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INNOVATIVE GEOSCIENCES, CIRCULAR ECONOMY AND SUSTAINABILITY

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BEHAVIOR OF SHALLOW TUNNEL IN SOFT SOIL UNDER SEISMIC CONDITIONS

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Abstract: In these modern times of space scarcity, underground tunnels play an important role as a key component of an urban transportation and utilities network. Tunnelling in urban areas is growing in response to the increased needs for efficient transportation, many urban tunnels are constructed in soft ground at shallow depths. A great demand for these multi-functional structures is increasing due to the fast population growth and limited aboveground spaces in urban areas. Any instability of tunnels will be highly detrimental to their performance thereby posing a threat to public safety, consequently cause life-threatening and infrastructure crippling consequences. Recent experiences show that tunnels become vulnerable during an earthquake event. In this paper, an effort was made to develop an integrated methodology for evaluation of the probabilistic future performance of underground circular tunnels subjected to seismic loadings.

Keywords: Stability; Underground construction; Tunnel; Earthquake; Seismic analysis.

1. INTRODUCTION

Tunnels are classified as complex engineering structures, often requiring careful and detailed procedures starting from an early analysis and design stages until the completion of construction works. Such structures behave differently compared to the surface structure mainly due to the impact of the surrounding soil, which results in the complex soilstructure interaction system. The interaction will become more complicated due to the impact of extreme loading such as strong ground acceleration of earthquake load (Tsinidis et al., 2020). The stability of tunnels is crucial as even minor tunnel instability can have severe lifethreatening and infrastructure crippling consequences. The importance of this type of structures, for life safe and from an economic point of view, reveals the need for proper seismic design. Taking into consideration the specific conceptual features of tunnels that makes their seismic behavior very distinct from aboveground structures and the lack of knowledge on many crucial issues, their seismic design becomes a very demanding procedure. Some notable case histories in relatively recent earthquakes have proven that under certain conditions tunnels may experience severe damage or even collapse due to strong seismic shaking. These include the collapse of the Daikai metro station during the notorious 1995 Kobe earthquake (Iida et al., 1996; Nakamura et al., 1996), of several “horseshoe”-shaped tunnels in Taiwan during the 1999 Chi-Chi earthquake (Ueng et al., 1999), and of the Bolu tunnel during the 1999 Kocaeli earthquake in Turkey (Tsinidis et al., 2020).

2. SEISMIC TUNNEL DAMAGE: TYPES AND CHARACTERISTICS

In general, during the event of an earthquake, underground structures may suffer significant damages due to the ground shaking and ground failure. Ground shaking is referred to vibration of the ground due to seismic wave propagation, while the ground failure is due to ground instability such as landslides, liquefaction, fault rupture.

The recorded damages of tunnels during an event of strong earthquake provide sufficient evidence that the tunnel becomes vulnerable to earthquake loads. Although the geotechnical engineering community has paid attention to critically understand the behaviour of tunnels during earthquakes, such studies are minimal and further research is required to evaluate the performance and integrity of tunnels under such unpredictable seismic hazards. Besides, particular attention should be given to the influence of uncertainties parameters that may have a significant ability to modify the response and performance of tunnels under earthquake loads. This type of damage often occurred where the tunnel opening was in a steep slope of highly weathered rock, especially when the tunnel opening lacked sufficient protection. The main phenomena are avalanches, rock falls and sliding from higher parts of the slope, which usually caused damage of the tunnel portals and partial or even complete obstruction of tunnel openings. The most important tunnel collapses occurred in the weak carbonaceaous mudstones and the mudstones surrounding the entrance of the Longxi tunnel (Figure 1), with five collapses near the left and right tunnel portals. A collapse of the secondary lining occurred in the plain concrete above the haunch (Figure 2) near the Longxi tunnel entrance and in the Longdongzi tunnel.

Figure 1. Collapse of the tunnel at the crossing of the fault (Tianbin, 2012).

Figure 2. Collapse of the tunnel roof of secondary lining concrete in the Longxi tunnel (Tianbin, 2012).

Figure 3. Rock fall at the Longxi tunnel portal from a high and steep slope (Tianbin, 2012).

Tensile and shear failures were visible in the concrete lining. Rock falls often occurred on slopes consisting of strong, relatively unfractured rock masses with stress release fractures and frequently destroyed the portal and slope protection structures. For instance, during the earthquake the granite from the top of the slope near Longxi tunnel fell across the opening and destroyed the slope protection structure (Figure 3).

3. SEISMIC BEHAVIOUR OF TUNNEL LINING

In the design of the tunnel lining against seismic loading, tunnel engineers attempt to adopt closed-form analytical solutions to estimate the maximum induced forces of circular tunnel structures. In early studies of racking deformations, Peck et al. (1972), based on earlier work by Burns and Richard (1964) and Hoeg (1968), proposed closed-form solutions in terms of thrusts, bending moments, and displacements under external loading conditions. The response of a tunnel lining is expressed as functions of the compressibility and flexibility ratios of the structure, and the insitu overburden pressure and at-rest coefficient of earth pressure of the soil. The solutions are developed for both full-slip and no-slip condition between the tunnel and the lining. Full-slip condition results in no tangential shear force due to absence of normal separation.

3.1. Wang's Analytical solution (1993)

In this analytical solutions, Wang (1993) suggested that the relative stiffness between a circular lining and the medium are often estimated based on two ratios designated as the compressibility ratio (C) and the flexibility ratio (F). The ratios are calculated by the following equation:

$$C = \frac{E_m\left(1-v_1^2\right)r}{E_1 t\left(1+v_m\right)\left(1-2v_m\right)}, \tag{1}$$

$$F = \frac{E_m\left(1-v_1^2\right)r^3}{6E_1 I_l\left(1+v_m\right)}, \tag{2}$$

where: E_m is the modulus of elasticity of soil, kPa, υ_m is the Poisson's ratio of soil, E_1 is the modulus of elasticity of the tunnel lining, kPa, υ_1 is the Poisson's Ratio of the tunnel lining, r is the radius of the tunnel lining, m. t is the thickness of the tunnel lining, m. I_l is the moment of inertia of the tunnel lining.

Meanwhile, Wang (1993) proposed the equations for calculation of the induced structural forces for both full-slip and no-slip condition of interfaces. The formulations are as follows:

a) Full-slip condition:

$$M_{\max} = \pm\frac{1}{6}K_1\frac{E_m}{\left(1+v_m\right)}r^2 g_{\max}, \tag{3}$$

$$T_{\max} = \pm \frac{1}{6} K_1 \frac{E_m}{(1+v_m)} r g_{\max}, \tag{4}$$

$$K_1 = \pm \frac{12(1-v_m)}{2F+5-6v_m},$$

g_{max} is the peak ground particle acceleration at surface.

b) No-slip condition:

$$T_{\max} = \pm K_2 \frac{E_m}{2(1+v_m)} r g_{\max}, \tag{5}$$

where:

$$K_2 = 1 + \frac{F[(1-2v_m)-(1-2v_m)C] - \frac{1}{2}(1-2v_m)^2 + 2}{F[(3-2v_m)-(1-2v_m)C] + C\left[\frac{5}{2} - 8v_m + 6v_m^2\right] + 6 - 8v_m},$$

The maximum bending moment is assumed equal to that in full-slip conditions (Equation 3).

Where: T_{max} is the maximum tangential thrust, M_{max} is the maximum moment, K_1, K_2 are the lining response coefficient.

3.2. Penzien's Analytical Solution (2000)

Penzien and Wu (1998) and Penzien (2000) developed similar analytical solutions for thrust, shear, and moment in the tunnel lining due to racking deformations.

a) Full-slip condition:

Assuming full slip condition, solutions for thrust, moment, and shear in circular tunnel linings caused by soil-structure interaction during a seismic event are expressed as (Penzien, 2000):

$$T_{(\theta)} = -\frac{12 E_1 I_l \Delta D^n_{lining}}{D^3(1-v_1^2)} \cos 2\left(\theta + \frac{\pi}{4}\right), \tag{6}$$

$$M_{(\theta)} = -\frac{6 E_1 I_l \Delta D^n_{lining}}{D^2(1-v_1^2)} \cos 2\left(\theta + \frac{\pi}{4}\right), \tag{7}$$

$$V_{(\theta)} = -\frac{24 E_1 I_l \Delta D^n_{lining}}{D^3(1-v_1^2)} \sin 2\left(\theta + \frac{\pi}{4}\right), \tag{8}$$

where: ΔD^n_{lining} is the Lining diametric deflection under normal loading only:

$$\Delta D^n_{lining} = R^n \Delta D_{free-field} = R^n \frac{g_{\max} D}{2},$$

where: R^n is the Lining-soil racking ratio under normal loading only: $R^n = \pm \frac{4(1-v_n)}{\alpha^n + 1}$,

$$\alpha^n = \frac{12E_1 I_l (5-6v_m)}{D^3 G_m (1-v_1^2)}$$

b) No-slip condition:

In the case of no slip condition, the formulations are presented as:

$$T_{(\theta)} = -\frac{24E_1 I_l \Delta D_{lining}}{D^3(1-v_1^2)} \cos 2\left(\theta + \frac{\pi}{4}\right), \quad (9)$$

$$M_{(\theta)} = -\frac{6E_1 I_l \Delta D_{lining}}{D^2(1-v_1^2)} \cos 2\left(\theta + \frac{\pi}{4}\right), \quad (10)$$

$$V_{(\theta)} = -\frac{24E_1 I_l \Delta D_{lining}}{D^3(1-v_1^2)} \sin 2\left(\theta + \frac{\pi}{4}\right), \quad (11)$$

where: ΔD_{lining} is the lining diametric deflection:

$$\Delta D_{lining} = R \Delta D_{free-field} = R \frac{g_{\max} D}{2},$$

$\Delta D_{free-field}$ is the free-field diametric deflection in non-perforated ground, R is the Lining-soil racking ratio, $R = \pm \frac{4(1-v_m)}{\alpha + 1}$,

$$\alpha = \frac{24E_1 I_l (3-4v_m)}{D^3 G_m (1-v_1^2)}$$

4. FINITE ELEMENT MODELLING

The finite element method is widely accepted numerical method for analysis and design in almost all branches of engineering. Plaxis 8.6 is a finite element code for soil and rock analyses, originally developed for analyzing deformation and stability in geotechnical engineering projects. In Plaxis, only no-slip condition between the tunnel lining and ground is simulated.

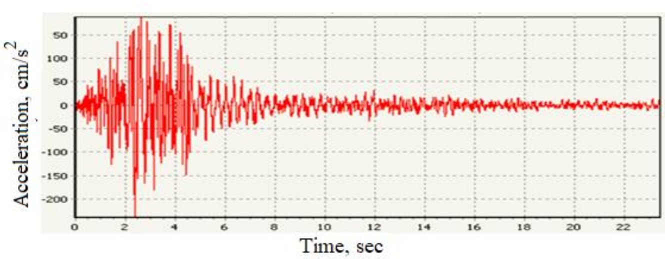

Figure 4. Acceleration time history at the base of the model.

Earthquake: Moment magnitude $M_w = 6.5$ and source to site distance 100 km, Peak ground particle acceleration at surface, $a_{max} = 2.4g$. Apparent velocity of S-wave propagation in soil only, $C_s = 250$ m/s. The ground motion recorded at rock site (Figure 4) with an amplitude of 240 cm/s^2 and a duration time of 22 s is selected as the seismic input after high frequency cutoff.

The Mohr-Coulomb model is adopted to define the behavior of the soil in the numerical analyses, and the calculation parameters of the soil and tunnel lining are shown in Table 1.

Table 1. Soil and tunnel linng

Parameters	Soft soil	Tunnel lining
Unit weight, γ_m (kN/m^3)	17.3	
Poisson's ratio, υ	0.35	
Internal friction angle, φ (^{0})	18	
Cohesion, c (kPa)	25	
Coefficient of lateral earth pressure, K_0	0.5	
Diameter, D (m)		6.3
Depth, h (m)		20
Axial stiffness, EA (kN/m)		10.5E+06
Flexural rigidity, EI (kNm^2/m)		7.875E+04
Weight, W (kN/m/m)		7.5
Poisson's ratio, υ_1		0.15

Model analysis was performed to calculate the mode shape and the vibration frequencies of the soil-tunnel systems. To analyze the nonlinear time history for calculating acceleration, the following three phases were considered:

(1) Plastic analysis and staged construction. In this step, the lining of the tunnel was activated and the soil inside the tunnel was deactivated.

(2) In the second phase, the amount of reduced volume is simulated by applying the contraction to the tunnel lining. This contraction will be defined in the phase of in-stage construction calculation, and the contraction of 3% will be applied to the tunnel center.

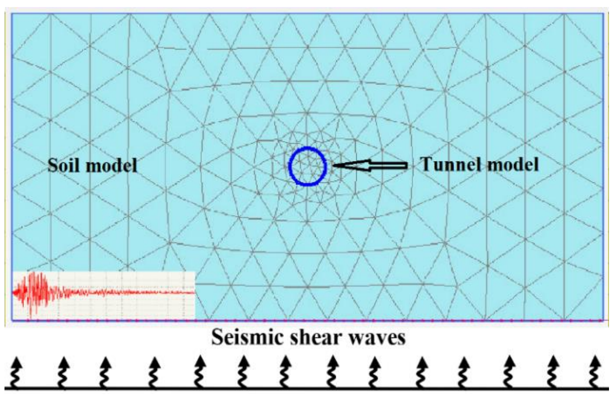

Figure 5. Seismic shear waves applied at transverse direction (x-axis) of the soil-tunnel model.

(3) In the third phase, the FE model is subjected to nonlinear dynamic time history analysis. The numerical models used in this study for validation of numerical model is described in Figure 5.

Numerical modeling results:

Seismic induced bending moments (M, kN.m/m), shear forces (Q, kN/m) and axial forces (N, kN/m) in tunnel linings as shown in Figures 6, 7 and 8.

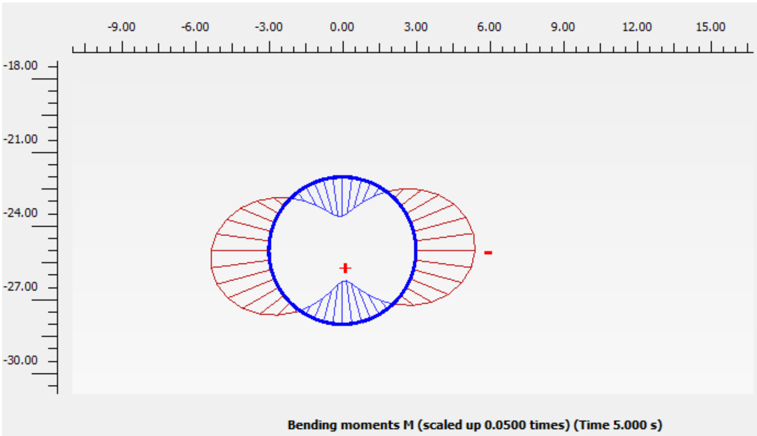

Figure 6. Seismic induced bending moments (M, kN.m/m) in tunnel linings.

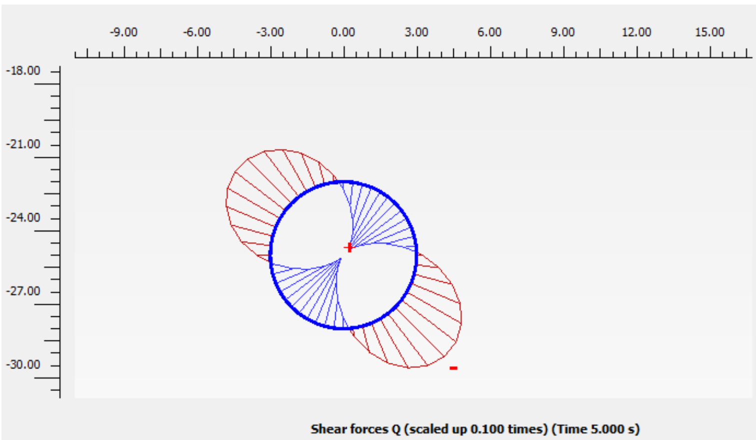

Figure 7. Seismic induced shear forces (Q, kN/m) in tunnel linings.

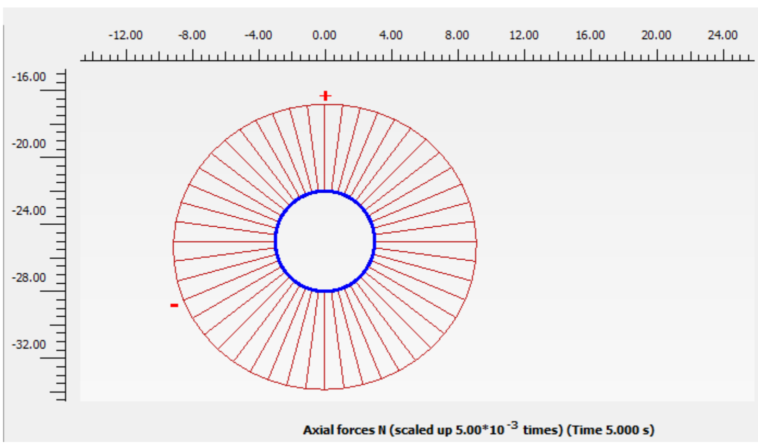

Figure 8. Seismic induced axial forces (N, kN/m) in tunnel linings.

The analysis results, using Wang (1993), Penzien (2000) and numerical method are shown in Table 2.

Table 2. The closed form and numerical method analysis results

Item		**N_{max} kN/m**	**Q_{max} kN/m**	**M_{max} kN.m/m**
Wang 1993	full slip	22.02	-	66.03
Wang 1993	no slip	427.6	-	66.03
Penzien 2000	full slip	22.02	44.02	66.03
Penzien 2000	no slip	41.43	41.43	62.156
Plaxis 2D	no slip	32.00	28.50	48.16

5. CONCLUSIONS

Two available analytical solutions to compute induced forces of a circular tunnel are presented. The solutions provide identical results for the condition of full-slip between the tunnel lining and the ground but differ in values of the calculated thrust for the condition of no-slip. Two-dimensional finite element analyses are performed to validate which of the two analytical solutions provide the correct solution. Comparison with numerical analysis demonstrates that Penzien's solution significantly underestimates the thrust in the tunnel lining for the condition of no-slip.

Results showed the significant roles of tunnel lining properties and burial depth in modifying the response and performance of tunnels. The proposed fragility curves highlight that the high stiffness of soil and tunnel lining may reduce the probability of tunnel damages. Meanwhile, the effect of burial depth is less significance for tunnels buried in poor soil condition. As a conclusion, the tunnels are vulnerable to seismic effects and their impact cannot be neglected.

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