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# Significance of Soil Compaction on Blast Resistant Behavior of Underground Structures: A Parametric Study

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**Abstract:** Dynamic response of underground structures has always been a topic of concern for designers and researchers. The behavior of these complicated systems under blast loading is affected by various factors and parametric studies are required to investigate their significance. The importance of soil density around the underground structure through which, the waves of explosion of a penetrator bomb is transferred, has been studied in this paper by using finite difference method (FDM). According to the results, soils with higher degrees of compactioncan absorb explosion energy more significantly. Therefore, the displacements and stresses of underground structure lining in denser soils are moderately lower. Thebending moment of the lining should be given attention, as regards being a critical design parameter.

**Keywords:** Bomb, Finite difference method, Passive defense, Underground structure.

## INTRODUCTION

A secondary goal in designing some underground structures (facilities like subway tunnel) is to provide blast-resistant structures, which can be used as civilian shelters. These structures are designed based on their primary objectives andcontrolled for blast loading situation, to evaluate their performance in the time of need(e.g., Desai et al., 2005). The same problem exists for underground facilities such as marine pipelines (e.g., Shah Mohammadi and Mohammadi, 2010). The most critical situation that these underground structures may encounter is the explosion of a penetrator bomb, which is designed to penetrate the soil, rock, or concrete and detonate inthe nearest possible position to an underground facility.

Extensive researches have proven that numerical simulations are valuable and accurate tools for investigating the effects of blast loading on underground structures. Different numerical techniques can be used to simulate the dynamic behavior of underground structures caused by blasting. Considering thelinear behavior of the medium, Boundary Element Method (BEM) is suitable for studying wave propagation problems such as blasting (e.g., Rahimian et al., 2010; Safari and Noorzad, 2009). Among the numerical methods, however, the Finite Element (FEM) and Finite Difference methods (FDM) are more common in practice for studying the effect of blasting, since it is possible to consider the problems in the cases of internal blasting as well (e.g., Feldgun et al., 2014; Feldgun et al., 2008; Gholizad and Rajabi, 2014; Lu and Wang, 2005; Song and Ge, 2013). Stevens and Krauthammer (1991), and Stevens et al. (1991) used a series of full-scale tests to

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constitute a combined numerical model for analyzing response of buried reinforced concrete arches to surface explosions. They used finite element method (FEM) for modeling an underground structure and finite difference method (FDM) for soil media.

A viscous cap plasticity model was used for modeling soil media. Reinforced concrete was simulated by using a nonlocal continuum damage/plasticity model. The FEM/FDM hybrid model was used to predict stresses and displacements of the arches. Results of the simulations were compared with the full scale tests outcomes, in order to assess the accuracy of the combined numerical model. Yang (1997) investigated the response of buried shelters to confined blast loading using the FEM model. In this research, soil and concrete were modeled linearly and the goal was to investigate the influences of soil, explosion, and structure characteristics on the response of buried shelters. Gui and Chien (2006) studied the blast-resistant behavior of a tunnel passing beneath the Taipei Shongsan airport. They used Flac 2D software, which is based on finite difference method, to investigate the effect of soil parameters as well as penetrator bomb parameters on the underground structure'sresponse under blast loading. The well-known linear elastic–perfectly plastic Mohr-coulomb model was assumed for the soil, in order to consider undrained behavior.Parametric studies were performed with a range of undrained shear strength (cohesion), Young's modulus and damping of the soil. The soil Young's modulus was found to be the most influential parameter among other parameters. Nagy et al.(2010)numerically investigated the effects of subsurface blast on buried structures. For the soil, the elastic-plastic Druker-Prager cap model was selected.

Using the Abaqus software, a numerical formulation called the Arbitrary Lagrange Euler Coupling defined by Hu and Randolph (1998) was used to model explosive charge and the soil region near the explosion, to eliminate distortion of the mesh under high deformation, while the conventional finite element method was used to model the rest. The interface between the soil and the tunnel was also taken into consideration, by using the Mohr-Coulomb model. The behavior of the whole system was evaluated using a numerical example which shows that the approach of the proposed model, was capable of producing a realistic simulation of the behavior of the physical system in a smooth numerical process.

This paper is a numerical investigation based on the effects of soil density over dynamic response of underground structures which were attacked by explosion of a penetrator bomb. To this aim, a parametric study was performed using the Finite Difference Method (FDM) with Flac 2D code (Itasca Consulting Group, 2005). FDM has already proven to be a sufficient technique for investigating wave propagation through soil media (Stevens et al., 1991). The geometry and embedment depth of the underground structure in this study, represent the metro subway tunnels corresponding to Mashhad city, located at the north-east of Iran. Most parts of the soil are comprised of medium dense to dense silty sand. In this study, the focuswas on the plastic parameters of the soil, with regards to the soil's behavior in terms of its stress-strain relationship.

## NUMERICAL MODELING

Numerical models were created based on the characteristics of the Mashhad subway tunnel, which is a circular tunnel with an outer diameter of 9.1 m with 35 cm thick reinforced concrete lining. The buried depth of the tunnel varies between 15 and 25 m along the route. An average amount of 20 m depth was chosenfor the underground structure modeled in this study. Figure 1 shows the models' size and their boundary conditions. Numerical models were extended 40 m (about nine times as big as tunnels radius) from right, left and bottom of the tunnel. Therefore, the models length and height were 80 and 60 m, respectively. In Figure 1, special boundaries called viscous boundaries are shown according to the Flac manual (Itasca Consulting Group, 2005). These were added to the models in the dynamic analyses in order to prevent blast waves from reflecting into the grids (Lysmer and Kuhlemeyer, 1969). As already mentioned, numerical analyses were performed using Flac 2D software.

**Soil Modeling**

The US Army manuals suggest that underground facilities, which are used as shelter should be confined in sand (TM5-855-1 1986). The Mashhad subway tunnel also passes through different sandy soils. The underground water table is below the tunnel level.

The simple linear elastic-perfectly plastic Mohr-Coulomb model with a non-associated flow rule was used as a constitutive model,to represent the behavior of soil which experiences large deformation. In the literature, there are advanced constitutive models developed for high and rapid loading over soils (e.g., An et al., 2011; Higgins and Chakraborty, 2013; Tong and Tuan, 2007; Wang et al., 2004). However, it should be noted that these models often have a number of parameters whose values can hardly be measured and the model's implementation in numerical codes would not be interesting. Instead, one can use simpler constitutive models such as the Mohr-Coulomb model by considering its limitations. In this case, the complexity of the analysis is reduced, but the solution would be given along with some estimation.

Soil parameters for the Mohr-Coulomb model can be divided into elastic and plastic parameters. The elastic parameters are Poisson's ratio ($\upsilon$) and Young's modulus (E), while the plastic parameters comprise of cohesion (c), internal friction angle ($\phi$), and dilation angle ($\psi$). Elastic parameters define the stress-strain relationship of the soil in the domain of reversible deformations, while the irreversible deformation portion (defined by dilation angle), as well as magnitude of ultimate shear strength (defined by $\phi$ and c) of the soil are controlled by plastic parameters. In the following, elastic parameters are introduced first and then, the selection of plastic parameters is discussed.

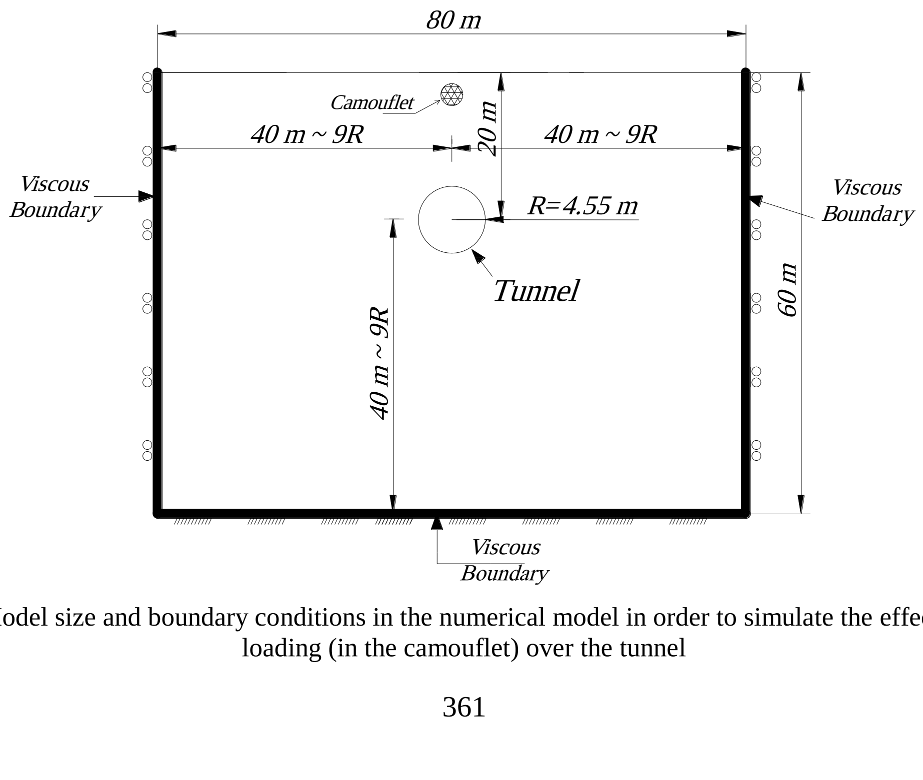


**Fig. 1.** Model size and boundary conditions in the numerical model in order to simulate the effect of blast loading (in the camouflet) over the tunnel

Table 1 presents non-plastic parameters of the soil whose values correspond to the characteristics of the majority of the soil in the Mashhad subway project, which are medium dense to dense silty sand. The parameters including soil density and Young's modulus can be easily obtained from the geotechnical site investigation, reported by geotechnical consultants. Poisson's ratio has a small range of zero to 0.5 and its value usually does not have a major effect on the analyses. In this research, $\nu = 0.3$ is taken into account which is acceptable for the soil condition in the project. Regarding the dynamic elastic modulus, it is noted that this parameter of the soil varies significantly under blast loading. Investigationsby Jackson et al. (1980) revealed that the Young's modulus of the soil increased up to ten times in sub-millisecond loading. However, Farr (1990) discovered that a large increase in soil stiffness does not occur in loading with sub-millisecond peak pressure time. Ishihara (1996) suggested that a gradual increase of up to 100% (two times) would happen to the Young's modulus of the soil which undergo blast loading. It should be noted here that some soil explosion parameters in this study were obtained from charts of TM5-855-1(1986) (US Army manual for designing protective structures published in 1986). Since this manual isbased on the works of Jackson et al. (1980), the dynamic Young's modulus of the soil was selected to be about 6 times bigger than its static modulus in all the analyses.

In this study, the influence of soil plastic parameters ($\phi, c, \psi$) on the response of underground structures was studied by varying their value in the analyses. In general, changes of soil plastic parameters may happen due to soil compaction. Although the non-plastic parameters were also altered in the compaction process, their variations were neglected in this study. Five model analyses were performed with different soil plastic parameters. Table 2 shows a list of soil plastic parameters used in thefive analyses. In order to properly simulate the behavior of the soil, plastic parameters should be selected, provided that their value is matched with each other. The selection procedure of these parameters is explained in the following paragraphs.

Internal friction angle ($\phi$) is one of the parameters that affects soil strength. In addition, this parameter is usually considered as an index of soil compaction state. This parameter usually varies from 35 to 45 degrees for medium dense to very dense sandy soils. According to Vesic (1973), changes in strain rate of dense sandy soils alter their friction angles and increase in strain rate leads to a decrease in soil friction angle. Based on Vesic's works, friction angle of dense sandy soils in dynamic loading ($\phi_{dyn}$) is defined thus:

**Table 1.** Soil elastic parameters used for all soil types

| $\rho(\frac{kg}{m^3})$ | $\upsilon$ | $E_{Static}$ $(MPa)$ | $E_{Dynamic}$ $(MPa)$ |
|---|---|---|---|
| 1700 | 0.3 | 48 | 300 |

**Table 2.** Soil plastic parameters in different analyses

| Analysis No. | $\varphi$(deg) | $\varphi_{dyn}$(deg) | $\psi$(deg) | $c\,(kPa)$ |
|---|---|---|---|---|
| 1 | 35 | 33 | 2.5 | |
| 2 | 38 | 36 | 6.25 | |
| 3 | 40 | 38 | 8.75 | 1 |
| 4 | 43 | 41 | 12.5 | |
| 5 | 45 | 43 | 15 | |

$$\phi_{dyn} = \phi - 2^{\circ} \tag{1}$$

Based on these findings and the fact that soil strain rate varies vastly during blast loading, in all the analyses, dynamic soil friction angle ($\phi_{dyn}$) was assumed to be 2 degrees smaller than its static condition. In this paper, five values of $\phi = 35,38,40,43,45^{o}$ were chosen in the parametric studies. It is here noted that the values of $\phi$ mentioned in this study regards the peak value, which is often obtained in the laboratory.

Cohesion ($c$) is another soil parameter whose value is very small in sandy soils and its effect on the whole system behavior is negligible. Consequently, in all the analyses, the cohesion value remained unchanged in the dynamic analyses and it is considered small enough ($c = 1kPa$) in order to have stable numerical results.

Dilation angle ($\psi$) is the other soil plastic parameter, which links shear and volumetric deformations of the soil together. Bolton (1986) pointed out a friction-dilatancy relationship in the following form:

$$\phi_{\max} - \phi_{crit} = 0.8 \cdot \psi_{\max} \tag{2}$$

where $\phi_{\max}$: is the peak friction angle (whose values are defined before) and $\phi_{crit}$: is the soil's critical friction angle that is the friction angle of a loose soil with zero dilation in critical state. Laboratory tests showed that the typical value of $\phi_{crit}$ for quartz sand is 33 degrees (Bolton, 1986). For all the analyses in this study, $\phi_{crit} = 33^{o}$ is assumed. The parameter $\psi_{\max}$: corresponds to the maximum dilation angle observed in the soil behavior during shearing. In this study, based on the introduced friction angles as well as predefined $\phi_{crit}$, the dilation angle ($\psi$) of the Mohr-Coulomb model was measured from this equation.

## Tunnel Lining Model and Interface

Tunnel lining was simulated with two-dimensional elements with three degrees of freedom. These elements behave as linear elastic-perfectly plastic that can simulate reinforced concrete. In each step of the analyses, the numerical model calculates incremental forces and moment in the lining elements, which are induced by gradual displacements and then, by accumulation of increments, total values are measured. Ifaxial stress in a lining element reaches the peak tensile or compressive strength of the introduced material, the element would be marked as cracked and its stress could go no further than material specific residual strength. Tunnel lining is made from precast reinforced concrete segments withunit weights of 2600 kg/m$^3$. These 35-cm thick segments are reinforced by 13 numbers ofD12 steel bars at the top and bottom of each segment. Table 3 presents properties of lining segment materials.

Figure 2 shows the moment-axial force interaction diagram for the concrete lining segments. This chart was constructed based on the segment's geometry and its ultimate bearing capacities as mentioned in Table 3. These parameters (concrete and reinforcing bars) are used to define pure maximum tensile and compressive bearing capacities of a 35m-thick lining and then, these capacities are defined as input to Flac2D code in order to consider the moment-axial force diagram (Itasca Consulting Group, 2005). It is assumed that the tensile strength of the lining section is only due to steel bars while in compression, both the concrete and steel bars can tolerate compression forces. All the points inside the envelop represent combinations of moments and axial forces that lining segments can resist. As can be seen, the maximum moment that can be applied to the segments is approximately 155 kN.m in conjunction with axial compression force of 2600 kN.

Numerical models that simulate the dynamic response of underground structures,normally address soil-structure interaction with the Coulomb law. In 1986, Mueller (1986) presented results from a dynamic experimental test which pointed that interface properties of sand and rough grout are similar to sand. Based on Mueller and other researchers' results, Stevens and Krauthammer(1991) used no special elements or constitutive law to present interface in their hybrid model. Furthermore, Liu (2009) studied dynamic response of subway structures under internal blast loading and did not use any special elements for modeling the interaction between tunnel lining and the surrounding medium. The only means that was used for accounting for the soil-structure interface was reducing cohesion and friction angle of interface elements to 75%. In the present study, the interface between the lining and soil medium was presented without using any special elements or constitutive law and just by reducing cohesion and friction angle of interface element to 75%.

## Modeling of Blast Loading

This section discussesthe procedure of simulating the loading of penetrator bombs. In this study, explosion of a general-purpose bomb known as MK82 (Mark 82) was investigated. MK82 with a nominal weight of 227 kg is a very common non-guided bomb, which can be carried by different aircrafts. This bomb has 87 Kg of Minol or Tritonal explosives. Average TNT equivalent coefficient for these two explosives is equal to 1.26. Therefore, the bomb detonation is as powerful as explosion of 110 kg TNT. Mk82 penetrates about 3 m in sand, if the bomb is released from an altitude of 2750 m (Stipe 1946). In this study, it was assumed that the corresponding camouflet was generated at a depth of 3 m from the ground surface.

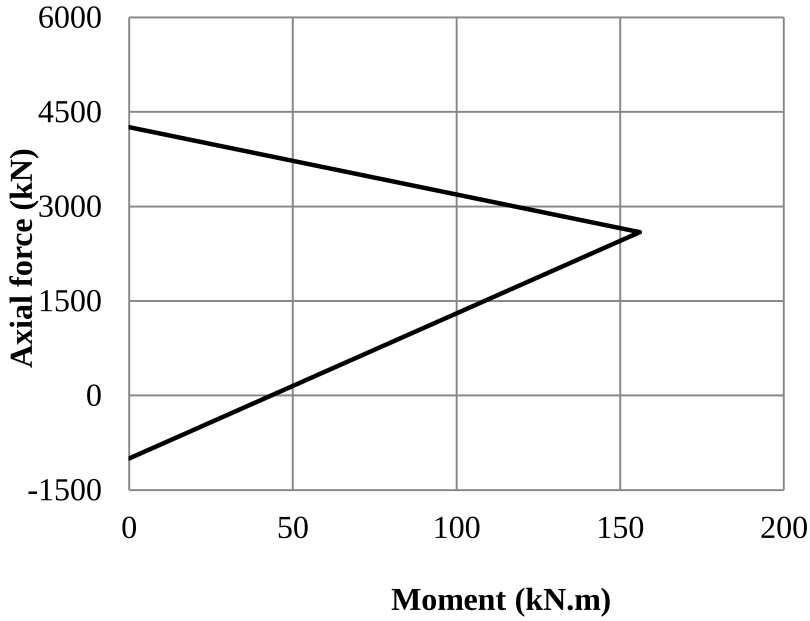


**Fig. 2.** Moment- axial force interaction diagram for concrete lining segments: Negative value for the axial force represents tension and positive value means compression.

**Table 3.** Material properties of concrete lining segment

| $\upsilon_{conctrete}$ | $E_{concrete}(GPa)$ | $f'_{concrete}(MPa)$ | $\upsilon_{Steel}$ | $E_{Steel}(GPa)$ | $f_{y_{Steel}}(MPa)$ |
|---|---|---|---|---|---|
| 0.2 | 28.2 | 30 | 0.3 | 200 | 400 |

In order to predict the loading of penetrator bombs, their explosion type has to be specified. Explosions are different and can be categorized based on their various specifications. A useful categorization of explosions divides it into two branches of unconfined and confined explosion. Each of these types has three subcategories. Free air, air, and surface burst are different types of unconfined explosion. Fully vented, partially confined and fully confined are three subcategories of confined explosions (US Army Corps, 1969). Peak pressure, the time taken to reach peak pressure and pressure dissipation time of these different blasts are completely different. Investigations show that the burst of penetrator bombs are partially or fullyconfined explosions.

A surface burst or a shallow explosion creates a hole in the ground, which is calleda crater (partially confined explosion). Camouflet is a small cavity that forms when an explosion occurs deep enough to be entirely confined in the soil. The amount of explosive, bomb surrounding materials and its penetration depth arefactors that control the formation of a crater or camouflet (Bulson, 1997). The TM5-855-1 manual suggests that explosion of 110 kg TNT at 3 m belowground level is completely confined. Therefore, a camouflet is formed by explosion of the MK82 bomb. The camouflet's dimension has not been investigated in academic researches yet and there is no method for estimating its size. On the other hand, some investigations studied crater size and suggested formulas for predicting dimensions of craters formed by explosion (Bulson, 1997). In the present study, it was assumed that the volume of the camouflet generated by a penetrator bomb be equal to that of a crater generated on the surface. Based on this assumption and relationships of predicting crater size introduced by Walley (1944), an estimation of camouflet diameter was reached. Explosion of 110 kg TNT detonated 3 m below soil surface creates as pherical camouflet with diameter of 3 m (as shown in Figure 1).

Numerous researches tried to calculate the penetrator bombs loading. Most of these have investigated free field pressure, which is the pressure induced by a penetrator bomb in a semi-infinite soil medium without any underground structure. Pressure of a confined explosion adjacent to an underground facility would be distinct from free-field pressure, because of the waves' reflections within underground structures. Lampson (1946) initiated an investigation to predict free-field pressure of penetrator bombs. Researches in this regard were conducted until the first years of the 80's, when a series of full scale tests on penetrator bomb with different strength had been carried out by US Waterways Experiment station. Drake & Little (1983) used the results of these tests to come up with empirical formulas for calculating free-field pressure of penetrator bombs detonation. Westine and Friesenhahn (1983) also suggested a method for estimating free field pressure of penetrator bomb in saturated and unsaturated soils. Drake and Little's (1983) method was mentioned in TM5-855-1. In the other editions of this design manual published in 1990 and 2008, readers were advised to use TM5-855-1 for designing underground structures (US Army Corps 1969, 1990). These formulas were also cited in papers of other researchers (Gui and Chien, 2006; Yang, 1997). Eq. (3) shows Drake and Little formulas for predicting free-field pressure of penetrator bomb:

$$P = P_i . e^{-\frac{t}{t_a}} \tag{3a}$$

$$P_i = 160. f .\rho .c.\left(\frac{R_i}{W^{0.33}}\right)^{-n} \tag{3b}$$

where $P$: is the free-field pressure created by a penetrator bomb during the time. $P_i$: is peak free-field pressure of the bomb. $P$ and $P_i$: are in terms of psi. $t$: is the time (second) after detonation when the pressure is being calculated. $t_a$: is the arrival time (second) of detonation waves to a point where pressure is being calculated and is equal to $\frac{R_i}{c}$ where $R_i$: is the distance in terms of foot between detonation center and point of calculation, which is equal to the radius of camouflet ($R_i$ = 1.5 m in this study). $c$: is the seismic wave velocity (foot/sec) of the soil. $\rho$: is mass per unit volume of soil medium ($lb/ft^3$). In Equation (3b), $f$: is called ground shock coupling factor whose value is defined as a factor of scaled depth of explosion. $W$ (in terms of pound): is the weight of the explosive (if the explosive is not TNT, then TNT equivalent weight has to be calculated and used instead) and $n$: is Attenuation Factor. The product of $\rho.c$ and $n$ can be estimated for different situations based on the charts and figures provided by Drake and Little (1983). According to Drake and Little(1983) for dense dry sand, the parameters are selected as $n = 2.75, \rho.c = 25 psi / ft/\sec, c = 870(ft/\sec)$, and $f = 1$. The arrival time ($t_a$) is then measured as $t_a$ = 5.75$ms$. It was also noted that the free-field pressure of the bomb reaches peak pressure linearly in time equal to $\frac{t_a}{10}$. Figure 2 presents the time history of the induced pressure from the bomb blast (calculated based on Eq. (3)), which is applied over the periphery of the camouflet.

## Analysis Procedure

First and foremost, it should be noted that the accuracy of numerical modeling of blast loading was investigated and verified in a similar procedure performed by Nagy et al. (2010). In this procedure, a blast loading was applied in the middle part of the soil medium without the existence of any underground structure; thus, a free-field pressure was generated in the soil medium. The model accuracy wasexamined by comparing the measured maximum pressure at specified distances from the blast point with the peak free-field pressure calculated from TM5-855-1 (similar to Eq. (3b)). For the present study, two different grid sizes including 0.5*0.5 m and 1*1 m were considered for numerical investigations. The results of the numerical simulations of peak free-field pressures at different scaled distances ($R/W^{0.33}$) are presented in Figure 4. As can be seen, the obtained results for the scaled distances before $R/W^{0.33}=1$ are different while the numerical results are coincident well with that obtained from TM5-855-1. Since the grid size of 1*1 requires less calculation time and the accuracy of the results are sufficiently enough (because $R/W^{0.33}>1$ for the present model), this grid size is selected in all numerical analyses. Asimilar investigation has been performed to obtain the damping parameters of the numerical model. More explanations in this regards is out of the scope of this paper and extra information can be found in Seyedan (2014).

All the simulations were carried out in three steps. First, the soil medium was loaded by gravity force. Therefore, *in-situ* stresses were induced in the model. In the second step, the underground structure was excavated and lining elements were added to periphery of the tunnel. In this step, displacements, lining forces, moments and deformations were calculated. Finally, in the third step, explosion of penetrator bomb was simulated by applying a time dependant value of normal pressure (as shown in Figure 3) over the periphery of the camouflet. Referring back to Figure 3, the effected duration of the blasting is less than 6ms; however, the required time to trace the dynamic response of underground structure should be longer because of wave propagations and reflections in the medium. In this study, the numerical simulations were taken into consideration to be 50 ms with dynamic time increment of $2.5*10^{-6}$ second during the dynamic analyses.

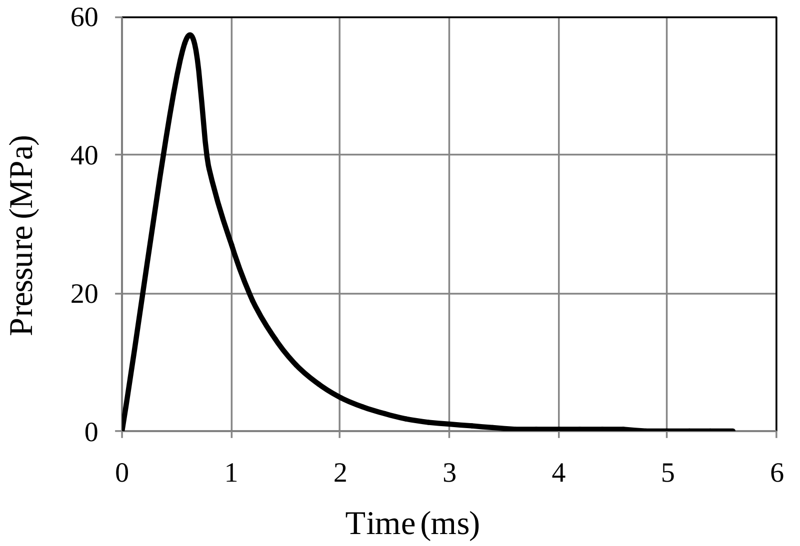


**Fig. 3.** Variation of induced pressure from blast loading in the camouflet with time

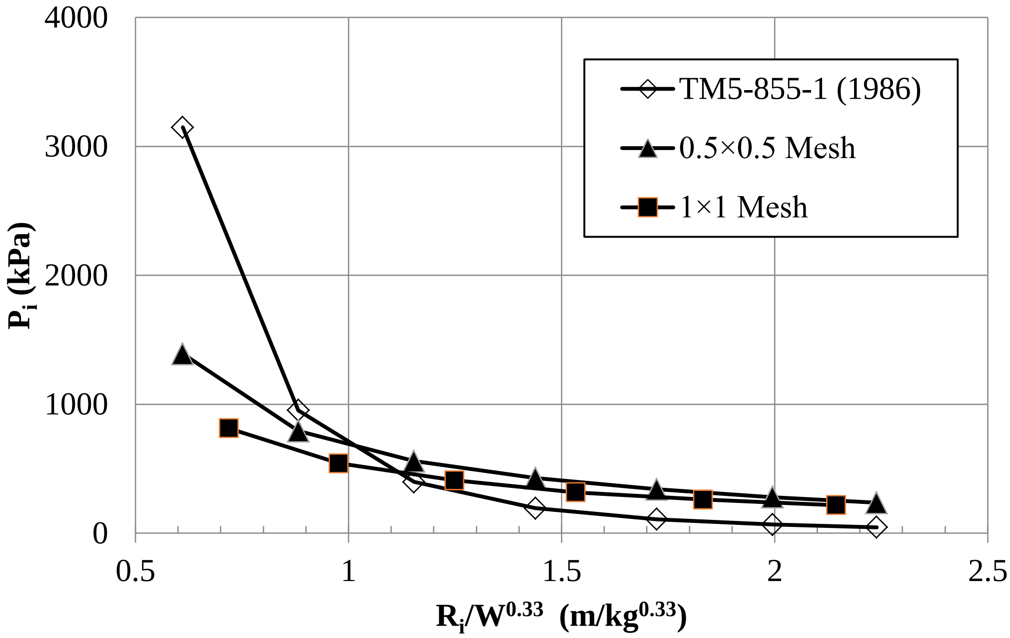


**Fig. 4.** Comparison of peak free-field pressure induced by blast loading from numerical method and that introduced by TM5-855-1 for two different grid sizes

## RESULTS OF SIMULATIONS

### General Response

Figure 5 shows the variation of lining parameters at the tunnel crown after the bomb explosion in analysis No. 5. After the bomb detonation, blast waves were propagated through the soil medium and a fraction of them reached the nearest tunnel point i.e. tunnel crown in about 10 ms. These waves increased the pressure in the soil adjacent to the tunnel crown dramatically from 100 kPa to more than 400 kPa (Figure 5a). After the peak value, the pressure returned to the initial value at a similar rate. In addition, large displacements as well as extra axial force, bending moment,and shear force were induced in the lining alongwith a rapid rise. According to Figure 5b, large and residual displacements were imposed on the tunnel lining (about 0.3 m), while the values of axial force (Figure 5c), moment (Figure 5d) and shear force (Figure 5e) of the lining returned to the initial value after about 40ms. Among these, the rise in shear force of the lining only occurred in a fraction of time, while the distortion time of other parameters had taken more than 20 ms. The other point in the behavior of the lining is that the peak value of the parameters was reached at different times (vertical displacement in 48 ms, axial force in 25 ms, bending moment in 16 ms, and shear force in 30 ms).

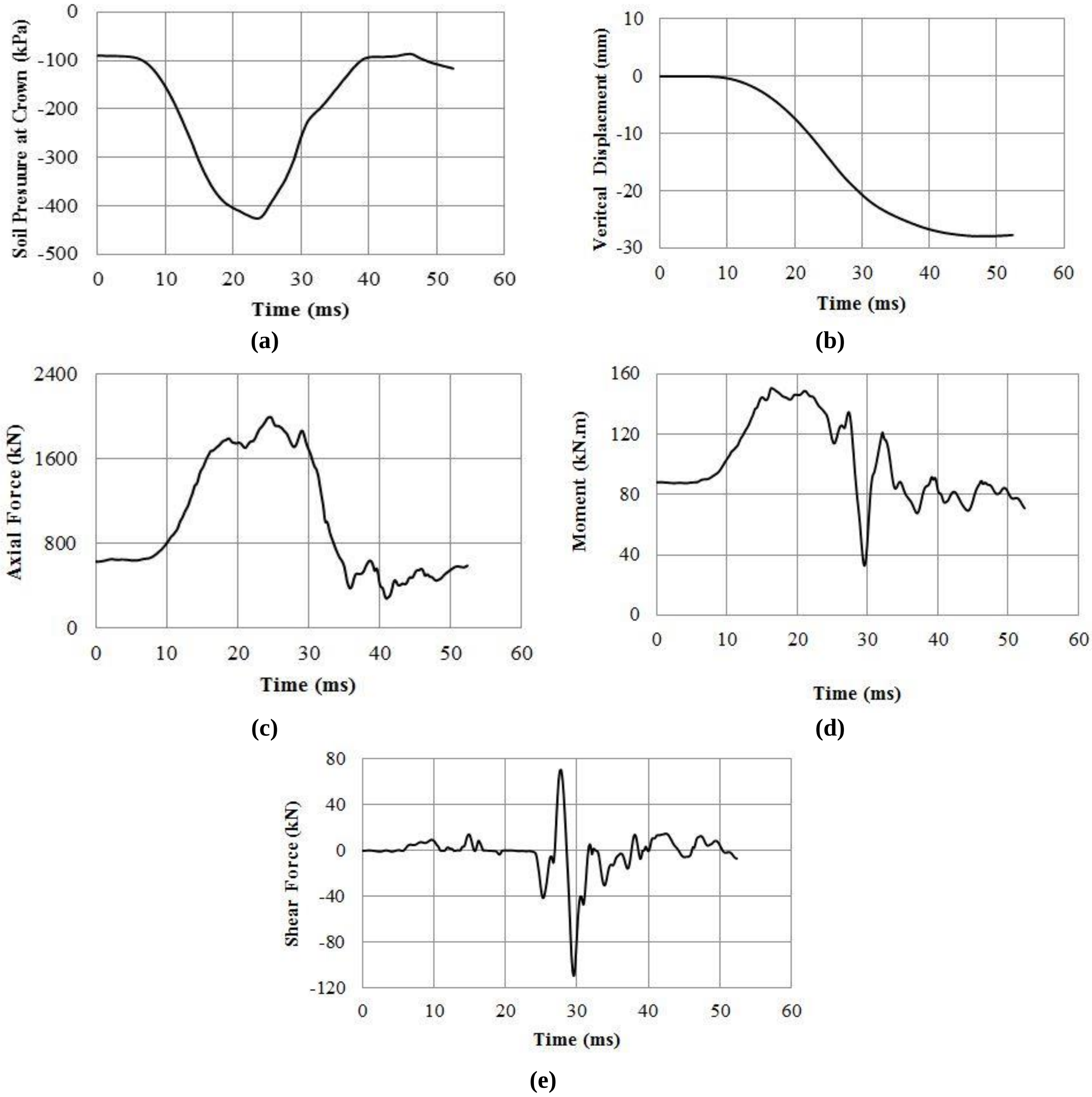


**Fig. 5.** Time history responses of different soil and lining parameters at tunnel crown in analysis No. 5:(a) soil pressure at crown; (b) vertical displacement; (c) axial force; (d) moment; (e) shear force.

## Soil and Lining Parameters at Tunnel Crown

Changing the soil plastic parameters in simulations, affects the blast-induced pressure in soil adjacent to underground structure. As can be seen in Figure 6, increments of plastic parameters result in reduction of peak soil pressure. In other words, the denser the soil, the lower the peak induced pressure from the blast. The maximum pressure induced in the medium soil (with parameters corresponding to $\phi =$

$35^o$) is about 750 kPa, while it reduces largely to about 420 kPa in the very dense soil (with the parameters corresponding to $\phi = 45^o$).

Residual displacement of the lining at the crown also depends on the plastic soil's parameters, i.e., soil densification. Figure 7 shows maximum vertical displacement of the tunnel crown lining in the simulations. It can be seen that the predicted deformation of the tunnel is also sensitive to the selection of the soil plastic parameters. It should be recalled that the Young's modulus of the different soils were assumed constant for all the analyses. For the soil with $\phi = 35^o$, a vertical displacement of 0.33 m is predicted but was reduced to about 0.28 m for very dense soil ($\phi = 45^o$), which indicates about 16% tolerance in prediction.

Figure 8 illustrates the maximum magnitudes of axial force and moment of tunnel lining at the crown for all five numerical analyses. It can be seen that the maximum moment remained unchanged in all the analyses, while the soil compaction influenced the axial force of the lining. This should not be interpreted as a sign that the variation of plastic parameters does not affect the maximum lining moment, since the maximum moments obtained in Figure 8a are almostequal to the maximum capacity of lining segments as shown in Figure 2. In other words, in all the analyses, the bending moment reached its ultimate capacity in the interaction with the axial force. Variation of maximum axial force in Figure 8b shows that this parameter can be changed in a small range about 13% along with different estimation of soil compaction. The denser the soil, the lower the maximum axial force. It can be noted that in all the analyses, maximum axial force of tunnel lining was almost 50% of lining compression capacity. This means that the tunnel lining behavior is the most influenced by the moment rather than the axial force.

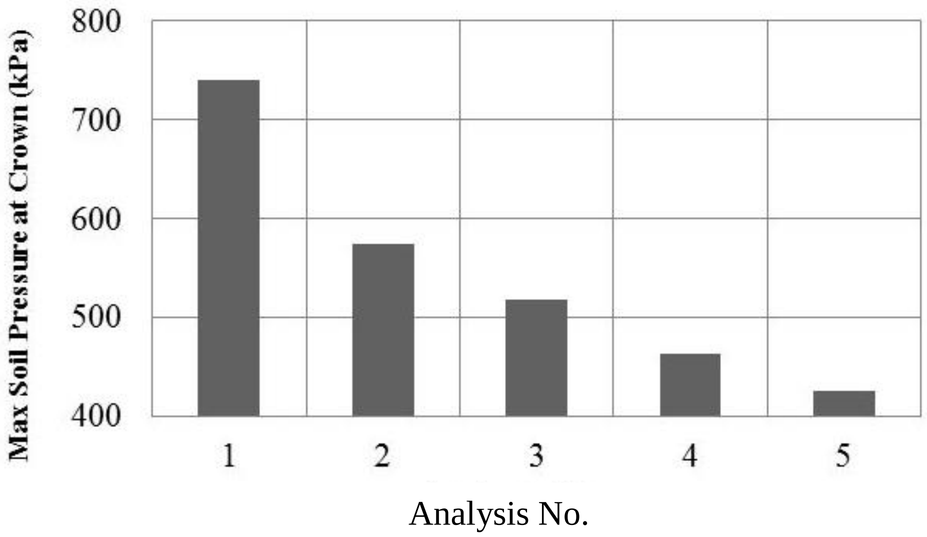


**Fig. 6.** Maximum pressure induced in the soil adjacent to the tunnel crown in different analyses

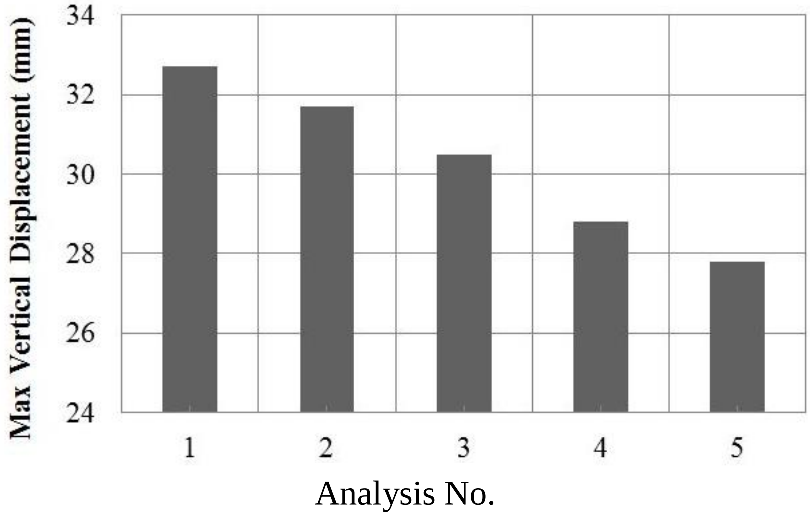


**Fig. 7.** Maximum vertical displacement of tunnel crown lining for different analyses

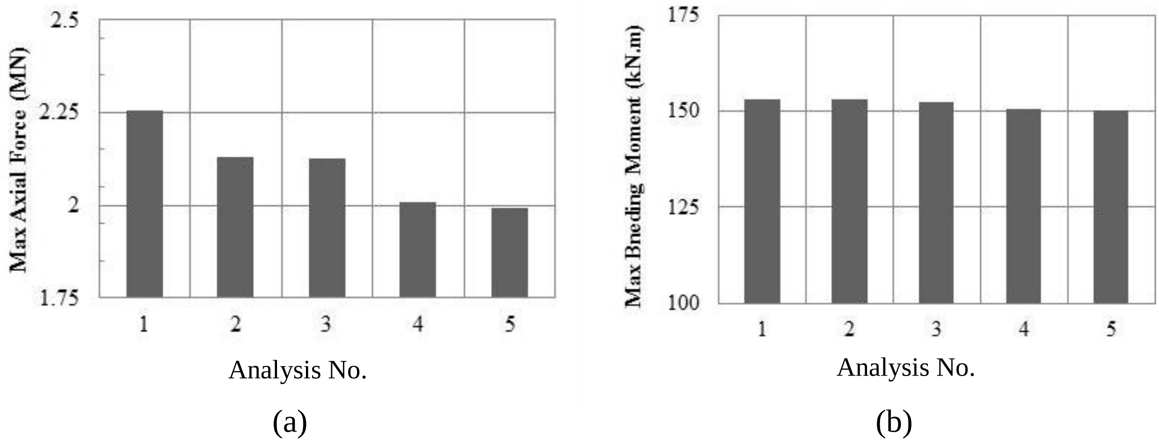


**Fig. 8.** Presentation of maximum values of (a) axial force; (b) bending moment of tunnel crown lining for different analyses

## CONCLUSION

Using finite difference method, the response of underground structures to explosion of penetrator bomb was investigated. A series of simulations were carried out in order to study the significance of soil plastic parameters. Actually, these parameters are related to the soil compaction state. In the analysis, the well-known and practical linear elastic-perfectly plastic Mohr-Coulomb model was adopted as soil constitutive model. This model has totally five parameters including two elastic parameters ($E$, $\nu$) and three plastic parameters ($\phi$, $c, \psi$). Among the parameters, elastic ones were kept constant and the plastic parameters were changed with the so called Bolton friction-dilatancy relationship. The parameters were selected forsilty sand. The tunnel properties were chosen similar to those of Mashhad Subway tunnel.

It was found from numerical analyses that the soils with higher plastic parameters, which correspond to higher degrees of compaction, play more adequate roleof refuge against blast loading situations. Blast waves expand with more energy to propagate through these soils. Therefore, underground structures buried in soils with higher densifications are more protected from the threat of external explosions. It can be said that the performance of underground structures in soil media with higher degrees of compaction would be better and safer.

Analysis also revealed that due to the blast loading, large and residual displacements can occur in the tunnel lining. Based on the axial force-moment interaction diagram of the lining, it was found that bending moment is the other most critical parameter of the tunnel lining when underground shelters encounter blast loading. Linings of underground structures would fail due to its excessive bending moment, which is induced by detonation of penetrator bombs. Therefore, it can be suggested that lining of underground structure be designed based on this parameter. Shear force in the lining only increases in a small portion of time and it is not critical.

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