The effect of earthquake frequency content on seismic response of horseshoe tunnels

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Abstract. Two-dimensional finite difference element analyses are performed to evaluate the tunnel response under earthquakes with different frequency content. The nonlinear model is used to consider the behavior of soil elements, while the elastic model is used to simulate the structure element response. The elastic base is applied at the bottom to absorb propagating waves, and the free-field column is used at the lateral boundary to consider the free-field motion in the soil medium. This study also comprehensively investigates the effect of soil-structure relative stiffness and soil-structure contact. The numerical results are validated with analytical solutions. The parametric study results show that the earthquake frequency content significantly affects the tunnel response. The higher frequency content of the earthquake results in a lower lining response. The effect of earthquake frequency content is more evident when the intensity level of input motion is increased. A similar trend is observed with the effect of soil-tunnel contact. The tunnel response decrease with increasing soil stiffness.

Keywords: Dynamic FDM analysis, nonlinear analysis, horseshoe, earthquake, internal forces, earthquake frequency content.

1 Introduction

Tunnels are essential components of metropolitan transportation and utility networks. The seismic performance of underground structures is typically thought to perform better than aboveground structures during past earthquakes. However, the recent strong earthquake demonstrates that underground structures may undergo extensive damage. For example, the collapse of the Daikai station was observed during the 1995 Kobe earthquake[1]; damage of tunnels was documented during the 1999 Chi-Chi earthquake [2], 1999 Kocaeli earthquake [3], and 2008 Wenchuan earthquake [4]. Therefore, it is needed to effectively evaluate the seismic performance of the underground structure, especially the horseshoe tunnels.

The earthquake frequency content is an important characteristic of the seismic analysis. There are many parameters used to evaluate the frequency content of an earthquake motion, such as predominant period, mean period, power spectrum intensity, and the ratio between peak ground acceleration (PGA to peak ground velocity (PGV) [5-7]. Among those, PGA/PGV ratio is a simple and useful parameter to provide information on earthquake frequency. Usually, earthquake motions are divided into three types based on the ratio of PGA/PGV [8]: (i) high frequency (HF) content when PGA/PGV > 1.2, (ii) intermediate frequency content when 1.2 ≥ PGA/PGV ≥ 0.8, and (iii) low frequency (LF) content when PGA/PGV < 0.8.

There are some studies on the effect of earthquake frequency content on different structures considering soil-structure interaction, such as storage tanks [9] and retaining walls [10]. However, to the knowledge of the authors, the specific study of the effect of the earthquake frequency content on the seismic soil-structure interaction problem of horseshoe tunnels with the nonlinear analysis is still limited. Due to the importance of the problem, a study is necessary to investigate the soil-structure interaction and earthquake frequency content effects on tunnel lining behavior in an accurate analysis.

In this study, two uniform soil types with different properties and twenty earthquake records with different frequency contents are used to evaluate the responses of a horseshoe tunnel. The investigation leads to some new findings useful for practical applications in terms of soil-structure interaction and earthquake frequency content.

2 Numerical simulation

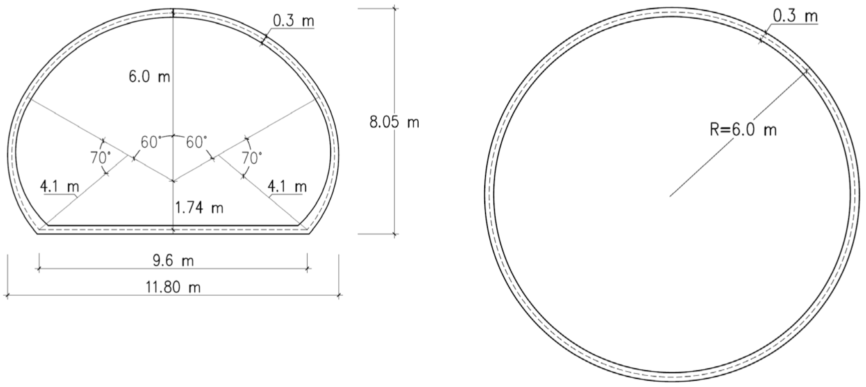
Numerical analyses were performed using the FLAC^{2D} software [11]. This software, a two-dimensional (2D) explicit finite difference code, was implemented because it is capable of modeling the nonlinear behavior of the soil and soil-structure interaction.

2.1 Structural modeling

The tunnel structure modeled in the numerical simulation was chosen from an existing road tunnel in Korea, as presented in Fig. 1(a). The tunnel is located 29 m below the ground surface. The cross-section of the tunnel has a total width of 11.8 m and a maximum height of 8.05 m. The lining thickness is 0.3 m. The tunnel structure was modeled as an elastic behavior material using beam elements with an element size of 0.5 m. The input properties used for structural elements are listed in Table 1.

Table 1. Properties of tunnel elements.

Parameter	Value
Density (kg/m^3)	2500
Young's modulus (GPa)	22.8
Poission’s ratio	0.2



(a) Horseshoe tunnel (b) circular tunnel (reference model)

Fig. 1. The geometry of the (a) horseshoe tunnel and (b) circular tunnel cross-section.

2.2 Soil modeling

Two uniform soil profiles, shear wave velocity (V_S) of 200 m/s and 400 m/s, were used in this study. Properties of the soil are summarized in Table 2.

Table 2. Properties of the soil material.

Parameter	Value
Density (kg/m^3)	1800
Shear wave velocity, V_S (m/s)	200, 400
Poission's ratio	0.3
Small strain damping (%)	1

The numerical model of the tunnel and soil profile is displayed in Fig. 2. The dimensions of the 2D soil model were set to 120 m x 60 m (width x height). The width of the soil model was selected based on a sensitivity study conducted such that the waves reflected from the lateral boundaries do not influence the seismic response of the tunnel. The height of the soil domain is the depth from the ground surface to the bedrock at the investigated site. The soil medium was modeled using plane-strain quadrilateral elements. The element size of 0.5 m was selected based on the recommendation of Kuhlemeyer and Lysmer [12]:

$$\Delta l = \frac{\lambda}{10} \div \frac{\lambda}{8} \tag{1}$$

where λ is the wavelength of propagated wave corresponding to maximum frequency interested. The $Sig3$ model, which is available in FLAC^{2D}, was employed to simulate the nonlinear behavior of soil.

The Rayleigh damping was used to model small strain damping, is expressed as follows [13]:

$$[C] = \alpha[M] + \beta[K] \tag{2}$$

where $[C]$ is the damping matrix, $[M]$ is the mass matrix, $[K]$ is the stiffness matrix, α and β are the Rayleigh coefficients.

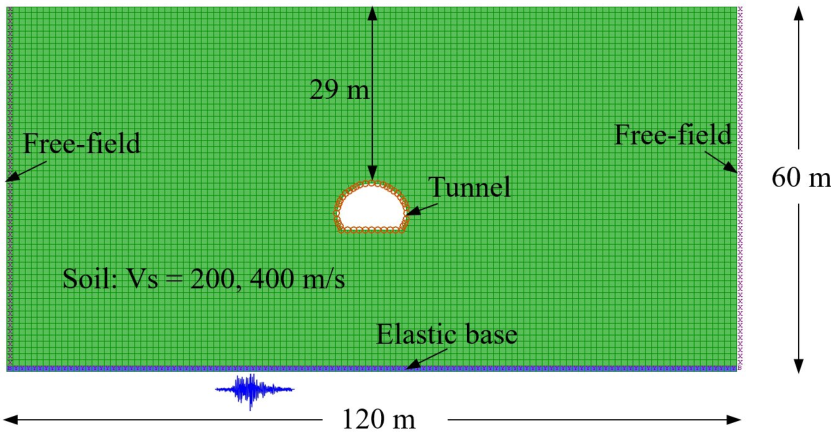


Fig. 2. Numerical model of the tunnel and soil domain.

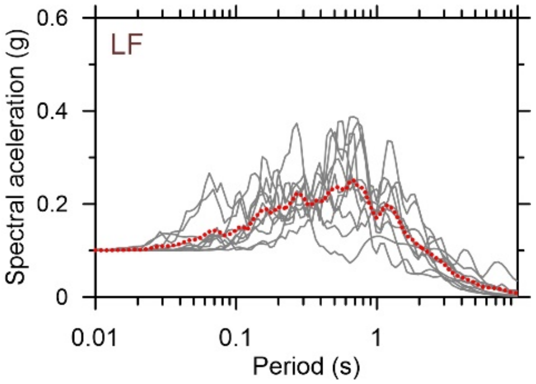
2.3 Soil-tunnel contact and boundary conditions

The soil-structure contact was simulated using the interface elements. The interface option UNBONED in the FLAC^{2D} software was used in this study. This contact interface can model a realistic partial-slip condition, considering the gapping and the slipping phenomena between soil and tunnel under loading.

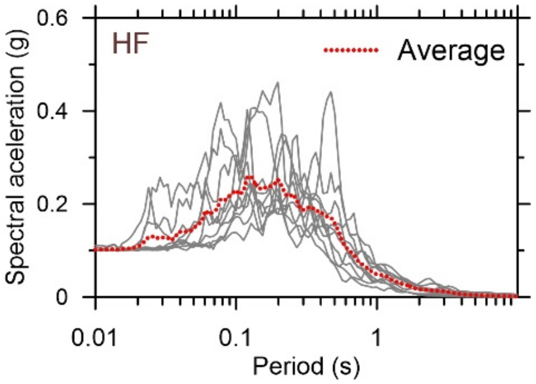
The bottom boundaries are used elastic boundary (a quiet boundary) to absorb reflected waves. The input motion was applied as a stress-time history at the base of the numerical model. Lateral boundaries of the numerical models were simulated using free-field boundary. The free-field boundary aims to create an infinite soil domain in the horizontal direction, as the actual condition. It thus avoids the wave reflections at the boundaries.

2.4 Earthquake motions

For this study, 20 motion records were selected from worldwide earthquakes provided by the NGA-West2 database (https://ngawest2.berkeley.edu). Two groups of ground motions, including low and high frequency content, are classified separately. Each group contains ten ground motions. The input ground motions are then scaled to 0.1 g and 0.6 g PGA. The response spectra of the 0.1 g input ground motion are shown in Fig. 3.



(a) Low frequency content



(b) High frequency content

Fig. 3. Response spectra of input ground motions.

3 Model validation

Before the parametric study step, the numerical model is first needed to validate. This research focuses on the parametric study of an unconventional shape to evaluate the tunnel lining response. As studied for the first time, it lacks analytical solutions or experimental tests. A reference model was developed and compared the results with available analytical solutions of Wang [14] and Penzien [15] to validate the numerical modeling method. The reference model has a circular section with an equivalent diameter of a horseshoe tunnel. The detail of circular section geometry is presented in Fig. 1(b). The material properties, boundary conditions, soil domain size of the reference model are identical to those of the baseline numerical model mentioned earlier. Notably, one soil profile, which has V_S = 200 m/s, was employed in the validation step. Two 0.6 g ground motions, one was selected from low frequency content (case 1), and the other was selected from high frequency content (case 2), were used as input loading in the reference model. Moreover, the no-slip condition was adopted to be consistent with analytical solutions.

Fig. 4 shows the comparisons of the bending moment calculated from numerical simulation and analytical solutions for two cases. The dashed line (i.e. the 1:1 line) indicates target values. The closer scattering to the 1:1 line, the lower difference between numerical and analytical methods. As can be seen from the figure, the bending moment from numerical results is an almost perfect match with analytical solutions. It proves that the numerical model is reliable for further parametric studies.

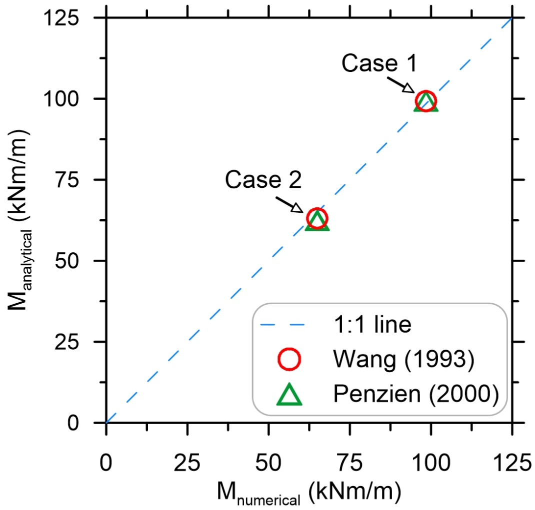


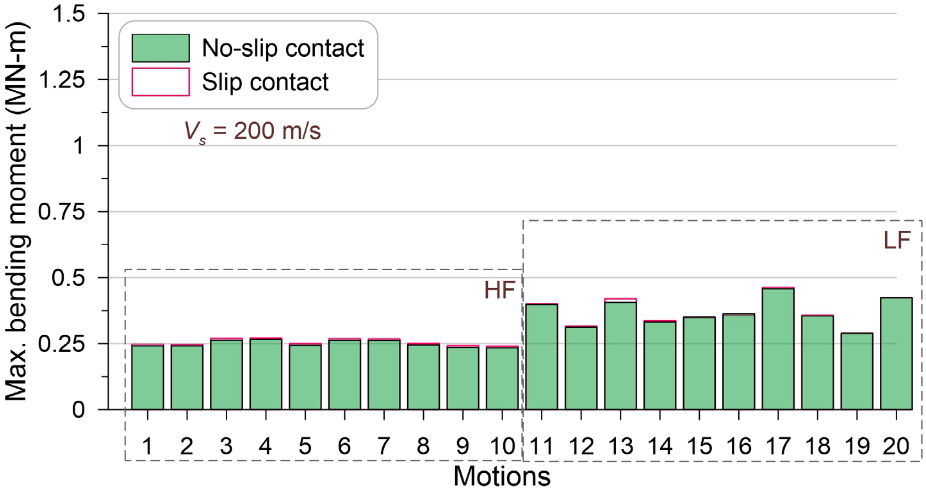
Fig. 4. Comparisons of the bending moment between numerical and analytical solution methods.

4 Results and discussions

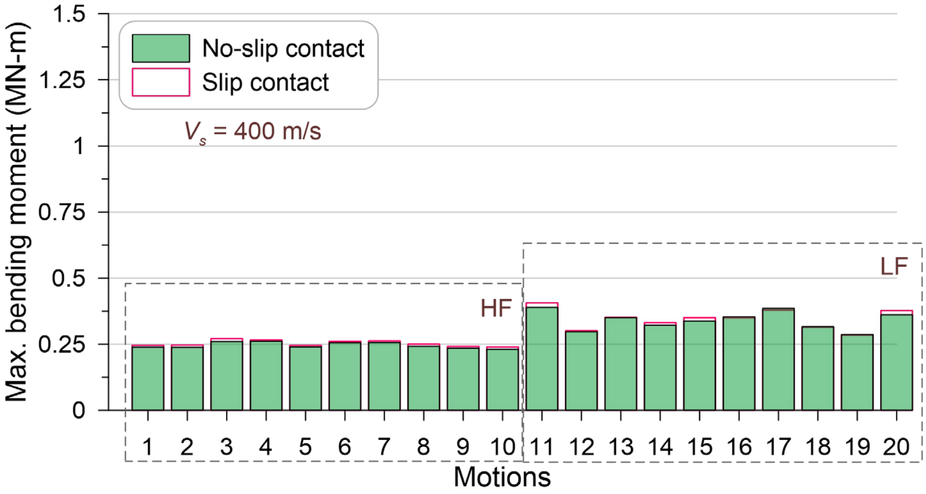
A total of 160 analyses was performed in the parametric study step. The numerical results are presented in Fig. 5 and Fig. 6.

The bending responses of the tunnel lining in the case of 0.1 g input motions are shown in Fig. 5. The effect of earthquake frequency content is evident for both V_S = 200 m/s and V_S = 400 m/s. The lower frequency content of earthquake causes a higher tunnel response (i.e. bending moment) compared to that of higher frequency content. However, the effect of soil-tunnel contact seems to be negligible in this case.

Fig. 6 presents the lining bending moment response in the case of 0.6 g input motions. The difference of bending moment between LH and HF is more obvious compared to that of the lower intensity of input motions (i.e. 0.1 g PGA). The effect of soil-tunnel contact is also more apparent. This is because the nonlinearity of soil increases when the increasing intensity of the input motion. It results in higher flexibility of tunnel lining. The tunnel located in the case of V_S = 400 m/s generally causes a lower lining response compared to that of V_S = 400 m/s. It indicates that tunnel embedded in stiff soil would be safer than soft soil when subjected to a seismic loading.

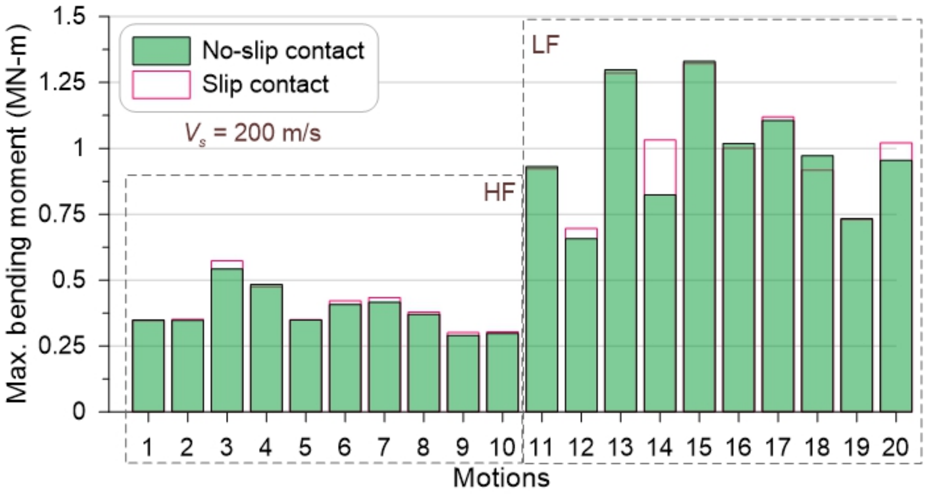


(a) V_S = 200 m/s

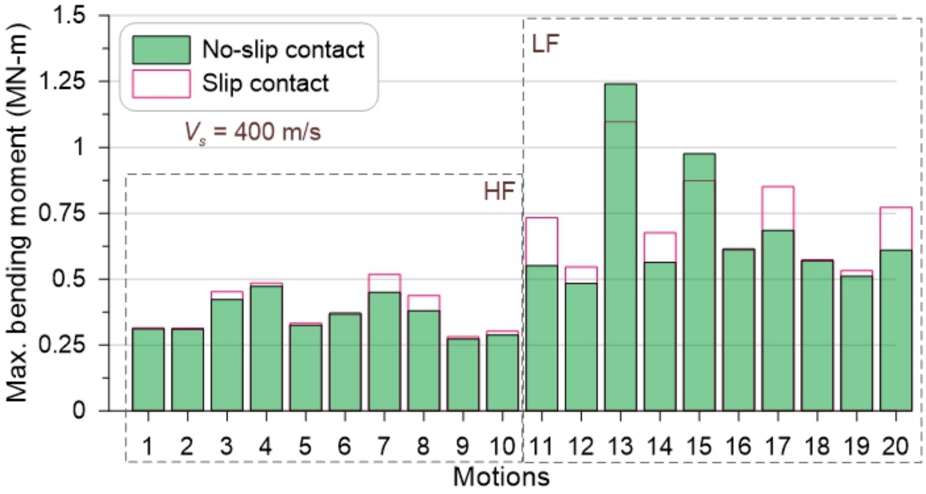


(b) V_S = 400 m/s

Fig. 5. Tunnel responses in the case of 0.1 g input ground motions.



(a) $V_S = 200$ m/s



(b) $V_S = 400$ m/s

Fig. 6. Tunnel responses in the case of 0.6 g input ground motions.

5 Conclusions

This study presents the effect of earthquake frequency content on the seismic response of horseshoe tunnels. Effect of the soil-tunnel contact, the intensity of input motions, the soil stiffness are also investigated. The numerical simulation models were developed and validated with analytical solutions. A nonlionear constitutive model was used for surrounding soil. A total of 160 analyses were performed for parametric studies. The following conclusions are drawn.

(1) The earthquake frequency content has a significant effect on the tunnel response. With the increase of ground motion intensity, the difference in tunnel response between LF and HF is increased.

(2) The effect of soil-tunnel contact rises with increasing intensity levels. In the case of 0.1 g input motions, that effect is small and can be ignored. However, in the case of 0.6 g input motions, the effect is noticeable.

(3) The higher soil stiffness, the lower the tunnel response is observed. It indicates that the tunnel located in stiff soil would be less vulnerable than soft soil when subjected to seismic loading.

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