Starting and end shafts of the Sophia railway tunnel

Le puits de départ et le puits de extrémité du tunnel de chemin de fer Sophia

R.A.Schiphouwer – *Heijmans Engineering, Rosmalen, the Netherlands*
J.W.Bosschaart & A.L.J.Timmermans – *M.U.C. Engineers, Terheijden, the Netherlands*

ABSTRACT: The Sophia railway tunnel in the West of the Netherlands is at the moment under construction as part of the Betuwelijn railway line, a freight line from the port of Rotterdam to Zevenaar near the German border. The tunnel is located near Papendrecht and is 8115 meters long, crosses two rivers, two highways (the A15 and the A16), eleven local roads and the railway line Rotterdam-Dordrecht. The tunnel consists of 3875 meters of ramp sections (traditionally built, open and closed box sections) and 4240 meters of bored tunnel sections with a slurry shield. The bored tunnel sections consist of two tubes with an inner diameter of 8.65 m. The newly developed continuous advance method has been used to construct the Mixshield. In this paper the design of the shafts at which the bored tunnel starts and ends will be discussed from a geotechnical point of view.

RÉSUMÉ: Le tunnel de chemin de fer a été situé en l'ouest de Pays-Bas est en fase de réalisation. Ce tunnel consiste à une partie à forer d'une longueur de 4.250 mètres et 3.875 mètres d'ouvrages en tranchees (une partie ouverte et l'autre couverte). Le tunnel réalise un passage sous les autoroutes A15 et A16, onze routes departementales, la ligne de chemin de fer Rotterdam-Doredrecht vers le sud et la voie navigable entre Rotterdam et Allemagne. Le procédé de foirer a été réalise continuellement sans interruption pour fixer les éléments préfabriqués du tunnel. Le tunnel consiste de deux tubes de 8,65 mètres de diamètre intérieur. Cet article présent la conception géotechnique des deux puits.

1 SOPHIA RAILWAY TUNNEL

The tunnel was awarded by the NS Railinfrabeheer B.V. Mangementgroup Betuwelijn to the consortium Tubecon I v.o.f. on the basis of a design and construct contract for € 298 million. Tubecon I v.o.f. consists of the contractors Royal Boskalis Westminster N.V., Heijmans N.V., NBM Amstelland N.V., Hochtief AG and Phillipp Hozmann AG.

The construction of the Sophia tunnel is to finish in 2003 and the Betuwelijn railway line will open in 2005.

The principal feature of the continuous advance method is the possibility of simultaneously advancing and erecting rings, i.e. the interruption of the advance during the ring building is omitted. With the continuous operation of the TBM the trust jacks can be controlled in semi-automatic and automatic modes.

2 STARTING SHAFT

2.1 *Layout*

The starting shaft of the Sophia railway tunnel consists of a building pit with a width between 29 and 34 m and a length of 65.8 m. Both the bored tunnel tubes are for the greater part located at a depth in excess of NAP –25.0 m (the tunnel then has a cover of 7 m in the Pleistocene sand underlying the soft top layers). This depth will guarantee a stable operation of the TBM-shield. However, in order to start as shallow as possible, the soft top layers along the sections directly in front of the starting shaft and the end shaft have been excavated and have been replaced by sand fill. The depth of the starting shaft is thereby reduced to 18.7 meters.

In Figure 1 an aerial photograph shows the ramp sections behind of the starting shaft and the soil improvement in front. Ramp section 16 is being excavated at the time of taking the photograph, so the dewatering is not yet started. The starting shaft itself nevertheless is already dewatered. The sand fill in the soil improvement is being placed with hydraulic equipment at time of the photograph. After finishing both the bored tunnel tubes, the starting shaft will be transformed into an underground control center for the tunnel.

Figure 1. Construction of the starting shaft.

2.2 *Soil profile*

The soil profile at the starting shaft is characterized by a very soft clay/peat top layer of 11.5 m (Holocene), which overlies 16.0 m of medium dense sand (Pleistocene), which in turn overlies a clay/peat layer of 7.7 m and finally a medium dense sand (the latter two belonging to the Kedichem formation). Typical for the West of the Netherlands is the very high ground water table at NAP –2.3 m (i.e. ground level –0.8 m).

Figure 2 shows a typical CPT at the starting shaft with a schematic soil profile.

2.3 *Vertical equilibrium*

The contract was based on a reference design that used slurry walls (1.2 m thick) until NAP –36.0 m, an underwater concrete floor (1.3 m thick) at NAP –20.2 m and twelve slurry wall sections until NAP –36.0 m in the center of the building pit acting as tension piles. Tubecon proposed a combined wall, large diameter pipe piles with infill of sheet pile walls, and a reinforced underwater poured concrete floor.

Instead of using tension piles or anchors, vertical equilibrium of the floor is provided by lowering the groundwater table inside the sand layer between NAP –20.2 m (base of the underwater concrete floor) and NAP –29.0 m (top of the deeper Kedichem clay/peat layer) to NAP –21.0 m. This is generally referred to as the polder principle. By using the polder principle, the hydraulic uplift pressure will not act directly on the underwater concrete floor, but will act on the base of the Kedichem clay/peat layer at NAP –36.7 m. The (measured) seepage through this clay/peat layer and the combined walls during construction is limited to 3 m^3 per 24 hours, partly due to using water sealing gel in the sheet pile interlocks and by incorporating a cautious sheet pile driving protocol.

By using the nett weight of the underwater concrete floor, the nett weight of the soil layers within the building pit between NAP –20.2 m and NAP –36.7 m and the wall friction along the combined walls, sufficient down force is mobilized to ensure vertical equilibrium, see Figure 3.

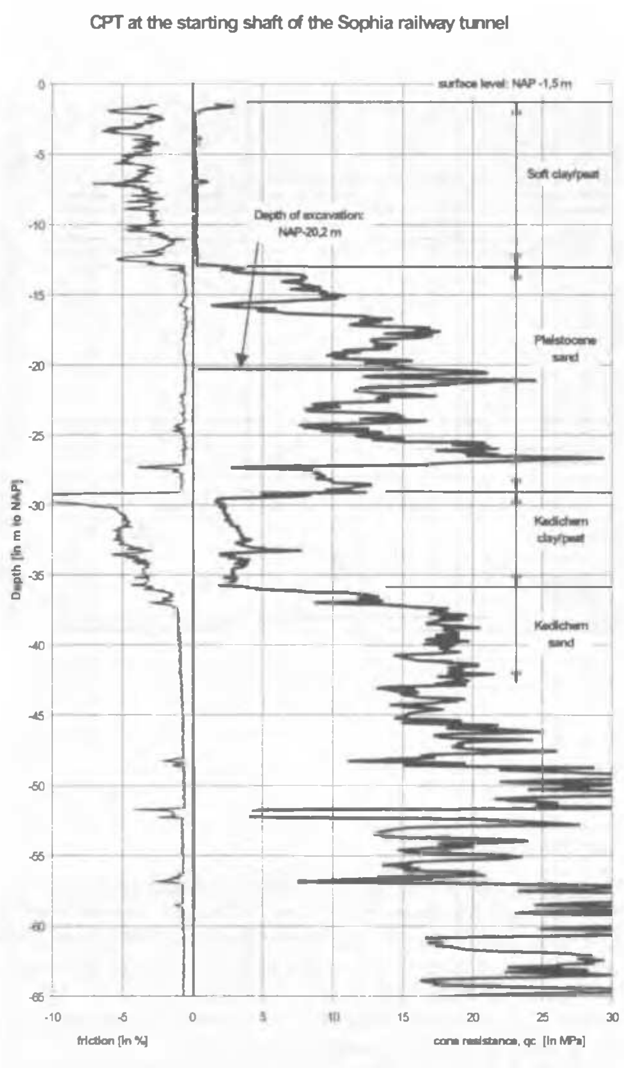

Figure 2. CPT and soil profile at the starting shaft.

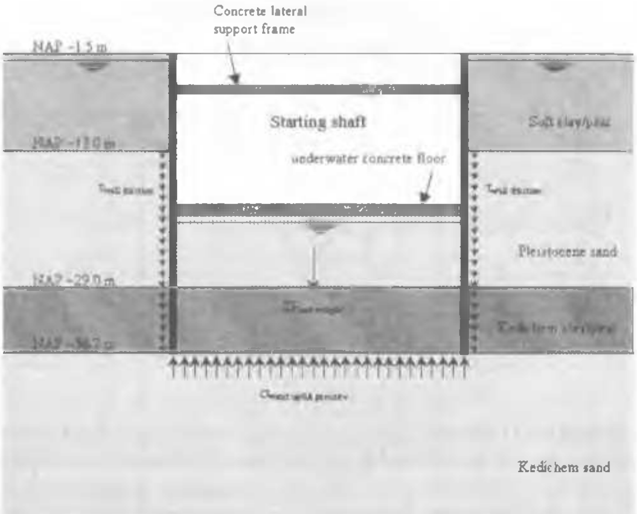

Figure 3. Vertical equilibrium of the starting shaft in construction phase 14 (excavated and dewatered).

2.4 *Deformations of the combined wall*

During construction of the starting shaft the deformations and the equilibrium of the shaft were monitored. The entire construction process of the building pit was observed using inclinometers attached to the combined wall and by measuring the deformations of the supports and the top of the combined wall. These measurements were compared to the predicted deformations of the wall made with an analytical model (Technosoft version 4.00 sheet pile wall analysis) and a Finite Element Method program (Plaxis version 7.1). By validating the measured deformations with the predictions, it proved feasible to follow the construction process and to determine the safety margins of the building pit. This observational method was also used to further optimize the deformation predictions for future construction phases. The possibility of optimizing the design of the end shaft with the data recorded at the starting shaft was an important motivation for installing the inclinometers. In Figure 4 the results of the predictions are compared with some of the measurements of the two representative inclinometers (at the center of the building pit).

A significant difference in the two predictions is for example that in the analytical model the combined wall bends inwards (below NAP –25.0 m) after construction phase 3, while in the FEM-prediction the lower part of the combined wall hardly deforms at all. This difference is attributed to the fact that in the analytical model all calculations of the numerous construction phases are based on drained soil behavior, a common assumption in analytical sheet pile analysis. In the FEM-analysis however, the construction phases are calculated taking into account the soil permeabilities and a time-line consistent with the estimated construction planning. Some construction phases therefore are fully drained, some are partly drained some are almost entirely undrained. A separate Plaxis-analysis with fully drained soil behavior in all construction phases provides a more comparable results with the analytical model.

Another significant observation that can be drawn from the charts is that the stiffness of the combined wall in the predictions appeared to be too low in the top section, particularly construction phase 12. After finishing the predictions it was decided that, in order to increase the vertical equilibrium, the pipe sections should be filled with concrete to NAP –14.5 m. The resulting increase in bending stiffness of the combined wall had not been taken into account in the prediction, which would explain the difference between the measured and the predicted lateral deformations.

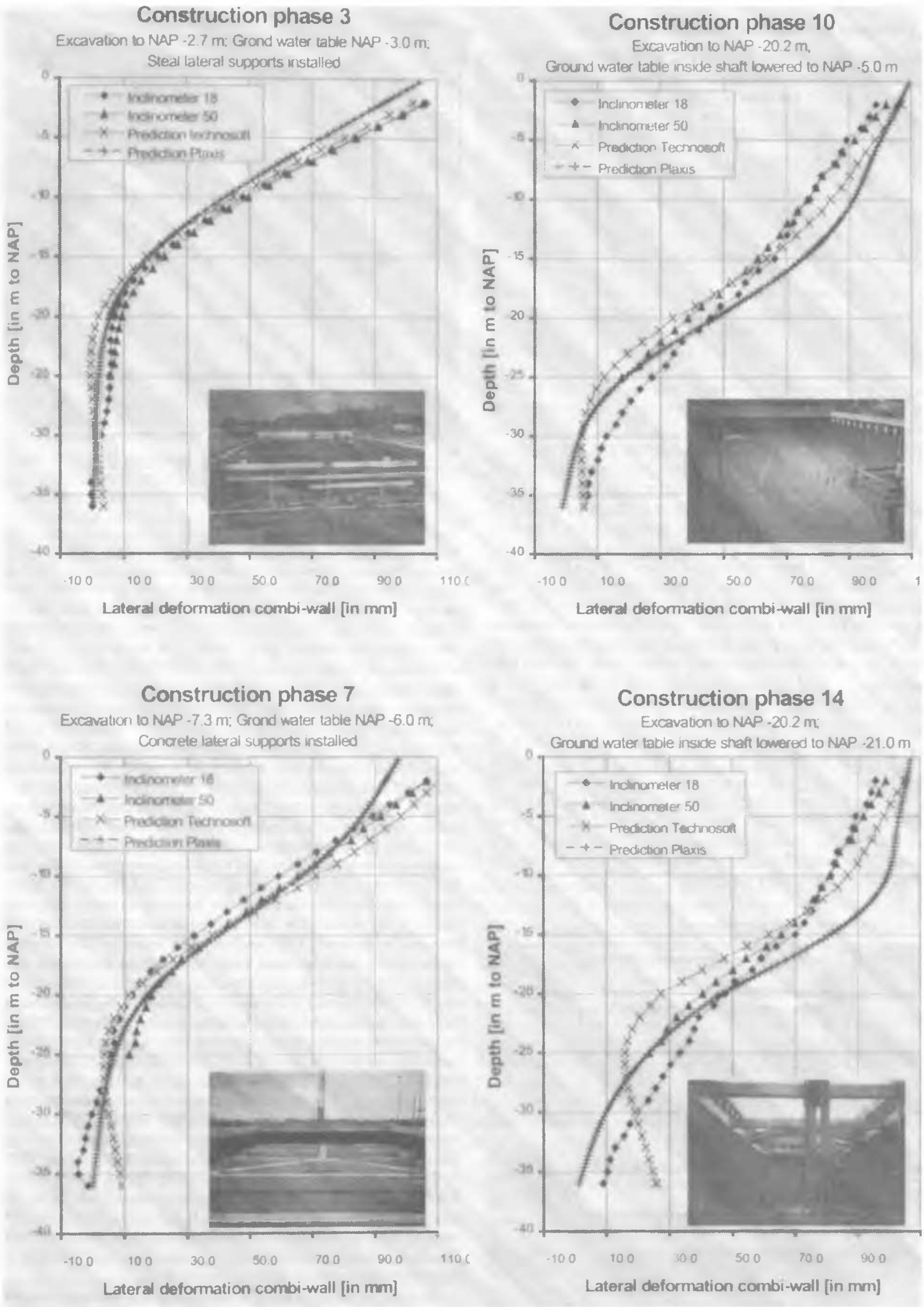

Figure 4. Measured versus predicted lateral deformations.

2.5 *Horizontal and rotational equilibrium*

An important phase in the construction of the starting shaft was to lower the ground water table in ramp section 16 (the first section directly behind the starting shaft). In order not to impose any restrictions to the construction process it was preferable to finish the ramp simultaneously with the boring of the two tunnel tubes. Finishing the ramp could of course only be carried out after lowering the ground water table inside the ramp sections. Therefore, since the initial wet excavation of section 16 was not completed at the start of boring the first tunnel tube, the dewatering of this section was started during the boring process. Figure 5 shows the horizontal equilibrium of the starting shaft both before and after the dewatering of the ramp section 16.

Clearly, the water pressure on the back wall of the starting shaft (separating the starting shaft and ramp section 16) contributes to the horizontal and rotational equilibrium of the starting shaft before dewatering of the ramp section 16. After dewatering of the ramp section 16, the horizontal and rotational equilibrium of the starting shaft will rely much more on the wall friction along the combined walls (this wall friction is not drawn in the diagrams in Figure 5) and on the friction on the bottom of the Kedichem clay/peat layer. The friction forces that can be mobilized have been a subject of much discussion, since the vertical equilibrium showed a relatively low factor of safety of 1.1. The vertical effective contact pressure is thus in the order of 20 kPa and consequently the maximum shear strength at the bottom is also limited. Furthermore, after dewatering, the resulting counteracting forces will have a origin much closer to the base. In the Ultimate Limit State (ULS) the calculations showed that rotational equilibrium was only reached with zero effective soil pressures over the first 15 m of the shaft. The effective earth pressure distribution at the base of the Kedichem clay/peat is depicted in Figure 5. In order to prevent leaking of the already finished tunnel sections, the total horizontal deformation of the shaft had to remain within 20 mm. The horizontal deformations and the sensitivity to certain variables were investigated by varying important parameters in the calculations. The lateral deformation of the shaft due to dewatering the ramp section 16 in the Serviceability Limit State (SLS) were calculated at between 5 and 10 mm. The narrow margin in the horizontal, vertical and rotational equilibrium also warranted a monitoring protocol for the pumping procedure. The observed deformations of the shaft proved to be closer to the lower limit of the calculations. Also, no time dependent deformations were measured.

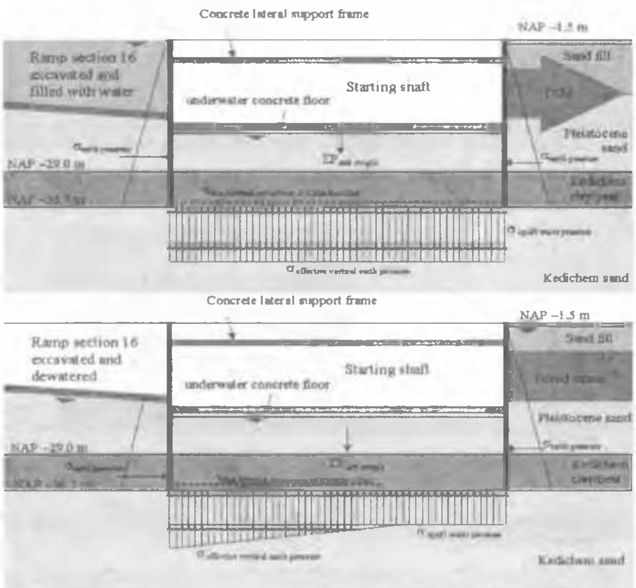

Figure 5. Horizontal and rotational equilibrium of the starting shaft before and after dewatering the ramp compartment.

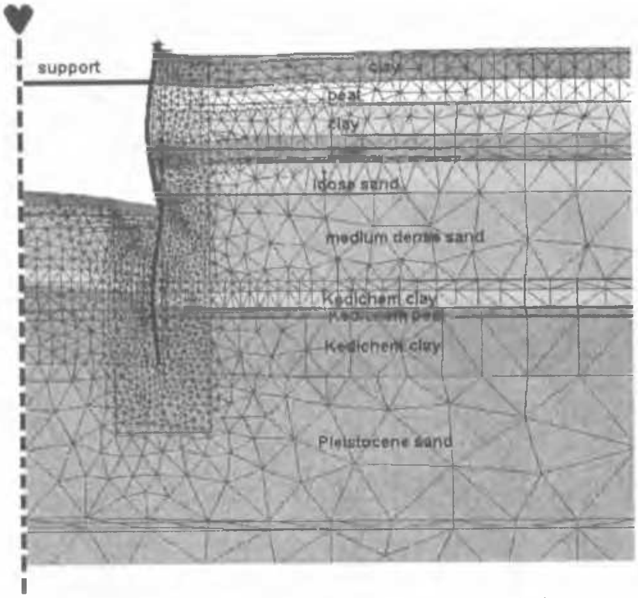

Figure 6. Calculated deformations starting shaft., FEM analysis

2.6 *Heave of the starting shaft*

Another uncertainty in the design of the starting shaft was the heave of the soil layers beneath the underwater concrete floor of the starting shaft. The total unloading pressure on the soils under the floor is 300 kPa. On the basis of the oedometer data the 7.7 m thick clay/peat layer was expected to heave (in the Netherlands commonly referred to as "swell") in the order of 50 mm under the unloading pressure. The underwater concrete floor counteracts part of this deformation, the heave pressure under the underwater concrete floor as a result hereof would amount to the order of 30 kPa. Since the underwater concrete floor is maximal 34.5 m wide and is restrained at both sides by the combined walls, the resulting bending moment in the underwater concrete floor necessitated the floor being reinforced. The challenge in the design phase proved to be predicting the amount of heave and the resulting heave pressure on the underwater concrete floor. The laboratory testing consisted only of 1-D oedometer tests, while the shaft would response to a 3-D situation involving numerous other variables. The heave of the Kedichem layer was calculated on the basis of existing analytical methods, by correlation with experimental triaxial extension tests and by means of FEM analysis. The heave and the sensitivity to certain variables were again investigated by running numerous calculations varying important parameters. On the basis of these calculations the heave was expected to be limited to 15 to 25 mm in the center of the building pit. In Figure 6 the resulting deformations are depicted (scaled up). Judging from the deformation pattern, the deeper sand layers also contribute significantly to the heave of the floor. In these analyses the in-situ macro permeabilities of the soil strata and the exact construction phasing were largely unknown parameters. The underwater concrete floor was therefore designed for a heave of 40 mm.

The measured deformation of the underwater concrete floor after dewatering the starting shaft turned out to be 15 mm, see also E.A.Kwast et al. (2001). At this stage, no significant time dependent deformations was recorded. It had been decided on the basis of this information and because the underwater concrete floor had been secured with anchors, to use less reinforcement in the underwater concrete floor of the end shaft.

Because the starting shaft is unique in the absence of tension piles or anchors, the Dutch Center for Underground Construction (CUR-COB) used the opportunity to install test equipment to observe the heave at three depths under the shaft and to observe the behavior of the earth and water pressures. These data will be

Figure 7. The starting shaft with part of the TBM assembled.

used for future deep building pits in the Netherlands, such as the North South metro line stations in Amsterdam.

The recording of the data started before excavating the shaft in order to follow the entire construction process. On the basis of these data the mechanisms under a deep building pit will be better understood. The COB is currently working on post processing of the recorded data. In E.A.Kwast et al. (2001) some of the results are described. In the future, improved design procedures can be developed for calculating vertical and lateral deformations of deep shafts.

3 END SHAFT

3.1 *Layout and soil profile*

The end shaft is comparable to the starting shaft, the soil profile is however slightly different: First a very soft clay/peat top layer of 11.5 m overlies by 12.0 m of medium dense sand, this in turn overlies the Kedichem clay/peat layer of 9.0 m, 10.0 m of Kedichem sand, again 8.0 m of Kedichem clay/peat and finally a medium dense sand below NAP –52.0 m. The ground water table is NAP –2,3 m. The reference design used slurry walls (1.2 m thick) until NAP –46.0 m (i.e. ground level –44.5 m), an underwater concrete floor (1.3 m thick) at NAP –20.3 m and once more twelve slurry wall sections until NAP –46.0 m within the building pit acting as tension piles.

3.2 *Vertical equilibrium*

Since the first Kedichem clay/peat layer at the end shaft was found to be 5 m shallower than at the starting shaft, the vertical equilibrium could no longer be served by the polder principle alone. Therefore an alternative design was chosen that uses the characteristics of the polder principle (i.e. lowering of the ground water table between the underwater concrete floor and the Kedichem clay/peat layer) in combination with ground anchors to increase the vertical equilibrium. The combined walls are extended to NAP –34.5 m. The Leeuw® anchors used utilize an injected grout body in association with normal quality steel connecting rods. These anchors do not require applying a prestress, therefore simplifying the underwater procedures significantly (the anchors are installed from a pontoon after wet excavation of the shaft).

Rather than applying a prestress to the anchors, the heave of the Kedichem clay/peat layer is used as a means for introducing a tensile force in the anchors. The hydraulic uplift pressure on the bottom of the first Kedichem clay/peat layer will try to uplift the end shaft, but this is counteracted by the anchors. Tubecon choose to use anchors instead of tension piles, because in the SLS the tensile resistance that can be mobilized with piles is limited due to the negative skin friction as a result of the heave. After all: between NAP –20.3 m and NAP –25.0 m the heave of the Kedichem clay/peat layer results in negative skin friction while the piles can only develop positive skin friction in the sand between NAP –34.0 m and NAP –44.0 m (this effect will only limit the tensile resistance in the SLS, in the ULS the negative skin friction between NAP –20.3 m and NAP –25.0 m will ultimately turn around into positive skin with ever increasing vertical deformation of the anchors). The grout of the anchors is injected between NAP –25.0 m and NAP –44.0 m, therefore only part of the heave pressure will act on the anchors.

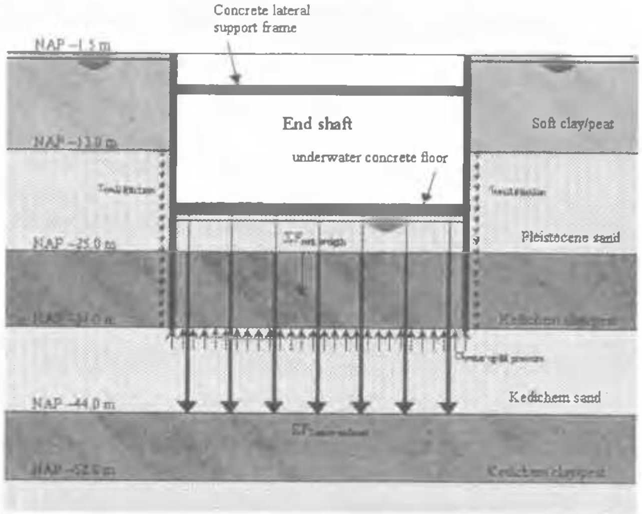

Figure 8. Vertical equilibrium of the end shaft in the final phase.

4 EXPERIENCES WITH PLAXIS ANALYSES

The design of the starting and end shafts are based on analytical design approaches. The FEM analyses with Plaxis was later added in order to verify the design and to investigate the sensitivity to variations in parameters. Both methods show quite similar results. A significant difference between both methods is the pore water pressure predicted in the Kedichem clay/peat layer due to the excavation of the shaft. This items is still under discussion, since the measurements of the in-situ pore water pressures are only just becoming available.

The FEM program Plaxis has proven to be a very useful design tool. Especially the fact that all (failure) mechanisms are incorporated into a single calculation is very useful when a large number of options have to be considered. Since the use of an integrated model allows a better definition of the safety factor the design will be more optimized: Adding up of safety margins due to model to model incompatibility or uncertainties, as often in analytical calculations, are not necessary.

Due to the nature of the excavation, predominantly unloading stress paths were expected. Hence, it was decided to use the Hardening Soil model for both the sands and the cohesive layers. The calculated deformations from this new model have proven to be realistic. Clearly, caution should be adopted when selecting inputs for the use in the FEM analysis method. The use of a relatively new soil model with a larger number of variables requires sufficient knowledge of the program, the models used, as well as knowledge of laboratory and field tests.

5 REFERENCES

Kwast, E.A. et al. 2001. *Swell investigation by the Sophia railway tunnel, Betuwelijn, The Netherlands*. Proceedings XVth Int. Conf. on Soil Mechanics and Geotechnical engng. Rotterdam: Balkema.