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Full Length Research Paper

Subway station seismic consideration based on geotechnical study: A case study approach

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Iran is a vast country with 1.6 million square kilometer area and more than 70 million inhabitants. Tehran, the capital of Iran, together with its satellite townships is home to more than one sixth of the country's population which make over 13 million, and mostly make motorized daily journeys. This work is based on the case study geotechnical investigation of a metro station in Tehran. The access gallery and tunnel inter-section problem is very sensitive and dangerous in critical zones and the necessity of the provision of tunnel lining has been discussed in this paper. This study is a concise case study which highlights the important soil parameters for designing of deep and shallow tunnels and substations. It shows the critical area for performance of tunnel intersection of and access galleries and the method to avoid collapse of the tunnel and access galleries while tunneling with a high safety. If boring of the access gallery and tunnel station, is made at the same time or the boring of the access gallery is carried out before the tunneling lining, the tunnel will be overburdened with loads leading to weakening of the access galleries crown, and its immediate collapse. One of the objectives of this paper is defining the seismic modeling of tunnel. Specifying the differences in seismic modeling using two softwares, PLAXIS and FLAC is another aim of this paper.

Key words: Tunnel, geotechnical investigation, seismic modeling, finite element method, finite difference method, Tehran, Iran.

INTRODUCTION

Theoretical and experimental investigations were carried out for the design of the tunnel and metro substation (Haftetirr) in Tehran, Iran. This case study shows the critical areas of the tunnel and the stability problem faced by the access gallery and identifies the method to avoid the collapse of the tunnel and access galleries enabling performing shotcrete and lining.

There are critical and unstable zones if the boring of the tunnel, station and accessory galleries are carried out at the same time. The method used to prohibit the collapse of tunnels was to stop boring of the access galleries in order to carry out boring the tunnel station quickly and the progress of the tunnel with higher safety. This paper presents the geotechnical investigation including the geological studies, identification of the test pits, subsurface studies, and *in-situ* density testing. This station is a junction station between line nos. 1 and 6.

The results of this study are based on site investigation, laboratory testing and theoretical analysis. This study is limited to laboratory, analytical and experimental analysis.

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This station is intersected between line nos. 1 and 6 and the line no.1 became operational in 2001. This station is located in downtown of Tehran with heavy traffic. This study is a precise and concise case study that gives the importance of soil mechanical parameters used in designing deep and shallow tunnel and substation. There are many studies carried out using laboratory and field test results for tunnel structures while keeping in mind the critical zones of the tunnel or subway station in this paper is highlighted in order to avoid collapse and destruction of natural and human resources. One of the most significant advantages of the numerical method is in predicting the critical area surrounding the tunnel and the tunnel structure before making the tunnel construction due to different loads. Numerical modeling is used as control method in reducing the risk of tunnel construction failures. As long as some factors such as settlement and deformation are not completely predictable in rock and soil surrounding the tunnel, using numerical modeling is a very economical and capable method in predicting the behavior of tunnel structures in various complicated conditions of loading. An overview of classification of numerical modeling in tunnels is presented. Moreover, the soft-wares and their applications showing the model versatility with easy input parameters and output computers are introduced.

Software outputs show rock and soil deformations related to tunnel stability and predict the zones of hazards relevant to the underground structures. There are not many conducted studies using numerical models to tunnel structures that estimate the critical zones. One of the objectives of this paper is defining the seismic modeling of tunnel. Specifying the differences between the two PLAXIS and FLAC modeling is another aim of this paper. The stress redistribution caused by tunnel excavations induces movements in the earth mass and ultimately at the ground surface (Sadaghiani, 2010). The need to control the ground surface settlements in urban area is widely recognized and new construction methods are continuously developed. The settlements induced by underground excavation may cause serious damages to nearby structures and subsurface underground utilities (Sozio, 1998; Sekimoto et al., 2001). Several methods for predicting the ground surface settlement are presented in the literature (Peck, 1969; Chow, 1994; Sampaco, 2000; ITA, 2007). Numerous geologic and geotechnical related issues involving regional seismicity, active faults, high groundwater, toxic and potentially explosive subsurface gases, abandoned oil wells/oil fields, and both soft-ground and hard rock conditions above and below the groundwater surface, have imposed constraints on the design and construction of the subway projects (Stirby et al., 1999). The metro system in Tehran consists of 12 lines, 4 of which are currently operational (Lines 1, 2, 4 and 5) and is 430 km long and has 276 stations. Population growth, number of daily-trips, and motor-vehicles traffic in addition to changes in the structure of urban life have all caused the traffic problems which have manifested itself in the city of Tehran. This article can help researchers, designers and consulting engineers who are active in tunnel engineering to achieve good interpretation of deformation and behavior of tunnels in different rock and soil layers.

PROBLEM STATEMENT

The diagram of the tunnel, access gallery and the critical zones are shown in Figure 1. According to Figure 1, the boring of the access gallery is very sensitive and dangerous in the critical zones of the tunnel cross-section and the access galleries. The important point to note is that the load of three ribs is bearing on the tunnel crown and for this reason, minimizing the lattice girder number is not necessary. However, to overcome this problem, it is necessary to increase the length of lattice girder and to increase the shotcrete thickness, and then the tunneling and boring can be continued with safety. According to Figure 1, if tunneling continues with the previous speed, the rib foundation appears to be unstable at the tunnel face. Therefore the engineer has to avoid disturbing the ribs before the final stage of the station construction. Priority is in providing lining to the tunnel and station with a high quality reinforced concrete.

A numerical modeling is used as control method in reducing the risk of tunnel construction failures. Another benefit of using numerical simulation is in the colorful illustrations predicting the tunnel behavior before, during, and after construction and operation. There are some theoretical and experimental methods available to predict the tunnel behavior such as convergence-confinement method, analytical methods, and numerical methods. Among them, the Finite Element Method (FEM) and the Finite Deference Method (FDM) are only used in this study. The numerical methods are very simple and quick to use and the results are conservative and practical for users. As some of the methods available have limitation in simulating and modeling the whole tunnel design factors, numerical modeling seems the best option for it is fast, economical, accurate, and more interesting in predicating critical zones in tunnel. However, what soft-wares predict are not always the same as real ground nature conditions in which there is tunnel. As some of the methods available have limitation in simulating and modeling the whole tunnel design factors, numerical modeling seems the best option for it is fast, economical, accurate, and more interesting in predicating critical zones in tunnel. However, what softwares predict are not always the same as real ground nature conditions in which there is tunnel. Ground movements arising from tunneling projects in urban areas have the potential to cause damage to overlying structures. The most effective way and third great tool to promote deep conceptual understanding of the real world in the history of engineering

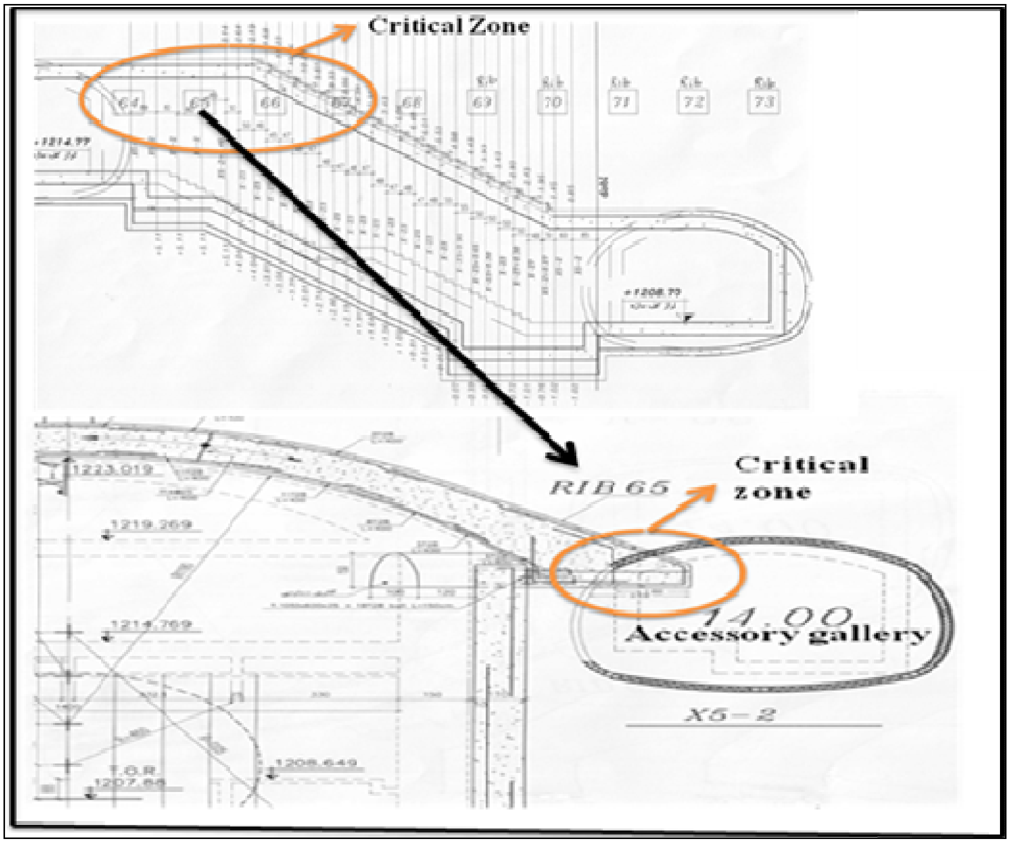


Figure 1. Cross section of metro tunnel showing critical zone of station and access galleries.

Table 1. Geotechnical properties of soil.

Soil type	Density	Shear strength parameters		Deformation parameters	
	γ (g/cm^3)	c (kg/m)	φ (°)	Et (kg/cm^2)	Poasion ratio (υ)
Aggregate (Sand and Gravel)	2	0.25	36	700	0.35

Table 2. Coefficient of B at different depth.

B (Coefficient depends on tunnel depth)	Tunnel depth (m)	
	6-15	15-30
	0.9	0.8

and sciences is through the investigation using computer simulation (Cai, 2000).

METHODOLOGY

Numerical analyses

Numerical analyses have been performed using the finite difference element code (FLAC) and finite element method code (PIAXIS) (Figures 4, 5, 6, 7 and 8).

Geology of project path

The main geotechnical investigation parameters are presented in Table 1. The underground water level in less than 50 m. Tunnel analyses are based on geotechnical parameters from soil or rock samples.

Seismic distortion

Angular distortion (γmax) based on the particles vibration affected by waves (VS) and velocity propagation shear waves (Cm) are as

Table 3. Coefficient depends on: α (Cm/S).

Earthquake intensity	Distance from focus		
	0-20	20-50	50-100
Rock			
6.5	66	76	86
7.5	97	109	97
8.5	127	140	152
Hard soil			
6.5	94	102	109
7.5	140	127	155
8.5	180	188	193
Soft soil			
6.5	140	132	142
7.5	208	165	201
8.5	269	244	251

Table 4. Soil specifications.

	ρm	vm	Em (kpa)
Soil specifications	2	0.35	70000

follows. Also, the coefficient of B for different depths are shown in Table 2, and the coefficient of α (cm/s) are shown in Table 3.

$$\gamma \max = \frac{Vs}{Cm} \quad (1)$$

$$C_m^2 = \frac{Gm}{\rho} \quad (2)$$

Soil shear Modulus (Gm),
ρ = Soil density

$$Vs = \frac{\alpha}{g}.\beta.A \quad (3)$$

A: Earthquake Accelerator: 0.35 g.

Distortion calculation of tunnel

Soil specifications are presented in Table 4.

Earthquake magnitude expected: Mw = 7.5
Focus distance< 20 km

$$\mathrm{G_m} = \frac{70000}{2(1+0.35)} = 25926kpa \quad (5)$$

(Gm= Calculation Maximum Distortion)

$$C_m^2 = \frac{25926}{2} = 12963 \longrightarrow C_m = 113.85m/s \quad (6)$$

A = 35 g

Tunnel depth is 20 -30 m and β = 0.8 and Cm<200 and soil is soft then α=208

$$Vs = 208 \times 0.8 \times 0.35 = 58.24Cm/s = 0.5824m/s \quad (7)$$

$$\gamma \max = \frac{0.5824}{113.85} = 0.0051 \quad (8)$$

RESULTS

Designing of tunnel

Designing of tunnel comprises two parts: First, temporary structures and final designing of the tunnel lining. The calculation and analyses were based on Plaxis and FLAC . The temporary support for tunnel with overburden 20 - 30 m was lattice and steel ribs. FLAC outputs represented the stable parts of tunnel which used steel frame for increase stability and high safety.

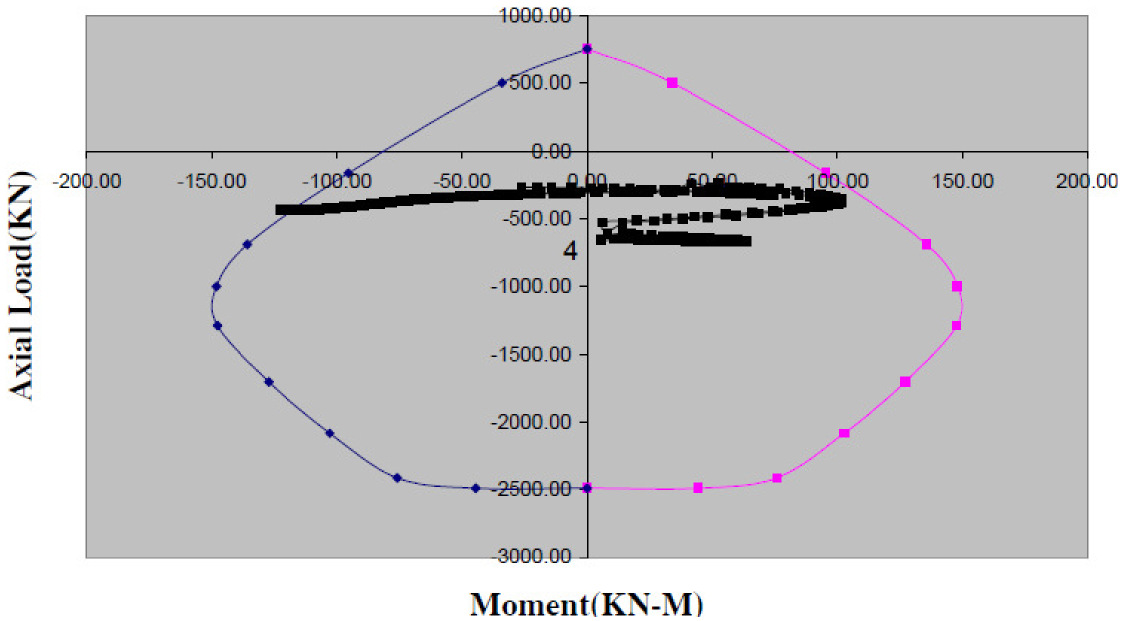


Figure 2. Interaction lattice frame (shotcrete =30 cm, bar = 25).

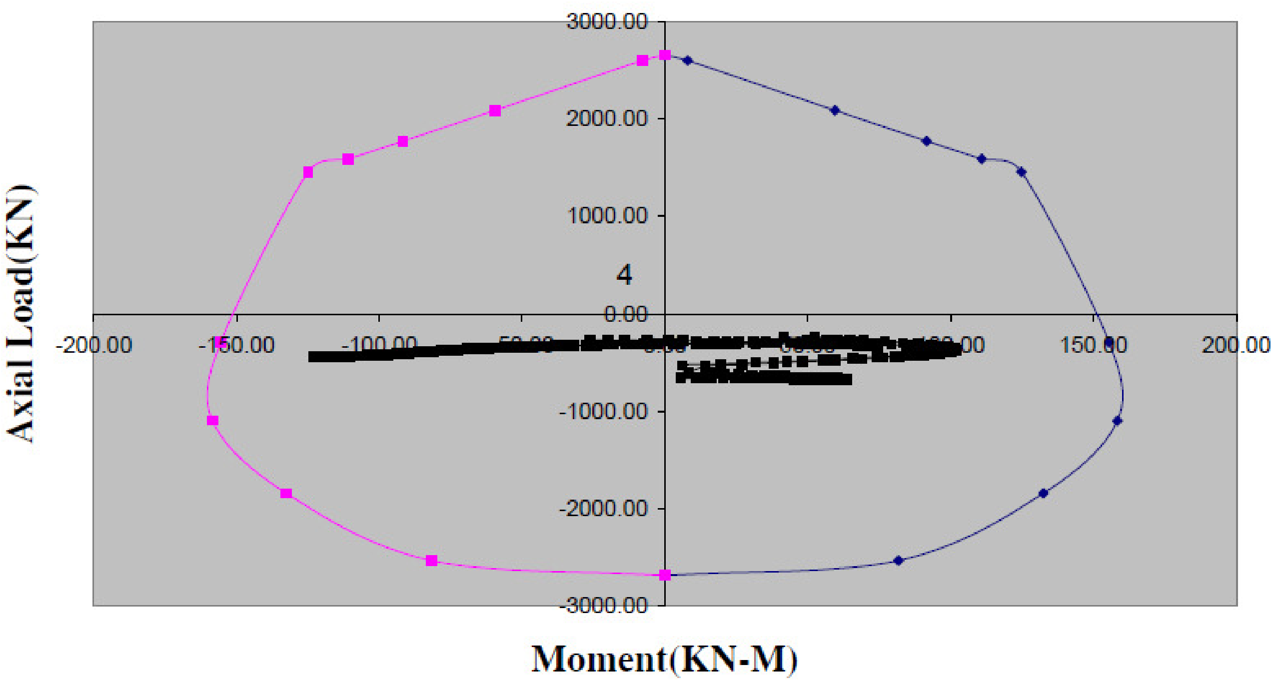
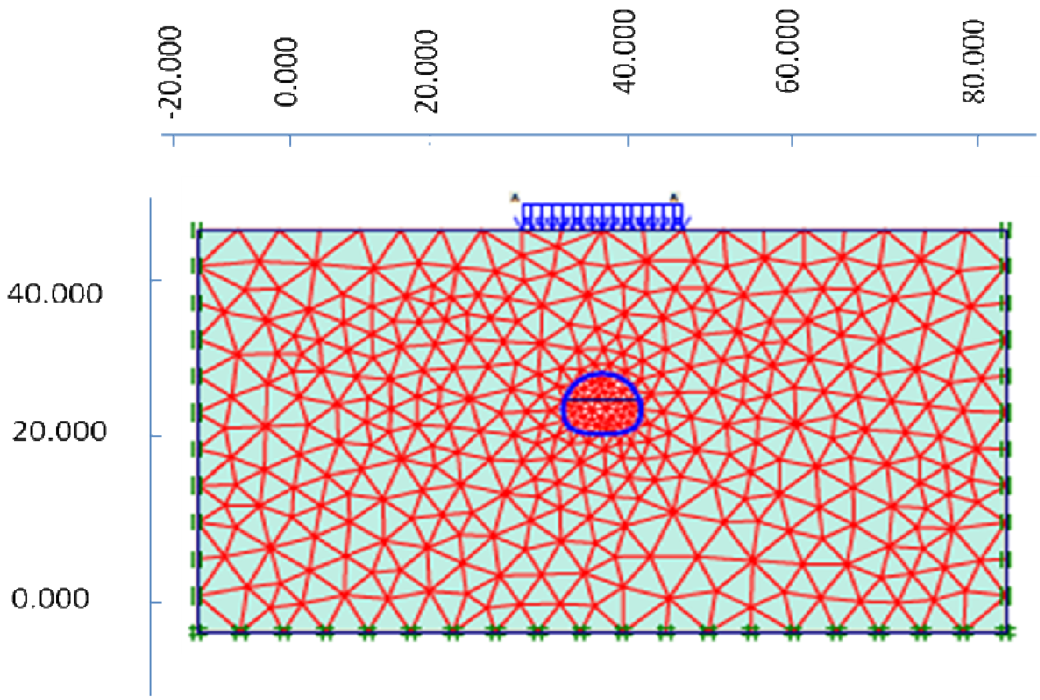


Figure 3. Interaction steel frame (shotcrete = 30 cm, IPE = 200).

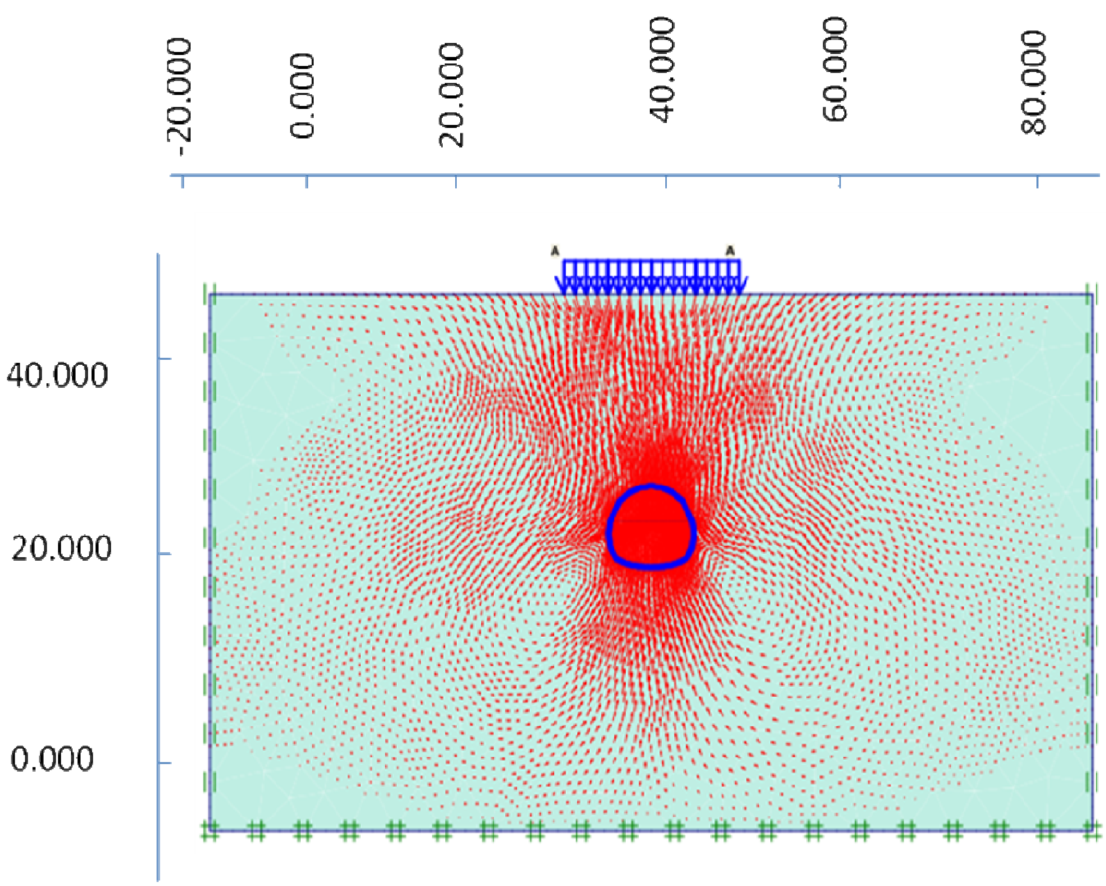
Tunnel with 20 - 30 m depth

It should be mention that concrete thickness was 26 cm and 4 # Φ22 mm bars were used. The axial load varied from (+150 kN) to (-150 kN) and the moment varied from (+750 kN-m) to (-2000 kN-m) as shown in Figure 2. As shown in Figure 3, the axial load varied from (+150 kN) to (-150 kN) and the moment varied from (+2500 kN-m) to (-2500 kN-m). The tunnel was modeled using PLAXIS. The results, in terms of, typical mesh, total displacement, horizontal displacement, bending moment and vertical displacement are presented in Figure 4. As seen in Figure 4 (B), the total displacement was 52.32 x 10^{-3} m. The extreme horizontal was 10.30 x 10^{-3} m, and the extreme

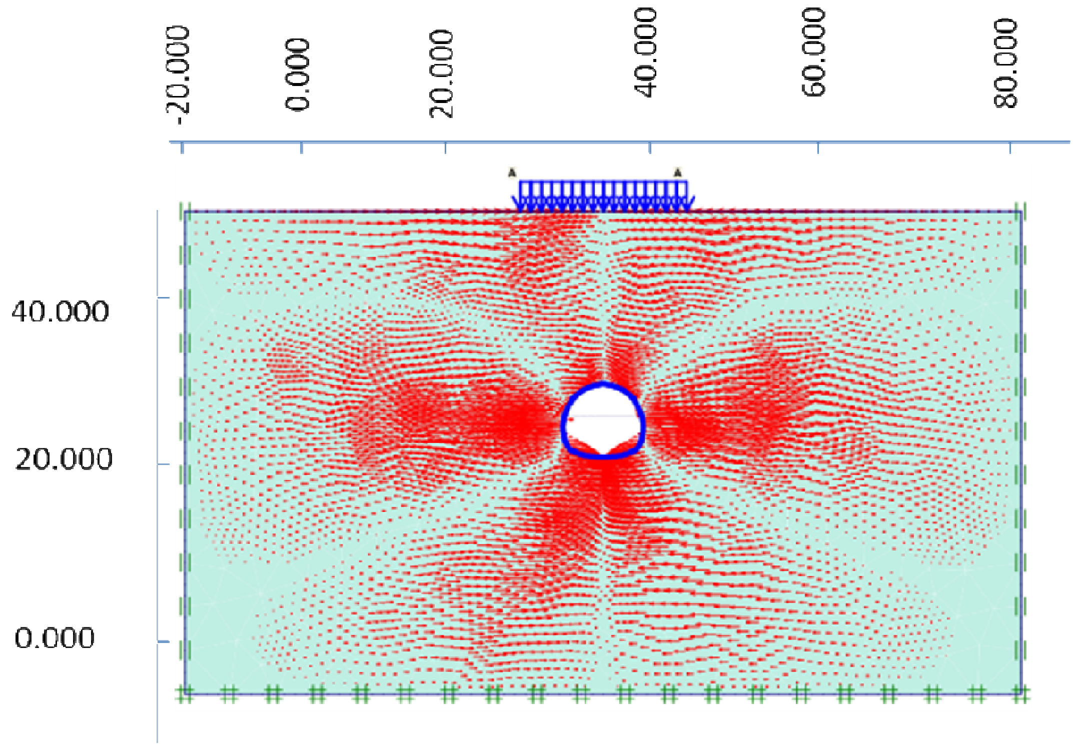
(A) Typical mesh.



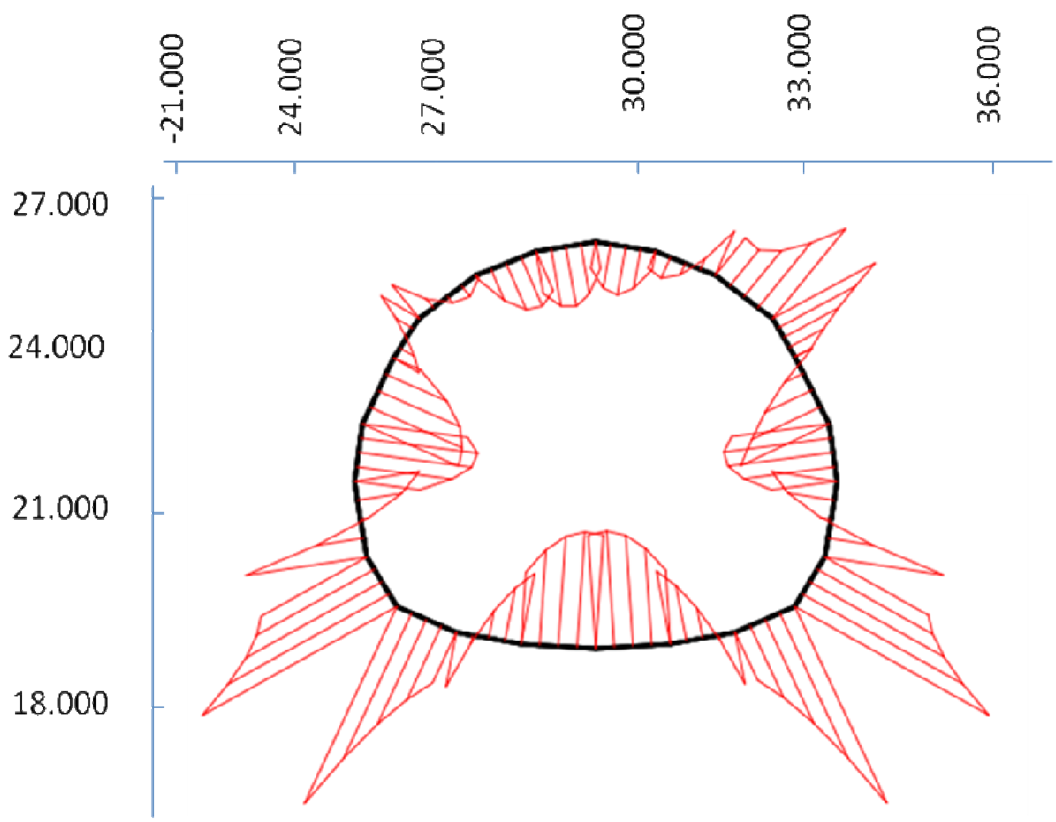
(4B) Total displacement (Extreme total displacement 52.32*10 $^{-3}$ m).



(C) Horizontal displacement (Extreme horizontal displacement 10.30*10^{-3} m).



(D) Bending moment (Extreme bending moment 296.13 kN-m/m)



(4E) Vertical displacements (Extreme vertical displacement 52.32*10^{-3} m).

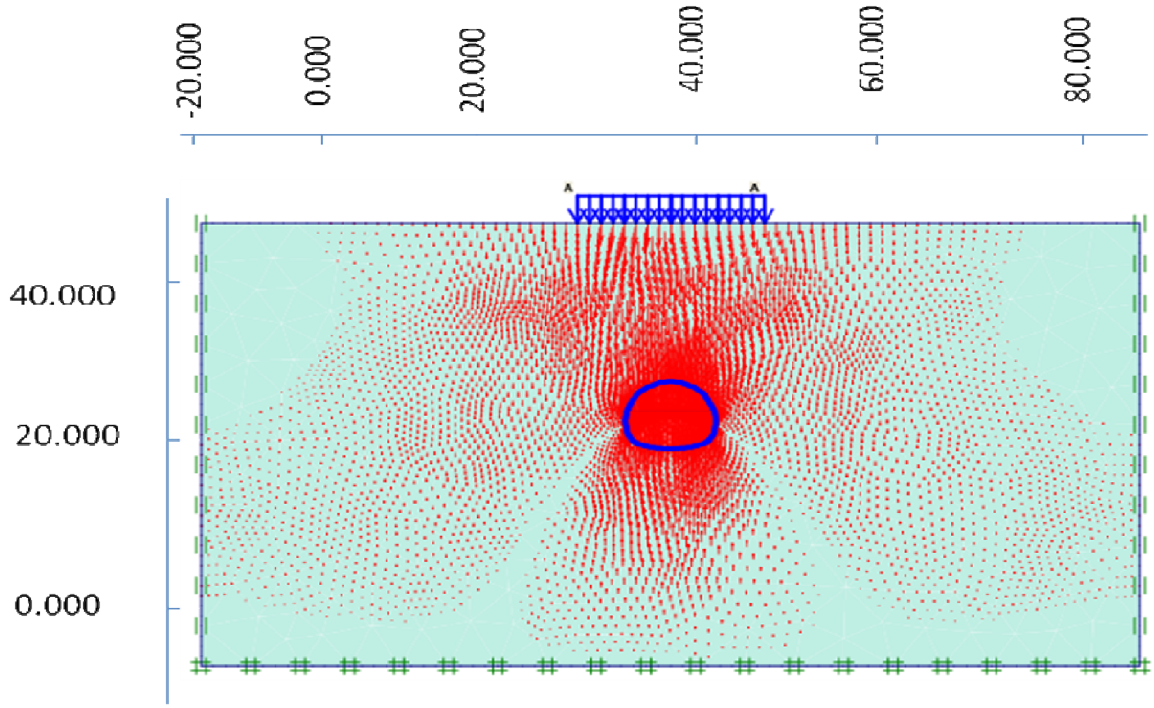
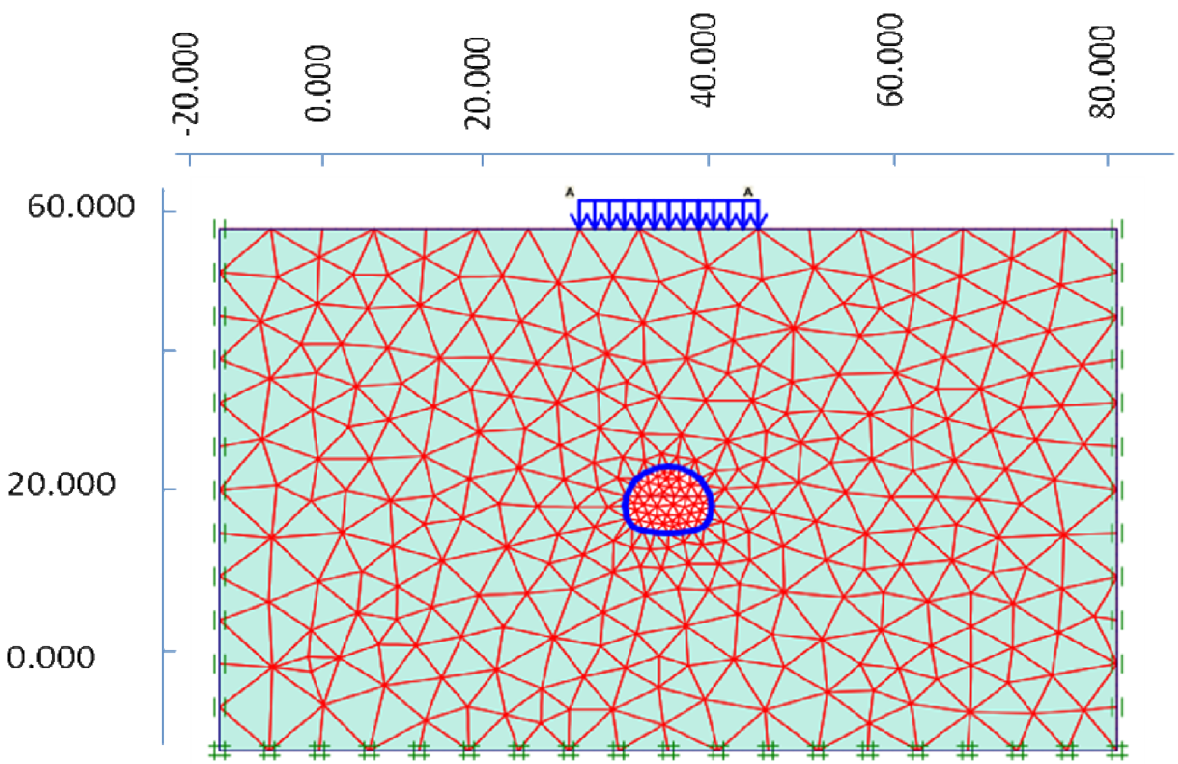
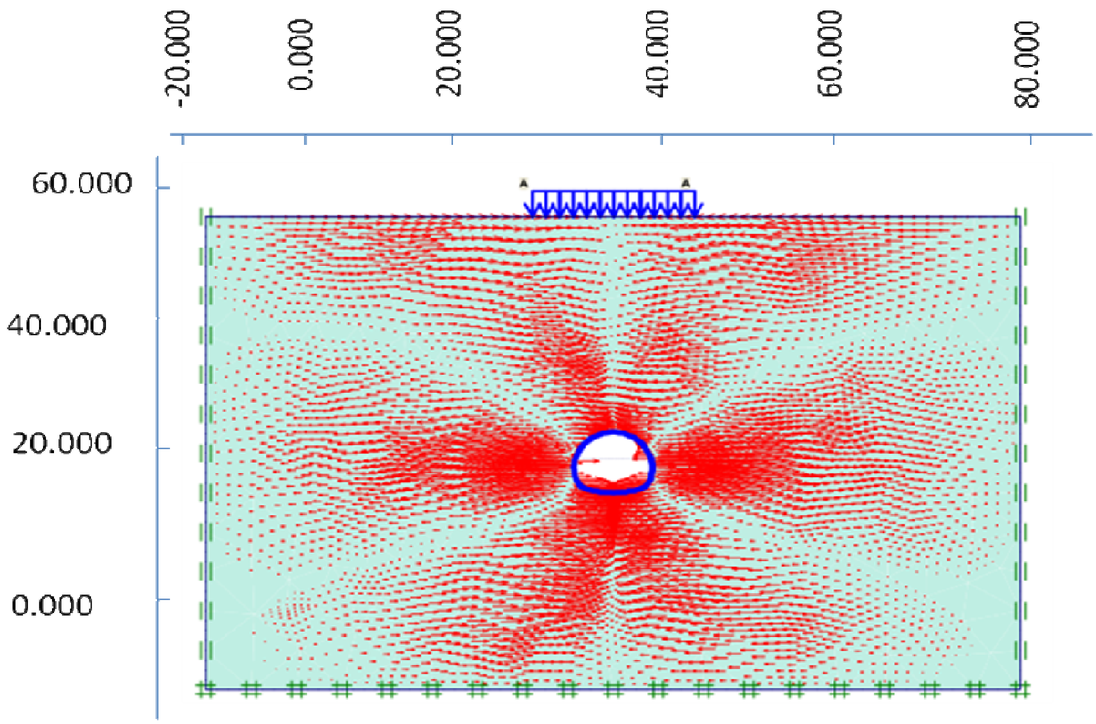


Figure 4. PLAXIS outputs for tunnel with 20 m overburden, (A) Typical mesh, (B) Total displacement, (C) Horizontal displacement, (D) Bending moment, and (E) Vertical displacement.

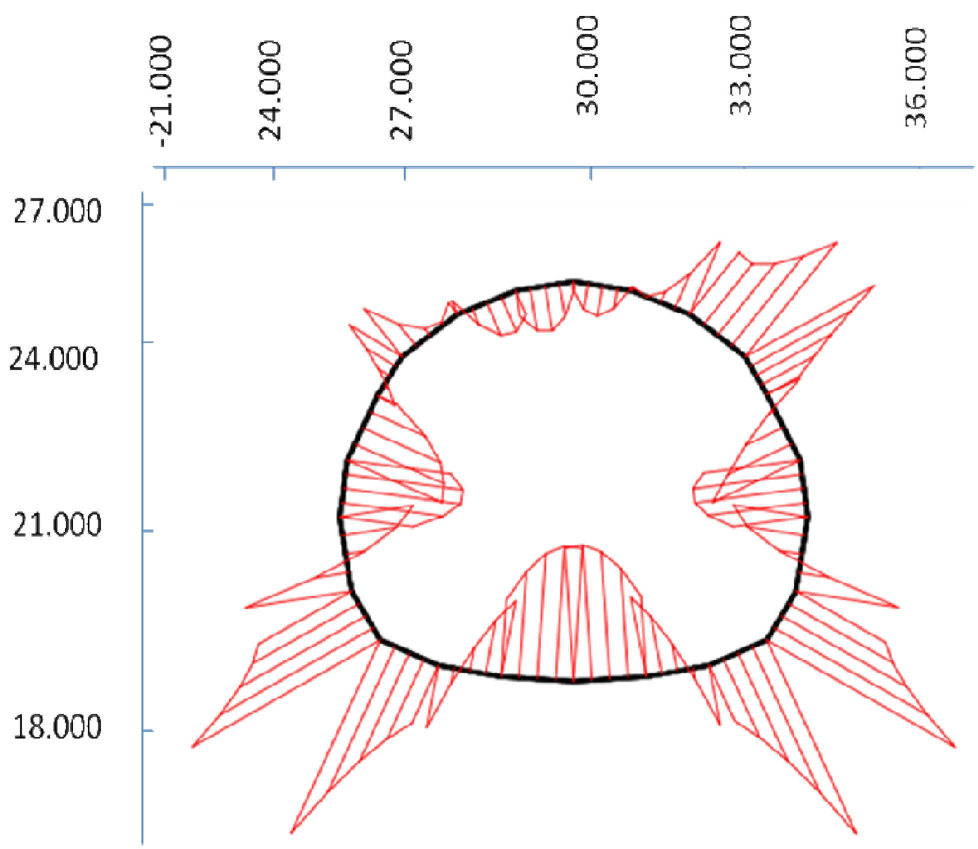
(A) Typical mesh.



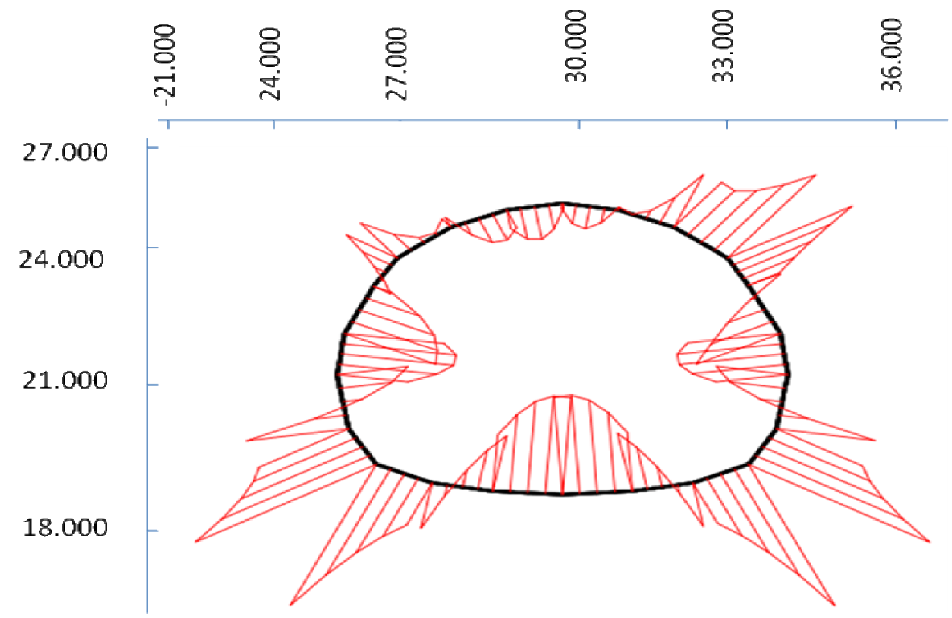
(B) Horizontal Displacements (Extreme horizontal displacement 12.89*10-3 m).



(C) Bending moment (Extreme bending moment -385.17 kN-m/m).



(D) Total displacements (Extreme total displacement 71.97*10^{-3} m).



(E) Vertical displacements (Extreme vertical displacement 71.97*10^{-3} m).

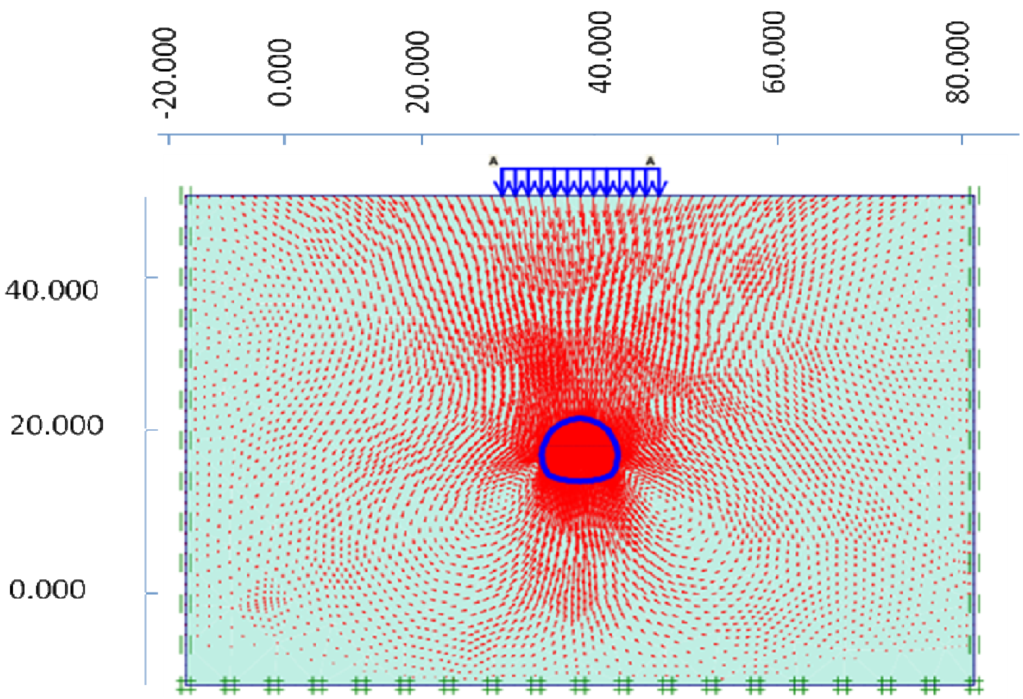
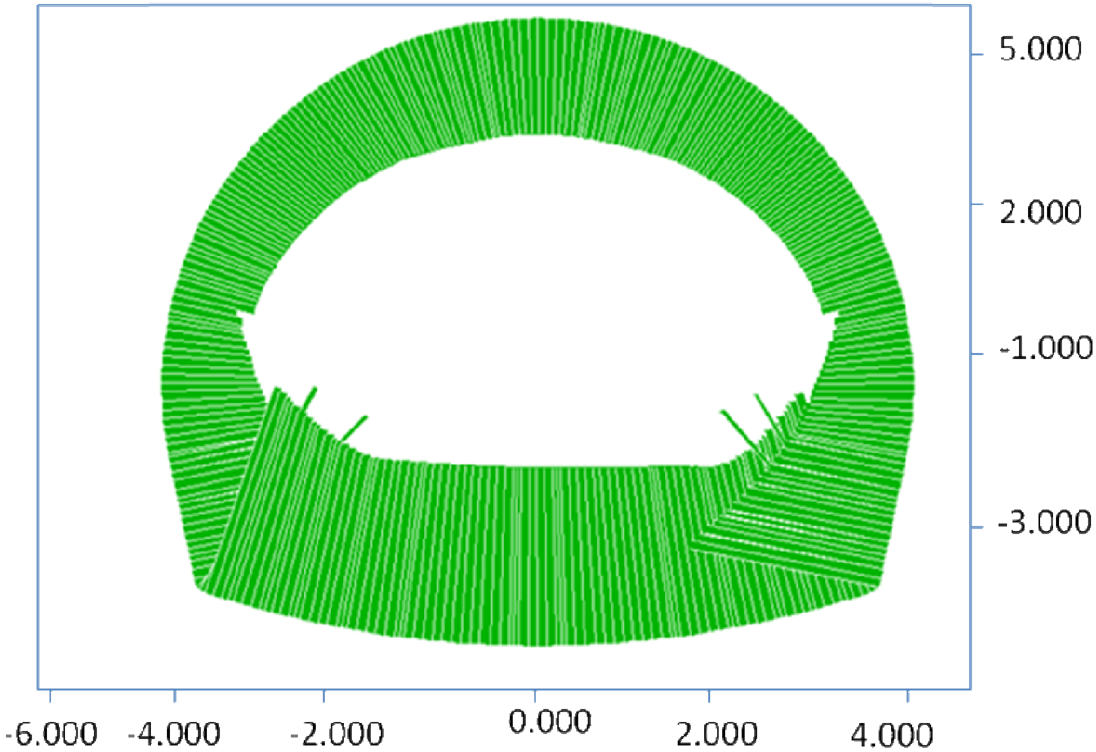


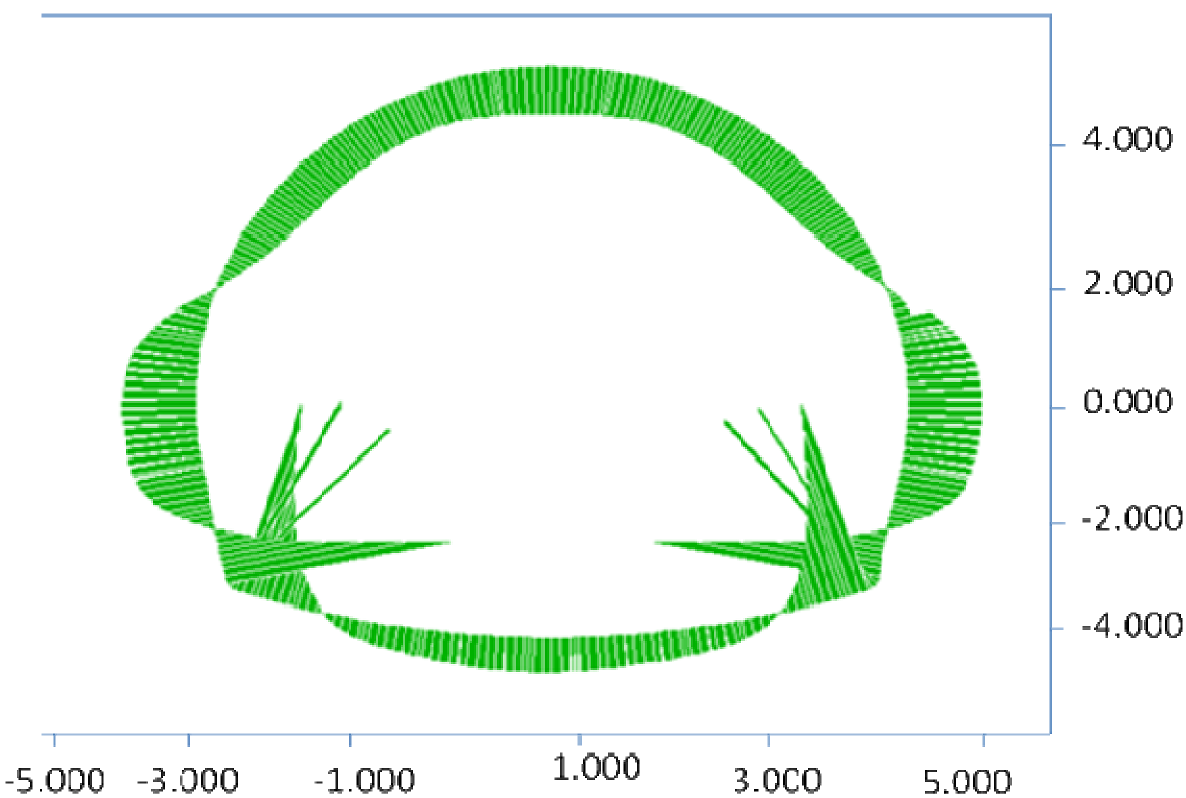
Figure 5. PLAXIS outputs for tunnel with 30 m overburden, (A) Typical mesh, (B) Horizontal displacements, (C) Bending moment, (D) Total displacements, and (E) Vertical displacements.

vertical displacement was 52.32 x 10^{-3} m. The extreme bending moment was 296.13 kN-m/m. A similar analysis of the tunnel was carried out for tunnel with 30 m overburden and the results are presented in Figure 5. As seen in the Figure 5(B), the total displacement was 71.97*10^{-3} m. The extreme horizontal displacement was 12.89*10^{-3} m, and the extreme vertical displacement was 71.97*10^{-3} m. The extreme bending moment was -385.17

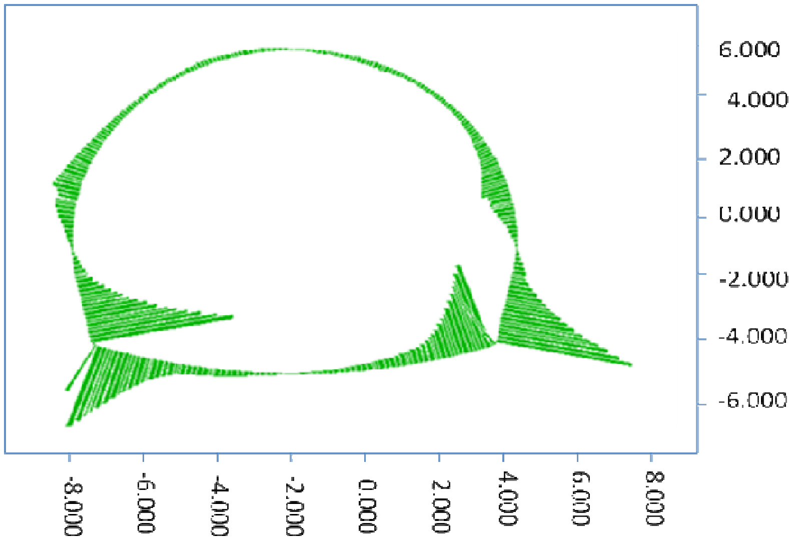
(A) [-5.70E<X<5.70E, -4.394<Y<5.759], Beam axial force (Max Value: 1.02E+06)



(B) [-6.348E<X<6.355E, -5.369<Y<6.733], Moment (Max Value: 1.13E^{+05})



C: [-5.936E<X<8.938E, -7.794<Y<7.080], Shear Force (Max Value: 2.22E^{+05})



(D) [-3.395E<X<3.395E, -3.663<Y<3.126], Contour interval=2.00E^{-02}

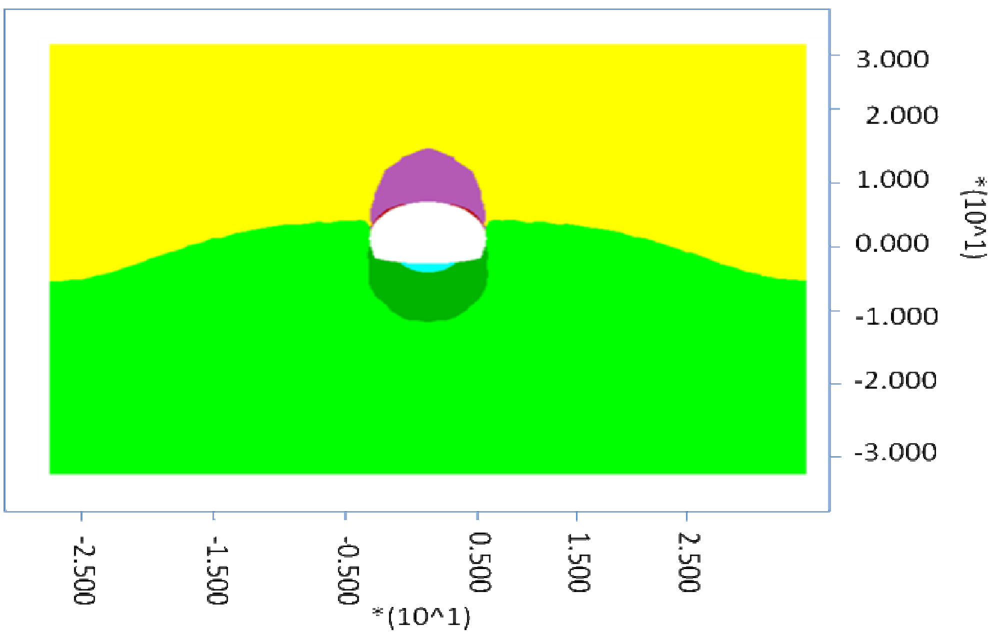
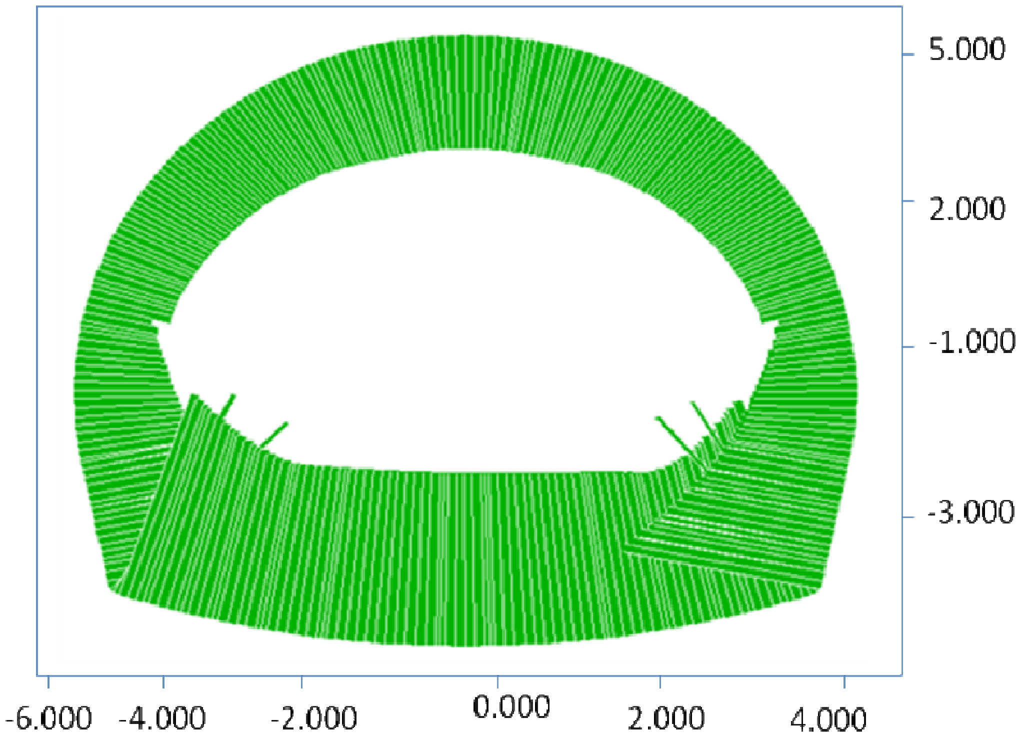


Figure 6. FLAC outputs for tunnel with 20 m overburden, (A) Beam axial force, (B) Moment, (C) Shear force, and (D) Contour interval.

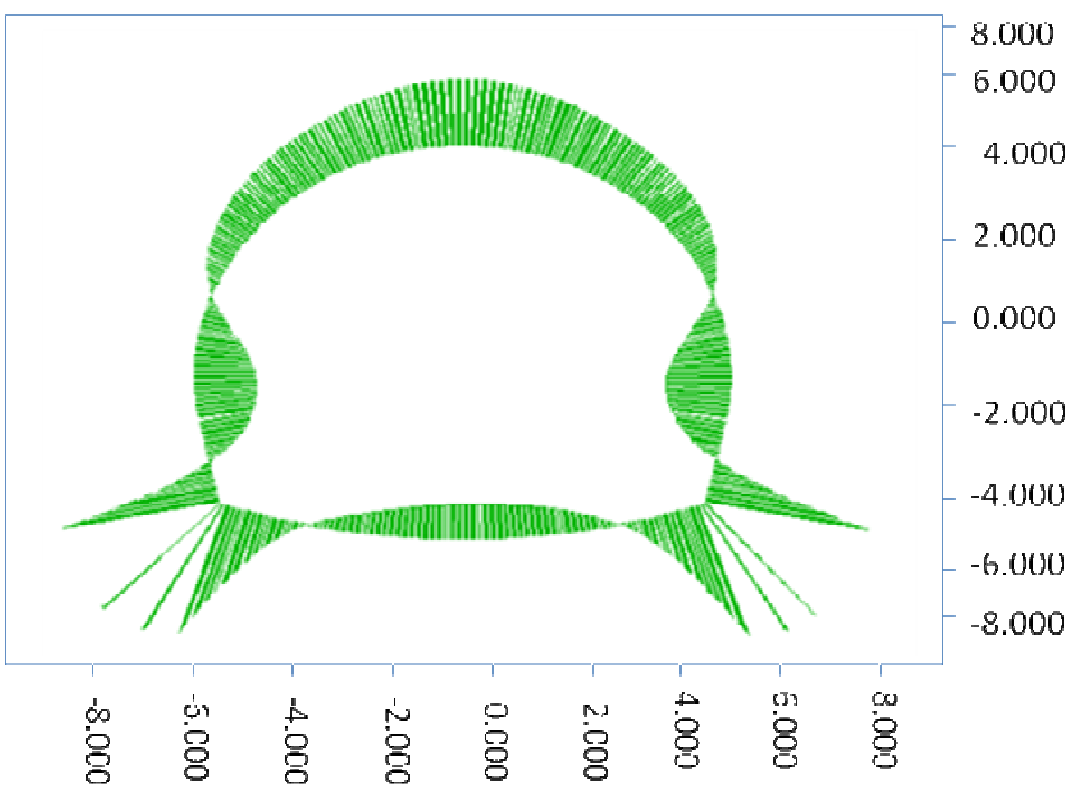
kN-m/m. Similarly, the analysis of tunnel with 20 and 30 m overburden were carried out using FLAC. The results with 20 m overburden, in terms of axial force, bending moment, shear force and contour interval are shown in Figure 6. As evident from the figure, the maximum axial force was observed to be 1.02E^{+06} kN, maximum moment was 1.13E^{+05} kN-m/m, maximum shear force was 2.22E^{+05} kN and the contour interval was 2.00E^{-02}.

Similarly, the results with 30 m overburden, in terms of axial force, bending moment, shear force and contour interval are shown in Figure 7. As evident from the figure, the maximum axial force was observed to be 1.14E^{+06} kN, maximum moment was -2.10E $^{+05}$ kN-m/m, maximum shear force was 3.09E^{+05} kN and the contour interval was

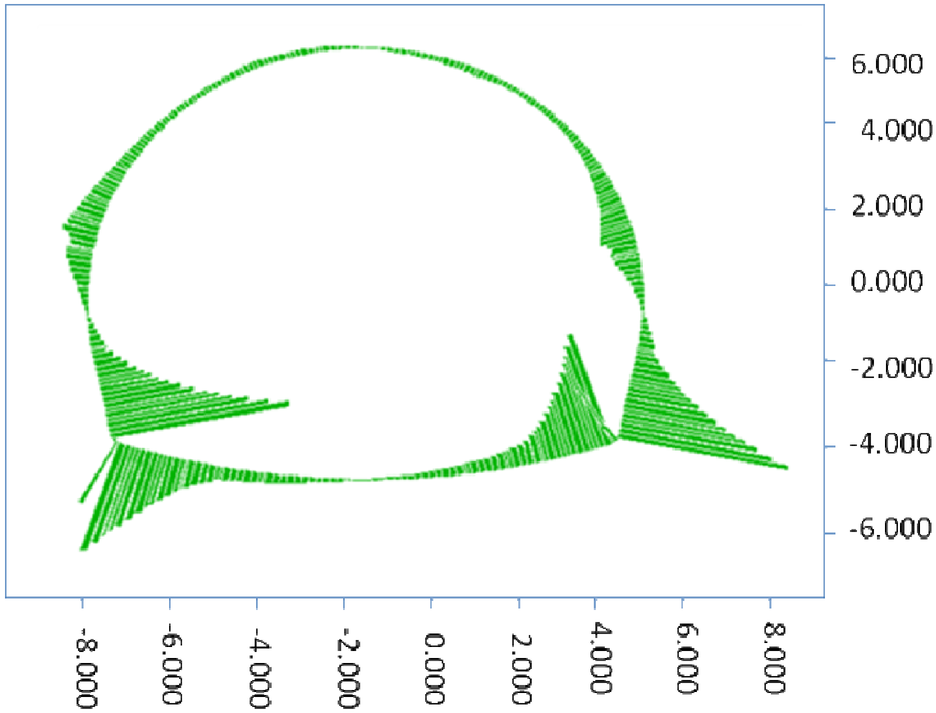
(A): [-5.076E<X<5.076E, -4.394<Y<5.759], Axial Force
(Max Value: $1.14E^{+06}$)



(B): [-9.142E<X<9.243E, -9.159<Y<9.226], Moment
(Max Value: $-2.10E^{+05}$)



(C): [-5.936E<X<8.938E, -7.794<Y<7.080], Shear Force (Max Value: 3.09E+05)



(D): [-3.561E<X<3.561E, -3.330<Y<3.793], Contour interval=2.50E-02

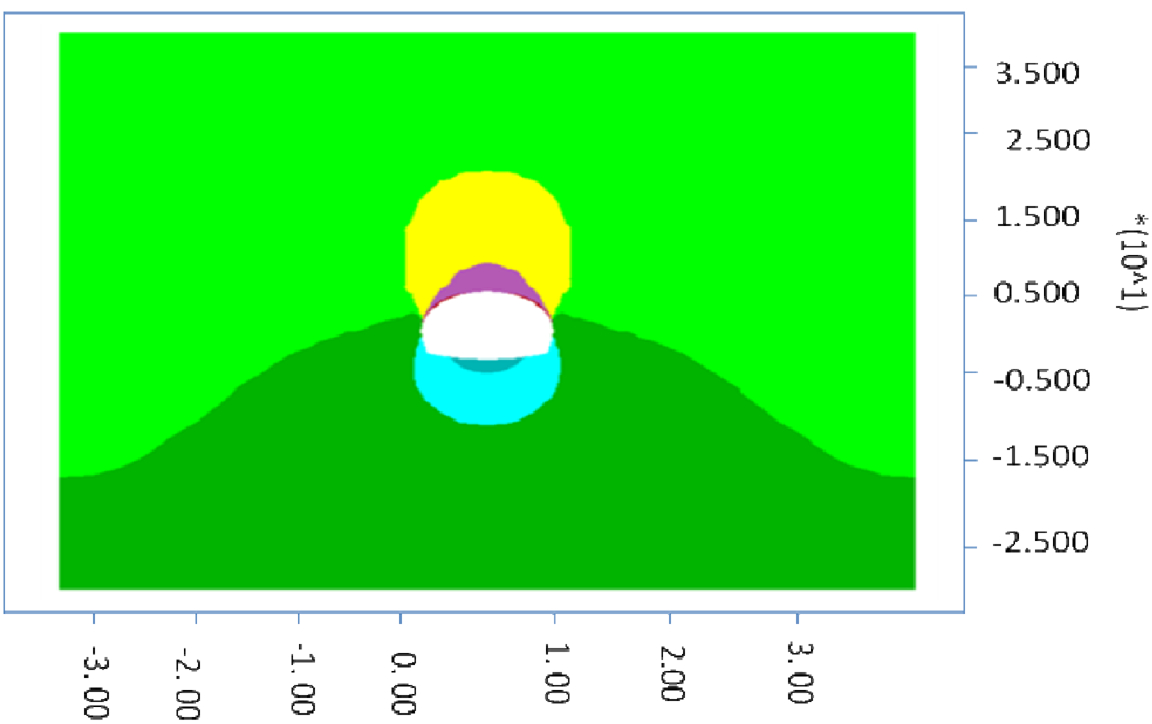
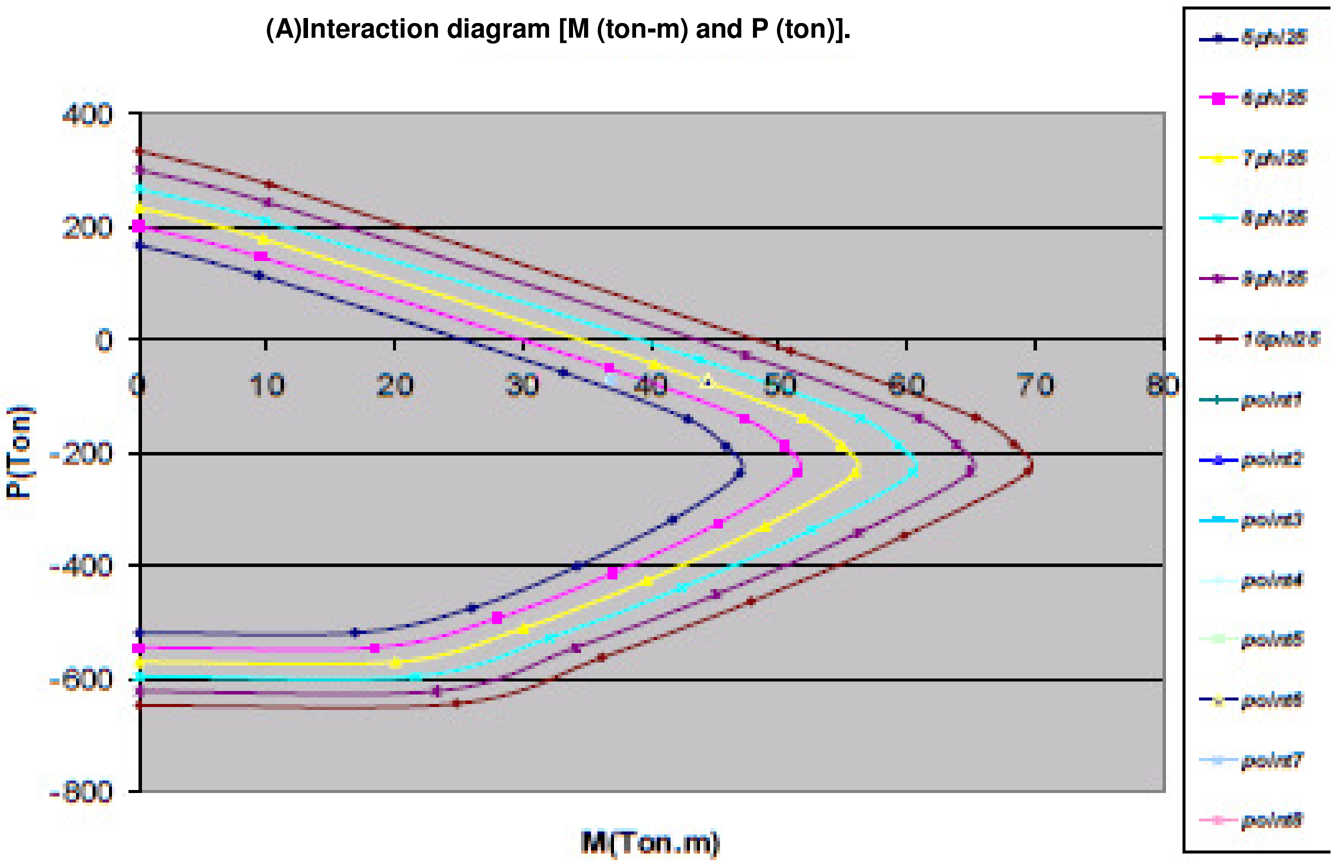
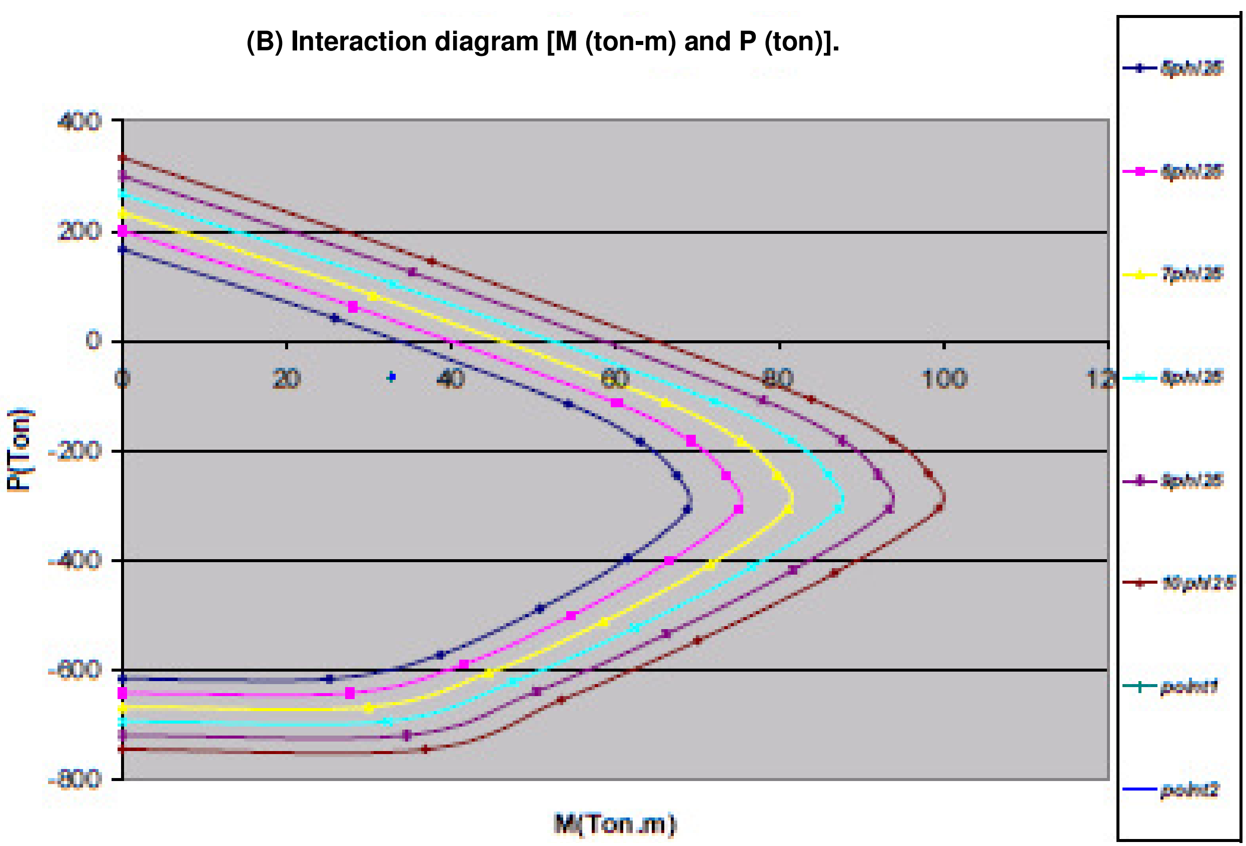


Figure 7. FLAC outputs for tunnel with 30 m overburden, (A) Beam axial force, (B) Moment, (C) Shear force, and (D) Contour

(A)Interaction diagram [M (ton-m) and P (ton)].



(B) Interaction diagram [M (ton-m) and P (ton)].



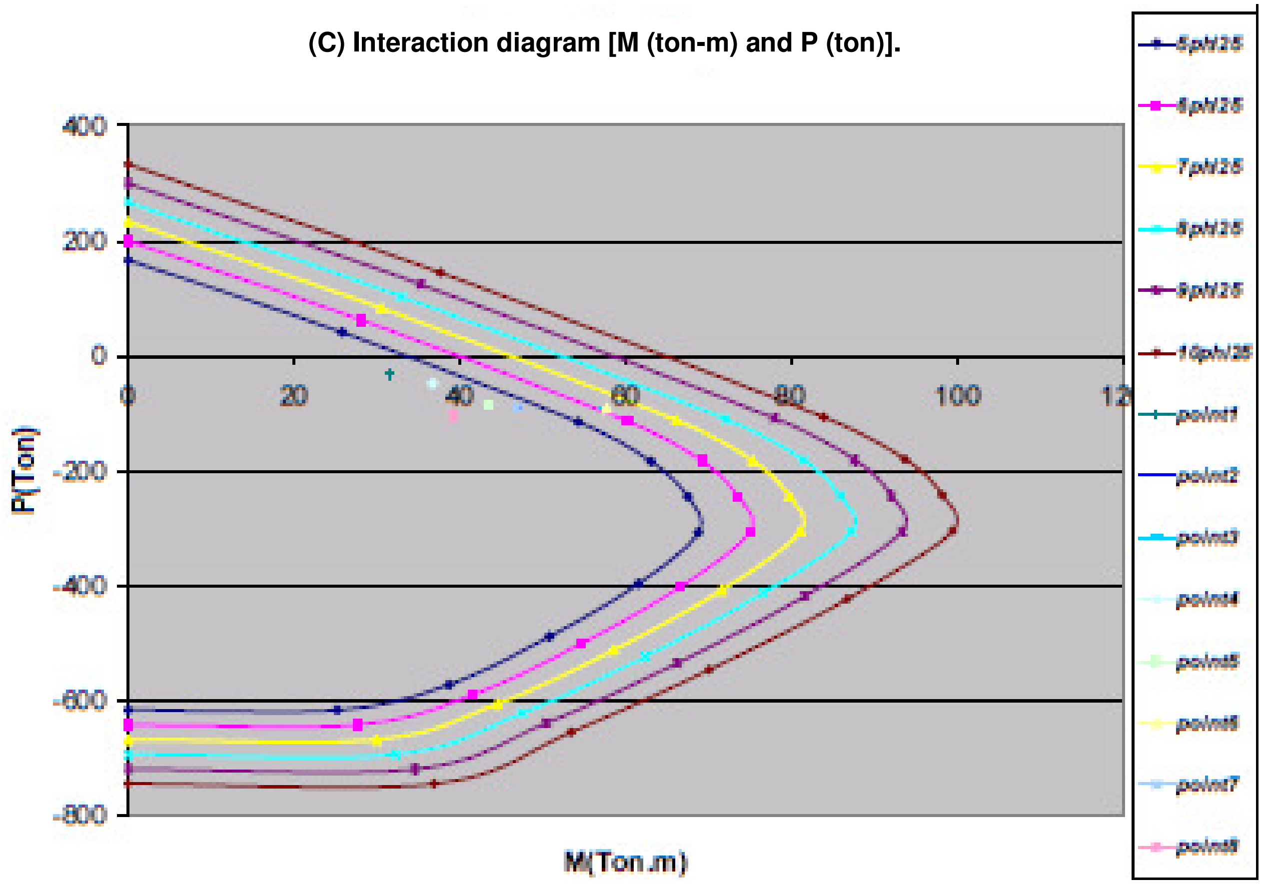


Figure 8. Interaction diagrams, (A and B) for 20 m overburden, and (C) for 30 m overburden based on M (ton-m) and P (ton) of tunnel with 20 m and 30 m overburden.

2.50E^{-02}. The interaction diagram with 20 m overburden is shown in Figures 8(A) and (B). Similarly, the interaction diagram with 30 m overburden is shown in Figure 8 (C).

RESEARCH STUDY

The role of numerical (FEM, FDM) simulation methods in tunnel design

The most basic difference between finite element and finite difference methods in numerical modeling is apparent from their names. In finite element method, a field variable (also known as a field function) the quantity of interest, temperature, water head, displacement, etc., is defined by a differential equation in mathematical physics, and solved through a desired field by the aid of shape functions that are geometric characteristics of the area under study. These shape functions (also known as interpolation functions) relate the quantity of the field variable to its value on the boundary of the domain of interest, that is, the numerical solution of the problem is approximated by these functions. In this method, the value of the field variable can be found from any desired point in the domain of the problem. In the finite difference method, an approximate solution of a differential equation of mathematical physics is presented by the aid of finite differences of the field variable in some certain nodes in the domain. Thus, the solution is obtained even from some predefined points and not at every point. Another method somewhat similar to FEM and FDM is known as Mesh Free or Mesh-Less method.

DISCUSSION

A number of softwares based on finite element method and finite difference method like, PHASE2, EXAMIN3D, PLAXIS, FLAC, PLAXIS 3D Tunnel, and other similar software can be used for the numerical modeling while hand calculations are also possible. In the later method (hand calculations), some simplifications are presented in professional books on rock mechanics and tunnel engineering in which, the theory of elasticity, and in some cases, theory of plasticity, are employed to introduce a solution to a general tunnel problem. A general tunnel problem in the theory of elasticity is commonly predicated as an axisimmetric problem of a circular tunnel in deep soil or rock mass. In this problem, the stresses are uniformly distributed in two perpendicular directions, that is, horizontal and vertical stresses. In fact, the variation of stresses in the depth, due to geostatic pressure, is neglected. In finite element or finite difference programs, every state of loading can be modeled. But, the first question is that which part of a tunnel is to be modeled? This case study presents the geotechnical studies of the Hafetir metro station excavation including; geological studies, the design of the tunnel, metro station and building foundations are usually based on the geological and geotechnical investigation results obtained from laboratory and field testing.

This study is a concise case study which highlights the important soil parameters for designing of deep and shallow tunnels and substations. This case study shows the critical area for performance of tunnel intersection of and access galleries and the method to avoid collapse of the tunnel and access galleries while tunneling with a high safety. If boring of the access gallery and tunnel station, is made at the same time or the boring of the access gallery is carried out before the tunneling lining, the tunnel will be overburdened with loads leading to weakening of the access galleries crown, and its immediate collapse. As long as this station is at the intersection station between lines 1 and 6, it needs a larger access gallery to the ground surface and other lines. There are critical zones and unstable zones for boring the tunnel, station and accessory galleries at the same time. The method to prohibit collapse is to stop boring of accessory galleries and to continue boring of the tunnel, after which the boring of the access gallery can be undertaken. The finite element and finite deference analyses of the tunnel and the access gallery was carried out by using the commercially available softwares PLAXIS and FLAC. The field stresses in soil used in the analysis is shown in Tables 1, 2, 3, and 4 and also Figures 3, 4, 5, 6, 7 and 8.

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