Analysis of pre-reinforced zone in tunnel considering the time-dependent performance

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ABSTRACT: Auxiliary support systems such as the reinforced protective umbrella method are applied before tunnel excavation to increase ground stiffness and to prevent the large deformation in soft ground and shallow depth tunnelling. This study suggests a method to characterize the time-dependent behavior of pre-reinforced zones around the tunnel using elastic waves and direct shear test. The results obtained from the laboratory tests are applied to numerical simulations of a tunnel considering its construction sequences. According to numerical analyses, the time-dependent tunnel stability is most critical in the initial installation part of pre-reinforced zone and the portal of tunnel. However, time-dependent effect on tunnel behavior is not significant during construction as long as a proper overlap length is applied. Finally, the suggested analysis method of combining experimental and numerical procedures that consider the time-dependent effect on the pre-reinforced zone on the tunnel behavior will provide reliable and practical design and analysis for tunnels in soft ground.

1 INTRODUCTION

Recently, instances of the construction of large underground structures in soil and soft rock layers are increasing. Auxiliary support systems such as the reinforced protective umbrella method are applied before or during tunnel excavation to ensure the stability of the tunnel face, to increase ground stiffness and to prevent large deformations in soft ground during shallow depth tunnelling. These auxiliary supports change the state of stress and derive the arching effect around the tunnel. However, the effects of pre-reinforcement on tunnel stability and waterproofing of large section tunnels in soft ground have yet to be clearly and quantitatively defined.

In conventional tunneling, there are typically 1 to 2 days between one face and the next face after the pre-reinforcement step. During this time interval, it is known that changes of the material properties caused by the effect of curing of the grouting material exist. However, 28 days of stiffness and strength after construction are generally applied to the material properties of the pre-reinforced zone in design stage without considering the effect of the time-dependent behavior of the injected grout material.

The present study suggests a method to characterize the time-dependent behavior of pre-reinforced zones around a large section of tunnel in soft ground using elastic waves. An experimental analysis was performed to characterize the time-dependent behavior of the pre-reinforced zone. Direct shear tests were performed at different time stages to obtain the time-dependent strength parameters. In addition, elastic wave velocities (i.e., \( V_p \) and \( V_s \)) were continuously measured using piezoelectric bender elements to obtain the time-dependent stiffness parameter. Time-dependent strength and stiffness parameters obtained from laboratory tests were applied in the numerical simulation of a large section tunnel in soft ground, taking into account its construction sequence. The proposed analysis method, which combines experimental and numerical procedures while considering the time-dependent effect on the pre-reinforced zone on the tunnel behavior, will provide a reliable and practical design basis and means of analysis for large section tunnels in soft ground.

2 TIME-DEPENDENT EXPERIMENTAL ANALYSIS

2.1 Measurement of elastic wave velocity and shear strength

As shown in Figure 1(a), S-wave and P-wave velocity were measured in this study using bender elements. The propagation direction was varied by arranging the installation direction of the bender element such that it was parallel for the p-wave and perpendicular for the s-wave. Wave velocities were measured continuously...
Figure 1. Schematic drawing of test setup.

Table 1. Material properties for samples used in test.

<table>
<thead>
<tr>
<th>Material properties</th>
<th>Joomunjin Standard Sand</th>
<th>Portland Cement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Friction angle (°)</td>
<td>39</td>
<td>–</td>
</tr>
<tr>
<td>Min. void ratio</td>
<td>0.62</td>
<td>–</td>
</tr>
<tr>
<td>Max. void ratio</td>
<td>1.13</td>
<td>–</td>
</tr>
<tr>
<td>Density (kN/m²)</td>
<td>25.9</td>
<td>30.8</td>
</tr>
<tr>
<td>Relative density (%)</td>
<td>60</td>
<td>170</td>
</tr>
<tr>
<td>Vertical Stress (kPa)</td>
<td>160</td>
<td>10(W) × 10(D) × 7(H)</td>
</tr>
</tbody>
</table>

for 28 days under 160 kPa vertical stress. From the wave propagation transition, arrival time was calculated. The wave velocities were calculated by dividing the height of the specimen by the measured arrival time (Dano, 2004; Khan, 2006).

A schematic drawing of the motorized direct shear apparatus is shown in Figure 1(b). The shear speed of the direct shear apparatus was maintained at 0.5 mm/min. A load cell of 2 ton capacity was installed to calibrate the load during shearing. Measured vertical and horizontal displacements were saved in a computer automatically using a LVDT. All material properties of samples used in the tests are listed in Table 1.

2.2 Results and analyses of experimental tests

2.2.1 Time-dependent characteristics of wave velocities

The results show that the wave velocity increases exponentially according to curing time and becomes almost constant after 7 days. Figure 2(a) shows typical velocity data for two types of measured waves. It is found that the strength of the mixture becomes an important factor from about 6 hours. As shown in Figure 2(a), P-wave velocity is faster than S-wave velocity and represents the wave that travels through the skeleton of sand and cement mixture. From Figure 2(b), it is seen that the wave velocity starts to increase when the curing time is 4~6 hours. This means that the mixture takes 4~6 hours to reveal a cementation effect.

2.2.2 Time-dependent characteristics of shear strength and strength parameters (c and \( \phi \))

The direct shear tests performed for specimens less than 7 days of curing time correspond to the wave velocity measurement. The results of the direct shear test were very similar to those of the wave velocity measurements. However, the tests were performed with intervals of a day or more, and thus the process of the strength revelation through cementation could not be observed.

Figure 3 shows the time dependent characteristics of shear strength parameters obtained from direct shear test. In Figure 3(a), the friction angle does not change in accordance with curing time. On the other hand,
Figure 3. Time-dependent characteristics of shear strength parameters.

Figure 3(b) shows time-dependent characteristics of cohesion. Through Figure 3(b), it is noticeable that the cohesion increases with curing time and after a certain amount of curing time the cohesion converges. It can be deduced that the bonding of cement causes this increase in cohesion. Therefore, strength is controlled by the normal stress (i.e., frictional) at the early stages and then by cohesion (Schnaid et al., 2001). In the numerical analysis, it is assumed that the friction angle does not change and the cohesion increases as curing time increases.

Ultimately, the cohesion controls the increase of the shear strength, and cohesion depends on time in a similar manner as elastic wave velocity. Hence, Young’s modulus and cohesion are selected as improvement properties to reflect time-dependent characteristics in order to facilitate accurate simulation via the numerical analysis program in this study.

2.3 Verification of testing results and acquirement of time-dependent design properties

An empirical determination method of pre-reinforced zones to investigate the stability in a tunnel is presented and the results are compared with characteristics of elastic wave velocity from laboratory test results.

As shown in Eq. (1), cohesion is related to the unconfined strength and friction angle. Herein, unconfined strength of grout bulb at 7 days of curing time is about 8 MPa from the uniaxial compressive strength tests. When the friction angle of a soil layer in natural condition is about 39 degrees, the cohesion of the pre-reinforced zone is about 1.91 MPa according to Eq. (1).

\[ c_x = \frac{q_{u(desig)}}{2 \times \tan(45^\circ + \frac{\phi}{2})} \]  \hspace{1cm} (1)

Young’s modulus of the grout bulb can be presented as a function of the weight density and design criteria strength in the concrete design specification, as given by Eq. (2).

\[ E_x = W^{1.5} \times 4270 \times \sqrt{q_u} \]  \hspace{1cm} (2)

Here, when the weight density is 18 kN/m\(^2\) and the unconfined strength at a curing time of 7 days is 8 MPa, Young’s modulus is about 8300 MPa. The initial Young’s modulus (i.e., 45 Mpa) can be calculated from equation (3)–(6), this represents an increase of 184 times.

Regarding the characteristics of elastic wave propagation, the maximum shear modulus can be calculated by Eq. (3) from the measured shear wave velocity.

\[ (G_{\text{max}})_{\text{field or Lab.}} = \rho \times V_s^2 \]  \hspace{1cm} (3)

The normalized shear modulus in the range of the tunnel strain level can be obtained by a resonant column and torsional shear test and the Ramberg-Osgood model (1943), as shown in Figure 4. Typically, the strain level of a tunnel is about 0.1–1%. Hence, the normalized shear modulus of sandy soil is about 0.3, as shown in Figure 4. Accordingly, the shear modulus of the pre-reinforced zone can be calculated by the maximum shear modulus from Eq. (3) and the normalized shear modulus in Eq. (4). Furthermore, the Young’s modulus of model can be derived by Eq. (5)
and Eq. (6), which delineates the relationship among Young's modulus, shear modulus, and Poisson's ratio.

\[ G_{\text{model}} = (G_{\text{max}})_{\text{Field or Lab.}} \times (G / G_{\text{max}})_{at \gamma} \]  
\[ \nu = f(V_p, V_s) \]  
\[ E_{\text{model}} = 2 \cdot (G_{\text{model}})_{\text{Field or Lab.}} \cdot (1 + \nu) \]

Although it is clear that the measured shear wave velocity has characteristics of infinitesimal deformation, it is reasonable that the increase in Young's modulus can vary according to any increase in the elastic wave velocity depending on the time.

With these equations and theory, Young's modulus of sandy soil in natural condition is 45 MPa and Young's modulus for a curing time of 7 days is 7400 MPa. Therefore, Young's modulus of a pre-reinforced zone increases 165 times compared to that before pre-reinforcement. It is apparent from the equations (3) ~ (6) presented here that stiffness increases at a similar rate to the result calculated by Eq. (2). Moreover, the Young's modulus of pre-reinforced sandy soil after 28 days of curing time increases by about 200 times in comparison with normal condition sandy soil. Based on these results and equations, numerical analyses should be accompanied to evaluate the time-dependent characteristics of pre-reinforced zones in a tunnel.

Young's modulus and cohesion of a pre-reinforced zone should be expected to increase by 200 times after a curing time of 28 days.

3 NUMERICAL SIMULATION OF LARGE SECTION TUNNEL IN SOFT GROUND

3.1 Large section tunnel model and boundary conditions

Comparative analyses were performed to analyze the time-dependent effect of a pre-reinforced zone on the large section tunnel behavior while tunneling. The material properties of the pre-reinforced zone have five different conditions that are either time-dependent conditions or constant time conditions (i.e., 1d, 2d, 3d, and 28d). A commercial 3D FEM analysis program (i.e., MIDAS-GTS) was used as a numerical simulation tool. Elasto-plastic ground material, which is linear elastic and perfectly plastic with following Mohr-Coulomb yield criterion and non associated flow rule, was used for analyses.

For the application of the stiffness and strength parameters depending on the time obtained from the experimental study, a tunnel model should be located at the same stress level of the experiment condition, as propagation characteristics of elastic waves are affected by the state of the stress.

Figure 5 shows the simulated 3D four-lane tunnel model, which was constructed 15 m under the surface of the ground. It has a radius of 9.4 m and a height of 10.4 m. The boundary of 3D FE model is horizontally fixed at the 4 end sections and vertically fixed at the bottom section. Water table is located at the top of the surface. This simulated tunnel is an actual section of a four-lane tunnel that was designed and constructed in weathered rock in Korea. Four sub-sectional excavations with a length of 0.75 m per stage were modeled. Steel pipes 114.3 mm in diameter were modeled as a beam element: a 12 m steel pipe was installed at an inclination of 11° with a 6 m overlap, with a 0.7 m transversal interval between the steel pipes that were installed along the tunnel crown 60° from the center. The grout expansion radius is 0.3 m from the pipe center.

3.2 Numerical analysis of a tunnel considering the time-dependent behavior

The time-dependent behavior of a pre-reinforced zone can be modeled using the procedures described below. The material properties (i.e., stiffness and strength) of the pre-reinforced zone are registered as the boundary conditions from Day 1 to Day 28. The registered initial boundary conditions were applied to a pre-assigned mesh in the pre-reinforcement construction. The boundary conditions were applied and updated according to the field construction process. Figure 5 shows the conceptual time-dependent stiffness and strength characteristics of the pre-reinforced zone after the third pre-reinforcement and 12 m of excavation at an excavation speed of 0.75 m/day.

The tunnel behaviors were analyzed considering the time-dependent effect of the pre-reinforced zone in terms of the vertical displacement and the horizontal displacement. The material properties used for the analysis are tabulated in Table 2. Figure 6 shows the time-dependent elastic modulus and cohesion values obtained from the experimental study as well as those used with the numerical analysis.
Table 2. Material properties used for the numerical analysis.

<table>
<thead>
<tr>
<th>Material</th>
<th>$\gamma$ (kN/m$^2$)</th>
<th>$E$ (MPa)</th>
<th>$\nu$</th>
<th>$c$ (kPa)</th>
<th>$\phi$ (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weathered soil</td>
<td>18</td>
<td>45</td>
<td>0.32</td>
<td>0</td>
<td>39</td>
</tr>
<tr>
<td>Weathered rock</td>
<td>21</td>
<td>200</td>
<td>0.3</td>
<td>250</td>
<td>35</td>
</tr>
<tr>
<td>Steel Pipe</td>
<td>33</td>
<td>210000</td>
<td>0.3</td>
<td>–</td>
<td>–</td>
</tr>
</tbody>
</table>

where $\gamma$: unit weight, $E$: elastic modulus, $\nu$: Poisson’s ratio, $c$: cohesion, $\phi$: friction angle.

Figure 6. Time-dependent stiffness and strength for numerical analysis and obtained experimental result.

Figure 7. Normalized vertical displacement variation.

4 COMPARATIVE ANALYSIS OF THE NUMERICAL SIMULATION RESULT

For quantitative analysis, displacements of each case are normalized with the result of a pipe-only case. With the normalized displacements, the effect of the time-dependent behavior of the pre-reinforced zone was examined.

4.1 Time-dependent effect on vertical displacement

Figure 7 shows the normalized vertical displacement at the portal according to the time-dependent condition and constant time conditions. The normalized vertical displacement behavior of the time-dependent condition is similar to the results of one day of the constant time conditions at the initial excavation section. As the excavation length increases, the results of time-dependent condition become nearly identical to the analysis result of 2~3 days of constant time condition, and the vertical displacement converges. The stiffness and strength of the pre-reinforced zone of 1~2 days of the constant time condition correspond to 30~50% of 28 days of the constant time condition. In other words, a reduction of the material properties of the pre-reinforced zone makes it possible to model the time-dependent effect of the pre-reinforced zone on the global tunnel behavior at the initial tunnel excavation.

4.2 Time-dependent effect on horizontal displacement at the tunnel face

Figure 8 shows the normalized horizontal displacement at the tunnel face according to the time-dependent condition and constant time conditions. From the analysis result, variation of the normalized displacement at the time-dependent condition ranges from 0.94~0.98, which is very similar to other cases during excavation. Therefore, pre-reinforcement can be considered as prevention of a collapse rather than a displacement reduction control at the tunnel face. It can be concluded that grouting reduces the horizontal displacement by approximately 2~6% at the tunnel face with the pre-reinforcement method.

4.3 Time-dependent effect on horizontal displacement at the tunnel side wall

Figure 9 shows the horizontal displacement variation at the tunnel side wall according to the time-dependent condition and constant time conditions. From the analysis result, the normalized horizontal displacement variation of the time-dependent condition ranges from 0.94~0.98, which is very similar to other cases during excavation. Therefore, time-dependent analysis of a pre-reinforced zone for the horizontal
displacement of a tunnel, which is identical to the variation of the vertical displacement.

Figure 10 shows the normalized vertical and horizontal displacements 10.5 m from the portal. Figures 10(a) and 10(b) are in good agreement with the result of 28 day of the constant time condition as well as the result of the time-dependent condition of the normalized vertical and normalized horizontal displacement. When the excavation length is 10.5 m, the pre-reinforced zone initially has 14 days of stiffness and strength; secondly, the pre-reinforcement zone had 6 days of stiffness and strength.

The stiffness and strength levels of 6 and 14 days of the constant time condition correspond to 87.8% and 99.3%, respectively, of 28 days of the constant time condition. Therefore, the time-dependent characteristics of the pre-reinforced zone do not affect tunnel displacement due to the sufficiently overlapped pre-reinforced zone. However, the time-dependent characteristics of the pre-reinforced zone should be considered in the case of a portal and an initial support section of a weak layer, which can cause a large displacement and unsafe construction conditions. Therefore, it can be concluded that 2~3 days for the stiffness and strength of pre-reinforced zones is appropriate to model the time-dependent behavior. In addition, a properly overlapped section of pre-reinforced zone can be assumed as 3 or 7 days of constant stiffness and strength without a time-dependent effect due to sufficient exposure of the stiffness and strength during the excavation stages in conservative designs.

5 CONCLUSIONS

Based on an experimental study of a pre-reinforced zone, the time-dependent characteristics were analyzed in terms of the strength and stiffness. Results show that the shear strength and the elastic wave velocities increase as the time stage increases. Shear strength and strength parameters (i.e., the cohesion and friction angle) can be uniquely correlated to elastic wave velocities. The results obtained from laboratory tests were applied to a numerical simulation of a tunnel, taking into account its construction sequence. According to the results of the numerical simulation, vertical displacement and horizontal displacement results for fewer than 2~3 days of constant time boundary conditions are nearly identical to the analysis results of the time-dependent condition. Therefore, it can be concluded that 2~3 days for the stiffness and strength of pre-reinforced zones is appropriate to model the time-dependent behavior of a large section tunnel. Finally, the suggested analysis method combining experimental and numerical procedures that consider the time-dependent effect on the pre-reinforced zone on tunnel behavior will provide a reliable and practical design basis and means of analysis for tunnels in soft ground.

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