DISTURBANCE TO NEARBY SURFACE STRUCTURES CAUSED BY TUNNELING INDUCED GROUND DISPLACEMENTS

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ABSTRACT

Damage to nearby surface structures caused by excavation-induced ground displacements is a major concern during excavation of tunnels in congested urban areas. Hence, developing a reliable numerical model for predicting the possibility of damage to nearby structures is needed. This paper presents the details of a comprehensive 3D finite element model developed to study the induced structural distortions of adjacent structures due to tunneling activities. The proposed 3D model used the newly developed shotcrete material model in Plaxis to model the reinforced concrete elements of the tunnel. Utilizing the developed FE model the characteristics of the soil-structure interaction were studied and the criteria for the safety of the structures were accordingly proposed.

Keywords: Tunneling, Construction, Buildings, Soil-structure interactions, 3D model, Damage

1. INTRODUCTION

One of the main challenges facing designers during drilling tunnels in crowded urban areas is to ensure that the proximate buildings are satisfactorily unaffected by the excavation-induced ground settlement. Therefore, reliable models to predict the jeopardy of destruction of the building are indispensable. A small differential settlement can initiate the progress of cracking in the walls and facades of the surface buildings. To assess the range of probable damage, Skempton and MacDonald (1956), Bjerrum (1963), and Polich and Tokar (1957) proposed empirical methods limited to the damage caused by settlements rising from the weight of the structures. Burland and Wroth (1974) and Burland et al. (1977) proposed an analytical approach to assess the damage of a building. They modeled building facades as a linear elastic deep beam undergoing sagging and hogging modes of deformation. Boscardin and Cording (1989) later included lateral strain using simple superposition to consider the role of horizontal extension induced by adjacent excavation and tunneling. El Naggar and Steels (2012 & 2013), El Naggar and Elgendy (2012) considered similar interaction problems using two dimensional finite element models.

In this paper, a comprehensive procedure is developed to assess settlement-induced damage to buildings and the associated soil-structure interaction (SSI). This procedure is based on a finite element method in which the building, the ground and the tunnelling processes are combined in a single numerical model. The lining of the tunnel is modeled using the shotcrete constitutive model modified to simulate the behaviour of reinforced concrete, in which, more realistic stress distributions can be obtained, as the non-linearity of the material behaviour and the distinct different strength performance in compression and tension is taken into account.

2. SHOTCRETE MODEL (USDM: USER DEFINE SOIL MODEL)

The new shotcrete constitutive model in PLAXIS is a versatile user defined model that can be modified to reasonably simulate the behaviour of reinforced concrete structures. In its original format, it can account for the increase of the strength and stiffness with time, while model the accompanying decrease in ductility and creep. In this model, the Mohr-Coulomb approach has not been pursued further to model the failure surface due to the
underestimation of the material strength in compression, even though, it was able to realistically simulate the tensile strength. Accordingly, Mohr-Coulomb was excluded. Thus a more advanced failure criterion was utilized in the new shotcrete model that takes into account the non-linearity of the material behaviour. Besides, this user defined model has the capabilities to take into consideration strain hardening or softening in both compression and tension which is particularly relevant for modeling concrete material in general and shotcrete in particular. As well, the time dependent strength and stiffness could be entered as a function of time. Moreover, it considers the creep and shrinkage.

2.1 Model specific input parameters

Table 1: Input parameters of the shotcrete model

<table>
<thead>
<tr>
<th>No.</th>
<th>Parameter</th>
<th>Description</th>
<th>Unit</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>$E_{28}$</td>
<td>Young’s modulus of cured shotcrete at $t_{\text{hydr}}$</td>
<td>kN/m²</td>
<td>30,000,000</td>
</tr>
<tr>
<td>2</td>
<td>$\nu$</td>
<td>Poisson’s ratio</td>
<td>--</td>
<td>0.2</td>
</tr>
<tr>
<td>3</td>
<td>$f_{c,28}$</td>
<td>Uniaxial compressive strength of cured shotcrete at $t_{\text{hydr}}$</td>
<td>kN/m²</td>
<td>45,000</td>
</tr>
<tr>
<td>4</td>
<td>$f_{t,28}$</td>
<td>Uniaxial tensile strength of cured shotcrete at $t_{\text{hydr}}$</td>
<td>kN/m²</td>
<td>4,500</td>
</tr>
<tr>
<td>5</td>
<td>$\Psi$</td>
<td>Dilatancy angle</td>
<td>°</td>
<td>0</td>
</tr>
<tr>
<td>6</td>
<td>$E/E_{28}$</td>
<td>Time dependency of elastic stiffness</td>
<td>--</td>
<td>1</td>
</tr>
<tr>
<td>7</td>
<td>$f_{c,1}/f_{c,28}$</td>
<td>Time dependency of strength</td>
<td>--</td>
<td>1</td>
</tr>
<tr>
<td>8</td>
<td>$f_{\text{con}}$</td>
<td>Normalized initially mobilized strength</td>
<td>--</td>
<td>0.15</td>
</tr>
<tr>
<td>9</td>
<td>$f_{\text{cin}}$</td>
<td>Normalized failure strength (compression)</td>
<td>--</td>
<td>0.1</td>
</tr>
<tr>
<td>10</td>
<td>$f_{\text{con}}$</td>
<td>Normalized residual strength (compression)</td>
<td>--</td>
<td>0.1</td>
</tr>
<tr>
<td>11</td>
<td>$\varepsilon_{\text{cp}}$</td>
<td>Uniaxial plastic failure strain at 1h, 8h, 24h</td>
<td>--</td>
<td>-0.03/-0.001</td>
</tr>
<tr>
<td>13</td>
<td>$G_{c,28}$</td>
<td>Compressive fracture energy of cured shotcrete at $t_{\text{hydr}}$</td>
<td>kN/m</td>
<td>100</td>
</tr>
<tr>
<td>14</td>
<td>$G_{t,28}$</td>
<td>Tensile fracture energy of cured shotcrete at $t_{\text{hydr}}$</td>
<td>kN/m</td>
<td>6.9</td>
</tr>
<tr>
<td>15</td>
<td>$f_{\text{un}}$</td>
<td>Ratio of residual vs. peak tensile strength</td>
<td>--</td>
<td>0</td>
</tr>
<tr>
<td>16</td>
<td>$L_{\text{eq}}$</td>
<td>Equivalent length (if no regularization is used)</td>
<td>m</td>
<td>0</td>
</tr>
<tr>
<td>18</td>
<td>$a$</td>
<td>Increase of $\varepsilon_{\text{cp}}$ with increase of $p'$</td>
<td>m</td>
<td>18</td>
</tr>
<tr>
<td>19</td>
<td>$\Phi_{\text{max}}$</td>
<td>Maximum friction angle</td>
<td>°</td>
<td>37</td>
</tr>
<tr>
<td>20</td>
<td>$\Phi_{\text{cr}}$</td>
<td>Ratio between creep and elastic strains</td>
<td>--</td>
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</tr>
<tr>
<td>21</td>
<td>$t_{50}$</td>
<td>Time for 50% of creep strains</td>
<td>days</td>
<td>1.5</td>
</tr>
<tr>
<td>22</td>
<td>$\varepsilon_{\text{shr}}$</td>
<td>Final shrinkage strain</td>
<td>--</td>
<td>-0.0005</td>
</tr>
<tr>
<td>23</td>
<td>$t_{50}$</td>
<td>Time for 50% of shrinkage strains</td>
<td>days</td>
<td>28</td>
</tr>
<tr>
<td>24</td>
<td>$\gamma_{\text{fc}}$</td>
<td>Safety factor for compressive strength</td>
<td>--</td>
<td>1</td>
</tr>
<tr>
<td>25</td>
<td>$\gamma_{\text{ft}}$</td>
<td>Safety factor for tensile strength</td>
<td>--</td>
<td>1</td>
</tr>
<tr>
<td>26</td>
<td>$t_{\text{hydr}}$</td>
<td>Time for full hydration (usually 28 days)</td>
<td>days</td>
<td>28</td>
</tr>
</tbody>
</table>
2.2 Model structure

The total strain is calculated according to hardening/softening elastoplasticity:

\[
\varepsilon_{\text{TOTAL STRAIN}} = \varepsilon_{\text{ELASTIC STRAIN}} + \varepsilon_{\text{PLASTIC STRAIN}} + \varepsilon_{\text{CREEP STRAIN}} + \varepsilon_{\text{SHRINKAGE STRAIN}}
\]

2.3 Yield surfaces

Figure 1: Yield surfaces and failure envelope

2.4 Strain hardening and softening

2.4.1 Compression

Schütz et al. (2011) proposed an approach for the behaviour of the model in compression. The stress-strain curve is divided into 4 parts (see Figure 2).

Due to the time dependency of the material:

\[
H_c = \left( \text{Normalised Hardening Parameter} \right) = \frac{\varepsilon_p}{\varepsilon_\text{cp}} = \frac{\text{Minor principal plastic strain (calculated from } F_p)}{\text{Plastic peak strain in uniaxial compression}}
\]

Figure 2: Normalized stress - strain curve in compression

2.4.2 Tension

In tension, the behaviour of the model is linear elastic until the tensile strength, \( f_t \), is reached after which the strength will be governed by the residual strength as shown in Figure 3.
Figure 3: Tension softening

\[ H_t = \left( \frac{\text{Normalised Tension Softening Parameter}}{E_{\text{uniaxial strain ultimate Plastic}}} \right) = \frac{E_1^p}{E_{tu}^p} \]

2.5 Time dependent material parameters

2.5.1 Elastic stiffness

The increase of Young’s modulus \( E \) follows the CEB-FIP model code (1990)

\[ E(t) = E_{28} e^{s_{\text{eff}}} \left( 1 - \frac{t_{\text{hydr}}}{t} \right) \]

\( E \) is constant for \( t < 1 \text{h} \) and for \( t > t_{\text{hydr}} \).

Where,
\( E_{28} \) = Young’s modulus of cured shotcrete;
\( t_{\text{hydr}} \) = time until full curing;
\( t = \) time in days;

\[ s_{\text{eff}} = \frac{\ln \left( \frac{E}{E_{28}} \right)}{\sqrt{t_{\text{hydr}}/1d - 1}} \]

2.5.2 Compressive and tensile strength

\[ f_c(t) = f_{c,28} e^{s_{\text{strength}}} \left( 1 - \frac{t_{\text{hydr}}}{t} \right) \]

\[ s_{\text{strength}} = -\frac{\ln \left( \frac{f_{c,1}}{f_{c,28}} \right)}{\sqrt{t_{\text{hydr}}/1d - 1}} \]

The ratio of \( f_c / f_t \) and the values of \( f_{\text{c,uniaxial}} \), \( f_{\text{t,uniaxial}} \) are assumed to be constant in curing.

For the time between 24 h to \( t_{\text{hydr}} \), Oluokun et al. (1991) suggested a method:

\[ f(c) = f_{c,28} \left( 1 - \frac{t_{\text{hydr}}}{t_{\text{hydr}}/1d - 1} \right) \]

\( t \) in days
3. PROBLEM DEFINITION

This section presents the development of the FE models that were used to carry out the numerical analyses presented in this paper. The considered problem involves a high rise building (15 stories with 1 basement) laying on a mat foundation resting on a thick sandy layer underlain by bedrock at a great depth (Figure 4). The 3D FE models were established using the computer program PLAXIS 3D AE.01 (PLAXIS bv, 2015). Sensitivity analysis was conducted and appropriate size mesh was reutilized accordingly. The FE models were employed to perform a comprehensive parametric study to investigate the interaction between the building and the tunnel, and evaluate the forces within the different structural members. This paper presents the results of the first stage of the undergoing research.

Figure 4: Plan view and section

4. THREE-DIMENSIONAL FE ANALYSES

4.1 Geometry

The considered FE model is 180 m wide that extends 162 m in the y direction and it is 60 m deep. These dimensions are sufficient to allow for any possible collapse mechanism to develop and to avoid any influence from the model boundaries. Figure 4 shows the geometry of the considered problem. The tunnels were assumed to be buried at different depths ranging from 1D to 2.5D in the vertical direction, where D is the tunnel’s diameter, and from 0B (under the centerline of the mat foundation) to 1B in the horizontal direction where B is the width of the mat foundation.

4.2 Soil Stratigraphy and Used Material Model

The Soil layer is assumed to be horizontal throughout the model. The ground water table is located well below the foundation level so there is no influence of the water table on the ultimate bearing capacity of the foundation. The analyses were conducted assuming drained conditions. The unit weight \( \gamma_{\text{mat}} = 20 \text{ kN/m}^3 \). The Mohr-Coulomb model is selected as the material model. The Mohr-Coulomb model involves only five basic parameters: (1) Young’s modulus, \( E = 40,000 \text{ kPa} \), (2) Poisson’s ratio, \( \nu = 0.3 \), (3) Cohesion, \( C = 0.2 \text{ kPa} \), (4) Friction angle, \( \varphi = 38^\circ \), and (5) Dilatancy angle, \( \psi = 0^\circ \).
4.3 Mat Foundation

The 20x18 m mat foundation considered in this analysis is located at the centre of the sand deposit. It consists of a 0.75 m thick concrete of unit weight, $\gamma = 24$ kN/m$^3$. The foundation was modeled using plate elements from the PLAXIS library with a linear isotropic behaviour. The Young’s modulus, $E_1 = 30,000,000$ kPa, and the Poisson’s ratio, $\nu_{12} = 0.15$.

4.4 Tunnel Lining

The tunnel lining was assumed to obey the USDM model defined above in the second paragraph. According to the considered cases the tunnel was modeled for lining thicknesses equal to 0.05D (i.e., 400 mm in this case). For all cases, the tunnel lining was modeled as shotcrete with unit weight, $\gamma = 24$ KN/m$^3$. See table 1 for the other parameters.

4.6 The Used FE Mesh and its Boundary Conditions

The model was built using about 250,000 3D 10-node tetrahedral elements. Figure 5 shows the generated mesh of the model.

5. RESULTS

A parametric study was conducted to examine the effect of the burial depth on the performance of the mat foundation. Also, the influence of the existing structure on the developed moments and thrusts in the tunnel lining was studied.

In the parametric study, the tunnel was assumed to be located just below the centerline of the foundation. In addition, burial depths ranging between 1D to 2.5D were investigated in the vertical direction and from 0B to 1B in the horizontal direction.

5.1 Effect of the Existing Structure on the Developed Moments and Thrusts in the Tunnel Lining

Figure 6 shows the percent increase (% increase from the base case, i.e., just the tunnel without any surface structure) of the thrust and the moment in the tunnel lining at the springline and the crown locations when the tunnel is located at the centerline of the foundation at different burial depths varies between 1D to 2.5D. whereas, Figure 7 shows the percent increase of the thrust and the moment at the springline and the crown locations when the tunnel is located at a constant depth of 1D and the horizontal distance between the center of the tunnel and center of the mat foundation varies between 0B to 1B in the horizontal direction.

Figure 5: Generated mesh
It can be noticed from the Figure 6 that the thrust and the moment increase considerably, especially when the tunnel is located under the centerline of the building close to the surface (almost by 120% for the thrust at the springline and 30% at the crown (See Figure 6(a)). When the tunnel is located at a depth of 1D-1B or more below the foundation (See figure 7), the increase in the thrust and moment drops (the tunnel is located away from the intense of the influence zone of the building).

![Figure 6: (a) thrust and (b) moment at the springline and crown of the tunnel lining when the center of the tunnel is located at the centerline of the mat foundation at a burial depth varied from 1D to 2.5D](image)

![Figure 7: (a) Thrust and (b) moment at the springline and crown of the tunnel lining when the center of the tunnel is located at a horizontal distance varied from 0B to 1B from the mat foundation](image)

5.2 Effect of the Existing Structure on the Displacements of the Tunnel Lining

Figures 8(a) and 8(b) show the vertical and horizontal deformation of the tunnel at the crown and springline locations for a tunnels located at the centerline of the foundation at a burial depth varies between 1D to 2.5D (Figure 8(a)) and at tunnels buried at a constant depth of 1D and the horizontal distance varies from 0B to 1B, respectively. It can be seen from Figure 8(a) that the vertical deformation increases as the burial depth of the tunnel increases. Similar trends in behaviour can also be observed for the horizontal deformation at the springline. It can be noticed that the maximum deformation of the lining is less than the tolerable deformation (less than 1% of the diameter). It remains on the safe side. The behaviour in Figure 8(b) is expected. As the tunnel is located away from the foundation, @ 1B the stress concentration from the foundation does not interfere with the tunneling zone and consequently, less deformation occur. It can be noticed from Figure 8(b), at 0.5B (part of the tunnel under the mat foundation) the drop in the displacement values due to the differential settlement.
Figure 8: Displacement of the springline and the crown of the tunnel lining when the center of the tunnel is located (a) at the centerline of the mat foundation at a burial depth varied from 1D to 2.5D and (b) at a burial depth 1D from the mat foundation and the horizontal distance varied from 0B to 1B

5.3 Effect of the Tunnel on the Pressure below the Existing Mat Foundation

It can be seen from Figure 9 which represents the distribution of the vertical stress underneath the mat foundation that the pressures in general are not uniform. The maximum applied pressure in all cases is less than the calculated allowable bearing capacity of the soil.

Figure 9: Pressure under the centerline of the mat foundation when the center of the tunnel is located (a) at the centerline of the mat foundation at a burial depth varied from 1D to 2.5D and (b) at a burial depth 1D from the mat foundation and the horizontal distance varied from 0B to 1B

5.4 Effect of the Tunnel on the Settlement of the Existing Mat Foundation

Figure 10 shows the deformed shape of the centerline of the mat foundation. The maximum value at the center of the mat is less than the tolerable settlement. It noticed from the figure 10(b) when the tunnel is located at 0.5B (part of the tunnel is under the mat foundation) the differential settlement occurred in the mat.
6. CONCLUSIONS

This study was conducted to examine the effect of the pre-existing high rise buildings on the forces and deformations developed in the tunnel lining and vice-versa. The conclusions are summarized as following:

1. When the tunnel is located within the overstressed zone (the tunnel is embedded within 2D and 0.5B) the deformations, bending moment and thrusts increase substantially due to the interaction between the foundation and the tunnel.
2. The horizontal deformation of the lining at the spring line and the vertical deformation at the crown, decrease as the burial depth of the tunnel increases.
3. All of the above effect reduces substantially or vanishes when the tunnel is embedded at a depth of 3D or more below the foundation.
4. Presence of the tunnel did not really affect the vertical pressure underneath the foundation and its deformed shape due to the high stiffness of the tunnel lining which acted as a support below the foundation.
5. The current engineering approach to model shotcrete linings in numerical simulations assumes a linear elastic material with a stepwise increase of the Young’s modulus in subsequent excavation stages. While realistic lining deformations may be obtained with this method, lining stresses are usually too high, in particular if the lining is subjected to significant bending.
6. With the new constitutive model more realistic stress distributions can be obtained, as the non-linearity of the material behaviour is taken into account. Furthermore, the stability of the tunnel can be checked at all intermediate stages without the need for additional capacity checks of the lining cross section.

REFERENCES


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