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Dynamic and Static Analysis of Circular Tunnel with Special Focus on the Hydro-Mechanical Coupling Behavior of Soil Leila Nikakhtar* and Shokrollah Zare**

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Keywords: Dynamic loading FLAC^{2D} Hydro dynamic coupling Tabriz Metro Line 1 MCE level One of the most important parts of a tunnel are the supporting systems, which must be sufficiently resistant to loads during the life of the structure. Critical loads on the tunnel support system are ground and water loads, that are usually calculated separately by analytical methods. These loads have coupled hydro-mechanical behavior in the materials around the tunnel and their effects on the structure should be considered as a connection phenomenon. Therefore, in this study, using two-dimensional finite difference method in FLAC software, a static design of the tunnel was performed, in which the water was initially considered only as the pore pressure, and subsequently, static analysis was carried out completely in the form of coupling. One of the twin tunnels of Tabriz Metro Line 1 was used as a case study. For these two states, axial force and bending moment were evaluated and compared. In order to investigate the tunnel behavior under dynamic load, the real Duzce earthquake at the MCE level was selected. The axial force and bending moment in pure dynamic state and hydrodynamic coupling using Byron behavioral model were evaluated. The results showed that in the static coupled design, higher axial force of about 27% and lower bending moment of about 36% were obtained compared to the no-coupled mode. Also in the dynamic coupled design, more axial force and lower bending moment, about 25% and 66% respectively, were seen. Therefore, it can be concluded that it is best to use this state of structural loading for the detailed design of reinforced concrete structures.

1. Introduction

The construction of underground structures, including tunnel excavation, affects the natural hydrodynamic behaviour of underground system. The tunnel, as a drainage structure, changes the water table. The coupling of stress and pore pressure has significant effect on the soil behaviour around the tunnel, the performance of the tunnel support system, and also the amount of loading on them.A few studies were done related to this topic. Some of the researchers have investigated the impact of tunneling on the groundwater regime in the surrounding environment [1, 2]. Schweiger et al studied the influence of seepage forces on the lining of a large undersea cavern [3]. Also Gunn and Taylor numerically investigated the steady state stress and seepage, but in this study, the fully coupled interaction behaviour between the tunnel construction and the underground water were not modelled [4].

Abstract:

Water pore pressure affects the geotechnical environment around the tunnel by preventing water line expansion and changing the stress field due to the decrease in effective stress. Analytical studies have been conducted in this field [5-8]. In addition to analytical methods, numerical methods have also been used to analyse the effect of pore pressure on the behaviour of the tunnel and its supporting system [9-11]. In recent years, the constructors and designers have been faced by complicated geological conditions for the construction of underground tunnel. The stability of tunnel structure is affected by stochastic vibration loading of earthquake, explosion, and oscillation, especially in conditions of groundwater in sediments. Therefore, investigation of the issue of hydrodynamic pressure and its effects on tunnel support system is required.

The behaviour of underground structures during an earthquake is one of the most interesting challenges in geotechnical engineering. Underground tunnels usually performed better than surface structures during earthquakes, but there are some reports of damage to some of these important structures during an earthquake (the 1995 Kobe, Turkey earthquake1999, the 2008 Sichuan and the 2001

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Valparaiso, Chile earthquake), which indicates the need to account for seismic loading in underground structures design. The effect of earthquakes on underground structures can be divided into two categories: ground shaking and ground failure [12].

Ground shaking is the vibration of the ground created by seismic waves that spread through the crust of the earth. Seismic motion causes three types of deformations in the underground structures (Axial, bending and ovaling or racking deformations). The oval deformation or racking in the tunnel occurs when shear waves are transmitted to the axis of the tunnel normally: A circular tunnel becomes oval and a rectangular tunnel is racking. The component that has the most influence on the tunnel lining behaviour under seismic loading, except in the case of a tunnel cut by a fault, is ovaling or racking deformation that is caused by seismic shear wave or S wave propagation [13, 14].

A diverse range of approaches have been used for the study of tunnels behaviour under seismic loads: empirical and analytical methods, physical model tests and numerical modelling.

The structural forces induced in a circular tunnel lining due to a seismic load have been determined by various elastic closed-form solutions, due to their simplicity [12-14]. These solutions have assumptions that limit them, such as soil and tunnel lining assumed to be linear elastic, uniformed circular tunnel without joints and so on[12].

So, physical model tests and numerical analysis have been used to gain a better understanding of the physical problem, especially the phenomenon of soil-structure interaction and the performance of underground structures. However, because of the complexity and high costs of these tests, the results obtained are still limited [15-17].

In most numerical simulations, seismic loads are considered quasi-static, and since these loads are calculated by the time history dynamic analysis or full dynamic analysis, it is the most complex level of seismic analysis that has been used in this study. This type of analysis is also the most accurate method and generally numerical. However, the downside to it is the long calculation time, which limits the applications of full dynamic analysis [18-21].

Liu and Song stated that in undrained conditions, increasing the pore pressure may be a source of damage to the underground structures, and if the amount of excess pore pressure reaches effective stress, liquefaction occurs and the underground structure will be damaged [22]. Reports indicate that some of the California tunnels became susceptible to buoyancy in the Lama Prieta (1989) earthquake [23]. On the other hand, sometimes even nonliquefiable soils have large and asymmetric deformation during an earthquake. Geological studies in different regions show that new deformations and forces occur in this area. According to monitoring data, linear underground structures such as tunnels may be demolished by the increasing pressure. Therefore, tunnel design should be executed considering this phenomenon. There have been many reports of damage caused by increased pore pressure in the soil around underground structures. Azadi and Hosseini investigated the effect of liquefaction on shield tunnels. They numerically investigated the impact soil liquefaction on forces and bending moment tunnel lining [24]. Unutmaz investigated the effect of tunnel diameters, depth of tunnel canter, support thickness of tunnel and the strength of soils in circular tunnel behaviour under the cyclic loads [25]. Chian et al studied the impact of soil liquefaction on the uplift of underground structures by constructing numerical and physical models [26]. Zheng modelled uplift displacement of tunnel caused by earthquake by finite difference method. They also used the multivariate adaptive regression spline model regression method to predict this behaviour of tunnel and estimate underground structure floatation in terms of structural specifications, soil and earthquake parameters[27].

According to these studies, loads on underground structures are considered as one of the most important causes of damage to these structures and should be considered more carefully while designing them. So, this study has focused on the existence of water in the surrounding soil of tunnel and its impact on the amount of load on tunnel lining. The study consists of two parts:

In the first part, the hydro mechanical behaviour of the surrounding soil was numerically investigated in static condition, and the effect of two approaches including fully coupled hydromechanics and the presence of water only as the pore pressure at the support system loading, was evaluated.

In the second part, the soil and structure behaviour was modelled in dynamic conditions, and the amount of the loading on the tunnel support system was compared for fully coupled hydrodynamic and pure dynamic analyses.

-Types of hydro mechanical coupling

In porous media such as rock and soil, the interaction between hydraulic and mechanical processes is called hydro mechanical (HM) coupling. In general, there are three algorithms for multi-physics simulations of the hydro mechanical process, which include full coupling, loose coupling and one-way coupling. A fully coupled simulator is an individual set of equations such as large system of nonlinear coupled partial differential equations and relevant physics that must be solved.

This type of H-M coupling has the most realistic results and is often a good method for simulation, but it is overly complex due to the solution of nonlinear equations involving inelastic environments and multiphase flow and is often feasible with some simplifications [28]. The other approach is one-way coupling where two sets of totally separate equations (including mechanic and hydraulic) are solved concurrently. Then the information is conveyed in only one direction. In other words, the outputs of one simulator are periodically transmitted as input to another. For instance, the pore pressure of the hydraulic computations is sent to calculate the mechanical stresses, strains and displacements [29].

The third type is loose coupling, in which two sets of equations are solved independently, and information is transmitted at defined intervals in both directions between fluid flow and geo-mechanical simulators. Although loose coupling has the advantage of simple implementation, such as one-way coupling, it is useful for estimating more complex nonlinear physics, in that it is closer to a fully coupled approach[30].

-Fin &Martin and Byron Models

The FLAC software is capable of performing fluid and dynamic analysis in porous environments simultaneously and fully coupled. By default, the pore water pressure reacts to the volume changes caused by dynamic loading, but the average pore water pressure remains constant during the analyses. There are various models for calculating the pore water pressure, but most of them are not suitable, because they require the results of specific laboratory or field tests. The software uses the modified Mohr-Coulomb failure criterion and the Finn and Byron models to perform this type of analysis. These models combine two equations and correlating the volumetric strain induced by the cyclic shear strain and excess pore water pressure produced during cyclic loading [31].

The following empirical relationship that correlates the decrease in volumetric strain ($\Delta \epsilon_{vs}$) of soil per cycle of shear strain with the magnitude of the cyclic shear strain amplitude (χ) is proposed by Martin et al.

$$\Delta \varepsilon_{vs} = C_1 (\gamma - C_2 \varepsilon_{vs}) + \frac{C_3 \varepsilon_{vs}^2}{\gamma + C_4 \varepsilon_{vs}}$$
(1)

Where C1, C2, C3 and C4 are constants and are interdependent of each other as follow:

$$C_1 C_2 C_4 = C_3 \tag{2}$$

Byron (1991) introduced an alternative and simple formula for calculating the volumetric strain of soil mass:

$$\frac{\Delta \varepsilon_{vs}}{\gamma} = C_1^b \exp\left(-C_2^b \left(\frac{\varepsilon_{vs}}{\gamma}\right)\right) \tag{3}$$

 $C_2^b = \frac{0.4}{C_2^b}$ and C_1^b can be obtained by Eq (4) from relative densities (Dr) and

$$C_1^b = 7600(D_r)^{-2.5} \tag{4}$$

Also, this constant can be determined by the rate of the normalized standard penetration test $(N_1)_{60}$ according to the Eq (5):

$$C_1^b = 8.7 (N_1)_{60}^{-1.25} \tag{5}$$

2. Statically investigation

2.1 Case study

In this study, the twin tunnels of Tabriz Urban Railway (TUR) located in Tabriz, the capital of East Azerbaijan Province of Iran were selected as a case study (Fig 1). These tunnels are located near the North fault of Tabriz, which is a right-slip fault geologically. Historically, the existence of this fault has led to a severe earthquake in the Tabriz region. The seismicity of this region is classified as VII to X on the Mercalli intensity scale [18].



Fig. 1: Location of Tabriz in Iran map

Geologically, the city of Tabriz is composed of formations of clay, sandstone, conglomerate, tuff, gypsum and alluvial sediments such as clay, marl, sand, sand and rubble with different classifications of grain size. A length of 7.2 km on line1 of the TUR tunnels was excavated by two TBM-EPB machines. This line starts from Station 7 in SE-NW until central Station 12 and connects to Station 19 in NE-SW (Fig 2). The cross section of the tunnel studied in this article is 13 station (Golestan Garden).

2.2 Numerical model

Finite difference method by applying FLAC^{2D} software was used for numerical analysis in this study. FLAC as a wellknown software can be used for analyzing, testing, and designing the geotechnical, civil and mining projects. The software can model fluid flow (e.g. underground water) inside a permeable solid such as soil. It can perform fluid flow independently of the usual mechanical calculation, or it may perform simultaneous mechanical modeling to obtain fluid/soil interactions [31].



Fig. 2: Tabriz Urban Railway line 1 map (The research location is marked by star).

HM coupling between liquids and porous solids has been known for over a hundred years [32, 33], but for the first time, in the 1920s, Karl Terzaghi tried to understand two apparently separate phenomena [34]. He made two principal ideas including effective stress and the fluid pressure diffusion of flow. This effective stress is described by the following relation:

$$\sigma_{v} = \sigma_{v} - p \tag{6}$$

Where $\sigma v'$ is the vertical effective stress or the portion of the vertical stress acting to compress the porous matrix, which is the difference between the applied load σv and the pore pressure p. Terzaghi also applied a diffusion-type equation to identify the dissipation:

$$\frac{k\partial^2 p}{a\partial v^2} \approx \frac{\partial p}{\partial t}$$
(7)

Where "k" is permeability, "a" is the factor of compaction and "p" is "hydrostatic overpressure in the pore water". This is Terzaghi's principle law in HM coupling and fluid flow through porous media [34]. In the case of underground structures, an increase in total stress may occur with increasing depth and drop in water pressure due to tunnel drainage [8].

For numerical static analysis, a model with 70 *50 dimensions was constructed. The excavation of the tunnel was in full face cross section with EBP TBM. The dimension of each mesh is 0.5 m, therefor total 14000 meshes were applied in this model (Fig 3).



Fig. 3: Mesh of model

The height of underground water from the ground surface was assumed to be 5.3 meters. As hydraulic boundary conditions, the top of the model was considered as the seepage boundary, so the water pressure at this boundary is zero, and also the bottom, left and right sides of the model are fixed as impermeable bounds. The geotechnical parameters of surrounding soil around the tunnel are shown in Table 1. The pour pressure distribution before the tunnel excavation is shown as Fig 4.

These assumptions were applied in numerical model: 1- soil is homogeneous, isotropic and continuously porous. 2-The head of the underground water is constant and 3-according to Darcy law, water flow is stable. The model parameters were estimated by considering these factors according to Table2.

Table.	1:	Physical	and	mechanical	parameters

Parameter	Value
Module of elasticity (MPa)	23
Poisson's ratio	0.35
Friction angle (°)	32
Cohesive force (kPa)	10
Density (kg/m3)	2150
Permeability (m/s)	1×10 ⁻⁵
Water head (m)	5.3
Tunnel depth (m)	12



Fig. 4: Distribution of pore water pressure before excavation

Supporting system in the mechanized tunneling with Earth Pressure Balance method is precast concrete segmental lining. In this model the tunnel support system is precast impermeable reinforced concrete with a thickness of 30 cm and a width of 140 cm. Each ring of the lining include 6 pieces. The properties of segmental lining are shown in Table 3.

The back filled grouting was ignored in the numerical model. In order to consider the effect of joints on segmental lining, the bending moment reduction factor 0.7 that it was calculated with Lee analytical method was used and lining was modeled as a uniform ring [35]. To validate the

numerical model, real data from Tabriz metro line1 was used. The estimated surface settlement with the model was in the good agreement with the surface settlement recorded in this area and the validity of the numerical model was confirmed.

Table. 2: HM Parameters of the model						
Parameter		Value				
Water bulk module (Pa)		1×10^{6}				
Porosity		0.5				
Soil dry density (Kg/m ³)		2510				
Soil saturated density (Kg/m ³	⁵)	2640				
Model characteristic length (1	n)	70				
Smallest zone characteristic		0.5				
length (m)		0.5				
Table. 3: Properties of	shield and	d segment				
Parameter	Shield	Segment				
Density (Kg/m ³)	7850	2500				
Young's modulus (GPa)	210	30				
Poisson's ratio	0.17	0.2				
Thickness (m)	0.1	0.35				

2.2 Results and discussion

The interaction between tunneling and groundwater has been investigated in two types of states, including existence of water only in the form of pore pressure(that in this study was named Just Pore Pressure as state 1), and fully coupled conditions in the numerical modeling (state 2). An important factor in the design of reinforced concrete structures is the amount of bending moment and the axial force on the structure, which is influenced by the amount of horizontal and vertical loads, and how these loads affect the structure. The maximum axial force and bending moment to the tunnel support in these two states are shown in Fig5 and Fig 6.

By comparing the results of these two states, it can be seen that the fully coupled tunnel analysis produces a higher axial force on the support system. As shown in Figure 7, this increase in axial force was about 26.7%. According to this diagram, the bending moment in the fully coupled state decreases by 36.2% compared to state 1, and the bending moment distributions also differ in both states.

3. Dynamical model with Earthquake

In this study, a modified acceleration of real earthquake was used for dynamic loading. Since geotechnical data and tunnel characteristics are in line with the Tabriz metro line1, the recorded acceleration in earthquakes that are most consistent with the seismicity and geology of the site were selected. So, the acceleration of the Duzce was chosen after the relevant reviews. On November 12, 1999, an earthquake known as the Duzce earthquake, struck with a moment magnitude $(M_W) = 7.2$ in Turkey [36]. Two railroad bridges and an underground tunnel were severely damaged. The earthquake is classified on the MCE (Maximum Credible Earthquake) level with 0.62 g. But this value differs from the maximum acceleration obtained from earthquake hazard analysis for the MCE earthquake in site which is 0.57 g. Therefore, it is necessary to scale the base acceleration relative to the desired values. The scale coefficient is defined as the ratio of the maximum target acceleration to the maximum base acceleration.



Fig. 5: Axial force and bending moment on the lining in state 1



Fig. 6: Axial force and bending moment on the lining in state 2



Fig. 7: Results of two modeling states

To do this, using Seismo-Signal software, earthquake frequency is converted to frequency domain using Fourier transform of time domain. After selecting the maximum target frequency and filtering the earthquake accordingly, by reversed Fourier transform, the improved acceleration history is obtained (Fig 8). In this study, in order to prevent model distortion and not increase the number of model meshes, the frequency of 25 Hz was used for the filter.

3.1. Dynamic numerical model

It is necessary to make some changes to the calibrated FLAC^{2D} model for dynamic modeling. These changes include applying new boundary conditions, dynamic loading and damping system. Undoubtedly, the behavioral parameters of the soil and the support system material during dynamic loading are distinct from their static values, but since no information is available in this area, the same values have been used in dynamic model. Dynamic analysis was performed in non-slip conditions.

3.2. Boundary condition

Dynamic boundary conditions according to Fig 9 include free boundaries at the sides of the model and viscous boundary at the base of the model and at the base of the free boundaries. These boundaries are connected to the model using horizontal and vertical dampers. The boundaries of the free field provide conditions in which the propagation of waves in the distant is not affected by the structure. In order to prevent reflection of waves into the model, the viscous boundaries of the model are used. As a result, seismic waves are reflected below after collision with the upper boundary of the model, which is absorbed by the dampers after impact with the base of the model, and is prevented from being reflected again. There is a similar trend in the free-field column.

3.3 Dynamic loading

During the earthquake, body waves from the source are propagated in all directions. When these waves reach the boundary of the layers, they reflect and refract. Refraction occurs as the velocity of the waves passing through the layers near the surface is usually lower than the layers below. This phenomenon causes the earthquake waves to propagate in vertical direction in the horizontal layers.



Fig. 8: Acceleration, velocity and displacement curves after filtering

Based on this fact, the dynamic loading is considered as a shear plane wave propagating from the base of the model in an upright direction. Therefore, it is necessary to convert the history of speed into the history of stress. Assuming a plate wave, this conversion is as follows:

$$\sigma_{vv} = -2(\rho C_s v) \tag{8}$$

Where v is the velocity of the earth's motion of Duzce earthquake in Fig 10 and Cs is the shear wave propagation velocity for the site, which is considered 228.62 m / s. ρ is the mass density [31].Coefficient 2 is because half of the stress input is absorbed at the viscous boundary and only half of the actual dynamic load value is applied to the model. The final earthquake loading is in accordance with Fig 10 of the model. According to Fig 10, the loading time is 26 seconds, which is longer than the velocity time in Fig 8 due to the zero displacement at that moment. Failure to pay attention to this leads to an induction of permanent displacement in the model that is essentially unrealistic.

3.4. Rayleigh damping

Rayleigh damping in FLAC^{2D} software is characterized by two dominant frequency values and critical damping ratio. The damping ratio is frequency dependent, but in some frequency bands, it can be found that the critical damping ratio range is independent of frequency. This frequency can be determined by a combination of the central input frequency and the normal frequency of the system.

The central frequency of the Duzce acceleration is 6.51 Hz. To determine the natural frequency of the system, the force of gravity is applied suddenly to the model in a no damping state. According to the Fig 11 the natural frequency of the system is 1 Hz. So, the dominant frequency 3Hz that is between these two values was chosen for the Duzce acceleration.



Fig. 9: Boundary conditions and dynamic loading of the calibrated model in FLAC^{2D} [31]



Fig. 10: Time history of Duzce earthquake loading

The critical damping ratio is the second parameter to be determined. This ratio is lower in rock than in soil .Although no measurement has been made in this area, based on experience, 2 to 5% for the environment and 2 to 10% for the support system are appropriate [48]. In this model 5 % for the environment and 2 % for the support system is considered.



Fig. 11: Vertical displacement time history for the model under free vibration conditions

3.5. Wave propagation in the model

One of the problems that can occur during dynamic modeling is the distortion and inadequate propagation of the wave in the model. This phenomenon usually occurs due to the large dimensions of the elements and incorrect boundary conditions and causes the waveform to differ greatly from the initial loading. So, a simple model without excavating space was constructed, and linear elastic behavior was chosen for the materials. By applying Duzce acceleration to this model, the velocity history at the base of the model and the base of the free field for the earthquake in Fig 10 show that the wave propagation in the original model and the free field column are properly simulated, and no distortions are present. The maximum velocity at the base of the model is 93 cm / s (Fig 12). The maximum velocity recorded at the

earth's surface was 144 cm/s, with little change due to wave reflected from the model surface (Fig 13). To validate the numerical dynamic model, Wong analytical method was used in non-slip conditions for state 1, which showed a difference of less than 10% in both numerical and analytical methods in axial force and bending moment. All the relationships used in the analytical method are presented in Table 4.

Where T_{max} is maximum thrust in tunnel lining, M_{max} is maximum bending moment in tunnel cross-section due to shear waves, E_m is modulus of elasticity of soil or rock medium, v_m is Poisson's ratio of soil or rock medium, r is radius of circular tunnel, γ_{max} maximum free- field shear strain of soil or rock medium, E_l is modulus of elasticity of tunnel lining , v_l is Poisson's ratio of tunnel lining, t is thickness of tunnel lining, R is lining- soil racking ratio and I is moment of inertia of the tunnel lining(per unit width) for circular lining. By including the parameters related to this study in the formulas, the values of γ_{max} , K_1 and K_2 have been estimated as 0,006, 0.6 and 1.27 respectively.

Table. 4:	Relation	onsh	ips	pres	ented	by W	/ang	in the	ground-
							1	4.7	

structure interaction method [14]				
Non-slip conditions				
$T_{\max} = \pm K_2 \frac{E_m}{2(1+v_m)} r \gamma_{\max}$				
$M_{\max} = \pm \frac{1}{6} K_1 \frac{E_m r^2 \gamma_{\max}}{(1 + \nu_m)}$				
$F[(1-2\nu_m)-(1-2\nu_m)C]-\frac{1}{2}(1-2\nu_m)^2+2$				
$K_{2} = 1 + \frac{1}{F[(3 - 2\nu_{m}) + (1 - 2\nu_{m})C] + C[\frac{5}{2} - 8\nu_{m} + 6\nu_{m}^{2}] + 6 - 8\nu_{m}}$				
$C = \frac{E_m (1 - \nu_l^2) r}{E_l t (1 + \nu_m) (1 - 2\nu_m)}$				
$F = \frac{E_{m}(1-v_{L}^{2})R^{3}}{E_{m}(1-v_{L}^{2})R^{3}}$				
$6E_{L}I(1+v_{m})$				



Fig. 12: Horizontal velocity time history at the base of the model and the base of the free boundary column for the Duzce earthquake



Fig. 13: Horizontal velocity time history on the model top and the top of the free boundary column for the Duzce earthquake

3.6. The dynamic coupled model

In numerical modeling the Byron model is used to investigate the simultaneous effect of dynamic and water loads. The used parameters are presented in table 5.

Table. 5: The coupled model parameters					
parameter	value				
N	27.7				
N60	23.5				
Dr	72.71				
C_1	0.168				
C_2	2.378				

Figure 14 shows the pore pressure changes relative to the earthquake loading time at a depth of 26 meters from the ground surface. As a result of the dynamic loads and the change in the volume of the elements, the pore pressure increases and causes change in fluid flow regime in porous

media. In other words, increasing the pore pressure changes the effective stress, which adds more load to the model. Figure 15 shows the axial force and bending moment of the tunnel roof caused by Duzce earthquake in state 1 and Figure 16 presents these parameters in state 2.



Fig. 14: Pressure changes at a depth of 26 m during the Duzce earthquake

By comparing the maximum axial force and bending moment obtained from the dynamic analysis in two states for this point, as shown in Fig. 17, like the static analysis, the axial force is 25% higher in the coupled state from state 1 and also, the bending moment in coupled state is 66% lower than in state 1.



Fig. 15: (a) Axial force, (b) bending moment for the roof of the tunnel in state 1



Fig. 16: (a) Axial force, (b) bending moment for the roof of the tunnel in state 2



Fig. 17: Results of two modeling states

4. Conclusion

One of the most important stages of tunneling is the design of the support system, which must withstand sufficient loads over the life of the project. Permanent loads on the tunnel support system include earth load, water load, earthquake load, structural weight, etc. In terms of the impact of each load on the tunnel structure, the loads of the earth and water can be considered as critical loads. The separation of these loads is not logical considering the hydro-mechanical coupled behavior of water and ground (soil and rock) and their effects on the structure. Therefore, this study investigates the amount of loads inflicted on the tunnel in both coupled state and separated water and ground loads state. The most important results are as follows:

1-The static design of the tunnel in two hydro-mechanical coupling states, and considering the water load as the pore pressure, the axial force in the coupling state is more than the pore pressure coupling state. Also, the bending moment in coupling mode showed a decrease of 36.2% compared to the other state.

2-By applying the dynamic load on the level of MCE earthquake on the dynamic model of the structure in pure and dynamic coupled states, the axial force and bending moment were compared in the two states. The results show an increase in the axial force and a decrease in the moment of bending in the dynamic coupling state.

3-Comparing the results obtained in static and dynamic analysis, it was observed that the axial force on the segment support system in dynamic analysis in both states is approximately 5 times the static analysis. Also, the bending moment increased about 10 times.

4-Due to the difference of axial load and bending moment on the structure in the two states under the static and dynamic design conditions, it is better to study the behavior of the structure in a coupled state, however, only circular tunnel behavior was investigated in this study and it is suggested that different geometries were also examined.

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