Mutual influence of foundation elements and anchor pile design of a top down diaphragm wall tunnel

H. Mortier  
*CFE, Dordrecht, The Netherlands.*

S. van Dijk  
*Voorbij funderingstechniek, Amsterdam, The Netherlands.*

A.C. Vriend  
*Acécon, Baarn, The Netherlands.*

**ABSTRACT:** In the Delft railway Tunnel project, the interaction between the applied foundation elements played an important role. Diaphragm walls were installed at close proximity to existing piled foundations of a railway viaduct and grout anchor piles were realized in between prior installed diaphragm or sheet pile walls. A large amount of cone penetration tests were performed to investigate the mutual influence of the installation of these foundation elements upon each other. The applied pile test methods were optimized during the execution process in order to eliminate the shaft bearing capacity of the upper cohesive layers. As rather slender piles were used, buckling stability was verified by calculation and by testing as well. Finally, the impact of a future big variation in the existing dewatering upon the bearing capacity of the piles will be explained.

1 INTRODUCTION

In the city of Delft, the existing two track railway viaduct will be replaced by a four track railway tunnel.

![Image](image_url)

*Figure 1. Phasing of the execution at Phoenixstreet.*

In order to enable railway traffic during the seven years lasting construction works, a two phased process was elaborated. In a first phase, a two track tunnel is being built besides the existing railway viaduct. Once the trains can use this new tunnel, the existing viaduct will be demolished in order to create the necessary space to realize the tunnel comprising the two remaining railway tracks.

As it was known that the installation of diaphragm walls could affect the bearing capacity of existing nearby foundation piles, an investigation was done prior to the execution of these walls at the critical sections.

In the central part of the project, the width between the two outer diaphragm walls is enlarged from merely 12 m to almost 40 m to enable the construction of the underground station. At midspan of these large cross sections, the columns are supported by diaphragm wall barrettes. To pin the floor slab down, loaded by the uplift force of the ground water pressure, grout anchor piles were applied.

As cone penetration tests (CPT’s), performed after pile installation, showed a reduction of the initial cone tip resistance $q_c$-values, the bearing capacity of these grout anchor piles as well as the nearby diaphragm walls were questioned (Mortier *et al*, 2013).

During this validation process, the applied pile test methods were optimized.
2 DESIGN OF THE GROUT ANCHOR PILES

2.1 Geohydrological conditions

The geological layers encountered in the Delft region are typical for the river delta in the western part of the Netherlands. The top layer is a rather thin layer of sand and recent debris. Below this top layer, the geological profile consists of an accumulation of almost 20 m of soft soil of Holocene origin underlain by medium to dense sands of Pleistocene origin.

Three different water tables are detected within these soil strata. The phreatic water level is about NAP – 0.40 m (NAP = national reference water level in Amsterdam). In the deeper Holocene layers a hydraulic head of approximately NAP – 3.75 m is measured. In the Pleistocene sands the hydraulic head varies from NAP – 6 m to – 9 m, depending on the distance to the location of the industrial extraction of large amounts of Pleistocene water by the DSM chemical plant.

2.2 Grout anchor installation method

All grout anchor piles were installed before any excavation works took place. A thick walled steel casing equipped with an expendable shoe was driven into the soil to the required depth by using a hydraulic vibrator. The casing was then filled with water up to existing grade to balance for ground water hydraulic head. After placement of the GEWI bar inside the casing, grout was pumped bottom up into the casing while extracting the casing at the same time.

2.3 Bearing capacity of the grout anchor pile

The tensile bearing capacity of the piles is calculated by using the formula (CUR – rapport 2001-4)

\[ F_{rt,d} = \pi \cdot \xi \cdot \alpha_t \cdot q_c \cdot \gamma_{m, var, qc} \cdot \gamma_{m, var} \cdot L_a / \gamma_m \cdot \varphi \cdot q_c \cdot \gamma_{m, var} \cdot \gamma_{m, var} \]  

where \( \Omega_t \) is the diameter of the grout plug, \( \xi \) a parameter that takes into account the amount of CPT’s used for the design and the ability of the superstructure to redistribute the loads. The coefficients \( f_1 \) and \( f_2 \) represent the effect of soil stress increase due to pile installation and soil stress reduction due to the simultaneous loading of a group of piles.

\( L_a \) is the length of the grout plug. In the design, only the plug length in the Pleistocene sand layer was taken into account. The factors \( \gamma_{m, var, qc} \) and \( \gamma_{m, var} \) are safety factors for variable load effects (tension – compression) and tensile loads.

The two remaining factors are explained in the following paragraphs.

2.4 Coefficient of friction \( \alpha_t \)

Prior to the installation of the first permanent anchor piles, the coefficient of friction valid for the project, had to be defined. Therefore, at three different locations, each time three test piles were loaded up to the level of geotechnical collapse. This moment of collapse was defined as the point when deformation due to creep exceeds the allowable threshold according to formula:

\[ k = (u_2 - u_1) / (\log t_2 - \log t_1) > 2 \text{ mm} \]
Small grout plugs (about 5 m) in the Pleistocene sands are tested by using hydraulic jacks. Soil anchor interaction in the Holocene soil layers was avoided by using a steel casing around the anchor. By using formula (3), the ultimate friction capacity $\tau_{\text{ult}}$ of the anchors could be defined:

$$\tau_{\text{ult}} = \frac{F_{\text{test, max}}}{\pi \cdot \Omega_{5} \cdot L_a}$$

For the determination of the test load $F_{\text{test, max}}$ no reduction factors are used, neither are the measured $q_c$-values of the nearby CPT’s limited to a certain maximum value.

Finally, for the project an $\alpha_t$-value of 0.018 in combination with a cutoff $q_c$-value of 20 MPa was adopted.

For the design of the grout anchor piles is based upon CPT’s performed before pile installation and excavation of the building pit took place. The friction force between soil and grout plug is depending on the horizontal soil pressure present, which on its turn is related to the vertical soil pressure and thus to the measured $q_c$-values.

Excavation of the building pit will reduce the vertical soil pressures. According to Dutch directives, the reduction of the vertical soil pressures results into a reduction of the horizontal soil pressures. That is why the $q_c$-values before excavation have to be reduced before application in the calculation of the bearing capacity of the anchor piles. This reduction has to be done for all soil layers contributing to the bearing capacity of the anchor pile, using the formula (CUR – rapport 2001-4):

$$q_{c,\text{red}} = q_{c,\text{CPT}} \cdot \left(\frac{\sigma_n}{\sigma_i}\right)^{0.5}$$

where $\sigma_n$ and $\sigma_i$ are respectively the vertical soil stresses after (new) and before (initial) excavation.

2.6 Production tests

Besides the tests mentioned above, a prescribed amount of 3% of the anchor pile production has to be tested. Therefore, the design value of the tensile force is applied on the anchor while the creep criterion as given by formula (2) is verified. As these production anchors were tested before excavation, the test loads had to be increased to compensate for the absence of soil excavation as well as for the absence of the pile group effect and the presence of Holocene shaft friction.

3 BUCKLING STABILITY

3.1 Calculations performed

In the existing design directives, it was put forward that for slender piles in soil layers with an undrained shear strength $c_u > 10$ kPa, the verification of the buckling stability was not necessary. However, as most of the anchor piles were grouted up to the level of the concrete floor slabs, designers took these piles into account as compression piles allowing always higher compressive loads.

At different locations (university of Munich, CUR in the Netherlands,...) research was done simultaneously to investigate if the above mentioned criterion was safe enough in the case of grout anchor piles with a typical outer diameter of merely 200 mm (Meinhardt et al., 2009). As no directive existed yet at the moment the design for the Spoorzone Delft project was performed, a calculation method to define a safe maximum compressive load was elaborated for the typical situation of the project.

The piles were modeled as completely fixed at the top (connection with floor slab) and articulated at the base level of the pile. The soil layers around the pile were modeled as horizontal linear springs with a stiffness according to the Menard theory correlated to the reduced CPT values $q_{c,\text{red}}$. An initial eccentricity $e = 1/300 \cdot L$ was assumed and introduced in the model as a linear horizontal load.
\[ q_d = 8.F_d.e / L^2 \]  

(5)

with \( F_d \) the compressive load and \( L \) the total length of the pile.

Second order geometrical non linear FEM calculations resulted in a representative maximum compressive load \( F_{\text{buck;crit;rep}} = 1800 \) kN. Considering the possible imperfections like non verticality of the piles, locally reduced grout, eccentric position of the GEWI rod, etc; a safety factor of 1.5 was applied, thus resulting in \( F_{\text{buck;crit;d}} = 1200 \) kN.

3.2 Validation by testing

As the client was of the opinion that these huge compressive loads exceeded the field of common practice, a validation test was requested. In order to exclude possible eccentricity of the test load on the pile head, a temporary concrete slab, incorporating four permanent test piles, was poured underneath the absent future permanent floor slab. To prevent any load bearing by the underlying soil, a void was created by installing honeycomb cardboard panels underneath the concrete test slab that could be taken away by water jetting before the test took place.

![Figure 5.Buckling capacity test – lay out](image)

Four additional, neighboring piles were used as tension piles countering the test load and keeping down the testing frame while hollow hydraulic jacks were acting in between testing frame and test piles. The temporary concrete slab was kept leveled while increasing the jack loads. At the very moment the first of four test piles was bearing 1200 kN, the pressures in the jacks were maintained at that level for 180 minutes. During that period, deformations of the pile heads were monitored. The test validated the assumed acceptable compressive load of 1200 kN. Further on, no excessive creep deformations were measured, the temporary slab was maintained leveled and the loads on the other three test piles were all above 84% of the ultimate test load.

3.3 Comparison with new directives

Shortly after these test were done and approved by the client, the Dutch directive CUR236 – Ankerpalen (anchor piles) was published. In this directive, the maximum compressive load on anchor piles had to be calculated by using the formula:

\[ F_{\text{buck;crit;d}} = \beta . (c_u. E_{I_p})^{0.5} / \gamma_{m;\text{buc}} \]  

(6)

With \( \beta = \) coefficient with a value in between 8 and 14 (advised value = 11), \( \gamma_{m;\text{buc}} = \) safety factor equaling 1.5 and \( E_{I_p} = \) combined flexural stiffness of the GEWI rod and the grout cover. It was advised to consider the contribution of the grout cover only partly, and certainly not more than 50%. Adopting this formula for the specific case of the Spoorzone project, a value \( F_{\text{buck;crit;d}} = 1300 \) kN was obtained, thus showing good correlation with earlier calculations.

4 INSTALLATION EFFECTS OF PILES AND SHEET PILE WALLS

At the Bolwerk (Dutch for “stronghold”) building pit, the tunnel was realized using the traditional cut-and-cover method. Due to several circumstances, a lot of CPT’s were performed at different stages of the construction process. Before the works took place, the owner conducted some CPT’s to incorporate them in the contractual documents sent out to all tendering parties. Once the sheet piles of the cofferdam were installed, the contractor conducted some additional CPT’s in order to optimize the problematic dewatering design by upgrading the knowledge of the existing soil layers. Due to severe defor-mations of the nearby railway viaduct, CPT’s were made after all grout anchor piles had been installed in the non-excavated cofferdam. And finally, two post excavation CPT’s were executed.
The contractor made an analysis of all these data in order to detect the impact of each event upon the \( q_c \)-values - and thus the bearing capacity - of the soil. This analysis was thought to approve the questionable bearing capacity of the diaphragm walls and barrettes in the neighboring underground railway station, as explained in the following chapter. First, the analysis made is presented.

4.1 Uniformity of the initial CPT population

As the evaluation of the various construction stages can only be made per part (north, middle and south) of the building pit because not in every part of the pit the CPT’s after each stage were available, firstly an investigation was made to define whether the initial CPT population could be considered as resulting from one geotechnical formation. Therefore the mean CPT values per building pit part were compared to the mean CPT value of the whole population. For the Holocene layers in between the level – 10 m NAP (= underside floor slab) and – 19 m NAP (= top of Pleistocene sand layers), the ratio of both mean values varied between 0.85 and 1.07. The same ratio for the Pleistocene sand layers varied only between the values 0.97 and 1.06.

4.2 Installation effect of sheet pile walls and grout anchor piles

Comparing the ratio’s between the averages of the CPT values before and after sheet pile wall installation could only be done at the central part of the building pit. In the Holocene layers no \( q_c \)-reduction was measured, on the contrary, an increase of \( q_c \)-values up to a ratio of 1.34 was measured for an individual CPT. As such an increase could hardly be related to the rather marginal impact of the sheet pile installation – certainly in the areas at greater distance to the sheet pile walls, the variation was appreciated as heterogeneity of the soil layer. For the Pleistocene layers a reduction up to 10% was measured.

The installation of the anchor piles showed mostly a densification of the loosely packed Holocene layers and a rather significant relaxation of more than 20% of the dense packed Pleistocene layers. This resulted in average settlements of nearby structures of about 20 to 30 mm.

4.3 Effect of excavation works

Only in the southern part of the building pit, two CPT’s after excavation were performed. The main reason for these CPT’s was the impossibility to apply the required test load upon the existing piles before excavation. As already explained in paragraph 2.6, the test load takes into account the effects of future excavation, the absence of the pile group effect and the contribution of the Holocene layers to the measured bearing capacity. For the piles in the Bolwerk building pit, the total effect of these scenario’s resulted into a theoretical test load that exceeded the tensile capacity of the GEWI bar as can be seen in the formula’s below.

\[
F_{\text{test}} = F_{s,d} \cdot \gamma_{var} \cdot f_{\text{exc}} / f_2 + F_{\text{hol}} = 454 \cdot 2.94 + 554 = 1892 \text{ kN} \quad (7)
\]

Which is bigger then

\[
F_{\text{ult;GEWI}} = 0.9 \cdot f_{yd} \cdot A_{\text{GEWI}} = 0.9 \cdot 555 \cdot 3167/1000 = 1580 \text{ kN} \quad (8)
\]
In this formula $f_{exc} (= 1.35)$ is the coefficient for the beneficial effect of the unexcavated situation, $F_{hol}$ is the bearing capacity due to shaft friction in the Holocene layers, $f_{yd}$ the tensile strength of the GEWI bar (0.2% elongation) and $A_{GEWI}$ the cross sectional area of the applied 63.5 mm diameter GEWI rods.

In previous performed production tests, sometimes the creep criterion could not be fulfilled. At that moment an explanation was given that the Holocene cohesive layers were offering a substantial part to the bearing capacity. As these cohesive layers showed a slow transition from the loads taken by the enclosed (ground)water to the soil grains (this is the well known consolidation process), deformations were bigger than the allowable values of the creep criterion, but the final bearing capacity nor the stiffness of the anchor pile should be doubted. Both aspects, together with the encountered huge reductions of the $q_c$ values of the CPT’s after pile installation, made the client ask for a small amount of pile tests performed at the excavated level and without disturbance of Holocene shaft friction.

Therefore, steel casings were installed around the intended test piles, and this from excavated level till 1 m above top of the Pleistocene sand layer. This 1 m separation was chosen to maintain the watertight barrier in between two distinct water tables. In addition to these pile tests, two CPT’s were conducted from excavated level.

The CPT’s showed a huge reduction of the $q_c$-values in the prior densified Holocene layers (29% reduction compared to initial situation and even up to 50% compared to situation after pile installation) and a rather moderate additional reduction in the Pleistocene layers (merely 4% additional reduction compared to situation after pile installation).

The contractor concluded that there was indeed a reduction of the $q_c$-values due to (especially) the grout anchor pile installation, but it was too conservative to add this measured reduction to the calculated reduction due to excavation of the building pit. As the tests showed a sufficient bearing capacity of the anchor piles, the necessity to apply similar tests at the location of the neighboring underground station was considered negligible.

5 INVESTIGATION OF THE BEARING CAPACITY OF THE DIAPHRAGM WALLS

5.1 Problem definition

The tests and evaluations as described in chapter 4 convinced the client that the bearing capacity of the anchor piles was guaranteed, even though significant $q_c$ reductions were measured in the CPT’s executed after pile installation and excavation. Apparently, the traditional safety factors applied in the calculation of the bearing capacity of the piles incorporated these effects. However, as a vast amount of the anchor piles was installed in between the prior constructed diaphragm walls and barrettes of the underground station, serious doubts arose about the bearing capacity of these foundation elements, especially as the new town city hall was going to be built on top of the underground station (Everaars et al, 2011).

The conclusion that the theoretical (calculated) reduction factor $f_{exc}$ should not be added for the full amount to the measured reduction factors related to pile installation was not fully accepted by the client. As the conservative approach of the client led to an insufficient bearing capacity of the diaphragm...
walls and barrettes at the underground railway station, additional CPT tests became unavoidable.

5.2 Validation tests underground station - execution

The lateral diaphragm walls of the underground station have a pile base level that varies between – 24.50 and – 27.50 m NAP. For the barrettes under the central row of columns, this base level varies between – 31 and – 44 m NAP. The base levels of the anchor piles reached from – 33 to – 36 m NAP. As the purpose of the additional CPT tests was to determine the impact of the anchor pile installation upon the diaphragm walls and barrettes, it was decided that CPT’s to a level of – 36 m NAP were sufficient.

As already explained in paragraph 2.1, three different ground water tables are present in the project subsoil. In the design of the foundation elements a conservative approach was implemented. When dimensioning the foundation elements for compressive loads, the Pleistocene water table was assumed to be active underneath the floor slabs. This implied that the Holocene ground water pressure was taken away during the excavation process and could not regain its old head afterwards, due to a good water tightness of the diaphragm walls. On the other hand, when dimensioning the foundation elements for tensile loads, the Holocene ground water pressure was thought to be present. Moreover, if the watertight soil layer, present in between phreatic and Holocene water table was actually located at a lower level than the floor slab, even the phreatic water pressure was assumed to be present.

As the additional CPT’s penetrated the floor slab, measures had to be taken to avoid huge water ingress. In the case of significant leakage through the diaphragm walls the Holocene water pressure could cause this huge ingress in combination with possible sand transport from underneath the floor slab thus lowering the ground water table outside the building pit that could affect nearby buildings. The risks in the case of water tight diaphragm walls were estimated much smaller to almost negligible.

Figure 8.Cross section underground railway station showing foundation elements toe depths

Therefore, at first core drillings were made through the floor slab. Once arrived at the bottom of the floor slab, a poly urethane plug was grouted just underneath the floor slab. Then, a special device containing a valve and a box with an inner inflatable rubber sealing was used. Once the cone of the CPT had passed the box, the rubber was inflated forming a watertight closure around the CPT elongation.
tube. At that moment the valve under the cone could be opened and the CPT could be pushed downwards thus penetrating the prior grouted poly urethane plug. By using this method, all CPT’s could be made without any risk.

5.3 Validation tests underground station – results

In order to define the reduction factor due to anchor pile installation, the following method was adopted. As the top of the bearing soil layer was located at a level of – 19.20 m NAP and the new CPT’s reached to a level of – 36 m NAP (= deepest anchor pile base level), all qc-values of the original CPT’s within this zone were reduced by the theoretical reduction factor due to excavation \( f_{exc} \) and afterwards limited to the maximum value of 15 MPa. By doing this, the bearing capacity of this soil layer as implemented in the original design calculations was defined. Then, all \( q_{c,\text{red}} \) values were cumulated resulting in the value \( \sum q_{c,\text{red};\text{init}} \).

The same soil layer was investigated afterwards using the new CPT’s. It is evident that these values didn’t require reduction for excavation effects. The cutoff value of 15 MPa was however implemented again. By adding all values once again, the value \( \sum q_{c,\text{new}} \) was obtained. The reduction factor for pile installation effects could be defined as

\[
f_{\text{red;pile inst}} = \frac{\sum q_{c,\text{new}}}{\sum q_{c,\text{red};\text{init}}} \tag{9}
\]

Nine additional CPT’s were performed, three per part (north, middle and south) of the underground station. If we compare the average of all initial \( \sum q_{c,\text{red};\text{init}} \) values to the average of all new \( \sum q_{c,\text{new}} \) values per part, the resulting \( f_{\text{red;pile inst};\text{avg}} \) values for northern, middle and southern part are respectively 1.11 / 1.01 and 0.99. This means that the theoretical value of the reduction due to excavation incorporates all unfavorable effects mentioned before. Even though the fact that CPT’s might show strong reductions in \( q_c \) values due to pile installation, the final reduction after excavation remains the same as the (conservatively) theoretically anticipated value.

Comparing each individual new CPT to the average value of the initial CPT’s, a maximum reduction of 15% and 13% was found. All other ratios showed no reduction.

As the client was concerned about the bearing capacity of the diaphragm walls and barrettes, the new CPT’s were executed at close distance to these panels. Investigations made in prior diaphragm wall projects had shown that the \( q_c \) values could be reduced due to installation of the diaphragm walls (Handboek diepwanden, 2010). On basis of these investigations, the graph shown in figure 10 was defined.

![Figure 10. Reduction due to diaphragm wall installation – empirical boundaries and project values](image)

For the new CPT’s, the ratio \( a/l_s \) varied between 0.17 and 0.47. The CPT’s that resulted in the reductions of 15% and 13% had an \( a/l_s \) value of respectively 0.3 and 0.17. According to the graph, reductions up to 72% could be expected for the worst case scenario. This was another proof that the installation of the anchor piles didn’t influence the bearing capacity of the diaphragm walls to a greater extent than anticipated in the directives. The low \( \alpha_s \) value (= 0.055) applied for defining the shaft bearing capacity of diaphragm walls is based upon empirical data, but apparently takes the reduction (relaxation) of the soil nearby the installed diaphragm wall panels into account.

6 RISK ASSESSMENT OF DIAPHRAGM WALL INSTALLATION NEARBY EXISTING RAILWAY VIADUCT

Part of the Phoenixstreet western railway tunnel can already be built while the railway viaduct is still exploited. Piles of the piers of the existing viaduct have a pile base level of merely – 21 m NAP, and the distance of the designed diaphragm walls to the pile base of the raked piles can be as small as 2.92 m. Therefore a risk assessment had to be made, prior to the...
installation of the diaphragm walls. As initial CPT’s were available along the project alignment, it was decided to make additional CPT’s in the neighborhood of the existing CPT’s and at distances of 2, 4 and 6 m to the already installed diaphragm wall between eastern and western railway tunnel tube.

By comparing the initial CPT’s to the new CPT’s, the graph shown in figure 10 could be validated and the risk associated to the installation of the western wall of the western tube could be defined. The same approach as explained in paragraph 5.3 was adopted and the following results were found.

Only for the CPT with a distance of 1.54 m to the diaphragm wall, a reduction of 7% was found in the soil layer ensuring the bearing capacity. The new CPT at this location was only 1.54 m out of the diaphragm wall as the location at the foreseen 2 m distance was impossible due to obstacles. All other (8) CPT’s showed no reduction of the $q_c$ values in the investigated soil layer. This means that the graph of figure 10 must be considered as a worst case scenario, even for the upper boundary line.

![Figure 11. Validation of assumed relation $q_c$ reduction to DW installation by CPT’s](image1)

**Figure 11.** Validation of assumed relation $q_c$ reduction to DW installation by CPT’s

**Figure 12.** Comparison of initial and new CPT’s in bearing soil layers

7 IMPACT OF DEWATERING SHUT DOWN

The chemical plant of DSM is dewatering the subsoil of the Delft region for decades. Up to 1500 m$^3$/hour were extracted from the Pleistocene sand layers to cool the chemical processes in the factory. Nowadays, processes have been altered and DSM wants to shut down the dewatering. Agreements were made between the Dutch government and DSM to maintain a minimum dewatering level of 1100 m$^3$/hour during the execution of the railway tunnel project as this simplifies the works significantly.

The contract requirements however stipulated that shutdown of the dewatering process should be taken into account within the design. Stopping the dewatering will generate an increase of the Holocene water table by 1.25 m and the Pleistocene water table by 3.45 m. These values varied in relation to the distance to the DSM factory. The phreatic water level wasn’t influenced, as this water level was continuously fed by the Delft water channels.
For the tensile anchor piles, the higher loads were correctly implemented in the design, but the effect of decreasing vertical soil stresses and thus bigger reduction factors, were omitted in the basic design.

Figure 13. Comparison of original CPT with theoretical CPT’s after excavation and shut down dewatering

On the other hand, as already mentioned before, the shaft bearing capacity of the anchor piles in the cohesive Holocene soil layers was systematically neglected. This was done in order to avoid calculations comprising the effect of negative skin friction.

As the shutdown of the dewatering can never coincide with a negative skin friction scenario, it was acceptable to take the shaft bearing capacity of the Holocene layers into account. At first, the extra reduction of the measured $q_c$ values was defined applying the same methodology as when defining the reduction factor for excavation effects. When comparing the total shaft bearing capacity after dewatering, thus from underside floor slab down to pile base level, with the shaft bearing capacity during dewatering, but only in the Pleistocene sand layers, it appeared that the anchor piles had about 2% higher bearing capacities. Even more, when the $q_c$ values after the required reduction were limited to maximum values of 15 MPa, the ratio rose to 1.06.

This means that the redundancy of the Holocene shaft bearing capacity neglected during design was capable to cope with the omission in the design, related to the soil stress decrease due to the stopping of the dewatering.

8 CONCLUSION

Various tests upon grout anchor piles within the Delft railway tunnel project gave a better understanding of the behavior and performances of the applied anchor piles. As cohesive soil layers can cause a noncompliance to the creep criterion, it is advised to exclude the effect of this layer by using steel casings. Buckling stability can seriously decrease the bearing capacity of the compressed anchor piles. A fairly good value can be obtained by using the new Dutch directive CUR 236.

Apparently, the $q_c$ reduction factor for post CPT excavation, comprises also the measured $q_c$ reductions after pile installation. Measured $q_c$ values after pile installation don’t have to be cumulated with theoretically defined reduction factors for excavation effects.

Analogue conclusions can be made for the $\alpha_s$ coefficients used for shaft bearing capacity of diaphragm walls and barrettes. These values should be increased (a value of 0.8 could be a conservative approach) for recalculating the bearing capacity of the diaphragm walls using nearby post installation CPT’s.

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