Innovative Numerical Modeling of Ring Joints for Bored Tunnel Cairo Metro Line III

A. Abu-krisha
National Authority for Tunnels (NAT), Cairo, Egypt

1. Introduction:

Tunneling in Cairo has been activated in the last thirty years. Up to now, all of the Egyptian metro tunneling projects were preformed using Cut and Cover method and Tunnel Boring Machines (TBM) technology. A new metro line, line III, with 34-km long is planned to extend from Cairo International Airport, crossing under the downtown district and the River Nile and ending with two branches in urban areas of Giza, [1]. The first and the second phase of this line have been under construction and the other phases have been under preparing of tender documents. Different ground conditions, ground water levels and tunnel depths characterize every phase. Consequently, a suitable excavation and construction method have been assigned for phase one and two by slurry and EPB machines.

In this paper, numerical studies in the context of plane-strain, 2D finite element (FE) analyses are performed, comparing different layouts of support structure and construction schemes for Cairo Metro line III phase I. The numerical analyses were carried out using Plaxis v8 and CESAR-LCPC v4 packages software to investigate the different values of deformational behavior and internal forces. The sequence of the TBM tunnel excavation was idealized and modeled in two steps. It has been pointed out by Abu-Krisha [2] that more simulation steps can be performed in the 2D context to account for the machine stiffness and the grouting pressure. The new model is characterized by the realistic consideration of the radial ring joint behaviors and construction processes. Hence, in addition to support the design, the model is able to assist the steering lining joints behavior during tunneling process.

2. Geotechnical conditions

Stratigraphy of deposits under the Greater Cairo consists of several hundred meters of recent alluvial and diluvial layers. Within the zones of phase I & II, these deposits consist mainly of clay and sand layers with different geotechnical properties. The soil layers stratification and properties are based on the geotechnical investigation report [3]. A summary of the model geotechnical parameters that used in the current analyses is given in Table 1. The ground water level is recorded between 3 – 8 m from the ground within the area of phase I and no ground water within the area of phase II. According to the current alignment, the tunnel is located either within the clay and sandy or sand gravel layers. The geological longitudinal profile at the adopted section is shown in Figure 1.

<table>
<thead>
<tr>
<th>Soil Layer</th>
<th>Depth (m)</th>
<th>γ</th>
<th>E</th>
<th>ν</th>
<th>K0</th>
<th>φ</th>
<th>C</th>
</tr>
</thead>
<tbody>
<tr>
<td>Made ground</td>
<td>0-2.5</td>
<td>17.0</td>
<td>4.0</td>
<td>0.30</td>
<td>0.53</td>
<td>28</td>
<td>0</td>
</tr>
<tr>
<td>Upper Sand</td>
<td>2.5-8</td>
<td>19.0</td>
<td>35.0</td>
<td>0.30</td>
<td>0.38</td>
<td>36</td>
<td>0</td>
</tr>
<tr>
<td>Middle Sand</td>
<td>8-20.8</td>
<td>19.5</td>
<td>70.0</td>
<td>0.30</td>
<td>0.64</td>
<td>38</td>
<td>0</td>
</tr>
<tr>
<td>Sand &amp; Gravel</td>
<td>20.8-22.1</td>
<td>20.0</td>
<td>100.0</td>
<td>0.30</td>
<td>0.38</td>
<td>38</td>
<td>0</td>
</tr>
<tr>
<td>Lower Sand</td>
<td>22.1-70</td>
<td>20.0</td>
<td>12.0</td>
<td>0.30</td>
<td>0.64</td>
<td>41</td>
<td>0</td>
</tr>
</tbody>
</table>

Table 1: Soil layers and geotechnical parameters
3. **TBM lining analysis**

In general the stiffener of the lining is higher than the ground mass so the lining tends to attract loads. All joints in the real TBM tunnel are designed as fully structural joints. The hinges are only an artifice for 2D numerical modeling to account for 3D effects. The usage of plastic hinges produces a prediction of worst case for ground movements, and since the moment capacity of the lining is limited to its actual capacity. This provides a good check on the overall stability of the tunnel. In reality it is supposed that, the actual bending moments in lining do not occur because the local ground arching spreads the load.
4. Numerical modeling

In the present analysis a 2D plane strain elasto-plastic law soil model based on Mohr-Coulomb failure criterion has been developed. The precast concrete lining elements are modeled as an elastic material, Young’s Modulus ($E = 28500 \text{ MPa}$) and Poisson's ratio of 0.2. The concrete stiffness has been divided by a factor of 1.5 to account for long term. The incremental application of loads in a 2D model replicates the 3D effect seen in a real tunnel whereby the load comes onto the lining as excavation face advances. These values are based on an assessment of the load development from 2D numerical models and calibration against monitoring data from Cairo metro real tunnels. The finite element model adopted for the analysis by finite element programs CESAR-LCPC and PLAXIS. The de-confinement ratio for the distance corresponding to lining erection for both programs is taken equal to 30%. The tunnel lining is composed of semi-articulated rings made of 8 precast reinforced concrete segments, 5 standard segments, 2 counter segments and one key segment, as shown in Figure (2), [4].

For the numerical analysis by program CESAR-LCPC, model type 1, the tunnel segmental lining is assumed to be a continuous circular elastic element. This accounts for presence of permanent dowels between rings. The flexural stiffness reduction due to the joints is taken into account implicitly via a global approach based on Muir-Wood theory [5]. Consequently, the tunnel lining is modeled as a continuous ring with a reduced thickness from 0.40 m to 0.32 m. The finite element model adopted for the analysis of the section at KP 19.2 is shown in Figure (3), [4].

For the numerical analysis by program PLAXIS, model type 2, the tunnel lining is modeled as a semi-articulated rings made of 8 segments and 8 joints, not continuous rings, with thickness 0.40 m. The stiffness of radial joint is coupling the bending capacity of the joint with its rotation and its normal force. The joint is a connection that allows for a discontinuous rotation in the point of connection (joint). The angle of rotation at each joint in the model is checked against allowable limits. The analysis are carried out for two independent locations of key segment, one at $12^\circ$ from vertical axis (crown position) and second at $84^\circ$ from vertical axis (spring position). The finite element model adopted for the analysis of the same section at KP 19.2 is shown in Figure (4), [4].

The simulation process is started by initializing the geostatic state of stresses and specifying the existing surface load (buildings/traffic...). Then the tunnel excavation procedure is simulated taking into account the stress redistribution around the tunnel zone. The sequence of the tunnel excavation was idealized and modeled in two steps for each ring. In the first step, the stiffness of the soil inside the tunnel part is reduced by a reduction factor (de-confinement ratio). In the second step, the excavated soil is eliminated simultaneously with the installation of the segments lining. The ($\lambda$) value can be determined by comparing the numerical results with in-situ measurements. It has been pointed out by Abu-Krisha, [6, 7] that more simulation steps can be performed in the 2D context to account for the machine stiffness and the grouting pressure. However, the results of the described basic and simple simulation are more conservative in this case for the final resulted deformation. Calculation of the stresses and deformations of the support elements (lining) and the surrounding soil was based on nonlinear FEM, [8, 9]. The calculation sequences for both systems of the lining in both model types were done and can be summarize as follows:

- Phase (0): Geostatic stresses.
- Phase (1): Surface loads (building and traffic loads) application.
- Phase (2): Tunnel excavation with “Lamda = 30%” de-confinement.
- Phase (3): Lining installation, loading with remaining de-confinement.

The soil masses around the tunnel are modeled as a natural case without any soil improvement. The system of the lining support is modeled as an elastic beam element with eight hinges. Two systems of ring segment configurations are modeled.
5. Results and discussions

The profiles of vertical surface settlements for different model types after tunneling are illustrated in figure 5. All the values are beyond the acceptable limits for the safety of surface building. The vertical settlements at ground surface were evaluated for every construction phase in both models. The result of settlement was found to be more congruence for the analysis by model type 2 comparing with the site monitoring. The results of these analyses in the two models types are illustrated in figures 6 & 7. The plasticity in both models is investigated and it looks reasonable.
The results of the two numerical models for the lining that extracted from both final and intermediate phase are checked for internal forces to see if they look realistic. An empirical manual check is performed and compared with the numerical calculation results. The models procedures are calibrated against monitoring data for the lining. In the calibration study, the reduction factor ($\lambda$) is varied until the surface settlement and lining displacements match with field data. Figure 8 presents the calibration comparison at three points on the lining for both models with field measurements.

Bending moments and axial forces in the circumferential direction are extracted from both models type 1 and 2. The normal forces resulted from the model type 2 had very small change compared with model type 1, that can be negligible. The bending moments resulted from the model type 2 had much change compared with model type 1, which should be considered. Figure 9 portrays the comparison of the bending moment in the two numerical models. Table 2 presents the comparison of the internal forces in the lining for the two numerical models and empirical calculation. The value of bending moment of model type (1) is closer to the empirical value, while the normal and shearing forces of model type (2) are closer to the empirical value. Thus the model type (2) is technically preferable.
Figure 7: Deformed mesh of section at KP 19.2 by model type 2.

<table>
<thead>
<tr>
<th>Points</th>
<th>Field (mm)</th>
<th>Model (1)</th>
<th>Model (2)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>0.351</td>
<td>0.875</td>
</tr>
<tr>
<td>2</td>
<td>0.5</td>
<td>0.2</td>
<td>0.35</td>
</tr>
<tr>
<td>3</td>
<td>-0.5</td>
<td>-0.2</td>
<td>-0.35</td>
</tr>
</tbody>
</table>

Figure 8: Comparison of lining deformation for the calibration study.

Figure 9: Comparison of bending moment between model type 1 and type 2.
<table>
<thead>
<tr>
<th>Internal Forces</th>
<th>Empirical Calculation</th>
<th>Model type (1)</th>
<th>Model type (2)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bending moment KN.m/m</td>
<td>72</td>
<td>81</td>
<td>108.5</td>
</tr>
<tr>
<td>Normal forces KN/m</td>
<td>-1400</td>
<td>-1150</td>
<td>-1200</td>
</tr>
<tr>
<td>Shearing forces KN/m</td>
<td>66</td>
<td>40.6</td>
<td>53.11</td>
</tr>
</tbody>
</table>

Table 2: Comparison of internal forces between empirical calculation and numerical models.

6. Conclusion

The goal to develop a tunnel lining design with innovative the numerical modeling simulation to be more reality was reached. The innovative numerical modeling of ring joints for TBM tunnel therefore not only results in a robust but also economical tunnel structure. The innovative analysis has proven that the model with eight joint simulations is more accurate and realizable than the model with the stiffness reduction with continuous circular ring elastic element. This model can then provide valuable information on the stability and safety of the segmental lining design.

The radial joints in bored tunnels present the weakness link in the tunnel structure as they only derive their strength from the compressive normal force in the tunnel ring. The bending moment in the tunnel ring at radial joints are transferred to the reinforced cross section of the segment field in the neglecting joint.

The use of powerful simulation techniques will encourage updating/optimizing modern construction techniques, minimizing construction cost and will help to choose the optimum system for the tunnel lining ring design. Following the above mentioned principles for 3D numerical model, this will produce more realistic results.

REFERENCES


