Seismic failure mechanism of shallow cut-and-cover tunnels

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ABSTRACT
The objective of this paper is to study the failure mechanisms and damage states of rectangular cut-and-cover tunnels under seismic loading. Single, double, and triple box tunnel sections are selected from various tunnel sections that are designed and constructed for subway systems in Seoul, Korea. For each type of tunnel sections, various aspect ratios of the tunnel opening are considered. Four different soil types are considered to investigate the effect of the soil stiffness. Tunnel structures are modeled using nonlinear beam-column elements to simulate an inelastic structural behavior due to the seismic load effect. Load-displacement relationships in flexural and shear modes are assigned independently to a nonlinear beam-column element. A series of normal and shear springs are attached to beam-column elements to consider the soil-structure interaction. Nonlinear static analyses (pushover) were performed to monitor the development of plastic hinges in flexure or shear. On the other hand, elastic analyses are also performed to obtain the corresponding elastic moments at the plastic hinge formation. Four damage states, namely, minor, moderate, extensive, and collapse are defined based on the development of plastic hinges, and failure mechanisms of box tunnels are identified. A damage index (DI) is proposed as the ratio of the elastic moment to the yielding moment at a critical cross-section. Each damage state corresponds to a damage index (DI). The numerical results reveal that single box tunnels are vulnerable to a flexural failure, while multiple box tunnels are vulnerable to a shear failure occurred at the interior columns. DIs of the single box tunnels are slightly changed with the increment of the aspect ratios, and DI ranges from 1.0 to 2.3. DIs of multiple box tunnels are significantly varied at which shear plastic hinges occurred with various aspect ratios, and DIs range from 1.0 to 3.5. The proposed damage states are readily applied for the performance-based seismic design of shallow rectangular tunnel structures.

1 INTRODUCTION

In general, underground structures are more well-performed than above-ground structures when subjected to an earthquake (Dowding and Rozan 1978). Nevertheless, some recently large earthquakes have revealed that underground structures can experience a severe damage during strong ground motions (Hashash et al. 2001). There are a number of researchers surveyed and observed the damages of tunnel structures through various historic earthquakes (Dowding and Rozan 1978; Hashash et al. 2001; Owen and Scholl 1981; Sharma and Judd 1991; Wang 1985; Wang et al. 2001; Wang and Zhang 2013). Kitagawa and Hiraishi (2004) and Nakamura et al. (1996) presented a detailed observation on the damage of Daikai station in the 1995 Kobe earthquake. These catastrophes give a reminder of the need to carefully take into account seismic loading and sufficiently understand the seismic performance of underground structures for the minimum vulnerability.

Underground structures are fully embedded in soil layers and primarily conforms to the ground deformation. While a seismic excitation is transmitted to on-ground structures through inertial forces, underground structures are directly subjected to the surrounding soil deformation. The seismic performance evaluation of such structures can be achieved by a pseudo-static or a full dynamic analysis. The pseudo-static analysis utilizes the free field horizontal deformation obtained from a one-dimensional site response to impose on the surrounding soil of the structure. This method omits the dynamic interaction between the soil and the structure as well as inertial effects. Some researchers applied pseudo-static methods to evaluate seismic responses of tunnels such as Bobet (2003), Debiasi et al. (2013), Hashash et al. (2001), Hashash et al. (2010), Huo et al. (2006), Park et al. (2009), and Wang (1993). Argyroudis and Pitilakis (2012) performed pseudo-static and dynamic analyses for a specific single rectangular and a circular tunnel where the authors pointed out that the difference between two methods is insignificant.

A nonlinear analysis ensures a more accurate evaluation of the seismic performance of underground structures than a linear elastic analysis. Liu and Liu (2008) applied a pushover analysis method for identifying the failure mechanism of the Daikai station during the 1995 Kobe earthquake. They concluded that the occurrence of shear plastic hinges in the center column caused the collapse. Lee et al. (2016) performed pushover analyses with the pseudo-static analysis for three specific types of cut-and-cover tunnels in South Korea. Capacity curves, sequences of plastic hinges, and proposed damage states of the box tunnels were presented. The authors noted that the different geometry and dimensions of the box tunnels, the various depth of overburden, and shear behavior of tunnel frames should be investigated for completeness and enough generalization of proposed damage indices (DIs).

This study focuses on seismic failure mechanisms and updates the damage states of the cut-and-cover rectangular tunnels from pseudo-static inelastic frame analysis considering a variation of aspect ratios, effect of shear behavior of tunnel linings, and soil conditions. Three kinds of shallow box subway tunnels, which are single, double, and triple tunnels with various aspect ratios including 1.0, 1.5, and 2.0 are investigated. A total 36 combinations of nine structural shapes and four different soil conditions are modeled. Nonlinear beam-column elements are used for modeling tunnel linings and pushover analyses are carried out to develop capacity curves of tunnels and identify the failure mechanism of the tunnels. Finally, a set of damage states of the box tunnels is updated based on the number of plastic hinges formed at the structural members and the DI which is defined as the ratio of the elastic moment to yield moment of the structural members.

2 CONFIGURATION OF CUT-AND-COVER TUNNELS AND SITE CONDITIONS

In this study, three kinds of rectangular cut-and-cover reinforced concrete tunnels with various aspect ratios of the cross section are used as numerical examples, as shown in Fig. 1. Those are single, double, and triple box tunnels designed and constructed for subway systems in South Korea. The surrounding soil medium is assumed to be uniform and all the tunnel structures are located in soil with the overburden thickness of 7.0 m. The horizontal at-rest earth pressure factor K_0 of 0.5 is assumed for all soil profiles. The height (H) of all box tunnels is 6.0 m, while the width (B) varies from 6.0 m to 12.0 m corresponding to aspect ratios (B/H) from 1.0 to 2.0, respectively. It should be noted that H and B are center-to-center dimension. The thickness of sidewalls, top slab, and bottom slab is 1.0 m. The cross-sectional dimension of the inner columns of double and triple tunnels is 0.4x1.0 m, in which the longest side is parallel to the tunnel axis. The dimensions and reinforcement details of structural members are described in Fig. 2. It should be noted that the reinforcement ratio of structural members is compatibly increased with increment of the aspect ratios.

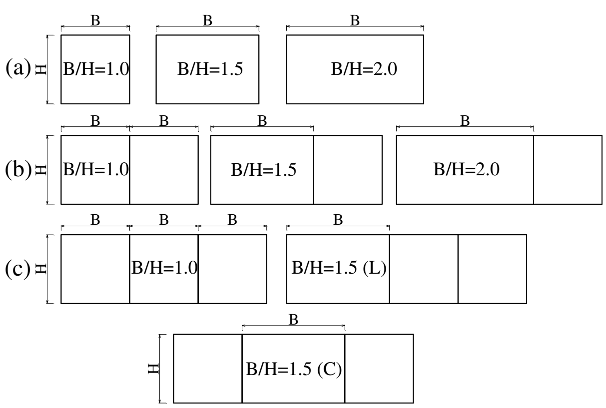

Figure 1. Investigated typologies of cut-and-cover tunnels (a) single, (b) double, (c) triple boxes

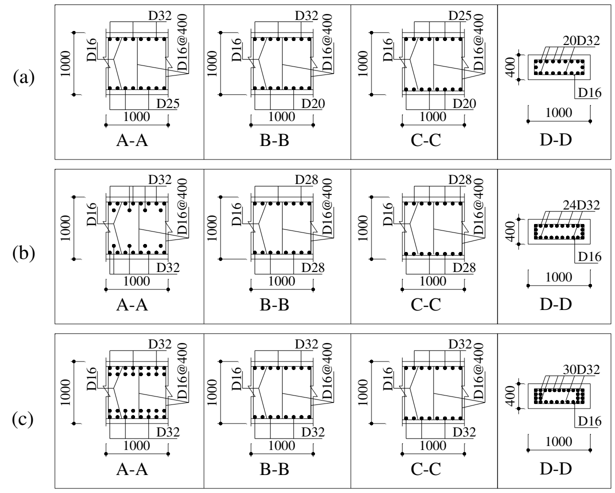

Figure 2. Dimensions and reinforcement details of structural members (a) B/H=1.0, (b) B/H=1.5, and (c) B/H=2.0. Note that A-A is used for top slabs, B-B for sidewalls, C-C for bottom slabs, and D-D for interior columns.

3 NUMERICAL MODELING

The pseudo-static procedure is applied for performing nonlinear analyses of the tunnel structures. Three types of tunnels are modeled in terms of two-dimensional (2D) inelastic frame elements. Only the racking of the tunnels is considered in the analysis because it is the predominant deformation under an earthquake and a common approach for the seismic design of a rectangular tunnel. The 2D frame model is selected for the following reasons: (1) a continuum model including soil and structure elements is relatively complicated in practice; (2) if the subgrade springs are appropriately calculated, the difference of numerical results from a continuum and frame model is unnoticeable (Chang et al. 2014).

The tunnel structures are modeled using SAP2000 (ver. 15), a finite element structural analysis program (CSI 2011). Structural members are divided into a number of elements. It is noted that the joint-offset features are used to model the rigid wall-slab and column-slab connections. Sixty-four elements per each structural member are selected for boxes with aspect ratio B/H = 1.0, while 96 and 128 elements are used for slabs with aspect ratios B/H = 1.5 and 2.0, respectively, after convergence tests. Plastic hinge models in flexure and shear are used for all frame elements. The formation of a plastic hinge, in either flexure or shear, is considered as an indicator of damage. The performance of a plastic hinge is dictated by the moment-curvature and shear force-shear deformation relationships that are determined by a nonlinear section analysis. Fig. 3 shows stress-strain relationships of concrete and reinforcing bar used in section analyses. The nominal compressive strength and the elastic modulus of concrete are 27.5 MPa and 24.8 GPa, respectively. Meanwhile, the yield strength and the elastic modulus of reinforcement are 413 MPa and 200 GPa, respectively. Fig. 4 shows moment-curvature relationship of section A-A. For the shear strength-shear deformation relationship of a section, we adopted the definitions and guidelines of ACI-318 (2008), FEMA-356 (2000), and Park and Paulay (1975) to determine the yield strength (V_y), ultimate strength (V_u), and the corresponding yield deformation (Δ_y) and ultimate deformation (Δ_u), as shown in Fig. 5.

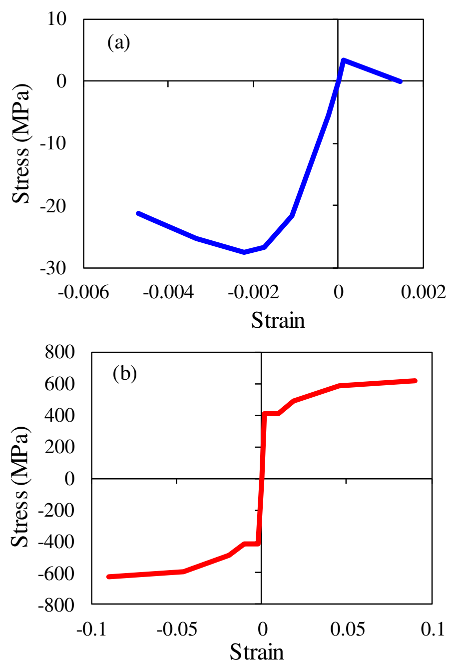

Figure 3. Material models for (a) concrete and (b) reinforcing bar

A series of normal and shear springs are attached to the nodes of the nonlinear frame elements to simulate soil-structure interaction. According to the seismic design code for metropolitan subway of Korea (MLTM 2009), the normal spring coefficients in horizontal (K_H) and vertical (K_V) directions are defined as

$$K_H = k_{h0}\left(\frac{h}{30}\right)^{-3/4} \quad [1]$$

$$K_V = k_{h0}\left(\frac{b}{30}\right)^{-3/4} \quad [2]$$

where, $k_{h0} = (\frac{1}{30})E_D$, h and b are the height and the width of the tunnel, respectively, E_D is dynamic elastic modulus as defined as $E_D = 2(1+\nu_D)G_D$, ν_D (=0.3) is Poisson's ratio of soil, G_D is dynamic shear modulus as in $G_D = (\gamma_t/g)V_s^2$, γ_t (=18KN/m^3) is density of soil, V_s is the shear velocity of the surrounding soil, and g is the acceleration of gravity. The shear spring coefficients in horizontal (K_{SB}) and vertical (K_{SS}) directions are defined as

$$K_{SB} = \frac{1}{4}K_V \quad [3]$$

$$K_{SS} = \frac{1}{4}K_H \quad [4]$$

Fig. 6 shows the boundary conditions and imposed loads on the tunnel. The pseudo-static analysis procedure in Iai (2005) is adopted in this study. Based on this guideline, the loads acting on the tunnel are (1) the overburden and horizontal geostatic pressures, (2) the free field soil deformation, and (3) the shear stress at the structure interfaces. In order to determine the free-field deformation and shear strain, a one-dimensional (1D) site response analysis is carried out. The shear strain is assumed to be uniform within the soil profile, thus, the soil displacement is applied as an inverted triangular shape.

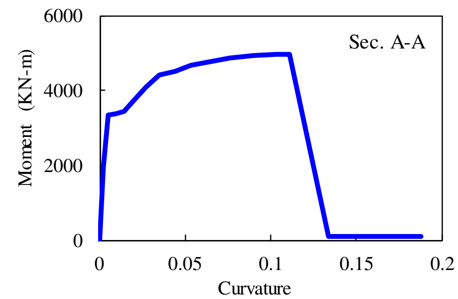

Figure 4. Moment-curvature relation of section A-A for B/H = 1.5

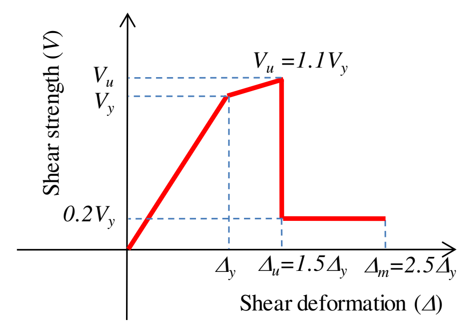

Figure 5. Force-deformation relation of shear behavior

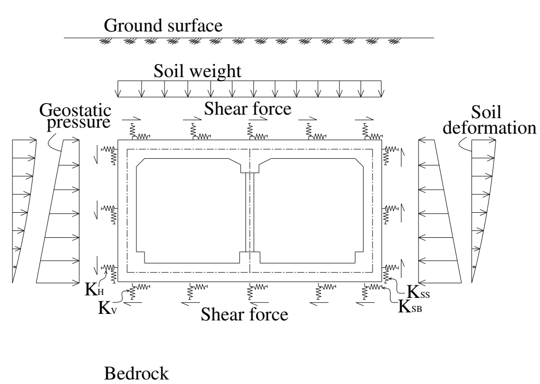

Figure 6. Boundary condition and loads to the tunnel

To evaluate the nonlinear behavior of the tunnel, we performed a series of nonlinear static analyses, also known as pushover analyses, for various soil profiles represented by shear wave velocities. Four kinds of site profiles corresponding to shear wave velocities, which are 100, 200, 300, and 400 m/s, are selected. Due to an assumption of soil linearity, the reduction of shear modulus with the shear strain is neglected. Based on the pushover analysis, the base force-shear deformation curve, formation and propagation of plastic hinges in the structure, and failure mechanism of the tunnels are adequately captured.

4 FAILURE MECHANISM OF THE TUNNELS

The capacity curve is one of the most useful outcomes of the pushover analysis, which is described in terms of a force-displacement relation. Based on this result, we can easily evaluate the change of the strength as well as the ductility capacity of the structure. In addition, another beneficial outcome of the pushover analysis is the formation of plastic hinges in the structural members. This effective indicator shows not only the nonlinear performance but also the failure mechanism of the tunnels. The cumulative failure of tunnel structures is thoroughly presented in this section.

4.1 Single box tunnel

The capacity curves of the single box tunnels are shown in Fig. 7 for a variation of soil profiles and aspect ratios. It can be seen that the base force increases with the increment of the soil stiffness in terms of the shear wave velocity. Because of the difference in the stiffness of soil, a stiffer soil (with a larger shear wave velocity) having a smaller free field shear strain, yields larger base force, and vice versa. Fig. 7 also implicates that the capacity of the tunnel is increased with an increment of the aspect ratios. It can be attributed that the larger aspect ratios, the more lateral resistance capacity produced by shear springs at the slabs of the tunnel frame. Besides, the longitudinal reinforcement ratio of the tunnel linings is compatibly increased with the widening of the aspect ratios.

Figs. 8-10 show the formation and development of plastic hinges for the single box tunnels with the variation of aspect ratios and shear wave velocities, where the dots indicate plastic hinges. It can be seen that the free field shear strain at which plastic hinges formed is slightly increased together with an enlargement of the aspect ratios. The plastic hinges form at lower strains, but at higher base force, for higher shear wave velocities (i.e., for V_s = 400 m/s the first plastic hinge forms at about a shear strain of 0.06%, whereas the plastic hinge does not form until about a strain of 0.3% when V_s = 100 m/s). The single box structure would be collapsed if it was located on the ground when four plastic hinges formed. However, the tunnel structure is always enclosed by the soil medium that supports and prevents the tunnel from a collapse state. It should be noted that a shear plastic hinge did not form.

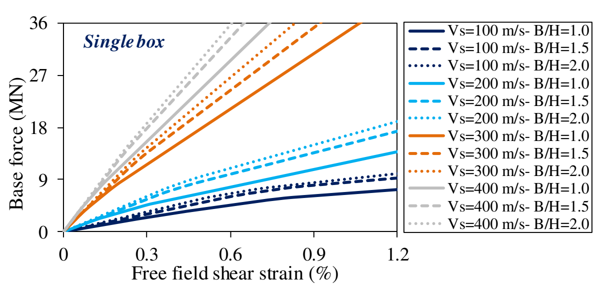

Figure 7. Capacity curves of the single box tunnels

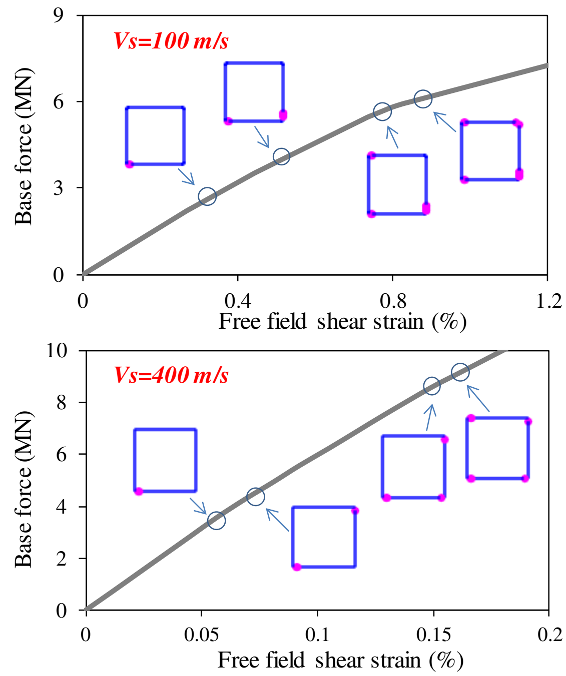

Figure 8. Development of plastic hinges for single box tunnel with B/H=1.0

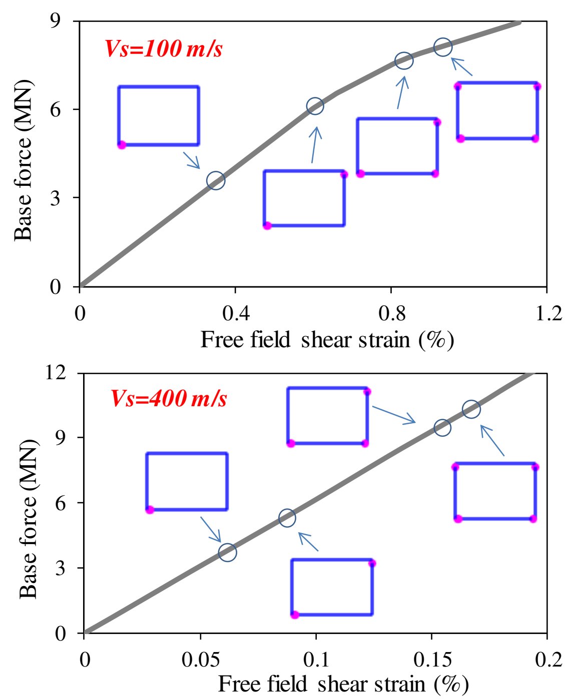

Figure 9. Development of plastic hinges for single box tunnel with B/H=1.5.

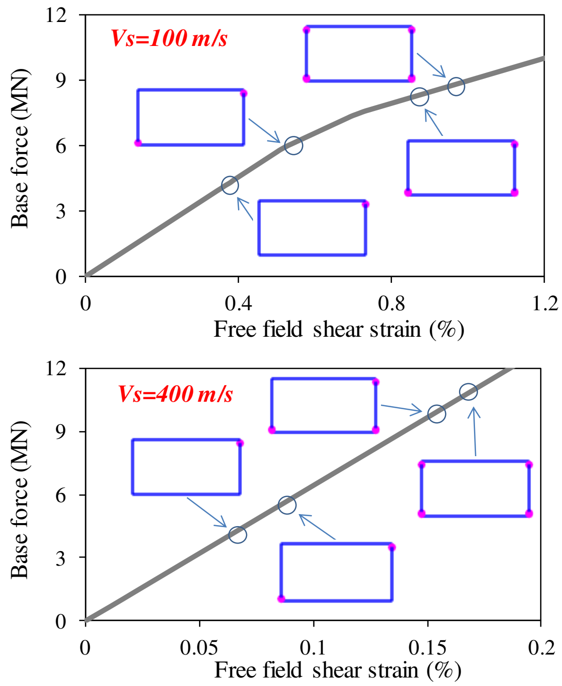

Figure 10. Development of plastic hinges for single box tunnel with B/H=2.0

4.2 Double box tunnel

Fig. 11 illustrates the capacity curves of the double box tunnels for various aspect ratios and soil conditions. Similar to the single tunnels, it also can be seen that the base force of the double tunnel structure is increased with the increment of the shear wave velocity of the surrounding soil medium. In other words, a softer soil condition (with a smaller shear wave velocity) having a larger free field shear strain, produces a lower base force, and vice versa.

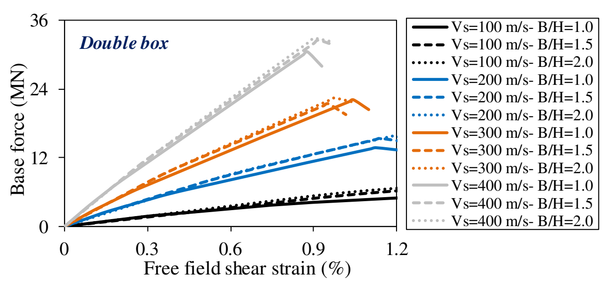

Figure 11. Capacity curves of the double box tunnels

Figs. 12-14 present the formation and development of plastic hinges in the double box tunnels with various aspect ratios for the cases of *Vs* = 100 m/s and *Vs* = 400 m/s. The pink dots and heavy pink lines indicate locations of plastic deformation and heavy orange line indicates failure. It should be noted that the shear plastic hinges formed in the interior column after four flexure plastic hinges formed at all corners of the outer frame. The free field shear strains at the initiation of shear failure of the double tunnels are approximately 1.5% and 0.7 % for *Vs* = 100 m/s and *Vs* = 400 m/s, respectively. Moreover, the shear plastic hinges promptly propagate to the center of the interior columns, consequently led to losing axial load capacity in the center column, and eventually caused the collapse of the box tunnel structure. This trend is observed for all three aspect ratios of the double tunnel.

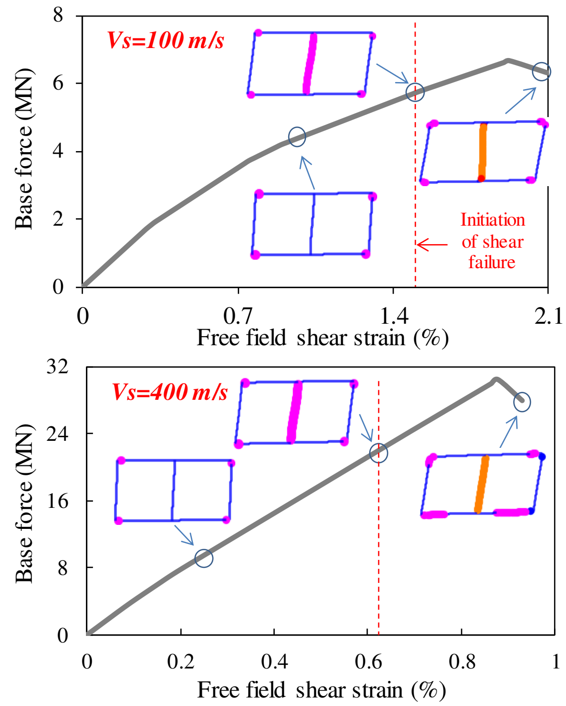

Figure 12. Development of plastic hinges for double box tunnel with B/H=1.0

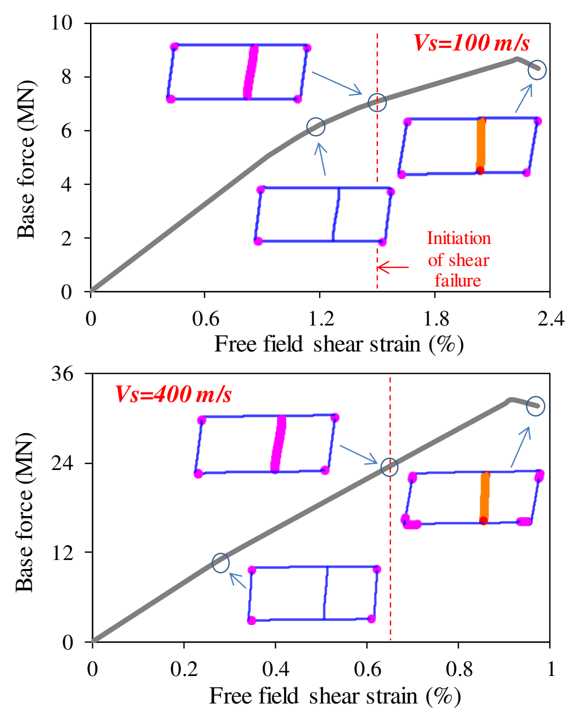

Figure 13. Development of plastic hinges for double box tunnel with B/H=1.5

The similar collapse mechanism of reinforced concrete columns under earthquakes were also found in elsewhere (An and Maekawa 1997; Moehle et al. 2001; Yoshimura et al. 2004).

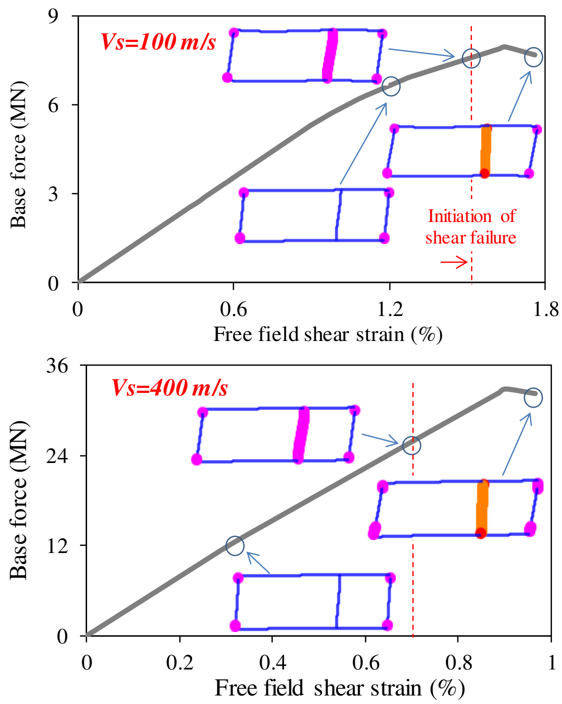

Figure 14. Development of plastic hinges for double box tunnel with B/H=2.0

4.3 Triple box tunnel

The capacity curves of the triple box tunnel for the variation of soil conditions and aspect ratios are shown in Fig. 15. Similar to the single and double box tunnels, a softer soil (a smaller shear wave velocity), having a larger free field shear strain, produces a lower base shear, and vice versa. It should be noted that the capacity of the cases B/H=1.5L and B/H=1.5C are almost identical. Fig. 16 illustratively shows the formation and development of plastic hinges for various aspect ratios and the soil conditions of *Vs* = 100 m/s and *Vs* = 400 m/s. It is also seen that the shear plastic hinges in the interior columns form after flexure plastic hinges form at all four corners of the outer frame. The free field shear strains at the initiation of shear failure of the triple tunnels are approximately 1.75% and 0.8% for *Vs* = 100 m/s and *Vs* = 400 m/s, respectively. Even though the shear plastic hinges formed in the interior columns, the complete collapse state is not really observed for the triple box tunnel. Because of the important role of the interior columns, as soon as the plastic hinges occurred in such column we may consider the multi-box tunnel structure is closely approaching to a possible collapse state.

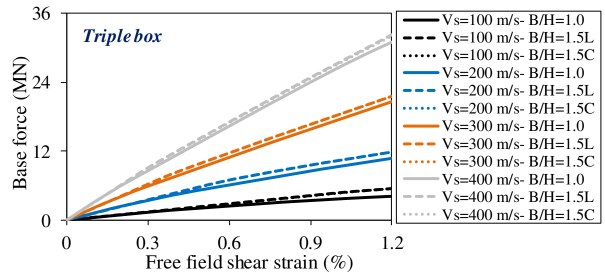

Figure 15. Capacity curves of the triple box tunnels

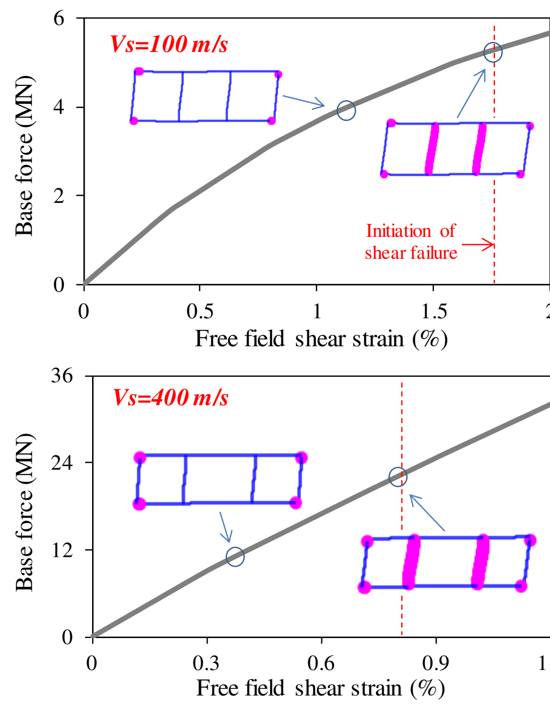

Figure 16. Development of plastic hinges for triple box tunnel with B/H=1.0 (up) and B/H=1.5C (down)

5 DAMAGE STATES OF TUNNELS

A set of damage states plays an important role for the seismic vulnerability assessment of structures. It is closely related to damage indices which are generally defined in terms of the structural response. In this study, we adopted the definition of damage states proposed by Lee et al. (2016) to apply for various aspect ratios of the cut-and-cover tunnels. DI is expressed as the ratio between the elastic moment demand (M) and the yield moment (M_y). Moreover, each damage state is also linked to the number of plastic hinges formed at the structure corners. For a single box tunnel, three damage states including minor, moderate, and extensive are classified. Similar to the proposal of Lee et al. (2016), the collapse state is not defined herein because the single box structure is not likely to collapse due to the formation of plastic hinges. Meanwhile, four damage states for the double and triple box tunnels are defined, which are minor, moderate, extensive, and collapse. We suggest adding the ‘collapse’ damage state for the multiple box tunnels because of a severe damage at the interior column, thus, a collapse is very probable. Fig. 17 shows representative examples of damage indices (M/M_y) for different tunnel types, aspect ratios, and soil conditions. The free field shear strain is shown on a logarithmic scale for a visualized purpose. The markers on the curves represent the development of plastic hinges in the tunnels. As displayed in the figures, the first four plastic hinges at the outer corners of all tunnel types are governed by a flexural yielding. For the cases of double and triple tunnels, the fifth plastic hinge in the interior columns is mostly due to a yielding in shear. Moreover, the shear plastic hinges form at entire interior column sections almost simultaneously. Table 1 shows the damage states, number of plastic hinges (NPH), and DIs of the investigated box tunnels in this study.

It should be noted that this study considers only a constant thickness of tunnel frames. A future study on the effect of various tunnel linings thickness on DI should be conducted.

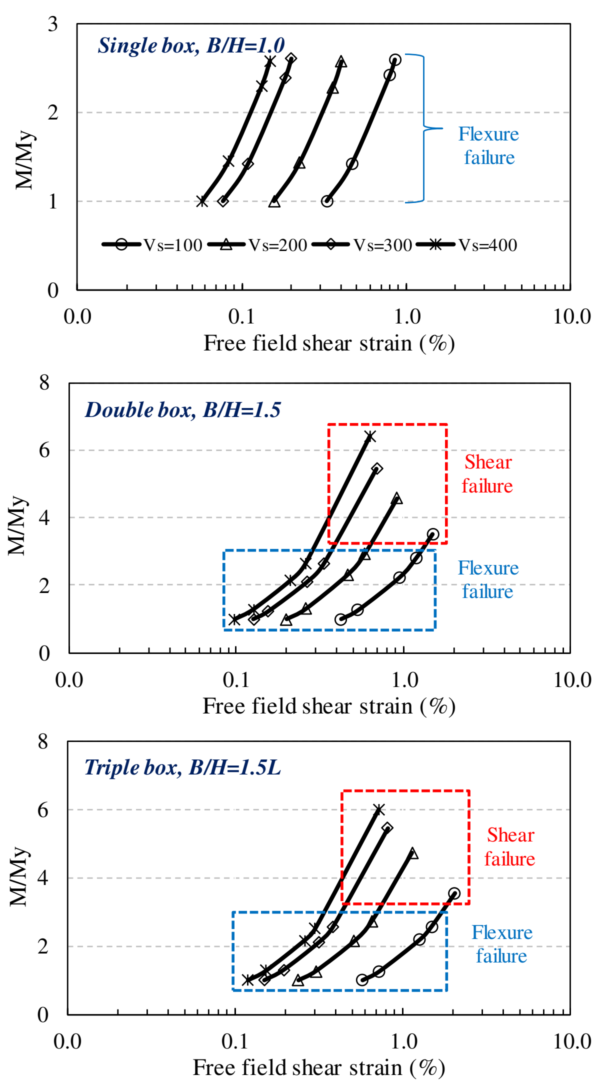

Figure 17. *M/My* ratios at the formation of plastic hinges for the box tunnels

Table 1. Proposed damage states and damage indices

Tunnel type	Damage state	Number of plastic hinges (NPH)	Damage index (DI, M/My)
Single box	None	0	DI < 1.0
	Minor	1 ≤ NPH < 2	1.0 ≤ DI < 1.4
	Moderate	2 ≤ NPH < 3	1.4 ≤ DI < 2.3
	Extensive	3 ≤ NPH	2.3 ≤ DI
Double box Triple box	None	0	DI < 1.0
	Minor	1 ≤ NPH < 2	1.0 ≤ DI < 1.2
	Moderate	2 ≤ NPH < 3	1.2 ≤ DI < 2.1
	Extensive	3 ≤ NPH < 5	2.1 ≤ DI < 3.5
	Collapse	5 ≤ NPH	3.5 ≤ DI

6 CONCLUSIONS

The seismic failure mechanisms of rectangular cut-and-cover tunnels were identified and damage states of such tunnels were proposed. Three kinds of structures designed for subway systems in South Korea including single, double, and triple box tunnels with various aspect ratios (B/H) were investigated. Four different soil conditions characterized by shear wave velocities, which are Vs = 100, 200, 300, and 400 m/s, were also considered. Nonlinear frame elements are applied for modeling tunnel structural members with a series of normal and shear springs to take into account the soil-structure interaction. The pushover analyses with the pseudo-static analysis procedure are implemented to capture the sequence of plastic hinges and capacity curves of the box tunnels. Based on the numerical results, the following conclusions are drawn:

(1) The formation and development of plastic hinges mostly depend on the relative stiffness between the tunnel and the surrounding soil medium. For double and triple box tunnels, shear plastic hinges formed at the interior columns after flexural plastic hinges formed at all four corners of the outer frame.

(2) For the single box tunnels, a collapse state was not observed even when plastic hinges formed at all corners, primarily due to the support from the surrounding soil medium. Meanwhile, the occurrence of a shear failure at the interior columns can lead to a collapse of multi-box tunnels (i.e. double and triple boxes). In a design practice, a ductile detailing in center columns should be considered for preventing the shear failure.

(3) A set of updated damage states of box tunnels and corresponding DIs are proposed. The NPH is used for defining the damage states of the tunnels because it clearly reflects the cumulative failure of the structures under seismic loading. A qualitative DI is expressed as the ratio between the elastic moment (M) and the yield moment (My) at the corners of the box tunnel. DIs range from 1.0 to 2.3 for damage states of the single tunnels and range from 1.0 to 3.5 for damage states of the double and triple tunnels.

(4) The variation of aspect ratios does not change the DIs of the single box tunnels significantly. However, for the

double and triple tunnels, DIs are notably varied at which shear plastic hinges occurred with various aspect ratios.

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