Combination of undrained and drained analysis to design the temporary supports of a large diameter tunnel

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Abstract: The purpose of this paper is to present the analysis and design methodology used for soil-support interaction of a large diameter tunnel in a saturated soil mass. PLAXIS software was used to evaluate the required support for both short and long term. Based on the characteristic curve of the unsupported tunnel, considering installation of the supports before the tunnel face, 50% stress release was considered before support installation. At this stage of analysis, undrained behaviour was considered through analysis, which resulted in the reduction in the pore water pressure around the tunnel due to tunnel converging. Reduction of pore water pressure in the soil mass was balanced with the parameters of the reinforced concrete required for the stability of the tunnel in a short time. The analysis was followed by a fully coupled analysis with drained soil parameters to control and design the support for the long term condition. Based on this analysis, a cost effective solution was made for the temporary support of the tunnel to bear the overburden pressure in both short and long term. Temporary support involves the installation of a 5-segment lattice girder per 75 cm of the tunnel plus 30 cm thickness shotcrete and wire mesh.

Keywords: temporary supports, drained analysis, clay soil, pore water pressure, characteristic curve.

1. INTRODUCTION

Garmsiri water transfer system situated in western Iran is aimed, through constructing a number of dams and tunnels and culverts, to provide 750 million cubic meters of water for agricultural uses in downstream areas. A part of this aqueduct system is T2 tunnel with 3090 m length, in which 850 m of the tunnel is being built within the depth of 25 to 40 meters all through alluvial sediments. The initial water level in the alluvial deposit is above the tunnel face, which means that the surrounding soil mass is almost saturated. The majority of the alluvium deposit comprises cohesive material such as clay (CL) and gravel with fine material (GC). With this situation and due to the low permeability of the soil mass, the stability condition of the tunnel in short term is different from long term because of the undrained behaviour of the soil mass in short term and drained behaviour in long term. This fact has a great impact on the design of temporary supports. If the analysis is done by applying hydrostatic water pressure from the first step of the calculation, an overestimated primary supports will result. Negative excess pore water pressure in the soil around the tunnel (due to excavation) in the short time results in the design of a safe and economic primary supports. This is in parallel to the need of the short term uniaxial compressive strength of the shotcrete in the primary supports to bear the soil pressure right after support installation.

In this paper combination of undrained and drained coupled analysis of soil mass–tunnel supports interaction is discussed in designing the primary supports required for T2 tunnel in Garmsiri aqueduct system. First, the geotechnical profile of the tunnel is discussed and then the geometry and constitutive model used in the analysis are explained. The steps of the analysis in the short and long terms are included considering the role of excess pore water pressure around the tunnel face.

2. GEOTECHNICAL PROFILE

Majority of the T2 tunnel is located on Amiran formation (1) with consecutive layers of Shale and Marn. The remaining part (850 m) of the tunnel is in alluvial deposit. The geotechnical profile along the tunnel is shown in Fig. 1. Different boreholes were drilled up to 20 m below the tunnel base to investigate the geotechnical specifications of the soil and rock mass. In addition to the stratigraphy of the soil, index
properties and triaxial shear strength parameters were determined from disturbed and undisturbed samples. Standard Penetration Test (SPT) was also performed during drilling to evaluate the in-situ stiffness and strength of the soil strata.

In Fig. 1 the overburden pressure on the tunnel increases from 22 m to 39 m from borehole T2-BH2 to T2-BH3. Also water level increases from the left side to the right side with the fact that the tunnel crest is almost below ground water level. The ground water level in this figure is a maximum level determined across one year monitoring the bore holes.

Soil mass in alluvial deposit includes clay (CL), sandy clay (SC) and gravelly clay (GC). In general material with the description of SC and GC are in thin layers inside the clay soil close to the bedrock. The average plasticity index (PI) and liquid limit (LL) of the clay soil are 20% and 40%, respectively. The average value of N60 (Standard penetration modified by energy) is 42 with a standard deviation of 18 (the lowest and highest SPT value are 13 and 75, respectively). Based on the study by Stroud (2), the elasticity of the soil mass lies in the range of 22 to 38 MPa. Results of drained triaxial tests on undisturbed clay samples showed that cohesion, friction angle and elasticity of the soil mass are 0.25 kg/cm², 26° and 300 kg/cm², respectively. Results of in-situ permeability test showed a permeability value in the range of $10^{-6}$ to $10^{-5}$ cm/s, which is very low and is an indication of undrained behaviour of the soil mass during excavation.

3. GEOMETRY AND FINITE ELEMENT MESH OF THE SELECTED SECTION

For the design of the support system, three different sections were chosen with tunnel base to ground distance of 22, 29 and 39 m. In this paper, the results of the analysis for the tunnel with maximum overburden pressure (39 m) is presented. In order to analyse the soil-support interaction, PLAXIS 2D (3) with six-noded triangular elements was used. Fig. 2 shows the selected cross section with finite element mesh, in which ground water level is almost three times the tunnel height.
4. CONSTITUTIVE MODELS AND PROPERTIES

In this study, the non-linear elastoplastic Mohr-Coulomb model was applied for both alluvial deposit and the bedrock. The model has non-associated flow rule in shear and associated flow rule in tension. Model parameters for different geotechnical materials are shown in Table 1.

<table>
<thead>
<tr>
<th>Material</th>
<th>$\gamma_d$ kN/m$^3$</th>
<th>$\gamma_{sat}$ kN/m$^3$</th>
<th>$C$ kPa</th>
<th>$\Phi$ ($^\circ$)</th>
<th>$E$ MPa</th>
<th>$\nu$</th>
<th>$k$ cm/s</th>
</tr>
</thead>
<tbody>
<tr>
<td>Alluvial deposit</td>
<td>18</td>
<td>21</td>
<td>25</td>
<td>26</td>
<td>30</td>
<td>0.3</td>
<td>$10^{-6}$</td>
</tr>
<tr>
<td>Bedrock</td>
<td>-</td>
<td>24.6</td>
<td>50</td>
<td>41</td>
<td>800</td>
<td>0.3</td>
<td>$10^{-5}$</td>
</tr>
</tbody>
</table>

where $\gamma_d$ = dry density, $\gamma_{sat}$ = saturated density, $C$= Cohesion, $\Phi$ = friction angle, $E$= modulus of elasticity, $\nu$ = Poisson’s ratio and $k$ = permeability.

Due to the use of shotcrete and steel set (lattice girder) for primary supports, an elastic model with plate element was used to model the support system in the software. The model parameters are “Normal Stiffness” and “Flexural Rigidity” for the equivalent section with a width of $b$, an equivalent section thickness $t_{eq}$ and the equivalent modulus $E_{eq}$.

Normal Stiffness = $D_{eq} = E_{eq}A_{eq} = E_{eq}b_t_{eq}$

Flexural Rigidity = $K_{eq} = E_{eq}I_{eq} = E_{eq}\frac{b_t_{eq}^3}{12}$

where $E_{eq}$, $A_{eq}$ and $I_{eq}$ are elasticity modulus, cross section and moment of inertia of the equivalent section of the steel set and shotcrete. To calculate normal stiffness and flexural rigidity of the equivalent section, the method presented by Hoek, Carranza-Torres (4) was used, which generally incorporates elastic parameters and geometry of both shotcrete and steel sets. To design the support system a trial and error method was applied to reach a final required support. The final support includes steel sets spaced at 75 cm centres and embedded in a 30 cm thick shotcrete lining. Each one of the steel sets has 5 segments: S1 segments for the roof, S2 segments for the side walls and S3 segment for the invert (Fig. 3). For S1 and S2, 4 bar lattice girders ($4\Phi25$, 25 cm centre-centre) and for S3, IPB20 was designed. Properties of the shotcrete and steel segments are presented in Tables 2 and 3.

Fig. 3. Tunnel lining for temporary supports

The normal stiffness and flexural rigidity of the primary support system are shown in Table 4. These parameters are shown for both short and long term due to the need of the compressive strength for concrete for both conditions.
Steel set properties

<table>
<thead>
<tr>
<th>Segment</th>
<th>E (GPa)</th>
<th>( \nu )</th>
<th>( f_y (MPa) )</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1=S2 (4Φ25)</td>
<td>200</td>
<td>0.25</td>
<td>400</td>
</tr>
<tr>
<td>S3 (IPB20)</td>
<td>200</td>
<td>0.25</td>
<td>240</td>
</tr>
</tbody>
</table>

Shotcrete Properties

<table>
<thead>
<tr>
<th>Property</th>
<th>( f'_{c} (MPa) )</th>
<th>E (GPa)</th>
<th>( \nu )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Short term</td>
<td>12</td>
<td>16.5</td>
<td>0.15</td>
</tr>
<tr>
<td>Long term</td>
<td>30</td>
<td>26</td>
<td>0.15</td>
</tr>
</tbody>
</table>

Parameters for equivalent segments

<table>
<thead>
<tr>
<th>Segment</th>
<th>Normal Stiffness (GN/m)</th>
<th>Flexural Rigidity (MNm²/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Short term</td>
<td>Long term</td>
</tr>
<tr>
<td>Roof (S1) and Side Wall (S2)</td>
<td>5.4</td>
<td>8.8</td>
</tr>
<tr>
<td>Invert (S3)</td>
<td>6.85</td>
<td>10.25</td>
</tr>
</tbody>
</table>

5. STAGES OF THE ANALYSIS

In order to analyze the soil-support interaction and design the tunnel support, the following steps were taken:
1. Tunnel excavation and stress reduction
2. Support installation and undrained analysis with short term parameters for supports
3. Drained analysis with long term parameters

At the first and second steps, due to high compressibility and low permeability of the surrounding soil, the material behaves in undrained condition. In these steps, soil expansion due to tunnel excavation results in negative excess pore pressure in the soil mass and reduction of the pressure on the support system. At the third step, due to the gradual increase in the pore water pressure in a long time, the total pressure on the supports increases while soil material is in drained condition.

5.1. Stress reduction before support installation

In this paper, the 2D analyses were carried out using the stress reduction method (the \( \beta \)-method or \( \lambda \)-method) to define the amount of stress relaxation of the ground before installation of the lining. Ground deformation due to tunnelling is of paramount importance at the time of support installation. There is always a delay before support installation. The necessity for the proper design is to install the support in a moment that uses most of the soil capacity and reduces the pressure on the tunnel face while tunnel face is stable. This concept is usually applied to design an efficient and economic support to stabilise the surrounding mass. To define the stress reduction parameter, the concept of the ground response curve and longitudinal displacement profile is used. This is similar to the analytical convergence-confinement calculations presented by Carranza-Torres and Fairhurst (5) with a difference that the ground response curve (convergence-confinement curve) is determined from the analysis by PLAXIS software without any support. This curve is depicted in Fig. 4.a for the tunnel face in the short term. In this curve, the horizontal axis shows the radial displacement of the tunnel face with a maximum 26 cm at the top of the tunnel face. The vertical axis is the ratio of the current stress in the tunnel face (P) to the initial stress (Po) without excavation. P/Po is related to “stress reduction factor” (\( \beta \)) or confinement loss (\( \lambda \)) as follow:

\[
\beta = \lambda = 1 - P/Po
\] (3)
Due to the lack of information on the longitudinal displacement profile in 2D analysis, relationships presented by Chern, Shiao (6) and Panet (7) were determined based on tunnel radius of 3.55m and radial displacement of 26 cm (Fig. 4.b).

If the support system is installed at the distance of 1 m from the tunnel face, the displacement of the tunnel before support installation from Panet and Chern equations are 15 and 10 cm, respectively. For the purpose of this design, Chern equation is used due to the fact that Panet equation is from elastic solution, while Chern equation is based on the measured plastic data. Considering 7.5 cm displacement in the face of the tunnel and short-term ground response curve the stress reduction factor was chosen as 50%.

6. RESULTS OF THE ANALYSES

In this section results of the analyses in different steps in short and long terms are discussed. Fig. 5 shows the vertical displacement of the soil mass after support installation in the short and long-term conditions. The maximum displacement at the tunnel crest and ground level before installation of the supports were 7.4 cm and 2.2 cm, respectively. After support installation in both short term and long term, there is very little increase in the displacement in the soil mass, at the ground level and at tunnel perimeter. The maximum vertical displacement reaches 8 cm and 2.5 cm at the tunnel crest and the ground level. At the bottom of the tunnel, a heaving of 2.1 cm is seen from the short term to long term. In fact, the maximum increase in the displacement from short term to long term happens at the middle point of the invert. This is in conjunction with the maximum bending moment in the invert (can be seen at Fig 8).

Maximum displacements at the ground level and tunnel face are shown in Table 5. It can be seen that more than 90% of the displacement occurred during the tunnel excavation and before support installation. The combination of the shotcrete and the steel sets reduced the deformation rate to a very large extent.
### Table 5. Vertical displacement at different point in the soil mass

<table>
<thead>
<tr>
<th>Displacement (cm)</th>
<th>Before support installation (50% stress reduction)</th>
<th>Support installed (short term)</th>
<th>Support installed (long term)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ground level</td>
<td>2.2</td>
<td>2.3</td>
<td>2.5</td>
</tr>
<tr>
<td>Tunnel roof</td>
<td>7.4</td>
<td>8.0</td>
<td>7.8</td>
</tr>
<tr>
<td>Tunnel floor (Heave)</td>
<td>1.8</td>
<td>2.8</td>
<td>4.9</td>
</tr>
</tbody>
</table>

Plastic zones in the soil mass around the tunnel are shown in different stages in Fig. 6. It can be seen that before support installation, plastic points are concentrated on both sides of the tunnel with the thickness of almost 1.5 times of the tunnel radius. After installation of the supports, the number of plastic points decreases in both short term and long term. However, at the long term condition, the plastic zone thickness is reduced to a thickness of less than tunnel radius with the development of plastic points in the soil below the invert.

![Fig. 6. Plastic zone around tunnel a) after excavation, b) short term after support installation and c) long term after support installation](image)

The pore water pressure distribution in the short and long term conditions are shown in Fig. 7. It can be seen that in the short term, because of the expansion of the soil around tunnel due to tunnel excavation, the pore water pressure decreases around tunnel face in comparison with the hydrostatic pressure before tunnel excavation, which can be seen in the left and right boundaries of the model. This increases the soil shear strength in short time. At the long term condition, the pore water pressure changes to reach the initial hydrostatic pressure if the tunnel is completely water tightened.

![Fig 7. Pore water pressure around tunnel a) short term and b) long term](image)

### 6.1. Load distribution at the supports

As it was discussed earlier, the support system was designed to bear the soil and water pressure in both short and long term. In the short term, there is a decrease in the pore water pressure due to excavation, which results in lowering the pressure on the supports. In the long term, pore water pressure around the tunnel will increase due to drainage of the water and reaches the hydrostatic pressure. In this section, load distributions at the support system are illustrated. These include axial forces, bending moments and shear loads, which are shown in Fig. 8 for both short and long term. Due to the symmetry of the model, only the right side of the support system is shown.
As can be seen in Fig. 8, the load distribution on the supports increases in the long term in comparison with the short term. There are 23%, 33% and 48% increase in the axial force, shear load and bending moment, respectively, in the long term in comparison with the short term. The maximum bending moment moves from the corner to the middle of the invert from short term to long term. Due to the high value of bending moments in the invert structure, IPB20 was designed to support the loads. It should be mentioned that in the invert support, the plastic bending moment was considered to reduce the maximum bending moment occurred in the invert and redistribute it to the rest of the structure. The axial load distribution is almost uniform across the invert and side wall, while the maximum shear load occurs in the corner of the invert and reduced to almost zero in the middle.

7. CONTROL OF THE SUPPORT DESIGN

In order to design the support system in different segments, interaction diagram for combined bending and axial load (M-P) was used. The method of analysis is ultimate load, in which the dead load increased by the factor of 1.3 due to the temporary condition of the supports. The M-P curves at failure were determined via PCACOL V3 (8) for each segment of the tunnel support for the short and long term based on the geometry of the segments, shotcrete and steel properties at table 4 and 5. With the selected parameters the M-P curves for different segments (S1, S2 and S3) are shown in Fig 9. The critical M-P interactions due to the applied loads on the tunnel support are shown by bullet symbol in these figures. It should be mentioned that in these curves, the longitudinal spacing between steel sets is considered 75 cm.

As can be seen in Fig 9, the resulted loads in all segments are inside the M-P curves, which shows that the support system in both short term and long term satisfies the design requirements. The applied load in the invert structure is almost on the M-P curve. This is because a plastic bending moment was considered in this segment, which is equal to the maximum capacity of the IPB20 profile in this segment.

Although the analysis showed that the tunnel was safe in short and long term, two additional steps were taken into consideration to increase the safety of the project. First, excavation by top heading and bench approach; and secondly, use of umbrellas of slotted drainage pipes ahead of tunnel face to drain concentrated waters in sandy soil clusters. Both these steps were taken in to consideration in the next steps of the study.

8. CONCLUSION

In this paper, PLAXIS software was used to analyze and design the primary supports required for a large diameter tunnel in a saturated alluvial deposit. The convergence-confinement method was used to define strength reduction parameter before support installation. During this analysis and after support installation, undrained parameters were used for soil material and short term parameters for
lining system. For the long term interaction, drained parameters were used for soil material and long term parameters for the lining system. Combination of the undrained analysis in the short term and drained analysis in the long term resulted in optimized design for the support system. The combination of steel sets embedded in the shotcrete provided enough resistance to control the convergence of the tunnel in both short and long terms.

<table>
<thead>
<tr>
<th></th>
<th>Roof (S1) and Side Wall (S2)</th>
<th>Invert (S3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Short term</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Long term</td>
<td></td>
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</tbody>
</table>

Fig 9. Interaction diagram for combined bending and axial load in different segments of the support system

9. ACKNOWLEDGMENTS

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10. REFERENCES