own weight, the equation is:

$$\frac{u}{\gamma' X} = \frac{1}{1 + \frac{1}{2R^2}}$$

With R being the thaw consolidation ratio:

$$R = \frac{\alpha}{2\sqrt{c_v}}$$

In which:

u	= excess pore pressures	[kN/m^2]
γ'	= own weight of soil	[kN/m^3]
X	= distance over which the thaw plane descends	[m]
R	= thaw consolidation ratio	[-]
α	= constant, depending on the rate of heat extraction	
c_v	= coefficient of consolidation	[mm^2/s]

This solution describes for one-dimensional thaw consolidation the descending thaw plane relative to the surface when a frozen soil is subjected to a positive temperature at the upper surface. The solution is not directly useful for an artificial circular frozen body, where drainage may not be one-dimensional. Horizontal drainage may occur, defined by the coefficient of consolidation c_h. c_h is normally greater than c_v. Though at the inside of the frozen body thawing will occur under undrained conditions, since the lining of the tunnel at one side and the frozen body at the other side prevent drainage initially.

4.1.3. Creep of Frozen Soils

The tendency of a material to keep developing deformations, with a constant load is called creep. With soil this means deformations will occur even after the pore pressures have been reduced to zero. The compression of the soil will continue at a very slow rate. Frozen soils are more susceptible to creep and relaxation effects than unfrozen soils. As a result of compression stresses between the grains and the ice inside the frozen body, ice will melt. The melted ice will flow to areas with a lower prevailing stress where it will start to freeze again. According to Assur (1963) creep of a frozen soil is defined by temperature, time, load and material characteristics of soil.

In Figure 4.3 a typical strain/strain rate - time diagram is shown for constant stress and isothermal conditions. ε is the strain and ε with a dot above is the strain rate. Three phases can be distinguished in the figure:

1. Phase 1 - strain hardening. This phase starts at an instantaneous strain ε_0 and is characterised by a strengthening factor and a continuously decreasing strain rate.
2. Phase 2 - linear. This phase has an approximately constant strain rate and is also called the steady-state creep phase.
3. Phase 3 - strain softening. In phase 3 the strain rate starts to increase which leads eventually to failure.

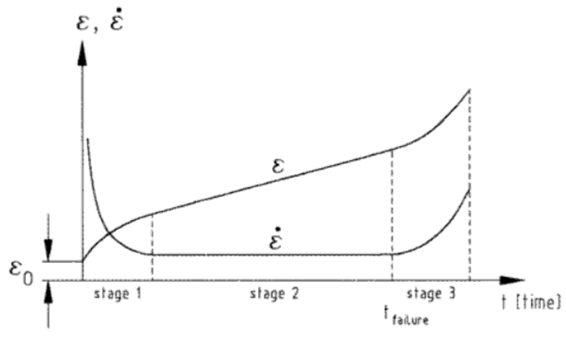


Figure 4.3 - Typical creep curve of a frozen soil (after Jessberger et al, 2003)

In frozen soils it is observed that phase 2 is often absent. It is replaced by an inflection point, which corresponds to a minimum strain rate. When designing the inflection point is seen as 'failure'. With uniaxial creep tests creep parameters *A*,*B* & *C* can be determined to describe uniaxial creep behaviour. The equation of Klein (1979) describes the behaviour as follows:

$$\varepsilon_1 = \sigma_1/E_0 + A^* \sigma_1^{B} {}^* t^C$$

In this equation creep is described subjected to constant temperatures. Creep in (artificially) frozen soils is however a process during which the temperature varies. In the equation of Klein creep parameter *A* can be replaced by $A_{(T)}$, a temperature-dependent creep parameter:

$$A_{(T)} = \tilde{A} \exp\left(-\left(C^*\Delta H\right)/\left(R^*T\right)\right)$$

In which:

σ_1	= constant axial stress	[*N/m*^{2}]
t	= time	[*s*]
A,B, C, $\tilde{A}$	= creep test parameters	
ΔH	= thermal activation energy	[*J*]
R	= gas constant	[*J K*$^{-1}$ *mol*$^{-1}$]
T	= temperature	[*K*]

In Table 4.1 for some frozen soils the creep parameters are given (after Jessberger, 1987). These values shown are not the parameters for the Isotachen model but the parameters A, B and C for Klein ‘s formula.

Table 4.1	**T [°C]**	**A [mPa^{-B} x h^{-C}]**	**B [-]**	**C [-]**
Ottawa sand	-9.4	3.50×10^{-4}	1.28	0.44
Manchester fine sand	-9.4	1.90×10^{-4}	2.63	0.63
Clayey fine sand	-10	8.20×10^{-3}	2.25	0.24
Sand	-10	1.90×10^{-3}	2.80	0.42
Bat-Baioss clay	-10	1.60×10^{-3}	2.50	0.45
Callovian sandy loam	-10	5.50×10^{-4}	3.70	0.37
Emscher marl	-10	7.60×10^{-5}	4.00	0.10
Silt	-10	7.90×10^{-6}	5.60	0.88
Silty clay	-10	5.99×10^{-3}	2.63	0.38
Oil sand	-10	1.18×10^{-2}	1.60	0.44

4.1.4. Changing Soil Properties due to Freeze-Thaw Cycle

The freeze-thaw cycle changes the structure of a soil. Physical and mechanical soil properties are changed due to a change in structure. The structure of a soil is defined as the linkage and arrangement of particles. According to Jilin et al. (2000) the difference in mechanical properties of soils has internal and external factors. Internal factors, such as water content, particle and pore characteristics, lead to an initial structure of the soil. This initial structure is being influenced and changed to a secondary by external factors, of which the main one is load (temperature and time are also regarded as load).

Clay soils are built up out of peds. During the freezing process ice lenses are formed and they will compress the peds and break some of the bonds between them. Eventually new peds will be formed. However sometimes in clays, especially sensitive clays, bonds are broken irreversibly. The greatest structural changes due to the freeze-thaw cycle therefore occur in clays.

An increase of the permeability of clays is a result of the fissuring which occurs due to freezing and thawing. The size of the increase of permeability depends on the liquid limit of the soil, the void ratio and the initial permeability, shown by Nagasawa and Umeda (1985). The coefficient of consolidation depends on the permeability. c_v will increase when the permeability increases, thus c_v is directly influenced by the freeze thaw cycle.

In coarse grained materials, such as sands, usually no structural changes occur. The soil properties of sands are seen as unchanged due to the freeze-thaw cycle. In practise sometimes frozen soil is even sampled in the field to gain an 'undisturbed' sample. In the laboratory it is thawed and then tested.

4.1.5. Failure Behaviour of Frozen Soils

This yielding and failure behaviour of frozen soils is different from most other materials. The type of failure depends on soil type, temperature, strain rate and confining pressure and can vary from brittle to plastic.

Under a hydrostatic pressure frozen soil will first weaken and then start to melt, which is the cause of the creep process explained in a previous chapter. Stress sharing within the frozen soil body is governed by mechanical effects and pressure melting phenomena are governed by thermodynamic effects. Shear stresses will cause other failure behaviour then compression; low strain rates will result in ductile yielding and the higher the strain rate gets the more brittle the yielding becomes. The shear behaviour of frozen soils is governed by four physical mechanisms according to Ting et al (1983):

- Pore ice strength
- Soil strength (interparticle friction, particle interference, and dilatancy effects)
- Increase in effective stress due to the ice bonds resisting dilation when a dense soil is sheared
- Synergistic strengthening effects between soil and ice preventing the collapse of the soil skeleton

The behaviour of a frozen soil under uniaxial tension is more brittle than under uniaxial compression. The behaviour is also less sensitive to strain rate and temperature variations. Limited tensile testing is done on frozen soils.

The strength of a frozen soil increases as the temperature decreases. For dense sand, silty sand and Boom Clay the temperature - strength relationship is shown in Figure 4.4 (after Bourbonnais, 1984). For the sand it can be seen that up to -40 [$°C$] the strength increases and a maximum is reached at about -100 [$°C$]. The Boom Clay shows an exponential increase in strength for temperatures at least down to -100 [$°C$].

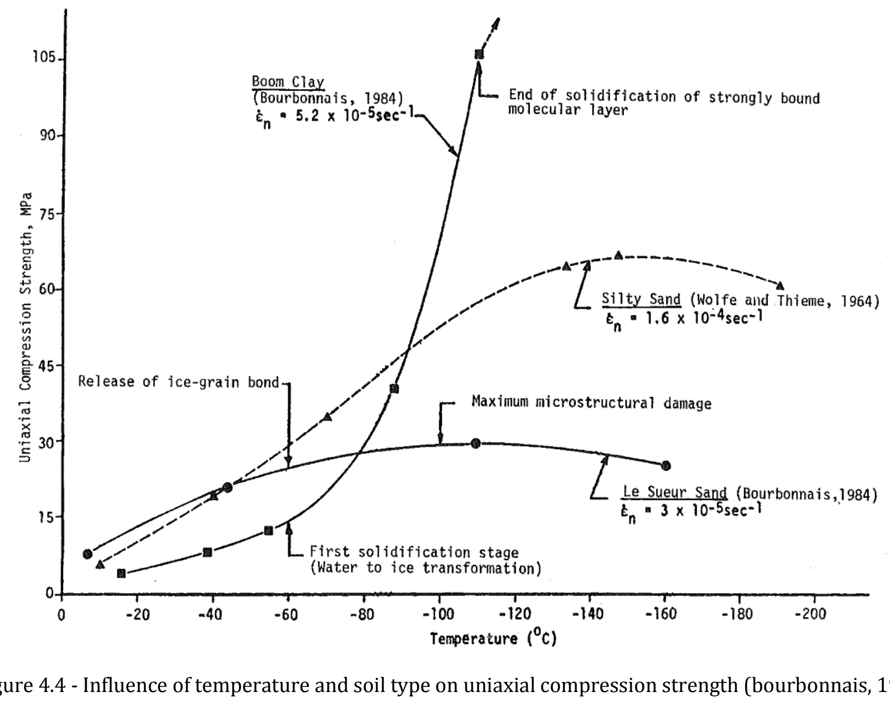


Figure 4.4 - Influence of temperature and soil type on uniaxial compression strength (bourbonnais, 1984)

However, also the brittleness of the material increases with temperature, which means that after a peak strength is reached the strength reduction is large. Brittleness occurs at higher temperatures with sands and silts than with clays as clay will require lower temperatures to freeze all pore water. The unfrozen pore water keeps the frozen soil plastic. Bourbonnais and Ladanyi (1985) conducted tests on the frozen Boom Clay which has lead to the conclusion that the frozen soil kept its plastic behaviour down to temperatures of -110 [°C] and axial strains larger than 5 [%]. For sand brittle behaviour already occurs from temperatures lower than -150 [°C] and failure occurs for strains larger than 0.5 [%].

4.2. Soil Conditions Requisite for Application of Freezing

Artificial soil freezing is a method suitable in almost all soil conditions. The only important requisite of the soil is a large enough water content, at least 10 [%]. As every soil type occurring in the soil profile of Amsterdam has water content higher than 10 [%] this will not lead to a problem. The characteristic lower value of the water content is shown in Table 4.2.

Layer	Water content [-]
Backfill	0.62
Trench fill	1.49
Mudflat deposit, Hydrobia clay	0.70
Peat	1.70
1st Sand layer	0.16
Alleröd	0.28
2nd Sand layer	0.20
Marine Eem clay (zone 1)	0.36
Marine Eem clay (zone 4)	0.46
Layer of Harting	0.73
Glacial Drenthe clay	0.23
Glacial Warven clay	0.31
3rd Sand layer	0.15

Table 4.2 – Water contents of soil layers in the soil profile of Amsterdam

Two other important factors influencing the application of AGF are the groundwater flow and the chemical composition of the groundwater. Groundwater flow can lead to serious problems when trying to close a freeze wall. Groundwater flow with large velocities provides a continuous source of heat and could lead to a steady state in which the frozen body stops growing. In general one can say the maximum flow velocities for the two freeze methods are (Jessberger et al, 2003):

→ 2 [m/day] for brine freezing
→ 4 to 6 [m/day] for liquid nitrogen freezing

If the velocities exceed these guidelines additional measures are an option to keep the ground freezing method possible.

- Reduce the freeze pipe spacing
- Installation of addition freeze pipes (an additional row)
- Lowering the temperature of the coolant
- When using brine, the use of liquid nitrogen on critical locations
- Reduce the permeability and groundwater flow by grouting
- Installation of wells to reduce the flow gradient

Chemical contaminants or dissolved salts may lead to difficulties when trying to create a frozen soil body. As stated before salts lead to a lower freezing point of the water, which will lead to a lower strength of the soil body at the same temperature. Contaminants concentrations might not freeze of freeze at a low rate which can lead to zones with a lower strength or even partially unfrozen zones.

In Amsterdam the groundwater flow is of such small magnitude that it does not affect the freezing process. Values are presented in Table 4.3[6]. The freeze works will take place in the 2nd Sand layer.

[6] Memo between SATURN and ABNZL (2007) *Thermische Vereisungparameter, DO.RAP.DWV.120, 24-5-2007*

Table 4.3	**q_{max} [$m^3/day/m^2$]**
1st Sand layer	$9.3 \cdot 10^{-3}$
2nd Sand layer	$7.0 \cdot 10^{-3}$
3rd Sand layer	$3.0 \cdot 10^{-3}$

The salt content in the ground water lower than 100 [*mg/l*], which is not likely to influence the freeze process. No problems are yet encountered while freezing cross connections of the two tunnel tubes related to contaminants or salty groundwater. Assumed is therefore that this will not lead to any problems in this feasibility study.

CHAPTER 5

CONSTRUCTIVE DESIGN OF AN UNDERGROUND STATION USING FREEZING TECHNIQUES

In this chapter the construction phasing of the design chosen in Chapter 3 is determined. Also the processes occurring during and after a freeze-thaw cycle found in Chapter 4 are further elaborated for practical application in a Plaxis calculation. Modelling criteria for this calculation are determined and some execution aspects are listed.

5.1. Construction Phasing

The construction process of an underground station constructed using freezing techniques can roughly be divided in to six phases. These phases are explained shortly in this chapter with the use of schematic drawings.

Phase 1 - Installation of the diaphragm walls

Two building pits are necessary as two entry points to the station are a requirement in the DPvE '96. To construct two building pits, 110 [*m*] apart, diaphragm walls are installed. The walls have a thickness of 0.5 [*m*] and are installed up to a depth of NAP -35 [*m*] (up to the Eem Clay layer). Dimensions of the pits are equal; both have a length and width of 28 [*m*] x 10 [*m*]. Excavation is done after the tunnel tubes have been bored through the building pits.

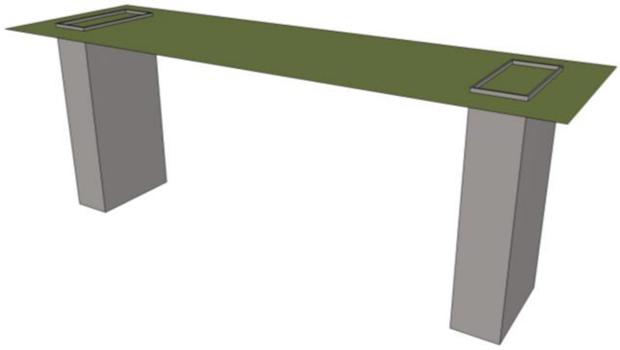


Figure 5.1 – Installing diaphragm walls for two entrance shafts

Phase 2 - Boring of the tunnel tubes

Two tunnel tubes with an external diameter of 6.52 [m] are constructed using a tunnel boring machine. The depth is equal for both tunnels, the centre of the tunnel face is at NAP -21.5 [m], and their relative distance is 5 [m]. To let the boring process be continuous the tunnels need to be bored through the diaphragm walls. The cross sections where the TBM passes through the diaphragm walls are reinforced using glass fibre, instead of the usual steel reinforcement cages.

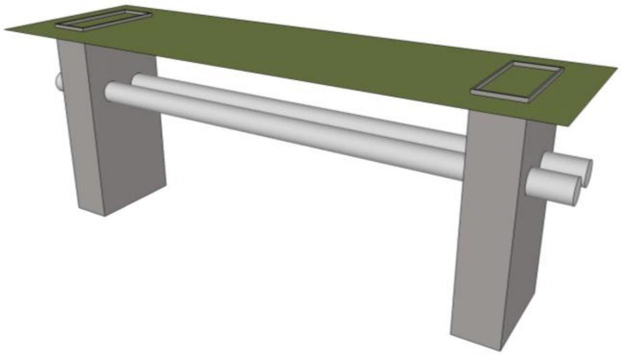


Figure 5.2 – Boring tunnels through the diaphragm walls

Phase 3 - Excavation of the building pits

The excavation of the building pits is done in steps while simultaneously lowering the groundwater level inside the building pit and installing struts to ensure a stable situation. Tension piles are needed to prevent uplift of the floor of the building pit. Grout injections are used to ensure a watertight connection at the crossing of a tunnel tube and diaphragm wall. Inside the pits the tunnel elements are removed. In the schematic drawing of each building pit two walls have been removed to give a view inside the building pits.

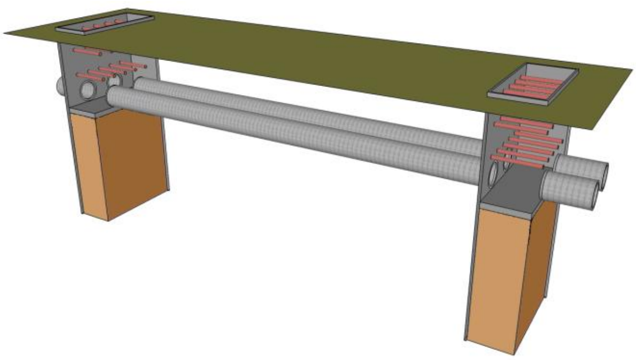


Figure 5.3 – Excavation of building pits using struts and if necessary tension piles

Phase 4 - Installing of freeze pipes

Freezing could be done from the building pits parallel to tunnel tubes or from one tunnel tube towards the other. The choice has been made to install from both building pits freeze pipes, parallel to the tubes, towards each other. When freezing from tube to tube the length of the freeze pipes would have been smaller but some freeze pipes will need to be turned off to be able to excavate soil. This will lead to difficulties in the freeze process. Since freezing over large lengths has been proved possible and it will lead to a simpler freeze process this option will be used. Also construction works in the tunnel tubes are more flexible.

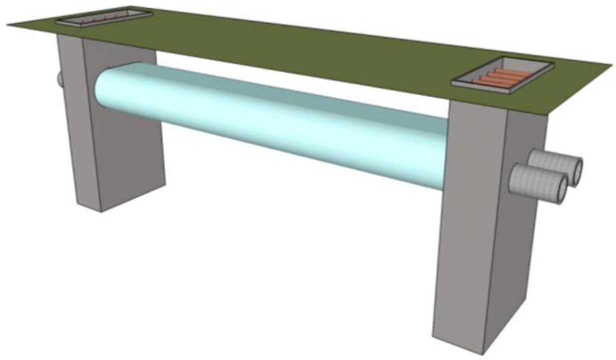


Figure 5.4 –Freezing the soil body surrounding the tunnel tubes

Some overlap at the meeting point of pipes of different pits is needed therefore the length of a pipe varies between 50 [m] and 60 [m]. The freezing will be done in the first and second sand layer and in the Allerod layer in between. Freeze pipes will be installed in multiple rows to be able to grow a thick enough frozen soil body. The pipes are installed at a centre to centre distance of maximum 15 times the outer diameter of a freeze pipe (Zentrum Geotechnik). Usually a distance of 1 [m] is chosen. A smaller distance between the centres is possible, however when freezing over such large distances the risk exists the freeze pipes will cross each other due to deviations when installing them. Installing the freeze pipes with a maximum deviation of 1 [%] is possible, recently also maximum deviations of 0.5 [%] have been proved possible.

Phase 5 - Excavation of the station

When the soil body is closed and at the required thickness excavation inside can start. Proving the soil body is closed may be a challenge when freezing such large volumes. Excavation of the total station will be done in parts. Temporary stability of all parts will be ensured with shotcrete. The final concrete lining is constructed to withstand full soil and water pressures in the final situation. When the final lining is completed the freezing can stop and the frozen soil body will start thawing as a result.

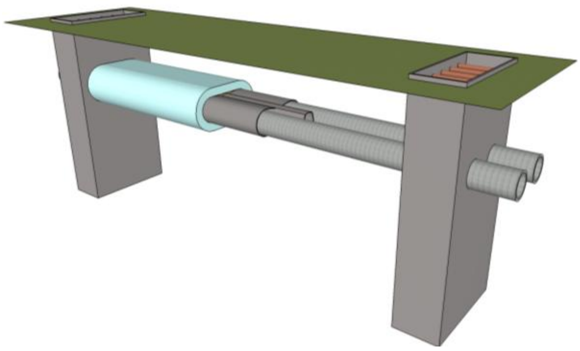


Figure 5.5 – Excavation of the soil within the frozen soil body

In the schematic drawing the frozen body is not drawn over the whole length of the station to be able to show the construction works within. First the middle part is excavated and then the side parts. The shotcrete wall separating the middle and side parts is demolished simultaneously with the construction of side parts. To excavate the side parts the tunnel lining has to be removed first. Step by step a part of the tunnel lining is removed, soil is excavated and shotcrete is applied to stabilize the wall. The New Austrian Tunnelling Method is used to excavate all parts.

Phase 6 - Finalizing the station

After completion of the final lining the station can be finalized, with the application railway tracks, a platform, elevators and all other installations.

Figure 5.6 – Final station with thawed soil body

5.2. Soil Properties

5.2.1. General

For the location of the design station Rokin in Amsterdam is chosen. Parameters used in the calculations for this station are written down in *Grondonderzoek Noord/Zuidlijn - Parameterset definitief ontwerp* prepared by Adviesbureau Noord/Zuidlijn. The soil profile contains the layers shown in Table 5.1.

Top layer relative to NAP [m]	Bottom layer relative to NAP [m]	Layer	Layer description	Layer number
+1.0	-4.5	Backfill	CLAY, peat, sand	1
-4.5	-9.8	Trench fill	PEAT, with clay layers	4
-9.8	-12	Mudflat deposit, Hydrobia clay	CLAY, strongly sandy	11
-12	-12.5	Peat	Clayey PEAT	12
-12.5	-14.8	1st Sand layer	SAND, moderately fine, slightly silty	13
-14.8	-16.5	Alleröd	LOAM, strongly sandy	14
-16.5	-31	2nd Sand layer	SAND, moderately fine to coarse	17
-31	-40	Marine Eem clay (zone 1)	CLAY, slightly silty to slightly sandy	19
-40	-43	Marine Eem clay (zone 4)	CLAY, slightly silty to slightly sandy	19c
-43	-43.5	Layer of Harting	CLAY, slightly sandy, slightly peaty	20
-43.5	-51	Glacial Drenthe clay	CLAY, slightly sandy	22
-51	-53	Glacial Warven clay	CLAY, slightly silty	23
-53	-100	3rd Sand layer	SAND, moderately fine, slightly silty to slightly gravelly	24

Table 5.1 - Soil profile at the location of Rokin, Amsterdam

5.2.2. Initial Soil Properties

In the document "*Grondonderzoek Noord/Zuidlijn - Parameterset definitief ontwerp*" all parameters needed to perform a finite elements method calculation are defined. For each soil layer occurring in the soil profile along the North/South Line weight, strength, permeability and stiffness parameters are determined by executing laboratory tests. Minimum, maximum and average parameters are determined. The thickness of the soil layers is determined by cross sections of the soil provided by Witteveen+Bos. The soil parameters can be found in Annex C.

5.2.3. Soil Properties Frozen Soil

CDM, a research institute in Bochum, Germany, conducted laboratory tests to determine the properties of frozen samples of the Allerod, 2nd Sand layer and Eem Clay. Uniaxial compression and uniaxial creep tests were performed to be able to describe the strength and deformation behaviour. The properties of frozen soils change over time due to the creep process, as mentioned in paragraph 4.1.3. Characteristic soil properties for several stand up times, 24 hours, 1 week, 2 weeks, 4 weeks and 8 weeks, are therefore determined and can be found in Table 5.2. The stiffness moduli are calculated using Klein's creep equation for which the creep parameters A, B and C are determined during creep tests. The stiffnesses are secant stiffnesses.

Table 5.2		**Short term values** (lab conditions)		**Time dependent values**														
Layer	Temperature T [°C]	Uniaxial compressive strength q_f [MN/m²]	Shear parameters φ_f/c_f °/[MN/m²]	Young's modulus of elasticity E_f (t) [MN/m²]					Compressive strength (allowable) σ_P (t) [MN/m²]					Cohesion intercept (allowable) c_f (t) [MN/m²]				
				24h	1w	2w	6w	8w	24h	1w	2w	6w	8w	24h	1w	2w	6w	8w
Eem Clay	-10	3.6	13.5/1.27	85	65	55	50	45	1.30	0.95	0.83	0.70	0.65	0.51	0.37	0.33	0.28	0.26
Allerod	-10	4.5	16.0/2.5	137	127	123	118	115	1.65	1.50	1.45	1.35	1.30	0.62	0.57	0.55	0.51	0.49
Sand	-10	8.5	27.7/1.9	200	170	160	145	140	1.85	1.45	1.30	1.15	1.10	0.56	0.44	0.39	0.35	0.33

It can be read from Table 5.2. the compressive strength decreases in time. Andersland and Ladanyi (2003) endorse this finding. They state that the strength of a frozen soil decreases with time from its short term strength to its long term strength.

The Poisson's ratio and internal friction angle were not determined during the tests performed by CDM. In literature values of Poisson's ratios for several soils are given by Tsytovich (1975). Böning (1992) gave values for the friction angle in relation to time. Based on their findings the Poisson's ratio and friction angle are chosen.

Material	**% Passing**			**Water content [%]**	**Temperature [°C]**	**Stress [kPa]**	**Poisson's ratio**
	250μm	**50μm**	**5μm**				
Sand	7	9.4		19.0	-0.2	196	0.41
				19.0	-0.8	588	0.13
Silt		64.4	9.2	28.0	-0.3	147	0.35
				28.0	-0.8	196	0.18
				25.3	-1.5	196	0.14
				28.7	-4.0	588	0.13
Clay			>50	50.1	-0.5	196	0.45
				53.4	-1.7	392	0.35
				54.8	-5.0	1176	0.26

Table 5.3 – Poisson's ratios for frozen soils (after Tsytovich, 1975)

Soil type	Condition	Short-term properties				Long-term properties			
		σ_c	ϕ	c	Young's modulus	σ_c	ϕ	c	Young's modulus
		MN/m^2	-	MN/m^2	MN/m^2	MN/m^2	-	MN/m^2	MN/m^2
Non-cohesive	Dense	7	38	2	600-900	4	22	1.4	260-400
	Medium dense	5	30	1.5	500	3.5	15	1.2	250
Cohesive	Semi-stiff	3	20	1	400-500	2	10	0.8	200-260

Table 5.4 - Strength properties of frozen soils as a function of the stand-up time, for T=10°C , after Böning

5.2.4. Soil Properties of Thawed Soil

As stated before it is known undisturbed clays lose their loading history after a freeze-thaw cycle due to structural changes. After thawing the clay is remoulded and the consolidation process has to start from the beginning. The void ratio has increased due to the phase change of water into ice and the attraction of water to the frost front through cracks.

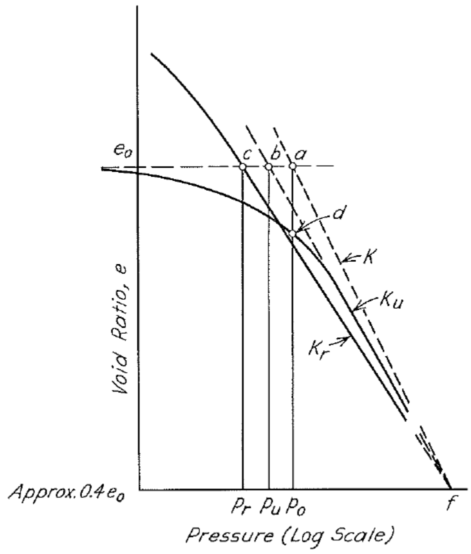


Figure 5.7 - Relations for e and p for clay of ordinary sensitivity corresponding to K_r remoulded and K_u undisturbed stated in the laboratory, and K natural state in the field

In de graph e_0 and p_o correspond to respectively the initial void ratio and the initial effective overburden pressure of normally consolidated clay. The overburden pressure is gradually removed from the clay sample during the process of sampling and when starting the consolidation test the load is gradually increased again. Line K_u in the graph is a result of the loading of the undisturbed sample.

K_r represents the e-log p line for remoulded clay. K_r has a slightly smaller slope than the line for undisturbed clay, K_u. As the compressibility index C_c is the tangent of these lines, this index is smaller for remoulded soils. For normally loaded clays the compression index can be determined for both undisturbed and remoulded clays using the liquid limit. The liquid limit is defined as the water content in percent of the dry weight at which the soil transfers from plastic to liquid behaviour. For remoulded clays the following equation yields, with a scattering of 30 [%] (Skempton, 1944):

$$C_c' = 0.007(L_w - 10\%)$$

The compressibility of undisturbed clay with medium or low sensitivity can be roughly determined with the following equation:

$$Cc \sim 1.30Cc' = 0.009(Lw\text{-}10\%)$$

The sensitivity of a clay is defined by the water content, *w*, in relation to the liquid limit. For normally loaded clays *w* is close to L_w. If *w* is considerably lower, the sensitivity is low and if *w* is considerably greater, the sensitivity is high (Terzaghi et al, 1967). For pre-compressed clay layers the compressibility not only depends on the liquid limit but also on the ratio $\Delta p/(p_0'\text{-}p_o)$. If this ratio is less then 50% the compressibility is likely to be 10 to 25 [%] of the compressibility of normally loaded similar clay. In Amsterdam the soil is normally consolidated.

In what way the Allerod layer precisely will react to the freeze-thaw process is unknown. The soil layer is sandy at some locations but clayey at other locations. Practical experience of Witteveen+Bos has led to the opinion the layers parameters do not change much. In calculations made by Witteveen+Bos a stiffness decrease of 5% is taken into account. However when the Allerod would react more as clay and would be remoulded the effect of the freeze-thaw process will have more effect on the parameters. The determination of those parameters explained below.

The liquid limit before freezing is 30.5 [%] and the plastic limit is 26.1 [%] as tested by CDM. This does not differ much from the liquid and plastic limit of Eem Clay determined by Deltares, which are respectively 31.3 [%] and 26.4 [%]. After a freeze-thaw cycle the liquid limit of the Eem Clay was increased to 38.5 [%]. For the Allerod no tests are done, therefore the liquid limit of the Eem Clay is used to determine the compressibility index after thawing.

$$Cc' = 0.007(Lw\text{-}10\%)$$
$$\Rightarrow\ Cc' = 0.007(38.5\text{-}10) = 0.1995\ [\text{-}]$$

The void ratio can be determined using the fixed relation between porosity and void ratio. The original porosity is 0.44. The assumption for the increase of porosity of 3 [%] leads to a new porosity n_f of 0.47 and a new void ratio e_f of:

$$e_f = \frac{n_f}{1-n_f}$$
$$\Rightarrow\ e_f = \frac{0.47}{1-0.47} = 0.89\ [\text{-}]$$

The size of the increase of permeability after a freeze-thaw cycle depends on the liquid limit of the soil, the void ratio and the initial permeability. Nagasawa and Umeda (1985) found a relation between the normalized permeability ratio k_r and the liquid limit of a soil after a freeze thaw cycle, shown in Figure 6.8.

The normalized permeability ratio is defined with the following formula:

$$k_R = \frac{k_{FT}}{k_i}\left(\frac{1+e_f}{e_f{}^3}\right)$$

In which:

k_r	= normalized permeability	[-]
k_{FT}	= permeability after freeze-thaw	[*m/s*]
k_i	= initial permeability (before freeze-thaw)	[*m/s*]

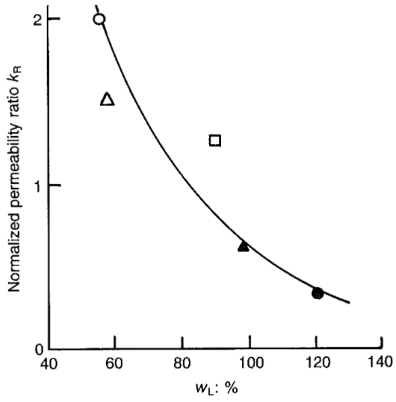


Figure 5.8 – Relation between the water content of a soil to the normalized permeability ratio k_r

Since k_R be read from Figure 5.8 and k_{FT} is the value to be determined the formula is rewritten. The calculation of k_{FT} is shown below:

$$k_{FT} = k_r \cdot k_i \left(\frac{e_f^{\ 3}}{1+e_f} \right)$$

$$k_{FT} = 5.0 \cdot 3.0 \cdot 10^{-5} \left(\frac{0.89^3}{1+0.89} \right)$$

$$\Rightarrow \quad k_{FT} = 5.59 \cdot 10^{-5} [m/s] = 4.83 \; [m/day]$$

A value of k_R corresponding to a liquid limit of 38.5% is difficult to read from the graph. For lower values of liquid limit (below 60%) the value for k_R goes up quite quick, therefore a value of 10. For the permeability after freeze-thaw then a value of $4.83 \cdot 10^{-5}$ [m/s] is obtained.

The compression index Cc is directly related to the modified compression parameter λ^* of the Soft Soil Creep model. With the use of empirical relations the other input parameters for the Soft Soil Creep model can be determined. These are the modified creep parameter μ^* and the modified swelling index κ^*. The determination of all three parameters is shown with the following equations.

$$C_C = 2.3\left(1+e_0\right) \cdot {\sigma_{ref}}/{E_{oed}}$$

$$\lambda^* = {\sigma_{ref}}/{E_{oed}}$$

$$\Rightarrow \quad \lambda^* = \frac{C_C}{2.3\left(1+e_0\right)} = \frac{0.1995}{2.3\left(1+0.89)\right)} = 0.0459 \text{ [-]}$$

A λ^* of 0.0459 corresponds to an E_{oed} of $2.2 \cdot 10^3$ [kPa] according to the relation $\lambda^* = 100/E_{oed}$. The original oedometer stiffness of the Allerod was $13 \cdot 10^3$ [kPa]. According to the Plaxis material model manual the ratio λ^*/ κ^* ranges usually between 2.5 and 7. In this manual also a range for the ratio of λ^*/ μ^* given, namely 15 to 25. Both values for the ratio are chosen in the centre of the range.

$$\kappa^* = \lambda^* / 4.75$$

$$\Rightarrow \quad \kappa^* = 0.0459 / 4.75 = 9.97 \cdot 10^{-3} \text{ [-]}$$

A κ^* of $9.97 \cdot 10^{-3}$ corresponds to an E_{ur} of $9.0 \cdot 10^3$ [*kPa*] according to the relation $\kappa^* = 0.9 \cdot 100 / E_{ur}$. The original unloading and reloading stiffness of the Allerod was $45 \cdot 10^3$ [*kPa*]. It can be seen this means a large stiffness reduction due to the freeze thaw cycle.

$$\mu^* = \lambda^* / 4.75$$

$$\Rightarrow \quad \mu^* = 0.0459 / 20 = 2.30 \cdot 10^{-3} \text{ [-]}$$

Allerod parameters after thawing are determined for three different scenarios. These three options will all be tested in Plaxis. An overview of the changed soil properties is given below.

1. Stiffness decrease of 5 [%]
   → E_{50}, E_{oed} and E_{ur} are lowered by 5 [%]
2. Allerod acts as a remoulded clay HS model
   → higher void ratio, compression index and permeability
3. Allerod acts as a remoulded clay SSC model
   → higher void ratio and permeability
   → Instead of E_{50}, E_{oed} and E_{ur} the SSC parameters μ*, κ* and λ* are used

5.3. Modelling Criteria

Criteria for settlements

For the North South Line maximum allowed settlements at ground level have been set at 20 [*mm*]. An additional requirement is crack width of 2 [*mm*] at the maximum. According to Boscardin and Cording this leads to a category of damage of 2. Category 2 includes typical crack widths up to 5mm, where as category 1 includes typical crack widths up to 1 [*mm*]. Category 2 corresponds with a limiting tensile strain of 0.075 to 0.15 [%]. When a tensile strain of 0.1 [%] is taken with Figure 5.9 the maximum angular distortion can be determined. A curve parallel to the already present curves starting at ε_h=1 [‰] is drawn. A maximum angular distortion of $2 \cdot 10^{-3}$ is derived which matches with a skew of 1:500.

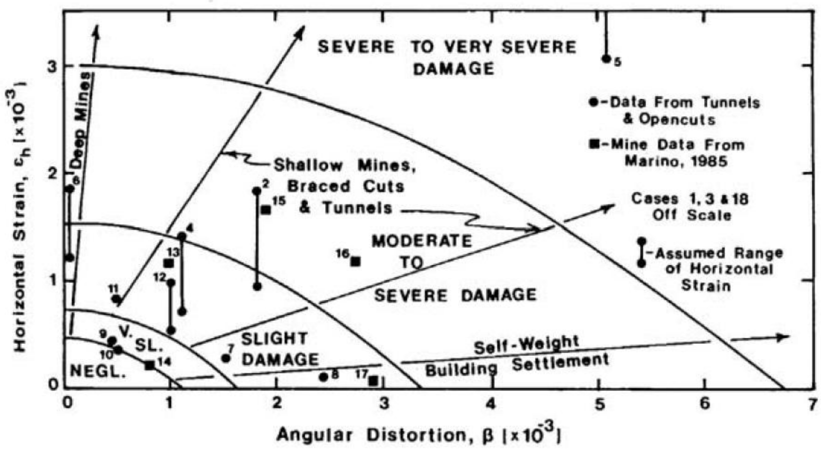


Figure 5.9 – Graph by Boscardin and Cording relating the tensile strain to the angular distortion

Angular distortion is defined as the relative rotation of a building, β. It is the angle between part of the building that settles the most and the tilt line of the entire building (the dotted line between the settlements of point A and D).

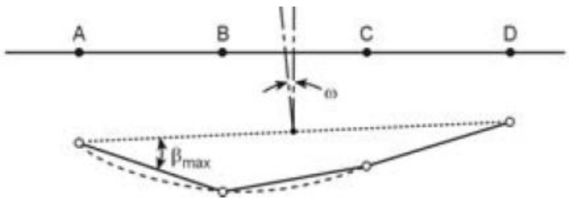


Figure 5.10 – Definition angular distortion

Criteria for structural forces

The maximum value for the bending moment in the bored tunnel lining as it is located near Station Rokin is 240 [*kNm/m*]. This value is valid for a lining with light reinforcement. For heavy reinforcement the maximum value is 330 [*kNm/m*]. The main part of the tunnel, about 80 [%], is constructed with light reinforcement in the lining. At locations where more reinforcement could lead to for instance a reduction of the thickness of the frozen body, heavy reinforcement is applied. The advantages of reducing the thickness of the frozen soil body outweigh the extra costs of heavy reinforcement.

For shotcrete lining the maximum moment depends on the amount of reinforcement applied, the thickness of the shotcrete and the governing normal force. With a net thickness of 250 [*mm*] a maximum value for the bending moment is assumed to be 100 [*kNm/m*], based on experiences with freezing cross connections.

If the columns inside the station have an assumed length and width of 1.5 [*m*] the maximum normal force can be determined. A geometrical reinforcement percentage ω is assumed of 3 [%] and a concrete quality B100 with a calculation value for axial compression of f'_b=60 [N/mm^2]. To determine the maximum normal pressure the following formulas are used:

$$N_d = A_b \cdot f'_b + A_s \cdot f_s$$

$$\psi = \omega \cdot \frac{f_s}{f'_b}$$

$$N_d = A_b \cdot f'_b \left(1+\psi\right)$$

$$\Rightarrow \quad N_d = 1500^2 \cdot 21\left(1+0.03 \cdot \frac{435}{60}\right) = 57.5 \cdot 10^6 [N] = 57.5 \cdot 10^3 \, [kN]$$

In which:

ψ	= mechanical reinforcement percentage	[-]
f_s	= calculation strength of reinforcement steel	[N/mm^2]
A_b	= area of concrete in cross section	[mm^2]
A_s	= area of steel in cross section	[mm^2]

The criteria set in this paragraph will be used to determine whether results of the calculations in Plaxis are allowable.

5.4. Modelling in Plaxis

5.4.1. Change of Soil Properties Throughout Construction Phases

In Plaxis the soil properties of several soil clusters will be changed in a number of the construction phases. Below in short the changes and possible effects are listed.

1. Initial conditions
   a. Soil properties (strength and stiffness parameters) as in "*Grondonderzoek Noord/Zuidlijn – Parameterset definitief ontwerp*"

2. Freezing (wished in place)
   a. Increase strength en stiffness parameters
   b. Heave
      i. Significant heave in fine grained materials: excess pore water cannot drain and due to slower freezing rate extra water is attracted at the freeze front
      ii. No significant heave occurs in coarse grained materials

3. Maintaining the frozen body
   a. Creep
   b. Growing of the frozen body -> further heaving

4. Thawed soil body (wished in place)
   a. Consolidation due to:
      i. The phase change of ice to water, decrease of volume
      ii. Self weight
      iii. Applied loading
   b. Excess pore pressures
      i. Possibly occurring in fine grained materials depending on the drainage rate -> shear strength reduction immediately after thawing
      ii. Not occurring in coarse grained materials

5. End conditions
   a. In fine grained materials:
      i. Permeability increases -> coefficient of consolidation increases
      ii. Loading history is lost -> soil acts as remoulded soil
   b. In coarse grained materials: no structural effects

5.4.1.1. Describing Frost Heave Behaviour

In Japan the following equation (by Takashi et al.) is often used to determine frost heave:

$$\varepsilon = \varepsilon_0 + \frac{\sigma_0}{\sigma} \cdot \left(1 + \sqrt{\frac{U_0}{U}}\right)$$

In which:

ε	= frost heave ratio	[%]
σ	= effective overburden pressure	[kN/cm^2]
U	= frost heave propagation rate	[mm/h]
ε_0, σ_0, U_0	= frost heave parameters, which are determined by laboratory tests	

Also a method developed by Konrad and Morgenstern (1980) can be used to determine frost heave rate. Their method describes a relationship between the velocity of arriving water and the frost-front advance rate. The heave rate is expressed as:

$$\frac{dh}{dt} = 1.09v + 0.09n\frac{dz}{dt}$$

In which:

dh/dz	= frost heave rate	[*mm/s*]
v	= velocity of arriving water at the frost line	[*mm/s*]
n	= soil porosity	[-]
dz/dt	= frost front advance rate	[*mm/s*]

A third method to determine frost heave behaviour is by using the segregation potential (SP). The SP-value depends on the soil type and overburden pressure, by the following equation:

$$SP_{(P)} = SP_{(0)} \cdot e^{-ap}$$

In which:

$SP_{(p)}$	= SP-value related to pressure p	[$mm^2/s * °C$]
$SP_{(0)}$	= SP-value for no vertical pressure	[$mm^2/s * °C$]
a	= factor depending on the type of soil	[$1/MN/m^2$]
p	= pressure	[MN/m^2]

Research centre CDM in Germany performed tests for the North/South line to determine this SP-value. Tests were done on samples of the Eem Clay and Allerod for different pressures. Results for the Allerod are shown in Table 5.5

Specimen No.	**Applied pressure** [MN/m²]	**SP-Value** Initial freezing phase [mm²/s * °C] *10⁻⁵
20853	0.023	30 to 130
20854	0.200	1 to 21
20846	0.300	0.5 to 6.5
20852	0.500	Sample was pre-frozen before test and partly thawed during test
20303	0.023	145
20305	0.100	4.5

Table 5.5 – Value for the Segregation Potential for different applied pressures

Soils are usually classified as non-frost susceptible when the value of the segregation potential does not exceed $1 \cdot 10^{-5}$. This implies applying a load such that the SP value decreases to this value will stop further heaving by a prevention of water suction towards the frost front. Freeze pressures are estimated between 150 and 475 [kN/m^2] for the Allerod using this assumption.

Tests performed by CDM gave a relationship between soil type, applied pressure and segregation value. With the following equation the SP-value is related to the frost heave rate, *h'*, and the temperature gradient over the frozen fringe, ΔT.

$$SP_{(p)} = \frac{h'}{gradT}$$

The heave rate is defined in millimetre per second and the temperature gradient in Celsius per millimetre. The prevailing total stress is 224 [kN/m^2]. In the report of CDM it is concluded that the result of the test with an applied pressure of 300 [kN/m^2] acts different than the other samples, therefore its result is being disregarded. A SP-value of $10 \cdot 10^{-5}$ [$mm^2/s \cdot °C$] is chosen for the Allerod in this case, by looking at the results of the test with an applied load of 200 [kN/m^2].

The frozen fringe is the area between the last ice lens and the total unfrozen zone and was first introduced by Miller (1972). More precisely it is defined as the zone between the warmest ice lens and the zero isotherm. Akagawa (1988) performed an experimental study on frozen fringe characteristics as the phenomena in this area seemed to underlie the entire heaving process. A 700 hour frost heave test was conducted on a silty clay sample and using X-ray radiographs the development of ice lenses could be monitored clearly. The varying thickness and temperature gradient of the frozen fringe were part of the determined data. Results are shown in Figure 5.13.

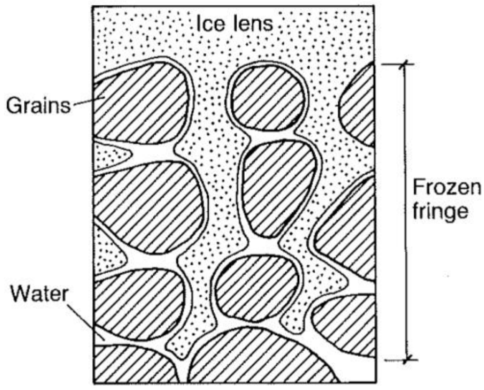
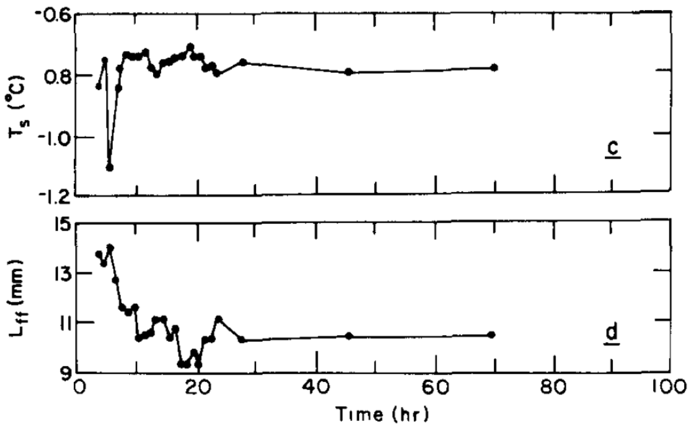


Figure 5.12 – Definition frozen fringe (after Harris)



The upper graph shows the temperature of the warmest ice lens over time. It varies around -0.8 [°C]. The temperature difference is now known as the other boundary of the frozen fringe is considered to be the zero isotherm.

The lower graph shows the thickness of the frozen fringe over time. After 30 hours it reaches a steady value of about 11 [*mm*].

Figure 5.13 – Results Akawaga (1988)

The Allerod is a sandy silt, therefore due to the different grain size distribution the values found in the experimental study of Akawaga are not directly valid for the Allerod. As the Allerod contains more coarse grained material the heave rate will have a lower value. The temperature difference will be larger and/or the thickness of the frozen fringe will be smaller. For Akawaga's silty clay the temperature gradient is $7.27 \cdot 10^{-2}$ [*°C/mm*]. As it is not known what the magnitude of the difference between the temperature gradient of the Allerod and the silty clay is, the value of $7.27 \cdot 10^{-2}$ [*°C/mm*] is used for further calculation. The heave rate, based on a conservative assumption, then becomes:

$$h' = gradT \cdot SP_P$$

$$\Rightarrow \; h' = 7.27 \cdot 10^{-2} \cdot 10 \cdot 10^{-5} = 7.27 \cdot 10^{-6} \; [mm/s] = 0.63 \; [mm/day]$$

The research institute Deltares conducted an extensive research on the development of heave, especially on the soil types present at the location of the North South line. Soil types on which the research has been done are Allerod, Eem Clay and the Transition layer. For testing a standard triaxial apparatus with additional measurement systems and refrigerating elements is used. Four radial deformation sensors and two axial deformation sensors are added.

Axial soil heave is found to have a very low value, radial soil heave however should be a point of interest in each design using artificial ground freezing. Radial frost heave is found to be significant in all three of the tested layers. For the station design the Allerod is of importance. This layer shows a radial heave expansion with a maximum of 3 [%] under drained conditions. The composition of the Allerod however varies quite largely, from clayey sandy silt to a silty sand. The minimum radial heave to be found by Deltares is 1.5 [%]. The in-situ effective stress and pore pressures are applied during the tests.

A radial expansion of 3 [%] means for the 1.7 [*m*] thick Allerod layer a heave of 51 [*mm*]. The calculated heave rate was 0.63 [*mm/day*]. If the freeze works are considered to last for three months, the heave will reach a value of 59 [*mm*]. The two results do not differ much.

Heave settlement of 51 [*mm*] or 59 [*mm*] may seem little, for instance compared to the freeze wall constructed for metro station Rotterdam Central. There heave settlements in the order of decimetres were found. However the soil layers that heaved in Rotterdam consisted of weak peats and clays, the soil layers were much thicker and the prevailing effective stresses were lower.

Experience with cross connections for the North South Line learned there were very few volume changes due to freezing in the Allerod layer. Only the volumes of frozen soil are significantly smaller than the frozen volumes proposed in this case study. The results of the tests of Deltares and CDM are therefore implemented in the Plaxis model. CDM also presented a graph with the percentage of heave according to the applied load. Soil pressure on top of the Allerod is 224 [kN/m^2], which according to the graph corresponds with a heave percentage of 0.3 [%]. Note on the shown relation in the graph is that the local soil structure of the Allerod layer may vary considerably from the tested soil structure. Deviations to the better or the worse side may occur.

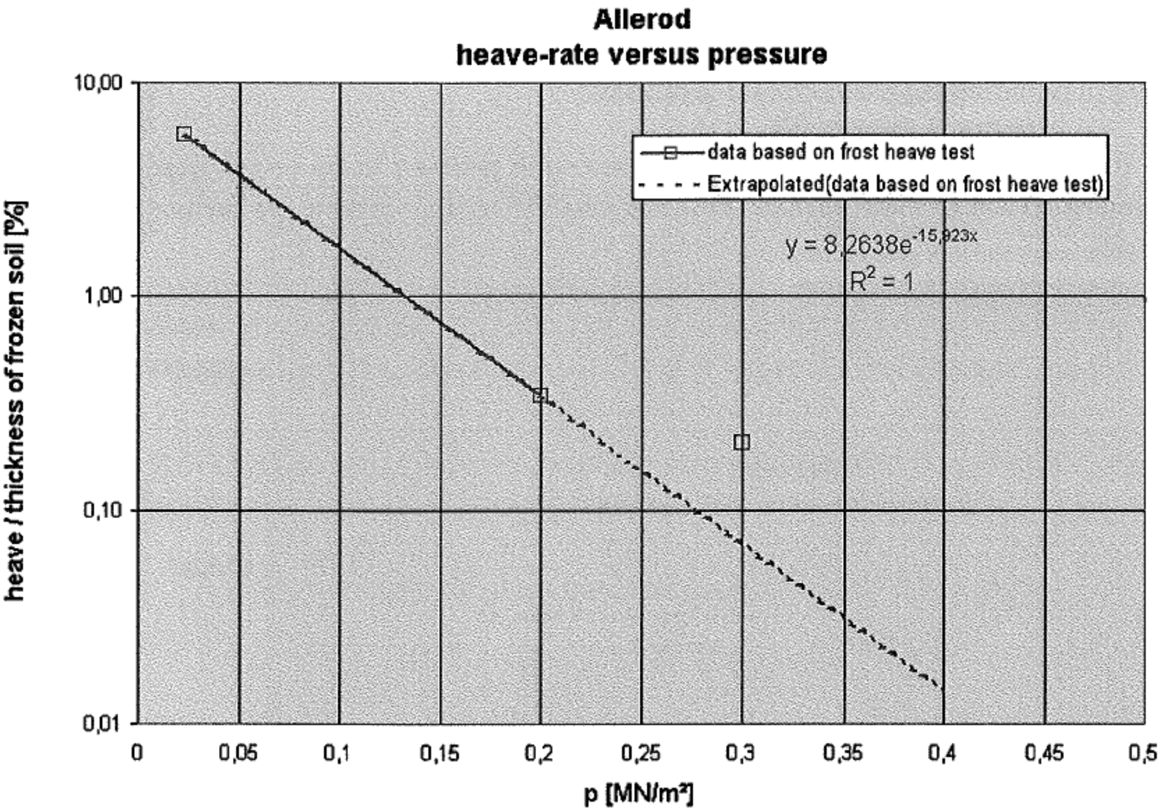


Figure 5.14 – Relation heave percentage to applied pressure tested by CDM

The largest value for the heave percentage found is 3 [%]. This value is applied in Plaxis as a volume change of the frozen Allerod soil clusters. Per phase in Plaxis an incremental strain can be applied to a volume element. At the end of a calculation step in which a strain increment is imposed a new equilibrium will be reached with a different volume. By applying a positive strain increment the stresses in the surrounding soil will rise. To stay at equilibrium with the stresses inside the volume these will also change. However part of the stress change will be diverted by arching in the soil surrounding the swelling soil. Therefore it is not possible to tell on beforehand how the stresses will develop.

The equilibrium reached after an imposed volume change will be kept in next phases if no new strain increment is applied. Rebound of the volume will not occur. If in different construction phases several strain increments are applied the sum of those increments is the total volume strain applied.

For the Allerod a total volume strain 3 [%] is aimed at. The volume change will occur over an amount of time and therefore can not be applied just to one construction phase. Chosen is for the following distribution of strain increments:

Phase		Strain increment
6.	Freezing soil body	1%
7.	Lowering pore pressures inside frozen soil body	0.1%
8. to 21.	Excavation and shotcrete of all parts	0.1%
22.	Final lining	0.5%
23.	Thawed soil body	-3%

Table 5.6 – Applied volume strain increments per construction phase

It is expected that in time the soil subsides as much as it has heaved. The attracted extra water towards the Allerod layer during the freeze period will drain when the soil warms up to positive temperatures again. This may take some time due to the soil composition of the Allerod. It consists of sand, but also clay and silt.

5.4.1.2. Describing Creep Behaviour

Frozen soils are susceptible to creep, which means they will keep developing deformations even with a constant load. In Plaxis the creep behaviour of the ice can be modelled in various ways. One way is to reduce the stiffness and strength stepwise in time.

CDM tested for the freeze works of the North South Line frozen samples of the Allerod, 2nd Sand layer and Eem Clay. The soil parameters that resulted from these tests can be found in Table 5.2 in paragraph 5.2.3. In Plaxis these parameters are entered for different construction phases. Before excavation has started the soil is given the parameters of 24 hours. No creep effects have occurred yet. When (part of) the soil inside the frozen body is excavated different parameters have to be subscribed to the frozen body itself.

If in Plaxis in successive stages only soil properties are changed no new stress distributions and according settlements will be calculated. The phase with the changed soil properties should therefore start not from the previous phase but from two phases earlier.

Another way to model creep behaviour in Plaxis is using the Soft Soil Creep model. The Soft Soil Creep model uses besides c', φ', ν_{ur}, ψ and K_0^{nc} the following parameters:

- λ* Modified compression index
- μ* Modified creep index
- κ* Modified swelling index

These three parameters are related to international acknowledged compression parameters according to the following equations.

$$\lambda^* = \frac{C_C}{2.3(1+e)} \qquad \kappa^* = \frac{C_\alpha}{2.3(1+e)} \qquad \mu^* = \frac{C_S}{1+e}$$

The Soft Soil Creep parameters are not known, but could be approximated by using the material curve generator which is available in Plaxis. Creep curves for several stresses are determined by CDM. One of these curves, the one for which the applied stress is the closest to the in situ stress of the Allerod, could be fitted with the curve generator. As a result of limited time this option is not further elaborated.

CDM research centre determined Klein's creep parameters with uniaxial creep tests. A third way to model creep behaviour in Plaxis would be to implement a new soil model according to Klein's creep equation into the program. This option is also not further looked into, as it will require quite some time. The formula of Klein is as follows:

$$\varepsilon_1 = \varepsilon_0 + A * \sigma_1^B * t^C$$

In this formula A, B and C are creep parameters. The tests were performed at a temperature of -10 [°C] and with several applied constant loads. The creep parameters obtained are shown in Table 5.7.

Soil	A [m²/(MN*h)]	B [-]	C [-]
Eem Clay	$8.0*10^{-4}$	1.75	0.31
Allerod	$1.0*10^{-3}$	3.30	0.17
Sand	$9.5*10^{-4}$	2.20	0.33

Table 5.7 – Creep parameters Klein's formula

To account for the creep in Plaxis the stiffness parameters are lowered manually according to Table 5.2.

5.4.2. Geometry and Material Properties

The inserted geometry of the problem is shown in Figure 5.15. As the problem is symmetric only half of the problem is modelled to increase the calculation speed. Objects in the model are:

- Soil profile as mentioned in paragraph 5.2.1.
- TBM tunnel, with a diameter of 6.52 [*m*]
- NATM tunnel which encloses the TBM tunnel, largest width is 9.0 [*m*] and largest height is 7.6 [*m*].
- Half a NATM tunnel at the boundary of the model, largest height is 7.6 [*m*].
- Geometry lines to function as the boundary of the frozen body. These lines enclose all tunnels. Thickness of the frozen body is set at 3 [*m*].
- Building, founded in the 1st Sand layer, modelled by plates and node to node anchors.
- A column at the intersection of the two NATM tunnels, modelled with a node to node anchor.

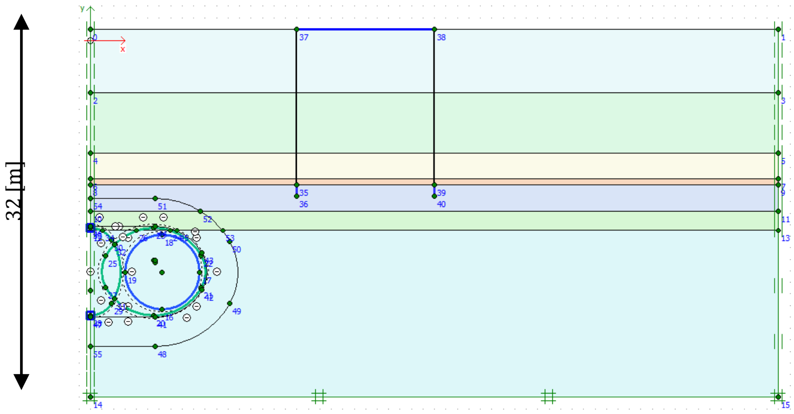


Figure 5.15 – Geometry of the model in PLAXIS

Characteristics of the bored tunnel

The bored tunnel consists of tunnel segments which mechanically work as hinges. Also longitudinal and transverse joints in place have their effect on the strength of the tunnel. In the model the tunnel is simplified to only a homogeneous plate which will show a more stiff behaviour than is actual the case. To account for the more stiff behaviour the modelled plate is assigned 40 [%] of the actual elastic modulus of the concrete. The elastic modulus and other characteristics of the bored tunnel are shown in Table 5.8. The flexural rigidity and normal stiffness are calculated in the out-of-plane direction.

Table 5.8 – Characteristics lining bored tunnel		
Thickness lining	0.35	[*m*]
Volumetric weight	2500	[kg/m^3]
Elasticity modulus	15	[*GPa*]
Poison's ratio	0.15	[-]
Flexural rigidity (EI)	$107.2*10^3$	[kNm^2/m]
Normal stiffness (EA)	$10.5*10^6$	[kN/m]

Characteristics of the NATM excavated station

The station will be excavated using the NATM. Temporary lining is made by applying shotcrete to the excavated surface, final lining will be made in-situ. The stiffness of the shotcrete is reduced to 50 [%], to account for cracks. Also for the final lining this reduction is made. The planned thickness of the shotcrete layer is 300 [*mm*]. However a thickness of 250 [*mm*] is used in calculations to account for the negative effect of low temperatures on the development of the shotcrete. The outer 50 [*mm*] of the lining is assumed not to gain its full strength.

Table 5.9 – Characteristics shotcrete tunnel lining		
Thickness lining	0.25	[*m*]
Volumetric weight	2500	[kg/m^3]
Elasticity modulus	14.25	[*GPa*]
Poison's ratio	0.15	[-]
Flexural rigidity (EI)	$37.1\cdot10^3$	[kNm^2/m]
Normal stiffness (EA)	$7.125\cdot10^6$	[kN/m]

Table 5.10 – Characteristics final station lining		
Thickness lining	0.35	[*m*]
Volumetric weight	2500	[kg/m^3]
Elasticity modulus	14.25	[*GPa*]
Poison's ratio	0.15	[-]
Flexural rigidity (EI)	$101.8\cdot10^3$	[kNm^2/m]
Normal stiffness (EA)	$9.925\cdot10^6$	[kN/m]

Characteristics of the column inside the station

The column is inserted in Plaxis as an node-to-node anchor. The normal stiffness should be entered for the anchor as a force per anchor. For Plaxis to be able to calculate the force per unit width also the out-of-plane spacing distance needs to be entered. To start with a column of 1.5m by 1.5m is assumed to calculate the normal stiffness.

Table 5.11– Characteristics column inside the station		
Volumetric weight	2500	[kg/m^3]
Elasticity modulus	15	[*GPa*]
Normal stiffness (EA)	$33.75\cdot10^6$	[*kN*]
$L_{spacing}$	7	[*m*]

Characteristics of the adjacent building

The building adjacent to the station had a width of 10 [*m*] and is founded on the second sand layer, as is common for most of the buildings in the historic centre of Amsterdam. The bearing piles of the building are modelled using a combination of anchors (pile) and plates (pile toe). The combination of plates and anchors is applied to take into account only a small part of the bearing capacity is the result of skin friction. The building it self is modelled using a stiff plate.

Table 5.12 – Characteristics pile toe	
Flexural rigidity (EI)	$8\cdot10^3$ [kNm^2/m]
Normal stiffness (EA)	$2\cdot10^6$ [kN/m]
Poison's ratio	0.2 [-]
Weight	2.0 [$kN/m/m$]

Table 5.13 – Characteristics pile	
Normal stiffness (EA)	$9.925\cdot10^6$ [kN/m]
$L_{spacing}$	1 [*m*]

Table 5.14 – Characteristics building	
Flexural rigidity (EI)	$1\cdot10^{10}$ [kNm^2/m]
Normal stiffness (EA)	$1\cdot10^{10}$ [kN/m]
Poison's ratio	0 [-]
Weight	25.0 [$kN/m/m$]

5.4.3. NATM Excavation

To excavate the station the New Austrian Tunnelling Method (NATM) will be used. NATM uses sequential excavation and support to construct a tunnel. Fowell & Karakuş (2004) combined the characteristics of NATM stated by various researchers before into to the following major principles:

- The strength of the soil or rock surrounding the tunnel is the main load bearing component.
- A thin semi-flexible sprayed concrete lining (fibre-reinforced shotcrete) is used to as a primary support system. In most cases permanent support systems are constructed in a later stage as a cast in place lining.
- The time of closure of a shotcrete ring should depend on the soil or rock conditions. A short closing time will lead to higher structural forces in the shotcrete, a long closing period will lead to larger deformations of the surrounding soil. Deformations must not be inhibited to such an extend that it leads to unduly large loads on the support system. In soft soil however with a short stand up time closing the invert should be done as soon as possible.
- Laboratory tests and close monitoring of the deformations of supporting structures and the ground itself should be performed.
- The length of the unsupported excavation should be as small as possible.

The cross section of the station will be excavated in several steps to ensure a stable situation and keep settlements to a minimum. The entire cross section of the station is located in the non-cohesive second sand layer. As most of the soil will not be frozen within this cross section all excavations will be done with an inclined excavation front to overcome internal instability problems. Start of the excavation will be at the centre part of the station and this will be done in two steps, first the top, then the bottom.

The side parts will be excavated each in five steps, starting at the inside of the bored tunnel. Tunnel segments will be removed piece by piece and shotcrete is applied as soon as a step is excavated. Eventually the shotcrete between the side and centre part will be demolished. The construction phasing that will be used at station Museumsinsel in Berlin in shown in Figure 5.16. Such an order of excavation is also intended for this design.

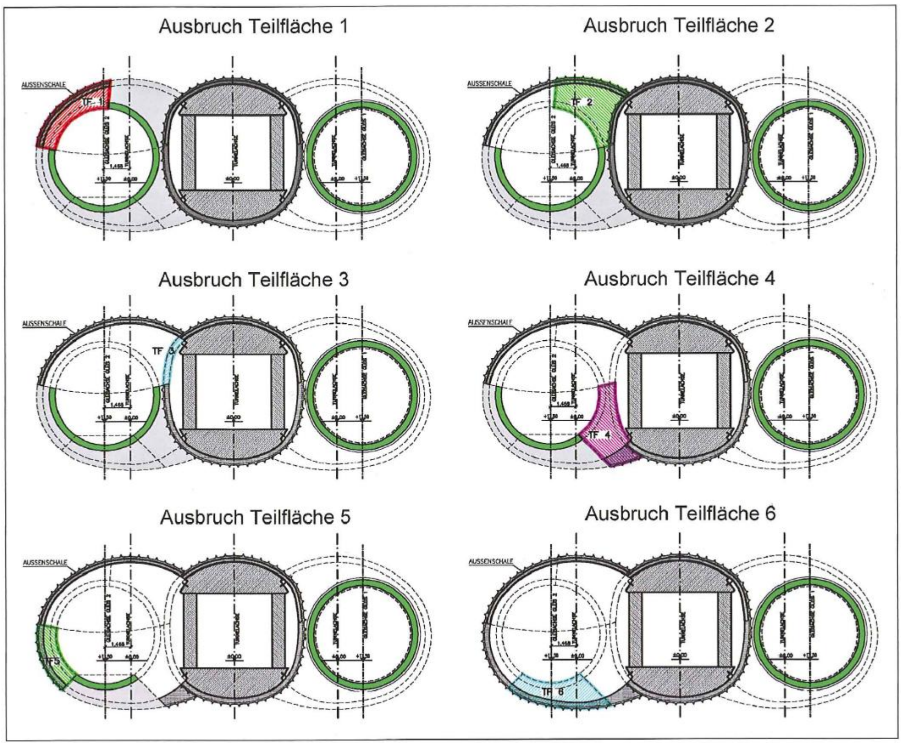


Figure 5.16 – Excavation sequence station Museumsinsel, Berlin

To model this sequence of excavation in Plaxis some concessions have to be made. If excavation step 1 (upper half of the centre part) is modelled the calculation reaches a collapse of the soil body. Location of the collapse is the unfrozen soil inside the frozen soil body, close to the already excavated soil. This soil shifts into the just excavated space, as can be seen in Figure 5.17. The largest deformation (indicated with the red colour) has a value of 160 [*mm*].

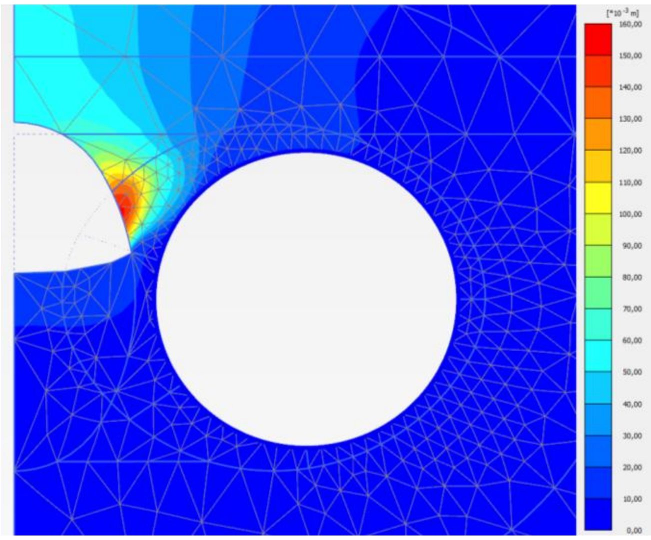


Figure 5.17 – Internal instability due to cohesionless soil

To be able to calculate all phases a small amount of cohesion is assigned to the soil inside the frozen body. As this type of failure does not lead to the failure of the frozen soil body this is purely to make the calculation with Plaxis possible.

Besides it is not unlikely that the soil at the inside of the frozen soil body starts to freeze also, and therefore gains some cohesion. The minimum cohesion that had to be applied to continue the calculation turned out to be 4 [*kPa*]. In reality due to the freezing effect the cohesion is probably higher.

The applied amount of cohesion influences the settlements of the frozen body. A larger value of cohesion leads to less settlements overall. The soil clusters with the applied cohesion then gain a bearing capacity. As the value of cohesion that is actually present is not known as a start the minimum value for cohesion is applied to make calculation possible and be on the safe side.

One of the emergency exits of the North South line, located at the 1e Jacob van Campenstraat, will be constructed using artificial ground freezing. The unsupported excavation length used for the emergency exit will be 2.0 [*m*], conform regulations of Saturn, the executive partnership at the North South Line consisting of Dura Vermeer and Züblin.

The NATM excavation steps will be modelled with Plaxis 2D. A plane strain analysis is used, an analysis suitable for situations for which the cross section is uniform over a large length. A plane strain analysis gives resulting forces of a prescribed displacement per unit of width in the out-of-plane direction. When the excavation steps are modelled soil clusters are switched off piece by piece. A manner of excavation will be sought that leads to a stable situation for all steps and thus results in an infinite unsupported excavation length in reality. If a stable way of excavation can be found with Plaxis without accounting for 3D effects, the excavation length can be chosen primary based on what is practical execution wise. 6 meter is chosen. Required space for the excavation equipment is 4 [*m*] plus 2 [*m*] to be able to apply shotcrete to a larger amount of surface at once.

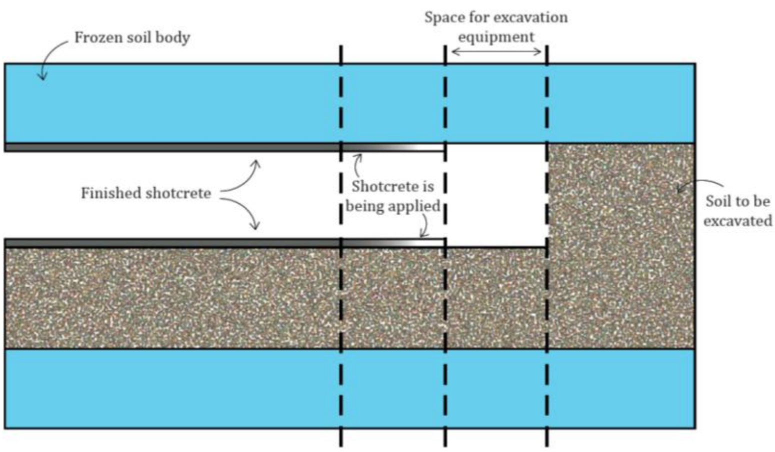


Figure 5.18 – NATM excavation in longitudinal direction

5.4.4. Thickness of the Frozen Soil Body

Excavation will take place in several steps to ensure the stability of the frozen soil body, the stability of the soil inside frozen soil body and keep settlements to a minimum. If the excavation goes according to plan the entire cross section will never be totally excavated but not unsupported. It might happen that one part (centre or side) is unsupported for a short period of time, though this is not intended when construction was planned. To prevent problems from occurring the frozen soil body therefore needs to be able to withstand earth and water pressures, when the centre (or middle part) of the station is excavated without support.

With Plaxis the minimum thickness of the frozen soil body is determined by decreasing the thickness en calculating of the structure was stable. This has lead to the results shown in Table 5.15, for excavation of the centre part. Excavating the side part leads to less settlements than excavating the centre part. Logically it follows that when the thickness of the frozen soil body decreases the settlements increase, up to a thickness for which frozen soil body collapses. First the thickness was modelled using the soil properties of soil frozen for 24h. When it was determined a thickness of 2.0 [*m*] was the minimum required thickness it was also tested if the soil body was stable with soil properties of 8 weeks. This proved to be the case.

Thickness [*m*]	Settlements top of tunnel due to excavation centre part [*mm*]	Settlements on ground level due to excavation centre part [*mm*]
3.0 (24h)	62.2	29.3
2.5 (24h)	72.9	33.1
2.0 (24h)	87.8	37.7
2.0 (8w)	118.5	47.1
1.5 (24h)	Frozen soil body collapses	

Table 5.15 - Settlements due to excavation centre part only

The deflection of the frozen soil body can be verified by a simple analytical calculation. The frozen soil body can be schematised as a beam fixed at both ends, with an uniform distributed load on top. The deflection in the middle of a beam can then be determined using the following equation:

$$w = \frac{1}{384}\frac{\sigma_{v,tot} \cdot l^4}{EI}$$

Young's modulus of elasticity after 24 hours is set at 137 [MN/m^2], after 8 weeks the value for E is set at 115 [MN/m^2]. As the 'beam' consists of both the Sand layers and the Allerod layer, but mainly the Allerod layer the values of E of the Allerod are used. The soil on top acts as an uniform distributed load of 270.84 [kN/m^2], in 2D 270.84 [kN/m] per meter of width in the out-of-plane direction. The settlements in Table 5.16 are determined with the length of the beam set at 13 [*m*].

Thickness [*m*]	Empirical calculated deflection (fixed ends) [*mm*]
3.0 (24h)	65.4
2.0 (8w)	262.8

Table 5.16 - Settlements calculated with an analytical calculation

These settlements should correspond with the settlements of the top of the tunnel due to excavation of the centre part calculated by Plaxis. For the frozen soil body of 3 [*m*] thick with soil parameters of 24 hours the analytical result matches the numerical result well. However the results for the soil body of 2 [*m*] thick with parameters of 8 weeks do not match, more than a factor two is between those results. Partly this can be due to stretching of the frozen soil body due to the deflection. Soil arching could also be part of the explanation, as is arching of the body which is due to the arch shape more stable than a beam.

Anderson & Ladanyi (2004) provide guidelines to design a frozen earth wall based on time-dependent strength by means of a hand calculation first written down by Sanger & Sayles (1979). The guidelines are meant for a circular vertical shaft. The equations below lead to the ratio of inner wall diameter (a) and outer wall diameter (b) of the circular frozen soil body. For soils with an internal friction angle the formula to be used is a little different than for soils with no internal friction angle. Both formulas are displayed here.

$$\frac{b}{a} = \left(\frac{\sigma_{tot}+H}{H}\right)^{1/\left(N_{\varphi}-1\right)} \qquad \text{for } \phi>0$$

$$\frac{b}{a} = \exp\left(\frac{\sigma_{tot}}{2c(t,\varphi)}\right) \qquad \text{for } \phi=0$$

In the formulas the symbols σ_{tot}, c, t and θ represent the following parameters:

σ_{tot}	= soil and water pressure acting on the freeze wall from the outside [kPa]
c	= cohesion [kPa]
t	= time, strength parameters are time-dependent
θ	= temperature, strength parameters are temperature-dependent

These formulas hold for a vertical circular shaft as stated before. The frozen soil body for this case will be horizontal and non-circular. Pressure on the frozen soil body will therefore be not uniform around the circumference. Where the largest value of pressure on the tunnel is exerted depends on K_0. As the soil is frozen mainly in the sand where K_0 is near 0.5 it is assumed the maximum shear force is exerted on top of the tunnel frozen soil body.

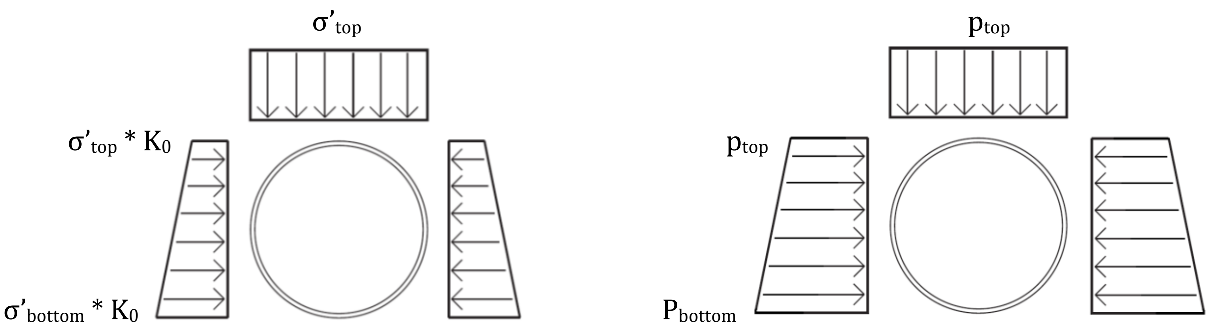


Figure 5.19 – Soil and water pressures acting on the tunnel

To determine the pressure on top of the tunnel the volumetric weights of the several soil layers in the soil profile are necessary. These are written down in Table 5.17. The values shown are the upper boundaries of the volumetric weights. The location of the ground water table, NAP -0.5 [m], is also of importance. The top of tunnel is located at NAP -16.9 [m].

Top layer relative to NAP [m]	**Bottom layer relative to NAP [m]**	**Layer**	**Volumetric weight dry [kN/m³]**	**Volumetric weight wet [kN/m³]**
+1.0	-4.5	Backfill	11.3	15.6
-4.5	-9.8	Trench fill	5.1	12.4
-9.8	-12	Mudflat deposit, Hydrobia clay	9.4	15.5
-12	-12.5	Peat	5.0	12.3
-12.5	-14.8	1st Sand layer	17.0	19.9
-14.8	-16.5	Alleröd	14.7	18.7
-16.5	-31	2nd Sand layer	16.0	19.1
-31	-40	Marine Eem clay (zone 1)	13.2	17.9
-40	-43	Marine Eem clay (zone 4)	11.7	16.9
-43	-43.5	Layer of Harting	8.6	14.7
-43.5	-51	Glacial Drenthe clay	16.0	19.8
-51	-53	Glacial Warven clay	15.2	18.7
-53	-100	3rd Sand layer	17.3	19.9

Figure 5.17 – Dry and wet volumetric weight of the soil layers

Water pressure at depth NAP -16.9 [m]:

$$p_W = \gamma_W \cdot h$$

$$\Rightarrow \quad p_W = 10 \cdot \left(-0.5 - -16.9\right) = 164 \; [kN/m^2]$$

Effective vertical stress at depth NAP -16.9 [m]:

$$\sigma_v' = \gamma' \cdot h$$

$$\Rightarrow \quad \sigma_v' = 11.3 \cdot 1.5 + 5.6 \cdot 4.0 + 2.4 \cdot 5.3 + 5.5 \cdot 2.2 + 9.9 \cdot 2.3 + 8.7 \cdot 1.7 + 9.1 \cdot 0.4 = 105.37 \; [kN/m^2]$$

Total vertical stress at depth NAP -16.9 [m]:

$$\sigma_{v,tot} = p_w + \sigma_v'$$

$$\Rightarrow \quad \sigma_{v,tot} = p_w + \sigma_v' = 164 + 105.37 = 269.37 \; [kN/m^2]$$

The following calculations are then made to derive the ratio of inner wall diameter and outer wall diameter:

$$H = c\left(t,\theta\right) \cdot \cot\varphi$$

$$\Rightarrow \quad H = 330 \cdot \cot 25 \approx 707.69 \; [kPa]$$

H is calculated using the long-term cohesion strength and friction angle at a temperature of -10 [°C]. Both parameters are time and temperature dependent.

$$N_\varphi = \frac{1+\sin\varphi(t,\theta)}{1-\sin\varphi(t,\theta)}$$

$$\Rightarrow \quad N_\varphi = \frac{1+\sin\left(25\right)}{1-\sin\left(25\right)} \approx 2.46 \; [-]$$

N_φ is called the flow value. φ depends on time and temperature and the long-term value is entered, 25°.

$$\frac{b}{a} = \left(\frac{\sigma_{tot}+H}{H}\right)^{1/\left(N_\varphi - 1\right)}$$

$$\Rightarrow \quad \frac{b}{a} = \left(\frac{269.37+707.69}{707.69}\right)^{1/\left(1.46\right)} \approx 1.25 \; [-]$$

The ratio b over a can now be calculated using the formula for soils with and internal friction angle.

With an inner diameter of 7.5 [*m*] of the NATM excavated side part, the thickness of the frozen soil body should be at least 1.88 [*m*] according to these guidelines. As the NATM tunnel is not circular this value should be seen as estimation. The analysis with Plaxis however confirms this minimum thickness.

Several other formulas can be found in literature to determine the needed wall thickness. First Lamé and Clapeyron (1833) proposed a simple formula using only elastic behaviour and overestimating the minimum wall thickness. Then Domke (1915) included plastic behaviour in the formula and Klein (1981) also included the internal friction angle as one of the input parameters for the formula. The frozen soil is assumed to behave elastically from the circle of freeze pipes inwards and assumed to show plastic behaviour from the circle of freeze pipes outwards. The following equation was derived by Klein:

$$\frac{d}{a} = \left(0.29 + 1.42 \cdot \sin\varphi\right) \cdot \left(\frac{\sigma_{tot}}{q}\right) + \left(2.30 - 4.60 \cdot \sin\varphi\right) \cdot \left(\frac{\sigma_{tot}}{q}\right)^2$$

In the equation the symbols d and q represent the following parameters:

d	= thickness of the freeze wall	[*m*]
q	= unconfined compressive strength	[*kPa*]

CDM also determined the unconfined compressive strength. For the sand layer this parameter is measured at 1100 [*kPa*] after a standup time of 8 weeks. With an internal diameter of 7.5 [*m*] the minimal thickness should be then 1.79 [*m*] according to this formula. For this estimation also yields that it is supposed to be for a circular shaft. The analysis with Plaxis however does not lead to an entire different minimum thickness, but is very close to this result. It could be concluded after two hand calculations and a Plaxis analysis suggest that the minimal thickness of the frozen body should be 2.0 [*m*]. The calculations are based on characteristic soil properties but no safety factor is yet taken into account.

When a safety calculation is performed with Plaxis the friction angle and cohesion are lowered stepwise to determine the value of safety for the calculation phase (commonly known as phi-c reduction). As the amount of cohesion inside the frozen soil body chosen is based on the being able to continue the calculation failure will occur immediately when the cohesion is lowered with a factor. To determine the safety factor of the frozen soil body therefore manually the friction angle and cohesion of the frozen soil body only are lowered stepwise. The critical phase is the excavation of the centre part. Performing a safety calculation on this phase has lead to the following safety factors:

Thickness [m]	**Factor of safety based characteristic soil properties**
2.0 (8w)	1.1
3.0 (8w)	1.6
3.3 (8w)	2.0
3.75 (8w)	2.5

Table 5.18 – Factor safety achieve for a certain thickness of the frozen soil body

Saturn, the executive partnership at the North South Line, prescribes a minimum factor of safety of 2.0 [*m*]. According to this safety philosophy the thickness of the frozen soil body should be 3.3 [*m*].

For the stepwise excavation also a safety analysis is done. It is found that for a thickness of 3.0 [*m*] the soil body has a factor of safety of 2.0. Under the condition that it may never occur that the centre part of the station is completely excavated without any support (shotcrete) the design thickness of the frozen soil body is kept at 3.0 [*m*]. There should be no problems meeting this condition. The thickness of the frozen body can be measured quite accurately. More information about this execution aspect is written in paragraph 5.6.

5.4.5. Cross section station

In the previous paragraph the minimum thickness of the frozen soil body was determined to be at least 3.0 [*m*]. In a row of freeze pipes the pipes are located with a centre to centre distance of 1.0 [*m*]. The distance between the rows is 1.5 [*m*]. The required amount of freeze pipes with these mutual distances is 108. 28 freeze pipes will be placed for each of the curved side parts of the frozen body. In the fairly straight top and bottom part of the frozen body 26 pipes each will be placed. The location of the freeze pipes in the cross section is indicated in Figure 5.20 with black dots. In the out-of-plane direction the freezing will take place over a length of 110 [*m*]. As the freeze pipes will be installed from two sides towards each other the length of these freeze pipes will vary between 55 and 60 [*m*] and the total amount of freeze pipes is 216.

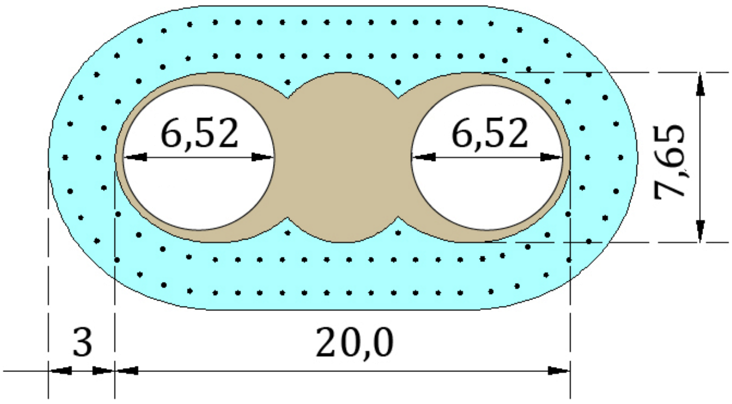


Figure 5.20 – Cross section of the frozen soil body with indicated the location of freeze pipes (lengths in [*m*])

The soil to be excavated is coloured brown in Figure 5.20. The maximum width of the excavation is 20 [*m*] and the maximum height is 7.65 [*m*]. The surface of soil in the cross section is approximately 70[m^2]. With an excavation length of 110 [*m*] the volume of soil to be excavated will be 7700 [m^3]. Part of this soil will be frozen. The intended volume of frozen soil is 20350 [m^3]. The actual volume of frozen soil will be larger.

Figure 5.21 shows the cross section of the station after the final lining is constructed. The thickness of the shotcrete lining is 250 [*mm*], the thickness of the final lining is 350 [*mm*]. At the junction of the arches columns are constructed. Construction of these columns is done after shotcrete at the centre part is applied together with the final lining of the centre part. Initially in the model columns are placed every 7 [*m*] and have a width and length of 1.5 [*m*]. The width of the platform is 9.0 [*m*], which was a preset requirement. The minimal free height is 3.0 [*m*]. Metro trains can run on both sides of the platform and have free space with height 4.3 [*m*] and width 4.0 [*m*]. The platform has a height of 0.8 [*m*] relative to the final floor of the station.

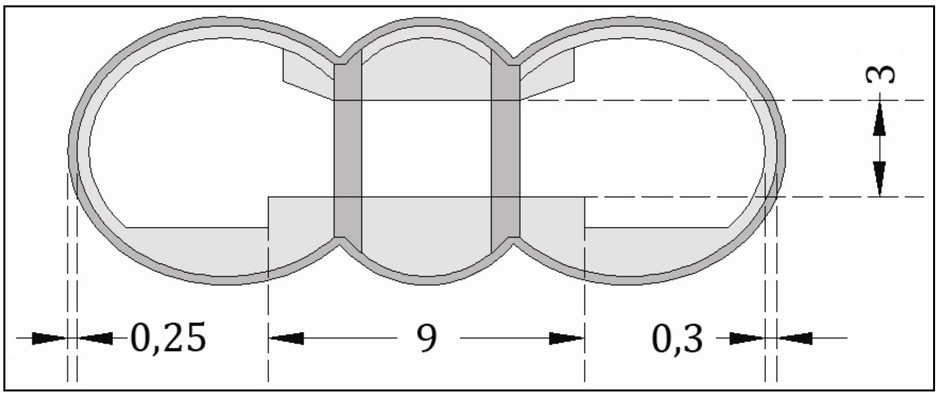


Figure 5.21 – Cross section of the station with final lining (lengths in [*m*])

5.4.6. Phasing Characteristics

The phases implemented in Plaxis are listed in this paragraph, from the initial phase (No. 0) to the final phase (No. 23) in which the station is completed an the frozen soil body is thawed. The phases actions per phase are explained and shown with a figure.

0. Initial phase (K_0 procedure)
In the initial phase initial pore pressures and initial effective stresses (horizontal and vertical) are generated using the K_0 *procedure*. The ground water level is set at NAP -0.5 [*m*]. With the K_0 *procedure* vertical effective stresses are generated using the self weight of the soil layers implemented. Horizontal effective stresses are generated using the specified value of K_0 and the vertical effective stresses. K_0, the coefficient of lateral earth pressure, is specified with Jaky's formula ($K_0 = 1 - \sin\varphi$) if the soil is normally consolidated. For overconsolidated soils K_0 is larger than the value given by Jaky's formula. K_0 is then also determined by the *OCR* and *POP* value.

1. Adjacent buildings (Plastic drained)
The geometry of the adjacent building (consisting of plates and anchors) is activated. A *plastic drained* calculation is done to calculate this phase, which means undrained behaviour is temporary ignored. In cases with excavations during undrained behaviour the effective stresses will rise temporarily. By performing a plastic calculation the long term (drained) effect modelled, and this temporary rise will be ignored. For all following phases the calculations will be performed *plastic drained*.

2. Excavation bored tunnel (Plastic drained)
The excavation of the bored tunnel is modelled by deactivating the soil inside the tunnel and activating the surrounding plate. The plate has the material parameter set of the tunnel shield. In the water conditions tab the cluster inside the bored tunnel is set dry.

3. Contraction bored tunnel (Plastic drained)
As the shield is larger than the actual tunnel diameter there is overcutting. The overcutting leads to a void in the soil. This is modelled by a contraction of the tunnel of 0.5 [%], which corresponds to an overcutting of 8cm.

4. Tail void grouting bored tunnel (Plastic drained)
The void caused by overcutting during the excavation process is injected with grout fill the space between lining and soil and to prevent large settlements from occurring. The grout is a fluid when injected and cures in time. This event is modelled by deactivating the tunnel lining and applying a predefined pore pressure of -280 [kN/m^2] to the cluster inside the tunnel lining. 280 [kN/m^2] is the in-situ pore pressure at some distance of the tunnel at the same depth.

5. Tunnel lining bored tunnel (Plastic drained)
To model the final phase of the bored tunnel the lining is activated again and addressed the material parameter set of a concrete lining. The interface parameter R_{inter} is set at 1 in this set, due to the tail void grouting. In the water conditions tab the cluster inside the bored tunnel is set dry again.

6. Freezing soil body (Plastic drained)
Installing freeze pipes and turning on the refrigerating system will lead to the freezing of the soil. In this phase the final frozen soil body needed to start excavation is modelled by changing the material parameters sets of several clusters of soil into sets for frozen soil. The development of the soil body to a frozen soil body is thus not modelled, it is 'wished in place'. The initial (24h) properties of frozen soil are used. The drainage type of all frozen soil types are set to non-porous. In the water conditions tab the frozen soil body is not set dry to obtain correct stresses at its boundary. A volume strain increment is applied of 1 [%].

7. Removing pore water inside the frozen body (Plastic drained)

Before excavation can start the unfrozen soil inside the frozen soil body is set dry. This is done performing relaxation drillings from the two building pits. In Plaxis in the water conditions tab the soil clusters inside the frozen soil body are set dry. A volume strain increment is applied of 0.1 [%].

8. Excavation side part 1 – Frozen soil properties 24h (Plastic drained)

When the frozen soil body is closed and at the required thickness excavation can start. First the middle part (1) will be excavated. Parameters of the frozen soil of 24h will be used in this phase. In the water conditions tab the cluster inside the NATM excavated tunnel is set dry. Cohesion, 4 [*kPa*] is added to the soil inside the frozen soil body, hence the change of colour of those soil clusters. A volume strain increment is applied in this phase of 0.1 [%].

9. Shotcrete side part 1 – Frozen soil properties 2 weeks (Plastic drained)

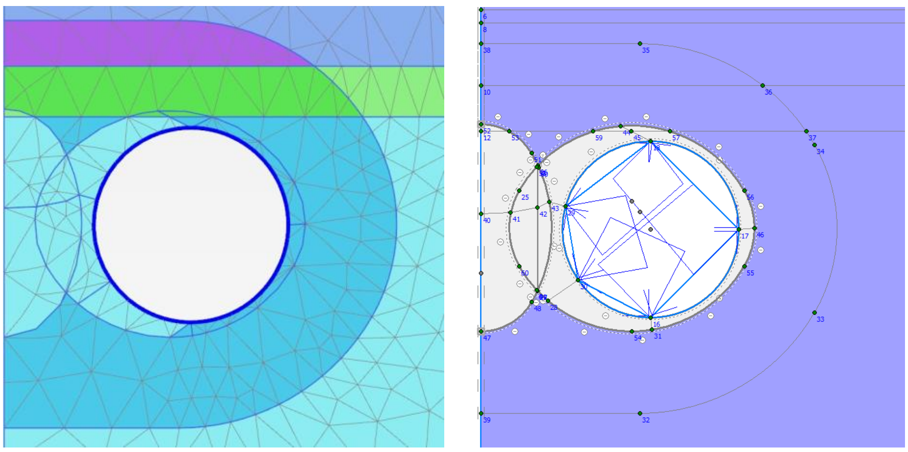
It is assumed that excavation over the entire length can take place in 2 weeks. At locations where the excavation started the tunnel will already be open for two weeks when the final excavations are done, therefore frozen soil properties of two weeks will be applied to the frozen soil body. The lining of the NATM tunnel will be activated in this phase, as the shotcrete is applied. A volume strain increment is applied in this phase of 0.1 [%].

Phases 10 up to and including 21

The principles of phases 8 and 9 will be repeated till all parts are excavated and shotcrete is applied. After the centre part is entirely excavated and shotcrete is applied, the final lining is already made. Also columns are constructed at the junction of the side and centre part. After all parts are excavated and shotcrete is applied the entire final lining can be made. The side part is excavated in 5 steps. During the construction phases the soil properties of the frozen body are changed to model creep. This can be seen by the changing of the colours of the frozen soil clusters. How the phases look in Plaxis can be seen in the figures below. In all phases a volume strain increment is applied of 0.1 [%].

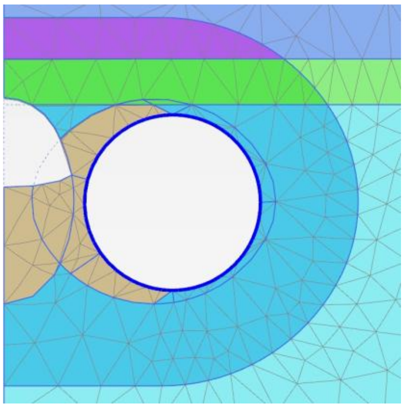
22. Completed station – Thawed soil properties (Plastic drained)

When the station the final lining is installed the refrigerating system will be turned off and the frozen soil body will start to thaw. This phase represents a total unfrozen soil body. The parameter sets of the former frozen soil body are changed into thawed parameter sets. For the coarse grained soil layers (1st and 2nd Sand layer) this means the original parameter set, for the more cohesive and less permeable layers (Allerod) this means an adjusted parameter set. A volume strain increment is applied in this phase of -3.0 [%].

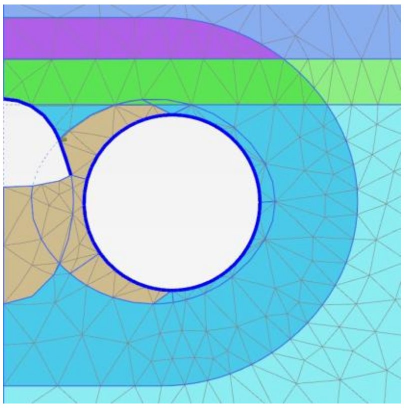


Phase 6 (Freezing soil body) *Phase 7 (Removing pore water)*

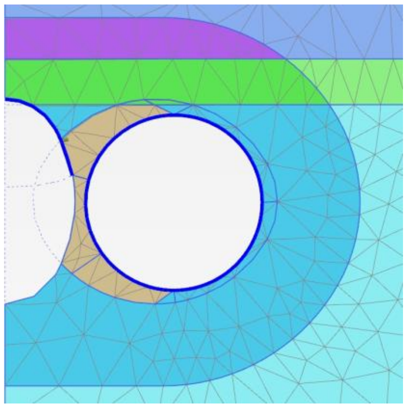
Figure 5.22 – Excavation sequence modelled in PLAXIS



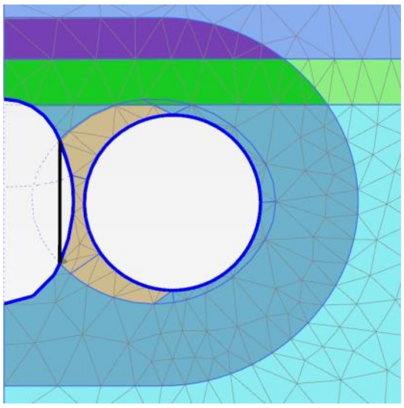
Phase 8 (Excavating centre part 1)



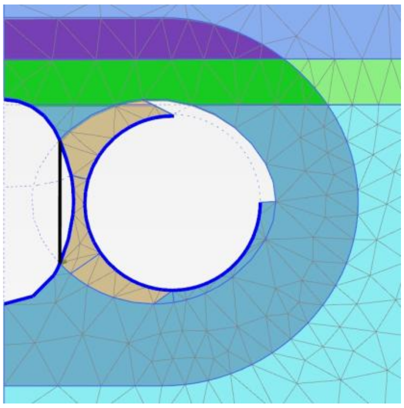
Phase 9 (Shotcrete centre part 1)



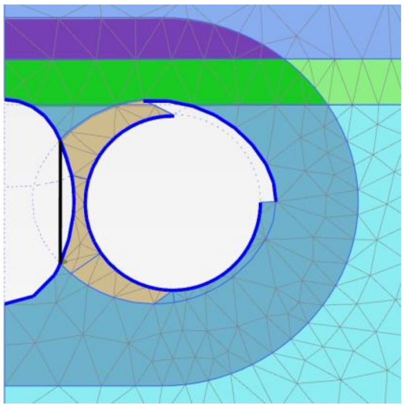
Phase 10 (Excavating centre part 2)



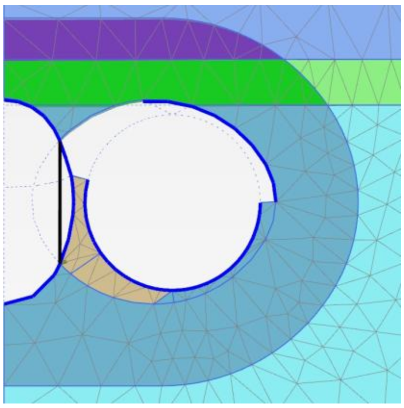
Phase 11 (Shotcrete centre part 2)



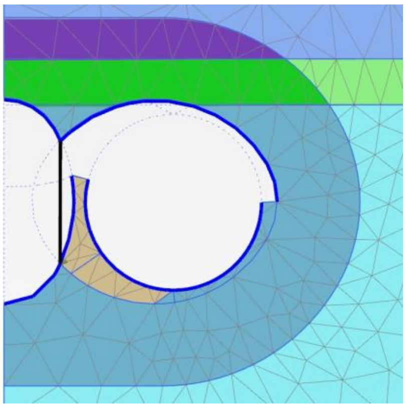
Phase 12 (Excavate side part 1)



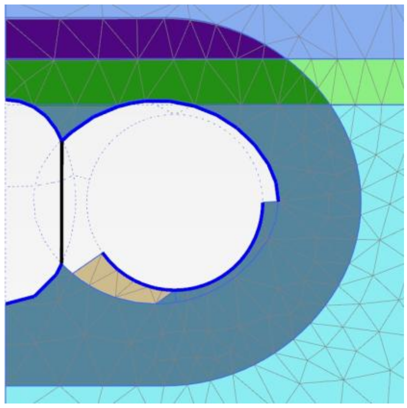
Phase 13 (Shotcrete side part 1)



Phase 14 (Excavate side part 2)

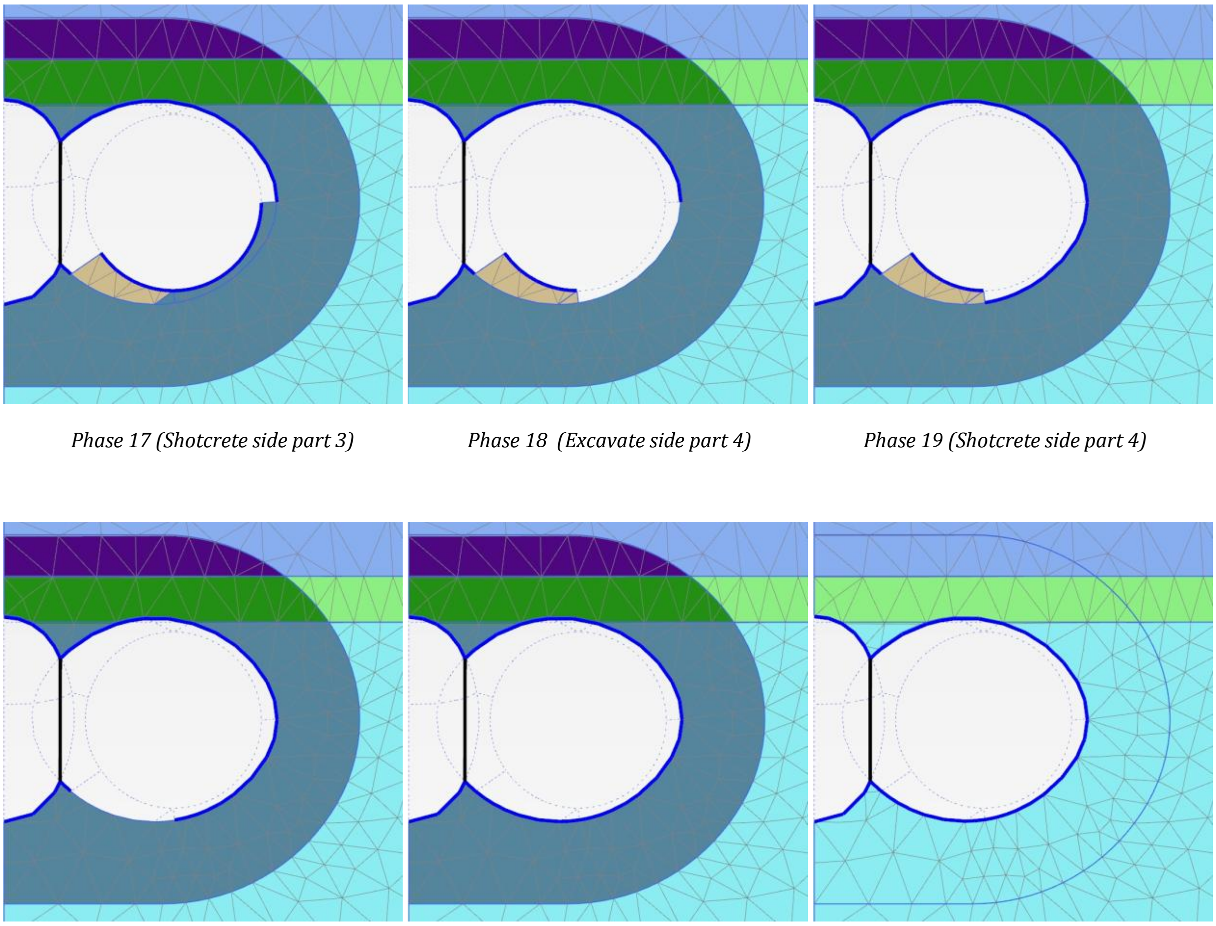


Phase 15 (shotcrete side part 2)



Phase 16 (Excavate side part 3)

Figure 5.22 – Excavation sequence modelled in PLAXIS



Phase 17 (Shotcrete side part 3) *Phase 18 (Excavate side part 4)* *Phase 19 (Shotcrete side part 4)*

Phase 20 (Excavate side part 5) *Phase 21 (Shotcrete side part 5)* *Phase 22 (Thawed soil body)*

Figure 5.22 – Excavation sequence modelled in PLAXIS

5.4.7. Accounting for the 3D arching effect in 2D

The excavation is temporary not supported due to the time span between excavation and applying shotcrete. A three dimensional arching effect will occur which is not simulated in a two dimensional model. To account for this arching effect the Load Reduction Method of Schikora & Fink (1982) may be applied.

The Load Reduction Method splits the construction of the NATM tunnel in two phases. In the first phase the excavation takes place and the lining is not yet installed. The initial acting stresses on the tunnel are in this phase divided in two parts with the help of *β.* Part of the stresses (1-*β*) will act on the unsupported tunnel and the rest (*β*) will act on the supported tunnel (tunnel lining). *B* is the load reduction factor, also called the unloading factor.

The value of *β* will depend on lining thickness, soil stiffness and strength, tunnel diameter and excavation length. Möller and Vermeer (2005) analysed the influence of these factors on *β.* In Figure 5.23 results of this analysis are shown for elasto-plastic ground. The graph is based on a NATM excavated tunnel with a diameter of 7 [*m*] and a tunnel cover of 21 [*m*]. The Young's modulus of the soil is 20 MN/m², K_0 is 0.67 and the friction angle is 20°. The graph is also valid for tunnels with a diameter of 9 [*m*] or 11 [*m*]. In the graph the influence of cohesion and excavation length on the value of *β* are shown. Case studies have been performed to prove that relationships found are representative for practical applications.

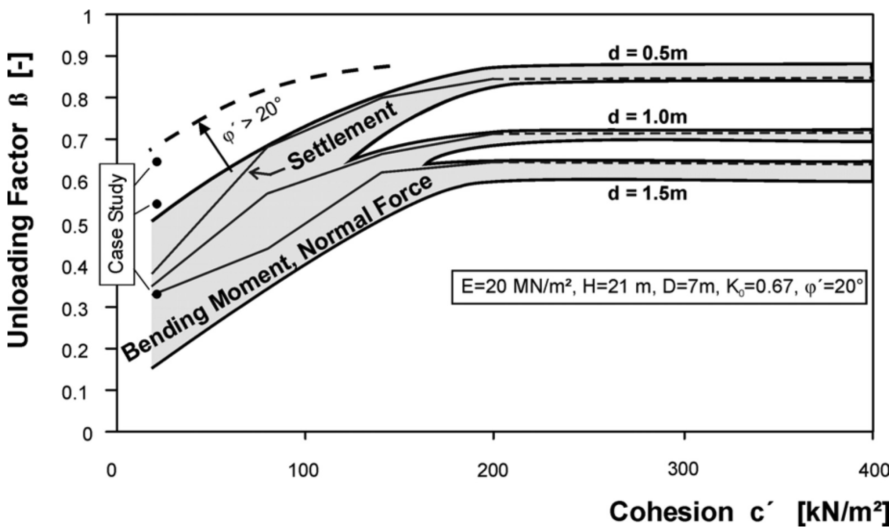


Figure 5.23 – The unloading factor related to the cohesion of a soil

For settlements, bending moments and normal forces different unloading factors have to be used to match exact three dimensional solutions. In the graph for bending moments and normal forces a bandwidth of unloading factors is given for a value of cohesion. The unloading factor for settlements is shown as a line and remains within the bandwidth of structural forces.

This effect is not taken into account in the calculations made. It is however something which could be further investigated to make less conservative calculations.

5.5. Results

In the previous paragraphs the geometry of the station was determined, as well as soil properties before, during and after thawing. In this paragraph the results of calculations using these data are shown and discussed.

5.5.1. Results First Model

A model with phasing characteristics as defined in 5.4.5. resulted in the outcome presented in this paragraph. A stiffness decrease of 5 [%] is applied in this calculation to the soil properties of the Allerod layer after the freeze-thaw process. Other options to model the Allerod layer after thawing are further elaborated in the next paragraph. Figure 5.22 shows the total vertical settlements after completion of the station and thawing of the previously frozen soil body.

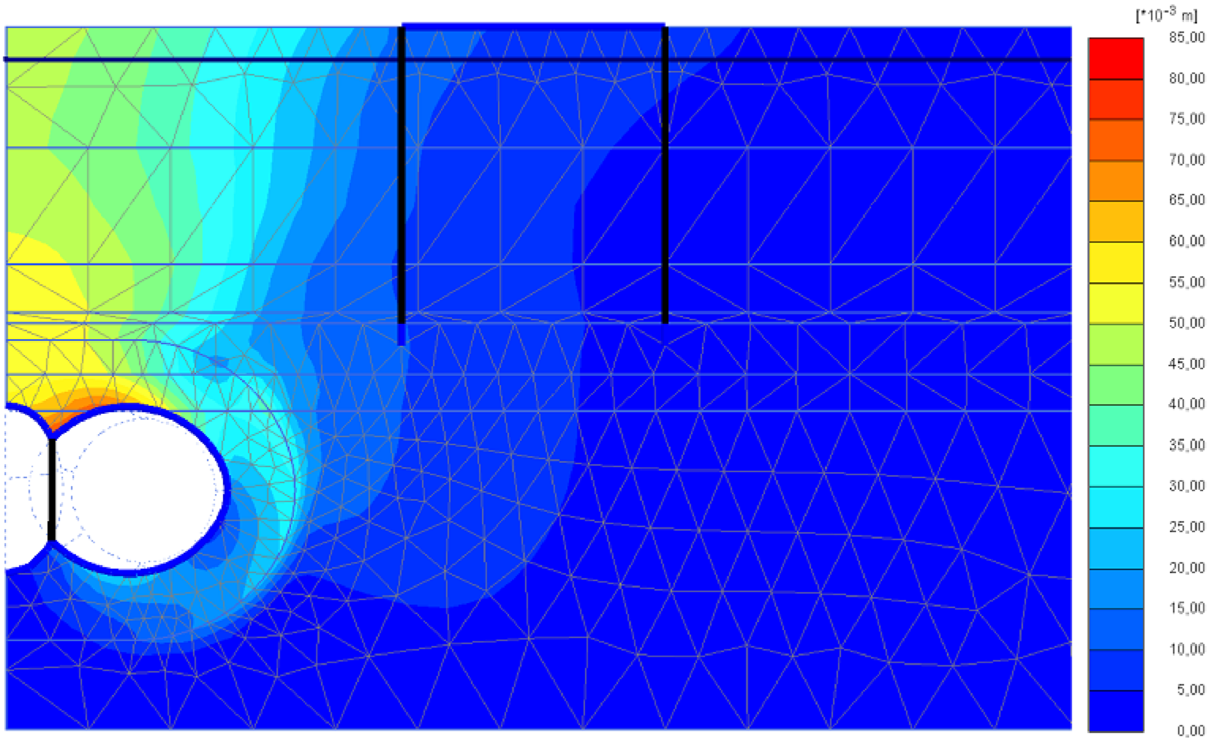


Figure 5.22 - Total settlements after completion station.

In paragraph 5.4. it was stated criteria have to be met concerning deformations and structural forces. Relevant values related to the criteria are listed below. All settlements noted are calculated in the y-direction.

- Maximum settlement on ground level: 47.4 [mm]
- Settlement pile closest to the construction works – at pile tip level: 9.4 [mm]
- Settlement pile furthest from the construction works – at pile tip level: 3.7 [mm]
- Maximum moment bored tunnel lining: 507.5 [kNm/m]
- Maximum moment shotcrete station lining: 221 [kNm/m]
- Maximum normal force column 14350 [kN]

When comparing to the modelling criteria it can be seen they are not met, except for the maximum skew of 1:500 and maximum normal force in the column of $57.5 \cdot 10^3$ [kN]. The other requirements concerning maximum bending moments and maximum settlements on ground level are exceeded.

In the design a frozen soil body was modelled surrounding only the station to be constructed. No intermediate frozen columns were modelled. As most preset requirements are not met these columns seem necessary. Most settlements are induced by deflection of the frozen soil body due to excavation of the top of the centre part. The soil between the centre part and side part is modelled as unfrozen soil and therefore does not support the frozen soil body to a large extent. The calculated settlements in the model due to this excavation are shown in the Table 5.19 and Figure 5.23.

Excavation step	**Phase settlement u_y [mm]**
Centre part 1	-32.0
Centre part 2	-8.2
Side part 1	-15.7
Side part 2	-1.0
Side part 3	-0.3
Side part 4	1.3
Side part 5	1.6

Table 5.19 – Vertical phase settlements at the top of the frozen soil body

Table 5.19 shows the largest settlements of the top of the frozen soil body occur due to excavation of centre part 1. Negative values are downward displacements and positive values are upward displacements. It can be seen that the phases in which the excavation of side parts 4 and 5 takes place leads to an upward deformation of the soil. This upward movement can be explained by the effect of heaving of the Allerod layer. Based on research by Deltares and information found in literature (Akawaga, 1988) a volume strain is implied in Plaxis on the Allerod for all phases where the layer is frozen. In the phases in which the excavation of side parts 4 and 5 takes place the heave is larger than the deflection of the soil body.

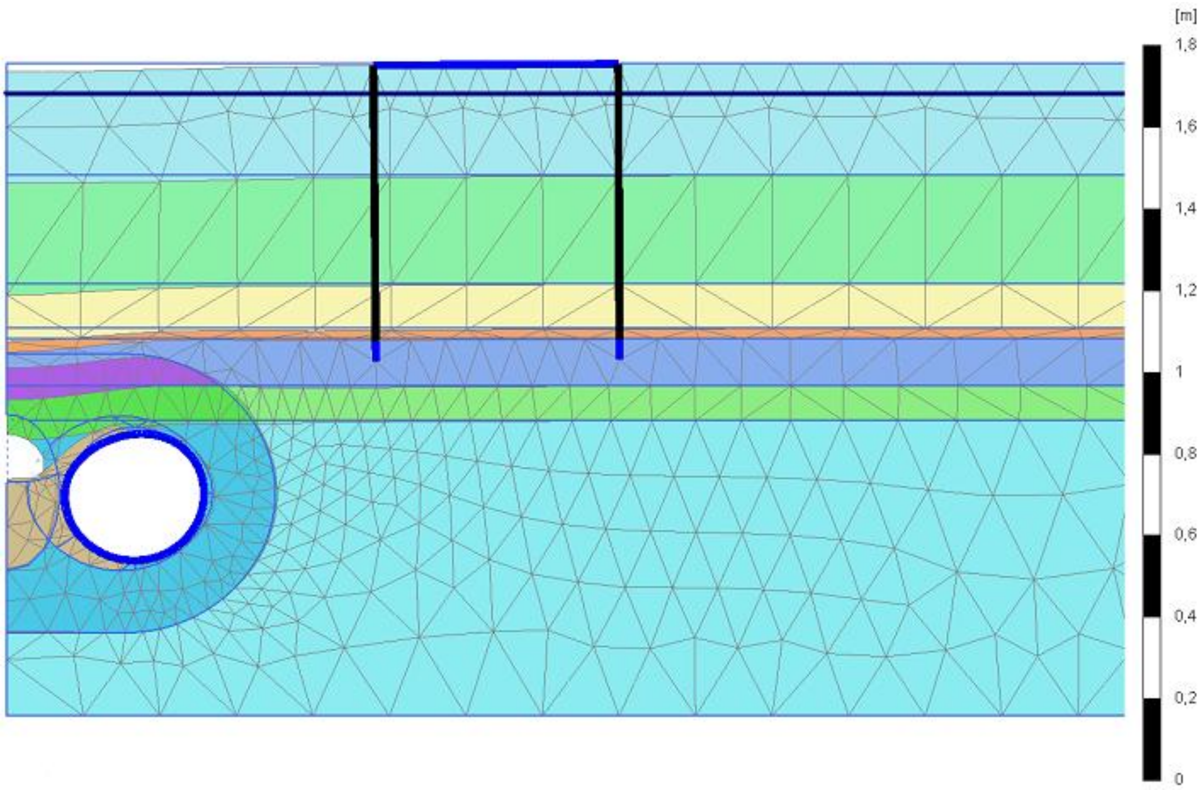


Figure 5.23 – Deformed mesh phase 8 (Excavation centre part 1)

The deformed mesh after calculation of phase 8, Figure 5.23, shows the unfrozen soil within frozen soil body (brown coloured soil clusters) does not give a large amount of support to the frozen soil. The unfrozen soil is compressed and pushed into the excavated space. The direction of the displacements is also shown in Figure 5.24.

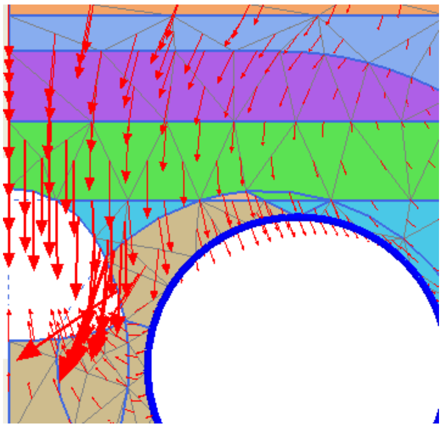


Figure 5.24 – Direction phase displacements phase 8

Furthermore this excavation step, due to the large deflection of the frozen soil body, causes too large bending moments in the bored tunnel lining. A bending moment of 507.5 $[kNm/m]$ occurs where a maximum of 330 $[kNm/m]$ is defined. Bending moments calculated in phase 8 are shown in Figure 5.25.

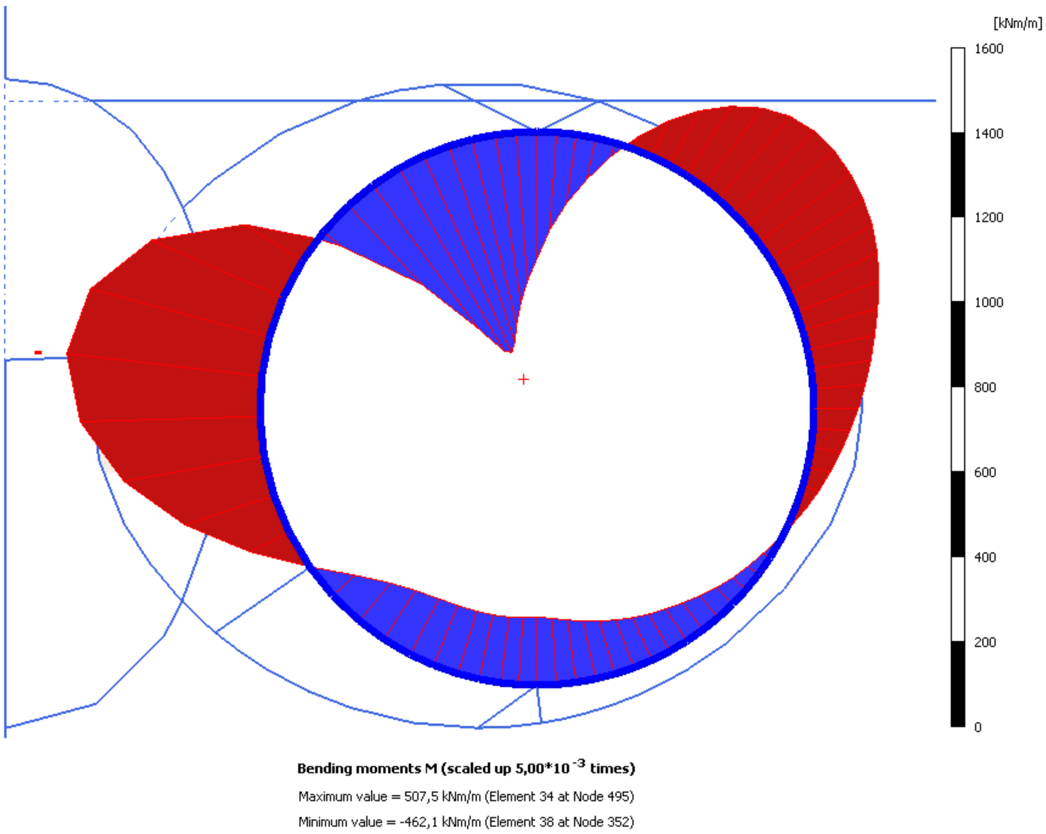


Figure 5.25 – Bending moments bored tunnel lining (Excavation centre part 1)

When looking at the outcome of the Plaxis calculation it can be concluded this alternative design using artificial freezing would not be feasible, mainly due to large deformations. To decrease deformations a freeze wall is implemented in an adjusted model. The results of this model are elaborated in the next paragraph.

5.5.2. Results second model

The deflection of the frozen soil body will decrease if the length of the span is decreased. As a result of the unfrozen modelled soil within the frozen soil body the length of the unsupported span is about 13 [*m*] in the first model. Based on experience using artificial freezing in Amsterdam (North/South Line) and in Rotterdam (subway station Rotterdam CS) it is very likely more soil is frozen than was intended to freeze. This is caused by continuous activity of the refrigerator system to ensure a required thickness of the frozen body. Especially in sands the frozen soil body will grow fast.

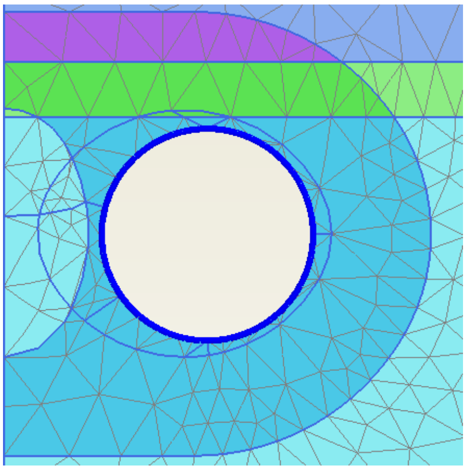


Figure 5.26 – Frozen soil body second model

Taking into account the previous findings, the second model assumes the soil in between the centre and side part is frozen. When executing this design evidently it must be verified. This will be done using temperature sensors. In Figure 5.26 it is shown to which clusters in this model frozen soil parameters are assigned. Dark blue clusters are the frozen 2nd Sand layer. The smallest thickness of the frozen wall is 0.6 [*m*]. The boundary on the left side is the soil to be excavated for the centre part. When executing it is likely the thickness will be larger.

Maximum settlements on ground level are reduced from 47.4 [*mm*] to 14.6 [*mm*] by implementing the frozen wall into the model. Other results in relation to the preset requirements are:

▪ Maximum settlement on ground level:	14.6	[*mm*]
▪ Settlement pile closest to the construction works – at pile tip level:	7.7	[*mm*]
▪ Settlement pile furthest from the construction works – at pile tip level:	3.6	[*mm*]
▪ Maximum moment bored tunnel lining:	256.8	[*kNm/m*]
▪ Maximum moment shotcrete station lining:	130.3	[*kNm/m*]
▪ Maximum normal force column	10450	[*kN*]

The second model meets all preset requirements. The application of the frozen wall causes the largest deflection of the frozen soil body to move from the centre part to the side part. Excavation of centre part 1 (phase 12) leads to a deflection of the top of the frozen soil body of 1.6 [*mm*] (and bottom 3.9 [*mm*]), instead of 32.0 [*mm*], a value which was found with the first model. The vertical total settlements after completion of the station are shown in Figure 5.27.

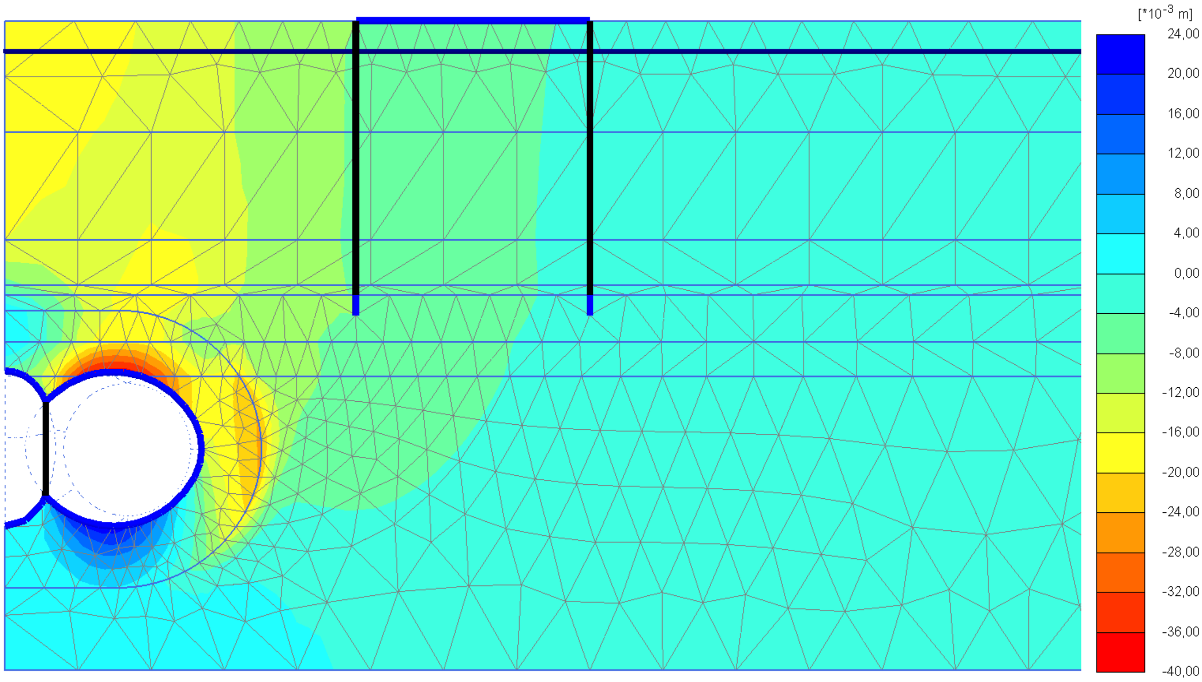


Figure 5.27- Total settlements after completion station.

When side part 2 is being excavated the freeze wall also needs to be demolished. This leads to a vertical settlement of the top of the frozen soil body of 10.2 [*mm*] and bottom 14.8 [*mm*]. In the first model this phase caused the top of the frozen soil body to settle 3.2 [*mm*] and the bottom 5.5 [*mm*]. Explanation for this dissimilarity is the difference in total displacements of the soil body up to phase 14. In the first model the soil body already deformed more, 72.3 [*mm*], than in the second model, 13.8 [*mm*]. Another explanation for the larger deformations of the side part with respect to the centre part is the larger unsupported span.

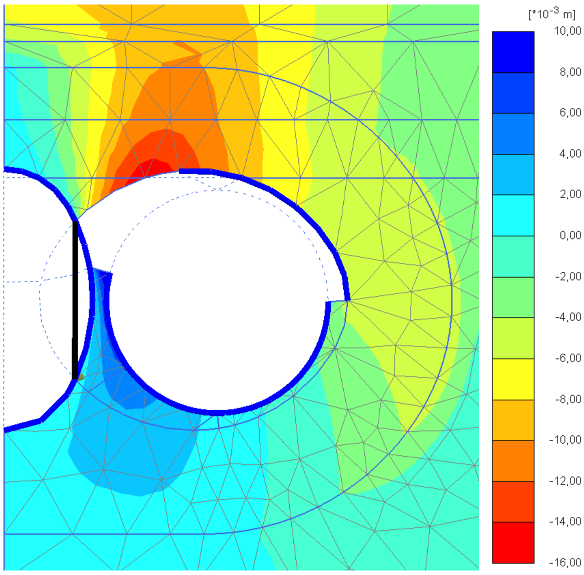


Figure 5.28 – Vertical phase settlements phase 14 (Excavation side part 2)

Explained in paragraph 5.4.1.1 is that heave is dealt with in the model by applying a strain increment in all phases where the Allerod has frozen soil parameters. A strain increment of 1 [%] is applied to the Allerod in the first phase in which the layer is frozen. Reaction of the soil is shown with Figure 5.29 and 5.30.

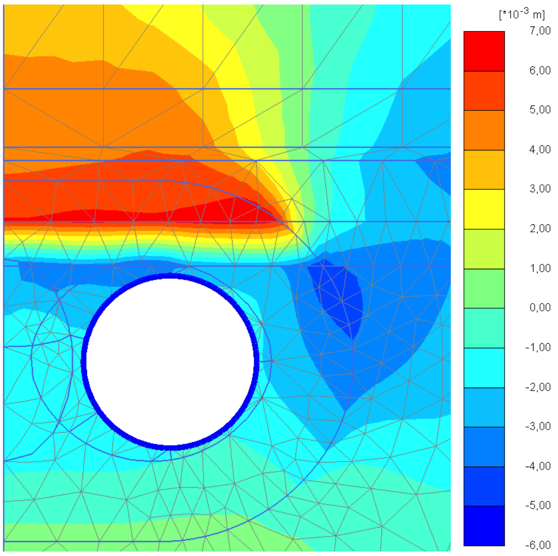


Figure 5.29 – Vertical phase settlements phase 6

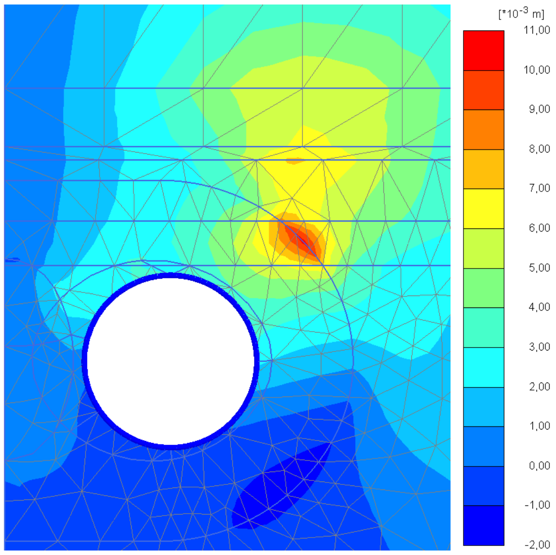


Figure 5.30 – Horizontal phase settlements phase 6

Applying a volume strain increment in Plaxis means applying a strain increment in each direction. Then Plaxis will search for an equilibrium with the surrounding soil. Due to surrounding boundaries or stiff soil it is therefore possible not the entire applied strain increment has occurred at equilibrium. In Figure 5.29 it can be seen displacements are mainly in upward direction from the Allerod layer. In that direction the soil can deform with the least resistance.

In Annex D the deformation of the mesh is shown per phase. In Figure 5.31 the total settlements in vertical direction are shown for three points in the modelled cross section. The location of these points A, B and C are shown in part of the cross section in Figure 5.32.

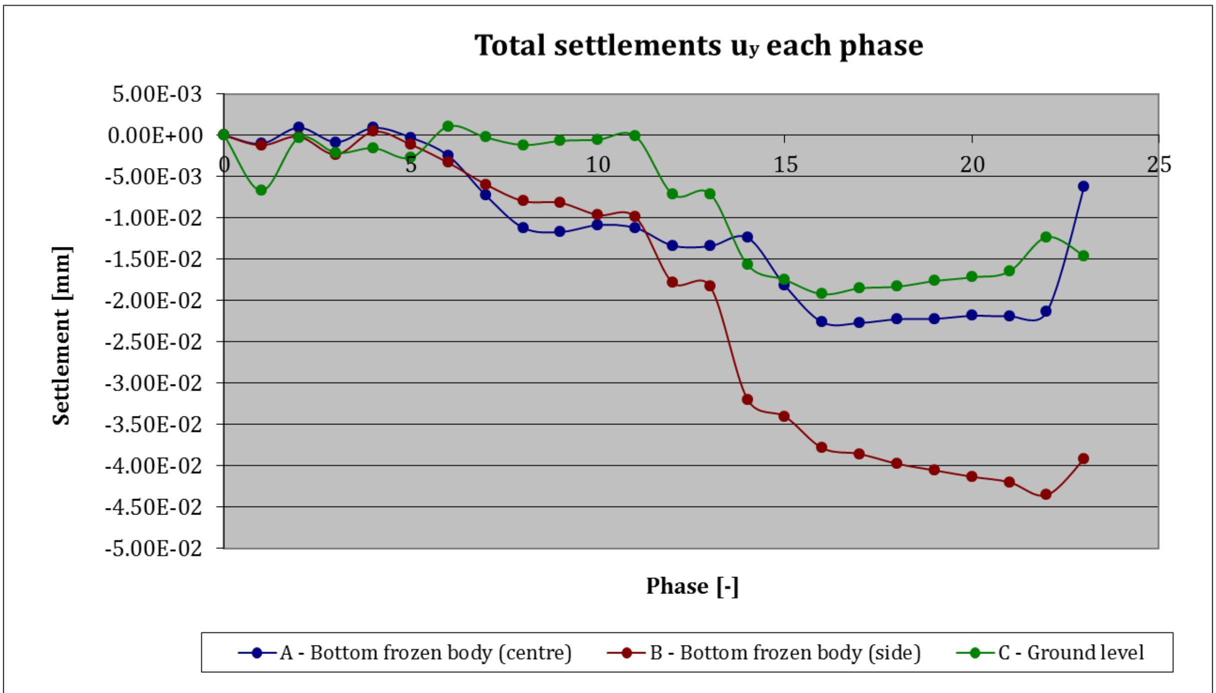


Figure 5.31 – Development of vertical total settlements for three points in the cross section

In the Plaxis simulation at the start of phase 6 (freezing soil body) displacements up to then are set to zero. Therefore the displacements from phase 6 on are only cause by constructing the station using freezing techniques. As a result the total settlements in phase 23 do not include settlements due to boring the tunnel. This is done because this study focuses on the effects of freezing on the soil and surroundings.

From the graph and the figures of the deformed meshes it can be seen phases 16 to 21 do not have a large influence on the settlements on ground level. As a result of heave, ground level even moves up. In these phases side parts 3 to 5 are being excavated and shotcrete is applied. The largest settlements have occurred in the four earlier phases, when excavation of side parts 2 and 3 took place. The cause of these larger settlements, removing the support of the frozen soil body, is already discussed in this chapter.

The final phase, thawing of the soil body, causes the top of the station to move upwards and ground level to move downwards. This can be explained by the fact that to the Allerod layer a negative volume strain is implied of 3 [%]. The lining of the station is probably deformed elastic and rebounds due to the decrease of stress. In practise this effect however is not often seen. Also the 5 [%] stiffness decrease of the Allerod parameters causes a small downward settlement of ground level.

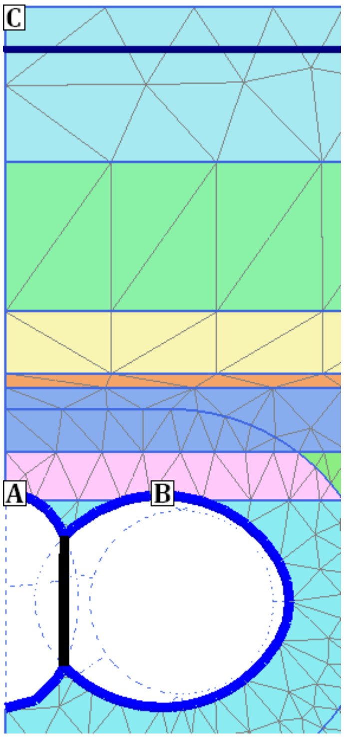


Figure 5.32 – Location calculated total settlements of Figure XX

The exact reaction of the Allerod to the freeze-thaw process is not known. In previous calculations a stiffness decrease of 5 [%] was used. If the Allerod however would react more as a clay its void ratio and permeability would be increased in the new situation, as is the compression index. The Allerod could also creep as result of the freeze-thaw process. The parameters for both situations are determined in paragraph 5.4.1.2. Results of calculations with these adjusted parameters are shown in Table 5.24, together with the previously obtained results. Only in the final calculation phase the parameters are adjusted. Therefore the maximum moments do not change with respect to the calculation using a 5 [%] decrease in stiffness.

Table 5.24 – Overview outcome different models	**Preset requirement**	**Model 1 Allerod 5%**	**Model 2a Allerod 5%**	**Model 2b Allerod clay**	**Model 2c Allerod SSC**
Maximum settlement on ground level [*mm*]	20	47.8	14.8	21.8	16.9
Settlement pile closest to the construction works – at pile tip level [*mm*]	*Maximum settlement*	9.4	7.7	8.1	7.8
Settlement pile furthest from the construction works – at pile tip level [*mm*]	*difference of 24 [mm]*	3.8	3.6	3.6	3.6
Maximum moment bored tunnel lining [*kNm/m*]	330	504.5	256.8	256.8	256.8
Maximum moment shotcrete station lining [*kNm/m*]	200	221.0	130.3	130.3	130.3
Normal force column [*kN*]	76600	10031	10450	10465	10535

In Annex D settlement graphs are shown for the three calculations made with model 2. When the Allerod is modelled with parameter changes that would occur if it acts similar to a clay, the maximum settlement on ground level increases with 1/3. Both the 5 [%] stiffness decrease calculation and the calculation with a changed e, C_C and k are made using the Hardening Soil model. The compression index C_C is calculated using the water content of the Allerod and an empirical relation to C_C set up for clays. The compression index was therefore changed in the calculations from 0.03371 to 0.1995. This change in C_C is the main cause of the increase in settlements.

The results of the Allerod modelled with the Soft Soil Creep model show smaller settlements with respect to the calculation for which the Hardening Soil model is used. Difference with the model 2b is adding time dependent (secondary) compression to the calculation. It was expected model 2c would lead to larger settlements than model 2b, but it turned out not to be the case. This cannot be explained yet.

Every calculation made, met the skew related requirement. In all these calculations the geometry of the modelled cross section did not change and the smallest horizontal distance between the frozen soil body and the left pile was 5.25 [m]. It would be interesting to know at what distance adjacent buildings should be at minimum to still satisfy the skew requirement. Model 2a is used to determine this minimum stiffness. The horizontal distance between the piles is 12 [m]. With a maximum skew of 1:500 the maximum settlement difference of both pile tips should be 0.024 [m], thus 24 [mm]. This amount of differential settlement does not occur at depth of the pile tip with a mutual distance of 12 [m] even not right above the station and not in any of the construction phases. However with a structure above the tunnel stresses and deformations would have developed differently. Therefore additional calculations should be made to verify this statement.

The columns are modelled in all calculations with a length and width of 1.5 [m] and a spacing of 7 [m]. The maximum normal force, $57.5 \cdot 10^3$ [kN], is however much larger than any of the modelled normal forces. In the final phase of model 2b a normal force in the column is calculated by Plaxis of 1495 [kN/m]. With respect to normal forces a spacing of 38 [m] would be possible, however, such a span would lead to failure of the final lining in longitudinal direction. A spacing of 15 [m] leads to an increase in settlements to 15.2 [mm] and a moment in the upper beam of 9000 [kNm]. With concrete type B100, a width of 5 [m] and a height of 0.4 [m] the upper beam will be able to withstand this moment.

5.6. Execution aspects

This paragraph discusses shortly several practical aspects of artificial soil freezing. Some principles related to installation of freeze pipes, monitoring during execution and the number of required freeze pipe rows are explained.

→ *Installing freeze pipes*

When installing a freeze pipe through a wall of a building pit it is of importance no water enters the pit. The following steps are taken when installing a freeze pipe:

1. Execution core drilling
2. Installation steel tube in the core drilling
3. Closing the core drilling with a chemical substance
4. Installation preventer (a ring with a rubber part through which the freeze pipe can be installed)
5. Drilling of freeze pipe through the preventer

Freeze pipes can be installed under an angle by directional drilling. At the Damrak in Amsterdam freeze pipes are installed using directional drilling over a length of 85 [m]. Every 3 [m] the position of the freeze pipe was checked and if necessary the direction of installation of the next 3 [m] of pipe was adjusted. The installation could be done with an accuracy of 0.5 to 1 [%].

→ *Monitoring*

Whether the frozen body is closed or not can be detected with two methods. The first method is to make ‘relief drillings’ (=ontlastingsboringen). These drillings are used to determine the pressures inside the frozen body. Inside the frozen body excess pore pressures will develop due to the growing and expanding of the frozen soil. With a relaxation drilling the excess pressures can also be lowered by the drain of pore water. It is possible a relaxation drilling is being located inside the frozen soil. In this case the closing of the frozen body can not be detected by the drilling anymore. Temperature sensors can give a better view on the development of a frozen soil body. At the Damrak four of these sensors were installed at a distance of 1 [m] to the freeze pipes. When a temperature of -2 [°C] was reached the soil body was considered closed. Proving the closure of the frozen soil body in the station design might need additional measures due to the large volume of the frozen soil.

Freezing soil in de Allerod layer is occurring 1.5 times as slow as in the sand layers. Good monitoring to ensure the Allerod is at the required thickness is therefore necessary.

Due to relatively thin freeze walls compared to shafts, the frozen soil body for a tunnel is much more sensitive to additional heat exposure, such as (Jessberger et al, 2003):

1. Sun on the walls of tunnel access shafts
2. Tunnel ventilation
3. Heat of hydration from shotcrete or concrete during installation of the lining
4. Changes in the freezing system, such as reduction in the freeze capacity, malfunctioning of freeze plants and turn-off or loss of refrigeration pipes

During the constructing of the concrete lining inside the frozen body heat exchange must be monitored closely for two reasons:

- → The hydration heat of the shotcrete may not lead to significant thawing of the frozen body.
- → The shotcrete may not freeze before it is totally cured.

Usually these problems will be avoided by controlling the refrigeration power and ensuring a concrete temperature of 10 to 20 degrees Celsius at start (Zentrum Geotechnik).

→ *Boring through diaphragm walls*

Reinforcement in diaphragm walls is usually placed by using reinforcement cages. However, in the design the TBM needs to bore through four diaphragm walls. Reinforcement therefore needs to be placed differently in the cross section where the TBM passes through the walls. These cross sections are reinforced using glass fibre, instead of the usual steel reinforcement cages. To ensure a good connection and especially the water tightness of the connection between the tunnel and diaphragm walls a jet-grout body could be used.

→ *Number of freeze pipe rows and freeze period*

In a row of freeze pipes the pipes are located with a centre to centre distance of 1.0 [m]. A maximum centre to centre distance of 15 times the outer diameter of the freeze pipe is defined and usually a distance of 1.0 [m] is chosen. With a freeze pipe a thickness of 1.5 [m] can be reached therefore two freeze pipe rows are installed with centre to centre distance of 1.5 [m].

To freeze the required thickness, 3 [m] with two rows of freeze pipes, will take in sand approximately 25 days. In the Allerod this will be approximately 40 days. As ensuring the water tightness of the large frozen soil body will require some time too, the time from start of freezing to start of construction works is estimated at 50 days.

CHAPTER 6

ECONOMIC FEASIBILITY OF SOIL FREEZING TO CONSTRUCT UNDERGROUND SPACES

Whether the alternative station design can be competitive to other designs depends next to the technical feasibility mainly on costs. In this chapter, as accurately as possible, the costs of the station constructed using freezing techniques are estimated.

6.1. Costs Freezing Technique

Determining the costs of the station is done with the help of some documents provided by a freeze contractor. A price indication for a shaft using nitrogen freezing was available. As brine freezing is used costs cannot be adopted directly but an indication is possible. An overview of the costs is not presented in this report due to confidentiality of the numbers.

Since several costs are not mentioned explicitly in the cost overview 10 [%] of the total costs are added as 'Other works'. These costs include for example isolation between the shotcrete and final lining, water pumps and the demolishing of the diaphragm walls between the station and the tunnel. For engineering, execution and profit 35 [%] of the total costs construction costs are added. The total costs of only the structural works amount to 85 million euro. In this number finishing of the station is taken into account.

6.2. Economic Feasibility of the Design Compared to the Current Design

The costs for the current executed design of station Rokin are estimated at €60.000.000,-. The costs estimated for the design using freezing are €85.000.000,-. Both are very rough cost estimations, however, a conclusion can be drawn.

Although the freeze design is a more expensive design, the costs are not two or three times as much with respect to conventional techniques as is often thought. Also it should be noted that in the cost estimation costs of rerouting cables and ducts and hindrance to the surroundings are not yet taken into account. Also delays during execution due to a not continuous boring process are not added. This all works in the advantage of the freeze design. Though further research is necessary it can be stated from an economical point of view the freeze design can compete with the current design.

CHAPTER 7

CONCLUSIONS AND RECOMMENDATIONS

In the previous chapters research has been done to determine a suitable way to construct a station with as main construction technique artificial ground freezing. By performing a case study the feasibility of the method was determined. This chapter discusses the main conclusions and recommendations that can be drawn up as a result of this study.

7.1. Conclusions

The conclusions made are divided in three paragraphs; conclusions related to the design of the station, conclusions related to results of modelling with Plaxis and conclusions related to the applicability of soil freezing in general.

7.1.1. Design Station

A study has been performed to determine the current developments in the field of artificial ground freezing. A design similar to the design executed in Berlin for subway station Museumsinsel appeared the most promising for application in Amsterdam. From two construction shafts a soil body will be frozen around two already bored tunnel tubes. Soil will be excavated within the frozen soil body and after a final lining is constructed, the frozen soil will be thawed again. As a result of the alignment of the tunnel and the local soil profile station Rokin is chosen to make this alternative design for.

To obtain a factor of safety of 2.0, the thickness of the frozen soil body should be 3.0 [m] at the minimum. This value is verified with a Plaxis calculation and several empirical calculations. The thickness will be achieved by installing two rows of freeze pipes around the circumference of the station to be built. Meetings with people both from the engineering side and contractor side confirmed in the Amsterdam subsoil one row of freeze tubes can ensure a freeze wall with a thickness of 1.5 [m].

In a row of freeze pipes the pipes are located with a centre to centre distance of 1.0 [*m*]. The distance between the rows is 1.5 [*m*]. In the out-of-plane direction the freezing will take place over a length of 110 [*m*]. As the freeze pipes will be installed from two sides towards each other the length of these freeze pipes will vary between 55 and 60 [*m*] and the total amount of freeze pipes is 216. Figure 7.1 again the cross section of the frozen soil body is shown.

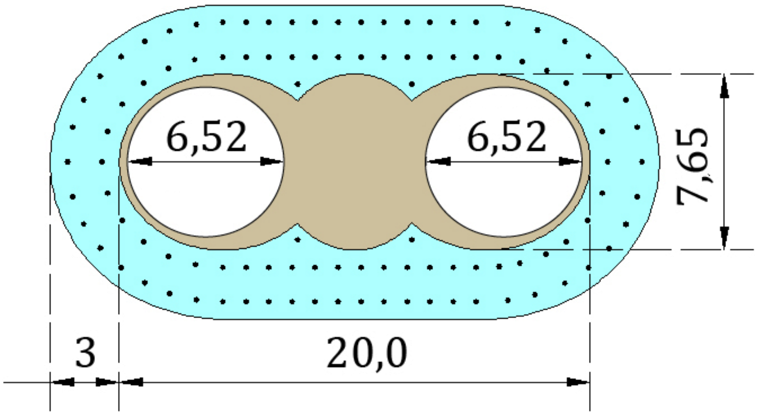


Figure 7.1 – Cross section of the frozen soil body with indicated the location of freeze pipes (lengths in [*m*])

Figure 7.2 shows the cross section of the station after the final lining is constructed. The thickness of the shotcrete lining is 250 [*mm*], the thickness of the final lining is 350 [*mm*]. At the junction of the arches columns are constructed. Construction of these columns is done after shotcrete at the centre part is applied together with the final lining of the centre part. Columns are placed every 15 [*m*] and have a width and length of 1.5 [*m*]. To support the station in length direction a beam is constructed on top of the columns.

The width of the platform is 9.0 [*m*], which was a preset requirement. The minimal free height is 3.0 [*m*]. Metro trains can run on both sides of the platform and have free space with height 4.3 [*m*] and width 4.0 [*m*]. The platform has a height of 0.8 [*m*] relative to the final floor of the station.

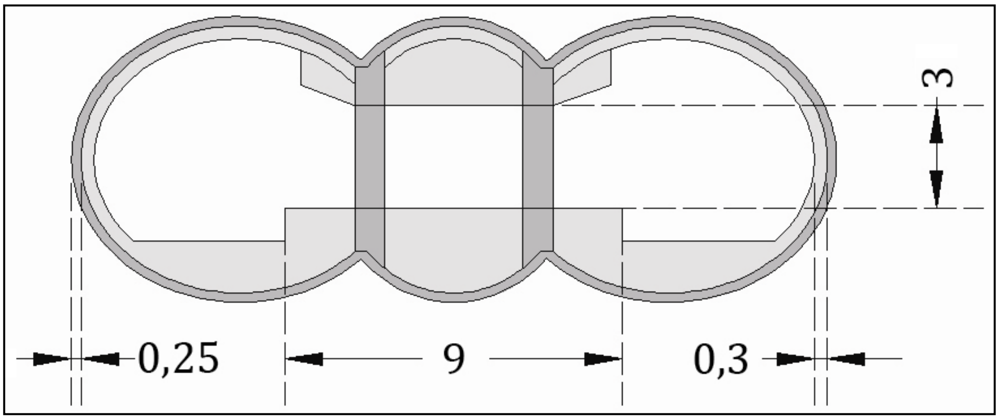


Figure 7.2 – Cross section of the station with final lining (lengths in [*m*])

7.1.2. Results Modelling with Plaxis

A start was made by modelling the cross section shown in Figure 7.1. In the first model the frozen soil was only surrounding the circumference of the station. Calculations showed most requirements on settlement and structural forces were not met with this approach. Due to absence of a column supporting the frozen beam, the first excavation step, excavating centre part 1, leads to a large amount of settlements. It was concluded, after seeing the results of this model, a column made of frozen soil is a necessity to construct the station within the preset requirements.

A second model was made in which the column, or in fact a freeze wall, was implemented. The maximum vertical total settlements were reduced to less than 1/3 of the original settlements, to a value of 14.8 [*mm*]. With the implementation of the freeze wall all preset requirements were met. The largest phase settlements occurred in the first model when the centre part was excavated. In the second model the largest phase displacements occur when the side part is excavated. For this excavation the freeze wall needs to be demolished.

Some uncertainty is present in the change of parameters of the Allerod due to the freeze-thaw process. Therefore three calculations were made to determine the influence of changing parameters. Assuming a stiffness decrease of 5 [%], and thus assuming an Allerod layer consisting of little clay, led to a vertical total settlement of 14.8 [*mm*]. Assuming the parameters of the Allerod change to remoulded clay parameters and thus assuming a large part of the Allerod consists of clay, led to a vertical total settlement of 21.8 [*mm*]. These are a the minimum and maximum boundary for the actual settlement value.

Calculations were started using a spacing between the concrete columns of 7 [*m*]. As the normal force in the columns resulted in a value 1/7 of the allowed normal force the spacing was increased in the calculations to 15 [*m*]. Settlements only increased to 15.2 [*mm*], however in the out-of-plane direction forces and moments on the span between two columns increases quickly. A maximum distance of 12 [*m*] between columns is advised.

It can be concluded that an alternative design for station Rokin using artificial freezing would be technically feasible. The behaviour of the Allerod after thawing should be further verified, but if the worst case scenario is true maximum settlements are not exceeded largely. Mitigating measures could be taken to keep the settlements within boundaries.

7.1.3. General Applicability of AGF

It may be clear that artificial ground freezing is an expensive method to construct with. There must be a strong motive to apply this method. Motives to use AGF for underground station construction can be:

- small hindrance on ground level is allowed;
- there is little space available on ground level for construction works;
- a watertight excavation has to be ensured, leakage will cause prohibited damage to surroundings;
- other methods lead to excessive risks;
- small infrastructure in the subsoil does not allow an open excavation.

The type of soil and hydraulic circumstances are very important for the number of additional measures that need to be taken. Related to hydraulic circumstances there are three points of interest:

1. To cause a sufficient bonding of the soil, a water content of at least 10 [%] should be present.
2. For large applications of artificial ground freezing brine freezing will have preference over liquid nitrogen freezing, as the construction time will be relatively long and brine freezing will lead then to a more economically attractive solution. For brine freezing in general a maximum ground water flow of 2 [*m/day*] would be acceptable.
3. The chemical composition of ground water also influences the possibility of freezing the soil. Salt water will require lower freezing temperatures, thus fresh water is ideal. If the soil does not meet these three requirements measures can be taken but this will naturally lead to an increase in costs.

The type of soil largely influences the reaction of a soil to a freeze-thaw process. In granular soils freezing will not lead to any problems. No significant heave will occur since excess pore pressures will not develop due to the possibility of drainage to unfrozen areas. Thawing frozen granular soils will not lead to any problems either. The structure of granular soil is equal to the soil structure before the freezing took place.

In soils with a lower permeability, clays and silts, freezing is more difficult. Firstly the time required to freeze the same volume of soil in clay would take 1.5 times longer than in sand. Secondly soils with low permeability experience heave. The formation of ice lenses causes a volume increase in the direction of the temperature gradient. Organic soils (peat) are most difficult to freeze. Ice crystals can be formed which are dangerous because they do not have a large strength. Freezing peat layers takes about 3 times longer than freezing sand layers. Thawing of fine grained soils will lead to changing soil properties. The soil behaves as a remoulded clay, due to loss of loading history.

Thus ideal situations for using artificial ground freezing to construct an underground station are:

- soils containing of fresh ground water without any chemical pollutions;
- a water content of at least 10 [%];
- little or no ground water flow;
- coarse grained soils.

This does not mean the method is not applicable in finer grained soils, but one should then take into account the consequences and take adequate mitigating measures.

If freezing partly will take place in clay the expected heave should be determined and assessed whether this heave is allowable. If structures on ground level are located above or in the vicinity of freeze works it should be checked if freeze pressures do not cause damage to those structures. If no structures on ground level are positioned above the freeze works it should be determined if heaving of the ground level is problematic for, for instance, roads or cables and pipes in the subsoil. If heave does cause problems in theory its magnitude can be decreased with several options, three of them are listed below:

- installing heating pipes surrounding the intended volume of the frozen soil body;
- start with intermittent freezing after the frozen soil body has reached its desired volume.

A risk analysis has to be made before making a decision on this point. It could be that the required measures to decrease heave cost more than handling the results of the heave.

The second main point of interest when freezing in clay are the changing parameters of the soil after thawing. If a part of the station is constructed in clay the negative effects can be mitigated if necessary. Soil improvement can be used to reduce settlements due to thawing of clays. However a station constructed entirely in clay is not feasible.

Feedback to the research question:

→ *Can soil freezing be a viable option as a construction method for constructing underground stations in densely built up areas?*

Overall it could be stated artificial soil freezing is a method which is due to costs mainly suitable for complex situations or small applications at the present. If there is, however, a motive such as lack of space on ground level, the method becomes interesting. Especially if more traditional constructing methods lead to risks and require expensive additional measures to reduce those risks. Artificial soil freezing is a robust method and will lead to a design with little risks, if on beforehand the consequences are determined accurately. An alternative design for station Rokin proved the technical feasibility of the construction method.

7.2. Recommendations

During the thesis assumptions and simplifications had to be made. In this paragraph recommendations for further research are listed.

At the start of the thesis the choice has been made to look only at the mechanical aspects of artificial ground freezing. Thermal aspects were not looked into during the thesis. The thermal aspects of freezing determine however the development of the frozen soil body in time, which is a very important side of freezing. For instance to determine the heave, effect of intermittent freezing and effect of placing freeze pipes closer together of further apart. A design using freeze techniques must include a thermal calculation.

The behaviour of the Allerod after thawing is not quite known, due to the varying composition of the layer and the unknown behaviour of a soil consisting of a part of clay. If a freeze design would be executed at location Rokin on beforehand tests should be performed to determine the thaw behaviour, soil properties and crimp, of the Allerod.

- → The thaw behaviour and final crimp could be determined using a special frost heave apparatus. The lay-out of this apparatus is similar to a triaxial cell. To determine the frost heave rate chilled brine is pumped through the upper plate and water supply through the base plate is possible to allow water to move to the frost front. To determine thaw behaviour and total crimp the reverse could be done; a frozen soil sample with a heat supply through the upper plate.
- → After thawing the soil shows different behaviour due to changed soil properties. Especially the compression index is of importance and this parameter can be determined by performing an Oedometer test on a sample of thawed soil. The permeability of the soil could be determined with a Falling Head test.

Creep of the frozen soil body is modelled by lowering the soil stiffness manually per construction phase. The creep of a frozen body cannot be modelled directly with current creep models in Plaxis as they are suitable for soft soils, not for frozen soils. Implementing a creep model suitable for ice creep would be a more accurate way to model the creep than lowering the stiffness manually. Klein's formula for creep, for which creep parameters are already determined by CDM, could be input for the model.

In the current model no 3D influences are taken into account, which leads to a conservative design. Each part of the excavation is temporary not supported due to the time span between excavation and applying shotcrete. A three dimensional arching effect then will occur which is not simulated in a two dimensional model. To include the 3D effect in a 2D calculation a factor β can be applied to obtain correct settlements and forces in the lining. Part of the stresses (1-β) will act on the unsupported tunnel and the rest (β) will act on the supported tunnel (tunnel lining). Vermeer and Möller (2005) performed research on the value of β, by comparing 3D calculations with 2D calculations.

CHAPTER 8

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ANNEX A – METHODS TO DESCRIBE CREEP

Klein's method which uses the following formula:

$$\varepsilon = A^* \sigma_1{}^{B}{}^* t^C$$

With creep parameters A, B and C.

NEN-Bjerrum isotache model which uses the following parameters:

RR	Recompression ratio (primary compressibility before the pre-consolidation pressure)
CR	Compression ratio (primary compressibility after the pre-consolidation pressure)
C_α	Secondary compressibility after the yield stress (creep parameter)

→ The model suits Dutch and international norms and regulations

Koppejan model which uses the following parameters:

C_p	Primary consolidation constant before the pre-consolidation pressure
C_s	Secondary consolidation constant before the pre-consolidation pressure
C_p'	Primary consolidation constant after the pre-consolidation pressure
C_s'	Secondary consolidation constant after the pre-consolidation pressure

→ The model is not used internationally
→ The model is only suitable for loading (not unloading/reloading)

a,b,c-isotache model which uses the following parameters:

a	Direct compression coefficient
b	Secondary compression coefficient
c	Secondary strain rate coefficient (creep parameter)
τ	Intrinsic time (time necessary to derive at the current strain under the current stress)

→ The model uses natural strain instead of linear strain to determine parameters which is more accurate, especially for soft soil layers.

International parameters:

C_c	Compression index
C_r	Recompression index
C_α	Creep index

Soft Soil Creep model (Plaxis) uses the following parameters:

λ^*	Modified compression index
μ^*	Modified creep index
κ^*	Modified swelling index

ANNEX B – SOIL MODELS USED

In Plaxis the relationship between strain and stress within a soil is given by a certain soil model. In general this is done using the following equation:

$$\underline{\dot{\sigma}}' = \underline{\underline{M}} \cdot \underline{\dot{\varepsilon}}$$

The material model consists of several mathematical equations, which relate increments of effective stress rates to increments of strain rates. In the equation above $\underline{\underline{M}}$ is the material stiffness matrix. This matrix is based on model parameters and may depend the stress or strain state of the soil or other state parameters.

The soil models used in Plaxis to model the case are described in short in this Annex.

Mohr-Coulomb model

The Mohr-Coulomb model is a linear-elastic perfectly-plastic model with the Mohr-Coulomb failure contour. The failure contour determines whether or not plasticity occurs. For stresses within the failure contour the strains are purely plastic and therefore reversible, according to Hooke’s Law. Parameters for the Mohr-Coulomb model are two elasticity parameters (E and υ) and three plasticity parameters (c, φ and ψ).

The Mohr-Coulomb failure criterion is defined by six yield functions formulated in terms of the principle stresses s'_1, s'_2 and s'_3. One of the failure criteria is given here:

$$\frac{1}{2}\left(\sigma'_3 - \sigma'_1\right) \leq c'\cos\varphi' - \frac{1}{2}\left(\sigma'_3 + \sigma'_1\right)\sin\varphi'$$

All six criteria together form the hexagonal area shown in Figure XX (Plaxis Material Models Manual). Corresponding with the six yield functions there are six plastic potential functions defined.

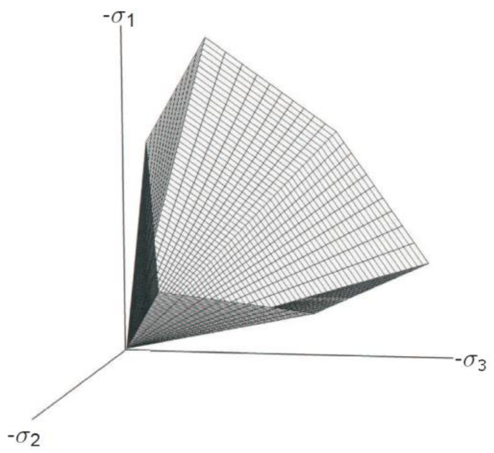

Figure XX - The Mohr-Coulomb failure contour for a cohesionless soil

With a certain *c* and *φ* the Mohr-Coulomb yield functions only depend on the stress state of the soil and do not depend on developed plastic strains. The model speaks of perfect plasticity because the hexagonal cone is therefore fixed in principle stress space.

Hardening Soil Model

The Hardening Soil model describes soil behaviour with a stress dependent stiffness according to the power law. In the yield function a state parameter is present which is related to the plastic strain. The yield surface can therefore expand due to plastic straining and is not fixed in principle stress space. In this model failure is also defined according to the Mohr-Coulomb criterion. Plasticity parameters are therefore the same as for the MC model (c, φ and ψ).

Two types of hardening are included in the model; shear hardening and compression hardening. Stiffness parameter E_{50}^{ref} is used to model shear hardening with irreversible strains due to primary deviatoric loading. Stiffness parameter E_{oed}^{ref} is used to model compression hardening with irreversible strains due to primary compression. A third stiffness parameter E_{ur}^{ref} is used to model elastic unloading and reloading. In the figures below the determination of the three stiffnesses is shown. E_{50}^{ref} and E_{ur}^{ref} are determined with triaxial tests and E_{oed}^{ref} is determined with an oedometer test.

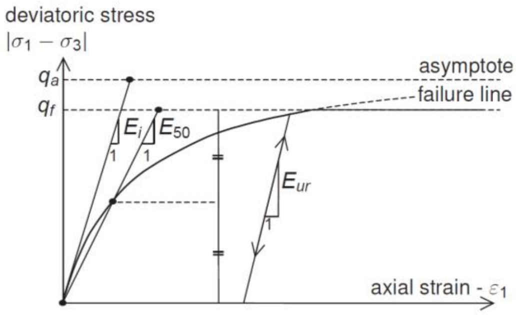

Figure B.1 – Standard drained triaxial test for primary loading

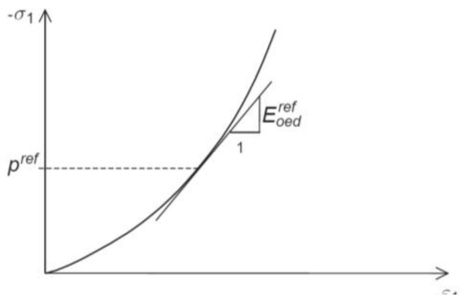

Figure B.2 – Oedometer test for primary loading

As the three stiffnesses are stress dependent, the input parameters are valid for a certain reference stress level, either a horizontal effective stress for E_{50}^{ref} and E_{ur}^{ref} and vertical effective stress for E_{oed}^{ref}. The value of the stiffnesses at different stress levels is calculated with the following equations.

$$E_{50} = E_{50}^{ref}\left(\frac{\sigma'_3}{p^{ref}}\right)^m \qquad E_{ur} = E_{ur}^{ref}\left(\frac{\sigma'_3}{p^{ref}}\right)^m \qquad E_{oed} = E_{oed}^{ref}\left(\frac{\sigma'_1}{p^{ref}}\right)^m$$

In these equations p^{ref} is the reference stress level, which usually has a value of 100 kPa. The level of stress dependency of the stiffness is given by *m*.

ANNEX C – INPUT PARAMETERS PLAXIS

In the tables in this annex the input parameters of the Plaxis calculation, related to initial soil properties can be found.

Tracé — **Noord tot en met Europaplein**

Parameters ten behoeve van Eindig Elementen Model berekeningen

				Massa		Sterkte				Doorlatendheid	Additioneel				Hardening Soil model					
	nr.	grondlaag	Omschrijving	γ_m [-]	γ_{sat} [kN/m³]	γ_m [-]	c' [kPa]	φ' [°]	ψ [°]	k_v [m/s]	ν [-]	K_0 [-]	POP [kPa]	OCR [-]	γ_m [-]	Pref [kPa]	E'50;ref [kPa]	Eoed;ref [kPa]	Eur;ref [kPa]	m
uiterste grens -				1.0	15.0	1.2		18		1.0E-05					1.3		6600	4800	19200	
ondergr. vervorming					15.0			22		1.0E-05							8557	6000	25000	
verwachting	01	Aanvulling	Aanvulmateriaal, puin, klei, veen en zand		15.0			26		1.0E-06	0.15					100	10000	8000	30000	0.8
bovengr. vervorming					15.0			28		1.0E-07							11433	10000	35000	
uiterste grens +				1.0	15.0	1.2		34		1.0E-07					1.3		14800	13000	45000	
uiterste grens -				1.0	18.4	1.2		23		1.0E-04					1.3		13200	11500	38500	
ondergr. vervorming					18.4			27		1.0E-04							17134	15000	50000	
verwachting	01A	Ophooglaag, zand	Ophoogmateriaal zand		18.4			30		1.0E-05	0.15					100	20000	20000	60000	0.8
bovengr. vervorming					18.4			32		1.0E-06							22866	30000	70000	
uiterste grens +				1.0	18.4	1.2		38		1.0E-06					1.3		29700	39000	91000	
uiterste grens -				1.0	12.1	1.2		15		1.0E-07			10		1.3		1300	600	2300	
ondergr. vervorming					12.1			18		1.0E-07			10				1713	800	3000	
verwachting	04	Geulopvulling veen met kleilagen			12.1		9	20		1.0E-08	0.15	0.60	10			100	2000	1000	5000	0.8
bovengr. vervorming					12.1			22		1.0E-09			10				2287	1200	9000	
uiterste grens +				1.0	12.1	1.2		26		1.0E-09			10		1.3		3000	1600	11700	
uiterste grens -				1.0	16.9	1.2		20		1.5E-08			10		1.3		6700	1900	11500	
ondergr. vervorming					16.9			24		1.5E-08			10				8700	2500	15000	
verwachting	07	Geulopvulling slappe klei	KLEI, met zandlaagjes		16.9		6	26		1.5E-09	0.15	0.50	10			100	10000	4000	25000	0.8
bovengr. vervorming					16.9			29		1.5E-10			10				11300	5500	35000	
uiterste grens +				1.0	16.9	1.2		35		1.5E-10			10		1.3		14700	7200	45500	
uiterste grens -				1.0	13.9	1.2		17		5.7E-09			10		1.3		3300	800	11500	
ondergr. vervorming					13.9			20		5.7E-09			10				4284	1000	15000	
verwachting	07A	Geulopvulling slappe humeuze			13.9		10	22		5.7E-10	0.15	0.55	10				5000	1500	20000	0.8
bovengr. vervorming					13.9			24		5.7E-11			10				5716	2000	25000	
uiterste grens +				1.0	13.9	1.2		29		5.7E-11			10		1.3		7400	2600	32500	
uiterste grens -				1.0	10.5	1.2		15		1.0E-07			10		1.3		1500	600	5400	
ondergr. vervorming					10.5			18		1.0E-07			10				2000	800	7000	
verwachting	08	Hollandveen	VEEN (bruin)		10.5		5	20		1.0E-08	0.15	0.65	10			100	2000	1000	10000	0.8
bovengr. vervorming					10.5			22		1.0E-09			10				2000	1200	13000	
uiterste grens +				1.0	10.5	1.2		26		1.0E-09			10		1.3		2600	1600	16900	
uiterste grens -				1.0	16.5	1.2		22		1.5E-08			10		1.3		5800	1500	15400	
ondergr. vervorming					16.5			26		1.5E-08			10				7500	2000	20000	
verwachting	09	Oude zeeklei	KLEI (matig vast), grijs		16.5		7	33		1.5E-09	0.15	0.60	10			100	9000	3000	25000	0.8
bovengr. vervorming					16.5			35		1.5E-10			10				10500	4000	30000	
uiterste grens +				1.0	16.5	1.2		40		1.5E-10			10		1.3		13700	5200	39000	
uiterste grens -				1.0	17.9	1.2		23		1.0E-06					1.3		7700	3100	19200	
ondergr. vervorming	10				17.9			27		1.0E-06							10000	4000	25000	
verwachting	10B	Wadafzetting, zandhoudend	ZAND (matig fijn), sterk kleihoudend/ KLEI, grijs,		17.9		2	35		1.0E-07	0.20	0.40				100	12000	5000	33000	0.5
bovengr. vervorming					17.9			38		1.0E-08							14000	6000	40000	
uiterste grens +				1.0	17.9	1.2		48		1.0E-08					1.3		18200	7800	52000	
uiterste grens -				1.0	15.2	1.2		23		1.0E-08			10		1.3		5800	1500	7700	
ondergr. vervorming					15.2			27		1.0E-08			10				7500	2000	10000	
verwachting	11	Wadafzetting, Hydrobia klei	KLEI (grijs), sterk zandig		15.2		8	34		1.0E-09	0.15	0.59	10			100	9000	3000	15000	0.8
bovengr. vervorming					15.2			34		1.0E-10			10				10500	4000	20000	
uiterste grens +				1.0	15.2	1.2		41		1.0E-10			10		1.3		13700	5200	26000	
uiterste grens -				1.0	11.7	1.2		15		1.0E-07			10		1.3		1500	600	5400	
ondergr. vervorming					11.7			18		1.0E-07			10				2000	800	7000	
verwachting	12	Basisveen	VEEN (bruin), met enkele kleilaagjes		11.7		6	21		1.0E-08	0.15	0.65	10			100	2000	1000	10000	0.8
bovengr. vervorming					11.7			23		1.0E-09			10				2000	1200	13000	
uiterste grens +				1.0	11.7	1.2		28		1.0E-09			10		1.3		2600	1600	16900	
uiterste grens -				1.0	19.8	1.2		28		1.5E-03					1.3		26900	15400	76900	
ondergr. vervorming					19.8			33	3	1.5E-03							35000	20000	100000	
verwachting	13	Eerste zandlaag	ZAND (grijs), matig fijn, zwak siltig		19.8			33	3	1.5E-04	0.20	0.40				100	40000	30000	200000	0.5
bovengr. vervorming					19.8			35	5	1.5E-05							45000	40000	230000	
uiterste grens +				1.0	19.8	1.2		47	17	1.5E-05					1.3		58500	52000	364000	

registratie	projectcode	status	
D3	01270L	Definitief	
autorisatie	naam	paraaf	datum
opgemaakt	HERJ		00-04-07
goedgekeurd	TEUE		
vrijgegeven	GRAF		

1) De onder- en bovengrens van de vervormingsberekeningen mbt het Hardening soil model, zijn afgeleid uit verschillende typen onderzoek.
2) Dilitantiehoek is gedefinieerd als: $\psi = \varphi' - 30°$
3) Laag 01, aanvulling is door de verschillende soorten materiaal (hout, puin, grind, klei, veen, zand) zeer divers van samenstelling.
4) De hoek van inwendige wrijving (φ) is gebaseerd op een rek niveau van 15%, dat wil zeggen na bezwijken.
5) De "engineering judgement" van het holoceen is grotendeels gebasseerd op archief gegevens van Omegam.
6) De dwarscontractiecoëfficient (ν) is afgeleid uit literatuuronderzoek.
7) De K0 waarde is gebaseerd op de hoek van inwendige wrijving bij bezwijken voor zand en op de Ip waarde voor klei.
8) De verticale doorlatendheid is gelijk aan de horizontale doorlatendheid, behalve voor laag 04, 08 en 12. Voor deze lagen is de horizontale doorlatendheid 2x zo groot.

10/04/0
Pagina 2/

Tracé — Noord tot en met Europaplein

Parameters ten behoeve van Eindig Elementen Model berekeningen

Tracé	nr.	grondlaag	Omschrijving	Massa γ_m [-]	Massa γ_{sat} [kN/m³]	Sterkte γ_m [-]	Sterkte c' [kPa]	Sterkte φ' [°]	Sterkte ψ [°]	Doorlatendheid k_v [m/s]	Additioneel ν [-]	Additioneel K_0 [-]	Additioneel POP [kPa]	Additioneel OCR [-]	Hardening Soil model γ_m [-]	Pref [kPa]	E'50;ref [kPa]	Eoed;ref [kPa]	Eur;ref [kPa]	m
uiterste grens -				1.0	18.5	1.2		23		3.0E-04					1.3		11500	5400	23100	
ondergr. vervorming					18.5			28		3.0E-04							15000	7000	30000	
verwachting	14	Allerod	LEEM, sterk zandig		18.5			33		3.0E-05	0.20	0.40				100	17000	13000	45000	0.5
bovengr. vervorming					18.5			35		3.0E-06							19000	20000	60000	
uiterste grens +				1.0	18.5	1.2		42		3.0E-06					1.3		24700	26000	78000	
uiterste grens -				1.0	17.8	1.2		23		1.0E-08					1.3		6500	1900	19200	
ondergr. vervorming					17.8			27		1.0E-08							8400	2500	25000	
verwachting	15	Geulvulling klei met zandlagen	KLEI, matig zandig, matig humeus, kalkrijk		17.8			32		1.0E-09	0.15	0.50				100	8400	3500	35000	0.8
bovengr. vervorming					17.8			35		1.0E-10							8400	4500	42000	
uiterste grens +				1.0	17.8	1.2		40		1.0E-10					1.3		10900	5900	54600	
uiterste grens -				1.0	18.6	1.2		23		1.0E-04					1.3		6500	3100	19200	
ondergr. vervorming					18.6			27		1.0E-04							8400	4000	25000	
verwachting	16	Geulvulling zand met kleilagen	ZAND (matig fijn), met kleilaagjes, schelpen		18.6			33		1.0E-05	0.15	0.47				100	8400	5000	35000	0.8
bovengr. vervorming					18.6			38		1.0E-06							8400	6000	42000	
uiterste grens +				1.0	18.6	1.2		46		1.0E-06					1.3		10900	7800	54600	
uiterste grens -				1.0	19.0	1.2		28		1.0E-03					1.3		24600	19200	61500	
ondergr. vervorming					19.0			33	3	1.0E-03							32000	25000	80000	
verwachting	17	Tweede zandlaag	ZAND (grijs), matig fijn tot matig grof, kalkrijk		19.0			35	5	1.0E-04	0.20	0.40				100	35000	35000	190000	0.5
bovengr. vervorming					19.0			39	9	1.0E-05							35000	45000	260000	
uiterste grens +				1.0	19.0	1.2		47	17	1.0E-05					1.3		49400	58500	364000	
uiterste grens -				1.0	18.4	1.2		23		2.4E-05					1.3		20800	11500	46200	
ondergr. vervorming					18.4			27		2.4E-05							27000	15000	60000	
verwachting	17A	Marien siltig Eemzand	ZAND (fijn), grijs, met kleilaagjes		18.4			33	3	2.4E-06	0.20	0.40				100	30000	20000	120000	0.5
bovengr. vervorming					18.4			35	5	2.4E-07							33000	25000	150000	
uiterste grens +				1.0	18.4	1.2		42	12	2.4E-07					1.3		42900	32500	195000	
uiterste grens -				1.0	18.0	1.2		23		4.0E-07		0.46		1.1	1.3		5800	2300	28600	
ondergr. vervorming					18.0			27		4.0E-07		0.46		1.1			7500	3000	35000	
verwachting	18	Overgangslaag	KLEI, uiterst siltig, zwak humeus, kalkrijk		18.0		15	34		4.0E-08	0.15	0.49		1.2		100	9600	4000	64000	0.8
bovengr. vervorming					18.0			34		4.0E-09		0.53		1.4			11000	5000	80000	
uiterste grens +				1.0	18.0	1.2		41		4.0E-09		0.53		1.4	1.3		14300	6500	104000	
uiterste grens -				1.0	17.9	1.2	10	24		2.0E-08		0.58		1.5	1.3		6200	2300	23100	
ondergr. vervorming					17.9		10	29		2.0E-08		0.58		1.5			8000	3000	30000	
verwachting	19	Mariene Eemklei (zone 1)	KLEI (grijsgroen), zwak siltig tot zwak zandig		17.9		20	32		2.0E-09	0.15	0.68		2.0		100	11000	4000	50000	0.8
bovengr. vervorming					17.9		30	34		2.0E-10		0.76		2.5			14000	5000	65000	
uiterste grens +				1.0	17.9	1.2	15	41		2.0E-10		0.76		2.5	1.3		18200	6500	84500	
uiterste grens -				1.0	17.1	1.2	10	24		2.0E-08		0.55		1.5	1.3		6900	2700	26900	
ondergr. vervorming					17.1		10	25		2.0E-08		0.55		1.5			9000	3500	35000	
verwachting	19A	Mariene Eemklei (zone 2)			17.1		20	34		2.0E-09	0.15	0.65		2.0			12000	4500	55000	0.8
bovengr. vervorming					17.1		30	35		2.0E-10		0.74		2.5			15000	5500	70000	
uiterste grens +				1.0	17.1	1.2	15	42		2.0E-10		0.74		2.5	1.3		19500	7200	91000	
uiterste grens -				1.0	17.2	1.2	10	23		2.0E-08		0.61		1.5	1.3		5400	1900	21500	
ondergr. vervorming					17.2		10	28		2.0E-08		0.61		1.5			7000	2500	28000	
verwachting	19B	Mariene Eemklei (zone 3)			17.2		20	30		2.0E-09	0.15	0.71		2.0			9000	3000	45000	0.8
bovengr. vervorming					17.2		30	32		2.0E-10		0.79		2.5			11000	3500	60000	
uiterste grens +				1.0	17.2	1.2	15	38		2.0E-10		0.79		2.5	1.3		14300	4600	78000	
uiterste grens -				1.0	16.6	1.2	10	23		2.0E-08		0.61		1.5	1.3		4600	1900	21500	
ondergr. vervorming					16.6		10	26		2.0E-08		0.61		1.5			6000	2500	28000	
verwachting	19C	Mariene Eemklei (zone 4)			16.6		20	30		2.0E-09	0.15	0.71		2.0			8000	3000	40000	0.8
bovengr. vervorming					16.6		30	32		2.0E-10		0.79		2.5			10000	3500	55000	
uiterste grens +				1.0	16.6	1.2	15	33		2.0E-10		0.79		2.5	1.3		13000	4600	71500	
uiterste grens -				1.0	14.5	1.2		22		6.0E-08		0.58		1.2	1.3		5400	1200	19200	
ondergr. vervorming					14.5			26		6.0E-08		0.58		1.2			7000	1500	25000	
verwachting	20	Laag van Harting	KLEI, groengrijs, licht zandhoudend,		14.5		10	28		6.0E-09	0.15	0.64		1.5		100	10000	2000	46000	0.8
bovengr. vervorming					14.5			30		6.0E-10		0.73		2.0			13000	2500	60000	
uiterste grens +				1.0	14.5	1.2		36		6.0E-10		0.73		2.0	1.3		16900	3300	78000	

registratie	projectcode	status	
D3	01270L	Definitief	
autorisatie	naam	paraaf	datum
opgemaakt	HERJ		00-04-07
goedgekeurd	TEUE		
vrijgegeven	GRAF		

1) De onder- en bovengrens van de vervormingsberekeningen mbt het Hardening soil model, zijn afgeleid uit verschillende typen onderzoek.
2) Dilitantiehoek is gedefinieerd als: $\psi = \varphi' - 30°$
3) Laag 01, aanvulling is door de verschillende soorten materiaal (hout, puin, grind, klei, veen, zand) zeer divers van samenstelling.
4) De hoek van inwendige wrijving (φ) is gebasseerd op een rek niveau van 15%, dat wil zeggen na bezwijken.
5) De "engineering judgement" van het holoceen is grotendeels gebasseerd op archief gegevens van Omegam.
6) De dwarscontractiecoëfficient (ν) is afgeleid uit literatuuronderzoek.
7) De K0 waarde is gebasseerd op de hoek van inwendige wrijving bij bezwijken voor zand en op de Ip waarde voor klei.
8) De verticale doorlatendheid is gelijk aan de horizontale doorlatendheid, behalve voor laag 04, 08 en 12. Voor deze lagen is de horizontale doorlatendheid 2x zo groot.
9) γ_{droog} is gelijk aan γ_{nat}

Tracé

Noord tot en met Europaplein

Parameters ten behoeve van Eindig Elementen Model berekeningen

	nr.	grondlaag	Omschrijving	Massa γ_m [-]	γ_{sat} [kN/m³]	Sterkte γ_m [-]	c' [kPa]	φ' [°]	ψ [°]	Doorlatendheid k_v [m/s]	Additioneel ν [-]	K_0 [-]	POP [kPa]	OCR [-]	Hardening Soil model γ_m [-]	Pref [kPa]	E'50;ref [kPa]	Eoed;ref [kPa]	Eur;ref [kPa]	m
uiterste grens -				1.0	19.4	1.2		25		1.0E-04					1.3		15400	7700	38500	
ondergr. vervorming					19.4			30		1.0E-04							20000	10000	50000	
verwachting	21	Tussenlaag zand	ZAND, zeer fijn, zwak siltig		19.4			33	3	1.0E-05	0.20	0.40				100	25000	15000	140000	0.5
bovengr. vervorming					19.4			35	5	1.0E-06							30000	20000	175000	
uiterste grens +				1.0	19.4	1.2		42	12	1.0E-06					1.3		39000	28000	227500	
uiterste grens -				1.0	19.7	1.2		27		1.0E-08		0.49		1.2	1.3		9200	2300	38500	
ondergr. vervorming					19.7			32		1.0E-08		0.49		1.2			12000	3000	50000	
verwachting	22	Glaciale Drenthe klei	KLEI, zwak zandig		19.7		15	34		1.0E-09	0.15	0.55		1.5		100	15000	5000	75000	0.8
bovengr. vervorming					19.7			36		1.0E-10		0.65		2.0			18000	8000	90000	
uiterste grens +				1.0	19.7	1.2		43		1.0E-10		0.65		2.0	1.3		23400	10400	117000	
uiterste grens -				1.0	18.5	1.2		22		1.0E-06		0.52		1.2	1.3		9200	2300	30800	
ondergr. vervorming					18.5			26		1.0E-06		0.52		1.2			12000	3000	40000	
verwachting	23	Glaciale Warven klei	KLEI, zwak siltig met zandlaagjes		18.5		5	32		1.0E-07	0.15	0.58		1.5		100	15000	4000	58000	0.6
bovengr. vervorming					18.5			34		1.0E-08		0.68		2.0			18000	5000	70000	
uiterste grens +				1.0	18.5	1.2		41		1.0E-08		0.68		2.0	1.3		23400	6500	91000	
uiterste grens -				1.0	19.6	1.2		28		1.0E-03					1.3		23100	26900	54600	
ondergr. vervorming					19.6			33	3	1.0E-03							30000	35000	71000	
verwachting	24	Derde zandlaag	ZAND, matig fijn, zwak siltig tot zwak grindig		19.6			35	5	1.0E-04	0.20	0.40				100	35000	40000	200000	0.5
bovengr. vervorming					19.6			39	9	1.0E-05							45000	45000	300000	
uiterste grens +				1.0	19.6	1.2		47	17	1.0E-05					1.3		58500	58500	390000	

registratie	projectcode	status	
D3	01270L	Definitief	
autorisatie	naam	paraaf	datum
opgemaakt	HERJ		00-04-07
goedgekeurd	TEUE		
vrijgegeven	GRAF		

Opmerkingen

1) De onder- en bovengrens van de vervormingsberekeningen mbt het Hardening soil model, zijn afgeleid uit verschillende typen onderzoek.
2) Dilitantiehoek is gedefinieerd als: $\psi = \varphi' - 30°$
3) Laag 01, aanvulling is door de verschillende soorten materiaal (hout, puin, grind, klei, veen, zand) zeer divers van samenstelling.
4) De hoek van inwendige wrijving (φ) is gebasseerd op een rek niveau van 15%, dat wil zeggen na bezwijken.
5) De "engineering judgement" van het holoceen is grotendeels gebasseerd op archief gegevens van Omegam.
6) De dwarscontractiecoëfficient (ν) is afgeleid uit literatuuronderzoek.
7) De K0 waarde is gebaseerd op de hoek van inwendige wrijving bij bezwijken voor zand en op de Ip waarde voor klei.
8) De verticale doorlatendheid is gelijk aan de horizontale doorlatendheid, behalve voor laag 04, 08 en 12. Voor deze lagen is de horizontale doorlatendheid 2x zo groot.

ANNEX D – RESULTS

Deformed mesh

For the second model the deformation of the mesh is shown for phases 6 to 22, which are all phases after the soil body is frozen. Deformations are scaled up 20 times for all figures.

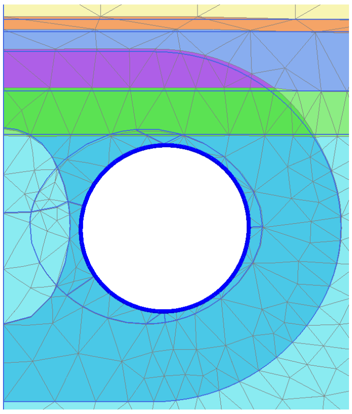

Figure D.1 – Phase 6
(Freezing soil body)

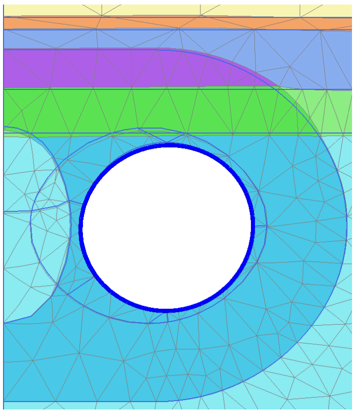

Figure D.2 – Phase 7
(Lowering pore water)

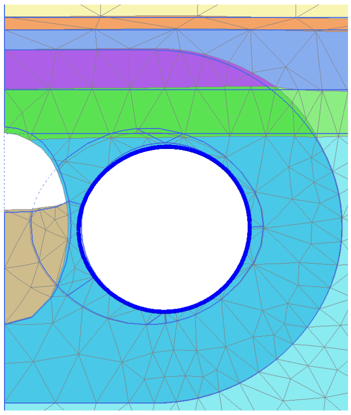

Figure D.3 – Phase 8
(Excavating centre part 1)

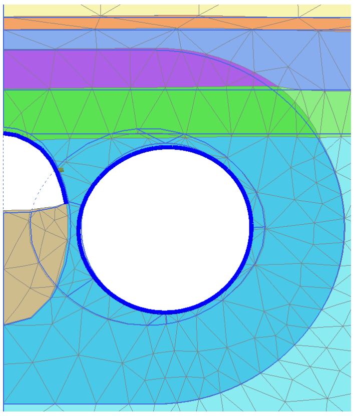

Figure D.4 – Phase 9
(Shotcrete centre part 1)

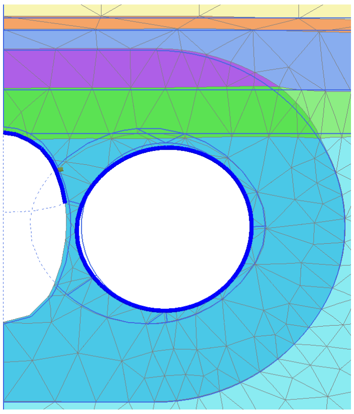

Figure D.5 – Phase 10
(Excavating centre part 2)

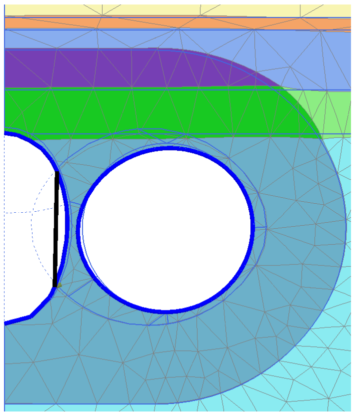

Figure D.6 – Phase 11
(Shotcrete centre part 2)

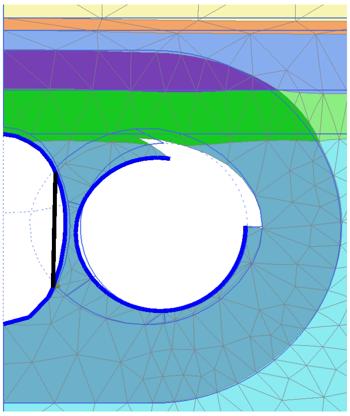

Figure D.7– Phase 12
(Excavating side part 1)

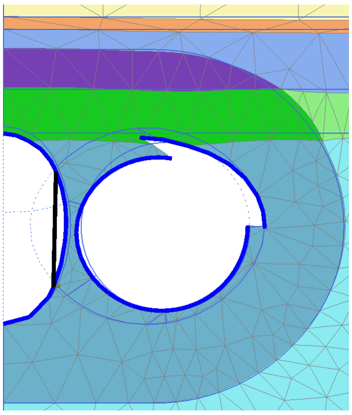

Figure D.8– Phase 13
(Shotcrete side part 1)

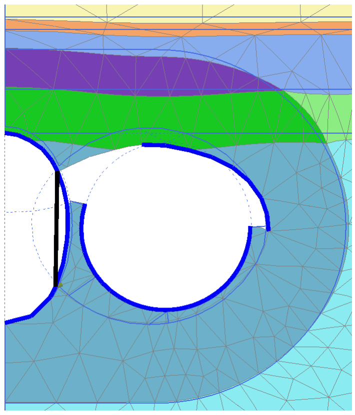

Figure D.9– Phase 14
(Excavating side part 2)

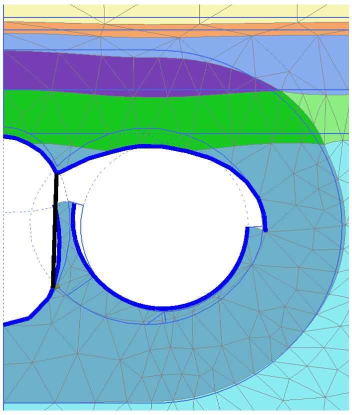

Figure D.10 – Phase 15
(Shotcrete side part 2)

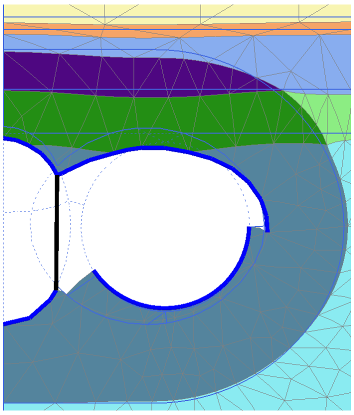

Figure D.11 – Phase 16
(Excavating side part 3)

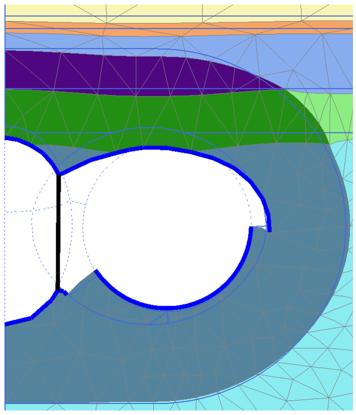

Figure D.12 – Phase 17
(Shotcrete side part 3)

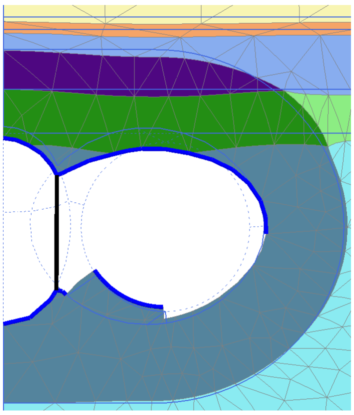

Figure D.13 – Phase 18
(Excavating side part 4)

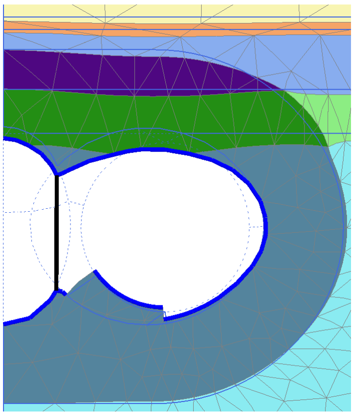

Figure D.14 – Phase 19
(Shotcrete side part 4)

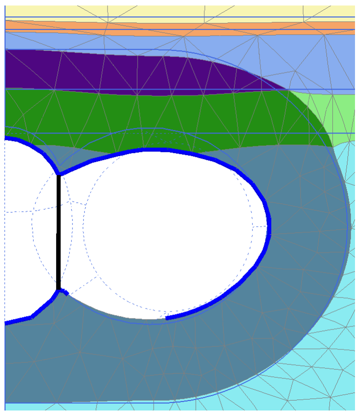

Figure D.15– Phase 20
(Excavating side part 5)

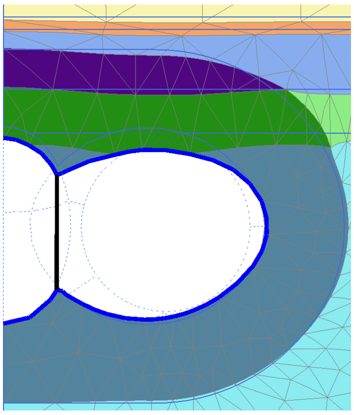

Figure D.16 – Phase 21
(Shotcrete side part 5)

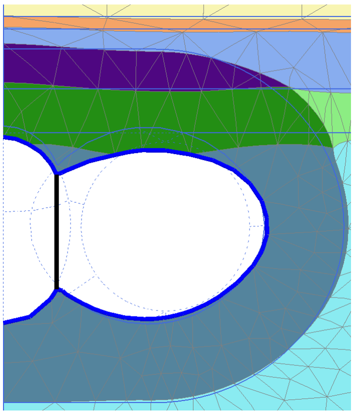

Figure D.17 – Phase 22
(Final lining)

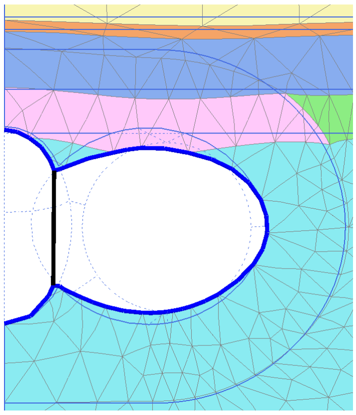

Figure D.18 – Phase 23
(Thawed soil)

Possibilities to model Allerod

Figure D.19, D.20 and D.21 show the total vertical displacements of several calculations. In the calculations the soil properties of the Allerod after thawing are varied. Figure D.22, D.23 and D.24 show the vertical phase displacements of those calculations.

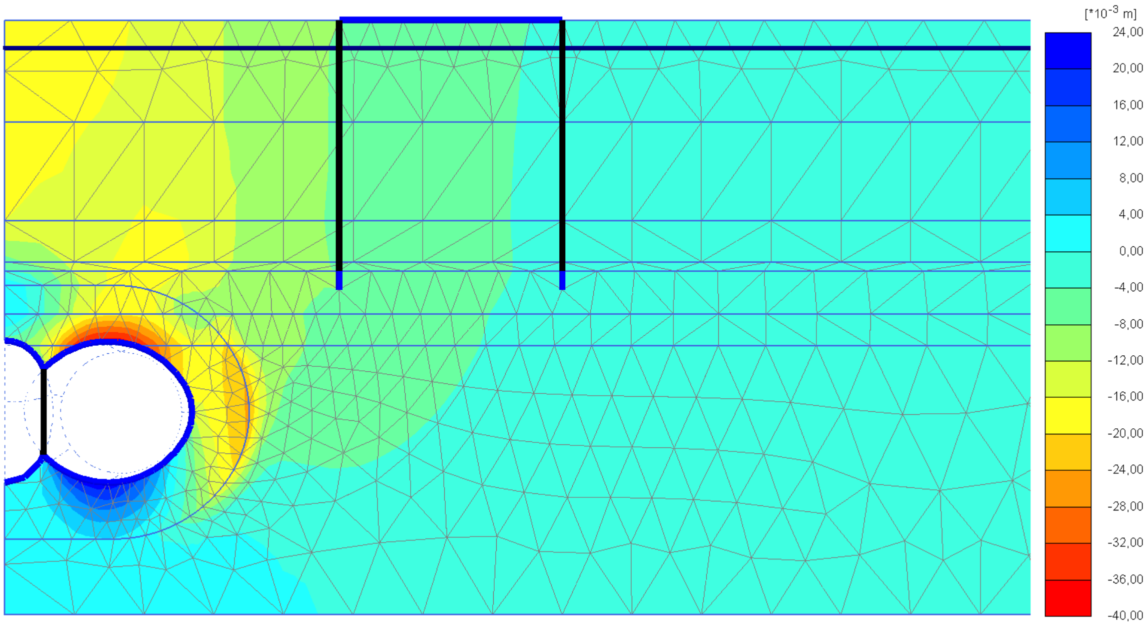

Figure D.19 – Allerod parameters lowered with 5% (Total vertical settlements)

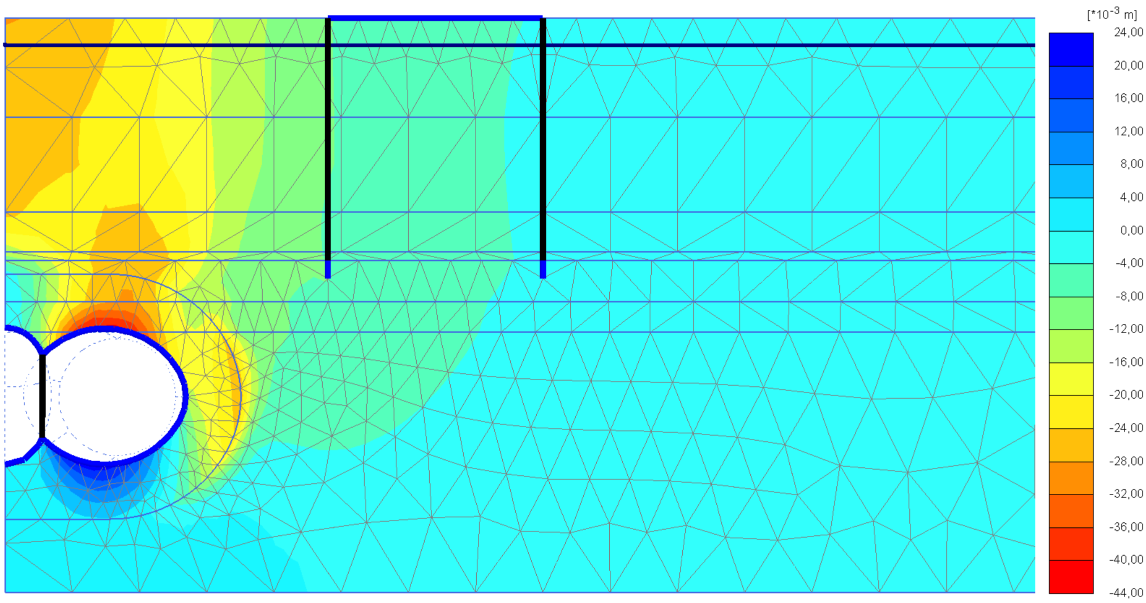

Figure D.20 – Allerod with adjusted C_c, e and k parameters (Total vertical settlements)

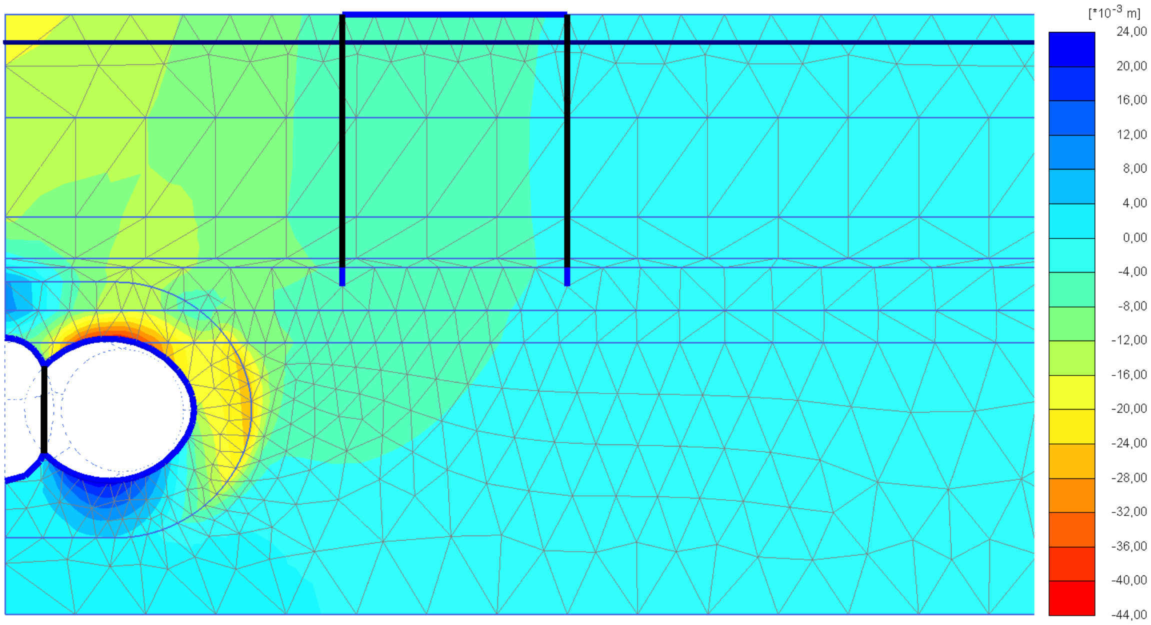

Figure D. 21– Allerod modelled with Soft Soil Creep model (Total vertical settlements)

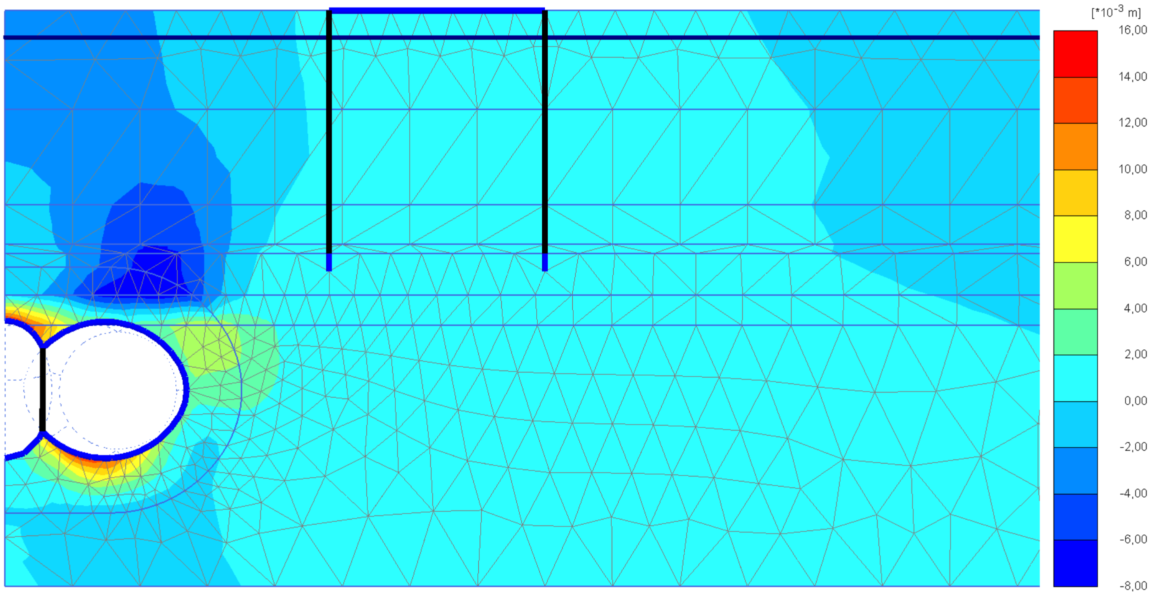

Figure D. 22 – Allerod parameters lowered with 5% (Vertical phase settlements)

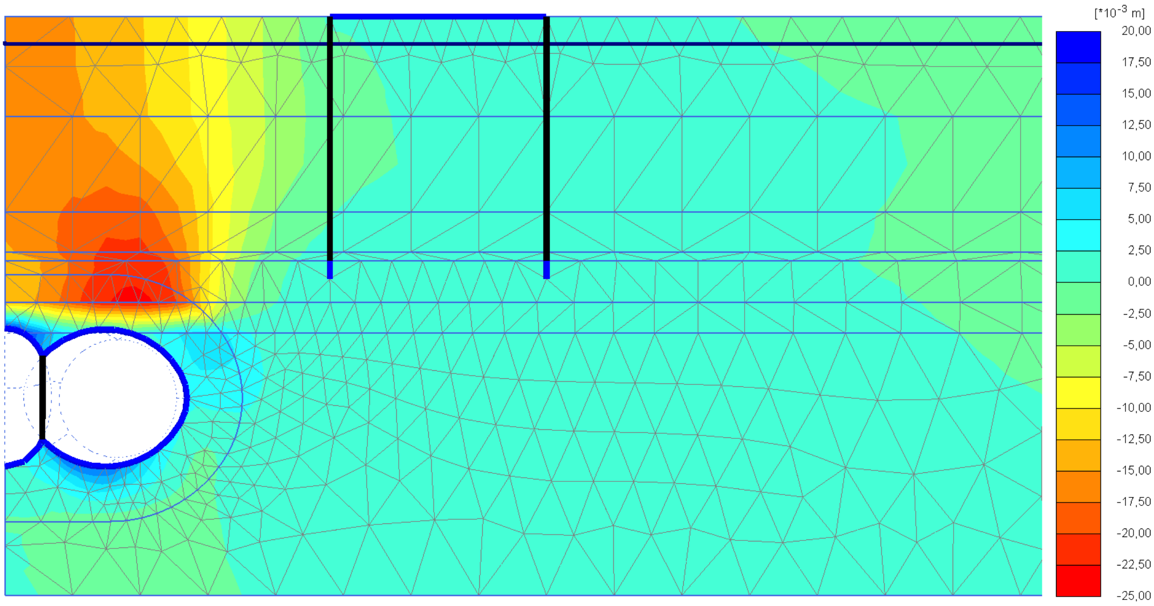

Figure D. 23 – Allerod with adjusted *Cc*, *e* and *k* parameters (Vertical phase settlements)

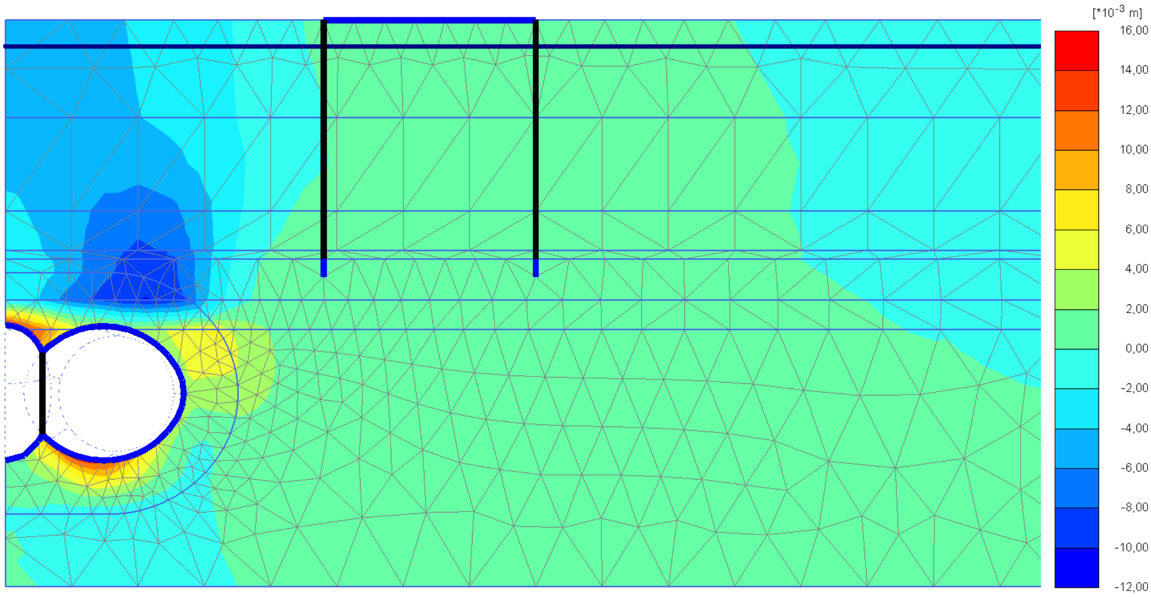

Figure D. 24 – Allerod modelled with Soft Soil Creep model (Vertical phase settlements)