Modelling the grouting process around a tunnel lining in a geotechnical centrifuge

Modélisation du processus d'injection de coulis de ciment du revêtement d'un tunnel dans une centrifugeuse géotechnique

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ABSTRACT: The grouting process around a tunnel lining is modeled in a geotechnical centrifuge. A model grout is injected in a scaled model of a tail void. In different tests grouting is performed with pressures equal to 80 and 90% of the total vertical stress of the soil above the tunnel. At the end of the test the pressure is increased to investigate the maximum pressure that can be applied before this will lead to large soil deformations. Deformations of soil and settlement of adjacent pile foundations were studied. It was concluded that for fully loaded piles, the pile deformations exceeded the soil deformation. To decrease settlements afterwards high pressures were needed, easily leading to fracturing and in some cases to further settlements of piles.

RESUME: Le processus d'injection de coulis de ciment autour du revêtement d'un tunnel est modelisé dans une centrifugeuse géotechnique. Un coulis modèle est injecté dans un modèle réduit de l'espace annullaire du tunnel. Par différents essais, l'injection est exécutée avec des pressions égales à 80 et 90% de la tension verticale totale du sol au-dessus du tunnel. A la fin de l'essai, la pression est augmentée afin d'étudier la pression maximale pouvant être appliquée avant que d'importantes déformations du sol n'aient eu lieu. Les déformations du sol et l'affaissement des bases adjacentes des pieux environnant le tunnel ont été étudiés. Il est conclu que pour des pieux supportant leur charge limite, les déformations de ces pieux ont excédés celles du sol. Afin de décroître l'affaissement de l'ensemble, de plus hautes pressions sont nécessaires conduisant à des fractures dans le sol et en quelques pieux. Dans certains cas, des affaissements de ces derniers ont été observés.

1 INTRODUCTION

The grouting of the lining is a critical process in shield tunnelling. Measurements, for example at the 2nd Heinenoord tunnel, (COB 1999) have shown that surface settlements are determined to a large extent by the quality of the grouting process. Grouting with a too low grouting pressure will lead to surface settlements, but a too high pressure can lead to a blow out of the grout and unpredictable deformations when the pressure is higher than the limit pressure that can exist in the soil at the depth of the tunnel. Furthermore a too low or too high grouting pressure can influence adjacent pile foundations.

The influence of tunnelling on surface settlement and pile foundations has been studied in a centrifuge (Bezuijen et al 1994, Loganathan et al 2000). In these studies the tunnelling process was simulated by a reduction of the volume of a model tunnel. However, with such a model it is not possible to find the maximum grouting pressure and it was found that the distribution of grouting pressure around the tunnel is of a major influence on the process. Therefore it was decided that the grouting process in the tail void has to be modelled by injection of a model grout in a scaled tail void.

In a research project commissioned and supervised by COB and performed by GeoDelft, equipment was made to investigate in a geotechnical centrifuge the influence of the grouting process on soil deformations and deformations of adjacent pile foundations.

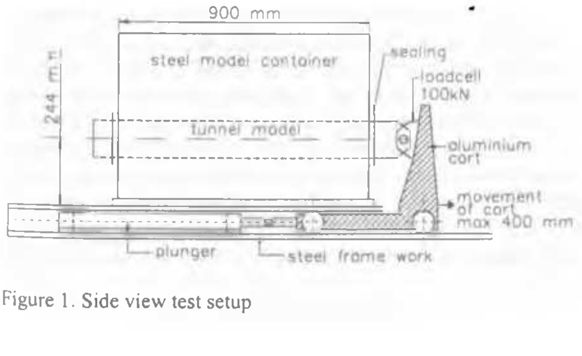

Figure 1. Side view test setup

Table 1. Scaling factors in centrifuge tests

Phenomenon	Prototype	Model
Volumetric weight	1	1
Dimension	N	1
Strain	1	1
Stress	1	1
Force	N^2	1
Time (consolidation)	N^2	1
Permeability	1	N

This paper deals with the equipment made and shows results of the tests which were performed. In this paper attention is focussed on the measured surface deformations and pile settlements. The measured blow-out pressures will be dealt with elsewhere (Bezuijen & Brassinga , in prep).

2 SCALE MODELS

In a geotechnical centrifuge the behaviour of soil can be studied in a scaled model. As the stress strain relation of soil is stress dependent, and if real soils are used as model material, the earth's gravity has to be enlarged. A model scaled 1:N has to be subjected to a gravity level of N times earth's gravity to comply with the correct strains and stresses. Each phenomenon in the soil is scaled in a specific way. In Table 1 some scaling factors are given.

3 TEST SET-UP

To simulate the tunnelling process, it is essential that equipment can move horizontally through the soil. A device was developed to enable this, see Figure 1. A plunger below the model container can move an aluminium 'cart' over a distance of 400 mm. Equipment connected to the cart can be moved horizontally through a watertight seal in the model container. The system can operate in tests up to 150 g. The maximum load that can be applied is 100 kN. To investigate the grouting process during tunnelling a so called tail void module was built. The module is shown schematically in Figure 2. An inner tube represents the lining. During the test, the outside tube, representing the TBM, is moved from the inner tube over a distance of 400 mm (on modelscale). During this process bentonite slurry is pumped through the supply system, connected with the outside tube, through the slits made in the inner tube. Bentonite slurry was injected by means of a plunger pump. The diameter of the outside tube is 130 mm, the diameter of the innertube is 125 mm. Some pore pressure gauges are placed at a distance of 200 mm behind the grout injection points, to measure the course of the pressure along the lining.

Bentonite slurry was used as a model grout. Slurry with 240 gr/l betonite was used to get a liquid with a scaled viscosity and yield strength as a grout in such a way that the penetration process of the grout in the sand is properly scaled.

The processes studied, surface settlement and limit pressure, occur when the grout is still in the liquid phase and therefore the cementing of the grout was not modelled.

Model piles have been placed into the soil above and next to the tunnel. Different configurations were used in different tests. In this paper most attention will be paid to the OLS tests, as shown

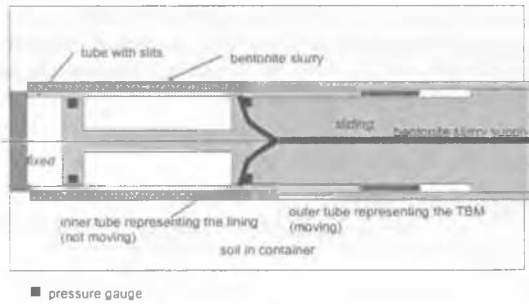

Figure 2. Sketch of the tail void module made to simulate the grouting process (not on scale).

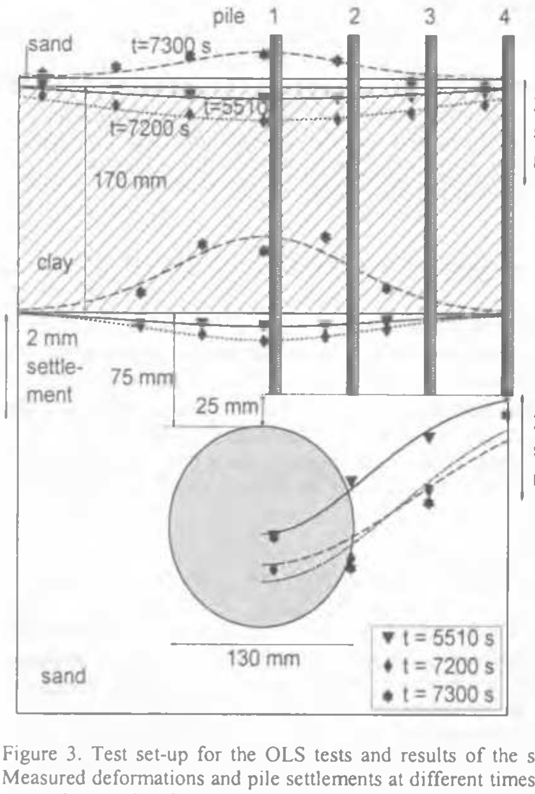

Figure 3. Test set-up for the OLS tests and results of the second test. Measured deformations and pile settlements at different times during the test and approximation with a gauss-curve.

in Figure 3. The piles were located in such a position that after the outside tube was moved over 290 mm the grout injection points were just below pile 1.

4 TESTS PERFORMED

4.1 *Type of tests*

Two types of test were performed. The first type was to test the equipment developed. This test was performed at the maximum design g-level of 150 g. This g level simulates a tunnel with a diameter of 18.75 m. A simple homogeneous sand model was chosen and only one pile was placed next to the tunnel.

The second type of test was performed at 40 g. This test was performed to test the possible influence of the drilling of a 5 m tunnel near Schiphol Airport as part of an underground logistic system. In this type of test the soil model was schematised from the typical soil conditions present in that area. The tunnel was placed in sand and a clay layer was placed above the sand, see Figure 3. A thin sand layer on top of this clay layer simulates the stronger unsaturated zone in the clay.

4.2 *Soil model and piles*

The sand model consisted of saturated sand. It was made by raining sand into the model container, which was filled with water and with the tail void module placed in position. The sand was brought to the desired density by repeated dropping of the model container on a concrete floor from a few centimeters height. The procedure is described in detail (Poel & Schenkeveld 1998). Measurements of the velocity of a compressive wave have shown that a very high degree of saturation could be obtained using this method.

The clay model was made from kaolin clay. The clay was preconsolidated at a pressure of 42 kPa. The undrained shear strength c_u of normally consolidated kaolin clay depends on the effective stress σ_v' (Bezuijen & Schrier 1994):

$$c_u = 0.21\sigma_v' \qquad (1)$$

In the OLS tests, performed at 40 g, 4 piles were used. The model piles were closed cylindrical aluminium piles with a diameter of 10 mm. Piles were placed at 10, 73, 136 and 199 mm from the tunnel axis, see Figure 4. After final installation during the test pile tips end 25 mm above the top of the tunnel, see also Figure 3. Before the test the piles were pushed through the clay, 7 mm into the sand at 1 g. The piles were connected to a loading frame by springs with a stiffness of 100 N/mm. During the test the freatic line was kept at 2 mm above the surface.

4.3 *Instrumentation*

Grout pressures were measured close to the injection points and at 200 mm behind the injection points with respectivily 4 and 2 total pressure gauges. Pore pressures were measured in the clay and in the sand at various locations. Furthermore, the soil pressure was measured below and next to the tunnel. The settlement of the soil surface and the settlement of the piles were measured by means of displacement gauges. The settlement of the top of the sand layer in which the tunnel was placed was measured by miniature piles placed 5 mm into the sand and connected to displacement gauges. The miniature piles were designed in such a way that the friction of the clay to the piles is much less than the weight of the piles at the desired g-level, but that the tip resistance is much higher than that weight. As a result the pile and the displacement gauge will follow the movement of the top of the sand. See Figure 3 and Figure 4 for the location of the instrumentation. Figure 4 shows that the location where the settlement was measured does not coincide with the location of the piles.

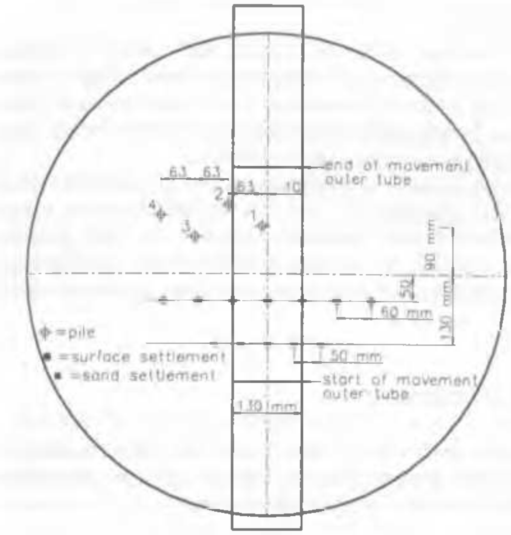

Figure 4. Top view of OLS test with instrumentation

During the OLS tests first the clay was reconsolidated to the desired g level, this took 5.2 hours. After that the piles were pushed 50 mm into the sand to acquire the desired depth and to generate a stress situation around the pile tip that is comparable to the prototype situation. The model was consolidated for another 2 hours to allow excess pore pressures to dissipate. Then the outside tube of the module was moved at a rate of 1 mm/s and simultaneously the tail void was filled with bentonite through the slits.
The pressure of the bentonite was controlled at the predetermined average level of 0.9 times the total soil stress at the axis of the tunnel. When the outside tube had reached a displacement of 390 mm its moving and the supply of bentonite were stopped and the betonite pressure was allowed to decrease. The second stage of displacement (10 mm) was performed at a rate of 0.01 mm/s without supplying bentonite, intended to cause a further decrease of the bentonite pressure. After the outside tube had reached maximum displacement (400 mm) the bentonite pressure was increased until the limit pressure was reached.

5 RESULTS

5.1 *Measured pressures*

The measured grouting pressures, the total horizontal soil pressure, the pore pressure in the soil during the experiment are shown in Figure 5. The pore pressure (p1) was measured in the sand layer, outside the influence of the tunnel. The grout pressures shown were measured 40 mm above (p3) and below (p4) the axis. The horizontal total pressure (p2) was measured at a distance of 10 mm from the tunnel, on the level of the axis of the tunnel. From this figure it turns out that the pore pressure in the sand around the tunnel hardly changes during the experiment. This means that the bentonite slurry, which acts as a model grout, hardly penetrates into the sand. The time during which the outside tube was moved under a controlled grouting pressure can be distinguished from the figure (from t = 5100 s until t = 5510 s), because during that stage there is some noise on the grouting pressures. Half way during this process the total pressure in the sand increases sharply (at t = 5350 s in Figure 5). At that time the end of the outside tube reached this pressure gauge. After the passage the pressure increase is caused by the model grout in the tail gap between the soil and the inner tube. Between t = 5550 s and t = 6500 s, while the moving and bentonite supply have stopped, the bentonite pressure drops because of the penetration of bentonite into the sand. Although the penetration is very small (see 5.3), the pressure drops because of the incrompressibility of the bentonite. It can be seen that during the second stage of the displacement of the outside tube, also without benonite supply, the grouting pressure drops to a level close to the pore pressure.

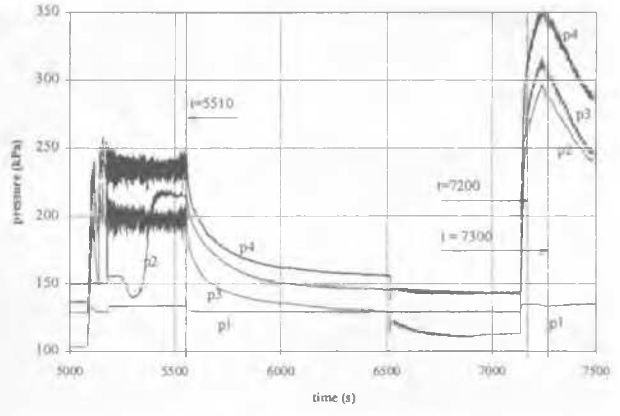

Figure 5. Measured pore pressure, grout pressures and total horizontal pressure

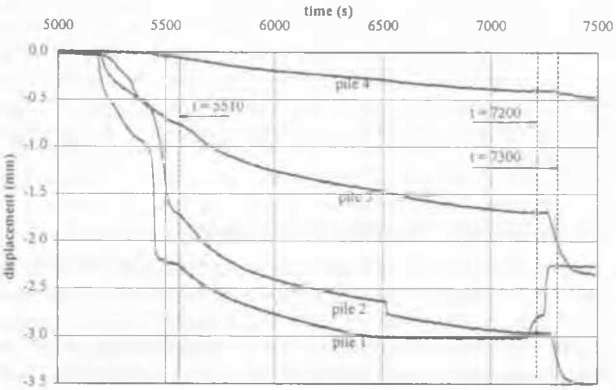

Figure 6. Measured settlement of piles

5.2 *Pile settlements*

The settlements of the piles are shown in Figure 6. The piles were kept loaded to the ultimate bearing capacity after they were pushed into the sand during the test. Because of the limit state situation around the pile tip in the test, any extra shear stress in the sand will cause settlement of the piletip. In practice the load will be less and therefore also the pile settlement caused by tunneling activities will be less. Settlement of the piles already occurs when the outside tube starts to move, even though the tail void is still far away from the piles. Settling of Pile 1 and Pile 2 increase sharply when the tail void passes underneath the piles (approximately at t = 5450 s). From Figure 6 it is also clear that the displacement of piles at a larger distance from the axis of the tunnel decreases. At a distance of 1.5 D_{tunnel} the influence of the tail void on the behaviour of piles has almost disappeared.
During the 'consolidation' stage (t = 5510 s to t = 6500 s) settlement of the piles continues caused by the decrease of the bentonite pressure in the tail void. The settlement of the piles exceeds the settlement of the surface and of the sand. This is caused by a decrease of strength of the soil around the pile tip. From t = 7200 s, when the grouting pressure is increased, the settling of the piles goes on, except pile 1 that moves upward. Figure 3 shows that soil heave is measured over a larger area, but the soil failure leads to a further settlement of the piles 2 and 3. From Figure 3 and Figure 6 it can be deduced that the soil failure has a large influence on the settlements. There is still a settlement trough at t = 7200 s, but at t = 7300 s (at approximately the same pressure, but after soil failure) heave is created above the

Figure 7. Model with bentonite after the test

tunnel. Pile 1 moves upwards, but less than the surface and the top of the sand. The capacity of the pile is increased, apparently in this case the soil strength does not decrease on top of the tunnel. Pile 2 and 3 both move downwards, while the surface and the top of the sand show heave. The loss of capacity of these piles has to be explained by loss of soil strength in the area next to the tunnel.

5.3 *Inspection of the model after end of test*

Figure 7 shows the model after removing the sand. From the inspection it turned out that the penetration depth of the bentonite into the sand was about 2 mm. The bentonite slurry remains around the tunnel due to the high yield stress. Reaching the limit pressure has led to the creation of lobs on both sides of the tunnel. At these locations a hydraulic fracture had occurred. This probably explains the decrease of the capacity of piles 2 and 3, placed aside of the tunnel.

The maximum pressure at which this fracture occurred was in between 2.2 and 2.5 times the vertical effective stress plus the pore pressure. More detailed information about this result will be published elsewhere (Bezuijen & Brassinga, in prep).

6 DISCUSSION

Surface settlements occur when the grouting pressure is less or equal to 90% of the vertical total stress. This means that under the soil conditions tested it is insufficient to use grouting pressures that avoid failure of the soil, as for example can be calculated by (Leca & Dormieux 1990). It is really necessary to grout with a pressure that is equivalent to the original soil stress to minimize settlements. Increasing the grout pressure after settlement of the soil surface has occured, has only a limited effect unless the pressure is increased to such a level that failure of the soil occurs. This is of course rather tricky because such high pressures can easily lead to a uncontrolable failure or blow-out of the grout.

The tests show that it is hardly to be avoided that heavily loaded piles will settle during tunnelling in the neighbourhood of the pile tips. The moving of the outside tube of the model tunnel already led to settlement, probably due to shear stress that is exerted on the sand. This means that during tunneling not only the grouting process but also the TBM itself can have an influence on pile foundations. For the conditions present during these tests, the influence decreases significantly within a distance of 1 time the diameter outside the tunnel.

7 CONCLUSIONS

1. The grouting process can be modelled in a geotechnical centrifuge, not only by using a contraction model but also, more appropriate, by really simulating the grouting process with a model grout.
2. To avoid surface settlement and pile settlement the grouting pressure has to be equal to the total vertical stress at minimum. High grouting pressures in the soil are necessary to reduce significantly settlements that have occurred before. Such high pressures can easily lead to fractures.
3. Pile settlements will be significantly for the conditions tested on, within a distance of 1 time the diameter from the tunnel. Pile settlement was sometimes reduced by high grouting pressures resulting in soil failure, but the tests showed that it is also possible, that such a process leads to further settlements

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