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Full scale test on environmental impact of diaphragm wall trench excavation in Amsterdam

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ABSTRACT: If everything goes according to plan the construction activities of the deep underground stations for the new Amsterdam metro line will start next year. In this year the tender design will be completed. As a part of the stations' design a full scale test on the excavation of diaphragm wall trenches has been carried out to investigate the impact on the environment. The results of the test will be used to validate a 3D FE model. In this paper the topics of the test will be illustrated.

1 INTRODUCTION

The North/South Metro Line in Amsterdam, 9 km long, will connect the northern and southern suburbs with the city centre (de Wit, 1998). For reasons of protecting the historic city centre and restricting the disruption of city life by the construction activities to a minimum, special construction techniques will be applied. For the line sections this will imply a bored tunnel that follows the street pattern as closely as possible and is lowered to a great depth. Consequently the underground stations are situated at a great depth as well. The stations will be constructed at extremely busy locations in Amsterdam. At excavation depths of over 30 m, the building pits for the stations will be the deepest ever to be dug in The Netherlands.

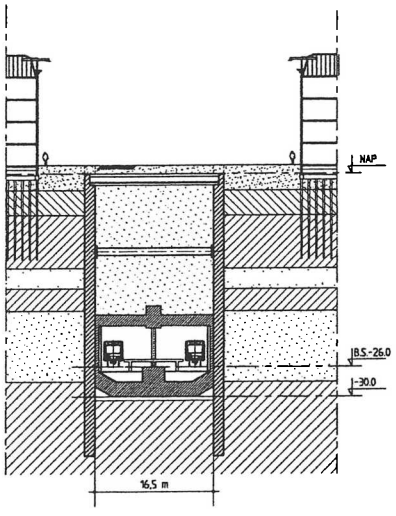

Figure 1a. Cross section of Vijzelgracht Station.

Furthermore high ground water levels and moderate soil conditions have to be considered as important environmental constraints. And last but not least the building pits are very near (3 to 5 m) to buildings of historical importance.

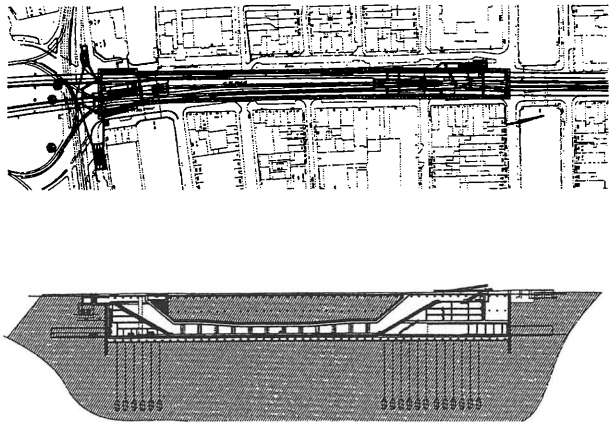

Figure 1b. Plan and elevation Vijzelgracht Station.

The underground stations will be constructed in a building pit of 1.4 m thick braced diaphragm walls reaching down to a level of over 40 m below surface level (figure 1). One of the most important aspects in the design of the underground stations is the influence of the construction on the built surroundings. This is of main importance because there are historical buildings closeby along a large part of the building pit. The following construction processes need to be examined:
- excavation of the diaphragm wall trench;
- excavation of the building pit.

To get a proper picture of deformations, it is necessary to use a program based on the Finite Element Method (FEM) or Finite Difference Method (FDM), in which the full building sequence with time schedule is processed. The excavation of the building pit can be analysed with a 2D FE model. The excavation of the diaphragm wall trench, however, is far more complicated because of the 3 dimensional behaviour. Little is known about the impact of diaphragm wall trenches' excavation on surrounding buildings. With the growing capacity of computers and the advanced constitutive 3D soil models that recently have become available, the 3D behaviour of diaphragm walls' trenching can be predicted. However, because of the new character of these features the link with construction in practice is quite limited. Since it is of great importance to know the impact of all construction activities on the adjacent historical buildings it was decided to carry out a research project on the diaphragm walls' excavation The project consists of the following three parts:

- prediction of the impact with a 3D FE model;
- full-scale test program at the Mondriaan Tower construction site in Amsterdam;
- validation of the 3D FE model based on the test results that have become available.

2 DIAPHRAGM WALL, CONSTRUCTION AND GROUND BEHAVIOUR

2.1 *Construction*

Diaphragm wall installation is carried out incrementally by the construction of individual panels to some planned sequence. The panels' dimensions can vary considerably depending on the design and the local circumstances. The construction of a diaphragm panel is carried out from surface level by means of a mechanical device such as a bucket grab (figure 2) or hydro fraise.

Figure 2. Excavation with a bucket grab.

A progressive excavation of a trench in the ground is allowed in such a way that stabilising fluid (bentonite) is introduced simultaneously as the trenching operation proceeds. Once the excavation is completed the reinforcement cage is inserted into the bentonite-filled trench. Furthermore the trench will be filled with concrete by way of tremie pipes, thereby displacing the bentonite from the bottom up.

2.2 *Stress Distribution*

To avoid instability of a single trench during excavation of a panel, the difference between bentonite pressure and water pressure must be equal or larger than the horizontal effective stress. Without 3D-effects in the ground instability will occur. These 3D-effects consist of redistribution of loads causing increasing stress in the surrounding soil and decreasing stress (beneath active state) along the panel. Stress transfer in the horizontal plane to both sides of the wall is referred to as arching. In the vertical plane stress transfer occurs to the toe of the wall and to relative stiff layers.

Directly after pouring the trench with concrete, the lateral pressure along the center of a panel increases to above the initial horizontal stress. A semi-3D FE-analysis on panels in the stiff London Clay demonstrates that the average horizontal stress behind a series of panels returns to its' initial K0-value (Ng. et al. 1995).

2.3 *Displacements*

Figure 3 shows field measurements on surface level along panels under various conditions on different locations throughout the world.

The vertical displacements at the centre of the panel can be neglected at 1 to 1,5 D away from the trench (D = depth panel). The maximum vertical settlement shows a large variation.

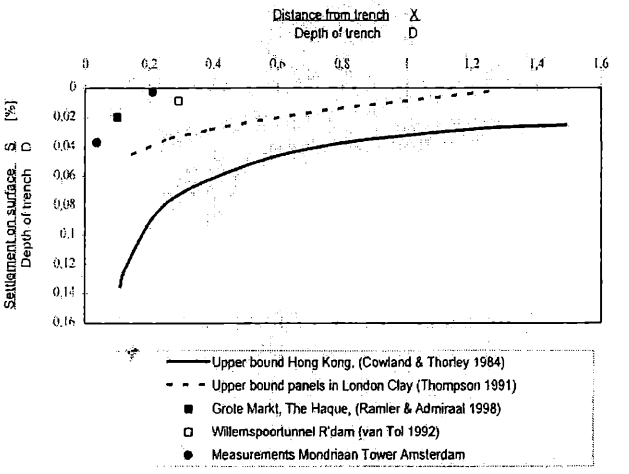

Figure 3. Field measurements on surface level.

3 FULL SCALE TEST

3.1 *General*

The objects of the full-scale test program at the Mondriaan Tower construction site is to monitor:

1. vertical and horizontal deformations of the ground adjacent to the excavated trench;
2. settlement of loaded piles;
3. impact on bearing capacity of piles; due to the excavation and concreting of succeeding diaphragm wall panels.

The test results regarding deformations of the ground is used to calibrate the 3D FEM model. The information regarding the pile tests will lead to direct insight in the effects on foundations.

3.2 *Geology, soil characteristics*

Surface level at the site is at + 2.0 m NAP. From the results of CPT's, borings and laboratory tests the geotechnical profile and characteristics of different soil layers were obtained. Some of the information is listed in Table 1.

Table 1. Geotechnical profile soil properties.

Type of soil	Bottom level (m NAP)	CPT (MPa)
Fill (sand)	-1.0	10-15
Clay, organic	-3.5	0.5
Peat	-7.0	0.5
Silty clay, organic	-13.0	0.5-1
Peat	-14.0	1.5
1st sandlayer	-18.0	8-30
Silty and clayey sand	-25.0	1-5
Clay (Eemclay)	-28.0	1.5
Sand	-42.0	10-30
Silt, clay	-51.0	3

The geotechnical profile is very similar to that at the locations of the future stations of the North/South line in Amsterdam.

Groundwater level is approximately NAP -0.4 m. The piezometric surface of the deeper aquifers is at NAP -3.0 m.

3.3 *Diaphragm wall and test location*

The future Mondriaan Tower will be a 100 m tall high-rise building. Underneath the office building a 2-storey underground parking garage will be provided. Diaphragm walls (length approx. 35 m) are implemented in the design both as building pit wall as well as structural wall for the underground parking. Some of the panels also serve as foundation elements of the building and have a length of 55 m. Thickness of all the panels is 0.8 m. In figure 4a the diaphragm walls at the test location and the layout of the instruments is illustrated.

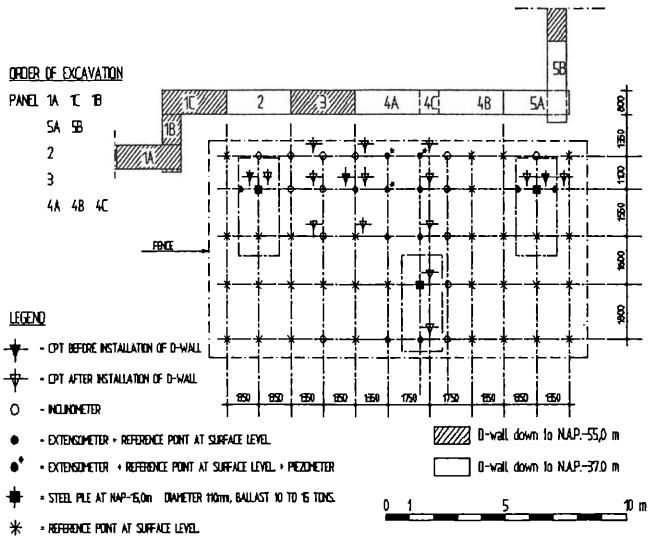

Figure 4a. Panel layout and layout of instruments.

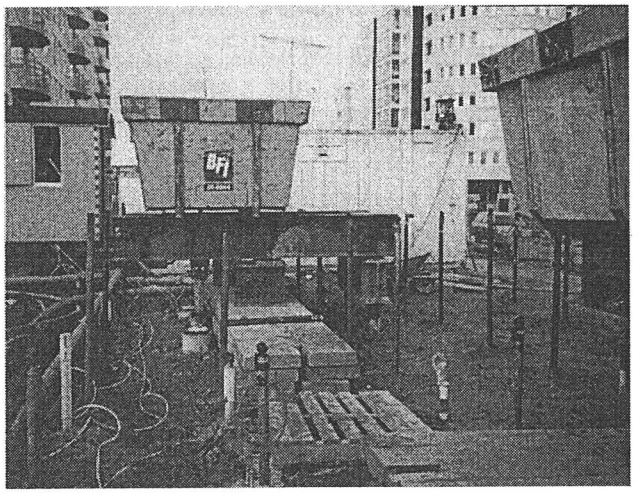

Figure 4b. Impression of the test site, the total station is visible in the background.

The sequence of excavation of the panels is also highlighted in figure 4a. The panels were excavated by hydraulic grab with a width of 2.7 m. The panel no's 2 and 3 were excavated in one course. The panel no's 1, 4 and 5 were excavated in 2 or 3 courses leading to different widths.

3.4 *Instrumentation and tests: deformation of ground*

It was decided to focus more on monitoring of deformations of the ground rather than on monitoring of changes in stresses. In earlier comparable tests (Teunissen et al. 1998) monitoring on deformations appeared to be more reliable.

To monitor the vertical deformation of the ground at surface level 48 reference points were placed in the testing area on a regular grid. These reference points were continuously monitored with a nearby total station. Underneath 11 of these reference points electronic extensometers were installed. With the extensometers the vertical deformation of the ground at deeper levels can be monitored. The extensometers were located at -15.0, -31 and -51 m NAP corresponding with respectively the 1st, the 2nd and the 3rd sand layer. Results of both the total station and the extensometers were stored in a datalogger. During the period of excavation of the panels 1 to 5, which lasted about 3 weeks, each of these instruments was monitored at least once in every 20 minutes. It was possible to monitor movements of 0.1 mm, which appeared to be sufficient to visualise the effects of the excavation on the surrounding stratum.

To monitor horizontal deformations 14 tubes for inclinometers were installed. The 70 mm tubes were placed in bore holes similar to those for the extensometers. The inclinometers were monitored by hand on specified moments, i.e. before excavation, during excavation at specific depths, after excavation and immediately after concreting each panel.

In two panels 3 piezometers were installed at different depths. This was realised by attaching piezometers to the reinforcement cages. The instruments allowed to monitor the changes in pressure of the bentonite and concrete in time. The wet concrete pressure is hydrostatic only to a certain depth beneath the concrete surface, the so-called "critical depth" (Lings et al. 1994). The critical depth is mainly influenced by the type and temperature of the concrete mix and by the rising velocity of the concrete table. In this way the lateral pressure of the wet concrte, which appeared to be an important parameter in the FE-model, could be determined

To investigate if relief of original stresses within the stratum has occurred CPT's (Cone Pressiometer Test) were carried out before and after the installation of diaphragm walls. Before excavation 3 CPT's were carried out as a reference. Some weeks after the test 6 CPT's were carried out at different distances from the diaphragm wall.

3.5 *Instrumentation and tests: bearing capacity and settlement of piles*

Most of the historical buildings in Amsterdam are founded on wooden piles. These piles were driven into the first sand layer at about NAP -12.0 m. The working load is usually in the range of about 10 to 15 tons. The ultimate bearing capacity of such piles varies between approximately 15 to 25 tons.

Before any excavation of diaphragm wall panels 3 piles with a diameter of 110 mm were driven into the first sand layer. The 3 piles were located in such a position that they were at a distance of approx. 1 panel width away from panel 2, 4 and 5 respectively. On each pile bearing capacity tests were carried out before and after the excavation of the diaphragm walls (Fig.5). The results of the test before excavation will be compared to the results of the test which was carried out after installation of all the diaphragm walls. The results of the tests as well as the predictions based on CPT's are listed in Table 2.

Table 2. Data pile tests and working load.

Pile tests	Pile 1 (kN)	Pile 2 (kN)	pile 3 (kN)
Prediction by CPT (min. max)	245-380	175 –300	175-300
Ultimate load capacity, test before	220	185	150
Ultimate load capacity, test after	360	280	270
Working load during excavation	150	120	100

From the tests before excavation the amount of ballast on the piles during the installation of the diaphragm walls was defined. It was decided that the amount of ballast in relation to the bearing capacity of the pile was to be similar to that of a representative wooden pile, i.e. 70% of the bearing capacity. The applied working load on the piles is listed in Table 2.

The piles with ballast on top were now considered to be representative for piles under typical houses along the North/South line.

During excavation the settlements of the piles were monitored by placing reference points on top of the pile and by sighting these points with the previously mentioned total station. Settlement of the pile can be compared with the settlement of the ground stratum monitored by the nearest reference

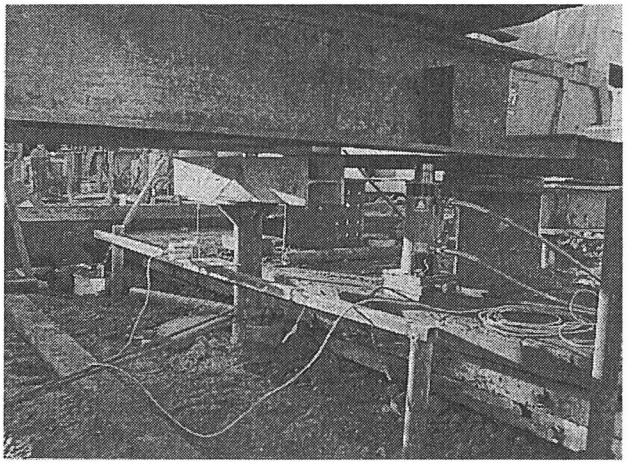

Figure 5. Pile testing

point at surface level and the nearest extensometer at the pile toe level. In this way it is possible to investigate if the settlement of the pile is larger than the surrounding stratum. This would indicate loss of bearing capacity.

After the excavation the ultimate bearing capacity test was repeated for each pile. By comparing these results with the first tests also a possible loss of bearing capacity is obtained.

4 MODELLING OF THE DIAPHRAGM WALL INSTALLATION PROCES

4.1 *Finite element mesh*

For the calculation of ground deformations and stress distribution during diaphragm wall construction, the finite element program. Two analyses are made :

1. the installation of a separate single panel with varying width and depth;
2. the installation of a number of successive panels to investigate the so called wall mechanism (semi 3D).

The mesh is built of 3D triangle-elements with 15 nodes and 6 gauss points. For the calculation of one diaphragm wall panel (2,7*0,8*35 m) a mesh with approx. 3000 nodes has been used (figure 6).

In the top-view of the mesh (50*80*50 m) only a quarter of the panel is modelled, which means that there are two planes of symmetry: in x, z- direction.

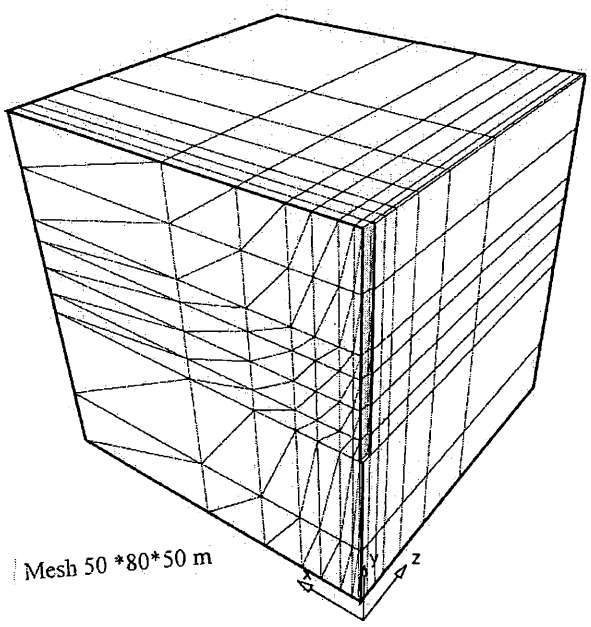

Figure 6. 3-D Finite element mesh

Between the level of NAP -1m and NAP -14m several thin (holocene) layers of silt, clay and peat were found (see table 1). The piles are founded in the 1st sand layer (NAP - 14/-18). Since the holocene layers have a minor effect on deformation of the 1st sand layer (and deeper layers) the holocene layers are modelled as one layer with equivalent material properties. All other layers as described before are modelled separately.

4.2 *Numerical models*

For the prediction all layers are modelled as a linear elastic material with the Mohr-Coulomb yield surface. Advantages of this model in relation to non-linear models are the use of simple model parameters, relative short calculation time and the interpretation of the results is normally easier to perform. The Mohr-Coulomb model does not take into account the variation of stiffness with depth and with strain. Therefore, in the validation calculations (back analyses), layers which have a large influence on the deformations of the piles are modelled as non-linear materials with cap- or friction hardening.

The elements representing the cohesive holocene and Eemklei layer are modelled with Poissons' ratio of 0,5 to simulate undrained behaviour. All other elements are modelled as drained.

4.3 *Modelling procedure*

The construction of the diaphragm wall is modelled using the following stages:

I. Excavate a single trench by switching the soil elements off and simultaneously, applying the bentonite pressure on the faces of the trench.

II. Fill the trench with concrete by increasing the lateral pressure. The lateral pressure of wet concrete can be described by a bilineair relation.

$$\sigma_h = \begin{cases} \gamma_c \cdot z. & z \leq h_{crit.} \\ \gamma_b \cdot z + (\gamma_c \cdot h_{crit}) & z > h_{crit} \end{cases}$$

In the calculations the critical depth h_{crit} is varied between 5 and 15 m.

III. To model the hardening of the concrete, the elements in the trench are switched on, and the stiffness and volumetric parameters are changed to those of concrete

IV. Construct the adjacent panels, one at the time, using the same procedure. Thus, the effects of the construction of individual panels (semi 3D) can be investigated. For the analyses the installation of three panels is considered, which in fact means six panels taken into account the planes of symmetry. Although this sequence is not quite corresponding to those on the test site, the effects of the different panels on each other (wall mechanism) can be investigated.

5 RESULTS

5.1 *Full scale test*

At this moment the interpretation of the test results and the validation of the FE model by means of back analyses is still going on. However some interesting first results will nevertheless be presented.

Ground movements:

- The targets on surface level show a minor settlement during excavation of the trench (stage I). While concreting (stage II) an instant heave of surface level occurred, that starts decreasing almost immediate after finishing concreting. Possibly the fairly light cohesive top ground layers are pushed aside / up by the lateral concrete pressure. Measurements in the trench during and after the concreting show the critical depth of approximately 10 m. Because of the undrained behaviour, the cohesive soils are not compressed but displaced. This assumption is confirmed by the inclino measurements that show little or no horizontal movement during excavation and a horizontal ground movement away from the trench during the concreting phase
- At the end of the test, after excavating and concreting five panels (stage IV), the maximum settlement on surface level, about 1.5 m away from the trench, is 4 to 10 mm (figure 7). The maximum settlement of approx. 10 mm is found at at the corner panel and the Z-shaped panel. A settlement of approx. 3mm is found at the 6 m wide panel, where the heave caused by the lateral concrete pressure is supposed to compensate the impact of excavating the panel. Further away from the trench (5 to 7.5 m) the measured vertical movements vary between + 1 mm (heave) to –1 mm (settlement).
- On a deeper level of 15 m below surface level the extenso meter results show maximum settlements that are within the same range as on surface level. Apparently the larger settlement one would normally expect on surface level is compensated by the heave of the top soil layers caused by the lateral concrete pressure. However when comparing the deeper levels (15, 30 and 50 m below surface level) the expected picture of a, with depth reducing impact, by diaphragm wall installation is confirmed.

Pile settlements and pile tests:

- The maximum settlement of the piles within a distance of 2.5 m from the trench was about 5-7 mm and appeared to be more or less the same as the vertical ground movement. This indicates that the bearing capacity of the piles is not affected by the diaphragm wall installation, which was confirmed by the pile tests afterwards.
- The settlement of the pile at a distance of about 5 m to the trench was negligible.
- The pile tests afterwards resulted in a higher bearing capacity. This can be explained by a contribution of the holocene layers that are pressed against the piles by the lateral wet concrete pressure. In fact the conclusion can be drawn that no decrease of bearing capacity is caused by diaphragm wall installation.

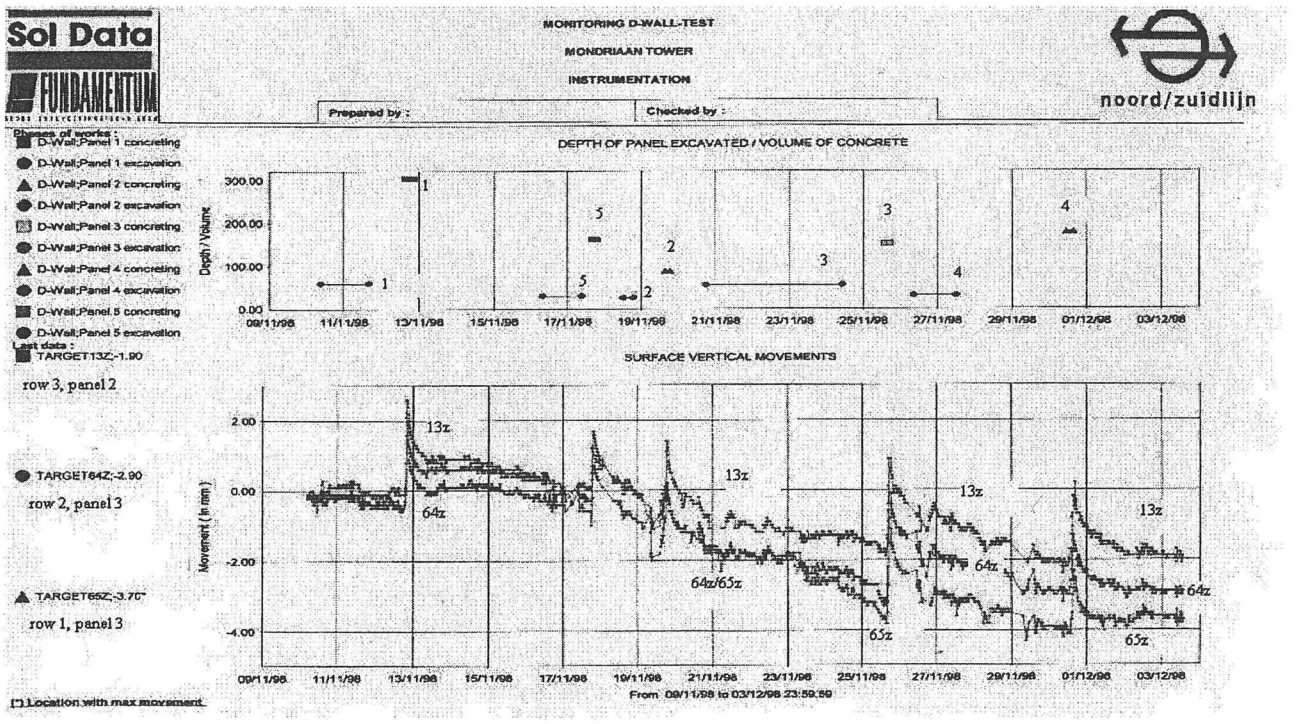

Figure 7. Settlements on surface level.

5.2 *FEM predictions.*

5.2.1 *Single panel*

Figure 8 shows the horizontal effective stress, at a depth of NAP - 15m, during the different stages (I, II) of a single panel. The horizontal stress (K0 situation) is about 70 kPa. In the bentonite stage, the stress at the centre of the panel decreases to about 45 kPa. At the edges of the panel the stress increases to above the K0 situation. After concreting, the stress at the centre of the panel increases significantly. At the corner a decrease of effective stresses will occur.

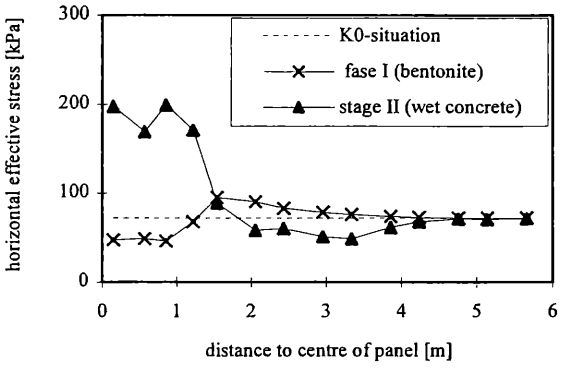

Figure 8. Effective horizontal stress in different stages (x=1,3, y=NAP –15m).

The horizontal and vertical displacements mainly depends on the width of the panels. In case of a narrow panel (B=2.7 m) the computed displacements in the bentonite stage at a level of NAP-15m (1st sandlayer) are –4 mm horizontally and –8 mm vertically (figure 9 and 10). Computations on a wide panel (B=6.2 m) result in a 100 percent increase of displacements, both horizontally and vertically. The vertical displacements at the center of the panel can be neglected at 0.3 D, which is confirmed by the test results.

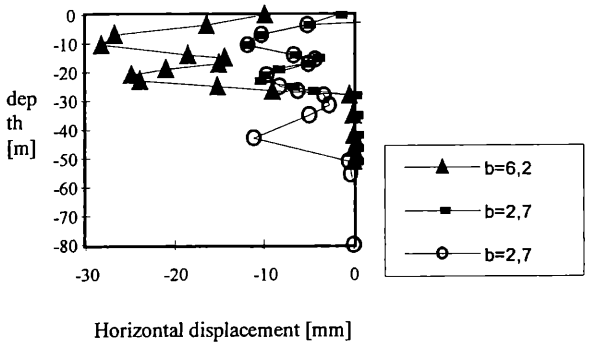

Figure 9. Horizontal displacements at centre of single panel in bentonite stage (x =1,3m, z=0m)

Due to the vertical load transfer to relative stiff layers (in this case: the sand-layers at NAP -14m and NAP -30m), the displacements in layers beneath those stiff layers decrease and are no longer of influence on displacements of the top layers (figure 9). Therefore, the influence of depth on horizontal and vertical displacements is very small.

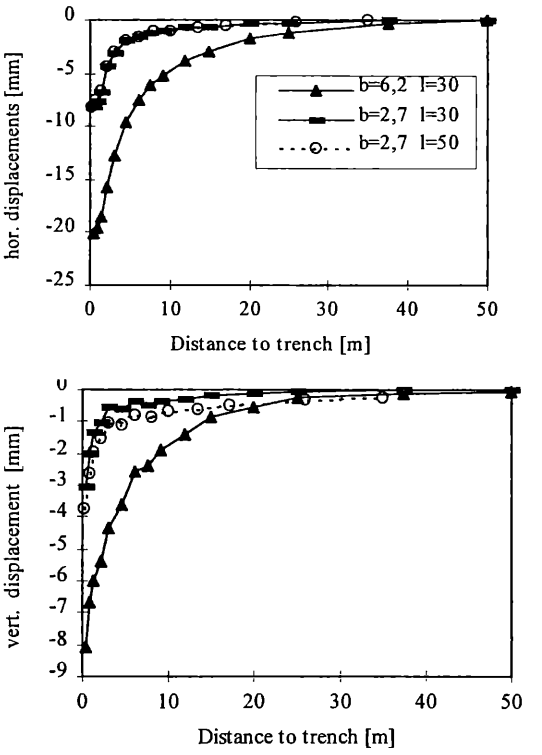

Figure 10. Deformations 1st sand layer in relation to distance to trench. Varying panel width and panel depth (x=1,35m, y=-15,5m).

In stage II (concreting panel) the pressures invert from unloading to reloading. At a level of NAP –15 m this increase of horizontal stress results in a lateral movement of ground away from the panel and in a vertical upward movement (figure 11).

Due to the activities in stages I and II respectively settlement and heave occur. In stage III (concrete hardened), the weight of the concrete panel is activated, which actually already happens in stage II. This increase of weight, caused by replacing soil by concrete results in settlement (in fact reduction of heave) of the panel and the surrounding soil. This deformation partly depends on the modelling of the interface between the wall and the soil stratum. With the assumption of a rigid interface the average total vertical displacements nearby the diaphragm wall, on the level of the first sand layer, are reduced to less than 5 mm (heave). Due to consolidation of the cohesive layers, in reality further settlement will occur, which is confirmed by the test results.

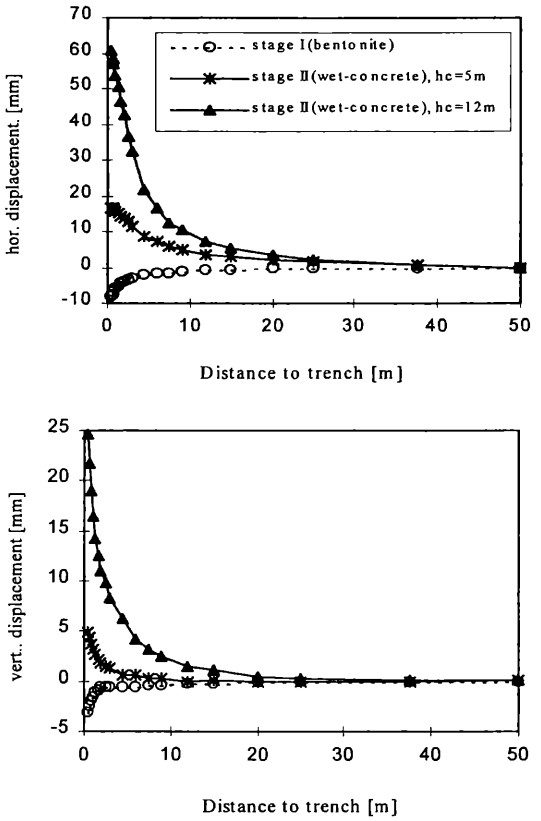

Figure 11. Displacements (stage I). Varying critical height of concrete (x=1,35m, y=-15,5m).

5.2.2 *Wall mechanism*

After the excavation of an adjacent panel, the horizontal stresses at the centre of this panel is decreasing. At the edge at the preceding panel, no significant increase of horizontal stress will occur. At the other edge of the panel, increase of horizontal stress will occur as shown in figure 8. Due to the concreting of the panel the horizontal stress in the preceding panel decreases significantly. This means that the increase of horizontal stress after concreting of a panel is temporary, caused by the compensating effect of concreting the next panel.

6 CONCLUSIONS

The main goal of the diaphragm wall test being a confirmation that the installation of a diaphragm wall has a minor effect on the surroundings and no significant impact on the piles of nearby foundations was satisfied. Secondly in relation to field measurements throughout the world (figure 3) one can conclude that:

- The test results at the Amsterdam Mondriaan Tower are at the left corner of the graph, which means relatively small settlements.
- The monitored settlement trough at the Amsterdam Mondriaan Tower is relatively steep; the width of the trough is approximately 0.3 D instead of 1.0 D.
- Heave occurs at the Amsterdam Mondriaan Tower caused by the lateral wet concrete pressure. Not known is if similar effects have occurred on other locations.

Finally a prediction by means of a 3D FE model is made to describe the diaphragm wall installation. This model used for the predictions proved, apart from consolidation effects, to be able to predict the deformations caused by the succeeding stages in the installation of the diaphragm wall. Mainly because of consolidation effects of the cohesive soil layers the test results in the final stage don't quite fit the FE predictions. In general the FE predictions result in larger deformations than monitored at the test program.

In the time to come the FE model will be validated/improved by means of back analyses. These results will be presented on a later occasion.

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