Investigating the influence of tunnel volume loss on piles using photoelastic techniques

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ABSTRACT: This paper presents the results of plane strain model tests on tunnel–pile interaction in a photoelastic material. The effects of volume loss are simulated by contracting a tunnel. The soil in this test is represented by crushed glass. This allows for the determination of stresses in the model by the photoelastic method. The influence of the stress change in the soil due to volume loss is shown, as well as the effect on three rows of piles at varying distance from the tunnel. From the tests it is clear that significant stress changes occur close to the pile tips.

1 INTRODUCTION

The upcoming boring of the North-South metro line in Amsterdam (Netzel & Kaalberg 2000, Kaalberg et al. 2005) will involve TBM excavation close to the pile tips of the wooden piles on which much of the historic inner city is founded. The influence of a volume loss due to tunnelling on the bearing capacity of these piles is relatively uncertain and hard to quantify even by numerical means (Broere & van Tol 2006). This prompted a different approach, to visualise the influence of volume loss on the stresses around the pile tips using the photo-elastic method. Photoelasticity has been used extensively to quantify stresses in homogeneous materials (e.g. Frocht 1948). And is extended to particle assemblies in index matching liquids by Drescher (1976).

A small research effort was started in 2002 to attempt to quantify the influence of a volume loss on the stresses close to pile tips in a photoelastic scale model. As the interpretation of the measurements proved more complicated than originally anticipated, it took considerably more effort to finalise the analysis and derive the complete stress state. An overview of these complexities is given in Petrucci & Restivo (2007). Standing & Leung (2005) had similar considerations as outlined above when they did comparable photoelastic scale model tests, but they decided to invert the problem and only quantify the influence of pile installation on the stress in existing tunnel linings.

2 STRESS MEASUREMENTS

It is possible to obtain detailed information on the stress distribution of a granular material using the photoelastic measuring method. The soil is replaced by grains of a photoelastic material, in this case glass particles (Wakabayashi 1957, Drescher 1976). Crushed glass behaves similar to sand particles, although, the grains are more angular. Therefore it is a reasonable substitute to investigate sand behaviour. However, other authors, most notably Lesniewska & Sklodowski (2005) argue that when uniform glass beads are used instead of crushed glass the stress trajectories a long failure planes can be better visualized.

When compared to the photoelastic measurement methods for continuous materials the quantification of the stress paths for a granular material is hampered. First, in order to eliminate light scatter the pores have to be filled with a liquid which has a similar refraction index as the glass particles. Secondly the analysis of the fringe patterns is impossible because of the stacked nature of the sample. Each layer of grains produces its own fringe pattern resulting in a mix of fringes which cannot be interpreted. The first condition leads to the qualitative visualization of these stress paths by a circular polariscope, see i.e. Wakabayashi (1957). For the second problem, quantification of the stress paths, Allersma (1982) developed an automated polariscope.

In general, granular material does not behave elastically, as is often assumed when formulating a
stress-strain relationship. Therefore it is not possible to derive the stress increments from strain measurements. The stresses have to be measured independently of the strains.

The method developed by Allersma (1987) and Allersma & Broere (2002) for measurement of the stress in a photoelastic material is followed. The method already published by Allersma in 1982, is most comparable to phase-stepping photoelasticity as described in e.g. Ajovalasit et al. (1998). The direction of the principal stress and the principal stress difference along the light path are measured with an automated circular polariscope. Therefore averaging of the stress in the direction normal to the plane is taken into account.

In a photoelastic material the refraction in each direction is dependent on the normal stress in that direction. The stress difference between the principal stress directions, i.e. twice the maximum shear stress, \( 2 \tau_{\text{max}} \), leads to a relative light velocity difference and subsequently a change in polarisation. This change in retardation can be measured to obtain the shear stress in a point.

The maximum shear stress in a point can be derived from the elliptical polarisation of the light. The light changes from circular polarised light into elliptical polarised light when travelling through the sample, caused by the stress in the material.

If at one or more points the complete stress state is known, the absolute values of the stress tensor can be determined by integrating the equilibrium equations. Unfortunately, this scheme is very sensitive for experimental noise, which hampers the results. In the present tests the vertical stress is calculated from the measured surcharge at the top of the sample.

3 TEST SETUP AND TEST PROCEDURE

3.1 Test setup

The test setup consists of a model container with inner dimensions of height \( \times \) width of 690 mm \( \times \) 530 mm and a depth of 70 mm. In Figure 1 a front view of the test setup is given. The front and rear sides are made of glass. The rectangular piles are made of stainless steel with a cross sectional area of 10 \( \times \) 10 mm. Two rows, with a c.t.c. distance of about 25 mm, of three piles are installed at respectively 0.5\( D_{\text{tunnel}} \), 1\( D_{\text{tunnel}} \) and 1.5\( D_{\text{tunnel}} \), simulating a piled foundation. Also a tunnel is embedded at 345 mm from the side and 232 mm from the bottom of the container.

The tunnel consists of an upper and a lower part milled of stainless steel which can be pushed out vertically by an inner stepper motor. Therefore, the volume change of the tunnel is primarily caused by vertical contraction. The mechanical realisation of this tunnel
can be seen in Figure 2. An additional surcharge is applied on a steel beam to increase the stress level in the sample. This surcharge is applied by spring loading the beam, i.e. between a fixed boundary and the beam a mechanical spring is applied, such that small displacements in the sample do not totally unload the boundary. Between the spring and the fixed boundary a load cell is monitoring the applied load.

The box is filled with crushed glass particles and the index matching liquid. The particle size of the glass particles is between 2 and 3 mm. The material behaves similar to sand of the same particle size as shown in triaxial tests performed by Allersma. According to Allersma, the angle of internal friction at constant volume, $\phi_{cv}$, for angular silica sand of this particle size is ca. 33° and for crushed glass ca. 39°.

3.2 Test procedure

Medium dense conditions are obtained by first pouring the glass into the strongbox, followed by densification of the sample, and finally pumping the liquid through the sample from bottom to top. The tunnel is undisturbedly embedded during preparation, while keeping the diameter at its maximum position. The piles are pushed in afterwards before applying the surcharge. A surcharge of 75 kPa is applied at the steel beam after preparation of the sample is finished. Before starting the test first an equilibrium stage has to be reached in which no additional creep is apparent. During this phase the surcharge is kept constant at 75 kPa.

The objective of this test is to investigate the effect of volume loss on existing pile foundations, therefore at the beginning of the test the tunnel is in its opened position. Also the piles are already embedded. Subsequently a photoelastic measurement is made by means of a mechanical polariscope. In the next phase, without altering the surcharge, the tunnel is contracted by decreasing the distance between the upper and lower tunnel parts by 2 mm or 1.2%. This results in a volume loss of only 0.6%, as the tunnel contracts vertically only and not uniformly. This will result in an unloading of the soil around the tunnel. Again a photoelastic measurement is made.

The mechanical polariscope is mounted on a computer controlled x-y scanner and scans approximately 1200 stress points spaced at 10 mm interval in an area of ca. 340 mm $\times$ 340 mm covering half of the tunnel and the lower part of the embedded piles. The measurement zone is also shown in Fig. 1 (dotted lines).

4 TEST RESULTS

The results of the performed tests are summarized in three figures. The first Figure, Fig. 3, is showing the principal stress directions and the magnitude of the principal stresses measured before tunnel contraction, the second Figure, Fig. 4, is showing the same data, but now for the situation after tunnel contraction. The principal stress crosses are all plotted with the same scale, i.e. the length determines the magnitude. Finally, the last figure, Fig. 5, plots the contour plot of the difference in the measured maximum shear stress. The “before” measurement is taken as reference, resulting in a positive value for the difference when the maximum shear stress in a point before contraction was higher than after contraction and logically a negative value when the value afterwards is higher than before. The measured surcharge...
4.1 Before tunnel contraction

When studying the results of the measurements taken before tunnel extraction, given in Figure 3 several things are noticed, for sense of scale the measurement points are horizontally and vertically spaced at 10 mm. Firstly the principal stress rotations right on top and below the tunnel are all negligible, the tunnel is loading the soil in compression, while to the side large stress rotations occur. The behaviour is therefore vertical symmetrical.

Another observation is that the spacing between the piles is such that arching seem to occur, spots without any rotation and with a lower stress magnitude can be observed. Clearly, a spacing of about seven times the pile diameter is still sufficient to cause this effect, however scale effects cannot be excluded as the grains are relatively large compared, $d_{\text{grains}} = 2–3 \text{ mm}$, to the pile diameter which is 10 mm. The most right pair of piles (remember two rows of piles are installed behind each other when seen from the front side), or the pile row next to the boundary of the model container is distributing its load immediately in the side wall.

As directly below a large undisturbed zone is seen. This is clearly a boundary effect. Lastly the pile closest to the tunnel is influenced by the tunnel, as between the pile and the tunnel a direct path of parallel stress directions can be seen. This pile row is at a tip–tunnel distance of about 12 cm or $12D_{\text{pile}}$ or $0.75D_{\text{tunnel}}$.

4.2 After tunnel contraction

Again the principal stress rotation and magnitude are plotted in Figure 4, this time for the situation after tunnel contraction. All former observations still apply, pile arching seems to still occurring, the closest pile row is still influenced by the tunnel and is unexpectedly in a higher stress regime. However, closer inspection reveals that the band of stress between the closest pile row and the tunnel seems smaller in width i.e. the stress intensity is lower. Also, the stresses directly below the tunnel seem changed also, the undisturbed zone below is smaller. From an experimental point of view this can be expected, because as the tunnel is decreased in height, the largest changes are expected in the vertical stress distribution. Not only unloading on the top side of the tunnel occurs but also an elastic rebound on the bottom.

4.3 Shear stress difference

It is rather difficult to observe differences in magnitude in principal stress from the already shown figures, therefore one additional figure is presented. In Figure 5 the difference in measured maximum shear stress between the situation before and after tunnel contraction is plotted as the difference is more clear from this data than from the processed data. The numbers plotted next to the x-axis and y-axis are the distance from the origin of the measurement in mm. The values given in the colourbar of the contour plot are in kPa, positive values indicate a lower stress than before tunnel contraction, negative values point towards a higher stress than before contraction.
It is immediately clear from this data that an increase in stress around the pile tips of the three pile rows is found. Another observation is that the first two pile rows are most affected by the contraction. The zones with the highest stress concentrations, in this case around the pile tips and at the point at which the stress arch reaches the tunnel are taking all the stress change, while the general field does hardly change at all. The last finding further supports the observation that creep in the sample during the tests is kept at a minimum. The increase in maximum shear stress can be explained by the fact that in the model setup due to volume loss the soil immediately begins to create movements of soil below the pile rows resulting in negative friction on the piles.

5 DISCUSSION

It is our opinion that for the case considered, a scaled plane strain situation with rather large grain size and low stress situation, the results are still convincing. The effects of nearby tunnelling are seen in the measured stress field. Increase of maximum shear stress can be explained by negative friction on the piles. However, the change of the horizontal or vertical stress is more difficult to acquire, both due to difficulties from noise in experimental data, as measuring properties of light is always very sensitive to experimental errors, and due to the integration of the measured stress field. Therefore, no conclusive answer can be given at this stage.

All pile rows in the presented setup suffer from the tunnel contraction effects, but the first row is clearly most influenced, not only by the tunneling but also in the reference situation interaction is seen between the two structural elements. The influence of the tunneling on the last pile row is minor, but this pile row is already located at $1.5D_{tunnel}$ to the center line of the tunnel.

When compared to the results of Standing & Leung (2005) a bit more information is derived from the measurements. In the current setup the model scale, especially when considering the dimensions of the pile, the model container and tunnel, are more realistic. Similar to Standing & Leung (2005) in our measurement nearby piles attract more stress lines, which reduce in intensity after the tunnel contraction. This indicates a stress release near the pile, which is consistent with the increase in shear along the pile as the pile starts to settle.

6 CONCLUSIONS

The influence of a contracting tunnel on the stresses near the tips of displacement piles has been studied using photoelastic model tests. From these tests, the changes in principal stresses and shear stresses near the tunnel and piles have been quantified. Given the limited size of the scale model, and the relatively large grains used to improve the visibility of the photoelastic effect, some limitations of the model have come to light and these have been discussed. Especially as there is some evidence of stress arches between the different piles, the results of these tests cannot be translated directly to a field situation but should be interpreted with care.

Nonetheless, the tests show a clear influence of a volume loss of the tunnel of 0.6% on the stresses near the pile tips up to $1D_{tunnel}$. Here stress bands are decreasing in width, indicating a lowering of the principal stresses, and at the same time an increase of the shear stresses along the pile shafts is seen for these piles. These observation corroborate those of Standing & Leung (2005), but also indicate that for displacement piles close to failure the influence zone of a tunnel volume loss is wider than suggested there. Contrary to their findings not only piles with their toes inside the influence zone defined by Jacobsz et al. (2003) are affected, but also those just outside of that zone.

This suggests that the influence zone to be taken into account for displacement piles, which depend on both end bearing and skin friction, might be slightly larger than for bored piles, which are mostly founded on end bearing alone. To determine this would require a combination of further field observations, model testing and numerical modelling of the problem.

REFERENCES


