Performance of CAPS Method Considering its Interaction with Adjacent Structures – The Q7 Station of Tehran Metro Line 7

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ABSTRACT: *In recent years, innovative underground construction techniques have been extensively utilized for many purposes in Iran. Using construction methods such as CAPS (concrete arc pre-supporting system) in the case of nearby special structures is regarded as a valuable technique. CAPS technique applied in this work is a supporting system implemented in urban areas, where the excavation-induced distribution of the soil settlement is considerably lower compared to other sequential excavation (SEM) methods. Our case study is Q7 station, which is an intersection station in Tehran metro line 7 located near Tohid Twin Tunnel and Gardoon Tower. Based on investigations carried out and presented in this paper, CAPS demonstrates an excellent performance and serviceability for structures located within congested urban areas. Q7 station was modeled using FLAC3D code. To ensure the accuracy of our model, monitoring data were compared with the numerical results. By performing sensitivity analysis on the shear parameters of the rehabilitated soil (c, φ) and the distance between beam elements (λ factor), we observed that increasing the shear parameters of the soil mass decreases the vertical displacement of the ground. The optimum value for the λ factor was estimated in this work based on the Rankin criteria for Gardoon Tower (a 20-story building) and Tohid Twin Tunnel.*

KEYWORDS: Surface settlement, CAPS method, Q7 station, λ factor, FLAC3D, numerical simulation

SITE LOCATION: Geo-Database

INTRODUCTION

Construction of large-span underground spaces, such as subway stations, in urban areas is commonly facing unforeseen problems affecting both the schedule and cost of the project. In this regard, the most important problem associated with construction of the substructures is the unexpected displacement under adjacent structures due to underground excavation; an issue more frequently occurring in soft soils and urban areas. Under such conditions, instrumentation, pre-construction, and construction inspection are the methods commonly used to prevent damage. The stability of the large-span excavation and relevant risk management of adjacent buildings are among the most important issues in the construction of a subway system. Since Navab Highway is one of the important areas above the Q7 station construction, accurate monitoring for this area is implemented. The available data from instrumentation can be used to verify the numerical simulation results (Ran et al. 2011, Liu et al. 2012). The Concrete Arch Pre-supporting System (CAPS) is one of the state-of-art methods used in the construction of underground stations, as it has numerous advantages including lower construction cost, fast excavation, ease of design and construction, and minimum soil mass disturbance leading to lower surface settlements. Analysis of the surface settlement using this method on one of the subway stations of Tehran Metro (Sadaghiani et al 2008) reveals the excellent efficiency of this method for underground excavation under specific conditions such as low overburden, congestion in urban areas and traffic flow disruption (Sadaghiani et al 2008, Sadaghiani et al 2010). In another project performed by Sadaghiani (2010) on Mellat station using Plaxis3D, it was found that in large-span underground spaces with low overburden, the surface settlement increases by decreasing the height of rib elements. Generally, the shape of the surface settlement induced by underground tunneling in transverse and longitudinal sections is an important issue investigated often by

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empirical methods (Peck 1969, Atkinson 2007, Guglielmetti 2008, Chakeri et al. 2014). Numerical models using finite element (FE) or finite difference methods (FDM) are among the recently applied approaches in underground space simulation. One of the advantages of the numerical methods is their ability to simulate complex situations; however, the results must be verified (Lambrughi et al. 2012). Other methods used to analyze surface settlement are empirical methods that do not consider the interaction between the underground tunnel and the nearby constructions. Recently, Mirhabibi et al. (2012) studied the interaction between a tunnel and surface structures through numerical simulation. They found that the rigidity and geometry of nearby buildings, soil conditions, and excavation technique are the main factors that influence the behavior of the surface structures. Their conclusion was confirmed by field data and centrifuge tests (Mirhabibi et al 2012, Mair 2013). In addition, Zhang et al. (2015) presented a damage assessment by monitoring a two-story building, simulating its frame and considering its interaction with underground space. Using a 3D model, they estimated the additional stress induced in the surface building by the underground tunneling (Zhang et al. 2015). Construction of the large-span substructures in urban areas has always been considered an important problem in tunnel engineering. Liu (2008) presented a pre-supporting system called “tunnel column method” - which is generally similar to the CAPS method, though different in the type of the pre-supporting elements used (Liu et al. 2000). Another innovation used in underground structures is the central beam column (CBC) method, a recent method with more rigidity as compared to CAPS that was applied by Valizadeh (2012) for Tehran Metro Line 3. The result of this study showed that this method is useful for certain conditions in urban areas. The sequential excavation method (SEM) has fundamental principles such as exploitation of the strength of native soil mass as the main component of tunnel support (Valizadeh Kivi et al. 2012).

SITE CONDITIONS AND GEOLOGY OF Q7 STATION

Tehran Metro Line 7 consists of two phases. The first phase has a 13 km tunnel in an east-west direction and the second phase has a 14 km tunnel in a north-south direction. The line starts from Thakhti stadium located in the southeast area of Tehran, and continues in the east-west. It changes its route to a south-north direction parallel to Navab Highway (Figure 1), and Q7 station intersects with line 2 of Tehran Metro (Navab station). Tunnel excavation was done by two Earth Pressure Balance (EPB) Tunnel Boring Machine (TBMs). This line involves 25 stations constructed using cut-and-cover, CAPS, SEM, top-down construction, and diaphragm systems. The main part of this phase is located downtown where there is constantly traffic congestion; therefore, the cut-and-cover method was not suitable.

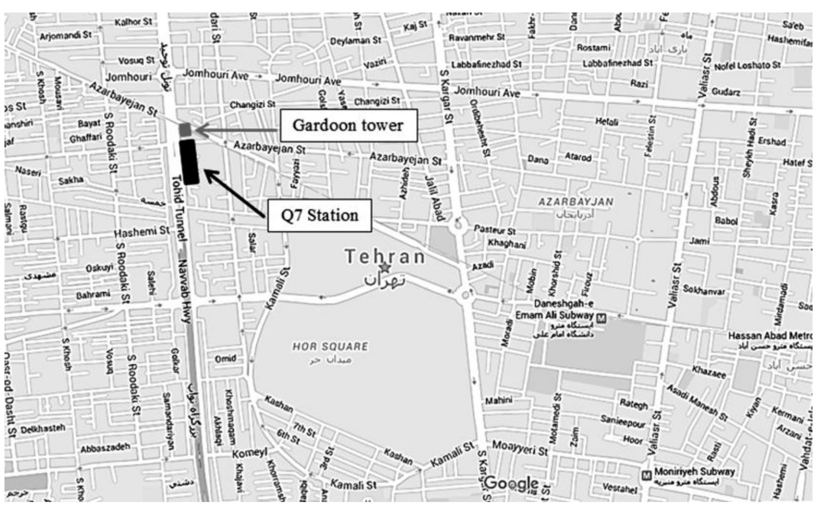

Figure 1. Location of the Q7 station.

The geotechnical soil classification and depth of the layers are shown in Figure 2 and Table 1. The data were obtained from four boreholes near the station. Boreholes were drilled using three rotary drilling machines through which continuous core drilling and SPT tests were conducted. In situ tests such as pressuremeter, plate load test, and in-situ direct shear were also carried out in the boreholes. The groundwater table in the vicinity of the project was below the base of the station; therefore, dry excavation was executed. The overburden layer in this project was about 29 m.

The Q7 station is located near the Tohid Tunnel, which is a major transportation underpass in the west of Tehran city. Main structural components of the Tohid Tunnel include a thick lining and lateral piles, with 1.5 m diameter and 4 m spacing. Total area and length of these tunnels are about 305 m^2 and 4272 m, respectively. The subway stations are located below Tohid Tunnel and parallel to it (Fig. 3). The Gardoon Tower, which is a 20-story commercial building, is located on the right-hand side of the station. Thus, this substation has two important adjacent structures. The Gardoon Tower and Tohid Tunnel, which were constructed in 2006-2008 while the Q7 station was built in 2015 (Ashrafi 2010).

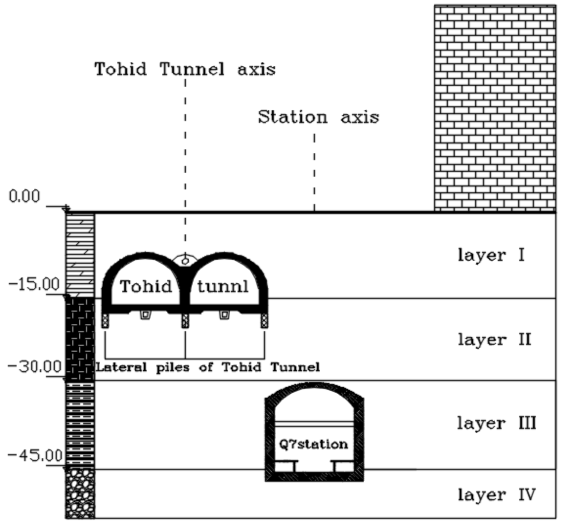

Figure 2. Geotechnical profile of the Q7 station.

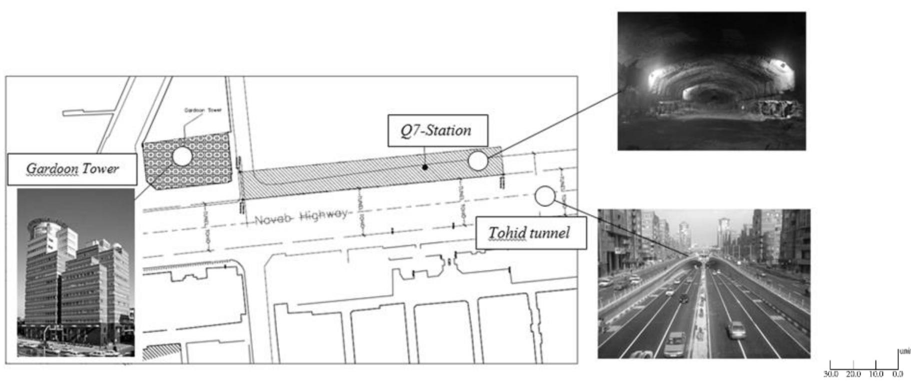

Figure 3. Location of the Q7 station with Tohid tunnel and Gardoon tower.

Table 1. Characteristics of the soil in Q7 station.

Layer .No	Unified classification	Depth	γ_d Unit weight	C_U^* Cohesion	Φ_U^* Friction angle	v Poisson's ratio	E Young modulus
Unit		m	kN/m^3	kPa	Deg		MPa
I	GM	0 - 15	17.50	12.5	33	0.35	100
II	GC	15-30	18.00	20	35	0.32	150
III	GM	30-45	18.50	30	37	0.30	190
IV	GM	>45	19.00	38	39	0.30	280

Geologically, the Q7 station is located within the Quaternary alluvial zone. As a widely spread soil stratum in Tehran, this zone is classified into four formations identified as I, II, III, and IV. The structure is located in zone III. The sediment along line 7 of the Tehran Metro is composed of silt, gravel, sand, and rubble. There are some active faults such as the Firouzeh castle and Rey near the project and the seismic activity of these faults is important for the dynamic analysis of the station (Ashrafi 2010).

CAPS METHOD

As construction of a subway station in urban areas is accompanied by deformations in the soil mass, it is essential to protect the pre-existing structures and underground utilities from potential damage. Thus, it is particularly important to know the effect that an excavation may have on adjacent structures as well as their interaction during subway construction. The cut-and-cover method in tunnel construction offers an alternative approach to underground construction techniques. This method involves constructing the tunnel structure in a trench-type excavation. However, in congested urban areas, cut-and-cover construction can be very disruptive as access to the ground surface over extended areas is difficult. Forepoling and grouting are other methods used in this situation, but they require special equipment and are costly. Therefore, the method used for the Q7 station was a method that does not require traffic deviation and the ground distortion is kept to a minimum (Sadaghiani and Taheri 2008). This method – called concrete arc pre- supporting system (CAPS) – is a low-cost system compared to other methods and does not need special expertise. The method involves sequential and pre-supporting systems, where first an initial 3D frame is built around the station, and then the soil is removed by Sequential Excavation Method (SEM). In this case study, two critical structures near our project existed including the 20-story high Gadroon Tower and the Tohid Tunnel, a twin tunnel connecting Navab Highway to Chamran Highway. The construction processes involved two phases, as explained below (Figures 4 and 5):

(a) Initial state of the soil mass;
(b) Excavation and construction of a 4 m wide forward gallery (horseshoe tunnel) in top floor (Thicket hall);
(c) Two small access galleries are excavated from the existing underground tunnel (initial tunnel) towards the side walls and continued along the station length on two sides;
(d) Excavation and construction of the rib elements (A horseshoe shape arc gallery on top) and the primary piles on the sides are excavated at two meter spacing along the station cross section (these piles were located below the toe of ribs and transmitted the ribs load to the ground). Using three small longitudinal access galleries, arc and pile are reinforced and concreted; these concrete arcs make a rib over the main tunnel (access galleries are plugged at the end);
(e) Excavation of the crown, side-drift and inert parts of the soil using the sequential excavation method (SEM) and stabilization of the area underneath by applying a layer of mesh and shotcrete;
(f) Complete the excavation of the ticket hall floor and build the secondary piles, that supports the slab of the top floor (these piles are located adjacent to the primary piles);
(g) Excavation of the platform area and building the slab of the top floor; and
(h) Final lining of the station.

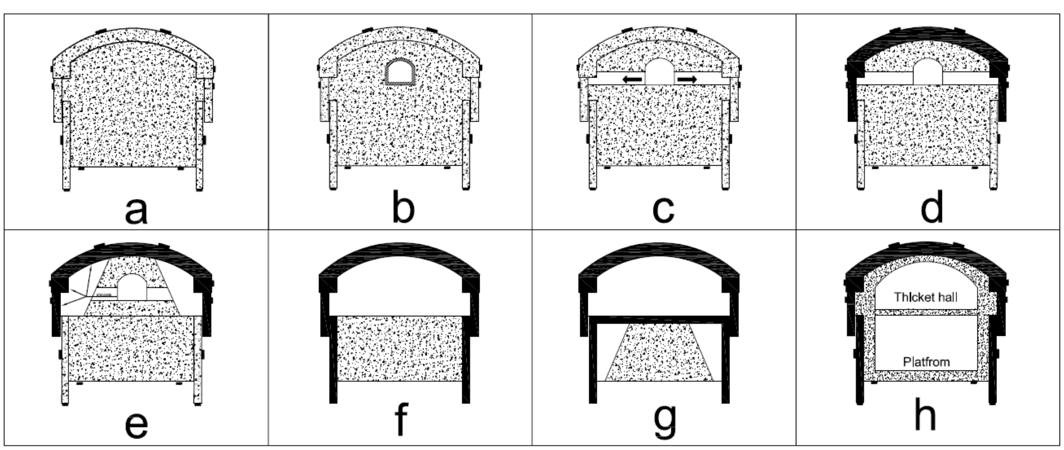

Figure 4. Schematic stages of the station construction.

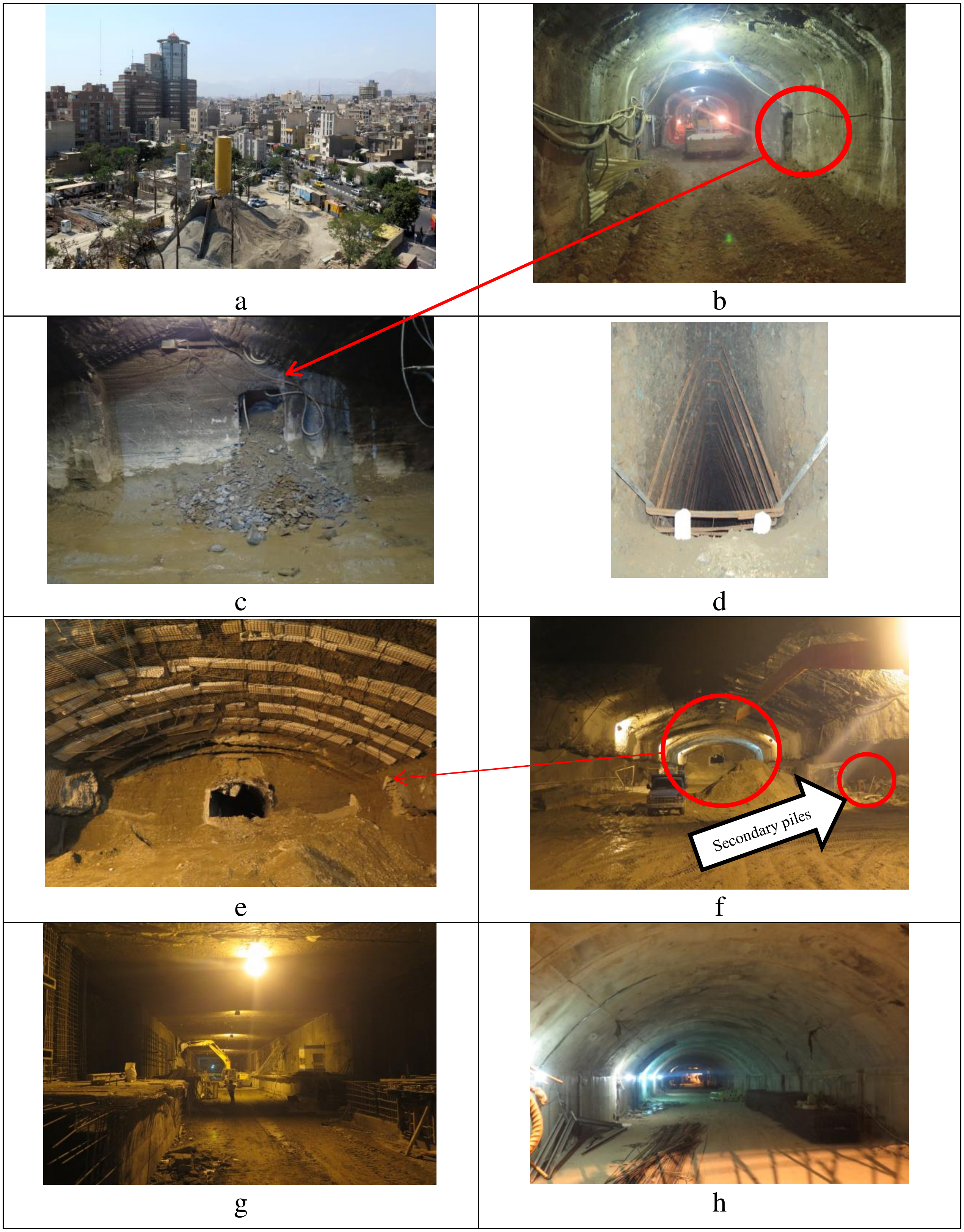

Figure 5. Actual stages of the station construction.

NUMERICAL MODELING

For the model, a 3D finite difference code was used(FLAC3D ver3. 0). To reduce the effect of virtual borders on the results, the dimensions of the model were selected as 6×L (Longitudinal), 6× L (Transverse), and 5× L (Vertical), with L being the width of the substation. The results of a parametric analysis indicated that these dimensions diminish the boundary effects (Mroueh et al 2008). The lateral borders along the faces of the model were simulated with rollers, while at the bottom were simulated with fixed support. The initial simulation of the model and the boundary are illustrated in Fig.6. Only short term loading is considered in the analysis; i.e. the consolidation process is not taken into account. Therefore, the unloading condition is undrained. Based on the actual parameters of the structural elements, the structure of the support system was modeled as beam and pile elements in FLAC. The FLAC3D model includes 89,000 grid cells. The 3D view of the CAPS method around the Q7 station is presented in Fig 7.

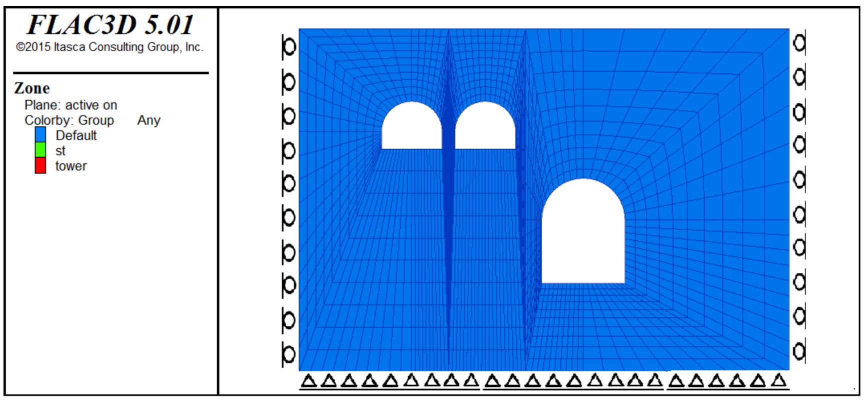

Figure 6. Initial sumulation of the model and the boundary condition.

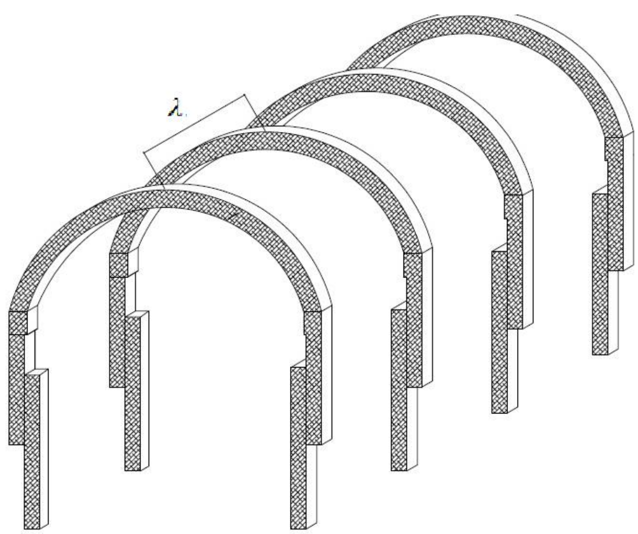

Figure 7. Schematic 3D view of CAPS element.

All ribs and piles in the numerical model were installed before the main excavation was performed. Because the Q7 station involves two floors, the excavation sequence for the ticket hall and platform floor was modeled exactly as constructed in the field.

To verify the results of the numerical model, we used the settlement measurements. Instruments were placed at or near the ground surface, building, or utilities to be monitored for displacement where measurements are commonly performed using traditional surveying methods. These instruments were located perpendicular to the station axis, with three points in each section. The detail of installation and the location of the monitoring equipment is shown in Figures 8 and 9.

To develop our model, the elasto-plastic Mohr-Coulomb failure model was used, with parameters presented in Table 1. The supporting systems, such as ribs and piles, are assumed to be elastic with the parameters summarized in Table 2.

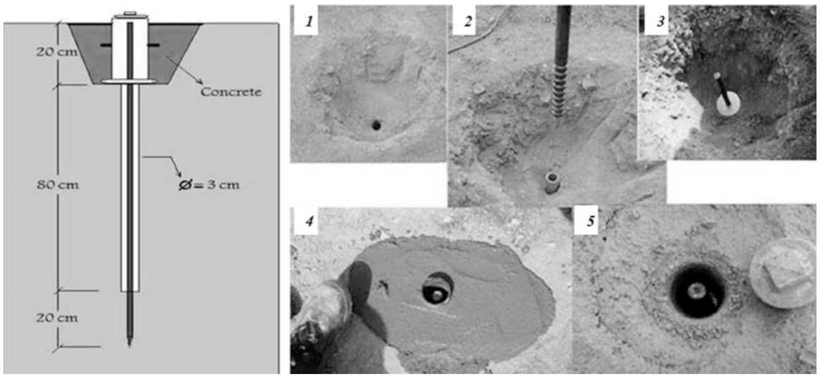

Figure 8. Installation of monitoring equipment 1-Making a ditch on the ground surface(depth of about 20 cm), 2- locating the pod in ground (depth 80 cm), 3- setting the pin in the pod,4- covering near the pin head by concrete, 5- positioning the protective cap on it).

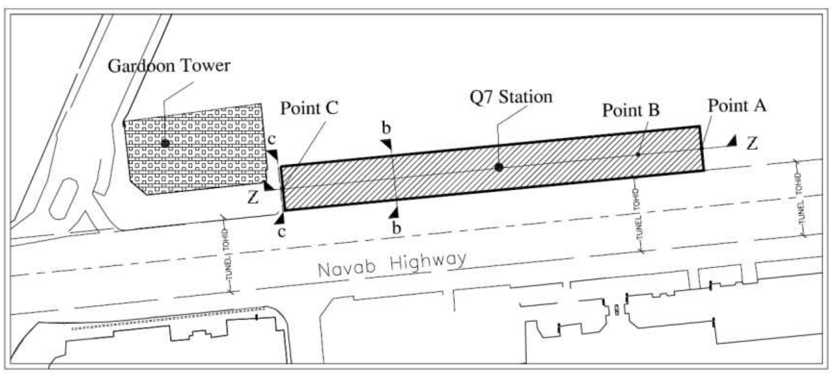

Figure 9. Layout of measuring surface settlement points around the Q7 station.

The traffic load of the Navab Highway was simulated by a uniformly distributed load of 20 KPa and the load of the Gardoon Tower was modeled using a uniform load on a surface applied to a rigid plate. The buildings were simulated

Table 2. Pre Support elements characteristics in Q7 station.

Support element	Young modulus (GPa)	Poisson ratio	Unit weigh(KN/m^3)	28-day Strength (MPa)
Beam(rib)	20	0.20	24.00	25
Pile	20	0.20	24.00	25
Shotcrete	20	0.20	23.00	21

by an elastic beam on the surface of the model and simulated as linear Timoshenko beam elements. Each surface beam had an equivalent moment of inertia (I) and thickness (t) representing the corresponding building (the thickness and Young's modulus of the mat foundation were assumed to be 1.5 m and 2×10^7 KPa, respectively) (Mirhabibi et al. 2012, Katebi et al. 2013). The Tower loading was applied by a uniform load of 140.0 KPa, which is the sum of the respective 5.0 kPa and 2.0 kPa dead and live loads for each floor.

Based on the numerical modeling and empirical method, and considering that the Tohid Tunnel was located within the influence zone of the Q7 station, its effect was incorporated in our modeling (Mair 2013). For the simulation, first the soil of the Tohid Tunnel was removed and then analyzed the soil mass around the station. In the next step, the rib and pile elements were modeled, then the excavation of the substructure in the ticket hall and platform were performed. Our model was developed based on the progressive excavation pattern, through which the station was modeled stepwise and the sequential excavation was performed with a 10 m lag between the top and the transversal section. The reference pins which were 120 cm long were arranged in three point arrays with 10 m longitudinal and 8 m transverse spacing installed on the ground surface. Surveying in this project could attain pin position with accuracy of

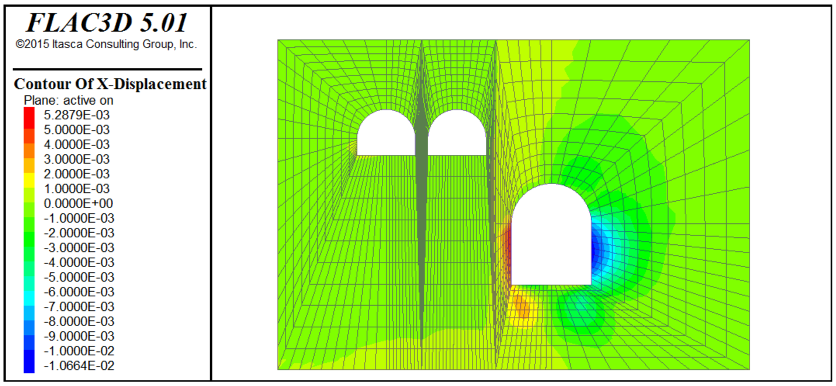

Figure 10. Horizontal displacement in soil mass after station excavation.

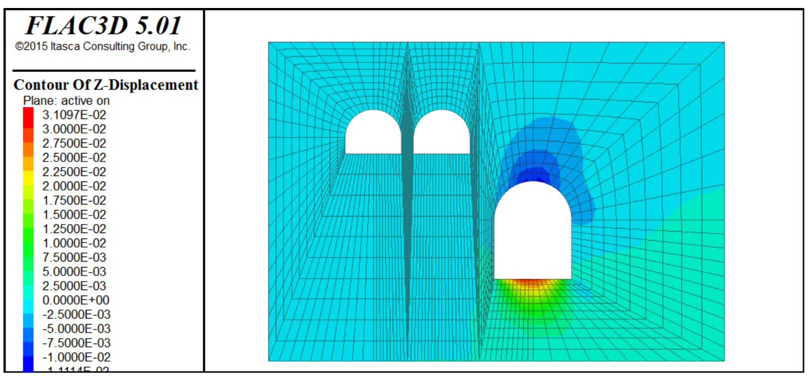

Figure 11. Vertical displacement in soil mass after station excavation

1 mm. To eliminate the effect of pavement deformation on results, the pins were installed about 40 cm below the street level. The displacements around the station excavation are shown in Figs. 10-11. In order to validate the results of our numerical model, vertical displacements were compared in FLAC3D with the measured data. The comparisons are shown in Fig.12 (the x axis is the distance to point C in Fig 9). As shown in the figure, the measurements are consistent with our simulation.

SENSITIVITY ANALYSIS OF THE VERTICAL DISPLACEMENT

The following parameters were selected in order to perform sensitivity analysis and to investigate their effect on surface settlement;

i. Distance between ribs (λ factor)
ii. Soil cohesion (C)
iii. Soil friction angle (φ)

To do this, a sensitivity analysis was performed and the variation of each parameter *vs*. the vertical displacement on the surface was plotted. When the underground excavation is constructed below the surface structures, in order to prevent large ground settlement, it is common to improve the soil's mechanical properties. One of the common

methods for soil improvement is deep soil grouting. This method aims to fill voids in the ground resulting in increasing the soil's properties such as C & φ and then decrease soil deformability due to tunnel excavation. So, mechanical properties of soil (C & φ) were selected to perform sensitivity analysis.
The output of this analysis is shown in Figs. 13, and 14. Based on our modeling, the shear strength parameters of the soil (c and φ) indicate an inverse relation with the vertical displacement. In other words, as the shear strength increases, the surface settlement decreases.

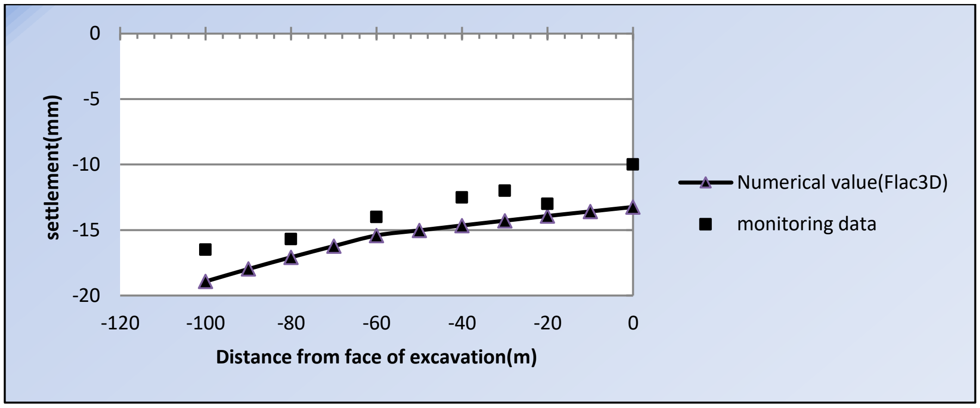

Figure 12. Monitoring & numerical results in longitudinal axis (for section ZZ in Figure 9).

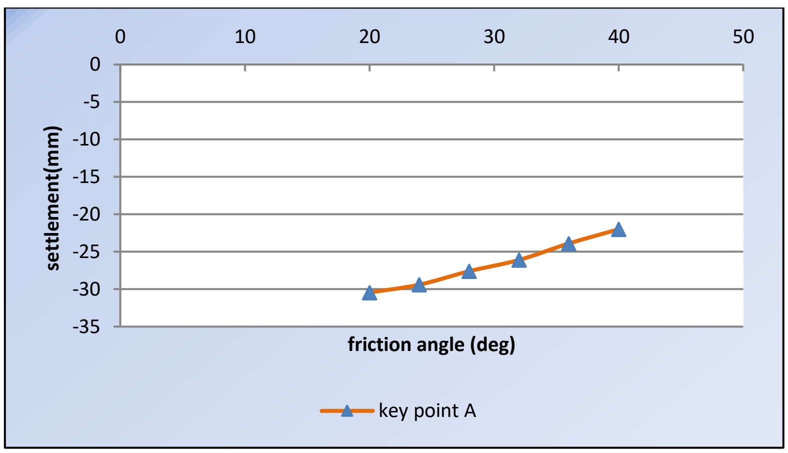

Figure 13. Effect of φ (friction angle)on the surface settlement (for point A in Figure 9).

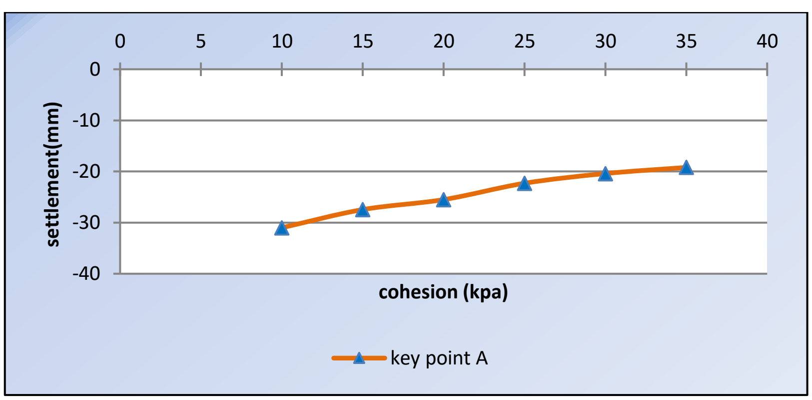

Figure 14. Effect of cohesion C on the surface settlement (for point A in Figure 9).

The spacing between the rib elements (λ value) is one of the most important geometry parameters determined by the project designer. Selection of the optimum value directly affects the final cost and time of construction. Our results show that an increase in the λ factor causes an increase in the surface displacement. The relationship between S_{max} (at points A, B, and C of Fig 9) and the λ factor is shown in Fig.15. This relationship, that is calculated based on section ZZ, clearly indicates that by increasing the λ factor, the vertical settlement increases.

One of the objectives of this work is to determine the optimum value of the λ factor by considering its relationship with the vertical displacement. Using these key points (i.e., A, B and C, shown in Fig 9) for the station construction, we concluded that the surface settlement and the λ value have a direct relationship; thereby an increase in the λ value causes an increase in the vertical deformation. In Fig.16, the effect of the λ parameter on the transverse settlement is shown.

DISTRIBUTION OF THE HORIZONTAL DISPLACEMENTS

The horizontal displacement of the soil profile in section CC was plotted based on the numerical results (see Fig 9) and is shown in Figs. 17 and 18. The location of Gardoon Tower and Tohid Tunnel are also shown. As shown, the horizontal displacement has a great variation near the crown station. This displacement increases gradually with depth and reaches its maximum at the connection of the rib to the pile, and decreases thereafter. On the other hand, the variation of the horizontal displacement with depth in the left side of the station indicates that the Tohid Tunnel has a negligible effect on the horizontal deformation. The maximum deformation was concentrated at the rib-pile connection point, and then, deformation reduces to zero with depth. Considering the real connection conditions of the rib to the pile, this connection was simulated as a rigid point.

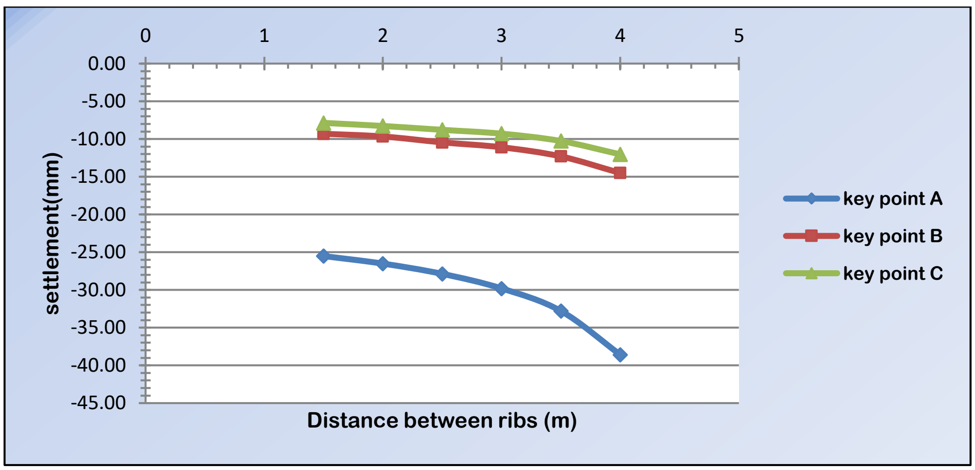

Figure 15. Effect of the λ value on the surface settlement.

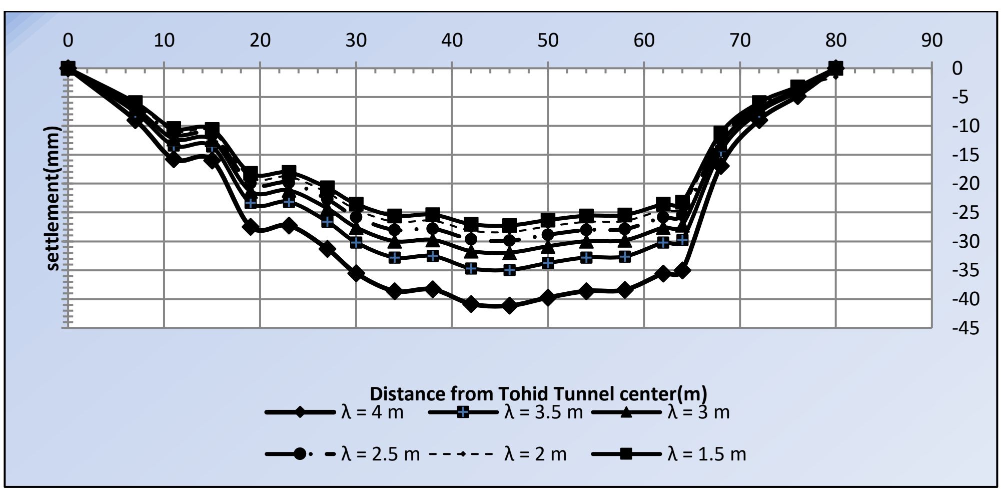

Figure 16. Sensitivity analysis of the λ factor for section CC.

Assessment of soil disturbance caused by tunneling construction on surface or subsurface structures is one of the most important aspects of tunneling in soft ground. One of the methods for damage classification, presented by Chiriotti et al. (2008), is based on the vulnerability index (I_V). This method is based on the history of the building, engineering judgment, and how far the building conditions are from being is optimum and perfect condition. The vulnerability index is obtained through an analysis of the collected information on the building condition investigated by engineering judgment.

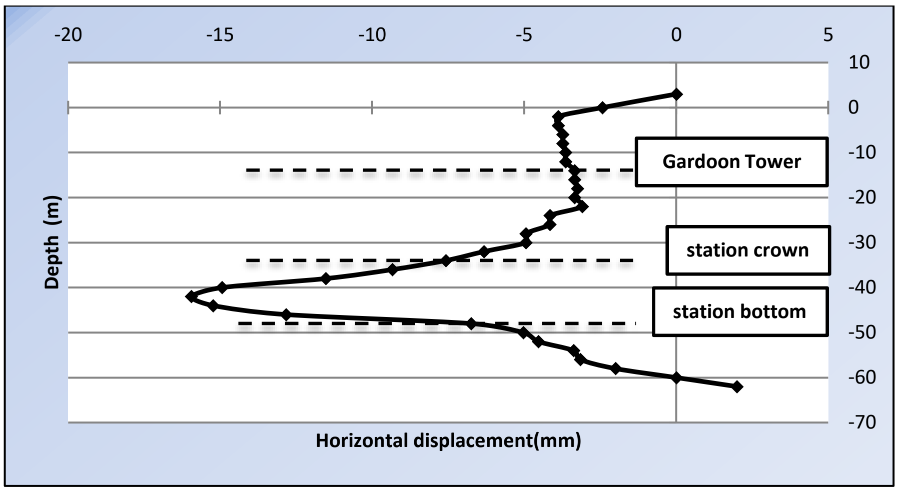

Figure 17. Horizontal displacement in section cc (right side of the station based on numerical analysis).

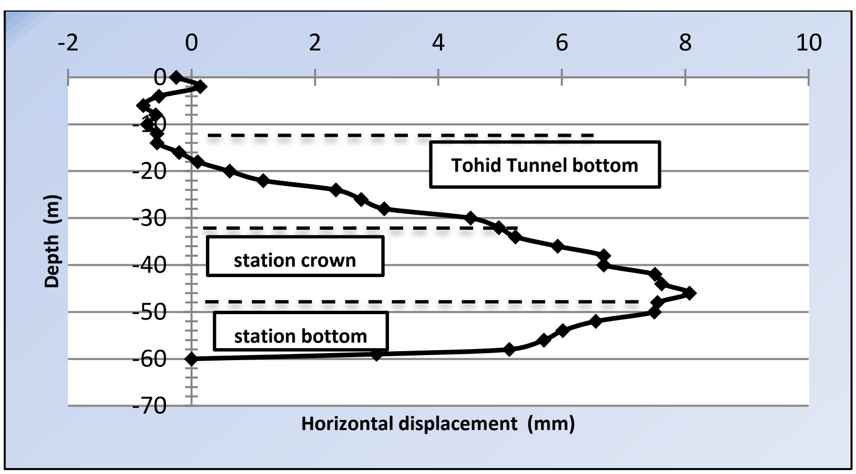

Figure 18. Horizontal displacement in section CC - left side of the station (based on numerical modeling).

The vulnerability index (IV) can be classified into 5 categories with different degrees of severity, using the following normalized scale, 1 to 100: 0–20, negligible; 20–40, low; 40–60, slight; 60–80, moderate; 80–100, high. According to the vulnerability index in our project, the long-term and short-term I_V values are equal to 64 and 14 for the Gardoon Tower and the Tohid Tunnel respectively. This score (vulnerability index) illustrated that the effect of the Q7 station excavation on the Gardoon Tower is moderate and on the Tohid Tunnel is negligible.

Since the level of damage is not determined using Chiriotti's assessment, the Burland method (1977) was used that is based on parameters quantifying risk. The most important factor in this approach is the deflection ratio (Δ_{max}/L) which is related to the maximum tensile strain ε_{max} under the surface structure. Here, we can either control ε_{max} or limit the maximum vertical displacement (approximately 30 mm) considering the building quality and serviceability condition of the tower (Guglielmetti et al. 2008).

Regarding the sensitivity of the Gardoon Tower (commercial complex), the *Aesthetic* damage level was selected that involves slight cracking of the structure affecting the internal walls and their finishes. Structural damages are related to cracking or excessive deformations of the bearing structures and can lead to partial or total collapse of the building. Based on the damage degree and Burland classification, the crack width is limited to 0.1 mm and the tensile strain is 0.05 %. Other parameters of a building in Burland's classification are explained below (Guglielmetti et al. 2008) (Fig.19):

S_{max}: maximum vertical settlement, ΔS_{max}: maximum differential or relative settlement, α_{max}: maximum angular strain (sagging when positive; hogging when negative), λ_{max}: maximum angular distortion, ω: tilt (rigid body rotation of the whole superstructure or a well-defined part of it), and Δ_{max}: maximum relative deflection (max displacement relative to the straight line connecting two reference points with a distance L).

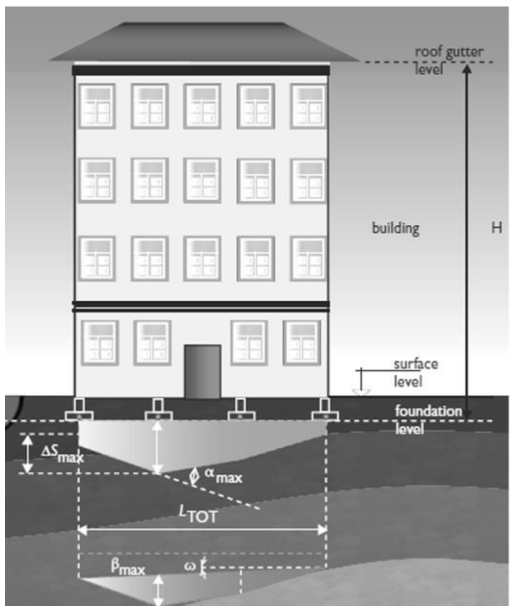

Figure 19. Related parameters in Burland Damage assessment (Guglielmetti et al. 2008).

Rankin is another (Guglielmetti et al. 2008, Chapman et al. 2010) classification that relates the damage due to differential settlements of isolated foundations, the angular distortion β and maximum settlement S_{max}. The control parameters found in the Rankin damage classification used in the present study are β_{max} that is in the range of 1/500 to 1/200 and S_{max} that is approximately 25 mm (See Table 3). Since the vertical displacement under the Tohid Tunnel is very low, we don't consider the effect of the substation excavation on the twin Tunnel. It is noted that the vertical displacement below the tower foundation had been derived from numerical modeling.

According to the Rankin approach, the maximum differential displacement ΔS_{max} under the tower is about 25 mm and β_{max} by 30 m width of the building is approximately 1/1250. This value is less than the allowable values (Tables 4 and 5). So, this result shows that the Gardoon Tower building is not excepted to experience problems related to the station excavation.

Finally, the maximum horizontal strain was determined based on the numerical model to be about 0.0036, which is less than the allowable value (0.005) in Burland's classification. Another method in damage assessment is the code of foundation design (GB50007-2002) which is based on the allowable differential settlement. The differential vertical displacement must be less than 0.003L (L is the width of the building) (Huang et al.(2002)). Considering these criteria for our building dimensions (30m ×53 m), the maximum allowable differential settlement must be 90 mm; these criteria determined the optimal value for our case study (See Tables 4 and 5).

Table 4. Summary of various approaches to damage assessment for Gardoon tower.

Damage assessment classification	category	Description of typical damage	Control parameter	Allowable value	Calcul. value	Status
Burland	aesthetic	Hairline cracks	Tensile strain	0.05(%)	0.0036 %	ok
Rankin	aesthetic	Possible superficial damage which is unlikely to have structural significance.	β_{max}	1/500-1/200	1/1250	ok
			S_{max}	10-50 mm	25 mm	ok
GB-20005 code	slight	High compressibility soil	ΔS_{max}(0.003L)	159 mm	25 mm	ok
			ΔS_{max}(0.003d)	90 mm	25 mm	ok
Functional	Moderate	Expected superficial damage to buildings and expected damage to rigid pipelines.			1/200–1/50	50–75
Serviceability And structural	High	Expected structural damage to buildings and damage to rigid pipelines; possible damage to other pipelines			>1/50	>75

Table 5. Summary of various approaches to damage assessment for Tohid tunnel.

Damage assessment classification	Category	Description of typical damage	Control parameter	Allowable value	Calcul. value	Status
			East Tohid tunnel			
Burland	aesthetic	Hairline cracks	Tensile strain	0.05(%)	0.043 %	ok
Rankin	aesthetic	Possible superficial damage which is unlikely to have structural significance.	β_{max}	1/500-1/200	1/2145	ok
			S_{max}	10-50 mm	9.40 mm	ok
GB-20005 code	slight	High compressive soil	ΔSmax(0.003L)	37.8 mm	25 mm	ok
			West Tohid tunnel			
Burland	aesthetic	Hairline cracks	Tensile strain	0.05(%)	0.012 %	ok
Rankin	aesthetic	Possible superficial damage which is unlikely to have structural significance.	β_{max}	1/500-1/200	1/6631	ok
			S_{max}	10-50 mm	4.58 mm	ok
GB-20005 code	slight	High compressibility soil	ΔS_{max}(0.003L)	37.8 mm	5.41 mm	ok

OPTIMIZATION METHOD FOR THE λ PARAMETER

Project cost and risk management are two important aspects of underground construction in urban areas, so the findings of this study can be useful to select the optimum dimension in design. In this study, the optimum value for λ value was investigated. Note that all parameters that may have an effect (e.g., geometrical situation, underground water table, the interaction between soil and structures, etc.) must be investigated carefully. The Rankin criterion was selected for determination of the λ value since the Gardoon Tower has a concrete frame structure and the default assumption in Rankin classification is considered for frame structures. The Gardoon tower is an important structure that, based on Rankin category, falls in the aesthetic class so that the maximum vertical displacement must be limited to 30 mm[(10+50) /2] = [30]. To find out the threshold value for the λ factor, a sensitivity analysis was implemented. Fig.20 shows the limitation of the vertical displacement for determination of the λ factor based on serviceability of Gardoon Tower. As shown in the last section, the effect of the station on the twin tunnels is shown to be negligible. On the other hand, the Gardoon Tower plays a more important role in the determination of the λ factor. As shown in Fig 20, the optimal value was obtained for the λ factor to be about 3 m. This investigation revealed that it is possible to determine the optimum value for the dimension parameters of the project by considering the interaction between large-span underground spaces and adjacent structures.

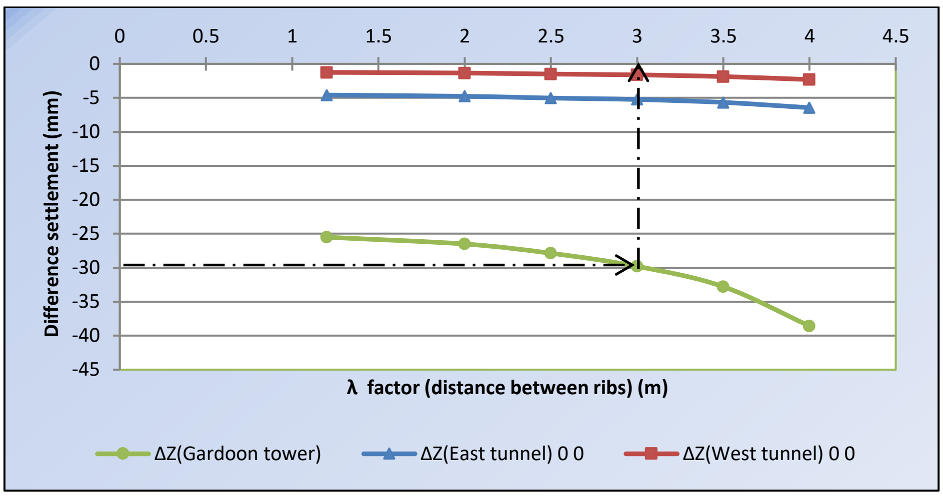

Figure 20. Determination of the optimal value for the λ factor.

CONCLUSION

The most important findings of this work can be outlined as follows:

1. The CAPS method is an innovative construction technique that is a promising candidate for constructing large-span underground structures, especially in highly urbanized areas.
2. In damage evaluation cases, the Rankin damage classification is a reasonable method especially when the surface structures have a frame skeleton.
3. The connection point of rib to pile element is a deficiency of the CAPS system since its stiffness is lower compared to other sections and the vicinity of high surcharges must be considered as a critical point. The height of secondary piles in CAPS system can lead to large horizontal displacement, and must be carefully considered in the design phase.
4. Evaluation of various damage assessment indicators showed that the surface structures are more vulnerable than the underground structures due to substation construction using the CAPS method.
5. In cases that the CAPS system leads to high surface deformation adjacent to critical structures, it is recommended that soil stabilization methods such as grouting are used to increase the shear strength of soil.
6. 3D FDM simulation is a useful tool for design optimization of CAPS method system in substructures by considering adjacent structures.
7. The condition of the adjacent structures and the soil influence the optimum distance between pre-supporting elements in CAPS method. The quantity and size of these elements are directly related to costs and construction schedule of the project.

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