Imperfections in jetgrout layers

Imperfections dans les couches de coulis de ciment injecté à haute pression

A.F.van Tol – *Rotterdam Public Works and Geotechnical Laboratory, Delft University of Technology, the Netherlands*
S.Koster – *Delft University of Technology, Ballast Nedam Engineering, the Netherlands*
J.P.G.Ramler – *Ballast Nedam Engineering, the Netherlands*
A.Verruijt – *Geotechnical Laboratory, Delft University of Technology, the Netherlands*
J.K.Vrijling – *Hydraulic Engineering, Delft University of Technology, the Netherlands*

ABSTRACT: In recent years, jetgrout layers and walls have been used as an impervious barrier in building pits. With high laying jetgrout layers small openings which may be present can lead to sand-carrying wells which are difficult to control and which have already caused major problems in some projects. This paper describes the results of experimental research carried on the failure mode and stability of the soil in the imperfections in jetgrout layers. The experiments confirm the type of failure described with an analytical solution for the stability of a sand prop in a hole with a high hydraulic gradient. The series of experiments have determined the governing parameters.

RÉSUMÉ: Durant ces dernières années, l'injection de coulis de ciment à haute pression nommée 'jetgrout' a été de plus en plus utilisée pour créer des barrières imperméables sous les excavations. Dans une couche de ce type d'injection placée juste au-dessous de l'excavation, de petites ouvertures, pouvant être présents, peuvent causer des transports d'eau et de sable difficiles à contrôler et qui ont déjà causer des problèmes majeurs dans plusieurs projets. Cette contribution décrit les résultats obtenus des expériences sur le mode de rupture et la stabilité du sol se trouvant dans les imperfections des couches d'injection. Ces expériences ont confirmé le mécanisme de rupture formulé par une solution analytique de stabilité de remplissage de sable dans une ouverture sous un gradient hydraulique élevé. Ces différents essais ont permis de déterminer les paramètres gouvernant ce mécanisme.

1 INTRODUCTION

For some time now, horizontal jetgrout screens consisting of short overlapping columns have been used to ensure the vertical stability of building pits and trenches. In recent years, there have been several, near-catastrophic events involving these jetgrout screens, which were intended to act as a water barrier under the building pit or trench. This scenario has arisen in projects in Cairo, Berlin, Bilbao and The Hague. In the latter case, it seems that the jetgrout arch was not fully watertight and this was unfortunately only detected during excavation, when groundwater and sand were flowing into the building pit. Nevertheless, the average permeability of this screen, determined with pumping tests, was satisfactory. This was one of the reasons for the Delft University of Technology to begin research into the design criteria and the reliability of a jetgrout screen as a groundwater barrier. The research focused on two aspects: the reliability of jetgrout screens (Tol, 2001), and the consequences of imperfections in jetgrout screens, the subject of this paper. Ballast Nedam Engineering sponsored the latter research.

According to the actual state of the art, all injection layers show imperfections. This means that the percolating water should be discharged in a controlled manner using a drainage system in the soil above the jetgrout layer, without the risk of erosion. A covering sand layer of a certain thickness is therefore necessary. In this research, an analytical model for the stability of the soil in a flow channel in the jetgrout screen was developed. In addition, analytical experiments (scale 1:5) were carried out to verify the model and to estimate the governing parameters. On the basis of this research, it is possible to determine the minimal thickness of the covering sand layer and the most important design criteria.

Earlier research followed also this approach (Bieberstein et al, 1999). In their study the governing failure mechanism starts with the complete reduction of effective stress at the top of the hole and considers the seepage pressure in the covering layer. In the present research a more critical failure mode for the hole was found.

2 ANALYTICAL SOLUTION

2.1 *Equilibrium of the soil in the hole*

It is assumed that an imperfection in the jetgrout screen is filled with sand. A difference in pore pressure is created across the grout layer when the construction pit is pumped empty. This leads to groundwater flow through the sand-filled hole. In a stationary situation, the seepage pressure acts as a uniformly distributed load over the height of the soil prop in the hole in the grout layer. In a hole with a constant cross-section and filled with uniform sand, a constant gradient across the hole occurs. It is also assumed that 100% of the differential pore pressure acts over the hole in the grout layer. This is a conservative estimate for the stress state in the hole. Calculations with groundwater models confirm the assumption as long as the hole is filled with sand and has a small diameter. Figure 1 shows the stresses acting on a soil element in the hole.

The following differential equation can be established from the vertical equilibrium of this element:

$$\frac{d\sigma_{z'}}{dz} - \frac{K.O.\tan\delta}{A}\cdot\sigma_{z'} = (\gamma_{sat} - \gamma_w) - i.\gamma_w \tag{1}$$

with:

γ_{sat} = density of saturated soil
γ_w = density of water
A = surface area of hole cross section

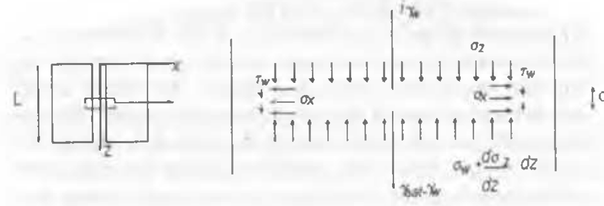


Figure 1. Equilibrium of a soil element in the hole

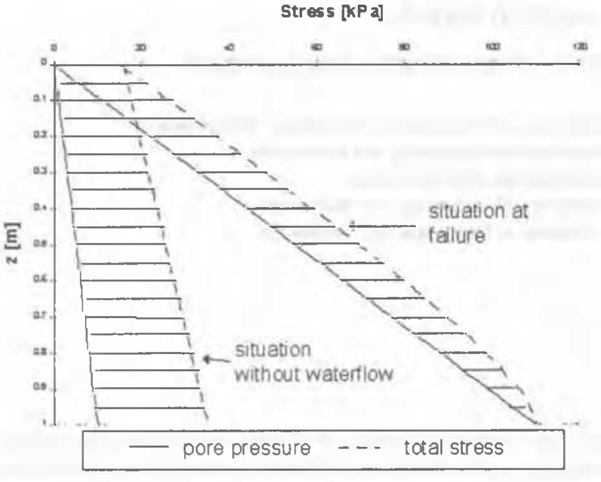


Figure 2. Stress state in a hole without seepage and at point of failure due to seepage.

O = circumference of hole cross section
K = horizontal earth pressure coefficient
σ_Z'= vertical effective stress
σ_Z = vertical total stress
δ = friction angle between wall and soil
i = gradient across hole

The solution of (1) can be determined using the boundary conditions for the vertical stress on the lower side of the hole.

$$z = L \quad \sigma_{z'} = \sigma_L$$

This gives the following description of the effective stresses over the height of the hole (see Sellmeijer, 1999):

$$\sigma_z' = \frac{[(\gamma_{sat} - \gamma_w) - i.\gamma_w]A}{K O.\tan\delta}\left(e^{\frac{KO\tan\delta}{A}(z-L)}\right) + \sigma_L'.e^{\frac{KO\tan\delta}{A}(z-L)} \tag{2}$$

Figure 2 shows the pore pressures, the total stresses and the effective stresses over the height of the hole. This figure shows both the stress states at the moment of failure as well as the initial stress state, without seepage.

2.2 *Evaluation*

Equation (2) describes the stress state in a hole in a jetgroutlayer. For the solution given, three aspects need to be considered.

1. The hydraulic gradient over the hole at the point of failure is far more than 1, so that the seepage pressure would normally lead to liquefaction. This does not occur because of the developed shear stresses acting at the wall of the hole. Moving upwards, the soil prop fixes itself. This phenomenon is similar to traditional arching in granular materials.
2. Increasing the hydraulic gradient over the hole reduces the effective pressure on the lower side of the hole to nil at a certain moment. Then there is just equilibrium in every cross-section over the height of the hole. Further reduction of effective stress on the lower side of the hole means that the lower section can no longer develop shear stresses. As the maximum shear stress has already developed across the remaining part of the soil column, the entire soil column will then be forced out of the hole.A condition for achieving the stress state described is that the upper side of the hole is stable. The covering layer must exert a certain minimum effective stress. If this is not the case, there is no reactive force to the flow pressures on the upper soil section. The flowing water then carries the particles away and the hole will be worn away from the upper side. The presence of a covering layer above the grout layer is therefore essential. The flow pressure in the covering layer will quickly fall due to the rapidly widening flow pattern.

3 EXPERIMENTAL RESEARCH

3.1 *Test set-up used*

Two types of tests were carried out, whose aims were: to test the developed analytical model on experimental data, and to determine the two remaining unknowns, namely:

- the size of the maximum shear stresses to be developed in the hole;
- the maximum effective pressures developed in the covering layer at failure.

In the first series of tests, only the hole in the grout layer was modelled. In the second series, both the hole as well as the covering layer were modelled. Separating the failure behaviour of the hole and the covering layer allows a controlled description of the stability of soil in the hole, without influence from the covering layer. The link is made, though, in the second tests series. There was no soil under the grout layer in either test series.

A cylindrical tube was used for modelling the hole in a jetgrout layer, measuring 100 mm in diameter, 550 mm in length, and made from Plexiglas lined with sand (see Figure 3). Pore pressure devices were fixed to the tube wall. Water can flow into the lower part of the tube at varying pressure. Excavation and pumping out of the construction pit in the field is modelled by slowly increasing the pore pressure differential across the tube.

The covering layer was not included in the first series of tests. The presence of effective pressure on the upper side of the hole is essential, as stated earlier. For this reason, a movable filter was fitted on the upper side of the sand column, permeable for water and impermeable for sand. Different loads could be placed on top of this filter. In the second series of tests, a larger cylinder was fitted above a similar tube, measuring 600 m in diameter, 400 mm in height, and in which pore pressure devices could be placed at the axis point. Both the tube (the hole in the grout layer) and the larger cylinder were filled with sand. Pore pressure was then increased over time, until failure occurred.

3.2 *Observed failure mechanism*

In the experiments carried out, a failure mechanism occurred which followed different phases over time. The different

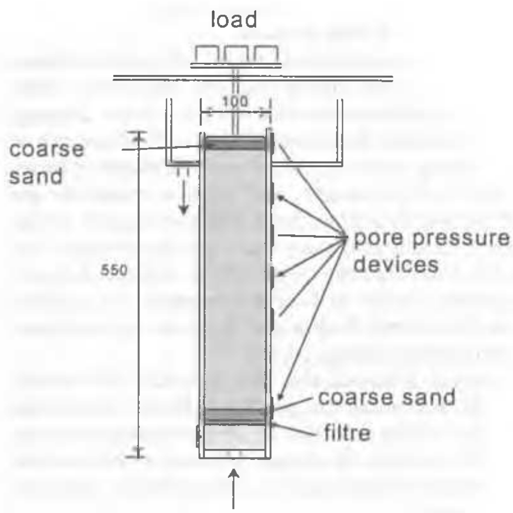


Figure 3. Test set up

Table 1 Overview of tested parameters

Parameters		lower limit	reference test	upper limit
top load	[kN]	1	3	6
Porosity	[-]	0.34	0.38 - 0.40	0.44
wall friction	[°]	12,5°	0.8 ϕ	ϕ
length	[m]	0.25	0.50	0.50
grain size sand		fine	Fine	Coarse

phases can be described as follows:

1. Crack formation

As pressure gradually increases, horizontal cracks form at the point where the pressure first builds up. This is often above the sand filter.

2. Fluidisation and compression

The section between the lower side of the set-up and the crack becomes fluid. The sand column (above the crack) becomes somewhat compressed from the lower side. This increases the density of the sand above the crack. The upper side of the sand column has not yet been affected.

3. First shift of upper column

When the pore pressure is increased at a certain moment, the entire sand column moves upwards. The maximum movement of the upper side amounts to a few millimetres. This shift mobilises the wall friction along the entire height; this results in sufficient resistance to prevent further column movement.

4. Stable situation as pressure increases further

Once friction has developed over the entire height, the sand column remains stable as the pore pressure increases. In this way, the load can be increased considerably.

5. Failure mode (1)

At a certain moment, the pore pressure is so large that the shear stress, which has developed no longer, offers sufficient resistance to a sand column shift. The column then slowly moves up. This movement continues until the entire sand column is pushed upwards. The water underneath the sand column is perfectly bright, no particles fall back against the flow.

6. Failure mode (2)

The entire stable sand column is forced out of the hole.

3.3 *Test series*

In the first test series, different sets of parameters were tested. Table 1 gives an overview of the range of the parameters that were varied. Most of the tests were carried out with sand glued at the inner wall of the Plexiglas tube. Some had 100% sand in these tests while others only 80% of the inner surface, leaving small vertical zones without sand so that the behaviour of the sand fill in the tube could be observed. The angle of wall roughness δ in these tests was supposed to be 0.8ϕ. A few tests were carried out with a tube without a sand liner, with a roughness $\delta = 12.5°$.

3.4 *Determining remaining unknowns*

As stated earlier, determining the two remaining unknowns (the maximum shear stresses in the hole and the effective stress to be mobilised in the covering layer at failure) was one of the reasons for performing the experiments. As equation (2) determines the vertical effective stress in the soil in the hole the unknown parameter is K.tanδ. Figure 4 gives the values of K.tanδ as a function of the porosity, found in the experiments. It appears that there are two branches, one for medium to loose sands with a K.tanδ value between 0.4 and 0.5 and one for dense sands with a K.tanδ value of 1.0 to 1.2. This corresponds to an active and passive condition in the hole at failure along a vertical wall.

The fact that the values of K.tanδ at the point of failure in those experiments with a 80% to 100% sand liner in the tube are similar for medium and loose sand can be explained by the compression of the sand column from underneath, before actual failure occurs. In figure 4 only the experiments without a sand liner in the Plexiglas show a considerable lower value for the friction.

The second unknown is the influence of the covering layer on the failure behaviour, or the maximum effective stresses that the covering layer can deliver. If this is modelled as a cone pushed upwards in this layer the tests indicate that the apex of this cone is about 8°. If the maximum stress is back-calculated from friction along a cylinder with a diameter D, based on effective vertical stress in he covering layer multiplied by K_r tanϕ it appears that the horizontal earth pressure coefficient $K_r = 0.27$.

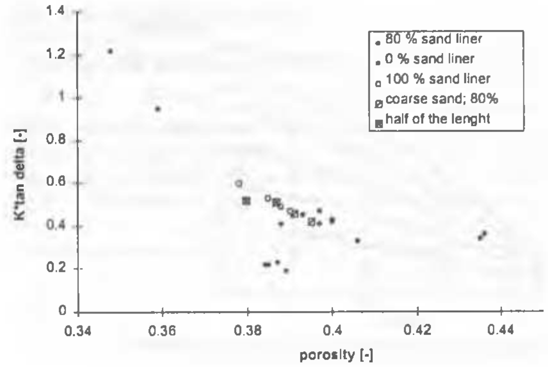


Figure 4. Results from the experiments, K.tanδ versus the porosity

4 DESIGN

4.1 *Design chart*

From (2) follows the required vertical stress $\sigma'_{z;s}$ at the top of the hole in a jetgrout layer:

$$\sigma'_{z;s} = \frac{D}{4K\tan\delta}(i\gamma_w + \gamma_w - \gamma_{sat})(1 - e^{-\frac{4K\tan\delta L}{D}}) \tag{3}$$

The available vertical stress $\sigma'_{z;r}$ at failure, in case a cone is pushed upwards in the covering sandlayer with thickness d is:

$$\sigma'_{z;r} = (\gamma_{sat} - \gamma_w)(1 + \frac{2}{D} d.K_r\tan\phi)d \tag{4}$$

In this equation the seepage pressure (in the covering layer) is neglected, which corresponds with the back-analyses of the experiments.

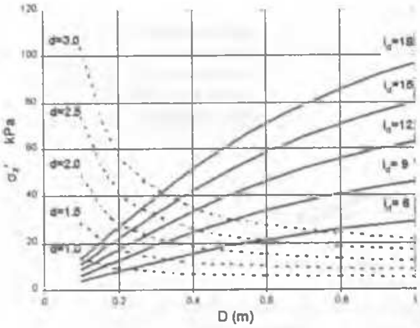
In figure 5 both equations 3 and 4 are depicted for an increasing hole diameter, different gradients and thickness' of the covering layer. Other parameters are fixed and the design values are giving in the figure. The overall factor of safety η =2.0 is applied to the height of the covering layer.

The required height of the covering layer can be determined, for the used set of parameters with the curves in figure 5. If for example a hole diameter of 0.2 m is chosen and a gradient $i_d = 12$ then $\sigma'_{z;s} = 17.4$ kPa and the required thickness of the covering layer d is 1.5 m (the continuous curve for i_d = 12 intersects the dotted curve for d=1.5 at D=0.2).

4.2 *Conclusions*

The following conclusions can be drawn:

- There is considerable resistance to failure when there is a sufficiently large, effective pressure on the upper side of a hole. This means that a gradient between 10 and 15 can be resisted.



parameters	γ
$K_d \, tg \, \delta_d$ = 0 32	1.20
L_d = 1.25	1 25
i_d = var.	1.2
$\gamma_{sat;\, d}$ = 18,2	1 1
$K_{r,\, d} \, tg \, \varphi_d$ = 0.125	1.5
η = 2	

Figure 5. Required cover as function of the hole diameter

- A long hole, or a thick grout layer, without ground coverage can only resist a hydraulic gradient of about 1 and is therefore far less favourable than a hole in a much thinner grout layer with some ground coverage, even if this is quite small.

Design calculations can in principle be performed using equation (2). Failure occurs when the effective stress at the bottom of the hole equals nil: ($\sigma'_Z = 0$ at $z = L$) giving equation (3) and the required effective stress at the upper side of the hole (σ'_Z at $z = 0$), which determines the required thickness of the covering sand layer. All parameters are known in this equation, except for the factor K.tanδ, which was determined in the experiments.

The available effective stress from the covering layer can be determined using equation (4). The unknown factor K_r was also determined in the experiments.

It should be noted that the presented design method is valid for homogeneous soil in the hole and the covering layer. Heterogeneous soil can have a negative influence on the stability. Especially a thin clay layer in the upper part of the hole requires a thicker covering layer.

4.3 *Design approach*

To design a jetgrout layer as a water barrier, it is important to realise that imperfections will always occur and that they can lead to a failure mechanism as described in this paper. A safe design can be achieved by using a sufficiently thick covering layer above the grout layer. A design approach for such a jetgrout layer may be as follows:

- using the probability model described by Tol (2001), the hole with the largest surface area is defined, with a required probability of failure such as 1 in 10,000;
- using the model for hole stability described in this paper the required thickness of the covering layer for a hole with this diameter is determined;
- although the seize of the hole is determined with a probabilistic model it is recommended to calculate the thickness of the covering layer with design values of the parameters and to apply an additional safety factor of about 2.0 to the thickness of the covering layer.

REFERENCES

Bieberstein, A., J. Herbst & J. Brauns, 1999. Hochliegende Dichtungssohlen bei Baugruben-umschliessungen, *Geotechniek nr. 2.*

Sellmeijer, J.B., 1999. Safe excavation level above jetgrout arch, *CO-390471/8,annex 1,* Geodelft

Tol, A.F. van, A.J.E. van Riel, & J.K. Vrijling 2001. Towards a reliable design for jetgrout layers. *Proc. of the XV International Conference on Soil Mechanics and Foundation Engineering,* Rotterdam: Balkema.