Construction of an underground station around an existing tunnel

Construction d'une gare souterraine autour d'un tunnel existant

H.E. Brassinga & G. Hannink – *Department of Geotechnical Engineering, Gemeentewerken Rotterdam, Netherlands*

ABSTRACT: On the south bank of the river Nieuwe Maas, the extension of Rotterdam's present city centre is being carried out. Within 10 years the Kop van Zuid, a former harbour area, will be transformed into a prime location with an international character. A new stop on the north-south Underground line, station Wilhelminaplein, will be located at the Kop van Zuid. The building method of the station is spectacular. The control of groundwater is of key geotechnical concern.

RESUME: Sur la rive sud de Nieuwe Maas, s'opère en ce moment l'extension du centre-ville actuel de Rotterdam. D'ici à une dizaine d'années, Kop van Zuid, une zone portuaire surannée et desaffectée, se sera muée en un emplacement de choix à rayonnement mondial. Une nouvelle station de métro, baptisée Wilhelminaplein, assurera une excellente accessibilité à Kop van Zuid. La méthode de construction de la station est spectaculaire. La domination d'eaux souterraines est d'une importance capitale geotechnique.

1 INTRODUCTION

The expansion of the port of Rotterdam began at the south bank of the river Nieuwe Maas a hundred years ago. Later, port activities shifted gradually to the west. It resulted in a decrease of activities on the south bank. The development of the Kop van Zuid, the creation of an undivided city with its centre on both banks of the river Nieuwe Maas, will stimulate revitalisation of the abonded harbour areas. By mid-1997, the Rotterdam Underground will also stop at the Kop van Zuid: an existing Underground line will be extended with a new station. The realisation of this new Wilhelminaplein station is a very complicated operation due to the conditions and requirements that have to be met:

- the station must be built around the present tunnel without disturbance of the transport service on the existing line;
- the station will be the deepest station in the Netherlands and has to be realised in a building pit with difficult geotechnical conditions, especially concerning the lowering of groundwater;
- the station will be in a gradient of 1:26, which fact not only requires a well-considered design but also causes varying building conditions at both ends;
- the execution of the station will take place at the same time as the erection of a large complex of buildings alongside the tunnel, due to which there is a strong relation between both projects.

The design of the station as well as the choice of a building method were influenced importantly by these conditions and requirements. This paper describes the geotechnical aspects of the construction of this Wilhelminaplein station.

2 PRESENT TUNNEL

The tunnel of the Rotterdam Underground runs from the city-centre to the southern suburbs. Due to the deep crossing of the river, the tunnel lies at a depth of approx. 7 to 12 m below sealevel (NAP), about 10 tot 15 m below streetlevel. The gradient of the tunnel is 1:26. The present tunnel, built in the sixties, consists of elements that were immersed upon piles with adjustable pileheads and has no fixed connection with these foundation piles. Due to the groundwater pressure against the bottom, the tunnel will float if the ground cover is removed completely.

3 GROUND AND GROUNDWATER CONDITIONS

The groundlevel at the Kop van Zuid is about 3 m above sealevel. Below the ground surface the subsoil consists of 3 to 5 m of sandfill resting on soft holocene clay and peat layers to a depth of 17 m below sealevel. Below this level fine to rather coarse pleistocene sand is found up to 30 to 35 m below sealevel. At this level a firm layer of loam, that is part of the Formation of Kedichem, forms the base of the geohydrological system. In figure 1 the result of a Dutch Cone Penetration Test (DCPT) is depicted.

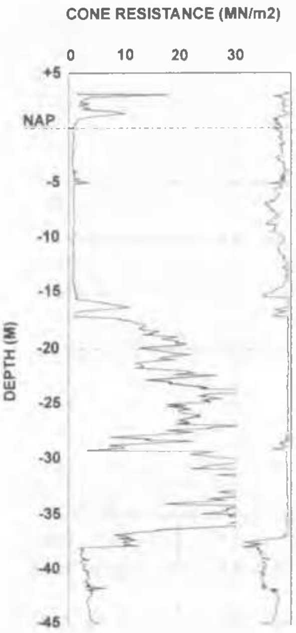

Figure 1: Dutch Cone Penetration Test at the building location.

Two water regimes are present, the first being the phreatic water of which the surface is about 2 m above sealevel and not in hydraulic continuity with the underlying pleistocene sand in which the second regime is found. The underlying sand in fact is a semi-confined aquifer which receives its recharge from the river Nieuwe Maas. The piezometric head in the pleistocene sand is a little below sealevel.

4 DESIGN

The new station is integrated with the present tunnel. Figure 2 shows de ground plan and a longitudinal section of the station and the adjacent underground hall. Due to the condition that the service on the Underground line goes on undisturbed during building, the station is erected without disrupting walls and roofslab of the existing tunnel. It was necessary to avoid inadmissible forces and displacements in the existing structure in any stage of the building process. Especially the stability of the tunnel is at stake as a direct consequence of the former building method.

Uplift of the tunnel in the final stage is prevented by a 2.5 m thick concrete roofslab. By means of V-shaped columns between roofslab and inner wall of the tunnel, part of the weight of the slab can be used as ballast. In order to keep the bending moments and shear forces in the existing structure within acceptable limits, the V-shaped columns will be provided with hydraulic jacks. By tensioning or detensioning the jacks, the flow of forces in the present tunnel can be regulated. It is expected that the jacking procedure has to be repeated a few times before the equilibrium in the new situation will be reached. The floorslab of the station at both sides of the tunnel will get a watertight connection with the bottomslab of the tunnel. Tension piles maintain the equilibrium of the new structure.

5 BUILDING METHOD

One of the most difficult problems to be solved was the design of the building pit. In order to realise a sufficient watertight connection between old and new structure it was of utmost importance to create a dry environment to work in. A dry building pit was therefore a necessity. Several possibilities for the design of the building pit were studied (Feijen & Hannink 1993). Special attention was paid to the necessity of closing the void underneath the tunnel which is also a consequence of the immersed tunnel technique of the sixties when hydraulic sandfill under the tunnel was not yet common use.

Finally the choice was made for a closed building pit, combined with a lowering of the piezometric head in the pleistocene sand layer although this will cause settlements of the lowest parts of the holocene layers. The dewatering is limited to a level that was reached in earlier projects. At some distance of the building pit part of the water was recharged into the sand to prevent exceeding of the allowed lowering.

The Figures 3 and 4 present two stages of the building process.

6 GROUNDWATER CONTROL

Groundwater control is one of the most important geotechnical aspects of the construction. A complicating factor is the vicinity of the river Nieuwe Maas. The following boundary conditions had to be taken into account for the design of the dewatering system:

- the required lowering of the piezometric head at the building pit to 12.5 m below sealevel at the northern end and to 7.5 m below sealevel at the southern end;

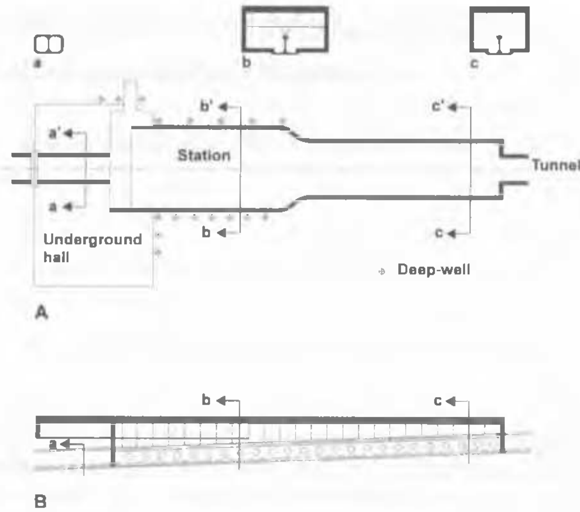

Figure 2: Ground-plan (A) and longitudinal section (B) of the new station with the position of the deep-wells.

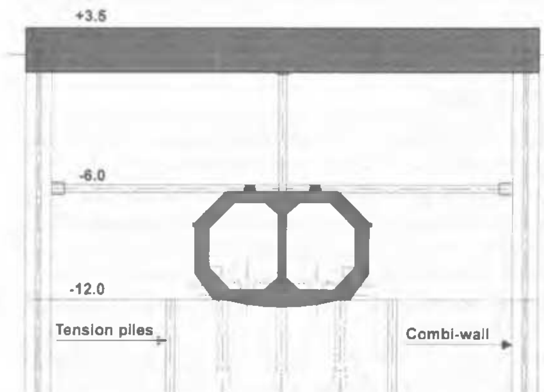

Figure 3: Excavation to bottom level of existing tunnel:
- combi-wall driven in
- roofslab finished
- horizontal and vertical tracing applied
- tension piles installed
- reinforcing bars glued in boreholes

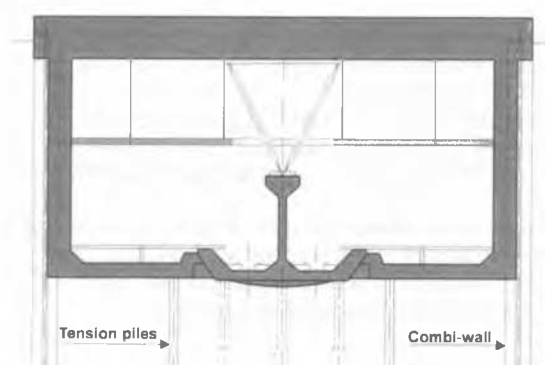

Figure 4: Final situation:
- brackets, bottom slab and wall finished
- V-shaped columns put in place
- intermediate floor suspended from roofslab
- walls and roofslab existing tunnel removed

- the maximum allowable lowering in view of possible damage of buildings in the surrounding;
- the required maximum dewatering capacity in case of high waterlevels of the river.

The average amount of dewatering during maximum excavation was estimated to be 900 m^3/hour of which 480 m^3/hour was to be recharged. High river waterlevels require a dewatering capacity of 1.100 m^3/hour. Recharge wells were placed in three lines at 200 to 500 m distance of the building pit.

Drainage is performed by 18 deep-wells ø 250 mm up to 35 m below ground surface, placed in bore-holes ø 500 mm. The wells, with perforations between 25 and 35 m, were placed outside the building pit, close to the combi-wall. The average capacity of the pumps in the deep-wells amounted to 65 m^3/hour.

In total 14 recharge wells ø 160 mm were installed, also up to a depth of 35 m. The perforations of the recharge wells are likewise between 25 and 35 m. The average capacity of the recharge wells amounted to 30 to 35 m^3/hour.

Figure 1 shows de position of the deep-wells with respect to the building pit. Deep-wells and recharge wells performed well during the period that dry conditions in the building pit were required. Dewatering started in September 1994 and was stopped in May 1996. The necessary amount of dewatering was in average smaller than estimated (850 m^3/hour), and the total amount of recharge in average larger (440 m^3/hour).

7 BUILDING PIT

For the building pit use was made of sheet pile walls. Due to the retaining height and the wish to use the wall as a foundation element, a combi-wall consisting of open tubular piles ø 1219 to 1420 mm, wall thickness 16 to 22 mm, up to a maximum depth of 28 m below sealevel and intermediate sheet piles Larssen IVS up to a depth of 18 m below sealevel, 1 m into the pleistocene sand, was installed. The amount of water that comes through the sides of the combi-wall from holocene clays and peats is negligable because of the low permeability of the holocene layers.

The biggest problem was to prevent water entering the excavation along the line of the tunnel which effectively acts as a drain. The problem is increased partly because sand was used as a replacement of soft layers underneath and above the tunnel, and partly because in the 1960's use of hydraulic sand fill under the tunnel after immersion was not common practice, and consequently there was a void of some centimetres below the tunnel. To overcome this a VHP grouted curtain cut-off was made at either end of the station. This operation was performed through vertical holes - treating the soil below the tunnel involves drilling through the tunnel - restricting work to weekends when only one of the tunnel's two tracks are needed. Jet grout work was started in April 1994; the bi-jet grouting process was used to treat the soil down to 17.5 m below sealevel with two rows of interlocking columns above the tunnel and one below (Figures 5 and 6). During the grouting process also the void underneath the tunnel was filled with grout over the complete length of the station. The horizontal stability of the jet grouted curtains above the tunnel against water pressure was ensured by a soil mass on the inside face of the curtains.

To check the waterthightness of the jet grouted curtain a test was carried out at the northern side in August 1994. Two pumps were installed in the sand fill next to the tunnel at 14 m below sealevel. Observation wells inside and outside the building pit recorded the waterlevel in the fill. The results show some reaction of the groundwater outside the building pit indicating a hydraulic resistance of the curtain of 1 to 2 days, which is equal to a permeability of 1 à 2 x 10^{-5} m/s. The leakage could have other causes than the jet-grouted curtain, and seemed to be managable by some additional pumping inside the building pit.

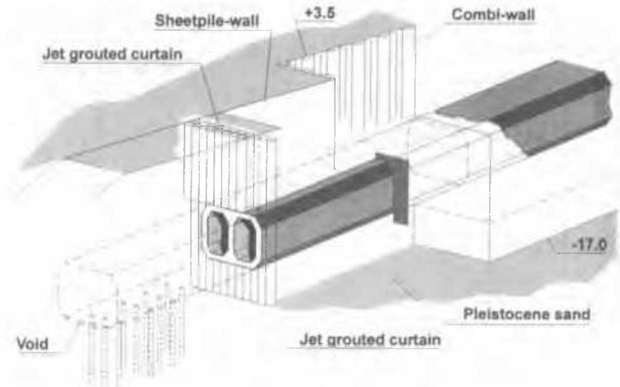

Figure 5: View of the jet grouted curtain at the northern side of the station.

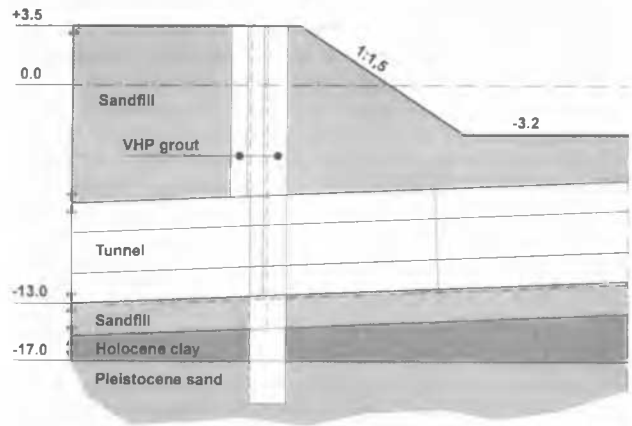

Figure 6: Cross-section of the jet grouted curtain and the northern side of the building pit.

The jet grouted curtains functioned well during a number of months, but in February 1995 leakage was noticed at the northern side. To avoid further damage, the building pit was temporary filled with water. The curtain was reinforced by installing a second jet grouted curtain in front of the existing one. During the repair process the Underground line functioned normally. The construction of the second curtain caused a delay of about three months in the building process, but no further problems did arise.

8 TENSION PILES

Tubex tension piles with grout injection, to be made from the bottom of the building pit, were foreseen in the design. The tension load amounted to about 600 kN/m at each piling row at both sides of the tunnel. The contractor proposed as an alternative Franki-piles with injection of a cement-water-mix afterwards. In stead of grout injection during penetration of the Tubex piles, fluidation takes place at the pile toe.

Three load tests on the Franki-piles were carried out in January 1995. The test piles were part of the foundation of the station. The level of the pile toes was 29.5 m below sealevel; the diameter of the piles 406 mm, the top of the pleistocene sand layer at the test location was 17.5 m below sealevel. The piles were made from 9 m below sealevel. Up the pleistocene sand layer a tube was made around the pile to eliminate fric-

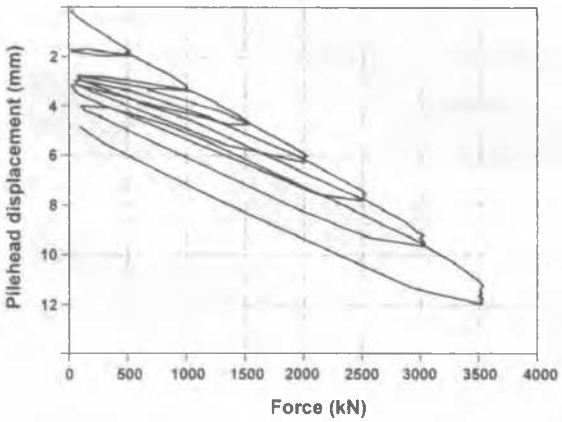

Figure 7: Result of a load test on a Franki tension pile.

tion in the holocene clay- and peat layers. After installation of the pile about 500 liter grout was injected, resulting in an extra thickness of the pile of about 30 mm. A maximum tension load of 3500 kN could be executed on the pile. The load was increased in seven equal steps which were maintained during 30 minutes or more if creep made this necessary (Figure 7). The ultimate load was not reached, but was estimated at 5000, 6000 and 7000 kN by extrapolation (Van der Veen 1953).

Cone Penetration Tests showed afterwards at 1,5 m distance of the pile a cone resistance in the pleistocene sand of about 50 MPa, considerably more than the original 20 MPa. The increase in cone resistance must be the result of the injection of the grout after the installation of the pile. The average shaft friction at the estimated ultimate load amounts to 325 kN/m^2, being 1,5% of the original cone resistance.

In the narrow part of the building pit micro tension piles were used. A tube ø 178 mm is brought to a depth of 23 m below sealevel by injection of water. After cleaning the tube a steel tube ø 114 mm is installed inside, and the original tube is pulled out injecting grout at the same time with a pressure of 20 bar at maximum. Also on these piles three load tests were carried out. The maximum load applied was 1.2 F_d, F_d being the design load. At that stage a shaft friction of 240 kN/m^2 was reached.

9 NEIGHBOURING BUILDINGS ACTIVITIES

At short distance of the station office-buildings are built with a height of maximum twenty floors. In Rotterdam it is a well-known phenomenon that high-rise and therefore heavy buildings, founded on piles in the pleistocene sandlayer at about 25 m below sealevel, cause settlements due to compression of the layer of loam at about 40 m below sealevel. This settlement not only occurs at the building itself, neighbouring constructions are affected as well. As a number of high buildings in Rotterdam was completed in recent years, a growing amount of data on this phenomenon is becoming available now (Hannink 1994). Using these data, analysis of the settlement behaviour of the Underground station Wilhelminaplein showed a differential settlement in the structure that could not be taken without damage. Measures to tackle this problem became necessary. A dilatation joint between new and old structure was constructed adjacent to the floor of the existing tunnel. The highest office-building, the Wilhelmina tower, was provided with two basements to reduce the load on the subsoil.

A further complication for the station was that work was taking place concurrently with piling for the new office-buildings. Piles were driven within a metre of the combi-wall and only 7 m from the tunnel. Measurements taken during driving recorded initial deformations of up to 20 mm in the tunnel. To minimise this, driving of the precast square section piles was symmetrical undertaken on either side of the tunnel at the same time. In addition, a row of 400 mm diameter holes was driven through the Holocene along one side of the combi-wall. The bores, spaced at 800 mm centres (leaving 400 mm of soil between each hole), were intended to absorb deformations, and deformation measurements showed that the 'trench' served its purpose.

10 MONITORING

Due to the completion of drainage activities the tunnel will move upwards. The vertical deformation amounted to about 6 mm in the period May 1996 to January 1997 after stopping the dewatering. The construction of the Wilhelmina tower will cause some downwards deformation of the tunnel. The design of the station has been based on a maximum differential settlement of 15 mm between the middle wall and the combi-wall. The effect of the building of the tower on the tunnel is negligible so far, as the tower is almost at its maximum height. Measurements will be continued after completion of the tower. Jack-up facilities have been installed in the station hall and in the V-shaped supports, to be able to compensate the deformations.

11 CONCLUSIONS

1. Deep-well drainage of the pleistocene sand proved to be a good method to create dry conditions in the building pit.
2. The jet grouted curtain cut-off appeared to be not watertight which caused delay.
3. Jet grouted curtains are only suitable to prevent water entering a building pit if a maximum of safety is demanded in the design stage.
4. Settlement problems were successfully tackled with projects at short distance where building processes take concurrently place.

LITERATURE

Feijen, Th.A. and Hannink, G. 1993. Underground Station Wilhelminapier, Rotterdam. *Proc. IC Options for Tunnelling 1993*: 231-240. Amsterdam: Elsevier.

Hannink, G. 1994. Settlement of high-rise buildings in Rotterdam. *Proc. 13th ICSMFE*: 441-444. New Delhi: Qxford & IBH.

Van der Veen, C. 1953. The Bearing Capacity of a Pile. *Proc. 3rd ICSMFE*: Vol II, 84-90. Zurich: Imprimerie Berichthaus.