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Stress-Displacement Distribution during Subway Station Construction Using CAPS Method

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Abstract - Construction of concrete arch pre-supporting system (CAPS) is mainly based on a method of ancient Iranian small water tunnels, Quanat, which is generally fast and more economical than usual methods such as fore poling. This paper intends to reveal the effects of four separate parameters (surcharge, rib intervals, pile diameter, and effective length of piles) in estimating the stress and moment distribution of subway station elements. Three dimensional finite element models show that axial and shear stresses and induced bending moments in ribs and piles decline when X/H ratio (rib intervals to depth of tunnel center) increases. Apart from this, it is observed in model study as well as in measured data that the maximum ground surface settlement is more comparable to the model results. Predicted amount of vertical movements with numerical models is lower than observed deformations at the main axis. In addition, there is an optimum settlement around 3m effective length of piles.

***Keywords*:** Settlement, Stress distribution, Metro station, FEM, Concrete arch pre-supporting system

1. Introduction

Construction of concrete arch pre-supporting system (CAPS) is generally fast and more economical than usual methods such as fore poling and is a rib like underground structure consisted of concrete piles and arch beams constructed around a proposed underground space as a retaining structure.

Some researchers presented some reports about deep underground excavation using CAPS method to estimate the amount of Ground Surface Settlement (GSS) due to tunneling, numerically and experimentally [1-7]. Pile Beam Arch (PBA) method was introduced by Xu et al. like CAPS as an economical and effective for determining ground surface settlement in presence of water [8].

Ding et al. used two-dimensional finite element model to simulate the shield tunneling and the platform construction by PBA method to enlarge the shield tunnel. The results showed that the arch primary lining was mainly in the compressive status [9]. Yan et al. carried out a series of 3D modeling using PBA method and showed that arch primary lining experienced more deformation than GSS, and high level of axial stress [10]. Valizadeh Kivi et al. analysed the stress and bending moment of large span underground station established by Central Beam Column (CBC) structure using full 3D Finite Element Method (FEM). The authors observed that the maximum axial force applied by buried columns took place at the end of tunneling whereas the amounts of bending moment were higher in some middle steps. By decreasing the column distances and increasing the columns diameter the compressive strengths were maximized on top of the tunnel crest, however, the latter had ales impact on stress distribution than column intervals [11]. Liu concluded that horizontal displacement of the pile body reduced significantly as the pile diameter increased or the spacing decreased, but the bending moment on the side pile increased greatly. The buried depth of the side pile had a large influence on constraint conditions at the pile bottom. Also, narrowing the pile spacing produces better effects in case of a lower requirement for displacement control [12].

This paper presents a 3D numerical analysis of retaining structure of subway station based on CAPS method of construction. The method and the analytical approach have been verified according to the monitoring data of the GSS using Plaxis 3D Tunnel code. In order to solve the problem, model development has been carried out for two different surcharges (depth of station) and two extra rib intervals. Finally, the stress-displacement of rib and piles were recorded for more interpretation.

2. Case Study
2.1. Site location
Tehran subway line 6 originates from Dowlatabad highway (South East of Tehran) and ends in Can Complex (North West of Tehran). The present length of this project is 30 kilometers and it passes through 28 stations. The final width of all stations, length and width of platforms are 14 m, 140 m and 4 m, respectively. The average distance of two consecutive stations is 1150 m. The Southern parts were excavated by TBM with 8.15 m of diameter, whereas northern sections were dogged up through NATM approach with 8.35 m of diameter.

2.2. Geotechnical characteristics
According to Standard Penetration Test (SPT), grading, Atterberg limits and natural water content determined from the attained specimens, the dominant soil type was estimated to be a main clayey sand (SC) with gravel and boulder represented by N_{SPT}>50 and unit weight of 20.5 kN/m^3 (Table 1). The topsoil with 19.5 kN/m^3 density, and water table were observed on 2-4 m and 42 m of surface ground, respectively. To achieve mechanical properties of soil layer, some direct shear tests and plate load tests have been done in LAB and in site.

Table 1: Soil mechanical properties

Layer	C (kN/m^2)	Φ (°)	E (kN/m^2)	υ	N_{SPT}	PI	ω (%)
SC	30	36	13e4	0.35	>50	12	8
Top soil	0-10	27-30	15e4-40e4	0.3	-	14	7

2.3. Station structure
Arch beam interval is 2.5 m and ranged from 0 to 60 axes across the station platform (length=142.9 m). The cross section of this part includes 15.9 m in width and 20 m in height. The length and the diameters of the piles are 15.25 m and 0.9 m, respectively. The cross section of concrete arch beams (ribs) is half-ovate with 1 m in width and 1.5 m in height. Table 2 exhibits the specifications of structural elements.

Table 2: Specification of structural elements

<table>
<tr><th>Position</th><th>Diameter (m)</th><th>E (kN/m^2)</th><th>γ (kN/m^3)</th><th>A (m^2)</th><th>I (m^4)</th></tr>
<tr><td colspan="2">Retaining structure</td><td>2.49e7</td><td>23</td><td>0.15</td><td>2.18e-4</td></tr>
<tr><td colspan="2">Middle access tunnel’s wall</td><td>2.49e7</td><td>23</td><td>0.28</td><td>1.5e-3</td></tr>
<tr><td colspan="6">Pile</td></tr>
<tr><td rowspan="2">2.5</td><td>0.8</td><td>6.36e6</td><td>21.13</td><td>0.502</td><td>2.01e-2</td></tr>
<tr><td>0.9</td><td>7.14e6</td><td>21.27</td><td>0.636</td><td>3.22e-2</td></tr>
<tr><td rowspan="2">3</td><td>0.8</td><td>5.32e6</td><td>20.94</td><td>0.502</td><td>2.01e-2</td></tr>
<tr><td>0.9</td><td>5.97e6</td><td>21.06</td><td>0.636</td><td>3.22e-2</td></tr>
<tr><td rowspan="2">3.5</td><td>0.8</td><td>4.58e6</td><td>20.81</td><td>0.502</td><td>2.01e-2</td></tr>
<tr><td>0.9</td><td>5.14e6</td><td>20.91</td><td>0.636</td><td>3.22e-2</td></tr>
<tr><td colspan="6">Rib</td></tr>
<tr><td colspan="2">2.5</td><td>8.24e6</td><td>21.47</td><td rowspan="3">1.226</td><td rowspan="3">0.192</td></tr>
<tr><td colspan="2">3</td><td>6.89e6</td><td>21.22</td></tr>
<tr><td colspan="2">3.5</td><td>5.92e6</td><td>21.05</td></tr>
</table>

In order to apply the distance impact of ribs, elasticity modulus (E_i) and specific density of concrete (γ_i) were modified considering concrete area (a_c)-total area of concrete and soil (a_c+a_s) ratio. Therefore, the two modified parameters were determined based on relations (1) and (2).

$$E_{equal} = \frac{\sum_{i=1}^{n} E_i A_i}{\sum_{i=1}^{n} A_i} \tag{1}$$

$$\gamma_{equal} = \frac{\sum_{i=1}^{n} \gamma_i A_i}{\sum_{i=1}^{n} A_i} \tag{2}$$

3. Numerical modelling

The height, width, and in-plane depth (station length) of model were chosen to be 55 m, 80 m, and 143 m, respectively. A uniform dead load of the near buildings was considered for each residential floor 800 kg/m^2 and for each car park 550 kg/m^2. Incessant traffic load of the near busy pathways and streets were determined 2400 kg/m^2. By the way, the northern part of station in where exists Saint Mary Park, a uniform load of 2000 kg/m^2 intensity was introduced. Noted that, there are two soils layer assigned to top soil (z=0 to z=-4.5 m) and SC (z=-4.5 m to z=-55 m). Soil layers, structures, and interfaces were assigned based on Table 1 and 2. Constitutive model of soil layers and structural elements were set to be Mohr-Columb and elastic, respectively. Other considerations are 1) to set standard fixities; 2) to neglect the probable instability due to excavation of access galleries, connection pins, and injection galleries. Figure 1 shows the steps of preparing model to be run.

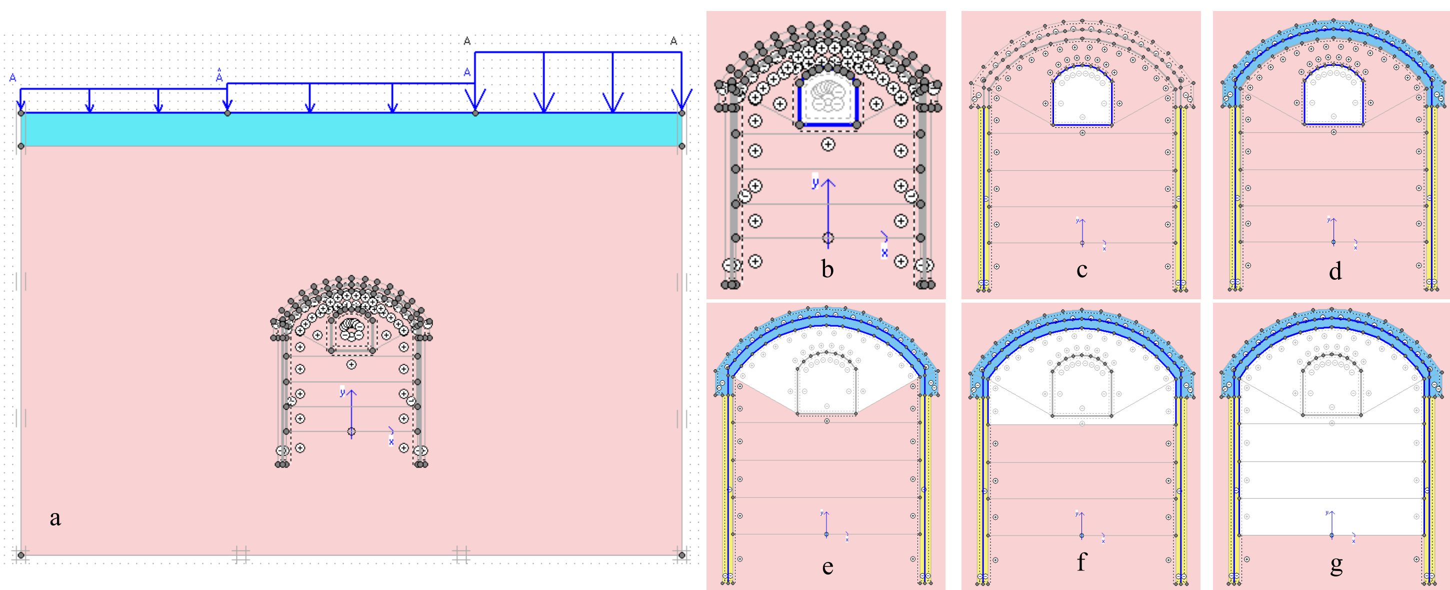


Fig. 1: Applying traffic and building load (a), Cutting and shutting middle access tunnel (b), piles installation (c), AB installation (d), Destroying middle lining and cutting upper zone (e, f), Excavating beneath soil layers to rail level in three separate steps (g).

4. Verification

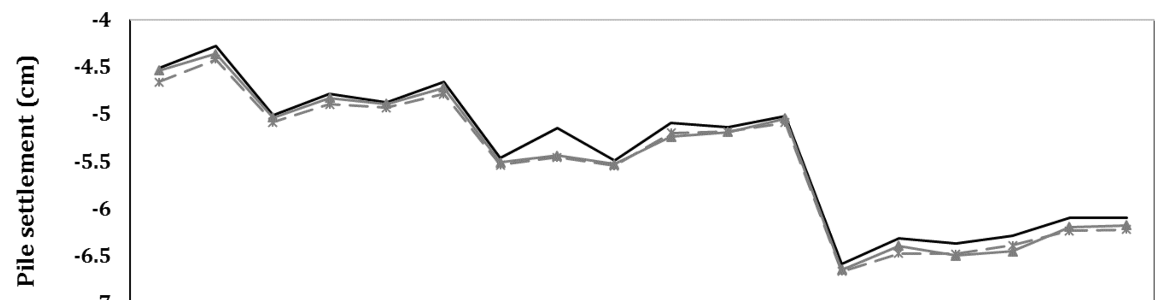
The station site provides reports related to monitoring of surface ground settlement in a period of 250 days in which all deformations become stable. According to existing conditions (20 m surcharge, 2.5 m rib interval, 0.9 m pile diameter, 3m buried length of piles) the attained numerical settlements have an acceptable error about ±5% from in situ collected data. As it is obvious, the equivalent analytical settlement estimated to be 4.9% which is in range of settlements achieved from monitoring with deviation about ±5%. As a challenging issue, this paper intends to investigate the impact of embedment length of piles (3 m, 3.5 m, and 4 m), arch intervals (2.5 m, 3 m, and 3.5 m), diameter of piles (80 cm and 90 cm), and geostatic load above station crest (10 m, 20 m, and 30 m).

Table 3: Comparison between numerical and real GSS with ±5% offset.

Type	Settlement (cm) at Monitoring period of 250 days		
	South axis	Main axis	North axis
Real settlement (deviation -5%)	4.617	5.082	4.322
Analytical settlement	4.68	5.083	4.60
Real settlement	4.86	5.35	4.55
Real settlement (deviation +5%)	5.103	5.617	4.777

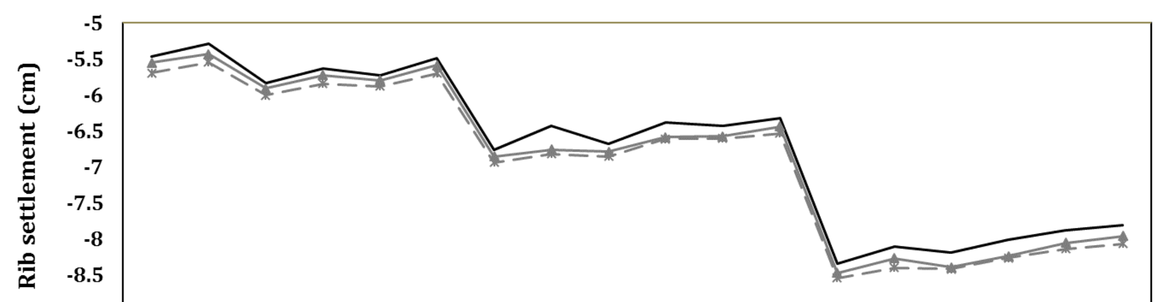
4. Results

Having done the development of the model, the results were classified in two groups of displacements and stresses for pile and rib response, separately. Figures 2 and 3 show the changes of displacement response of piles and ribs after changing the parameters.



Surcharge Eff. length Diameter$_{pile}$	10 m 3 m 80 cm	10 m 3 m 90 cm	10 m 3.5 m 80 cm	10 m 3.5 m 90 cm	10 m 4 m 80 cm	10 m 4 m 90 cm	20 m 3 m 80 cm	20 m 3 m 90 cm	20 m 3.5 m 80 cm	20 m 3.5 m 90 cm	20 m 4 m 80 cm	20 m 4 m 90 cm	30 m 3 m 80 cm	30 m 3 m 90 cm	30 m 3.5 m 80 cm	30 m 3.5 m 90 cm	30 m 4 m 80 cm	30 m 4 m 90 cm
X= 2.5 m	-4.51	-4.28	-5.01	-4.79	-4.87	-4.66	-5.47	-5.15	-5.5	-5.09	-5.14	-5.02	-6.59	-6.32	-6.37	-6.29	-6.1	-6.1
X= 3 m	-4.54	-4.36	-5.04	-4.83	-4.89	-4.72	-5.51	-5.44	-5.53	-5.24	-5.19	-5.05	-6.65	-6.4	-6.5	-6.45	-6.2	-6.18
X= 3.5 m	-4.66	-4.42	-5.09	-4.9	-4.93	-4.79	-5.54	-5.46	-5.55	-5.2	-5.19	-5.09	-6.67	-6.48	-6.48	-6.39	-6.24	-6.23

Fig. 2: Pile settlement during changes in the station depth, the buried length, and the pile diameter.

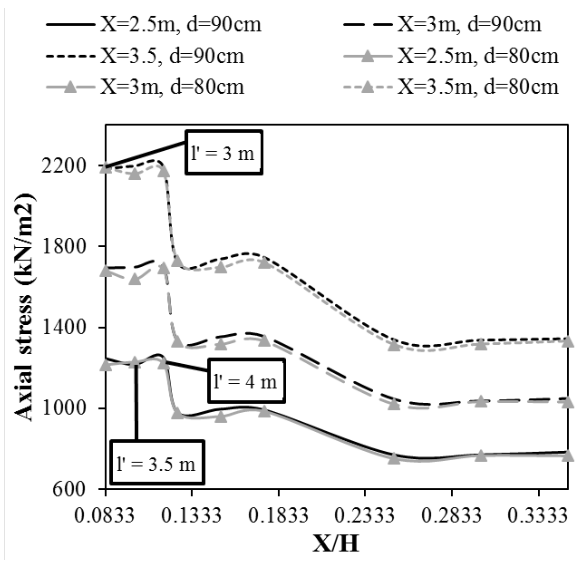


Surcharge Eff. length Diameter$_{pile}$	10 m 3 m 80 cm	10 m 3 m 90 cm	10 m 3.5 m 80 cm	10 m 3.5 m 90 cm	10 m 4 m 80 cm	10 m 4 m 90 cm	20 m 3 m 80 cm	20 m 3 m 90 cm	20 m 3.5 m 80 cm	20 m 3.5 m 90 cm	20 m 4 m 80 cm	20 m 4 m 90 cm	30 m 3 m 80 cm	30 m 3 m 90 cm	30 m 3.5 m 80 cm	30 m 3.5 m 90 cm	30 m 4 m 80 cm	30 m 4 m 90 cm
X=2.5 m	-5.47	-5.28	-5.84	-5.64	-5.73	-5.49	-6.76	-6.43	-6.68	-6.39	-6.43	-6.32	-8.34	-8.11	-8.2	-8.01	-7.89	-7.81
X=3 m	-5.55	-5.44	-5.91	-5.73	-5.8	-5.59	-6.86	-6.76	-6.79	-6.59	-6.58	-6.44	-8.48	-8.27	-8.4	-8.24	-8.06	-7.96
X=3.5 m	-5.69	-5.55	-6.01	-5.84	-5.88	-5.71	-6.94	-6.82	-6.86	-6.61	-6.61	-6.53	-8.55	-8.41	-8.42	-8.27	-8.14	-8.07

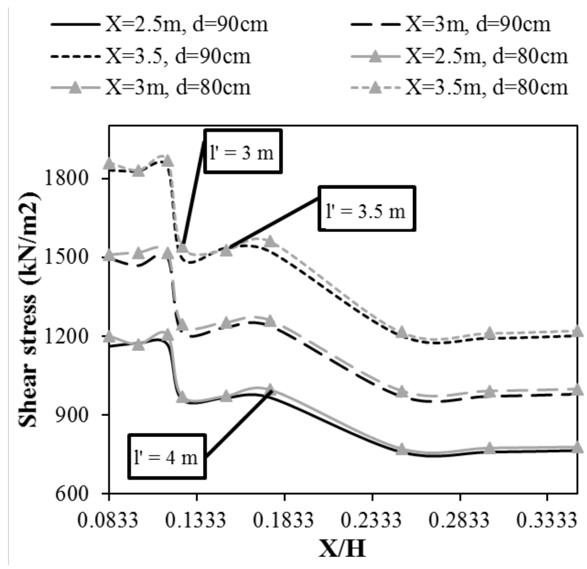
Fig. 3: Rib settlement during changes in the station depth, the buried length, and the pile diameter.

As shown in Figure 2 and 3, the least amount of pile and rib settlement occurred when the surcharge, effective length, pile diameter, and rib intervals were 10 m, 3 m, 90 cm, and 2.5 m, respectively. However, the maximum values were presented by 30 m surcharge, 3 m effective length of pile, 80 cm pile diameter, and 3.5 m rib intervals. The difference between these two limits was 2.45 cm which is remarkable. By increasing the pile diameter, the settlement quantity decreased in all other constant conditions, however, effective length of pile may compensate the values of smaller cross sections. A dramatic drop was observed when surcharge changed from 10 m to 20 m and from 20 m to 30 m. This means the most decisive parameter in pile settlement is the depth of station construction. In addition, an opposite trend can be drawn for the depths more than 10 m in where by increasing pile diameter and effective length, the settlement values decreased, especially when rib intervals are more than 2 m. Also, when installing ribs at closer distances, the amount of settlement increased because the weight of underground structures increased.

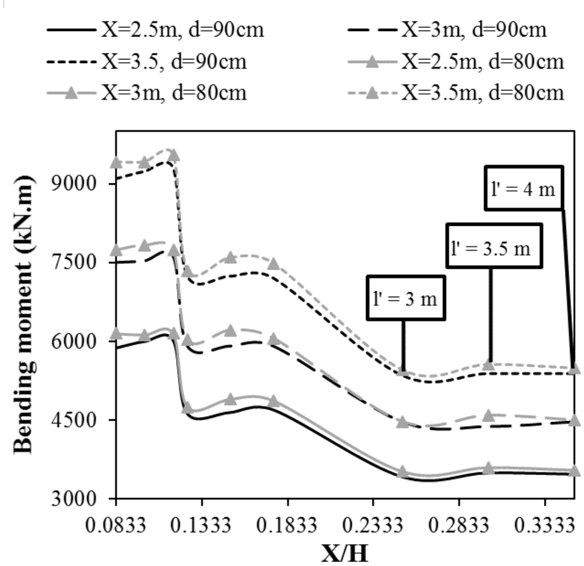
Figure 4 and 5 shows the changes in the axial stress, shear stress and bending moment during change in X/H ratio (rib intervals to depth of surcharge).



a

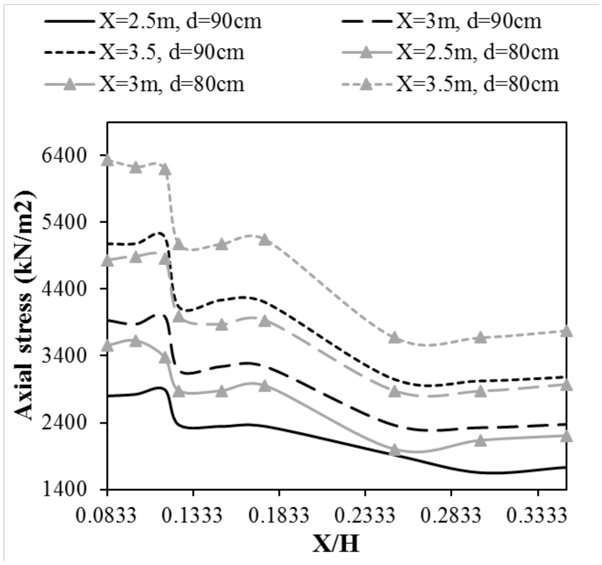


b

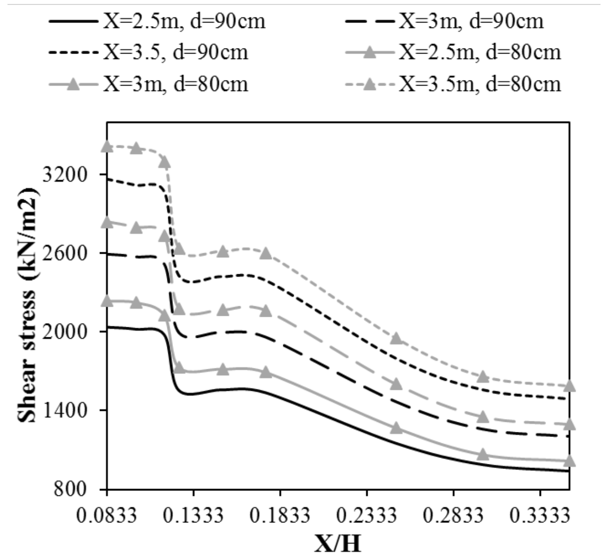


c

Fig. 4: X/H changes with a) axial stress, b) shear stress, and c) bending moment of ribs



a



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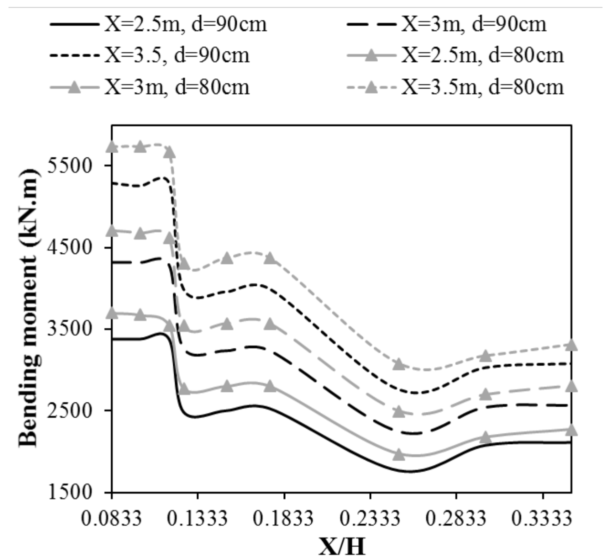

c

Fig. 5: X/H changes with a) axial stress, b) shear stress, and c) bending moment of piles

According to Figure 4, there are some patterns indicates that axial stress, shear stress and bending moment of ribs decrease by increasing X/H ratio, however, farther ribs result in higher axial stress, shear stress and bending moment in piles. In addition, when using thicker piles, the amount of all three parameters reduce. A sudden reduction in induced stresses and moment shows the effect of surcharge reduction from 30 m to 20 m and then from 20 m to 10 m. So, the maximum induced stresses and moments of rib belongs to lower rate of X/H, 90 cm pile diameter, and 3.5 m rib intervals. The effective length of piles has the least impact on stress induction. As a result, the best response can be achieved by the ribs with the minimum amount of stresses and moment, and refers to 2.5 m rib intervals, 10 m geostatic load, 90 cm pile diameter, and 3 m buried length of piles.

Figure 5 shows the same patterns of axial stress, shear stress and bending moment of piles decreased by increasing X/H ratio. With wider space of ribs, axial stress, shear stress and bending moment of piles increase in all other constant parameters.

Reductive trend of all three parameters is the same for ribs when using thicker piles. Similar to rib responses, depth of surcharge has the most impact on stress and moment induction in piles so that the maximum values refer to

3.5 m rib intervals, 3 m buried length of piles, 80 cm pile diameter and 30 m geostatic surcharge. In contrast, the minimum amount of bending moment is assigned to 2.5 m rib distance, 90 cm pile diameter, 3 m effective length, and 10 m geostatic surcharge.

5. Conclusions

In this paper, GSS above a case study of subway station constructed on the basis of CAPS method was investigated numerically to evaluate the efficiency of nominated finite element code. The model development was carried out by change in three four input data (pile diameter, buried length of pile, rib intervals, and depth of station construction). Therefore, the following conclusions will be drawn:

- 3D PLAXIS Tunnel attained an accuracy of ±5% in estimating GSS which is really acceptable.
- X to H ratio plays a leading role in the amount of stresses, moments and displacement of rib and pile. That is by increasing X/H the amount of stresses and moments, and vertical displacements decrease in both piles and ribs.
- Effective length of piles have less impact on the responses, however, the most decisive parameter is the soil column above the tunnel crest.
- The best rib intervals achieves when installing at farer distances, however, this parameter has a little influence on the response of settlements.

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