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The response of piled buildings to deep excavations

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ABSTRACT: This paper explores the influence of the piled foundation on the building response to excavation-induced deformations. The influence of the type of foundation, the position of positive and negative skin friction zones, and the flexibility of the piles is evaluated with respect to both horizontal and vertical soil deformations. Case histories from the Netherlands are included from Amsterdam (North South Line) and Rotterdam (a building adjacent to the Willemspoortunnel). Most of the buildings are founded on timber piles ranging in length from 12–17 m. Conclusions are drawn about the interaction between the piled building and the soil deformation.

1 INTRODUCTION

The response of piled buildings near deep excavations is governed by the effect of the deep excavation on the soil, the interaction between the soil and the pile and the interaction between the pile and the building. In general, the unloading effect of the deep excavation will lead to deformations and changes in stresses behind the wall. Due to these excavation-induced changes, the nature of the interface between the pile and the soil changes.

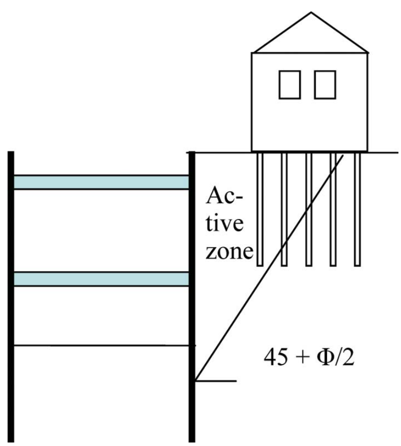

Figure 1. Active zone behind the wall.

This means that the soil around the piles is subject to vertical settlements and horizontal displacements, similar to the ground surface. In stress terms the vertical and horizontal stresses around the pile decrease. Outside the active zone (see Fig. 1 in the case of sandy soils) the stresses are assumed to remain approximately constant. For end bearing piles which settle less than the surrounding soil, negative skin friction may develop in the active zone.

Some general aspects of the behaviour of piled buildings due to horizontal and vertical ground deformations can be found in the literature, for example related to tunnelling-induced deformations.

2 PILE RESPONSE TO TUNNELLING

Jacobsz et al. (2005) describe the settlement predictions and measurements for the effects of tunnelling on piled structures for crossing of three bridges on the Channel Tunnel Rail Link project. Their cases indicate that a difference is found between end bearing and friction piles. End bearing piles were found to follow the green field settlement at the pile base for small volume losses. A reduction in the pile base capacity and subsequent load (due to stress relief caused by tunnelling) will result in the mobilisation of additional positive shaft friction. The soil and pile will settle the same amount as the neutral level. Friction piles alter the green field subsurface displacements and can be assumed to follow more or less the surface settlements as a conservative approach.

Chen et al. (1999) present an analytical method to determine the influence of tunnelling on a single pile or a pile group, which led to the following conclusions related to pile response due to tunnelling:

– Lateral pile deflections are very similar to the soil deflections.
– The pile-head settlement is less than the maximum vertical soil movement.
– Tunnelling below the pile tip leads to smaller bending moments and deflection of the piles than for shallower tunnels. The behaviour of a pile is influenced significantly by whether the pile tip is above or below the tunnel axis.
– Disturbing effects on the piles should be added to the existing loading and stresses in the piles.

Kaalberg et al. (2005) describe the results of an extensive programme in the Netherlands to find the influence of tunnelling on piles, for which measurements and a field test were performed at the Second Heinenoordtunnel. They showed that deformation of piles due to shield tunnelling consists of settlement of the soil layer around the pile toe plus settlement caused by stress relief around the pile toe.

The following similarities and differences between the effects due to tunnelling and deep excavations have to be taking into account when assessing the response of piled buildings to deep excavations:

– The general movement of the pile towards the tunnel or excavation and downwards is similar.
– An approximately triangular zone of influence behind the wall or beside the tunnel is expected.
– Below the mid height of the tunnel, the ground settlements reverse to heave, which is probably not the case for deep excavations.
– The 3D effect for a passing TBM is not present at deep excavations, but other 3D effects such as those arising from corners of the excavation or the limited size of an adjacent building could still be significant.
– Installation effects of retaining walls present additional changes of stress in the case of deep excavations (unloading in case of excavation for diaphragm wall or bored piles, loading due to concrete pressures or densification due to vibrations).

3 PILE RESPONSE TO DEEP EXCAVATIONS

Very few papers describe the response of piled buildings due to excavations, with some exceptions shown here.

Davies & Henkel (1982) describe the construction of Chater Station in Hong Kong. It is made clear that piled foundations adjacent to deep excavations can experience serious settlements due to negative skin friction caused by the lowering of the water table. The settlement of the building behind the wall during excavation was about 1.5 times the maximum deflection of the wall.

Lee et al. (2007) describe the result of a building damage assessment for a multi-propped excavation in Singapore. At 6 m behind the diaphragm wall, a 12-storey building founded on 37 m long piles settled less than 10 mm (30% of the surface settlement) due to the presence of the pile foundation. Since the deep excavation was 15–17 m deep, this effect is probably caused by an increase in negative skin friction leading to some (minor) pile tip deformations.

Negative skin friction development is an important contributor to piled building response. The concept of pile response to negative skin friction is reported by Fellenius (2006) based on long term measurements of piles. He describes the concept of the pile moving with the neutral plane at the location of the force equilibrium between positive and negative skin friction.

Elshafie (2008) has studied the effect of deformations caused by deep excavations on buildings with experimental modelling. Elshafie concludes that buildings with individual spread footings experience large differential settlements, because footings outside the zone of influence do not follow the influenced part of the building. This results in significant distortions and tensile strains concentrating at the weak parts of the buildings.

Conclusions for piled buildings adjacent to deep excavations based on both tunnelling and deep excavation related papers are:

– Piled buildings tend to follow the soil deformations at the neutral plane, leading to a difference in response of end-bearing and friction piles; end bearing piles follow the soil deformations at pile tip level, friction piles as a conservative approach relate more to ground settlement at the surface.
– Piles might settle by an extra amount due to negative skin friction caused by soil settlement due to draw down of the ground water and/or diaphragm wall installation.
– Stress relief around the pile tip can lead to mobilisation of positive shaft friction.
– The presence of the piles influences the subsurface ground displacements.
– Piles are rather flexible in horizontal loading and tend to follow the soil deformations.
– Disturbing effects on the piles should be added to the existing loading and stresses in the piles.

Several questions remain specific for the response of piled buildings to deep excavations, such as the effect of installation of the retaining wall. It is also not clear to what extent the presence of the piles will strengthen and reinforce the soil. To address some of these questions three cases of piled buildings adjacent to deep excavations are presented in this paper.

4 CASE STUDY 'WITTE HUIS' ROTTERDAM

4.1 *Introduction and soil conditions*

In a case study by Brassinga & van Tol (1991) the response of a piled high rise building to a deep excavation in Rotterdam is presented. Additional information can be found Sarlemijn et al. (1993), who report on the construction of the Willemspoortunnel. The 'Witte Huis (white house)' 11-storey high rise building, dating from the end of the 19th century is located at 10 m distance from the 18 m deep excavation.

The soil conditions are rather typical for Rotterdam and the Western part of the Netherlands. The high phreatic level is found at 1–2 m below ground level (NAP +3.0 m). About 2–7 m of sand is found as the top layer, underlain by clay and peat layers to a depth of about 20 m (NAP −17 m). Below that the Pleistocene sand layer, commonly used as a foundation layer for piles, is found. Figure 2 shows a typical CPT.

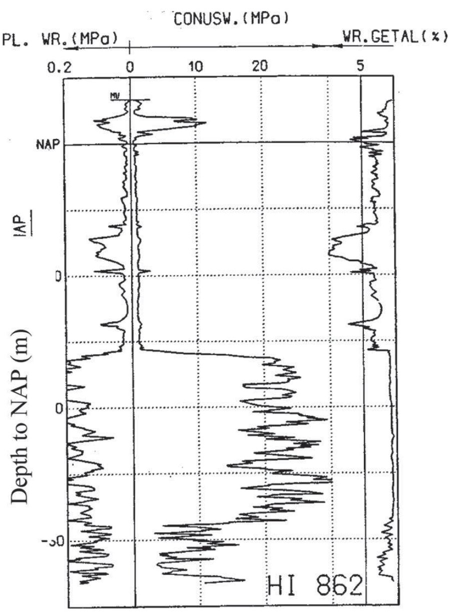

Figure 2. Typical CPT near Witte Huis.

4.2 *Details of the Witte Huis foundation*

The building is founded on wooden piles in the Pleistocene sand layer, with the diameter of the piles being about 250 mm. The design drawing shows in total about 750 piles for the foundation, placed in rows of 2–4 under the main walls, with a centre-to-centre spacing of 450–500 mm. Almost 9% of the total foundation area was filled with piles. During installation of the piles, a significant heave of the ground level of about 1.0 m was reported (Wikipedia).

Brassinga & van Tol (1991) report that the foundation had a low factor of safety before the deep excavation started. However, in the past no significant differential settlements of the Witte Huis had been noticed.

4.3 *Characteristics for tunnel construction*

Near the Witte Huis the cut-and-cover tunnel excavation crosses a canal from the harbour, leading to unequal loads on both sides of the excavation, see Figures 3 and 4.

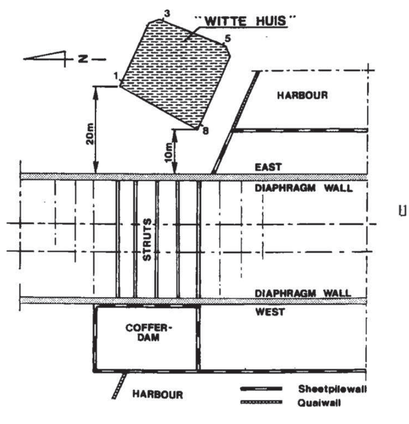

Figure 3. Plan view near the Witte Huis (Brassinga & van Tol, 1991).

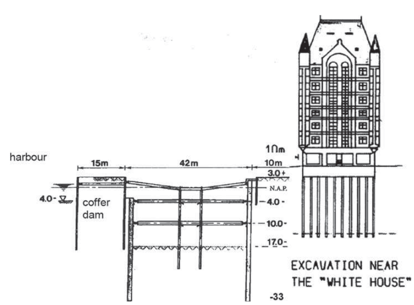

Figure 4. Excavation near the Witte Huis (Sarlemijn et al. 1993).

This caused horizontal displacements of the building. The excavation depth reached 20 m (NAP –17 m), just at the base of the pile foundation of the Witte Huis.

4.4 *Response of the 'Witte Huis'*

The building settled 6 mm during the D-wall construction, 12 mm in total including the first excavation phase (to NAP –4 m), and after extra prestressing of the struts an additional 5 mm (17 mm total) during the rest of the excavation. See Figure 5 for the development of the settlements of the four corners of the building with times.

Since no soil deformation was measured near the Witte Huis, it was calculated by FEM using the observed wall deflection as input. Vertical settlements of the building occur due to the combination of the ground deformation of the pile tip level and an increase in negative skin friction leading to increased pile tip load. Due to a low factor of safety of the original pile foundation, the neutral level is found clearly above the pile tip (where negative and positive skin friction meet). The measured deformation of the building is compared with the calculated green field settlement over the soil depth in Figure 6.

After the first excavation stage the settlement of the Witte Huis was 6 mm at measuring point 8 and 5 mm at point 1. Vertical soil displacements of this amount calculated using FEM were found at NAP –12 m at both distances from the wall for measuring points 8 and 5. This is assumed to be the level of the neutral point, which was used for prediction for the further excavation stages, see Figure 6.

It should be noted that the calculated ground settlement with depth in the soil turned out to be very sensitive to the assumed stiffness of the struts. This makes it difficult to define the neutral level accurately.

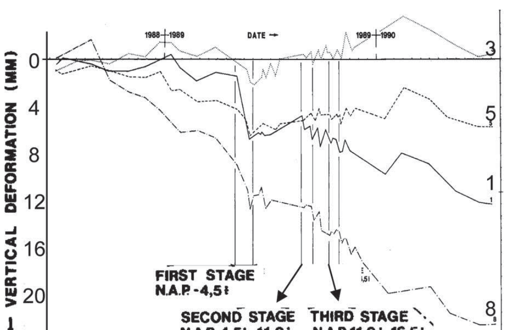

Figure 5. Vertical deformation versus time for Witte Huis (Brassinga & van Tol, 1991). Numbers relate to the settlement markers shown in Figure 3.

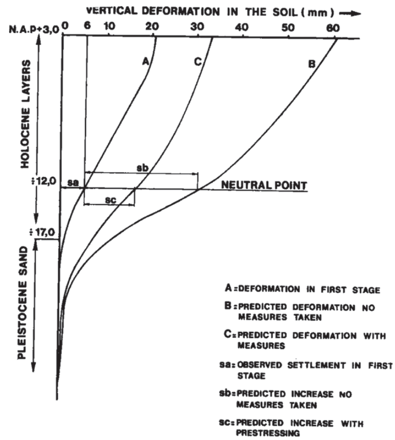

Figure 6. Calculated soil deformations with depth Witte Huis (Brassinga & van Tol, 1991).

Deformations after this stage (between August and November 1989) for the building were 10–12 mm. This value compares well with the calculated values based on the assessed neutral level, which was 11 mm.

4.5 *Analysis of the building response*

Figure 7 shows the settlement of the building at the points 1, 3, 5 and 8 at two stages: halfway down the full excavation depth and at full depth. Based on the regular distance between the lines of equal settlements, it is concluded that the building mostly tilted, without sagging or hogging deformation. The tilt is not exactly in the direction of the deep excavation due to the unequal loading on the harbour side, where the ground level is at NAP –4 m. Due to the extra struts in the second stage of the construction, the tilt at this stage is directed more towards the diaphragm wall.

The measured deflection is very small. No actual damage due to the deep excavation was observed in the building.

Horizontal deformation data are not available anymore, but deformations were larger than expected due to the soft response of the cofferdam on the opposite site of the excavation. No damage was observed as a consequence, so it is likely that the building moved rather uniformly. The maximum calculated horizontal deformation at point 8 was 20 mm after stage 1 and 25 mm at the end of construction. If the rear of the building did not move at all, the resulting horizontal strain would have been $7*10^{-4}$ and $1.2*10^{-3}$ respectively, based on the FEM results. The L/H ratio of the building is 20 m/43 m = 0.47. Combined with negligible deflection ratio this would have resulted in 'very

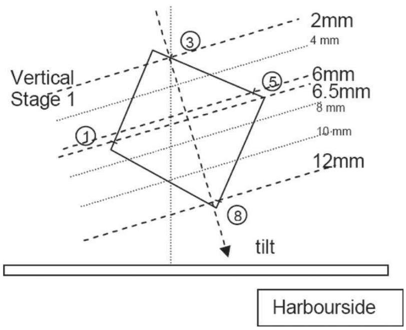

Figure 7a. Top view with vertical deformation of Witte Huis halfway excavation with direction of tilt indicated.

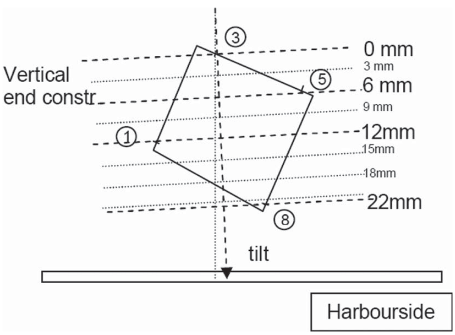

Figure 7b. Top view with vertical deformation of Witte Huis and at the end of construction (below) with direction of tilt indicated.

slight' or 'slight' damage respectively (Mair et al. (1996) and Cording et al. (2001)).

4.6 *Stiffness of the building*

The building shows no signs of bending in the structure at all. This means it is relatively stiff compared to the soil. The relative bending stiffness, defined as the stiffness of the building relative to the stiffness of the underlying ground, is compared using different methods. The concept of relative bending stiffness is described for tunnelling by Potts & Addenbrooke (1996) and Franzius et al. (2006). Since these require a depth of the tunnel, they can not easily be used for deep excavations. Recently, Goh (2010) and Goh & Mair (2011) adjusted the relative bending stiffness for deep excavations to:

$$\rho^*_{Goh} = EI/E_s L^3 \qquad (1)$$

where EI is the building stiffness, E_s a representative soil stiffness and L the length in either hogging or sagging.

The bending stiffness of the building can be assessed by taking into account the base slab, the floors and/or the walls and the amount of interaction between them. In this case the masonry walls, with a width of over 1 m in the basement and about 0.4 m at the top of the building, provide most of the stiffness. Based on an average width of 0.6 m, a height to the roof of 30 m and 2 walls in the 20 m building, the EI of the building would be $810*10^6$ kN m^2/m. If a correction is made for the openings in the wall, based on Dimmock & Mair (2008), the actual stiffness would be about one order of a magnitude lower at about $80*10^6$ kN m^2/m. (If only the stiffness of the slab and the floors is considered, the EI would be about a thousand times smaller at $100*10^3$ kN m^2/m.)

The estimated relative bending stiffness values are calculated and presented in Table 1 and Figure 8.

Table 1. Calculated (relative) bending stiffness Witte Huis.

Description	Calculated value
Number of storey's	11 + basement, floor slabs 0.2 m
Width building	$20*\sqrt{2} = 28$ m
Length of building	$20*\sqrt{2} = 28$ m
Foundation	Slab cement/masonry 0.3 m + piles
E soil	$E_{0,01\%} = 3$ MPa soft clay
E slab and floors/wall	10 GPa/6 GPa
I building m^4/m:	
Slab + floors	$1\ m*0.3^3/12 + 11 \times 0.2^3/12 = 0.01$
2 walls 0.6 m	$2*b_{wall}/b_{building}*H_{wall}^3/12 = 2*0.6/20*30^3/12 = 135$
$\rho^*_{Goh} = EI / E_s L^3$	
Slab + floors	$1.5*10^{-3}$
2 walls 0.6 m	1.2 (including reduction for openings)

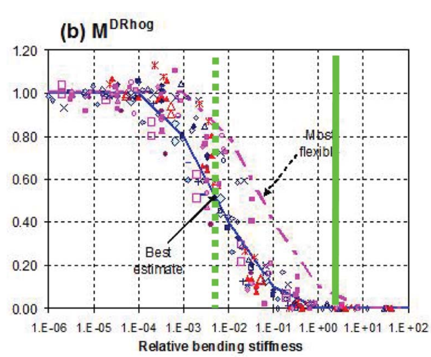

Figure 8. Design curves from Goh (2010), modified modification factor for hogging with results for Witte Huis in green lines (solid for walls, dashed for slab and floors).

In reality very little hogging deformation was observed, which is consistent with the estimated relative bending stiffness based on the walls (allowing for openings) leading to a modification factor of 0. The interaction based only on the individual slabs and floors underestimates the relative bending stiffness for this building. Methods for frame structures such as presented by Goh (2010) and Potts & Addenbrooke (1996) are not considered realistic for this type of masonry bearing structures.

4.7 *Conclusions*

The deep excavation mainly caused tilt in the Witte Huis. The amount of differential settlement (and associated deflection ratio) was negligible and so was the corresponding damage. The building shows no signs of bending in the structure at all. This means it is relatively very stiff compared to the soil, which is confirmed by Goh (2010) for the upper bound of EI values calculated for the building based on the stiffness of the walls (allowing for openings).

The effect of the pile foundation could only be evaluated using FEM results, because no deformations of the wall were measured locally. These calculations indicate that the piles follow the soil deformations at about NAP –12 m, while the bearing layer is situated at NAP –17 m. The neutral point is thus assumed at that level. The level of the neutral point did not change between stage 1 and the end of construction, based on the FEM analysis.

Horizontal deformations were larger than expected due to the translation of the whole excavation in the direction of the cofferdam.

5 CASE STUDY NORTH-SOUTH LINE

5.1 *Deep excavation and soil conditions*

The North-South Line in Amsterdam passes under the historical centre of the city in twin tunnels. Five underground stations are under construction. Ceintuurbaan Station is one of the deep stations in the historic city centre. The station is 220 m long, only 11 m wide and a maximum of 31 m deep. It is built by means of a top down construction, with 1.2 m thick diaphragm walls extending to a depth of NAP (Dutch reference level) –45 m. Details of the construction and soil profiles can be found in Kaalberg et al. (2005).

Fill and soft Holocene deposits are present to a level of about NAP –11.0 m (ground level around NAP +1.0 m). These are underlain by the 1st sand layer. The 2nd sand layer is found at about NAP –16 m, extending to NAP –25 m. Below the 2nd sand layer a stiff clay layer of around 15 m thickness (the Eem clay) is underlain by the 3rd sand layer.

A cross section of the excavation, soil profile and monitoring instruments is shown in Figure 9. The monitoring instruments include extensometers behind the wall, inclinometers in the soil and in the wall, manual levelling of the surface and the buildings and automatic monitoring of the buildings. Details about the construction and monitoring of Ceintuurbaan Station are given in De Nijs & Buykx (2010).

The buildings considered, see Figure 10, are built with masonry walls, wooden floors and timber pile foundations, reaching into the First Sand Layer at about 12 m below the surface level. Based on several pile load tests in the historic centre it is known that the wooden pile foundations have low factors of safety of around 1.0, if negative skin friction is assumed over all but the first sand layer.

Over a period of about 8 years, preparations for the construction and the subsequent excavation of the deep station to a depth of about 25 m took place, see Table 2.

The preliminary activities include raising the ground level (≈0.7 m), diaphragm wall construc-

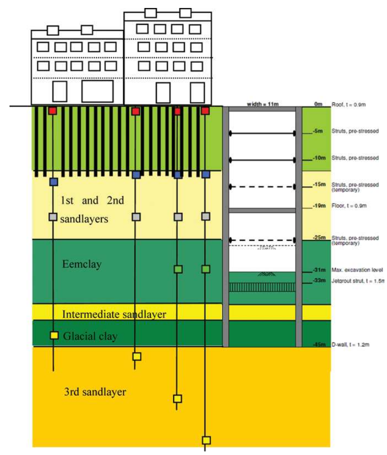

Figure 9. Cross section of Ceintuurbaan station with soil profile and extensometer locations.

Figure 10. Historic buildings at Ceintuurbaan station (dated 1880–1920).

Table 2. Construction activities and dates for Ceintuurbaan.

Construction activity	End date
Base monitoring start 2001	2003-11-01
Preliminary activities	2007-04-01
Excavation to NAP –6.2 m	2007-09-13
Excavation to NAP –15.3 m	2007-12-10
Excavation to NAP –19.4 m	2008-03-01
Excavation to NAP –24 m	2008-08-01
Excavation to NAP –25.6 m, floor construction, pumping test	2009-06-24

tion, jet grout strut installation, excavation to NAP –2 m, construction of the roof, refill on the roof and a pumping test for water tightness of the D-wall in the 1st and 2nd sand layers.

5.2 *Response during construction*

The building deformation and the soil deformations are compared in Figure 11 for the final date in Table 2.

Ground surface settlements measured by ground points (manual, 'GroundSurface') and extensometers (automatic, 'ExtensoSurface') are similar, but not exactly the same. This might be caused by the construction of the instruments itself, such as the size and weight of the base plates and some axial stiffness of the extensometer instruments in the borehole.

The settlement of the piled buildings (Leveling N and Leveling S) is larger than the ground settlement at pile tip level (ExtensoNAP –12 m), but smaller than the ground surface settlements. This indicates mobilization of positive shaft friction over parts of the 'soft layers'. The neutral level is found well above the foundation layer. For the buildings presented the neutral level is found at about NAP –4 m to NAP –9 m, assuming a linear variation of settlement between ground surface level and pile tip level.

Figure 12 shows the development of the building deformation during the construction stages presented in Table 2. The largest settlement of the buildings is due to the preliminary works and background settlement over 4 years between 2003 and 2007. During excavation of the station, the additional building deformations are less than 10 mm. Relative rotations are small (of the order of 10^{-4} or smaller).

Figure 13 shows the horizontal deformations of the building for both the North and South facades and for the MidHeight and Toplevel (respectively at heights of about 5–6 m and 12–13 m from street level). The amount of tilt can be derived from the difference between the prisms at these levels. This difference is 3–7 mm, leading to a tilt of 1: 1000 to 1:2000. Extrapolating this tilt results in a horizontal

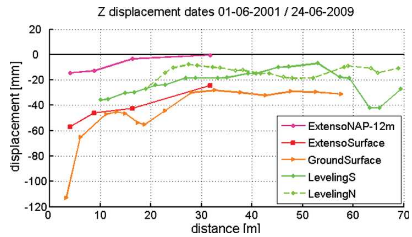

Figure 11. Building and soil deformations for North and South façade at Jan van der Heijdestraat East.

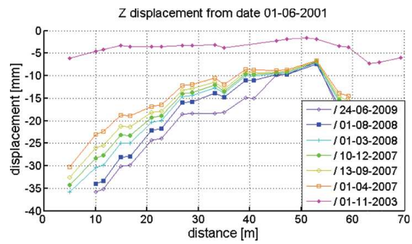

Figure 12. Building deformations for different construction stages for South façade with distance from the diaphragm wall perpendicular to the station.

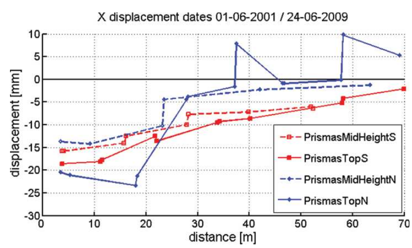

Figure 13. Horizontal building deformations (North and South facades) with distance from the diaphragm wall perpendicular to the station.

extension of 8–12 mm at street level. This is about 25–50% of the vertical settlement of the building at that point.

The horizontal extension diminishes to zero at a distance of about 28 m (north façade) and 52 m (south facades) respectively. The resulting average horizontal strain is of the order of 0.02–0.03%. Horizontal deformations of the soil have been measured automatically, but the results can not be presented here due to accuracy problems with the readings.

5.3 *Conclusions*

The settlement of the piled buildings is larger than the ground settlement at pile tip level, but smaller than the ground surface settlements. This indicates a neutral level above the pile tip level.

The largest settlement of the buildings is due to the preliminary works and background settlement over 4 years between 2003 and 2007. During excavation of the station, the additional building deformations are less than 10 mm. Horizontal building deformations at street level are about 25%–50% of the vertical building settlement.

6 CONCLUSIONS

Some of the conclusions for piled buildings adjacent to deep excavations set out at the start of this paper are confirmed by the case studies presented, as follows:

- Piled buildings tend to follow the soil deformations at the neutral level. Usually it is assumed that the building follows the ground deformation at the tip of the piles. The case studies presented in this paper confirm that it would be better to focus on the neutral level of the piles. For piles that are completely end-bearing in a competent stratum, this level would be the same as the tip level. For all other pile foundations the deformation should be calculated at a higher level, which depends on the equilibrium of the piles.
- Piles settle due to negative skin friction and soil settlement due to draw down of the ground water and/or diaphragm wall installation may cause a significant part (or even the main influence) of the overall construction effect.

Case studies with piled buildings adjacent to deep excavations are rare, especially information related to pile deformation and/or soil deformation at depth. The North-South Line project presents possibilities to investigate the vertical interaction, but unfortunately not the horizontal interaction, due to the limited availability of reliable inclinometer data.

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