1-g model tests of tunnels with a surrounding cement-treated soil ring

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ABSTRACT

The construction of large diameter tunnels can be challenging, especially in soft soil conditions. Stability intervention by means of ground improvement has been identified as a possible method to allow for the safe construction of such tunnels. A 1-g physical model study of the stability of a tunnel with a cement-treated soil layer surrounding it, along with related failure loads and mechanisms, is reported here. The failure mechanism of such tunnels is found to be very different from conventional tunnels with no ground improvement as cement-treated soil is very brittle. Failure occurs locally, with cracks developing in the tensile region in the cement treated layer. These cracks consistently occur at the crown, springline and invert, over a range of improved soil layer thickness-to-tunnel diameter (t/D) ratio. These cracks are also distinctly flexural tensile in nature. Prior to the onset of cracking, the deformation of the tunnel is minimal. This in turn results in a negligible settlement trough. This implies that caution is needed on the use of ground movement as a measure of the strength mobilization, owing to the lack of warning signs. The stability of such tunnels is found to be related to the t/D ratio and strength of the improved soil layer, while the significance of the unimproved surrounding soil is found to be minimal. This leads to the formulation of a new stability equation.

Keywords: tunnel stability, ground improvement, cement-treated soils, 1-g model experiments

1. INTRODUCTION

One way to use underground space efficiently is to build tunnels of large diameter and at great depth. However this can be challenging if the site is in soft soil. The complexity of the problem also increases in urban areas, where surface disruption needs to be kept to a minimum.

Tunnel stability intervention by means of ground improvement can potentially facilitate safe construction of such tunnels. This can be done by installing a cement-treated soil zone surrounding the tunnel cavity using deep cement mixing or jet grouting methods. Such intervention is being used, for example, in the Marina South tunnels for Thomson Line, currently under construction in Singapore.

The stability of the tunnel is often quantified by a stability number. Broms and Bennermark (1967) carried out both extrusion and intrusion tests for soft clays, which motivated the formulation of an equation for the overload factor (also known as stability ratio) for tunnels. Kimura and Mair (1981) carried out centrifuge experiments of tunnels with varying unlined lengths and depths-to-cover, from which a stability chart was populated. Caporaletti et al. (2009) also examined the stability of tunnels in layered soil using centrifuge model tests.

Much remains unknown about the behaviour and failure mode of the cement-treated soil zone around a tunnel. Hence it is important that first the failure mode of such tunnels be identified, following which a suitable stability equation can be adopted or modified accordingly.

A series of 1-g model tests were proposed to investigate these failure modes. A small scale cement-treated soil ring will be installed in a clay bed. A surcharge load will be added to the surface till failure occurs.

2. 1-g MODEL TEST PROCEDURE AND MATERIALS

Figure 1 illustrates the various relevant strength and geometric parameters that can affect stability. The...
1-g model test setup is as shown in Figure 2.

**Figure 1 – Relevant strength and geometric parameters**

![Geometric parameters and physical parameters](image)

**Table 1. 1-g model test setup and procedure (after Sun, 2008)**

<table>
<thead>
<tr>
<th>Chemical composition</th>
<th>Values</th>
<th>Physical Properties</th>
<th>Values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Latite Saturation, Factor (L.S.F)</td>
<td>0.83</td>
<td>Consistency (%)</td>
<td>29.0</td>
</tr>
<tr>
<td>Magnesia, MgO (% wt)</td>
<td>2.25</td>
<td>Penetration (mm)</td>
<td>2</td>
</tr>
<tr>
<td>Sulphate, Asbestos in SO₃ (% wt)</td>
<td>2.06</td>
<td>Initial Setting Time (min)</td>
<td>100</td>
</tr>
<tr>
<td>Losses on ignition (% wt)</td>
<td>2.53</td>
<td>Final Setting Time (min)</td>
<td>210</td>
</tr>
<tr>
<td>Silica, SiO₂ (% wt)</td>
<td>20.46</td>
<td>Soundness (rpm)</td>
<td>&lt;1</td>
</tr>
<tr>
<td>Calcium Oxide, CaO (% wt)</td>
<td>63.15</td>
<td>Fineness (μm)</td>
<td>363</td>
</tr>
<tr>
<td>Iron Oxide, Fe₂O₃ (% wt)</td>
<td>3.61</td>
<td>Specific Gravity, G</td>
<td>3.15</td>
</tr>
<tr>
<td>Aluminous Oxide, Al₂O₃ (% wt)</td>
<td>8.31</td>
<td>2-days Strength (N/cm²)</td>
<td>25.7</td>
</tr>
<tr>
<td>Sodum Oxide, Na₂O (% wt)</td>
<td>0.25</td>
<td>7-days Strength (N/cm²)</td>
<td>55.5</td>
</tr>
<tr>
<td>Potassium Oxide, K₂O (% wt)</td>
<td>0.3</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The strength of the cement-treated layer is determined by its soil-cement-water mass ratio. Unconfined compression tests and Brazilian split cylinder tests were carried out on test samples and compared to Xiao (2009). These proved to be fairly similar (within 5%) of each other.

### 2.2 Experiment procedure

The first part of the experiment involves setting up the clay bed. Kaolin clay slurry was poured into a container measuring 700mm (l) by 300mm (b) by 600mm (h) container. A drainage layer was installed at the bottom of the container. The clay was then consolidated to a certain specified strength using a hydraulic actuator applying surcharge at the surface. A rubber bag filled with water on the surface ensures that the surcharge applied remains uniform.

The second part of the experiment is to cast the cement-treated soil layer. Singapore Marine Clay was mixed together with cement, with the mixing procedure consistent with Xiao (2009). After mixing, the cement-treated soils were casted in segmental polyformaldehyde moulds (See Figure 3). The outer mould (Figure 3) was designed to open outwards while the inner mould (Figure 3) was designed to be collapsible inwards; this being to facilitate removal of the cement-treated soil ring. These were light and easily dismantled, making it easy to transport and ensuring as little disturbance as possible during the de-moulding process. Some samples were set aside and cast in a smaller mould. Unconfined compression tests (UCT) were also conducted on these samples immediately after the conclusion of the experiment to determine the unconfined compressive strength of the cement-treated soil.

**Figure 2. 1-g model test setup**

**Figure 3. Collapsible polyformaldehyde mould**

After 4 days of curing a hole is cut from the front of the clay bed into which the cement-treated soil ring was to be installed. As Figure 4 shows, a circular cutter was used, with the cutting edge gently tapered on the inner diameter and flat on the outer diameter to reduce
disturbance of the surrounding soil. The over-cut is 1mm all round. This is needed so that the cement-treated ring along with the sensors could be installed. Once everything was in place the box was closed up and a surcharge was once again applied to the soil for 3 days to re-consolidate the soil. Dense sand is used to keep the cavity from closing.

During the 3-day re-consolidation period, excess pore water pressure was monitored using pore pressure transducer embedded in the kaolin clay layer. This confirmed that excess pore pressure arising from the swelling which occurred during the installation of the cement-treated soil ring, has indeed dissipated. Cone penetration tests were then conducted to measure the undrained shear strength of kaolin layer.

In the final part of the test, the dense sand was manually removed from the cavity and surcharge is increased incrementally on the surface till failure. A speed controller adjusts the ram speed of the hydraulic actuator. A load cell at the actuator, as well as stress transducers at the crown of the cement-treated soil layer recorded changes in pressure. Another transducer recorded the displacement of the actuator.

### 3. EXPERIMENT RESULTS

A total of 6 tests were carried out. Table 2 is a summary of all the variables of each test.

The first series of experiments aims to establish a baseline and ensure that the experimental procedure is repeatable. Subsequently the next series studies the effect that the t/D ratio has on stability. The effect of the undrained shear strength of the kaolin clay and the cement-treated soil, hereafter denoted by $c_u$ and $c_{ul}$, respectively, is examined in the final series.

The surcharge applied in each test was plotted against the displacement of the hydraulic actuator (see Figure 5). It can be observed for each test the actuator load will gradually increase to a certain maximum value after which its continued displacement does not result in significant increase in surcharge. The flattening of the load-displacement curve was observed to coincide with the collapse of the improved soil ring, as indicated by visual observations.

<table>
<thead>
<tr>
<th>Series</th>
<th>Identifier</th>
<th>Experimental parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Baseline and repeatability</td>
<td>1G-N20-301</td>
<td>$c_{ul} = 198kPa, c_u = 308kPa, C=12cm, t = 3cm, D = 15cm, t/D = 0.2</td>
</tr>
<tr>
<td></td>
<td>1G-N20-303</td>
<td>$c_{ul} = 198kPa, c_u = 308kPa, C=13cm, t = 3cm, D = 15cm, t/D = 0.2</td>
</tr>
<tr>
<td>2. Varying t/D</td>
<td>1G-N45-300</td>
<td>$c_{ul} = 208kPa, c_u = 508kPa, C=15cm, t = 1cm, D = 11cm, t/D = 0.45</td>
</tr>
<tr>
<td></td>
<td>1G-N45-335</td>
<td>$c_{ul} = 208kPa, c_u = 508kPa, C=15cm, t = 1cm, D = 11cm, t/D = 0.27</td>
</tr>
<tr>
<td>3. Effect of $c_{ul}$ and $c_u$</td>
<td>1G-N20-226</td>
<td>$c_{ul} = 68kPa, c_u = 308kPa, C=13cm, t = 3cm, D = 15cm, t/D = 0.2</td>
</tr>
<tr>
<td></td>
<td>1G-027-280</td>
<td>$c_{ul} = 68kPa, c_u = 308kPa, C=15cm, t = 1cm, D = 11cm, t/D = 0.27</td>
</tr>
</tbody>
</table>

As Figure 5 shows, the transition from a linearly increasing loading trend to the flattening of the load-displacement curve is quite abrupt. This suggests that failure occurs rapidly when the maximum surcharge value is reached, after which the system can no longer take any further increases in stress. This is consistent with the brittle nature of the cement-treated soil. This maximum surcharge value must therefore be the critical surcharge to cause failure. Table 3 is a summary of all these critical surcharge loads.

![Figure 4. Coring tool for circular openings (left) and its cutting edge (right)](image)

![Figure 5. Actuator load vs actuator displacement for all tests](image)
The cracks tend to be smaller at the crown and wider at the springline, as seen in Figure 6. A possible explanation is that at the point of failure, compressive stress concentrates along the outer diameter at the crown and the inner diameter at the springline. These form ‘pivots’ from which the cement-treated soil layer can rotate about. This process is illustrated in Figure 7.

Subsequently the cement-treated soil was fully exhumed in order to more closely examine the rupture planes. The rupture surfaces, shown in Figure 8, display sharp breaks with jagged edges. There were no evidence of either plastic flow such as necking or plastic hinging, or shearing and sliding such as grinding or powdering of the rupture surfaces. This suggests that the failure occurs due to brittle fracture.

### 3.1 Series 1 Analysis

The objective of the series 1 experiments is to establish a baseline for all the upcoming tests and to show that such results are repeatable. Hence the conditions of both tests in this series are kept as similar as possible.

As can be seen in Figure 5, the trend for both test 1G-N20-308 and 1G-N20-303_REP is consistent. This trend persists for all the other tests as well. Their critical surcharge values (177kPa and 180kPa) are also very similar. This suggests that these experiments are repeatable.

### 3.2 Series 2 Analysis

The objective of the series 2 experiments is to study the effect of the thickness-to-diameter (t/D) ratio on the overall stability of the tunnel. Two different t/D ratios, 0.27 and 0.45, were tested and compared with the series 1 experiments. The undrained shear strength of both the kaolin clay and cement-treated soils were kept as consistent as possible.

The critical surcharge for tests 1G-N27-326 and 1G-N45-300 were 233kPa and 356kPa respectively. This represents a 32% and 101% improvement over the baseline test 1G-N20-308. This indicates that the t/D ratio has a positive effect on the overall stability.

It should be noted that the two tests in this series achieved the critical surcharge value at a lower actuator displacement than the series 1 experiments, as seen in Figure 5. This ‘stiffer’ response suggests that the flexural stiffness of the improved soil ring rather than just the elastic modulus (Young’s modulus) of the cement-treated soil may factor towards the stability of the system.
3.3 Series 3 Analysis

The objective of the series 3 experiments is to study the effect of the undrained shear strength of the kaolin layer and cement-treated soil ring on stability. Test 1G-N20-226 has a lower undrained shear strength of the cement-treated soil ring of 226kPa. The kaolin bed in Test 1G-O27-280 has an OCR of 3, with a higher undrained shear strength of the kaolin, of 60kPa.

Comparing test 1G-N20-226 with the baseline test 1G-N20-308, it is found to have a lower critical surcharge at 145kPa (18% decrease). This suggests that a lower undrained shear strength (of the cement-treated soil ring) has a negative effect on stability.

Test 1G-O27-280 is compared with test 1G-N27-326. These tests have critical surcharge values of 262kPa and 233kPa respectively. This suggests as well that an increasing the undrained shear strength of the kaolin layer helps to enhance the overall stability.

3.4 Quantifying stability

For all of the tests failure has occurred locally; cracks formed in the cement-treated soil with no major deformation in the surrounding soil. This is as the cement-treated layer manages to largely retain its shape post-failure.

It can be seen from the tests that the factors that have a positive effect on the stability of the system are t/D, as well as the undrained shear strength $c_{u2}$. Considering the observed failure mode, that of bending at the crown and springline, it is logical that t/D would affect stability positively as flexural strength can be related to thickness. The $c_{u2}$ value is related to the tensile strength of this material, as shown in Figure 9. Tensile strength in turn would directly affect the flexural strength of a material.

The tests also indicate that the undrained shear strength of the surrounding material $c_{u1}$ also factors into the stability of the tunnel system, which may seem counter-intuitive as failure occurs locally in the cement-treated soil. It is reasoned that this is due to the horizontal deformation of the cement-treated soil at the springline of the tunnel. As can be seen in Figure 5, the tunnel crown moves into the tunnel cavity at failure, while at the springline the deformation is towards the surrounding soil, resulting in a “flatter” improved soil ring. Thus a larger $c_{u1}$ value would provide greater resistance to this lateral deformation, adding to the stability of the system.

Thus any reasonable stability ratio for these types of tunnels should include all of these factors mentioned earlier. The stability ratio as proposed by Broms and Bennermark (1967) is as follows:-

$$N = \frac{\gamma (C + D/2) + \sigma_s - \sigma_T}{c_{u2}}$$  \hspace{1cm} (1)

Where $\gamma$ is the unit weight of the soil. Equation (1) can then be modified to be used for tunnels with a cement-treated soil layer. The following is proposed for the new stability equation:-

$$N = \frac{\gamma (C + D/2) + \sigma_s - \sigma_T}{Xc_{u1} + (1 - X)c_{u2}}$$  \hspace{1cm} (2)

Where X is some factor relating the contribution of the surrounding soil to the stability of the system. X is a real number between 0 and 1.

The suggested value of X for this study is 0.37. For the 1-g experiments, the load from the weight of the soil is assumed to be insignificant compared the surcharge value $\sigma_s$, while the value of $\sigma_T$ is 0. N can then be plotted against t/D to form a stability chart, as shown in Figure 10.

![Figure 9. Relationship between tensile strength and unconfined compressive strength of cement-treated marine clay (after Xiao, 2009)](image)

![Figure 10. Stability chart with 1-g test data](image)
The stability number is shown in Figure 10 to have a linear relationship with t/D. The equation for this stability chart is displayed on the graph. The region below the line plot is the stable zone.

4. CONCLUSION AND FUTURE WORKS

In order to acceptably quantify the stability of a tunnel with a cement-treated soil layer surrounding it, first the correct failure mode must be established. From several of these 1-g model tests, it can be seen that failure occurs as a result of bending stresses across the crown and springline of the cement-treated soil.

From the 1-g model tests, parameters relevant to the stability of the system are identified. These are \(t/D\), \(c_{u1}\) and \(c_{u2}\).

A stability equation can then be formulated based on this failure mode. Using this equation and the data from a 1-g tests, a stability chart has been populated. It can be shown that the stability number \(N\) has a linear relationship with \(t/D\).

The stability chart is however sparsely populated. In the future more tests should be carried out to see if the identified failure mode is still relevant for other \(t/D\) ratios than the 3 tested in this study.

A limitation of the 1-g test is that it is unable to simulate the effect of a surrounding soil profile that increases with depth. This can be achieved by using a geotechnical centrifuge. Some tests should be done to see if this would affect stability.

5. ACKNOWLEDGEMENTS

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