Geotechnical Aspects of Underground Construction in Soft Ground – Ng, Huang & Liu (eds)
© 2009 Taylor & Francis Group, London, ISBN 978-0-415-48475-6

Mechanisms that determine between fracture and compaction grouting in sand

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ABSTRACT: Laboratory tests have shown that injection of grout in sand can lead to different shapes of the grout in the sand. At low water cement ratios (1–2) there is usually compaction grouting, which leads to a spherical shape of the grout. At higher water cement ratios (5–20) fracturing of the sand will occur, leading to rather thin grout structures. In field observations this difference is less obvious. It is assumed that in most cases fractures occur, although it is hardly ever possible to examine what is made in the soil. This paper describes conceptual models and calculation models that explain, at least qualitatively the differences in the behavior of the grout and discusses a possible reason for the difference between the model tests and the field tests.

1 INTRODUCTION

Compensation grouting has been successfully applied in several projects to prevent or to compensate for surface settlements induced by for example tunnelling (Mair & Hight, 1994; Chiriotti et al. 2005; Christiaens et al. 2005). Compensation grouting uses hydraulic fracturing to get a heave that can compensate the settlement or compaction to densify the soil with limited heave. In the latter situation the compaction lead to improvement of the soil characteristics.

Laboratory tests have been performed to investigate the mechanisms that are of importance for compensation grouting in clay (Jaworski et al. 1981, Mori & Tamura 1987, Andersen et al. 1994, Chin & Bolton 1999, Au, 2001). Comparable tests have been performed for compensation grouting on sand (Chang, 2004, Kleinlugtenbelt et al. 2006, Bezuijen et al. 2007 and Gafar & Soga, 2006). These tests showed that compensation grouting by fracturing is not so straight forward in sand. After quite a number of tests with a water cement ratio of 1–2 it appeared that there were hardly any fractures in the sand, but a more or less spherical shape of grout is formed in the sand, see Figure 1. Fractures could be obtained when grout with a higher water cement ratio (5–20) is used (Bezuijen et al. 2007).

Similar results were found in at least 3 different laboratories. Different results were, however reported from the field (Grotenhuis, 2004) based on measured surface displacements. Normally it is not possible to investigate the fractures created, but this was possible during the construction of the station for the high speed train in Antwerp and during the construction of the cross-passages at the Hubertus Tunnel in The Hague, the Netherlands.

Figure 1. Results from Cambridge University (left) and GeoDelft that show more a spherical shape than fractures.

In Antwerp compensation grouting was applied to prevent settlements on the existing station (Christiaens et al. 2005). The resulting fractures were found in a soil layer with a lot of shells. A clear fracture was found between the shells. Here it is possible that the shells have influenced the fracture. The Hubertus Tunnel case was an unintended fracture that occurred when the lances for freezing were brought in and the grout pressure used to install these lances was large enough to create fractures.

The paper will describe the Hubertus Tunnel case more in detail, since this is one of the very few cases where fractures in relatively homogeneous sand at 11 m depth and 5 m below the waterline could be studied and describes the principle and results of calculation models that are developed to describe various parts of the grouting process. Possible causes of the differences between the field test and the model tests are: 1) the preconditioning has an influence. 2) the in-homogeneities in 'real' soil serve as a trigger for the start of the process. 3) The installation of the TAMs (TAM stand for Tube-a-Manchette, a tube with injection openings covered by a rubber Manchette through which the grout can be injected) which causes unloading of the soil has an influence. In this paper we will elaborate the 3rd possibility and mention some aspects of the 2nd. The first possibility is not dealt with in this paper.

Results of the model tests itself will be described and analyzed in another paper on this conference (Gafar at al., 2008).

2 THE HUBERTUS TUNNEL

The Hubertus Tunnel is a double track road tunnel that is constructed in dune sand. The tunnels have a diameter of 10 m. Five connections were made between the two tunnels. Soil freezing was used to make these connections. Lances were constructed from the tunnel lining into the sand to freeze the soil. A connection between the two tunnels was made through the linings and the concrete connection structure was made. For a cross-section see Figure 2.

The tunnel crown is placed 11 m below the soil surface and the phreatic level is 5 m below the surface. The vertical pressure at the top of the tunnel is estimated to be around 140 kPa.

Grout was used to install the lances for freezing. During the installation of these lances for the first connection, the pressure was increased so far that the grout created hydraulic fractures. These fractures became visible when the frozen soil was removed.

Figure 3 shows an example of fractures in the sand around the tunnel and between the tail void grout and the sand.

The picture is taken standing in the newly created opening, from left to right you see the opening, the tunnel lining, the tail void grout, grout from a lance that penetrated between the tail void grout and the sand and the frozen sand with a fracture in the upper part of the sand. There were also fractures that were only visible in the sand.

The normal pressure to inject the grout is 3 bar. However, according to the employees of the contractor on the site the pressure could have been higher during this injection because there were doubts on the pressure gauge. The grout had a water-cement ratio (W/C) of approximately 1 and 3% of bentonite was used in the mixture.

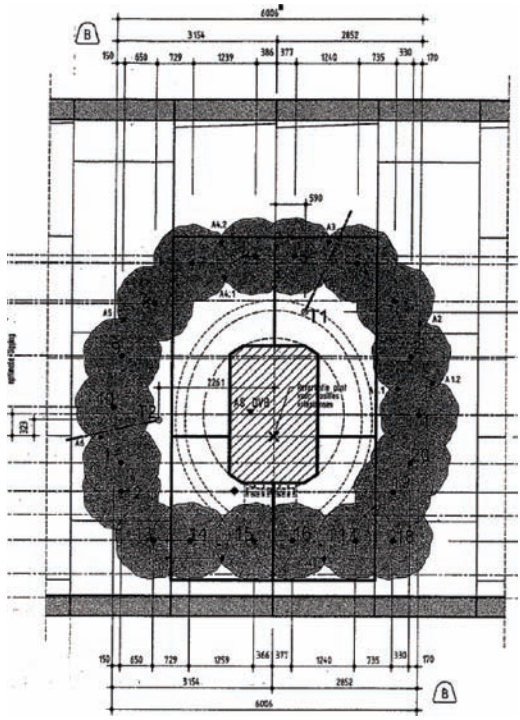

Figure 2. Sketch of a connection between the tunnels made by freezing.

3 COMPARISON WITH LABORATORY TESTS

The laboratory tests performed at GeoDelft, as described by Bezuijen et al. (2007) and Gafar et al. (2008) have been performed in a cylindrical tank with a diameter of 0.9 m and a height of the sand sample of 0.85 m. The sand was pressurized to get a vertical effective stress of 100 kPa. Different grout mixtures were injected and different injection procedures were used. Most mixtures injected had 5% or 7% bentonite. A fracture like behaviour only occurred when the W/C ratio in the mixture was 2 or higher. Lower W/C ratios would not create fractures, but lead to the shapes shown in Figure 1 (right picture). The maximum injection pressures varied in most cases between 20 and 30 bar.

Comparing the main results of the model tests with that of the field observations, there seems to be a discrepancy. The grout that does not lead to fractures in the model, does result in fractures in the field. The injection pressure in the field situation described is not known, but it is unlikely that where the normal

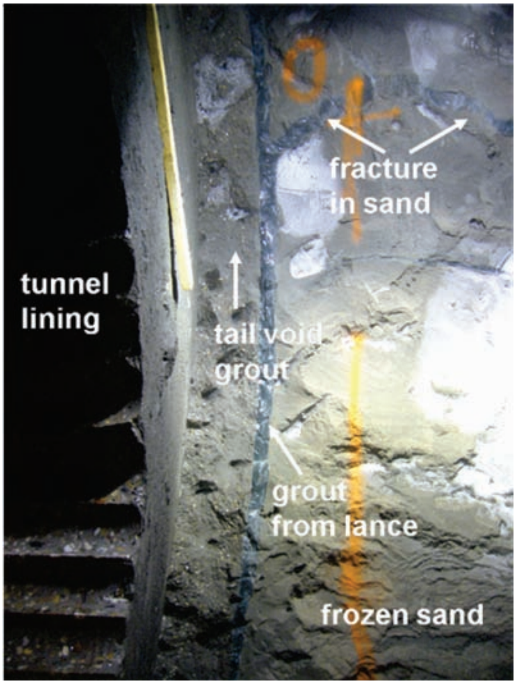

Figure 3. Fractures found during the construction of the first connection between the tunnels of the Hubertus tunnel.

injection pressure is 3 bar, it was 28 bar during the construction of the first cross-section (this would be the injection pressure based on the laboratory tests and extrapolating the results of these tests to 140 kPa vertical pressure).

A possible reason for the discrepancy between the model tests and the field behaviour can be the soil boundary condition, as will be explained in the next sections.

4 RELEVANT GROUT PROPERTIES

4.1 *Plastering*

Bezuijen & Van Tol (2007) have described how plastering of the grout influences the fracturing behaviour. Fracturing occurs because the radial stress in the sand around a fracture is higher than the angular stress. Using the Mohr-Coulomb criterion, the relation between radial stress and angular stress perpendicular on the radial stress can be written as:

$$\sigma_\theta = \frac{1-\sin\phi}{1+\sin\phi}\sigma_r \tag{1}$$

Where σ_r is the radial stress, σ_θ the tangential stress and ϕ the friction angle of the sand. Taking for example

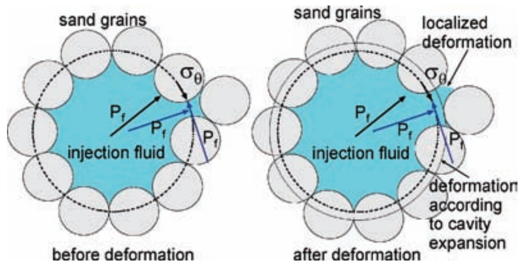

Figure 4. Sketch with possible deformation modes of the injection hole. In reality there will be more grains around the injection hole. The figure just shows the principle.

a friction angle of 35 degrees, the tangential stress is only 0.27 times the radial stress.

Looking at the scale of the grains, sand is not homogeneous and isotropic, as assumed in continuum mechanics. At the boundary of the opening there will be some grains that are in closer contact and some between which there is some space, see Figure 4.

Such space can be sufficient to have the fluid pressure not only in radial but also in tangential direction. The fluid pressure is with perfect plastering equal to the radial stress and much higher than the tangential stress and therefore this will lead to an opening of the space between the grains and a fracture can occur. Some in-homogeneity in the soil as is often present in a field situation (more than in the model test) will facilitate the fracture formation, because weak sections in the soil will deform. If there is a beginning of a fracture then the fluid pressure will penetrate further in the sand and the fracture will easily grow further. A fracture will stop to propagate when the pressure in the fracture tip drops due to friction losses, leak-off or increase of the volume with injection fluid.

Plastering and the formation of a filter cake in the injection hole will hamper fracturing of the sand, because now possible irregularities at the boundaries of the injection hole will be filled with plaster see Figure 5. This plaster also has certain strength and therefore prevents the fluid pressure from penetrating into the space between the grains.

As is also mentioned in Bezuijen en Van Tol (2007), plastering is caused both by consolidation of the grout and by leak-off of grout in the sand while larger particles in the grout are filtered by the sand, remain at in the injection hole and cause a plastering layer at the boundary between the injection liquid and the sand.

With a constant injection pressure, the consolidation of the grout mixture, without leak-off, can be approximated by formula (Bezuijen et al. 2007):

$$s_1 = \sqrt{2k\frac{n_i - n_e}{1-n_i}\Delta\phi\, t} \tag{2}$$

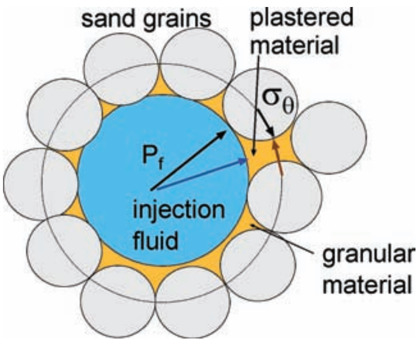

Figure 5. Influence of plastering.

Where s_1 is the thickness of the consolidated layer (the plastering layer), n_i the initial porosity, n_e the porosity after consolidation, k the permeability of the consolidated layer, $\Delta\phi$ the difference in piezometric head over the column and t the time.

This formula implies that the thickness of the plastering layer increases with the square root of the applied pressure, a lower pressure will lead to a thinner plastering layer.

4.2 *Leak-off*

Assuming that the flow during leak-off can be described as a Bingham liquid flow, the relation between the difference in piezometric head and the thickness of the plastering layer of courser materials can be written as (for $v > 0$, flow into the sand):

$$\Delta\phi = \frac{s_1 v}{k_1} + \frac{s_2 v}{k_2} + \alpha_1 s_1 + \alpha_2 s_2 + Rv \tag{3}$$

Where $\Delta\phi$ is the difference in piezometric head, s_1 the thickness of the grout cake (the plastering layer), s_2 the distance the liquid has penetrated into the sand, k_1 and k_2 the permeability of the cake and the sand respectively for the penetrating liquid, v the velocity, α_1 and α_2 factors for the cake and the sand respectively that determine the drop in piezometric head caused by the yield stress in a Bingham liquid and R the flow resistance from the soil around the fracture. When this equation is dominant, the thickness of the plastering will be proportional with the applied difference in piezometric head.

Experimental research by Sanders (2007) has shown that consolidation determines the thickness of the plastering layer for grout with a low W/C ratio, leak off is the dominant mechanism for the thickness of this plastering layer at W/C ratios of 5 and more. Normally a filter cake caused by consolidation of leakoff is made of finer particles than the sand of the subsoil. Due to this leak off will stop when there is some thickness of the filter cake.

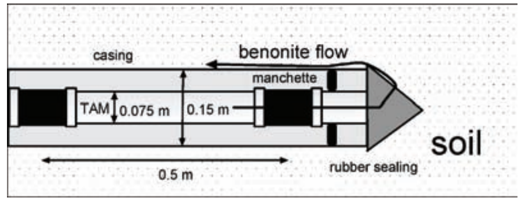

Figure 6. Sketch of casing and TAM during installation.

5 INFLUENCE OF SOIL DEFORMATION

In the experiments preformed in Waterloo, Cambridge and Delft, the sand around the injection point is reasonable uniform and the vertical confinement pressure is constant over the soil sample. Injection of grout will lead to cavity expansion until a fracture occurs.

A different pressure distribution is present around the injection points at the Hubertus Tunnel and in general when compensation grouting is applied in the field. At the Hubertus tunnel the freezing lances were applied between the two tunnels. Consolidation of the tail void grout leads to a stress reduction in the vicinity of the tunnel (Bezuijen & Talmon 2003). Arching between the 2 tunnels can lead to a further reduction of the stresses. Also the placement of lances, or in case of compensation grouting of TAMs, leads to a reduction of stresses. The TAMs are placed using a casing; see Figure 6 for a sketch. Figure 6 shows the situation where the TAM is placed by displacement of the soil. Another option is that the bentonite is used to remove the soil. The latter option leads to more unloading of the soil. Whatever option is chosen, when the casing is removed there will be some unloading of the soil although grout is injected to stabilize the bore hole, as appears from (minimal) settlements measured during the installation of the TAMs. (Kleinlugtenbelt, 2006).

The situation that occurred during compensation grouting can be compared with the situation of a cavity under repeated loading. Such a situation is analyzed by Wang & Dusseault (1994). They used cavity expansion theory to calculate the stress distribution in the soil around a bore hole during unloading and re-loading. In the case of compensation grouting, the removal of the casing will result in unloading and the reloading will occur during cavity expansion. Cavity contraction and expansion theory cannot describe when a fracture will start, but it can be used to calculate what pressures will lead to a plastic zone in the soil. Some plastic deformation in the sand is necessary before a fracture can occur. Plastic deformation will lead to dilatancy in the sand, which results in more space between the grains. This makes it easier for the grout to separate the grains further according to the mechanism described in Section 4.1 leading to a fracture, because there are locally zones with a higher permeability.

Parts of Wang & Dusseault's analyses will be presented here in a shortened and slightly adapted version to allow calculation of the influence of previous unloading on the loading pressure at which plastic deformation occurs. It is assumed that after installation of a TAM there is a pressure release due to the removal of the casing that leads to an active failure of the soil around the hole. This means that the radial stress, σ_r, is smaller than the tangential stress, σ_θ. After installation, the radial pressure is increased due to the fracturing process and passive yield will occur. In such a situation σ_r, is larger than the tangential stress, σ_θ and a fracture can occur. Here we assume that the fracture initiation pressure is related with the pressure that results to passive failure of the cavity.

Assuming a linear Mohr-Coloumb criterion, the plastic stresses must fulfill the criterion:

$$\sigma'_\theta - N\sigma'_r + S = 0 \tag{4}$$

Where the prime indicates the effective stress.

The material parameters N and S are different for active and passive yield and can be written as:

$$\begin{aligned} N_a &= [1+\sin\phi]/[1-\sin\phi] \\ N_p &= [1-\sin\phi]/[1+\sin\phi] \end{aligned} \tag{5}$$

and

$$\begin{aligned} S_a &= -2c_0\cos\phi/[1-\sin\phi] \\ S_p &= 2c_0\cos\phi/[1+\sin\phi] \end{aligned} \tag{6}$$

(The publication of Wang and Dusseault discriminates between peak values and residual values, here we use only one value.)

Here ϕ is the friction angle and c_0 the cohesion of the soil material. Active yield will occur when:

$$p_a < \frac{2\sigma'_0 + S_a}{1+N_a} \tag{7}$$

Where p_a is the pressure at the boundary of the opening in the active state (assuming a perfect plastering on this boundary) and σ'_0 the initial effective stress around the opening.

When the effective pressure around the TAM remains higher than the criterion mentioned in Eq. (7), there will be no plastic deformation and the limit pressure for passive plastic deformation will remain the same as if there was no unloading on the soil.

In case the criterion of Eq. (7) is fulfilled, there will be an active plastic zone. Increasing the pressure in the cavity afterwards will lead to, the situation sketched in Figure 7. There will be an active plastic zone and within that a passive plastic zone. When the pressure is increased further the active zone will disappear, but our interest is the pressure at which the passive zone started.

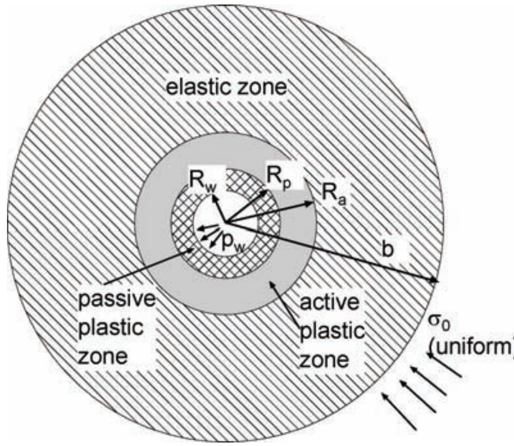

Figure 7. Definition sketch, for the situation with active and passive soil around the cavity, see also text.

Table 1. Input parameters used in calculation.

σ_0 original stress in sand	100	kPa
ϕ friction angle sand	35	degr.
c_0 cohesion	0	kPa
R_w (radius of tube, see Figure 7)	0.035	m
b (radius with constant press)	5	m

During unloading, when the active zone is formed P_r, Based on the equations presented by Wang and Dusseault, the pressure in the cavity, can be expressed as a function of the thickness of the plastic zone R_a and other parameters (see also Figure 7):

$$P_r = \left[p_0 - H\frac{b^2 - R_a^2}{b^2 R_a^2} - S_a \right] \left(\frac{r_w}{R_a} \right)^{N_a} \tag{8}$$

Where:

$$H = \frac{\sigma'_h (N_a - 1) - S_a}{b^2(N_a+1) + (1-N_a)R_a^2} R_a^2 b^2 \tag{9}$$

When after active yield the pressure is increased the first soil will come in the passive plastic state when:

$$p_p > \frac{2p_a + S_p - S_a}{1+N_p} \tag{10}$$

Without the active failure this relation would read:

$$p_p > \frac{2\sigma'_0 + S_p}{1+N_p} \tag{11}$$

Table 1 presents the input parameters used for calculations using the formula's presented above. Results

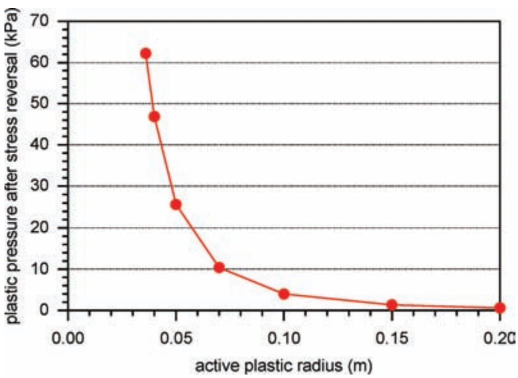

Figure 8. Pressure that is necessary to create passive plastic deformation after that first active yield is imposed as a function of the original active plastic zone.

are shown in Figure 8. Without active plasticity the pressure where passive plasticity starts is 157 kPa. From these results it is clear that plastic deformation in the active state can considerably reduce the borehole pressure at which passive plastic deformation occurs. The pressure needed to achieve passive plastic deformation decreases as the active plastic radius created by the low pressure before the pressure increase is larger. This presents quantitative information for the statement already mentioned by Wang & Dusseault: ‘Our study suggests that initial active formation damage reduces the pressure required to initiate such a fracture’.

6 DISCUSSION

In Section 4 it was shown qualitatively that only with a limited filter cake between the grout and the subsoil a fracture can occur in homogeneous soil. In in-homogeneous soil the formation of a fracture is easier, because also the initial deformation will not be symmetric. For this reason there was a fracture between the tail void grout and the sand at the Hubertus Tunnel.

The calculation model by Wang and Dusseault, as described in Section 5, shows that the pressure to create a fracture is considerably lower in case of a cavity that has been subjected to plastic unloading, compared to a cavity that is still in an undisturbed state. This last phenomenon is the most reasonable explanation that the injection pressure in the field is lower than the pressure measured in the laboratory. A lower injection pressure also means that the filter cake due to plastering and leak-off is thinner and thus that the grout mixture is less critical. This may be an explanation that fractures are created in the field for conditions that does not lead to fractures in the laboratory. Further experimental work has to prove and quantify this idea.

Consequence may be that the compensation grouting is sand is influenced not only by the grout, but also to a large extend by the stress distribution around the bore hole where the TAM is installed. The installation procedure will influence the shape of the fracture and the pressure needed to create a fracture.

7 CONCLUSIONS

From our study we came to the following conclusions:

1. Fractures can occur in a field situation where for comparable soil and using the same grout only cavity expansion is measured in a model test. This can be caused by the heterogeneity in the field and by the way the TAMs are installed in the field.
2. Unloading of the soil before it is loaded by injection of grout will lead to a reduction of the injection pressure.
3. A reduced injection pressure will lead to a reduced cake thickness because both bleeding and leak-off will be reduced.
4. In further experimental research it will be necessary to investigate the influence of the installation procedure and in-homogeneity of the soil.

ACKNOWLEDGEMENT

This work was supported by Delft Cluster. The authors wish to thank The Hubertus Tunnel Combination for the information presented on the first cross-connection of this tunnel.

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