Geotechnical Aspects of Underground Construction in Soft Ground, Mair & Taylor (eds)
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Ground movements due to tunnelling: Influence on pile foundations

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ABSTRACT: The influence of shield tunnelling on adjacent and overlying structures is an important factor for the design of tunnels in urban areas. In the framework of a joint industry project (CUR C 89, 1994), model tests have been conducted to study the influence of tunnelling on end-bearing foundation piles. For three different levels of the tunnel, model tests were carried out using the Delft Geotechnics centrifuge. As a function of volume loss due to shield tunnelling, pile settlements and ground surface settlements were measured. The influence on the pile bearing capacity was also determined. In order to increase the understanding of the influence of shield tunnelling on pile bearing capacity and to be able to make predictions for situations other than the situations tested in the centrifuge, extensive computer modelling has been carried out.

1 INTRODUCTION

In the near future tunnels will be bored in the western part of The Netherlands. This area is highly urbanized and it is likely that the tunnels will be driven at close distance from existing buildings. In the western part of The Netherlands the upper part of the soil consists generally of soft to very soft clay and peat. Therefore, most of the existing buildings are founded on end-bearing displacement piles. The influence of the tunnelling process on these piles can place significant restraints on the preferred horizontal and vertical alignment of the tunnel, especially in case where tunnelling is performed in the same sand layer in which the building foundations are located.

The interaction between a shield tunnel under construction and a loaded foundation pile is a complicated three dimensional problem. One of the difficulties to solve this problem numerically, is to incorporate the initial stress boundary conditions around the pile tip. Therefore it was decided to investigate the problem with model tests.

Between 1992 and 1995, a joint industry project has been performed to investigate the interaction between a loaded foundation pile and a shield tunnel under construction. The research has been carried out using the geotechnical centrifuge of Delft Geotechnics. In this paper test results are presented.

In combination with the physical modelling using the geotechnical centrifuge, extensive computer model-ling has been carried out. Predictions of the settlement trough have been made using the accepted empirical calculation methods based on a typical Gaussian settlement trough and a 2-D finite element method. Special attention has been given to mo-delling a loaded pile. The Eulerean large displacement approach is chosen to model a penetrating pile. Some results are discussed in this paper.

2 CENTRIFUGE TESTS

The stress-strain relation of soil is nonlinear and soil strength depends on the actual effective stress level. Due to the effective weight of the soil, the effective stresses increase with depth. In order to get reliable model test results, not only the mean effective stress in the model should be representative for the prototype under investigation, but also the distribution of stresses. This is especially true where locally high stresses are present around the pile tip.

When using the same soil material(s) in the model as present in the prototype, this stress requirement in the soil can be fulfilled by increasing the unit weight of the soil with a factor equal to the scaling factor by which the prototype size is scaled down. By increasing the acceleration that acts on the model (relative to the acceleration of gravity), making use of a geotechnical centrifuge, similitude is obtained between model and prototype regarding the stress-strain relationship of the soil. This is an essential requirement for the investigation of pile-tunnel interactions by means of physical modelling.

From model laws it follows that for a first interpretation of test results the following scaling rules may be used (Table 1).

For the modelling of the soil Speswhite Kaoline clay and Eastern Scheld model sand are used. The average particle size of the model sand (d_{50}) equals 150 μm. A geometrical scaling factor of 1:40 was choosen. In most of the figures the dimensions length is expressed in model quantities. To estimate the dimension length in prototype terms the model quantities must be multiplied with the geometrical scaling factor (N = 40).

Table 1. Scaling factors in centrifuge tests (provided prototype soil materials and the prototype pore fluid is used).

Quantity	Prototype	Model
Length	N	1
Acceleration	1	N
Time (laminar flow)	N^2	1
Stress	1	1
Strain	1	1
Soil density	1	1

3. TESTING PROGRAMME

3.1 *Prototype*

A section through the model is given in figure 1. The dimensions are expressed in model quantities. To approximate the dimension length in prototype terms the values given in figure 1. must be multiplied with the centrifuge model factor N (equals 40).

The prototype consists of a tunnel with a diameter of 7 m excavated (partly) in a sand layer that was covered by a 16 m thick clay layer. The diameter of the foundation piles equals 0.4 m. The tip level of the closed ended displacement piles is 2 m below the top of the sand layer.

Since the prototype geometry is scaled down with a factor 40 while the particle size of the soil materials in the model and in the prototype are the same, special consideration must be paid to the interpretation of the soil behaviour in prototype terms. From centrifuge model tests it is known that the clay behaviour in the model is representative for the prototype (taking in to account the time scale for laminar flow, see table 1). The particle size-effect is especially important with respect to the influence of dilatancy on soil strength in sand (Blinde, 1978). Therefore, in terms of the continuum stress-strain relationship the prototype sand behaviour in shearing may be understood as the behaviour of a soil with an average particle diameter of approximately 6 mm.

3.2 *Test set-up*

The volume loss due to tunnelling, defined as the reduction of the tunnel volume expressed as a percentage of the initial tunnel volume, is simulated by a well controlled diameter reduction of the model tunnel (Bezuijen, 1994). In contrast with the prototype, this volume loss is exactly known.

The range of volume losses that can be imposed equals 0 to 10 %. The tunnel itself is a stiff element in the soil and is not fixed to the container wall and can move freely during the imposement of the volume loss. The effective weight of the tunnel is almost equal to the effective weight of the soil.

The three-dimensional aspects which coincide with the progression of the tunnel boring machine are not taken into account and any movement in front of an advancing tunnel are ignored. Therefore, the model represents the prototype situation where full stability of the tunnel face is assured. The test set-up allows for a well defined comparison with 2 dimensional finite element analyses and for

A Cone penetration testing device
B Displacement transducers
C Piles
D Location tunnel in test 1
E Location tunnel in test 2
F Pore pressure transducers
G Loading frame
H Plunger
I Displacement transducer pile
J Load transducer pile
K Strain gauges
L Spring connection to loading frame
M Pile number
N Location tunnel in test 3

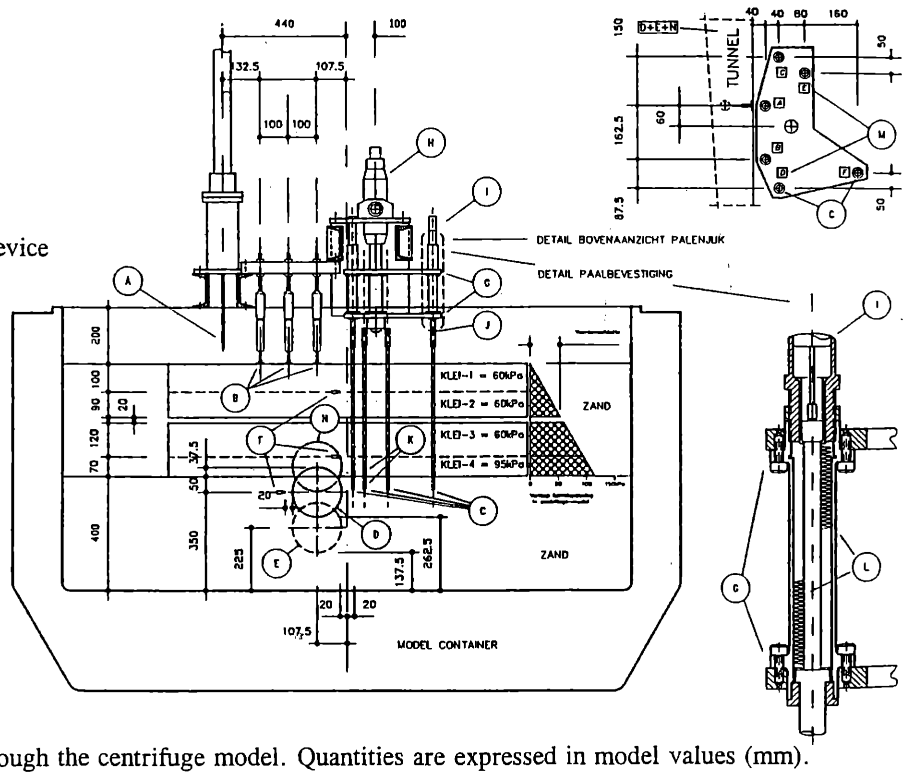


Figure 1. Section through the centrifuge model. Quantities are expressed in model values (mm).

comparison with 2 dimensional semi-empirical design approaches (Attewell et all, 1986).

The model tunnel is placed in the soil before centrifuge testing starts. The model piles are installed using 1 loading frame. The last 2 m pile penetration, expressed in prototype dimension, is performed "in flight". During pile penetration pile displacement and ultimate bearing capacity of the piles are measured individually. After the pile tips have reached the designed pile tip level, the piles are unloaded up to 70 % of the last ultimate bearing capacity recorded during penetration. Each pile is connected to the loading frame by a spring. Due to this spring connection the pile loading can be kept fairly constant in case a pile settles due to the volume loss at the tunnel.

During the reduction of the diameter of the model tunnel (volume loss), the pile behaviour and the development of the settlement trough at surface level are monitored continuously. So, information is obtained regarding small as well as large volume losses. The total volume loss of approximately 10 % was imposed in about 1 minute (the material behaviour of the clay during contraction is undrained).

4 TEST RESULTS

4.1 *Settlement trough*

In all tests the primary settlements at surface level develop linear with the volume loss imposed. confirm that the shape of the settlement trough, considered in a two dimensional section perpendicular to the tunnel-axis, can be described with a Gaussian curve as given in literature (Attewell et all, 1986). However, in test 3, the distance (i) from the axis of symmetry to the point of inflexion of the curve was not constant. At volume losses larger than 3 % of the initial tunnel volume, i decreased resulting in a smaller settlement trough (figure 2).

This is probably caused by the change from an elastic deformation pattern at relatively low volume losses to an elastic plastic deformation pattern at relatively large volume losses.

Predictions of the test results with existing empirical relations for the determination of i did not match with the results of the centrifuge tests.

Firstly, the empirical relations, used for this configuration, show a variety of i-values. Secondly, the centrifuge test results showed a phenomenon, that the value of i decreased with increasing tunnel depth (see tabel 2).

Table 2. Magnitude of parameter i for the three different model tests at a volumeloss of approximately 3 %.

Test nr.	i in model dimensions (mm)	i converted to prototype (m)
1	157,5	6,3
2	150	6,0
3	170	6,8

Although the number of data points on which the estimation of i was based is limited, the figures from table 2 indicate that the magnitude of i depends not only on the tunnel diameter, the thickness of the overburden and the stratigraphy of the soil layers, but also on the magnitude of the volume loss, the depth of the embedment of the tunnel in a dense sand layer and the stiffness of the soil layers.

An indication of the deformation pattern of the soil in case of test 3 is given in figure 3. The image is obtained by image processing and shows total displacement vectors representing soil displacements in the clay after imposing a volume loss of approximately 9 % (the scale of the arrows is a relative one, the size of the pile can be compared). The bottom line of the image is located approximately at the intersection between the clay and the sand layer. The results show that a shear band has developed from the bottom of the tunnel towards ground surface. This indicates that the deformation pattern at large volume losses is determined by plastic effects.

The reduction of the diameter of the model tunnel causes a rotation of the direction of the primary stresses. Around the tunnel high stresses develop (arching), causing excess pore water pressures in the clay. The dissipation of these excess pore water pressure takes some

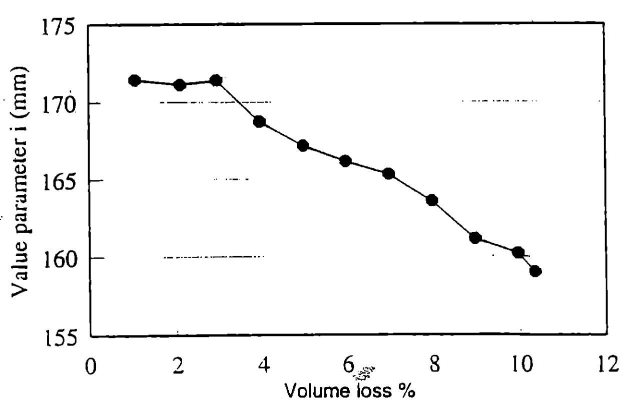


Figure 2. Parameter i as a function of the volume loss (test 3) expressed in model quantities.

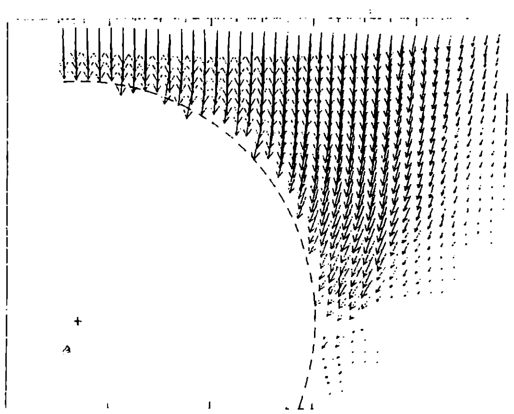


Figure 3. Image showing total displacement vectors in the clay around the tunnel.

time (consolidation). The effect of the consolidation is investigated in case of test 3 The settlement trough at surface level is given in figure 4. and represents the situation just after imposing the total volume loss of approximately 10 % (circles) and after full consolidation (squares). The lines fitted through the test data are determined using a simple regression analysis.

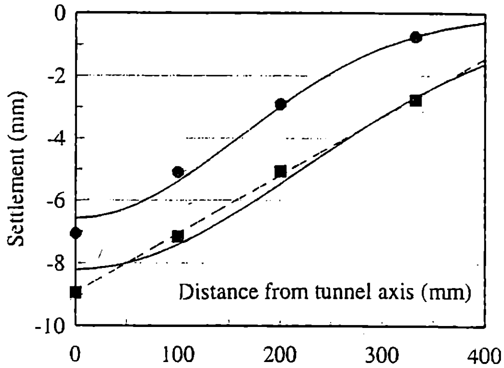


Figure 4. Settlement trough at surface level in case of test 3 (just after imposing the total volume loss of approximately 10 % and after full consolidation). The values are expressed in model quantities.

Just after imposing the total volume loss the volume of the settlement trough at surface level equals approximately the tunnel volume reduction. After full consolidation the volume of the settlement trough at surface level becomes larger than the tunnel volume reduction imposed (approximately 1,7 times larger). However, the angular distortion stays more or less constant as the trough becomes not only deeper but also wider (figure 4). This might be caused partly by the drainage boundary conditions of the test set-up. However large additional settlements may be expected in case the tunnel is located partly in soft clay.

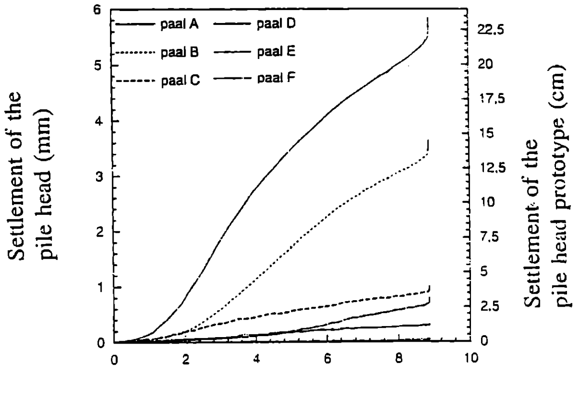
Another interesting aspect, which emerged from the results of the test 2, in which the top of the tunnel was located 3.5 m below the top level of the very dense sand layer, was that a reduction of the volume of the settlement trough at surface level occurred.

The test result indicated that the total volume of the settlement trough was approximately 50 % of the imposed volume loss at the tunnel depth. Probably due to arching on top of the tunnel, and dilatancy due to shearing within the very dense sand layer, a reduction of volume loss at ground surface is obtained.

4.2 *Pile settlement and reduction of bearing capacity*

The decompression of the dense sand layer caused by volume losses imposed at the tunnel have a significant influence on the settlements and bearing capacity of adjacent, end-bearing (displacement) piles. As expected, the centrifuge tests showed that the settlements and loss of bearing capacity was the largest for the closest piles, in this case located at approximately 0,25 x tunnel diameter distance from the edge of the tunnel. The largest settlements of the piles were found in a configuration, where the level of the centre line of the tunnel was equal to the pile tip levels (test 1). The smallest settlements were found in the configuration where the centre line of the tunnel was located above the pile tip levels (test 3). In the configuration, where the tunnel was located below the pile tip levels (test 2) the settlements of the nearest piles were less than in the situation where the centre line of the tunnel was equal to the pile tip levels. However, the piles at the largest distance showed a larger settlement in the case where the tunnel was located below the pile tip level, so the width of the influence zone was larger.

Test results are given for test 3 (figure 5). The top figure shows the settlement of the piles as a function of the imposed volume loss. In this figure 'paal A' is the foundation pile located at approximately 0,25 times the tunnel diameter from the tunnel edge. The distance between the tunnel edge and the piles B to F increases from approximately 0,5 to 1,8 x the tunnel diameter. The scale at the right side of the figure shows the pile head settlement converted to the corresponding prototype. In the bottom figure the change in pile head load as a function of volume loss is given. The deformations of the piles, located near to the tunnel, were so large, that a decrease of the pile head load occurred (in spite of the spring connection). From the figures it follows that the measurement of the volume loss ended at approximately 9 %, the real volume loss imposed was somewhat larger. In the situation where the tunnel was located above the pile tip levels (test 3), a detailed study was made of the



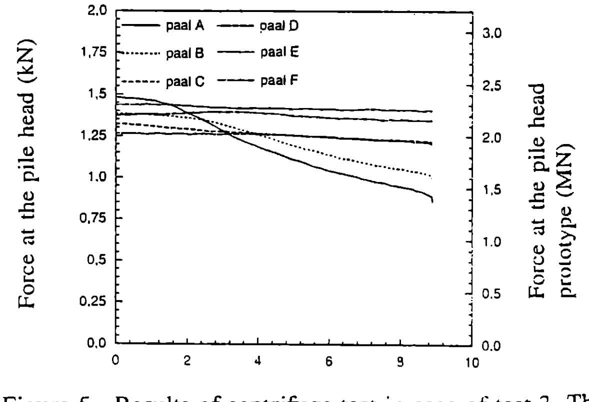


Figure 5. Results of centrifuge test in case of test 3. The piles are loaded up to a level equal to 70 % of the during the penetration phase recorded ultimate bearing capacity.

generation of the skin friction along the piles. Test results showed that for relatively small volume losses (smaller than 1% of the tunnel volume) the ground surface settlements were larger than the pile settlements, so negative skin friction developed. For larger volume losses of approximately 1% of the tunnel volume to 6% of the tunnel volume, the bearing capacity of the loaded piles was reduced by such an amount that the displacement of the pile tip increased more than the ground surface settlements. In this phase of the test positive skin friction was mobilised. For larger volume losses the ground surface settlement increased faster than the pile settlements, which eventually caused negative skin friction to occur at a volume loss of 10% of the tunnel volume. In the consolidation phase at the end of the test the piles showed additional settlements due to additional negative skin friction, because the bearing capacity of the piles was equal to the vertical load at the end of the test.

5. NUMERICAL MODELLING

5.1 *Settlement trough*

A back calculation for the experimental results has been made with a two dimensional finite element analysis. The tunnel was located for 2/3 within the soft clay layer (test 3). The constitutive model applied for the sand was the Mohr-Coulomb. For the modelling of the Spesswhite Koaline clay an undrained Mohr-Coulomb model was used. In the latter case, the undrained shear strength of the soil was coupled integrally to the actual stress level using the equations of Biot (Teunissen, 1991). The calculation parameters used are given in table 3.

Table 3. Calculation parameters used in the finite element calculation.

Parameter	Sand	Clay
Effective weight (kN/m^3)	10	7
Youngs modulus (MN/m^2)	70	3
Poisson's ratio (-)	0,3	0,35
Cohesion (kN/m^2)	0	0,5
Effective angle of friction (°)	40,0	14,6
Angle of dilatancy (°)	5	0

The volume-loss was simulated by imposing a fictive load uniformly distibuted along the outer boundary of the tunnel. For this simulation the stiffness of the tunnel was chosen relatively high compared to the stiffness of the surrounding soil. By doing so the axial symmetric diameter decrease of the model tunnel is well controlled and in accordance with the model test (the tunnel can move freely in the soil). Obviously, the stress distribution of stresses within the tunnel is wrong, but is in this particular case of no interest. The comparison between measured data and the calculation results is given in figure 6 for a volume loss of 3 % (directly after imposing the volume loss). Consolidation effects were not taken in to account.

The calculation results show that the total volume of the settlement trough was determined well, but that the value of i for the FE calculation was higher than observed in the centrifuge test. This is probably due to an underestimation of the values of the Young's moduli. The calculations showed the development of a shear band in a way, which is comparable with the test results.

The results indicate that, if soil parameters are determined carefully, 2D finite element calculations can be used to estimate the ground surface settlements accurately directly after the imposement of the volume loss at tunnel level.

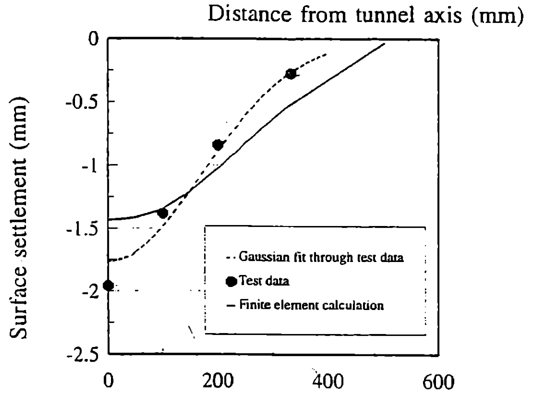


Figure 6. Measured and calculated settlement trough, expressed in model quantities.

5.2 *Pile settlement calculations*

Since it appeared not possible to model pile installation to entire satisfaction with the commonly used Lagrangean axial symmetric small strain approach, it was concluded that a large deformation analysis was needed to introduce the stress conditions just after the pile installation. This is especially important in case of stress-dependent (frictional) material behaviour. Therefore, the Eulerean large displacement approach was chosen to model the penetrating pile. In contrast with a Lagrangean approach, the material is not coupled to the elements. The finite element discretization including the penetrating pile is fixed in space and the soil flows through the fixed mesh. For the axial symmetric simulation of the pile installation the finite element programme DIEKA has been used (Berg van den, 1994). The calculation result is compared to the test result for test 2 (figure 7). The material model used was Drucker Prager. The material parameters are given in table 4. From the calculation result it was concluded that, in spite of the uncertainties in the choise of the exact soil parameters, it is possible to simulate the pile installation correctly.

Since the simulation was axial symmetric the influence of the volume loss had to be simulated by a diameter decrease of a ring located around the single pile. Although, the calculation result confirmed the influence of the decompression of the zone around the pile tip caused by the tunnelling process (in terms of bearing capacity of the loaded pile), the results are still qualitative.

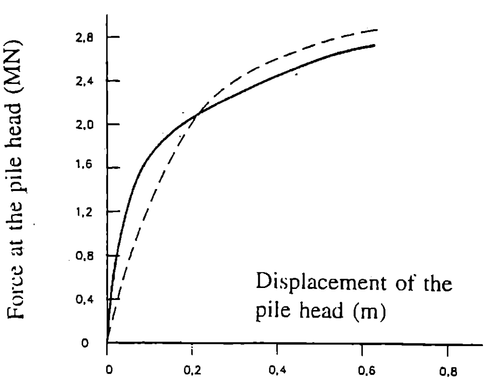


Figure 7. Pile installation. Experimental data compared to calculation result (quantities are converted to prototype values).

Table 4. Calculation parameters used in the finite element calculation for the pile installation.

Parameter	Sand	Clay
Effective weight (kN/m^3)	10	7
Youngs modulus (MN/m^2)	140	3
Poisson's ratio (-)	0,3	0,49
Cohesion (kN/m^2)	0	0,5
Effective angle of friction (degrees)	40,0	12,4
Angle of dilatancy (degrees)	15	0
Coefficient of earth pressure at rest K_0	0,5	0,5

Note: the angles of friction and dilatancy of the sand depend on the amount of volume strain and have been reduced to respectively 27 and 0 degrees (see for further information on this aspect Berg, 1994).

It was proposed to transfer the initial stress condition obtained with the DIEKA calculation to a three dimensional Lagrangean small strain FE-code and then to simulate the volume loss due to the tunnelling proces.

6. CONCLUSIONS

To investigate the influence of shield tunnelling on adjacent and overlying structures by model tests, it is important that the distribution of stresses in the model is representative for the prototype under investigation. The geotechnical centrifuge is a valuable tool to investigate this kind of interaction problems.

The centrifuge model tests have provided a direct relation between the imposed volume loss and the volume of the settlement trough at ground surface. Due to dilatancy and arching in sand, the volume of the surface settlement trough may be smaller than the volume loss imposed at tunnel level. However, in case the tunnel is bored (partly) in soft clay, the volume of the surface settlement trough becomes significant larger than the volume loss imposed at depth.

The settlement trough dimension parameter (i) is constant for "normal" volume losses (0-3%). But for volume losses > 3% the parameter i decreases. This means that the settlement trough described with a Gaussian form will become smaller and deeper with increasing volume loss.

For 3 tunnel levels the settlement of the piles is measured as function of the volume loss imposed. The pile-tip-level is kept constant in all cases investigated (2m below the upper boundary of the sand layer). In the first test the tunnel is located completely in the sand, in the second test a large part of the tunnel is located in the sand and in the last test the tunnel is located only partly in the sand layer. The test results show that, in this case (tunnel diameter equal to 7 m converted to prototype quantities), at distances smaller than about 0,25 m times the tunnel diameter from the tunnel lining the loaded piles are severely affected by the tunnelling process. Large settlements of the piles have been observed. At distances of between approximately 0,25 and 1 times the tunnel diameter from the tunnel lining, pile settlements varied depending on the amount of volume loss imposed and the distance between pile and tunnel. The pile at a distance of about 2 tunnel diameters was never affected. The decompression of the dense sand layer due to volume loss was the main cause of the pile settlement and the reduction of the bearing capacity of the end-bearing piles.

The combination of physical modelling and numerical modelling provides have increased the understanding of the complicated phenomena that occur during the interaction between loaded displacement piles and a bored tunnel under construction.

7. ACKNOWLEDGEMENTS

The authors gratefully acknowledge the participants in this project, who provided the necessary funds: Municipal Management Amsterdam, Public Works Rotterdam, Ho lland Rail Consult, Bored Tunnel Combination (Ballast Nedam Beton en Waterbouw BV, Hollandsche Beton- en Waterbouw BV, Structon Betonbouw BV, Ways und Freytag AG), Dirk Verstoep BV (member of Tubecon), Koninklijke van Drunen BV (member of Tubecon), van Hattum en Blankevoort, Delft Geotechnics, Ministry of transport and public works, Ministry of Economic Affairs

The authors further acknowledge J. Out for the design of the equipment (model tunnel and pile loading frame), Th. van Dijk, F. Kop and W. Löning for their assistance during preparation and testing.

8. REFERENCES

Attewell, P.B. & Yeates, J. & Selby, A.P. 1986. *Soil Movements Induced by Tunnelling and their Effects on Pipelines and Structures.*

Berg, P. van den. 1994. Analysis of Soil Penetration. *Phd. thesis, Delft University of Technology.*

Bezijen, A. & Schrier, J.S. van der. 1994. The influence of a bored tunnel on pile foundations. *Centrifuge 1994.* pp. 681-686.

CUR C89, 1994. Invloed tunnelboren op paalfundering. *Reportnumber 177.*

Teunissen, J.A.M. 1991. Analysis of plasticity and non-coaxiality in geomaterials. *Phd. thesis, Delft University of Technology.*