ENGINEERING SAFETY OF TUNNELS DURING EARTHQUAKES

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ABSTRACT: Although tunnels were considered among the safest structures under earthquake loads, the recent studies have established that some damages have been observed in different tunnels and underground structures during and after ground shaking. As a result, it was very important to perform engineering assessment for this special type of structures, from both geotechnical and structural engineering point of view. The designer should check the safety of the underground structures (including tunnels) to withstand adequately the different applied loads, considering seismic loads, as well as, the temporary and permanent static loads.

This paper is divided into two parts; the first one summarizes the current state of seismic analysis of tunnels and focusing on the earthquake effects on tunnels and their deformation as well as the different approaches of seismic analysis. The second part shows briefly the results of a recent extensive study that has been conducted by the Geotechnical Engineering Group at Ain Shams University, Cairo, Egypt. This study was performed to investigate the effect of tunnel geometry and rigidity, site conditions and earthquake intensity on the developing internal forces in tunnel lining, under seismic excitation. Through this study, an advanced nonlinear dynamic finite element model was developed to perform non-linear dynamic analysis for both of free field and soil-tunnel system. This methodology was used to perform the desired analysis considering circular tunnels having diameters range between 6 and 10 m. The soil formation consists of sand deposit having total thickness ranges between 30 and 50 m, followed by the bedrock surface. Different relative densities of the sand deposit were considered. The tunnel center was located at variable distances (d=7 to 19 m), below the ground surface. The thickness of the tunnel lining ranges between 0.3 and 0.7 m. Based on the results, conclusions and recommendations are presented, considering the seismic effect on tunnels and the engineering safety requirements for these vital structures.

INTRODUCTION

Underground facilities, especially tunnels, have a major role in the redevelopment of urban areas. Due to the population upsurge, there is a strong need for more underground structures. Up to about 15 years ago, underground structures were thought to be safe during earthquakes, as long as, they did not cross fault plans and tunnel were designed with no regard to earthquakes effect. The ITA working group on research was one of the first technical teams to act on reviewing the available design and analytical tools for underground structures (Hashash et al.[1]). On the other hand, recent studies focusing on tunnels damaged by near-field earthquake (ground shaking) enhanced the awareness of seismic hazards for underground structures. Kontogianni and Stiros[2] concluded from their study of tunnels in areas affected by strong earthquakes in the last 50-100 years that tunnels cannot be considered as structures invulnerable to earthquakes. The most important cases of underground structures damaged by earthquakes, which raised the attention to the vulnerability of underground structures to earthquake, are as follow:

(1) ***The 1923 Kanto Earthquake*** (with local magnitude (M) = 8.16), caused damage in 82 railway tunnels among the total of 116, in the area hit by the earthquake. The damage consisted of failure of portal sections, transverse and longitudinal cracking of the linings spalling and deformation[3]. Figure 1 shows cracks appeared in the lining of Nagasakayama tunnel after the 1923 Kanto Earthquake.

(2) ***The 1995 Hyogo-ken Nanbu (Kobe, Japan) Earthquake*** collapsed the Daikai Underground Station in Kobe, as sketched in Fig. 2. The station is located about 20 km from the epicenter of the earthquake. The moment magnitude scale (Mw) of the earthquake was 6.9 and strong ground motion lasted for 20 seconds.

(3) *During* ***The 1999 Chi-Chi earthquake*** in central Taiwan on September 21, many mountain tunnels suffered significant damage to various extents[4]. The earthquake magnitude (M) was 7.3 on the Richter scale and it was related to the reaction of the 60 km Chelungpu Fault. It was found that among the 57 investigated tunnels, 49 of them were damaged.

(4) ***The 1999 Duzce earthquake*** (Mw=7.2) hit Turkey on November 12 and caused an extensive damage in the 16-m wide under construction twin Bolu tunnel[5]. The length of the surface fault rupture was estimated to be 40 km.

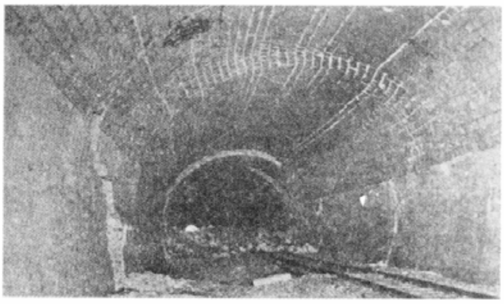


Figure 1. Cracks Appeared in the Lining of Nagasakayama Tunnel after the 1923 Kanto Earthquake (after Okamoto[3]).

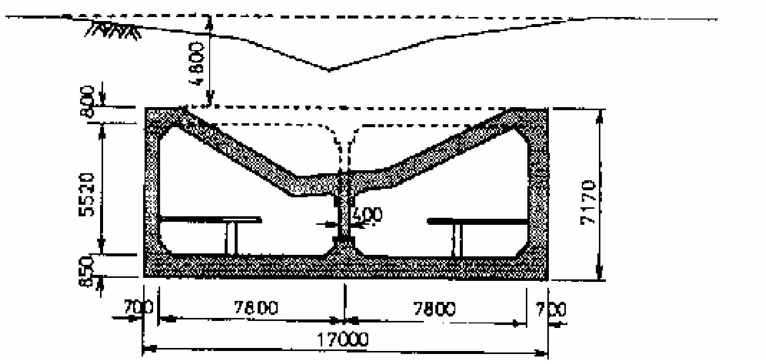


Figure 2. Sketch for the Cross Section of the Daikai Underground Station after the 1995 Hyogo-ken Nanbu Earthquake (after Uenishi and Sakurai[6]).

CURRENT STATE OF SEISMIC ANALYSIS OF TUNNELS

Earthquake Effect on Underground Structures

(1) Ground shaking: When seismic waves propagate through the Earth's crust, the resulting ground motion is considered ground shaking. The intensity of the shaking attenuates with distance from the fault rupture. During earthquake, different types of seismic waves are produced (body waves and surface waves). Figure 3 shows a sketch for the directions of the body waves (longitudinal P-waves and transverse shear S-waves), as well as, the surface waves (Rayleigh waves and Love waves) that generated by earthquake.

(2) Ground failure: It results in ground instability such as liquefaction, slope instability at tunnel portals, fault displacement and tectonic uplift and subsidence.

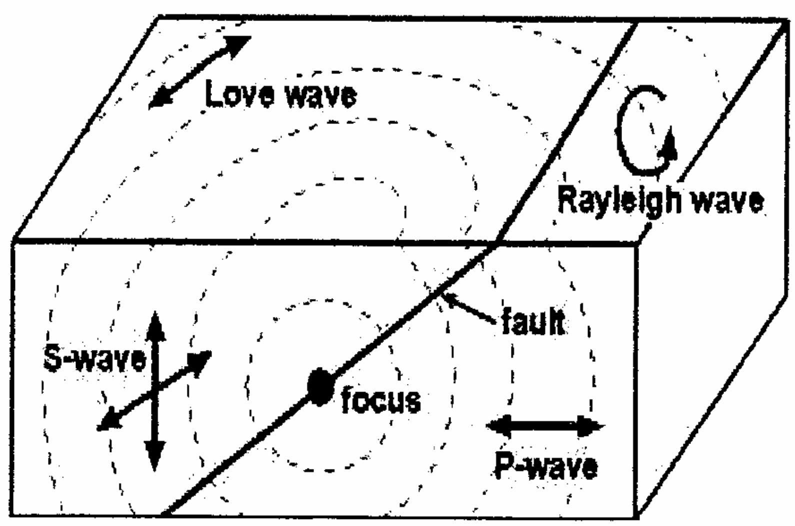


Figure 3. Direction of Body Waves (P and S-waves) and Surface Waves (Rayleigh and Love Waves) Generated by an Earthquake (after Resheidat and Hamdaoui[7])

Deformations of Tunnel Structures due to Seismic Motion

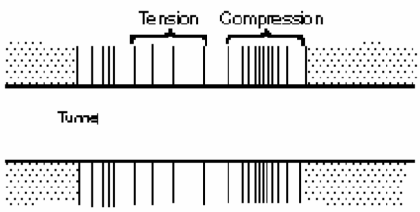
Owen and Scholl[8] illustrated the three types of deformations that express the seismic response of underground structures as follow:

(1) **Axial deformations**: When the compression waves propagate parallel to the tunnel axis, the shear stresses transferred between the ground and structure cause alternating compression and tension forces, as shown in Fig. 4.a. These forces are limited by the interface shear strength between tunnel surface and the surrounding soil.

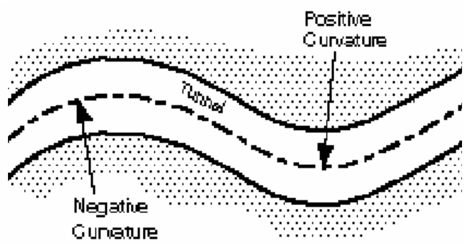
(2) **Curvature deformations**: When seismic waves propagate obliquely to the tunnel axis, the components of seismic waves producing particle motions perpendicular to the longitudinal axis, as shown in Fig. 4.b. These motions subject different parts of the structure to out-of-phase displacements. It results in a longitudinal compression-rarefaction-wave traveling along the tunnel.

(3) **Ovaling/racking deformations**: These deformations develop due to shear waves propagation normal or nearly normal to the tunnel axis, as shown in figures 4.c & 4.d.

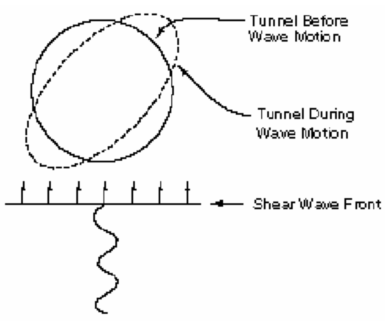
Previous researches[1,3,9,10] pointed out that the shear distortion of ground caused by vertically propagating shear waves (that cause ovaling/racking deformations) may be the most critical and predominant mode of seismic motions in many cases. On the other hand, axial and curvature deformations are limited by the interface shear strength that tends to be zero in the case of full-slip. Therefore, seismic response due vertically propagating shear waves is the main focus of this research.



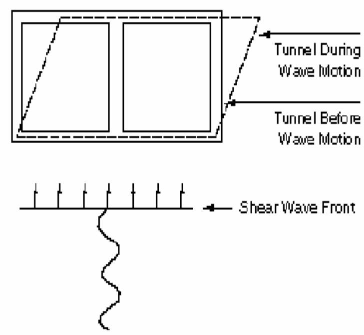
(a) **Axial deformation along tunnel**



(b) **Curvature deformation along tunnel**



(c) **Ovaling deformation of a circular cross section**



(d) **Racking deformation of a rectangular cross section**

Figure 4. Deformation Modes of Tunnels due to Seismic Waves (modified after Owen and Scholl[8]).

Seismic Analysis of Tunnels

The methods used for seismic analysis and design of tunnels may be categorized as shown on Figure 5 to the following approaches; (1) free-field deformation approach, (2) soil-structure interaction approach and (3) dynamic earth pressure method. Both closed-form elastic solutions and numerical analysis are used in these approaches.

The free-field approach describes ground deformations or strains caused by seismic waves without the structure being introduced within the analysis. The general behavior of the tunnel lining is similar to that of an elastic beam subjected to deformations and strains imposed by the surrounding ground. Structure deformations may be overestimated or underestimated depending on the rigidity of the structure relative to the ground. On the other hand, the use of the dynamic earth pressure approach for tunnels and underground structures faces several limitations[1]. Therefore, focusing in this paper will be on the soil-structure interaction approach only.

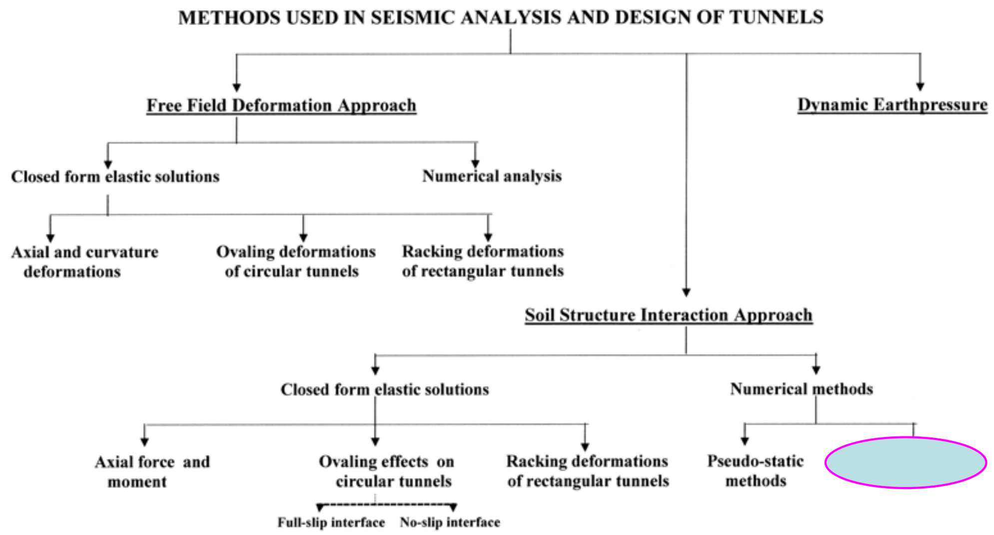


Figure 5. The Layout of the Methods Used in Seismic Analysis and Design of Tunnels

Soil-structure Interaction Approach

(a) Closed-form elastic solutions: In this approach, the tunnel-ground system is simulated as an elastic beam on an elastic foundation, utilizing the theory of wave propagating in an infinite, homogeneous, isotropic medium. The solutions ignore dynamic (inertial) interaction effects. Solutions were given for axial force and moment and ovaling effects on circular tunnels[1,9]. Penzien[10] developed closed-form solutions for thrust, moment and shear in circular tunnel linings considering the soil-structure interaction during seismic excitation.

(b) Numerical methods: Because of the complex nature of the seismic soil-structure interaction problem for underground structures, it may be required to use numerical techniques such as: the finite difference or finite element methods (FDM or FEM). Several computer programs are available for such analysis (FLUSH[11], ANSYS-III[12], ABAQUS[13] and others). For analyzing axial and bending deformations in the longitudinal direction, it is most appropriate to utilize three-dimensional models. While, two-dimensional models may be used to analyze the racking effect of the propagated seismic shear wave in planes perpendicular to the tunnel axis. Numerical analysis methods include both pseudo-static and dynamic solutions. The main advantage of the FEM is that it can provide the basic considerations required for a good analysis such as:

(1) Variation of soil characteristics with depth.
(2) Variation of ground motions with depth, which is a major requirement for embedded structures.
(3) Mutual effect between tunnel-soil system and nearby structures.
(4) Dynamic behavior of soil under cyclic loading.
(5) Any random ground motion (not particularly to be simple harmonic waves).
(6) It gives full seismic response analysis.

The conducted analysis for this research was performed using the dynamic numerical methods to model the interaction of the tunnel lining with the surrounding soil.

DYNAMIC SOIL-TUNNEL INTERACTION

Geometry of the Problem

Figure 6 shows the physical structure used for the simulation of the parametric study. It consists of a single circular tunnel with diameter (D) that varies between 6 and 10 m, surrounded by homogenous sand. Three types of sand have been studied considering a range of relative density (Dr = 25, 62 & 95%). The selected total thickness of the sand layer (H) was varied between 30 and 50 m above the bedrock surface. The tunnel center was located at variable distances (d=7 to 19 m), below the ground surface. The thickness of the tunnel lining (t) ranged between 0.3 and 0.7m.

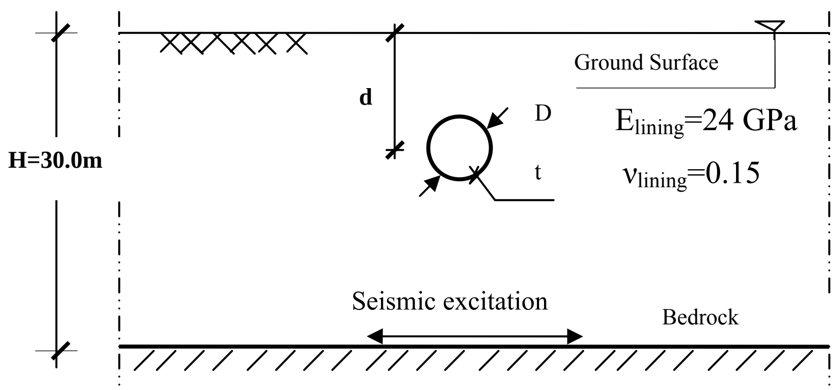


Figure 6. Geometry of the Studied Problem

Static and Dynamic Soil Properties

The soil properties for the three considered cases of sandy soil (S1, S2 and S3) are given in Table 1. The initial shear modulus (G_{max}) was evaluated according to Hardin and Drnevich[14] correlation;

$$G_{max} = \frac{3230(2.97 - e)^2}{1+e}) (OCR)^k \bar{\sigma}_o^{\ 1/2} \tag{1}$$

where the factor (k) depends on the plasticity index of soils and equal zero for sand, ($\bar{\sigma}_o$) is the mean normal effective principle stress, (OCR) is the overconsolidation ratio and (e) is the void ratio.

Table 1: Values of the Soil Properties Used for the Different Soil Types.

Soil Type	Static Friction Angle, (Ø^o)	Dry Density, γ_d (KN/m^3)	Void Ratio, e_o	Relative Density, Dr (%)	Poisson's Ratio, ν
Sand 1 (S1)	30 ^{o}	16.73	0.55	25	0.34
Sand 2 (S2)	35 ^{o}	17.96	0.45	62	0.32
Sand 3 (S3)	40 ^{o}	19.20	0.35	95	0.30

It should be noted that only the results of the first soil type with H= 30 m are presented in this paper. The remaining results are given by Abdel-Motaal et al.[15] and Khairy[16].

Soil Model Used to Represent Soil Behavior and Dynamic Analysis Algorithm

The constitutive models commonly used in soil dynamics are the equivalent linear models, cyclic nonlinear models and models based on elastic-plastic theories. Equivalent linear models have limited ability to represent many aspects of soil behavior under cyclic loading conditions. Moreover, this model does not estimate the values of the final residual soil deformation after dynamic excitation. At the other end of the spectrum, advanced constitutive models based on elasto-plastic theories can represent many details of dynamic soil behavior, but their complexity and difficulty of calibration currently render them impractical for many common geotechnical earthquake-engineering applications.

Hence, the nonlinear stress-strain was selected for the cyclic nonlinear models that follow the actual stress-strain path during cyclic loading using the Extended Massing Model[17]. The step-by-step time integration algorithms, which are adaptable to nonlinear problems and considered to be the most precise ones, were used in this analysis.

Earthquake Ground Motion

The source of dynamic excitation, applied at the surface of the bedrock perpendicular to the tunnel axis, is an artificial acceleration time history, Figure 7. For more details refer to Abdel-Motaal[18]. The records are adapted many times to give maximum accelerations at the bedrock surface (ab-max) equals 0.046, 0.064, 0.082 and 0.1g.

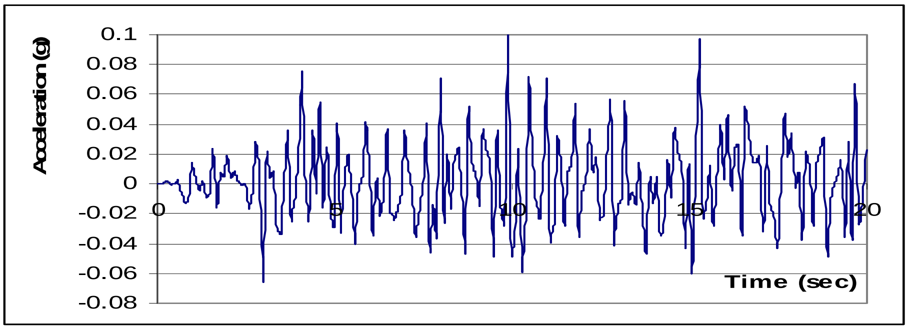


Figure 7. Acceleration Time-History - ab-max = 0.1g (after Abdel-Motaal[18])

Finite Element Meshes

Figure 8 shows one of the finite element meshes that were used for the free field analysis. While, Figure 9 shows the finite element mesh for one study case (d=11 m and D=8 m) of the soil-tunnel system.

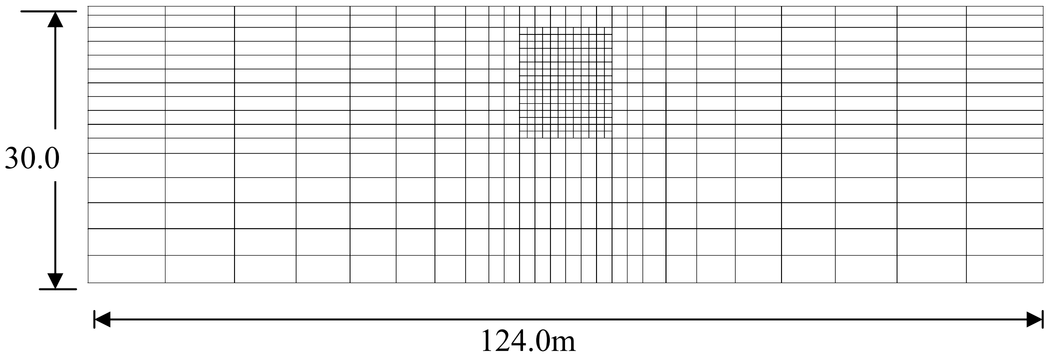


Figure 8. Finite Element Mesh Used in the Free Field Analysis.

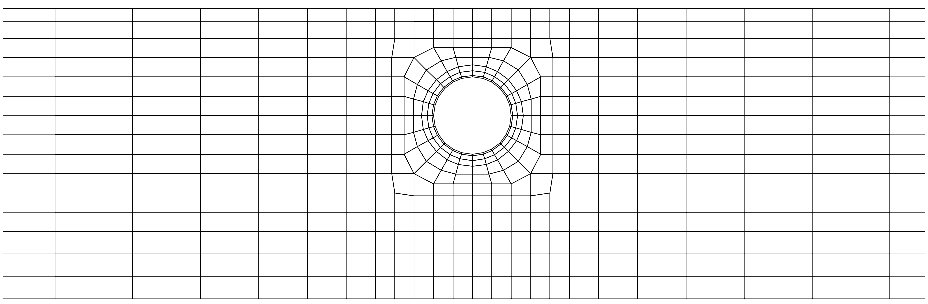


Figure 9. Finite Element Mesh Used in the Soil-Tunnel System Analysis (case of D=8 m and d=11 m).

RESULTS AND INTERPRETATION

Free field results

Figure 10 shows the variation of absolute peak accelerations values with depth for the considered four scaled seismic loading intensities (ab-max = 0.046, 0.064, 0.082 and 0.1g). The corresponding developed maximum accelerations at the ground surface (g-max) are 0.159, 0.196, 0.236 and 0.272g, respectively. Values of g-max will be used to identify each scaled loading. In general, the peak values of accelerations are magnified as it is raised from the bedrock towards the ground surface, which agrees with the results presented by Hashash et al.[1].

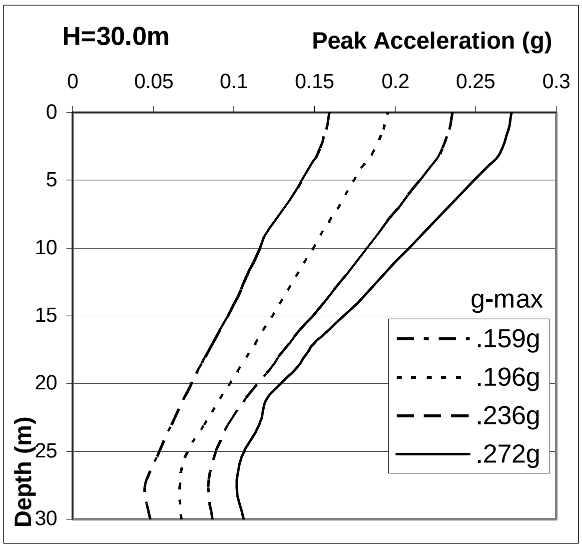


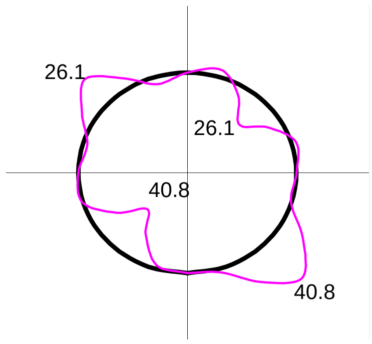
Figure 10. Variation of Absolute Peak Accelerations Values with Depth

Soil-Tunnel System Results

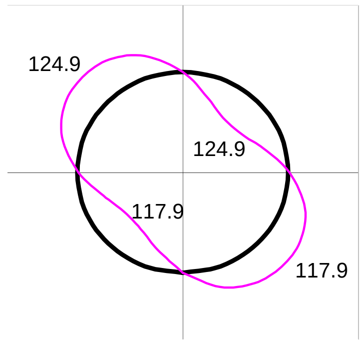
For the selected case (d= 19 m, D=6 m and t=0.5 m, soil type S1), Figure 11 shows the earth pressure acting on the tunnel lining, its deformed shape and the internal forces diagrams, resulting from seismic motion at specific time of 4.18 sec. of seismic motion (peaks of absolute values of internal forces are induced) during seismic excitation (g-max=0.272g). The shape of the internal force diagrams is similar to those obtained by Lee and An[19] and the shape determined by the formulas proposed by Wang[9] and Penzien[10].

It is noteworthy that combining the earth pressure resulting from seismic and static gravity loads shows that all points at the interface are subjected to compression stresses. The variation of peaks values of the bending moments (M-p), exerted due to the effect of seismic loads, with tunnel depth at different loadings intensities (g-max) are shown in Figure 12. While, these variations for the peaks values of the thrusts (N-p) are shown in Figure 13. In these figures the variations are shown for the three considered tunnel diameters (D=6, 8 and 10 m) and the different lining thickness (t = 0.3, 0.5 and 0.7 m).

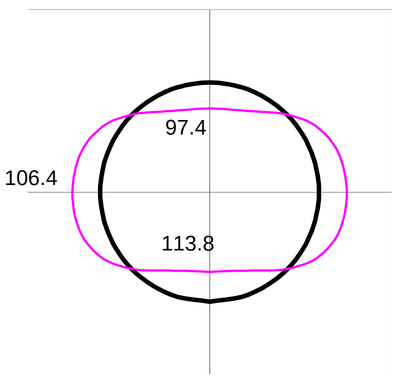
Case of: (d=19m, D=6 m, t=0.5m) **Sand S1** **H=30.0 m** **t=4.18 s**



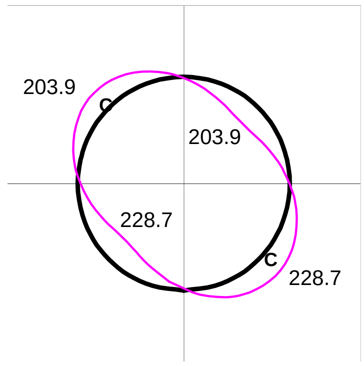
Normal Component of Soil P. on Liner (kPa)



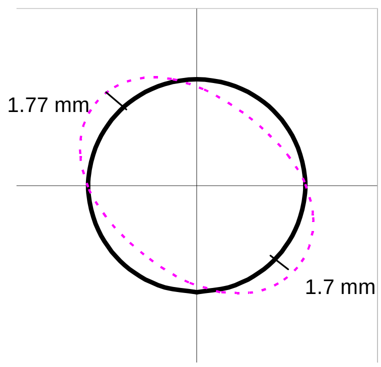
Bending Moment Diagram (kNm)



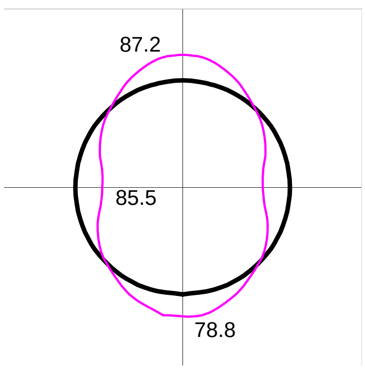
Shear Component of Soil P. on Liner (kPa)



Normal Force Diagram



Deforn **Deformed Shape (rigid body motion is cancelled)**



Shear Force Diagram

Figure 11. Earth pressure, Deformed Shape and Internal Forces Diagrams for case of (D=6.0m, t=0.5m, d=19.0m) Resulting from Seismic Motion at time=4.19s - g-max=0.272g.

Sand S1 (ϕ=30^o) H=30 m

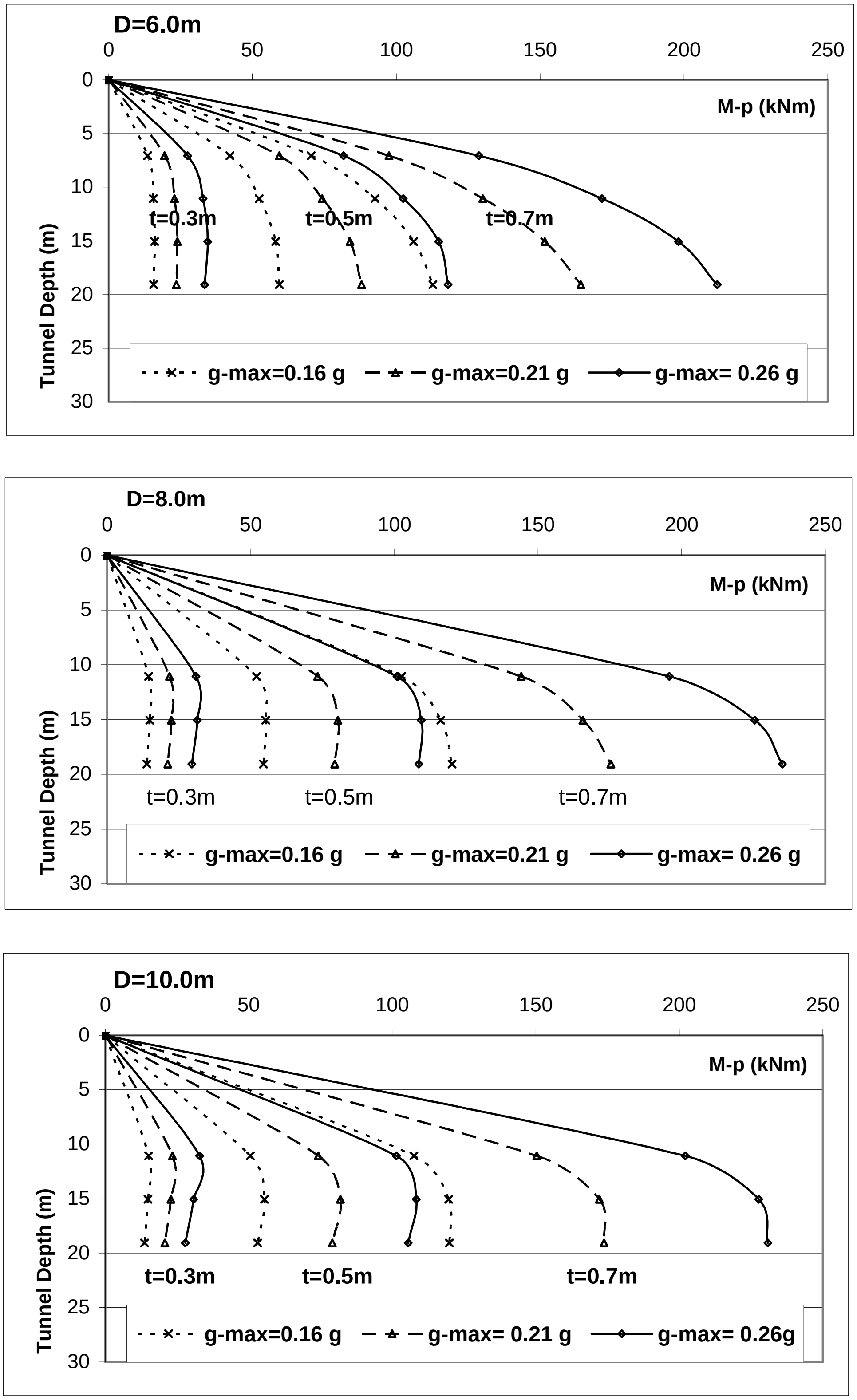


Figure 12. Variation of Peak Values of the Bending Moment (M-p), at Different Values of g-max, with the Tunnel Depth.

Sand S1 (ϕ=30^o) **H=30 m**

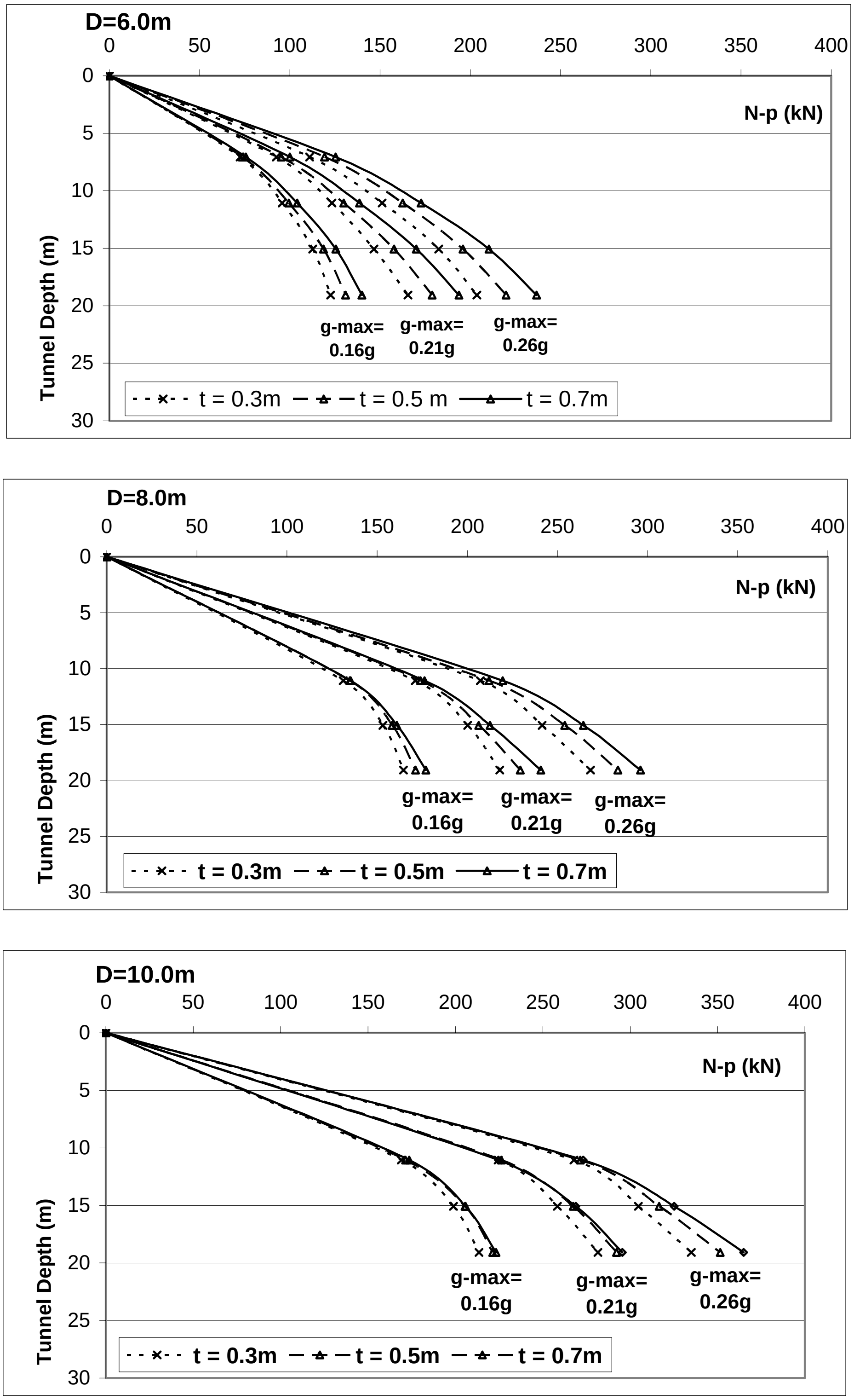


Figure 13. Variation of Peak Values of the Thrust (N-p), at Different Values of g-max, with the Tunnel Depth.

Based on the results of the parametric study, the following could be observed:

(1) The mutual interaction between tunnel and surrounding soil, under seismic loadings, depends on tunnel stiffness, its geometry properties and its location. These results agree with the conclusions of Sarfeld et al.[19].
(2) The results showed that, whatever the depth of the tunnel, the internal forces diagrams have almost typical shapes similar to those calculated using Wang[9] and Penzien[10] formulas and those obtained by Lee and An[20].
(3) The bending moment values increase with the depth starting from the ground surface up to a certain depth, and then the values begin to decrease. On the other hand, the values of the thrust increase with depth, at a decreasing rate, in the downward direction.
(4) The maximum values of the bending moment and normal force occur almost consistently at the knee or shoulder sections.
(5) In general, the maximum values of the bending moments are found at conditions where the tunnel centre is located at a depth ranging between 10 and 15 m.
(6) Both of the values of bending moment and thrust increase with the increasing of seismic intensity (maximum acceleration at ground surface).
(7) The peaks values of bending moment and thrust increase with the increasing the thickness of tunnel lining. The magnitude of the moments is more affected than thrusts. This means that strengthening a reinforced concrete lining by increasing its thickness must be accompanied with increasing the steel reinforcement in order to improve its ductility.
(8) The results showed that lining deformations increase as the flexibility of the tunnel increases (decreasing tunnel thickness and/or increasing tunnel diameter).

CONCLUSIONS

There has been a remarkable increase in the utilization of underground space in some urban areas during the past 15 years (El-Nahhas[21,22,23]) and there is mounting awareness for the vulnerability of tunnels and underground structures to ground shaking during earthquakes. A special study conducted by ITA[1] identified the need for improved numerical models to simulate the dynamic soil-structure interaction of tunnels, as well as portal and subway structures.

The study described in this paper aims at investigating the mutual dynamic interaction between a single circular tunnel and the surrounding soil. A sophisticated non-linear numerical model has been enhanced to simulate the problem, using the finite element method. The model has the capability to simulate the full dynamic interaction between the tunnel and surrounding soil under the effect of bedrock excitation, as a one-continuum system. The Extended Massing Model was selected to represent the two main soil behavior characteristics; the nonlinearity and the hysteresisty. The dynamic excitation at the bedrock surface is based on an artificial acceleration time-history.

Results of an extensive parametric study for the cases given in this paper indicated that the mutual interaction between a tunnel and surrounding soil, under seismic loadings, depends on tunnel stiffness, its geometry, properties and its location below ground surface. The maximum values of the bending moments and normal forces occur almost consistently at the knee or shoulder sections. In general, the maximum values of the bending moments occur when the tunnel centre is located at a depth ranging between 10 and 15 m below ground surface. Both of the bending moments and thrusts values tend to increase with the increasing of seismic intensity.

Tunnels with small-thickness lining tend to attract smaller additional internal forces during seismic motion than the rigid ones, but they will experience larger deformations. A more flexible configuration of tunnel linings, with adequate reinforcements to provide sufficient ductility, is a more desirable measure in some cases.

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