Three-dimensional numerical model of Baghdad metro with fluctuation of ground water

Aqeel Al-Adili¹ and Muammar Al-Taee²

¹ Dean of Petroleum Technology Department, University of Technology, Baghdad, Iraq,
² Department of Civil Engineering, College of Engineering, University of Misan, Misan, Iraq

Email: muammar_altaee@uomisan.edu.iq

Abstract. When a tunnel is excavated below the ground water table, water flows into the excavated faces of the opening tunnel and leads to affecting the stability by reducing the effective stress and shear resistance by generating seepage forces towards the excavation boundaries. The seepage flow may lead to ground water drawdown and time dependent subsidence due to consolidation and rearrangement of the pores. The effect of interaction between tunnelling and ground water on the soil around boring tunnel and ground surface has become an essential part of the planning, design, and construction of tunnelling project. This paper presents the analysis and the results of a numerical solution for a series of actual tunnelling processes of the proposed Baghdad metro as a case study. The finite element package, ABAQUS 2017, was used for this study. A fully coupled three dimensional stress-pore pressure model has been adopted to realistically capture the mechanical and hydrological interaction between tunnelling and ground water. The effect of fluctuation of ground water level on the pore water pressures for soil surrounding tunnel have been computed. Four sequential simulation steps have been done for the location approximately under Kasra Square where the tunnel will be approximately passed through different soil layer profiles. It is concluded that increased pore water pressures is associated with rising ground water and vice versa. It was also concluded that the left tunnel route will be under danger during the excavation only because of presence of negative pore water pressures along it. Therefore, preliminary proceedings of safety are recommended during the excavation of the left tunnel route.

1. Introduction

Proper control of the ground water during tunnelling requires a thorough understanding of interaction mechanisms between the tunnelling and the ground water within the context of the stress-pore water coupled effect. Despite the importance of understanding the stress-pore water coupled effect on tunnelling performance, studies concerning this subject are limited. Numerical methods have been used as primary tools in most of the available studies because of technical difficulties involved in physical modelling of the stress-pore pressure coupled behaviour in either small or large scale. Some of the available studies related to this subject performed numerical analyses with the steady-state seepage analysis or sequential seepage analysis and stress analysis, which cannot accurately model the fully coupled interaction behaviour between the tunnelling and the ground water where much needs to be investigated to better understand the three dimensional stress-pore pressure coupled interaction mechanism during tunneling [1; 2; 3 and 4]. This paper presents the fully coupled 3D finite element model and the simulation strategy results.
2. Description of the project and the selected site

The proposed Baghdad metro lies in Baghdad city. It has total length equal to 39 km including 42 stations. This project comprizes two lines that connect both sides of Baghdad city; Karkh and Rusafa. The central station in which the two lines of Baghdad metro encounter each other lies in Rusafa at Killani square on the Jamhurriya street as shown in figure 1 (a) [5 and 6]. Two routes for each line that were proposed in the previous study (in 1980) and the same proposition are adopted in this study by French firm Systra [7]. The tunnel shape is circular in cross section with 6.3 m outer diameter and 0.3 m of concrete lining thickness. The vertical depth of tunnel is approximately in variation along its extension depending on the geological section of Baghdad city. An advanced rate of excavation must be allocated beforehand according to the boring machine used in excavation, conditions of soil stratification, and other restrictions of urban area.

The site of Baghdad metro passing approximately under Kasra Square (44° 22' 36.7" E and 33° 21' 31.7" N) was considered in this study (figure 1 (b)). The two routes of tunnel at this location will be excavated at different depths from the ground surface where the depth of crowns of the left and right routes are 20.282 m and 10.5459 m, respectively and the horizontal distance between the outer diameters of the two routes equals to 45 m. Figure 2 shows the ground profile and the tunnel cross sections under Kasra Square approximately, deepest phreatic water level at this location was recorded from wells within Baghdad Governorate [8]. Table 1 summarizes the soil strata as well as the mechanical and hydraulic properties of the soil layers for this location [9].

Figure 1. Layout of Baghdad metro plotting on satellite image of Baghdad City (60 cm error).
Figure 2. Ground profile and tunnel cross sections of the proposed Baghdad metro at Kasra Square.

Table 1. Geotechnical properties of ground profile at location considered in this study.

| Layer no. | Depth of layer (m) a | Thickness (m) a | Soil description a |
|-----------|---------------------|-----------------|--------------------|
| Above W.T. b | From To | From To | From To |
| 1 | 0 | 3.26 | 3.26 | Fill (Grey silty clay with fine yellow bricks, CL) |
| 1 | 3.26 | 3.5 | 0.24 | Fill (Grey silty clay with fine yellow bricks, CL) |
| 2 | 3.5 | 8.8 | 5.3 | Stiff brown silty clay to highly plastic clay (CH to MH to CL) |
| 3 | 8.8 | 10.5 | 1.7 | Sandy silt with trace of clay (ML) |
| 4 | 10.5 | 15 | 4.5 | Brown silty clay with little |
| 5 | 15 | 16.9 | 1.9 | Brown silty clay or clayey silt |
| 6 | 16.9 | 30.8 | 13.9 | Very dense grey fine sand to silty fine sand (SP to SP-SM to SM, IV) |
| 7 | 30.8 | 36.03 | 5.23 | Hard grey silt clay (CL) |

| Layer no. | ρ (t/m³) b | E (kN/m²) b | Plasticity c |
|-----------|------------|-------------|--------------|
| Above W.T. b | ρₐ | ρₛ | E | ν | φ (°) | C (k) |
| 1 | 1.468 | 6600 | 0.2 | 9 | 60.2 | 3.00E-10 | 0.13 |
| 1 | 1.888 | 6600 | 0.2 | 9 | 66.5 | 3.00E-10 | 0.75 |
| 2 | 1.857 | 18500 | 0.45 | 0 | 120 | 2.00E-10 | 0.757 |
| 3 | 1.93 | 25366.7 | 0.35 | 20 | 100 | 8.00E-07 | 0.735 |
| Below W.T. | ρₐ | ρₛ | E | ν | φ (°) | C (k) |
| 4 | 1.884 | 22000 | 0.38 | 18 | 99.5 | 2.75E-10 | 0.65 |
| 5 | 1.963 | 45000 | 0.4 | 18 | 97 | 5.00E-09 | 0.819 |
| 6 | 2.25 | 25000 | 0.29 | 45 | 12 | 5.00E-09 | 0.8 |
| 7 | 1.85 | 22500 | 0.39 | 15 | 110 | 3.50E-10 | 0.813 |

b From the raster surface of lower GW through the last decade.

a Source: [9].
3. Coupled Stress-Pore Pressure Analysis

The commercially available finite element package ABAQUS 2017 was used for analyses. ABAQUS was selected in this study to take advantage of its effectiveness in coupled stress-pore pressure modelling as well as robustness in the numerical solution strategy for soil plasticity. Details of the finite-element model are described in the following subsequent sections:

3.1. Three Dimensional Finite Element Model

In order to realistically capture the interaction mechanism between the tunnelling of the proposed Baghdad metro at Kasra Square and ground water, a fully coupled 3D stress-pore pressure finite element model capable of simulating the sequential tunnelling process was adopted. Note that the importance of carrying out the fully coupled stress-pore pressure analysis for tunnelling cases where the interaction between the tunnelling and ground water takes place has been discussed by [10; 11 and 12]. The tunnel was assumed to be excavated full face. In the current study, three cases of ground water level were adopted. These cases are as follows:

Case 1: Ground water level at 3.26 m below the ground surface (figure 2).
Case 2: Ground water level at the ground surface.
Case 3: Existing 0.5 m height of surface water above ground surface (flooding case may occur from Tigris river, see figure 1 (b)).

Figure 3 shows the three dimensional finite element model of Baghdad metro under Kasra Square. This model extended to: (i) depth of 1.5 times the outer tunnel diameter below the deepest tunnel invert (left route) making total vertical depth (in Z-direction) of 36.03 m, (ii) horizontal distance (in X-direction) of 8 times the outer diameter from the centerline of each route of tunnel (8D₀ to the left of the centerline of left route and 8D₀ to the right of the centerline of right route) giving total width of 147.1 m, and (iii) the direction of the tunnel advance (in Y-direction) of 8 times of the outer tunnel diameter making total tunnel advance of 54.4 m. The locations of these boundaries were selected so that the presence of boundary beyond them does not significantly affect in the stress-strain-pore pressure field in the domain [1; 2; 3; and 4].

Figure 3. Three dimensional finite element model of Baghdad metro at Kasra Square.
In terms of the discretization, the regions closed to the tunnel sections were discretized in 1 m segment along the dimensions in the Z and Y directions and along 18.9 m distance in the X direction from the left and right of the center line of each sections to allow for realistic modelling of the actual construction sequence while in the remaining zones of the model the mesh size was made in 2.5 m to reduce the computational cost. Eight nodes trilinear displacement and pore pressure elements with reduced integration (C3D8RP) were used for discretizing the soil layers below the initial ground water table, and the shotcrete liners and the soil layer above the initial ground water table were discretized sing stress-displacement eight nodes brick elements with reduced integration (C3D8R).

In terms of the displacement boundary conditions (mechanical boundary conditions), roller boundaries were placed on the vertical faces of the model (left, right, front, and back boundaries) while pin supports were applied to the bottom boundary. In a coupled stress-pore pressure analysis, the hydraulic boundary conditions also need to be prescribed in addition to the displacement boundary conditions. With referencing to figure 3, a no-flow condition was assigned to the vertical boundaries perpendicular to the tunnel drive and the bottom boundary.

The pore water pressures on the left and right vertical boundaries were assumed to be constant at the original ground water level (3.26m below GS for Case 1 and at GS for Case 2) and to be constant under height of surface water of 0.5m throughout the analysis. Drainage boundary conditions were defined before and after tunnel excavation as follows: before tunnel excavation, all surfaces are undrained; after excavation, free drainage is allowed for the excavated surface (soil faces ambient concrete liners) as well as the inner faces of lining by assigning a zero pore water pressure flow boundary condition to allow for the water to occur during tunnel excavation. No recharge at the ground surface during the tunnelling was assumed for simplicity, although there may be near surface recharge from leaking water pipes in such urban situations.

Table 2 summarizes the initial conditions of the vertical effective stress (geostatic stress) and the pore water pressures for each layer within the model. Degree of saturation for ground below initial ground water level and concrete liners were defined equal to 1 as hydraulic initial conditions while the ground above this level was assumed to be fully dry (for Case 1 only). It is known that any material has permeable property, the initial condition of void ratio is also needed. Hence, the void ratio values mentioned in table 1 were considered as the initial values for each layer within the model.

With regard to the constitutive modelling, the soil layers were assumed to be an elastoplastic material conforming to the Mohr-Coulomb failure criterion together with the non-associated flow rule proposed by Davis (1968) [13], while the shotcrete lining was assumed to behave in a linear elastic manner. The time dependency of the strength and stiffness of the shotcrete lining after installation was not modelled in the analysis but rather an average value of Young's modulus representing green and hard shotcrete was employed. The mechanical properties of the shotcrete lining used in this study are 2500 Kg/m$^3$ of density, 30×10$^6$ kN/m$^2$ of the modulus of elasticity, and 0.2 of Poisson's ratio. The geotechnical properties of ground given in table 1 were used for analysis.
Table 2. Initial conditions of PWP and effective stresses of three dimensional finite element model of Baghdad metro at Kasra square.

| Layer no. | Pore pressure (kN/m$^2$) | Effective stress (kN/m$^2$) | Lateral coefficient (Rankine) ($K_a$) |
|-----------|--------------------------|-----------------------------|-------------------------------------|
| Above W.T. |                          |                             |                                     |
| Top       | 0.000                    | 0.000                       | 47.857                              |
| Bottom    | 0.000                    | 47.857                      | 0.729                               |
| 1         | 0.000                    | 2.400                       | 47.857                              |
| 2         | 2.400                    | 49.988                      | 49.988                              |
| 3         | 55.400                   | 95.409                      | 110.847                             |
| Below W.T. |                          |                             |                                     |
| Top       | 2.400                    | 55.400                      | 47.857                              |
| Bottom    | 55.400                   | 95.409                      | 100.000                             |
| 4         | 2.400                    | 55.400                      | 47.857                              |
| 5         | 55.400                   | 95.409                      | 110.847                             |
| 6         | 136.400                  | 275.400                     | 110.847                             |
| 7         | 275.400                  | 327.700                     | 343.028                             |

3.2. Simulation Strategy

In the current fully coupled analysis, the steps of simulation were comprised stages before, during, and after tunnel excavation. Hence, four steps of simulation were adopted in the model. These steps are geostatic, excavation, linings installation, and consolidation steps. In the present study, the time schedules of excavation and linings installation suggested in the newest study by French firm Systra were adopted. They are suggested that tunnel advance of 5 m to 5.5 m per one day can be excavated and of 1.5 m to 1.75 m per one hour can be lined. Therefore, with regards to figure 3, 10 days and 30 hours were adopted as the time period for the excavation and linings installation steps, respectively. Then, 10 days after the linings installation, the pore water pressure were also calculated.

4. Results and Discussions

After submitting the three dimensional finite element model shown in figure 3 for each cases mentioned in Section 3.1, many output field variables from the deformed shape of this model can be considered in the analysis. Figure 4 shows the observation paths along them the pore water pressures of soil at the crown and invert lines for each tunnel route after tunnel excavation and after completing linings installation.

Figure 4. Observation paths along them the PWP computed.
The results of the pore water pressures for each case are described in the following subsequent sections:

4.1 Case 1: Ground water level at 3.26 m below the ground surface
Graphical comparisons between the pore water pressures along the crown and invert lines of the two tunnel routes at this location after tunnel excavation were given in figure 5, which shows very clearly that positive pore water pressures occurred along the right route of tunnel. It was also noticed that pore water pressures had maximum values along the crown line of this route with range approximately from 29 kN/m² to 47 kN/m² while they were approximately close to zero along the invert line. On the other hand, negative pore water pressures were observed along the left route of tunnel of about -13.5 kN/m² along the crown line and with range from -3.6 kN/m² to -4.6 kN/m² along the invert line.

Figure 5 shows that positive pore water pressures took place along the two tunnel routes. It was observed that pore water pressures increased from those occurring after tunnel excavation. It was also observed that maximum pore water pressures were about 55.3 kN/m² along the crown line of the right tunnel route. Then, 10 days after the completion of the project construction, increase in pore water pressures along the two tunnel routes occurred as shown in figure 7. It was noticed that maximum pore water pressures was about 109 kN/m² along the invert line of the left tunnel route.
4.2 Case 2: Ground water level at the ground surface

Graphical comparisons between the pore water pressures along the crown and invert lines of the two tunnel routes at this location after tunnel excavation were given in figure 8, which shows very clearly that positive pore water pressures occurred along the right tunnel routes. It was also noticed that pore water pressures had maximum values along the crown line of this route with range approximately from 39.2 kN/m\(^2\) to 65.2 kN/m\(^2\) while they were approximately very close to zero along the invert line. On the other hand, negative pore water pressures were observed along the left tunnel route of about -13.5 kN/m\(^2\) along the crown line and with range from -3.6 kN/m\(^2\) to -4.1 kN/m\(^2\) along the invert line.

Figure 9 shows that positive pore water pressures took place along the two tunnel routes. It was viewed that pore water pressures increased from those occurring after tunnel excavation. It was also observed that maximum pore water pressures were approximately from 76.7 kN/m\(^2\) to 79.5 kN/m\(^2\) along the crown line of the right route of tunnel. Then, 10 days after the completion of the project construction, rise in pore water pressures along the two tunnel routes occurred as shown in figure 10. It was noticed that maximum pore water pressures was about 124.4 kN/m\(^2\) along the invert line of the left route of tunnel.

4.3 Case 3: Existing 0.5 m height of surface water above ground surface

Graphical comparisons between the pore water pressures along the crown and invert lines of the two tunnel routes at this location after tunnel excavation were given in figure 11, which shows very clearly that positive pore water pressures occurred along the right route of tunnel. It was also noticed that
maximum pore water pressure was 67.6 kN/m$^2$ taking place at 2.52 m on the crown line of this route while there were approximately approaching to zero for pore water pressures along the invert line. On the other hand, negative pore water pressures were observed along the left route of the tunnel of about -13.7 kN/m$^2$ and -3.6 kN/m$^2$, respectively.

Figure 12 shows that positive pore water pressures occurred along the two tunnel routes. It was seen that pore water pressures increased evidently from those occurring after tunnel excavation. It was also observed that maximum pore water pressure was 82.6 kN/m$^2$ taking place at 2.52 m on the crown line of this route. Then, 10 days after the completion of the project construction, increase in pore water pressures along the two tunnel routes occurred as shown in figure 13. It was noticed that maximum pore water pressures were about 127.2 kN/m$^2$ along the invert line of the left tunnel route.

![Figure 11. PWPs along Baghdad metro tunnel at Kasra Square for Case 3 after tunnel excavation.](image1)

![Figure 12. PWPs along Baghdad metro tunnel at Kasra Square for Case 3 after completing linings installation immediately.](image2)

![Figure 13. PWPs along Baghdad metro tunnel at Kasra Square for Case 3 after 10 days of the project construction complete.](image3)

5. Conclusions
A fully coupled 3D stress-pore pressure finite element model was used to realistically capture the mechanical and hydrological interaction between the tunnelling and ground water. The analysis of boring tunnel behaviour before and after tunnel excavation enables us to take into account the impact of tunnelling on surrounding environments. Hence, the following findings for three cases were obtained:
1. After completing excavation of tunnel, positive pore water pressures occurred along the right tunnel route and negative pore water pressures were obtained along the left tunnel route.
2. After finishing lining installation stage immediately, all pore water pressures were positives noticing that these pressures along the right tunnel route were greater than those along left tunnel route yet.

3. After 10 days of the project completion, all pore water pressures were positive but the maximum values were observed along the crown lines of the left and right tunnel routes, respectively and the minimum values happened along the invert lines of the left and right tunnel routes, respectively.

4. It was concluded that increasing pore water pressures is associated with rising ground water and vice versa.

5. It was concluded that the left tunnel route may be under danger during the excavation only because of presence of negative pore water pressures along it. Therefore, preliminary proceedings of safety are recommended during the excavation of the left tunnel route.

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