Ground freezing and groundwater control at underground station CS in Rotterdam

Sol congelé et contrôle des nappes d'eaux souterraines à la gare Rotterdam CS de metro souterraines

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ABSTRACT
Rather complex subsurface and groundwater conditions have played an important role for the design and execution of the extensive reconstruction works on the existing underground station Rotterdam CS. The retaining wall is designed as a diaphragm wall for about 90% of the building pit circumference. The remaining 10% has been generated by means of ground freezing. This paper describes the results of 3D groundwater calculations using the MicroFem code; this model has been used for the design and during execution of the project to determine the effect of dewatering activities. Two project case histories are presented, illustrating that knowledge of the local groundwater regime provides key-information in case of unexpected events, like leakage through diaphragm walls and during the process of coming to the vital conclusion that 100% watertightness of the frozen soil structure has been achieved.

RÉSUMÉ
Dans le projet des travaux de reconstruction de la gare Rotterdam CS de metro souterraines, les conditions du sous-sols et des nappes d'eaux souterraines ont joués un rôle important dans l'ébauche et l'exécution des travaux liés à l'excavation. Le mur de soutien est planifié comme paroi moulé pour 90 % du pourtour de la construction. Les 10 % restants sont obtenus par la congélation du sol. Cette publication décrit les résultats des calculs en 3 dimensions avec le programme MicroFem; ce modèle éprouvé a permis de mieux comprendre les conditions et les variations des conditions des nappes d'eaux souterraines. Deux cas historiques sont présentés démontrant que la connaissance des régimes de nappes d'eaux souterraines fournit des informations clé en cas d'évènements imprévus comme la fuite d'eau à travers les parois moulées et durant le processus de conclusion sur l'accomplissement des 100 % d'imperméabilité de la structure de sol congelé.

Keywords: large excavation, collar construction, 3D groundwater modelling, diaphragm wall, leakage, ground freezing, combined LN_2 (liquid nitrogen) and brine freezing, thermal erosion, pumping test, watertightness.

1 INTRODUCTION

In the centre of Rotterdam, The Netherlands, extensive reconstruction works are being carried out on the existing underground station CS between 2006 and 2012 (Hannink & Thumann, 2007). The present underground station, which is the terminal station of the Erasmus line, is being transformed from a two-track, single platform lay-out into a three-track, two platforms configuration (Figure 1).

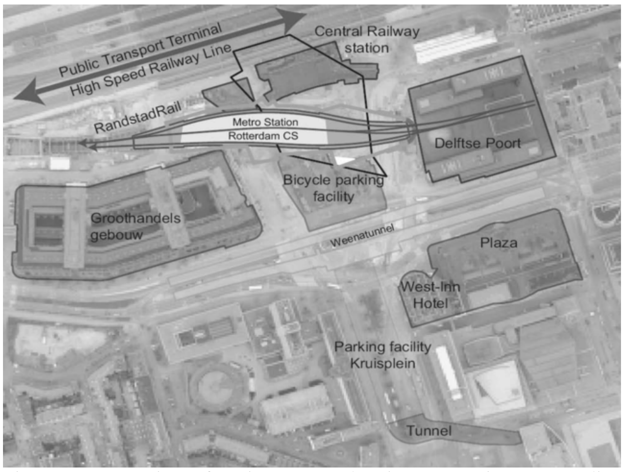

Figure 1 – Top view of station square showing excavation contour line (orange) and lay-out of re-built underground station CS (yellow); also nearby building projects (Weena tunnel, Central Railway Station, underground bicycle- and Kruisplein car parking) and adjacent buildings (Groot Handelsgebouw, Delftse Poort, Plaza, West-Inn hotel) are indicated.

The excavation method is based on isolating the water carrying sand layers inside the building pit by means of a diaphragm wall reaching to a depth of 38 m below reference level (i.e. NAP that corresponds with sea level). The ground surface level at the station square is about NAP -0.3 m. However, diaphragm walls could not be applied at all locations around the 7,500 m^2 excavation area, which is the reason for an extraordinary part of the retaining wall – the so called collar construction - being generated by means of ground freezing techniques. A more detailed description of the rebuilding of the underground station Rotterdam CS and the design of the collar construction can be found in Thumann & Haβ (2007).

Groundwater control is a very important issue while building in urban areas in the western part of The Netherlands. Where groundwater levels change significantly due to building projects, it may cause structural damage to nearby structures. Within this particular project, the rather complex sub-surface and groundwater conditions also played an important role for the design and execution of the works related to the excavation itself.

Engineering of this project is mainly performed by the Engineering Consulting Division of Rotterdam Public Works. Major design condition is that underground traffic and passenger transfer must not be affected during all stages of the building activities.

This paper focuses on (i) groundwater conditions of the site including three-dimensional (3D) computer modelling and validation, (ii) case history on the procedure to prove 100% watertightness of the collar construction, and (iii) case history on remedial activities to repair a substantial leak through the diaphragm wall.

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2 SUBSURFACE CONDITIONS AND GROUNDWATER REGIME

2.1 *General soil description*

The subsurface conditions at the building site are schematized as follows (Table 1):

Elevation (NAP m)		Origin -	Hydraulic head
from	to	Type of soil	(NAP m)
-0.3	-4.5	fill – sand	-1.5
-4.5	-5.5	Holocene – clay	
-5.5	-8.0	Holocene – peat	
-8.0	-17.0	Holocene – clay	
-17.0	-35.0	Pleistocene – sand	-2.3
-35.0	-37.0	Kedichem – clay	
-37.0	-40.0	Kedichem – sand	-2.4

Table 1 – Soil conditions

The following local variations in soil stratification are identified. In east-west direction a former canal is present, which has been excavated around 1960 for the floating transport of tunnel segments to the station square. The width of the channel is approximately 15 to 20 m; depth around NAP -11 m to -12 m. At the location of the highrise office building "Delftse Poort" a sand layer of varying thickness and size is present at depth of NAP -13 m to -16 m inbetween the clay and peat layers. The remaining clay and peat layers between the canal, the sandy layer (so-called "donk", a river dune sediment) and the Pleistocene aquifer are relatively thin and may even be locally absent (Figure 2).

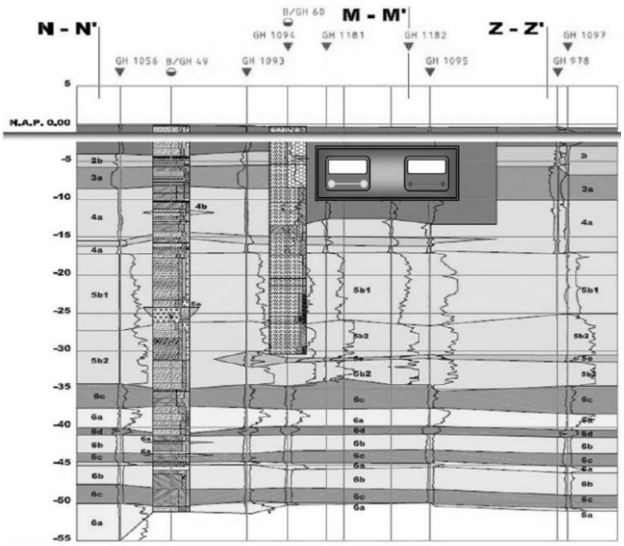

Figure 2 – Geotechnical profile of soil conditions underneath office building Delftse Poort (east side of the station square).

From a pumping test in 2004, it has been determined that locally hydraulic contact between canal, donk-layer and Pleistocene aquifer had to be anticipated for.

2.2 *Three dimensional modelling of groundwater conditions*

Several groundwater related questions have been solved using a 3D groundwater model, which has been generated with the finite element code MicroFem (Kemker & Boer, 2005). The model area boundaries are the river Nieuwe Maas in the south, lake Kralingse Plas in the east, Rotterdam Airport in the north and the river Schie in the west; the model covers an area of 60 km^2. The soil layers in the model have been based on available CPT's and borings as well as geological and geohydrological data, from the BIODIEP (Maas, 2003) regional groundwater model. The model consists of six soil layers with varying thickness, each with horizontal transmissivity [kD] and vertical hydraulic resistance [c] (Table 2):

Table 2 – MicroFem soil layers

Layer no.	Soil type	c (days)	kD (m^2/day)
1	Anthropogenic – sand		15
2	Holocene – clay/peat	1,500 – 1,800	
3	Pleistocene – sand		350
4	Pleistocene – sand		350
5	Pleistocene – sand		350
6	Kedichem – clay	3,000 – 4,500	
	Kedichem – 2nd aquifer		1,700

The Pleistocene aquifer is subdivided into layers no. 3 to 5; the transmissivity of the total Pleistocene layer is divided between the sublayers proportionally with layer thickness. The model has been calibrated using a large number of observation wells from the monitoring network owned by Rotterdam Public Works. Especially the values of hydraulic resistance have been optimised. The hydraulic resistance in the model varies between 500 days (near river Nieuwe Maas) and 11,500 days (northern part of the model). Bottom of the second aquifer has been taken as the hydrological base of the hydrogeological system.

In addition to the calibration to overall regional data as described above, the model has been fine-tuned in order to account for local soil phenomena like the influence of the donk sand layer and the former canal. This has been done using monitoring data of large dewatering works in Rotterdam carried out in the past (a.o. projects Willemspoortunnel, underground stations Beurs and Wilhelminaplein), resulting in a special set of parameter values for the centre of Rotterdam.

The present 3D model, that is the result of extensive calibration to both regional and local data, is considered to provide a realistic schematisation of the groundwater regime for a large part of Rotterdam.

2.3 *Groundwater monitoring facilities*

The building activities were to be executed in close proximity to the existing underground station. Therefore the probability of causing deformations to the underground tunnel due to related activities (e.g. dewatering of the excavation) had to be investigated. Another design condition that affected the risk profile was the requirement for undisturbed operation of underground traffic during all re-building stages of the project. Therefore, monitoring specifications have been defined considering these risks (Berkelaar et al., 2007). One of the main parameters to be monitored has been the phreatic groundwater level and hydraulic head around the construction site.

Monitoring of the groundwater levels has been performed using water- and strain gauges. The groundwater level in and around the building pits has been monitored by more than 110 standard piezometers, of which nearly 25 were equiped with fully automatic monitoring gear, a warning system with two types of hazard levels and monitoring data available on-line in real time. Also, the hydraulic head inside the sandfill of the former canal was monitored on-line, in real time with approximately 10 piezometers located directly underneath the existing underground tunnel structure.

Monitoring started prior to the pumping tests, that were performed to establish the watertightness of the diaphragm wall. Based on interpretation of this information, remedial actions can be prepared in case a relatively poor quality of diaphragm wall is encountered. Additionally, the detailed set-up of the pumping tests should provide information on different qualities of diaphragm wall sections, as these were made by more than one contractor. Also, confirmation of the expected pumping rates could be obtained which is important for validation of licences related to a.o. discharge of water volumes. About 80 of the piezometers were placed into the Pleistocene aquifer, along the inside perimeter of the excavation pit and close to the diaphragm wall. The other 30 piezometers were placed around the building pit.

Two periods are distinguished for groundwater monitoring in this project. At first, monitoring of the drawdown during the pumping tests, and subsequently, monitoring during the excavation and construction works for the underground station.

2.4 *Working method to determine the hydraulic resistance of the diaphragm wall*

No specific value for the resistance of the diaphragm wall has been defined within the contract. Authorization for the dewatering works is based on a hydraulic resistance of the diaphragm wall of 200 days. The quality of the diaphragm wall was rated in accordance with Table 3.

Table 3 – Hydraulic resistance of diaphragm wall

Diaphragm wall qualification	c (days)
Good	1,000
Avarage	800
Regular	500
Poor (necessary to take measures or repairs)	100

Two pumping tests have been prescribed in the construction specifications as to obtain sufficient monitoring data sets to determine the quality of the diaphragm wall. The first test had to be carried out after finishing the diaphragm wall, but prior to freeze-up of the collar construction. This implies that about 10% of the retaining wall circumferencing the excavation is still open. The second test was scheduled after freeze-up of the collar construction, thus for the fully enclosed excavation pit. It was recognized that in both cases the hydraulic head inside the construction area was also affected by groundwater extractions for the nearby construction sites of RandstadRail-Conradstraat and Weena tunnel.

During the first pumping test the hydraulic head in the Pleistocene aquifer has been lowered to NAP -14 m at the far west side of the building pit, for a period of two weeks. The hydraulic head of the Pleistocene aquifer below the highrise office buiding Delftse Poort – just east of the collar construction - reached its lowest level of approximately NAP -7 m by the end of the test. Hereafter, the pumps were relocated to the middle of the excavation (hydraulic head: NAP -12 m) to obtain another monitoring data set.

The entire excavation area was hydrologically isolated during the second pumping test, as the excavation had been closed after freeze-up of the collar construction. The hydraulic head in the Pleistocene aquifer inside the excavation was then reduced to NAP -18 m. Detailed set-up and additional goal of this test are described in one of the following case histories (section 3).

2.5 *Calculation of the hydraulic resistance of the diaphragm wall*

Modelling of the diaphragm wall in the MicroFem code requires the introduction of additional parameters, that are derived as follows.

According to Darcy's law the groundwater flow is defined by (1):

$$Q = q \cdot D = -k \cdot \frac{\partial h}{\partial x} \cdot D \tag{1}$$

in which:

Q: amount of leakage in m^3/day per meter length of the diaphragm wall;

q: Darcy velocity of groundwater in m/day;

D: thickness of the Pleistocene aquifer in m;

k: permeability of the soil in m/day;

$\partial h / \partial x$: hydraulic gradient in m/m.

To determine the design parameters for the diaphragm wall it is significant that the amount of leakage through the diaphragm wall (Q) at a known difference in piezometric level (Δh) computed with the model is consistent to Darcy's law, as in Figure 3.

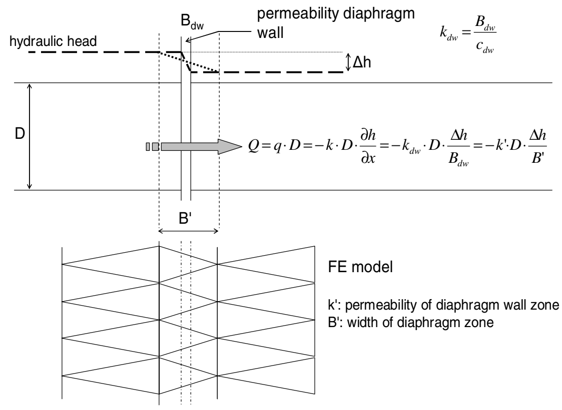

Figure 3 – Modelling of diaphragm wall hydraulic parameters in the MicroFem code.

Therefore, diaphragm wall properties are related to Darcy's law parameters as in (2):

$$k \cdot \frac{\partial h}{\partial x} = k_{dw} \cdot \frac{\Delta h}{B_{dw}} \tag{2}$$

in which:

k_{dw}: permeability of the diaphragm wall in m/day;

Δh: difference in piezometric level inside and outside the building pit, in m;

B_{dw}: width (thickness) of the diaphragm wall in m.

In accordance with water-resistance calculations for excavation walls such as sheet piles, permeability and hydraulic resistance are defined as in (3):

$$k_{dw} = \frac{B_{dw}}{c_{dw}} \tag{3}$$

in which:

c_{dw}: hydraulic resistance of the diaphragm wall in days.

Combining equations (1) to (3) yields (4):

$$Q = -\frac{\Delta h}{c_{dw}} \cdot D \tag{4}$$

The leakage in the MicroFem code is defined by:

$$Q = -k' \cdot \frac{\Delta h}{B'} \cdot D \tag{5}$$

in which:

k ': (equivalent) permeability of the diaphragm wall in m/day;

B': width (thickness) of the zone assigned in the model to represent the diaphragm wall, in m.

From equations (4) and (5) it follows that equivalent permeability in the MicroFem code has to be set to:

$$k' = \frac{B'}{c_{dw}} \tag{6}$$

To determine the actual hydraulic resistance of the diaphragm wall, the hydraulic heads in the Pleistocene aquifer have been computed using the MicroFem code with varying values for the resistance of the wall, based on equation (6). The calculations were carried out with a transmissivity [kD] for the diaphragm wall corresponding to an hydraulic resistance of 100, 200, 400, 600 and 1,000 days, including the flow rates as reported by contractor during the pumping test and a fixed transmissivity value of the first – Holocene - aquifer of 1,050 m^2/day.

The hydraulic heads and drawdowns as calculated by the MicroFem code have been compared with the time records of a number of piezometers along the inner perimeter of the building pit. Based on the degree of agreement between calculated and observed piezometric heads, the average value of the resistance of the diaphragm wall in the vicinity of the piezometer has been estimated to be approximately 200 days (Figure 4).

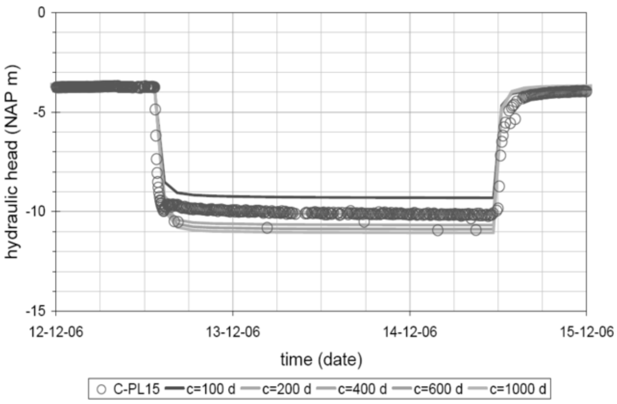

Figure 4 – Time record of hydraulic head at piezometer C-PL15 (points) during first pumping test vs. calculated values (lines) by the MicroFem code for varying resistances. Best fit is for c = 200 days.

Subsequently, the calculated hydraulic heads within the building pit (assuming a hydraulic resistance of the diaphragm wall of 200 days) have been compared to the measured data of all piezometers during the stationairy phase of the first pumping test, see Figure 5.

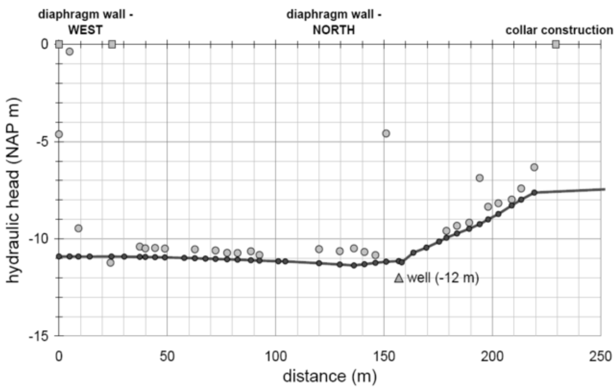

Figure 5 – Typical result of the first pumping test, showing measured hydraulic heads (points) in Pleistocene sand layer along northern diaphragm wall compared to calculated values (line), for dewatering at the middle of the excavation (well) to hydraulic head NAP -12.0 m.

The measured values of all available piezometers showed a reasonable fit with the calculated values for diaphragm wall resistance c = 200 days. The quality of the diaphragm wall therefore had to be considered as moderate (less than regular according to Table 3).

2.6 *Recommendations for contract specifications on quality of diapraghm wall*

The hydraulic resistance of a diaphragm wall is determined by means of a pumping test based on a method described by Elprama et al. (2006). The test is to be carried out in two successive steps to account for the phreatic storage of the Pleistocene sand.

During the first step the hydraulic head in the construction pit is reduced to a level of 2 m above the top of the Pleistocene sands. A quick response of the hydraulic head is visible, because the aquifer only responds to the change of pore water pressure (reversible elastic storage) in a confined aquifer. To identify the stationary situation, the hydraulic head within the construction pit has to show a more or less constant value for at least 24 hours.

During the second step, the hydraulic head is further reduced to a few meters below the top of the Pleistocene sands. This changes the topmost part of the Pleistocene sands in an unsaturated zone and the aquifer changes from confined into phreatic unconfined conditions. As a result of the phreatic storage of the Pleistocene sand, the response of the groundwater is (much) slower than during the first step. Literature (Elprama et al., 2006) states that the stationary situation is reached after 3 x 24 hours. The period of 3 x 24 hours is stated to be required to minimize the influence of phreatic storage in the measured leakage flow rate.

However, from data of flow rate versus hydraulic head reduction during the two years of construction period of this project (and other projects in Rotterdam), it appears that the flow rate which is needed for the required lowering of the hydraulic head is further reduced by a factor of 2 to 3 as time elapses. This applies to both the building pit of underground station Rotterdam CS and a similar building pit of another underground station (station Blijdorp). It seems that this effect is also due to the phreatic storage phenomenon.

Based on these experiences, contract specifications for the required watertightness of a diaphragm wall can for the best be based on an expected minimum hydraulic resistance of the wall (for example 200 days) in combination with the estimated hydraulic resistance of the Kedichem Layer at the bottom of the building pit (in this case 4,300 days). These values must comply with a certain amount of drainage of the building pit and can readily be checked by the pumping test for confined hydraulic conditions. Contract specifications may then be defined as follows:

- the drainage due to leakage are limited to a certain discharge flow rate;
- the contractor proposes measures for approval by the surveyor to restrict the amount of leakage in case the amount of leakage is larger than specified;
- the contractor shows the durability and the geotechnical stability of the building pit in case of local leakage, for approval by surveyor.

3 CASE HISTORY: WATERTIGHTNESS OF FROZEN COLLAR CONSTRUCTION

3.1 *Collar construction design*

About 10% of the retaining wall consists of a frozen soil volume of at least 2.5 m thickness reaching to NAP - 38 m. This so-called collar construction is generated by means of two rows of in total about 70 vertical freeze pipes. After freeze-up of the collar construction, it will be serving as retaining wall for excavating down to 14 m depth. The existing underground tunnel – with public transport in full service - is then embedded in the frozen soil body (Figure 6).

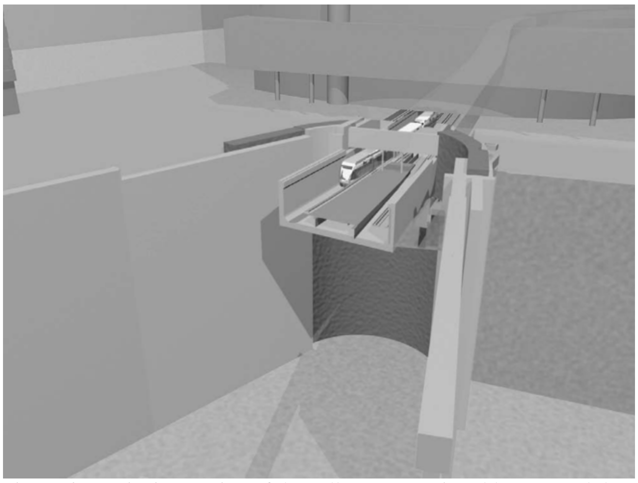

Figure 6 – Artist impression of the collar construction (blue) around the existing underground tunnel; the collar construction being supported by diaphragm walls (grey).

Basic condition of ground freezing is that groundwater flow has to be limited during freeze-up, as too high flow velocities will prevent closure of the frozen soil volume. Design condition for freeze-up of the soil has been a maximum groundwater flow velocity of 4 m/day in this case.

3.2 *Groundwater flow during freeze-up*

During the first start-up of the freezing process in December 2006, virtually no groundwater flow was present in the Pleistocene sand layer. Unfortunately, ground freezing had to be stopped due to structural damage of the freeze pipes; re-start was only possible by the end of March 2007.

Before re-start, dewatering of the nearby Weenatunnel building pit had been initiated, resulting in a hydraulic head gradient of about 0.02 with flow velocities of around 1.5 m/day as calculated by the MicroFem code. It was recognized that this would at least hamper/delay the freeze-up period. Based on additional MicroFem calculations, it was therefore decided to install so-called mirror-wells to reduce the hydraulic head gradient at the collar construction location, as illustrated in Figure 7. Monitoring records showed hereafter that almost no hydraulic head gradient existed at the ground freezing location, indicating that this measure is very effective.

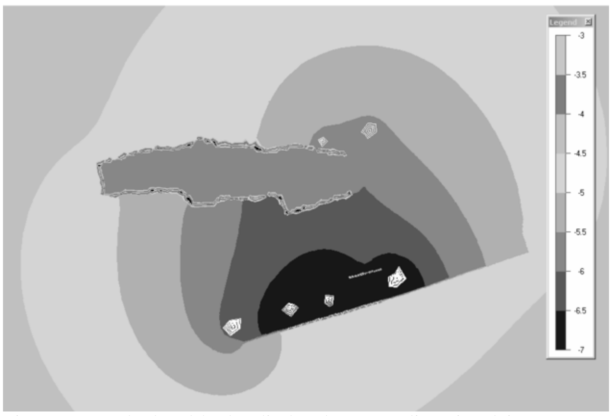

Figure 7 – Calculated hydraulic head contour lines in Pleistocene sand layer inside and around the building pit, including effect of the dewatering of the Weenatunnel (south) and mirror-wells (north).

However, during the second attempt to freeze-up the collar construction it appeared that closure was not achieved at least at one particular spot by end of June 2007, see Figure 8. Sensor T70_9 recorded temperatures stayed well above target value of -20 degrees Celsius at NAP -27 m in the Pleistocene sand layer, although the freezing process had been ongoing for more than eight weeks.

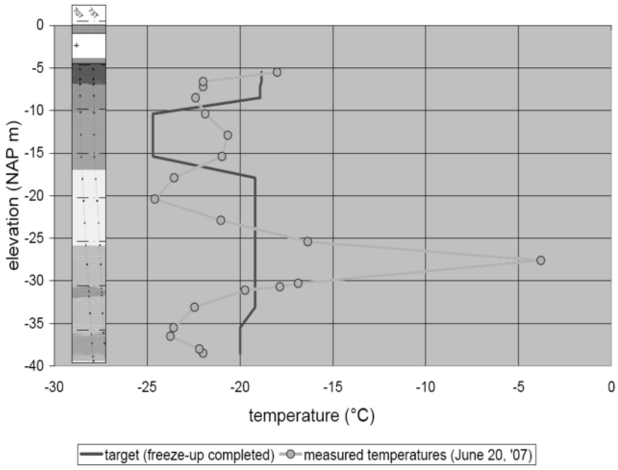

Figure 8 – Recorded temperatures (data by subcontractor MaxBögl) in monitoring casing close to diaphragm wall surface where the collar construction is generated. At NAP -27 m (sensor T70_9), temperatures (blue) are locally strongly deviating from target value (red) and temperatures at other depths, indicating closure has not yet been achieved there.

Detailed investigation of all available monitoring data revealed that an unexpected phenomenon had occurred. With help of the model calculations it became clear that, due to the local dewatering of the Weenatunnel project, the hydraulic head around the outer circumference of the building pit varied between NAP -4.5 m at the far west side of the building pit to NAP -5.8 m at the east side (collar construction) location. Consequently, this resulted in an average hydraulic head inside the building pit of around NAP -5.0 m induced by limited leakage through the diaphragm wall (between panel sections).

This implied that a rather steep hydraulic head gradient had developed at the collar construction location (NAP -5.0 m to NAP -5.8 m over a relative short horizontal distance), which effect was deteriorating in time due to the blocking effect of the growing (thickening) frozen soil columns around the freeze pipes. The blocking effect became noticeable in the monitoring data as off mid June 2007, as illustrated in Figure 9.

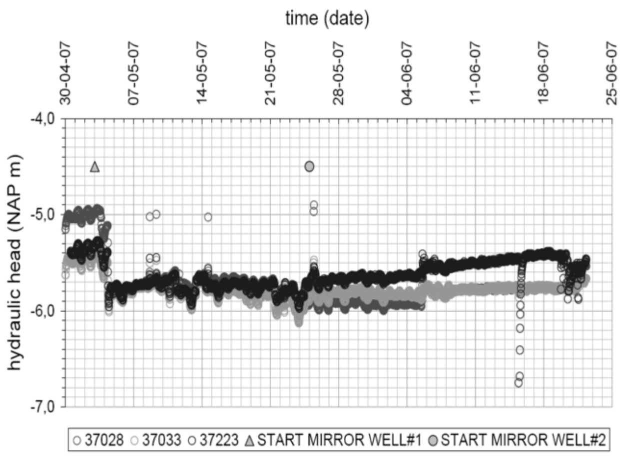

Figure 9 – Recorded hydraulic heads in time, showing a rising tendency inside the excavation (37223-blue) compared to outside (37033-green & 37028-red) due to the blocking effect. Note that the difference in hydraulic head outside the excavation (red at north vs. green at south) has been eliminated after activating mirror well#1.

As soon as it was recognized that this effect had almost certainly caused the delay, additional dewatering wells were activated inside the building pit in order to balance the hydraulic head to the level directly outside the building pit at the collar construction location. Again, the MicroFem code was used to determine a.o. the required pumping capacity. Already within a few days after activating of the additional wells, it showed that also the recorded temperature at monitoring sensor T70_9 dropped further below zero degrees Celcius, indicating that the closure had been reached at that particular location as well. Soon after that moment, monitoring of ground water pressures showed independent behavior of sensors inside and outside the building pit, indicating that freeze-up and closure of the collar construction had been completed.

3.3 *Watertightness of collar construction*

Excavation of the building pit to the final depth of 14 m (including lowering of the water table) could only proceed when the collar construction consisting of just frozen soil was declared to be 100% watertight, as the consequences of leakage through this part of the retaining wall (almost certainly leading to thermal erosion, failure of the frozen soil structure and severe damage) were not acceptable.

Thermal erosion occurs at a (remaining) gap in a frozen soil body, that cannot freeze-up due to too high groundwater velocities, and which is widening due to heat transport of the flowing water that is greater than the cooling capacity of the freeze pipes.

As it has been practically impossible to install a temperature monitoring grid that completely covers the collar construction wall area of about 1,600 m^2, and bearing in mind the freeze-up experience with many discussions on possible locations of unfrozen "hot-spots", all involved parties agreed on finding additional confirmation that the collar construction was undoubtedly completely watertight.

When a cylinder-shape frozen soil volume is generated, the watertightness of the frozen annulus is usually detected by a sudden increase of pressure of entrapped water inside the cylinder. However, for the collar construction, one had to realize that the frozen soil was only a limited section of the total retaining wall. Another aspect was that the diaphragm wall itself showed some leakage. This implies that the effect of the sudden pressure increase of entrapped water could not be used for detecting closure of the frozen soil body. Therefore, a special way of proving closure of the collar construction was required for these particular circumstances.

Basic concept of the adopted watertightness testing procedure is to generate the conditions that would certainly cause the thermal erosion to develop to such proportions that the effects will be noticeable in the available monitoring sensors.

Additional groundwater calculations with the MicroFem code revealed that an increasing gap resulting from thermal erosion would cause significant changes of monitored hydraulic head and water discharge volumes, see Figure 10.

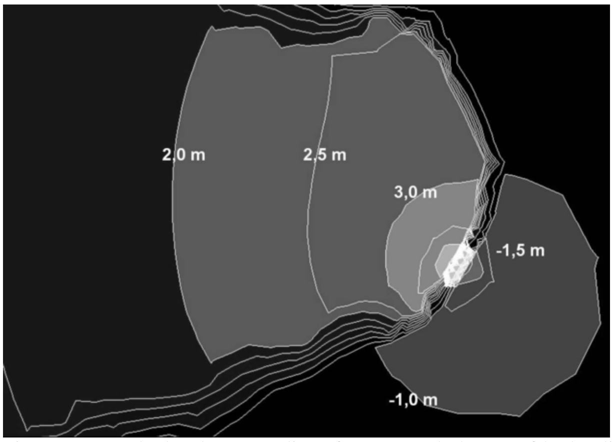

Figure 10 – Calculated contour lines for expected change of hydraulic head near gap in collar construction.

Additional thermal calculations as performed by CDM Consult GmbH have shown that thermal erosion will occur within a relatively short period of time, see Figure 11.

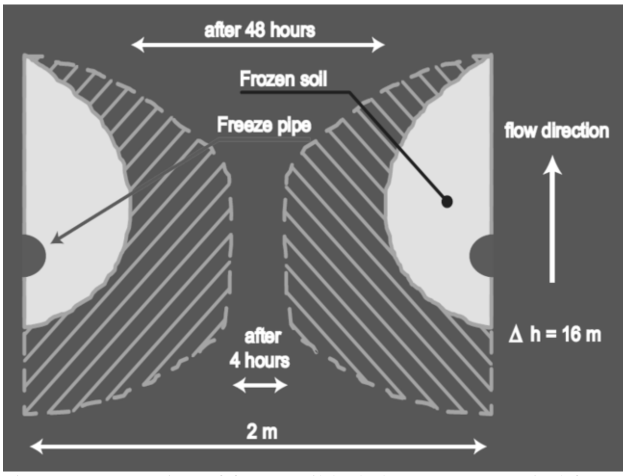

Figure 11 – Top view of frozen soil body between two vertical freeze pipes. Thermal erosion development is shown, 4 hours and 48 after introducing a large pressure gradient (difference of 16 m hydraulic head) over the initial gap (unfrozen soil) of 0.1 m in the collar construction.

The results of both types of calculation have been used to define the test set-up that facilitates free development of thermal erosion, if an initial gap should be present. For example, by generating a relatively large hydraulic head on one side of the collar construction, unlimited groundwater flow is ensured to have velocities remaining high even for increasing gap dimensions. Also, the test duration had to be defined as a sufficiently long period of time. It was concluded from the calculations that if no thermal erosion effects would occur within two weeks after introducing a large hydraulic head gradient, then 100% watertightness has been proven.

The watertightness test procedure was started after the initial observation that the collar construction was closed, based on temperature monitoring and thermal calculations. The testing protocol required daily interpretation of monitoring data, including approval by Rotterdam Public Works as well as contractor. As a result, the conclusion on 100% watertightness of the collar construction could be made immediately after the test period was completed, as none of the thermal erosion effects had shown up (see Figure 12). This allowed contractor to proceed with a.o. excavation works without further delay.

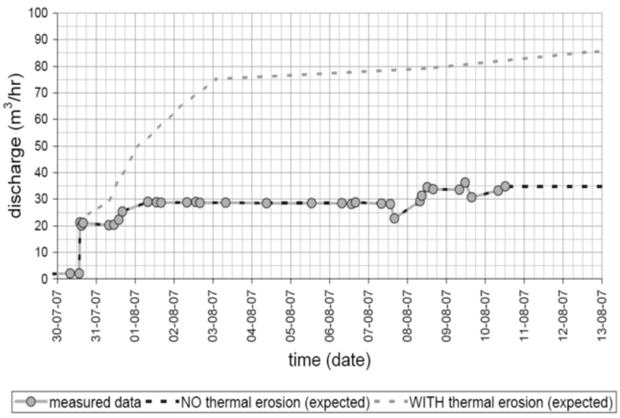

Figure 12 – Measured pumping rates for conditions without thermal erosion. In case of thermal erosion the measured values would progressively increase with time.

4 CASE HISTORY: DIAPHRAGM WALL LEAKAGE INCIDENT

Monday morning, 17 December 2007 a major leak occurred at a joint of the diaphragm wall at the moment that the maximum excavation depth of 14 m was reached. A huge amount of water and sand entered the building pit, Figure 13.

Figure 13 – Water and sand entering the excavation

Four rigs for installation of Tubex-grout injection piles under the existing metro station were immediately hoisted out of the building pit. Pumps to remove the water out of the building pit were installed, and the public area outside the building pit near the leak was closed off, because of rapidly developing ground surface settlements. The monitoring programme to check the possible deformations of nearby buildings was intensified.

The location of the leak is indicated in Figure 14. The distance between the location of the leak and the nearby monumental Groot Handelsgebouw is about 20 m.

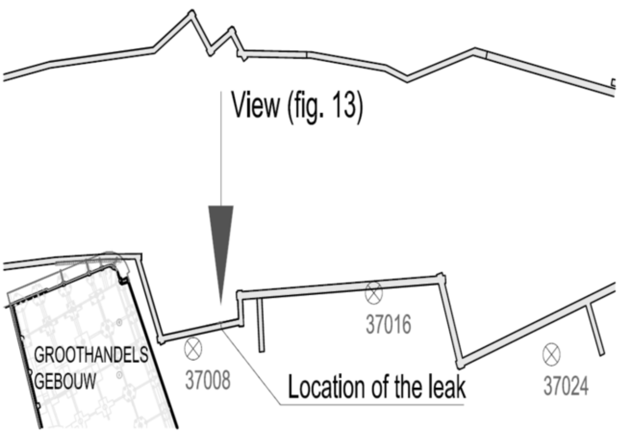

Figure 14 – Location of leaking diaphragm wall joint and of piezometers 37008, 37016 and 37024 (see also Figure 1 for orientation).

Almost immediately after the discovery of the leak, a PU foam was injected near the location where the leakage was visible and big sand bags were placed to limit the area with excess water in the building pit. The amount of water that entered the building pit was estimated to be 100 to 125 m^3/hour. It was suspected that the water was coming out of the sand layer below NAP -16 m. As the water entered the building pit at NAP -14 m, it was concluded that a short cut had developed between this sand layer and the bottom of the building pit despite the presence of rather impermeable clay and peat layers on top of it. The most probable short cut location was the joint between the diaphragm walls, see Figure 15.

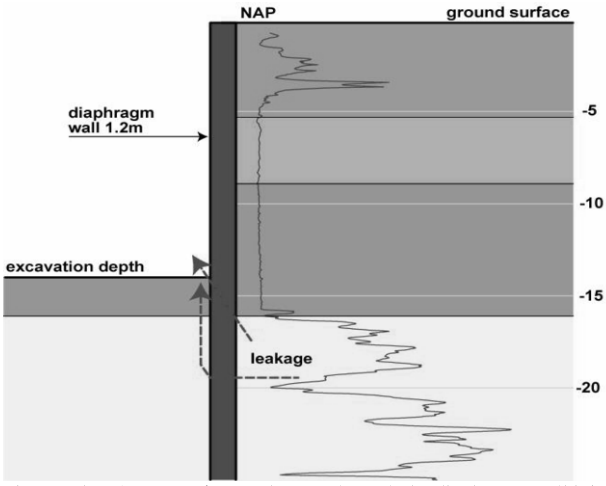

Figure 15 – Short cut of ground water through the diaphragm wall joint.

The construction of the nearby new Weenatunnel was at that time in operation. This building pit was surrounded by sheet pile walls. A dry excavation was made possible by using a dewatering system that decreased the water pressure head in the Pleistocene sand layer. As a result the hydraulic head near the leak before the calamity was about NAP -5 m. Piezometer 37008 nearby the location of the leak showed a sudden drop in the hydraulic head to NAP -9 m immediately after the break through. A recovery to NAP -6 m occurred during Monday due to the automatic correction of the Weenatunnel dewatering system, see Figure 16.

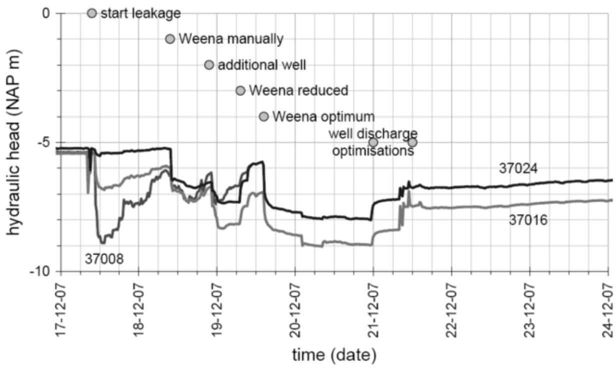

Figure 16 – Time recordings of hydraulic heads outside excavation in vicinity of leakage; piezometer identification numbers (37008 / 37016 / 37024) are also shown on Figure 14.

Tuesday 18 December 2007 it appeared that only injection of a PU based foam could not solve the problem. The PU foam was washed away due to the enormous amount of water entering the building pit. Efforts to put steel plates in place at the inner side of the diaphragm wall resulted in some reduction of the flow, but did not stop it.

The automatic correction of the Weenatunnel dewatering was stopped and the drainage was manually controlled from that moment. This allowed for futher lowering of the hydraulic head near the leak from NAP -6 tot NAP -7 m. Already on Monday it was decided to install two additional drainage wells outside the building pit to diminish the flow of water into the building pit and to increase the chances of stopping the flow by PU foam injection.

Wednesday morning 19 December 2007 the first drainage well at about 25 m distance of the leak was operational with a capacity of 100 m^3/hour. As a result the hydraulic head of the groundwater in the sand layer near the leak was lowered from NAP -6 m to NAP -9 m, see Figure 16. Because of this lowering, the flow through the leak was significantly reduced to about 50 m^3/hour, and the transportation of sand into the building pit had stopped. Due to the reduction of the flow, it was now possible to inject the PU foam effectively. As a result the leakage was stopped.

The hydraulic head near the leak was outside the building pit maintained at about NAP -9 m. It was recognised that this implied a certain risk to vulnarable foundations in the surrounding area. The piezometers at a distance of about 300 m of the drainage locations were therefore carefully monitored. Due to experiences of former projects a hydraulic head of NAP -5 m was considered to be acceptable at that distance for a period of not more than half a year.

The injection of the PU foam was considered to be a temporary measure for securing the watertightness of the joint between the diaphragm walls. However, due to the oncoming Christmas holidays the temporary measure had to be sufficient robust to prevent leakage during this period.

The final solution consisted of a sheet pile wall in front of the leaking joint, outside the building pit, at both sides connected to the diaphragm wall by jet grout piles. Both sheet piles and jet grout piles had to be installed to the same depth as the diaphragm wall: a depth of NAP -38 m. This was realised in the beginning of 2008, but it appeared to be impossible to install the sheet piles to the required depth. It was therefore decided to install a complete wall of jet grout piles around the leak (and the sheet piles). This did not result in a watertight secondary wall. Drainage tests showed that there were openings between the piles, apparently due to small inclinations of the piles. Fortunately, during the whole period the PU foam functioned well, and no new leakage occurred during the installation of the sheet piles and the jet grout piles. The pumping by the additional drainage wells was stopped at the end of March and the intensity of the monitoring programme was reduced. No leakage occurred during the remaining part of 2008.

During the leakage an estimated 500 to 600 m^3 of sand entered the building pit. This resulted in an extensive ground surface settlement in an area of about 25 x 25 m^2. Nearby the leak the ground surface settlement was estimated to be more than 2 m. Along the Groot Handelsgebouw the ground surface settlement appeared to be about 0.3 m, Figure 17.

Figure 17 – Settlement caused by wash-out of deep sand layer.

The Groot Handelsgebouw is founded on concrete piles with an enlarged toe. Because these pile toes only penetrate into the top of the Pleistocene sand layer, the foundation is vulnerable to the disappearance of sand in the top of the sand layer. However no settlement of the Groot Handelsgebouw was measured.

Afterwards CPT’s were carried out to check the ground conditions in front of the (closed) leak and along the Groot Handelsgebouw. No major differences were noticed compared to CPT’s carried out before the occurrence of the leak. The installation of the sheet piles in front of the leak had apparently densified the deep sand layer that was loosely packed due to the flow of water with sand.

5 CONCLUSIONS

The building pit for the reconstruction of underground station Rotterdam CS has provided unique experience for engineering issues governed by groundwater conditions. This is due to special techniques that have been used (a.o. ground freezing) and relatively long lasting dewatering activities. The presented case histories are illustrative, showing that detailed knowledge of the groundwater regime is essential for project design and execution. Within this context, the 3D groundwater model that has been developed for the region of Rotterdam is considered as a valuable tool.

The freeze-up of the collar construction appeared only to be possible by following a strategy that was based on the on-line measurements of the groundwater pressures at both sides of the wall. Closure was achieved by the combination of temperature measurements and use of drainage wells to minimize the flow of the groundwater along and across the almost frozen wall. A special pumping test was executed to prove that the frozen wall indeed was watertight, and thus no thermal erosion would occur during further execution of the works.

Also, closure and repair of a sudden gap in the diaphragm wall was made possible by the installation of additional drainage wells and extensive on-line monitoring to avoid damage in the surroundings.

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