

The deformation of overburden soil induced by thrust faulting and its impact on underground tunnels

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Abstract

Overburden soil beds situated above a fault are often deformed by propagation of bedrock thrusting from the fault during large earthquake. The deformed beds formed a triangular shear zone. This coseismic faulting often causes damage to underground tunnels located in the shear zone. The present research studies the deformation behavior of the overburden soil beds and the tunnel, the associated mechanism and the impact on the safety of tunnel linings induced by a large blind thrust slip. Based on sandbox experimental and numerical studies, it is found that results from numerical analysis are in agreement with the sandbox model tests with regard to growths of the shear zones within the soil beds, location of the tunnel in this shear zone and deformations of the tunnel. The potential major shear zone may be bent or bifurcated into two sub-shear zones owing to existence of a tunnel inside the shear zone. Furthermore, the occurrence of back-thrust faulting will threaten the safety of nearby structures. It was also identified that stiffness of the soil and the fault dip angles are among the major factors controlling the configuration of shear zones, the stresses within the soil, and the loads on tunnel linings. Based on the identified mechanisms, the strategies for hazard prevention are accordingly suggested and discussed.

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1. Introduction

Major earthquakes often induce coseismic ground ruptures, which are the consequence of bedrock thrusting and subsequent propagation of a thrust-fault tip upward to the surface of the ground. Results from trench investigation of fault profiles indicate that a major fault zone may contain several sub-faults within the overburden soils, which is often referred as “sediments” by geologists, and the fault may continuously slip upon

subsequent major earthquake events (Chen et al., 2004). Coseismic faulting is often accompanied by damage to engineering structures. These fault-induced damages included failures of houses, roadways, railways, bridges, viaducts, dams, tunnels, lifelines, foundations, and piles, etc. (Shearad et al., 1974; Bonilla, 1982; O'Rourke and Palmer, 1996; Prentice and Ponti, 1997; Murbach et al., 1999; Kung et al., 2001; Ulusay et al., 2001; Bray, 2001; Takada et al., 2001; Kelson et al., 2001a,b; Backblom and Munier, 2002; Kawashima, 2002b; Dong et al., 2003; Kontogianni and Stiros, 2003; Sorensen and Meyer, 2003; Cluff et al., 2003; Honegger et al., 2004; Anastasopoulos, 2005; Donmez and Pujol, 2005; Konagai, 2005; Bray and Kelson, 2006).

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Intensive studies of deformational behaviors of overburden beds induced by faulting have been conducted. (Roth et al., 1981, 1982; Cole and Lade, 1984; Lade et al., 1984; Tani et al., 1994; Bray et al., 1994a,b; Lazarte and Bray, 1996; Bray, 2001; Erickson et al., 2001; Johnson and Johnson, 2002; Finch et al., 2003, 2004; Lee and Hamada, 2005; Lin et al., 2005, 2006a; Hardy and Finch, 2006; Johansson and Konagai, 2006). Beds of overburden soil deformed by propagation of thrust fault situated underneath comprise severely distorted zone (fault zone) within a triangular shear zone. Although one major fault slip surface can be developed, if the slip is large enough, subsidiary faults may also form within the overburden soils. Lin et al. (2006a) explored the processes of thrust faulting within overburden soil and examined the influences of controls under a range of boundary conditions using physical sandbox models and numerical analysis. A much larger slip, up to 30% of the overburden soil thickness, was conducted in the research to observe consequences induced by such a high degree of distortion. The results revealed that Young's modulus and the dilation angle are the most influential parameters in models. A stiffer E leads to early fault propagation and the occurrence of back-thrusts, and causes more significant discrepancies in the deformation behaviors of models with different scales. A greater dilation angle of the overburden soil leads to a wider fault zone.

In-depth studies on the impact of fault movement on engineering structures based on numerical approaches (Duncan and Lefebvre, 1973; Kennedy et al., 1977; Wang and Yeh, 1985; Bray, 1990; Lazarte, 1996; Bray, 2001; Takada et al., 2001; Kawashima, 2002a; Dong et al., 2003; Chung et al., 2005; Anastasopoulos, 2005; Lin et al., 2005; Karamitros et al., 2007) and experimental analysis using scale models (Trautmann and O'Rourke, 1985; Trautmann et al., 1985; Burrige et al., 1989; Bray, 1990; Lazarte, 1996; Chung et al., 2005; Lin et al., 2005, 2006b; Johansson and Konagai, 2006; Nahas et al., 2006; Yilmaz and Paolucci, 2007) have also been carried out. Table 1 presents a summary of results of these studies. In these studies the main types of faults dealt with are thrust-fault and strike-slip fault, and the types of structures include tunnels, pipelines, foundations, buildings, dams, and bridges, etc. Both sandy and clayey soils have been studied. The development of major shear zone within the overburden soil is revealed and the mechanism is also discussed. Underground tunnel is the main topic of this study. The reported cases of fault induced tunnel damage include the Wrights Tunnel (San Francisco Earthquake, $M_L=7.9$, 1906), the Kern County Tunnel (Kern County Earthquake, $M_L=7.7$, 1952), the Inatori railway tunnel

(Off Izu Oshima Earthquake, $M_L=7.0$, 1978), the Rokko Tunnel (Kobe Earthquake, $M_w=7.2$, 1995), the Outlet Tunnel of Kakkonda hydropower station (Mid-North Iwate Earthquake, $M_w=6.1$, 1998), the Outlet Tunnel of Shih-Kang Dam (the Chichi Earthquake, $M_w=7.6$, 1999), etc. (Bonilla, 1982; Prentice and Ponti, 1997; Kung et al., 2001; Bray, 2001; Backblom and Munier, 2002; Kontogianni and Stiros, 2003; Konagai, 2005). Chung et al. (2005) and Lin et al. (2006b) discussed a case in which both the soil beds and the shield tunnel within these beds were deformed.

On the basis of these previous studies, the present research attempts to further explore the following issues:

- (1) The interaction between the growth of a shear zone and the tunnel within the overburden soil;
- (2) The stress states within soil, and load states on tunnel linings induced by blind thrusting. An index (danger factor), which has better capability than contours of plastic strain in illustrating the interaction between soil and tunnel, was used to evaluate how far the soil was from a failure state;
- (3) The influence of local controls (amount of slip, horizontal distance of the tunnel to the fault tip, soil properties) on the interaction between the growth of a shear zone and the tunnel;
- (4) The effect of loading induced by faulting on joints in segmental shield tunnel linings are considered in full-scale numerical analysis;
- (5) The influences of local controls (location of the tunnel, fault dip angle, soil properties) on loading states;
- (6) With the help of axial force–moment diagrams of segmental lining support, identify the parameters that truly relate to the safety of the tunnel lining.

2. Methodology

Full-scale experiment is undeniably the best method in studying the actual deformational behavior of overburden soil induced by thrust faulting, and its impact on underground tunnels, however such experiment, in reality, is far too costly and difficult to execute. In this research full-scale numerical analysis was adopted. However, to ensure that the numerical model could reveal reasonable predictions, physical sandbox model experiments, and numerical analyses were adopted. The results of physical sandbox experiments were compared with those produced by numerical analyses so that the correctness of the numerical simulation could be justified. As such, the numerical model, which had been calibrated to ensure its prediction on small-scale model,

Table 1

Summary of the existing studies for the fault movement on engineering structures

Methodology	Fault type	Overlying stratum/external action on structure	Type of structure	Deformation	Soil–structure interaction/soil/structure behavior
<i>Numerical analysis</i>					
Duncan and Lefebvre (1973)	Strike–slip faults	External force	Nuclear power plant	Cross-sectional	No/elasto-plastic/elastic
Kennedy et al. (1977)	Strike–slip faults	Beam–spring	Buried pipeline	Logitudinal	Yes/elastic /elasto-plastic
Wang and Yeh (1985)	Strike–slip faults	Beam–spring	Buried pipeline	Logitudinal	Yes/elastic /elasto-plastic
Bray (1990)	Normal/thrust faults	Clay	Earth embankments	Cross-sectional	Yes/elasto-plastic/elasto-plastic
Lazarte (1996)	Strike–slip faults	Clay	Earth embankments	Cross-sectional	Yes/elasto-plastic/elasto-plastic
Takada et al. (2001)	Thrust faults	Beam–spring	Buried pipes	Cross-sectional and longitudinal	Yes/elastic /elasto-plastic
Kawashima (2002a)	Thrust faults	Beam–spring	Bridge foundation	Logitudinal	Yes/elastic /elasto-plastic
Anastasopoulos (2005)	Normal faults	Soil	Building	–	Yes/elasto-plastic/elastic
Lin et al. (2005)	Thrust faults	Sand	Tunnel	Cross-sectional	Yes/elasto-plastic/elastic
Chung et al. (2005)	Thrust faults	Sand	Shield tunnel	Cross-sectional	Yes/elasto-plastic/elastic
Karamitros et al. (2007)	Strike–slip faults	Beam–spring	Buried pipeline	Logitudinal	Yes/elasto-plastic/elastic
Yilmaz and Paolucci (2007)	Normal/thrust faults	Undrained soil	Foundation	Cross-sectional	Yes/elasto-plastic/elastic
<i>Model experiment</i>					
Burridge et al. (1989)	Thrust faults	Sand	Tunnel	Logitudinal	–
Trautmann and O'Rourke (1985)	Strike–slip faults	Sand	Dam	–	–
Trautmann et al. (1985)	Strike–slip faults	Sand	Dam	–	–
Bray (1990)	Thrust faults	Clay	Earth embankments	–	–
Lazarte (1996)	Strike–slip faults	Clay	Earth embankments	–	–
Lin et al. (2005)	Thrust faults	Sand	Tunnel	Cross-sectional	–
Chung et al. (2005)	Thrust faults	Sand	Shield tunnel	Cross-sectional	–
Lin et al. (2006b)	Thrust faults	Sand	Shield tunnel	Cross-sectional	–
Nahas et al. (2006)	Normal faults	Sand	Foundation	–	–
Johansson and Konagai (2006)	Thrust faults	Sand	Pile; tunnel	Logitudinal	–
<i>Field studies</i>					
Shearad et al. (1974)	Normal/thrust faults	Soil	Dam	–	–
Bonilla (1982)	Normal/thrust faults	Soil	Pipeline	–	–
Lazarte et al. (1994)	Strike–slip faults	Soil	Foundation	–	–
O'Rourke and Palmer (1996)	Strike–slip faults	Soil	Pipeline	Logitudinal	–
Prentice and Ponti (1997)	Strike–slip faults	Soil	Tunnel	Logitudinal	–
Murbach et al. (1999)	Strike–slip faults	Soil	Foundation, transmission tower	–	–
Bray (2001)	Normal/thrust faults	Soil	Building, Dam, Bridge	–	–
Ulusay et al. (2001)	Strike–slip faults	Soil	Building, bridge, tunnel	–	–
Kelson et al. (2001a)	Thrust faults	Soil	Building	–	–
Kelson et al. (2001b)	Thrust faults	Soil	Building	–	–
Kung et al. (2001)	Thrust faults	Soil	Dam	–	–
Takada et al. (2001)	Thrust faults	Soil	Buried pipes	Cross-sectional and longitudinal	–
Kawashima (2002b)	Strike–slip/thrust faults	Soil	Bridge foundation	–	–
Cluff et al. (2003)	Strike–slip faults	Soil	Pipeline	Logitudinal	–

Table 1 (continued)

Methodology	Fault type	Overlying stratum/external action on structure	Type of structure	Deformation	Soil–structure interaction/soil/structure behavior
<i>Field studies</i>					
Sorensen and Meyer (2003)	Strike–slip faults	Soil	Pipeline	Logitudinal	–
Kontogianni and Stiros (2003)	Strike–slip/thrust faults	Soil	Tunnel	Logitudinal	–
Dong et al. (2003)	Thrust faults	Soil	Foundation	–	–
Honegger et al. (2004)	Strike–slip faults	Soil	Pipeline	Logitudinal	–
Donmez and Pujol (2005)	Strike–slip faults	Soil	Building	–	–
Konagai (2005)	Strike–slip/normal faults	Soil	Tunnel; foundation	Cross-sectional	–
Anastasopoulos (2005)	Normal faults	Soil	Building	–	–
Bray and Kelson (2006)	Strike–slip faults	Soil	Pipeline; building; Earth structure	Logitudinal	–

was used to simulate the full-scale model. The simulation results were close to reality as much as possible.

The framework for execution of this research is illustrated in Fig. 1. A small, 1:100-scale model was first built both in physical sandbox experiments and numerical analyses. The results of the small-scale model obtained from both approaches were then compared. This comparison not only justified the validity of the prediction but also helped validating the numerical model. The small-scale numerical model was subsequently modified, including input parameters, boundary conditions, and others, until its results met well with the experimental results (Step 2 and step 3 in Fig. 1).

The established full-scale numerical model was then adopted to simulate the deformational behaviors of the overburden soil as well as that of a hypothetical shield tunnel located within the soil bed when they were subjected to significant thrust-fault sliding triggered by an earthquake (Fig. 1; Steps 4 to 7). The studied factors include the horizontal distance of the tunnel to the fault tip, fault dip angle, and soil properties, etc. (Fig. 1, Step 6). Meanwhile, a danger factor, which is defined as the reciprocal of the conventionally known “safety factor”, was introduced to indicate the stress state of the soil around the shield tunnel (Fig. 1, Step 6). As well, the axial force–moment diagrams are used to evaluate the safety of the lining.

The load states of the tunnel lining were obtained based on the full-scale numerical model and compared to the axial force moment diagrams to evaluate the safety of the lining.

2.1. Physical model — sand box experiments

The dimensions of the sandbox were 100 cm in length, 60 cm in height, and 20 cm in width (Fig. 2). The bottom

of the sandbox was designed to be movable and capable of simulating the movements of a thrust-fault or normal fault sliding by moving the base plate upward or downward. The mobile hanging wall measured 40 cm, and the fixed footwall measured 60 cm. The maximum upward movement could be as high as 20 cm. Accompanying the vertical movements, horizontal movements could be simultaneously applied to simulate fault movements with dip angles ranging from 0 to 90°. A plexiglas wall was installed on one side of the sandbox to facilitate observation of the deformation. The vertical slip rate of the hanging wall was set to be 4.8×10^{-4} m/s and was controlled by a motor at a constant sliding speed.

During the sandbox experiment, the thickness of the overburden soil was 20 cm, and the maximum uplift of the hanging wall was 6 cm. The dip angle of the simulated thrust fault was mainly 60°. Furthermore, with reference to the hypothetical cross-section configuration of an actual shield tunnel case, a model tunnel of 0.2 cm tunnel thickness was selected with a radius of 3.05 cm. The model tunnel was installed at a depth of 9 cm and with different horizontal distances to the fault tip line at bedrock (e.g. FW2D, a horizontal distance of two times of tunnel diameter from the tip line of the bedrock, FW denotes footwall, HW denotes hanging wall, and D is the diameter of the tunnel).

The overburden stratum was simulated by quartz sand from Vietnam. The particle sizes of the sand ranged from sieve number 40 to 140 and comprised 99% quartz with a very small proportion of feldspar and mica. The sand had a specific weight of 2.65, and maximum and minimum dry density of 1.67 g/cm³ and 1.47 g/cm³, respectively. During the experiments, the relative density was controlled to be 55%, which represents a medium stiff state for all tests. The quartz sand had a frictional angle of 30° obtained from direct shear tests, and a secant shear

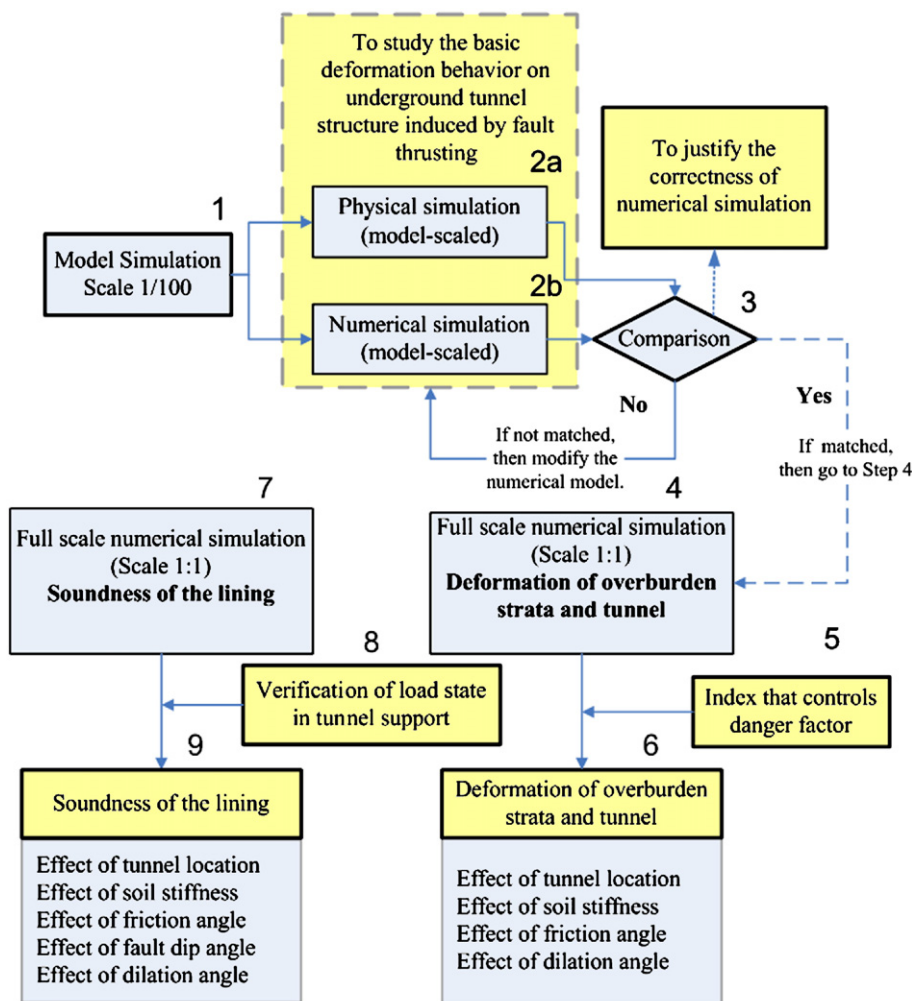


Fig. 1. Flowchart illustrating the research framework of this study.

modulus G of 0.5 MPa selected from simple shear tests at a relative density of 55% (Table 2). The adopted sand under various stress levels had an almost linear failure envelope, with a cohesion intercept close to zero. The dilation angle of sand was measured as 9° . Since the dilation angle in full-scale model will vary from site to site, a parameter study for dilation angle ranging from 6° to 30° was accordingly conducted.

In selecting a material for the model tunnel, the main concern lay in proper scale-down of the stiffness such that the model tunnel would be not too stiff compared to the stiffness of the soil used in the model. Meanwhile, since the research aimed at studying the deformation behavior, the deformation of the tunnel was intentionally exaggerated such that the deformation could be easily detected and observed. Therefore, besides the relative stiffness ratio between the soil and the tunnel, the com-

pressibility factors (C^*) and flexibility factors (F^*) should be considered (Einstein and Schwartz, 1979). With regard to these issues of concern, a carton paper was selected after several tests for the material for the model tunnel. The properties of the carton paper are also summarized in Table 2.

The faulting process during the sandbox experiment was recorded in photographs. Photos recording deformation of the strata were taken from the side of the box (Fig. 2). The photos were then processed to correct any distortion. To facilitate observation of the deformation, thin layers of dyed sand were spread in alternation with un-dyed sand at 2 cm intervals, enabling easy recognition of deformation of the sand. As a result, the visible distortion of dyed sand layers served as a clear indicator of the deformation of the overburden induced by blind fault slip and the development of the influenced zone, etc.

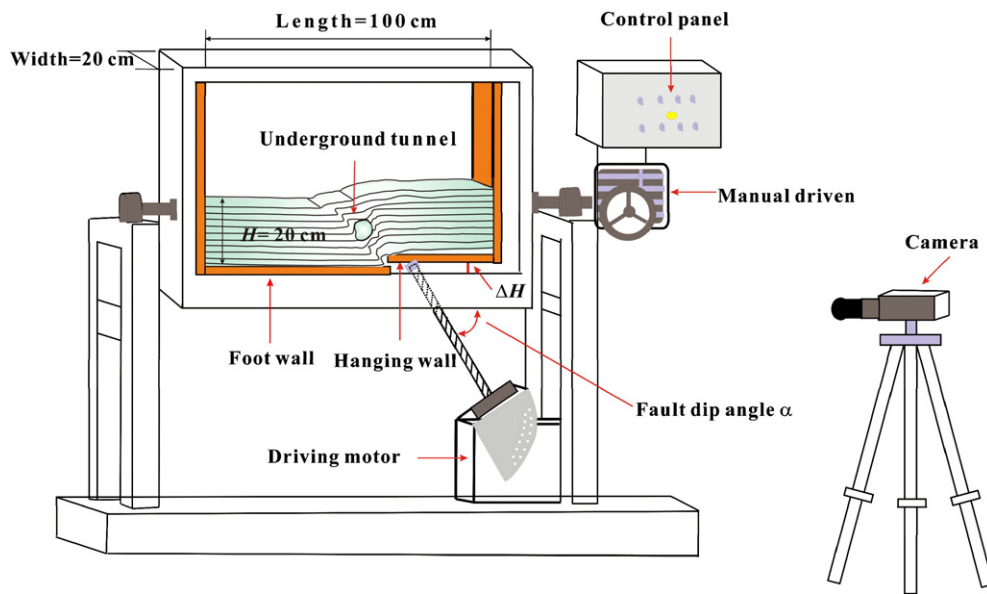


Fig. 2. Schematic setup of the experimental study in a model scale.

2.2. Numerical model

A finite element method based on the software ABAQUS (Hibbitt et al., 2004), was adopted for the numerical analysis. An elasto-plastic behavior was adopted for soil material. The material model, the Mohr–Coulomb model, which is often used for representing the behavior of geo-materials, was accordingly adopted in the numerical analysis.

2.2.1. Small-scale numerical model

The configuration of the numerical model was set to identical dimensions as those used in experiments (Fig. 3). The model box measured 100 cm in length, 20 cm in height and the hanging wall was 40 cm while the footwall was 60 cm across. Results from the sandbox experiments reveal that the influenced extent of the shear zone approximately is 20 cm wide and 60 cm of footwall in length should be sufficiently long without significant boundary effect. The right boundary could be moved to simulate the movement of the hanging wall (Fig. 3). The center of the simulated tunnel is situated at varied horizontal distances from the tip line of the bedrock (Fig. 3).

A roller condition was set for the left and the right boundaries of sand. Friction between sand and the bottom of sandbox was employed (a contact pair for interface simulation is readily available in ABAQUS). Both conditions of slip allowable and no slip were considered by means of interface between tunnel and surrounding soil. The material properties of sand and tunnel supports

shown in Table 2 were used for numerical simulation of the small sandbox model tests.

A fault zone is often produced within the overburden soil, which is a typical result for physical sandbox model tests when the underlying fault slips. For the numerical analysis, the material inside the fault zone, in fact, reached a plastic state. Therefore, the contour of the plastic strain served as a good indicator representing the fault zone within the overburden soil bed. The configuration and location of shear zones obtained by the numerical analysis and by the sandbox test were consistent.

Based on the above, the small-scale numerical model including the input parameters, boundary conditions and others was subsequently modified until its results met well with the experimental results.

2.2.2. Full-scale numerical model

Based on the calibrated small-scale model, which compared well to the model experiments, a full-scale model was then established. With a cross-section and configuration corresponding to an actual shield tunnel, the hypothetical full-scale tunnel would be located proximal to the Taipei Fault in the Taipei Metro System (Chung et al., 2005). A stretch of at least 1000 m of this designed lane of the underground tunnel would run close to the bedrock fault line and sub-parallel to the fault line with varied distances. Thus it is reasonable to consider the deformation induced by thrust faulting could be approximately in a two-dimensional plane strain condition. The deformation of the tunnel lining would be

Table 2
Summary of the material properties adopted in this research

Parameter	Experimental simulation	Full-scale simulation	Unit
Model tunnel	(carton paper)		
E_s	400	31,800	MPa
ν_s	0.3	0.2	
Model soil	(sand)		
γ (dry unit weight of soil)	15.7	20.0	kN/m ³
E	1.29	19	MPa
ν	0.3	0.3	
c	5	5	kPa
ϕ (frictional angle)	30	30	°
ψ (dilation angle)	6	6, 30	°
$k_o = 1 - \sin\phi$ (stress ratio at rest)	0.50	0.50	
Frictional property of interfaces $\mu = \tan(2/3\phi)$			
μ (soil–rock)	0.36	0.36	
μ' (soil–tunnel)	0.36	0.36	

Remark:

The relative stiffness of the ground mass to the tunnel support can be evaluated by two dimensionless parameters, the compressibility ratio, C^* and the flexibility ratio, F^* (Einstein and Schwartz, 1979). The compressibility ratio, C^* , is defined as $C^* = ER(1 - \nu_s^2)/E_s A_s(1 - \nu^2)$; The flexibility ratio, F^* , is defined as $F^* = ER^3(1 - \nu_s^2)/E_s I_s(1 - \nu^2)$, in which E , ν , and E_s , ν_s = the elastic constants for the ground and support; A_s = the average cross-sectional area of the support per unit length of tunnel = $t \times 1$; R = the tunnel radius; t = average support thickness; and I_s = the moment of inertia of the tunnel support per unit length of tunnel. The compressibility ratio is a measure of the relative stiffness of the ground support system under an antisymmetric loading condition, i.e. it reflects the flexural stiffness of the system. The flexibility ratio is a measure of the relative stiffness of the ground support system under a uniform or symmetric loading condition, i.e. it reflects the circumferential stiffness of the system. Increasing the values of C^* and F^* decreases the support thrust and moment and increases the net inward displacement of the tunnel. Since the research aims at studying the deformation behavior, the deformation of the tunnel was intentionally exaggerated such that the deformation could be easily detected and observed. Roughly estimating from Fig. 6 of Einstein and Schwartz (1979), the dimensionless support displacement of model is about twice of full-scale tunnel.

Model tunnel (Paper): $E/E_s = 0.003$; $t/R = 0.068$; $C^* = 0.054$; $F^* = 141.9$.

Full-scale tunnel: $E/E_s = 0.0006$; $t/R = 0.086$; $C^* = 0.008$; $F^* = 11.6$.

mainly cross-sectional deformation. The dimensions (L , H , T and D) of the full-scale numerical model were enlarged to 120 m, 20 m, 11 m and 6.1 m, respectively (Fig. 3). The hanging wall length was 40 m while the footwall was 80 m. The thickness of the tunnel lining was 0.25 m. Parameters used in the full-scale model are obtained from site investigation (Table 2). The center of the simulated tunnel is located within the shear zone of the fault at $FW7D$, $FW5D$, $FW3D$, $FW1D$, $HW1D$, and $HW3D$ (Fig. 3).

The segmental linings were modeled through elastic beam elements in ABAQUS. Before numerical simulation of a full-scale hypothetical tunnel model, ABAQUS had first validated as a suitable FEM model tool for tunnel lining. An elastically lined cylindrical tunnel under plane strain conditions in a linearly elastic ground mass was analyzed through ABAQUS. Axial loads, moments, and shear forces within the tunnel lining were obtained. These results were then compared with the analytical solutions derived by Burns and Richard (1964), Dar and Bates (1974) and Einstein and Schwartz (1979). Both full-slip and no-slip conditions at the ground-support interface were considered. The results validated that the segmental linings of tunnel rings could be modeled very well through FEM model under external loading conditions.

Subsequently, a numerical simulation of a full-scale tunnel model was built. The in-situ geostatic stresses were modeled firstly, the coefficient for lateral soil pressure was taken to be $K_0 = 1 - \sin\phi$. When the equilibrium of in-situ stresses was reached, the soil in the tunnel was then removed and segmental tunnel linings were installed. The segmental tunnel lining are under the loading from the self weight of the segments and the soil formation. This loading condition (the external loading condition) also implied that the tunnel opening had been excavated and supported before the loading corresponding to the free-field stresses was applied. Only the external loading condition was performed in this study without considering actual excavation unloading conditions that occurred during tunneling (Einstein and Schwartz, 1979). The assumption of external loading instead of excavation unloading may lead to support forces that are 50%–100% overestimated (Einstein and Schwartz, 1979).

The hypothetical shield tunnel comprised six circular segments; each of them had a width of 1.0 m, an inner radius of 2.80 m, an outer-radius of 3.05 m and a thickness of 0.25 m (Fig. 4). The segments were coded as *Types A, B* and *K* in accordance with the arc-length of the segments (Fig. 4b). Joints between rings were circumferential joints. Joints between segments in longitudinal direction were radial joints. A complete circular ring was formed by one *Type K*, two *Type B* and three *Type A* segments. Meanwhile, the *K*-segment was staggered from ring to ring to avoid a continuous distribution of the joints within longitudinal direction (Fig. 4b). The moment transferred at the joints would be smaller than the moment of the adjacent segment and the numerical model should properly reflect this phenomenon. According to the assembly of concrete lining in engineering practice, segments of beam elements were selected to

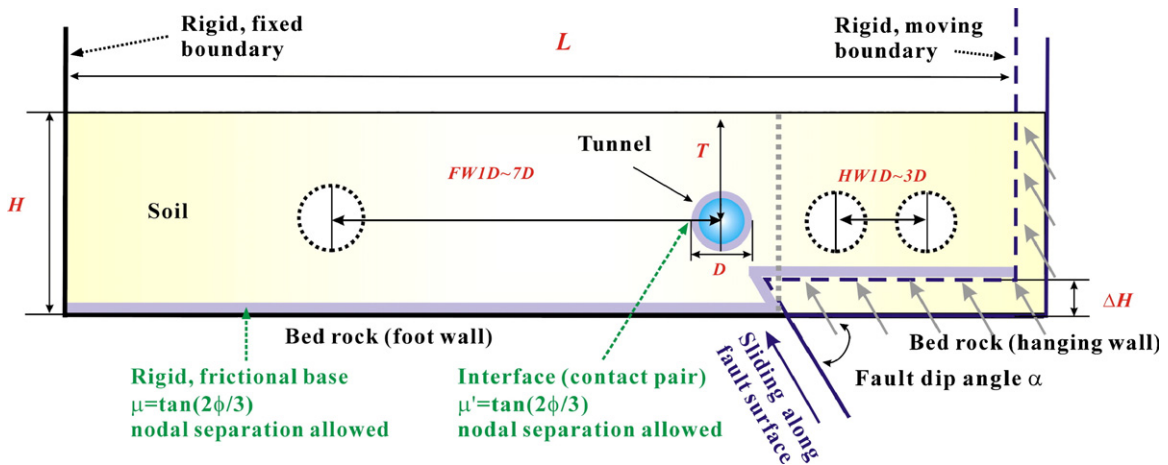


Fig. 3. Illustration of the configuration of the tunnel, thrust fault, and boundary conditions adopted for the numerical study. FW denotes footwall, HW denotes hanging wall, and D is the diameter of the tunnel, e.g. $FW2D$, a horizontal distance of two times of tunnel diameter from the tip line of the bedrock.

comprise a complete ring and one way to deal with the existence of joints was to consider the tunnel lining as a continuous ring with a discounted rigidity of the tunnel lining (Muir Wood, 1975; Lee et al., 2001b; ITA, 2000). In this study, the discounted rigidity is modeled by a reduction by half of the cross-sectional area of the beam elements located at the twelve radial joints of every two rings (Fig. 4c).

When the equilibrium between in-situ stresses and the excavation of tunnel was reached, the up-thrusting move-

ment of the bedrock by the fault was simulated through lifting 2.5 m in accordance with the fault dip angle. The corresponding axial loads and moments within segmental linings were obtained. The relationship between the design axial load capacity and the design flexural capacity of the segments is described as P – M curves. Therefore, the safety of the segmental lining for combined axial load and flexure moment were examined by confirming whether the load points of the lining were located inside the P – M curves, that is, at the side of origin.

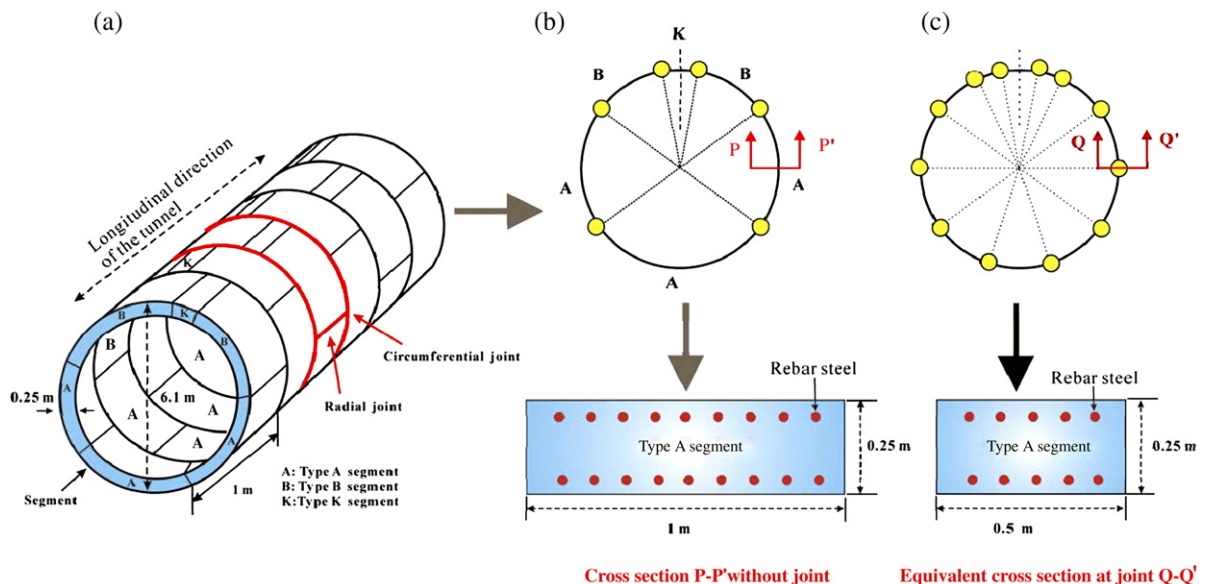


Fig. 4. (a) Typical setup of the lining inside a shield tunnel. A complete circular ring was formed by one Type K, two Type B and three Type A segments. The K-segment was staggered from ring to ring to avoid a continuous distribution of the joints within a longitudinal direction. (b) Cross-section of Type A segment with six joints and P – P' section without joint. (c) Cross-section Type A segment with twelve joints and Q – Q' section with joint rigidity reduction.

2.3. Danger factor

The typical output of analyses based on a finite element method includes displacements, strains, and stresses. To indicate how far the material was from a failure state, an additional measure, a danger factor, was also adopted in this research. The danger factor is defined as (Jeng and Huang, 1998):

$$D_f = \frac{f(\sigma_{ij})}{S(c, \phi, \sigma_{oct})}. \quad (1)$$

Where $S(c, \phi, \sigma_{oct})$ is the shear strength of the material and $f(\sigma_{ij})$ is the applied shear stress.

The Mohr–Coulomb strength criterion, which is conventionally used in geotechnical engineering, was also adopted in this research. Based on the Mohr–Coulomb criterion, the danger factor can be expressed in terms of c, ϕ as:

$$D_f = \frac{\sqrt{3}J_2 \times R_{mc}(\theta, \phi)}{c + I_1 \times \tan \phi} \quad (2)$$

$$R_{mc}(\theta, \phi) = \frac{1}{\sqrt{3}\cos\phi} \sin\left(\theta + \frac{\pi}{3}\right) + \frac{1}{3} \cos\left(\theta + \frac{\pi}{3}\right) \tan \phi \quad (3)$$

$$\theta = \frac{1}{3} \cos^{-1} \left(\frac{J_3}{\sqrt{3}J_2} \right)^3 \quad (4)$$

Where $I_1 = (\sigma_{11} + \sigma_{22} + \sigma_{33})/3$ is the first stress invariant; J_2 is the second deviatoric stress invariant; J_3 is the third deviatoric stress invariant.

The danger factor is the reciprocal of the conventional factor of safety. This factor is similar to the stress level (SL, Bray et al., 1994b). When D_f is less than one, the material has not reached a failure state, and on the other hand, when D_f equals to or is greater than one, it indicates that the material has failed.

3. Deformation of overburden soil and tunnel — small-scale model

3.1. Sandbox experiment

When the underlying bedrock thrusts upward during faulting activity, the overburden bed on the hanging wall is up-lifted, and a severely distorted zone (fault rupture surface) inside a triangular shear zone is formed between the footwall and the hanging wall (Fig. 5a). This severely distorted zone is often referred to as a “fault

zone” or a “failure zone”, and is considered to be due to the migration of the thrust fault within the bedrock to the ground surface. A detailed sketch of the distortion is also shown in Fig. 5. If the slip is large enough, subsidiary faults may also form within the overburden soils. As a fault moves, the fault zone in the overlying overburden bed extends to area W/H on the ground surface (W denotes the horizontal distance between where the fault moves and the point where the fault emerges on the ground; H is the thickness of the overlying overburden bed, Fig. 5). In our experiment, we used 60° as the fault dip angle and two fault zones were developed. The order of fault zones development was from the hanging wall towards the footwall, and affected area W/H on the ground surface was approximately 0.99 (Fig. 5a). The detailed information for the deformation of overburden soil induced by thrust-fault slip was described in Lin et al. (2006a).

Results of the (FW2D) experiment in which the fault dip angle was also 60° (Fig. 6). Three fault zones developed. The order of development of these fault zones is as follows: the first fault zone developed from the tip of the fault to the ground surface, then the second fault zone began to develop downward towards the footwall, however, this was hampered and bent by the underground tunnel, and the fault tip of this strand of fault did not extend to the ground surface. The third fault zone developed within the area between the first and the second fault zones. The affected area on the ground surface W/H was approximately 0.75 (Fig. 5b). There was no perceptible deformation or shifting of the tunnel due to ground uplifting. When compared with a site where there was no underground tunnel, it was clearly observed that the affected area on the ground surface W/H was 24% less, but that the soil inside the triangle zone was distorted more severely (Fig. 5a, b).

The results from experiment FW1D where the tunnel was much closer to the fault zone are shown in Fig. 5c. As the ground uplift increased during faulting, fault zones developed in an upward direction. The first fault zone was hampered and bent by the underground tunnel, then it passed directly underneath the tunnel. When development of the first fault zone reached the surface of the ground, the second fault zone started to develop upward toward the hanging wall and then passed above the tunnel. Ground surface measurement indicated that the affected area on the ground surface W/H was approximately 0.98 (Fig. 5c). The tunnel was uplifted, shifted, rotated, and severely distorted. Compared with the case without underground tunnel, the affected area on the ground surface W/H was

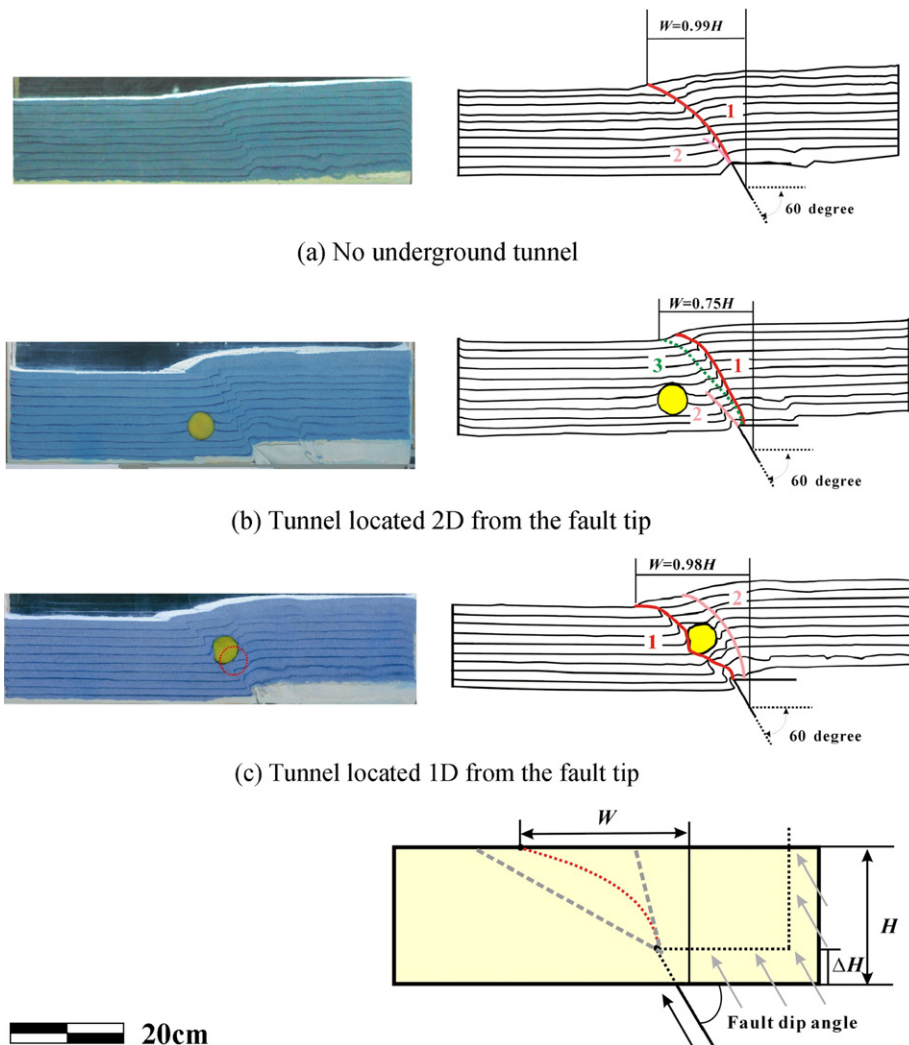


Fig. 5. The deformation and the development of fault zones of the overlying overburden soil, obtained from sandbox experiments. The distance (denoted as W) is accordingly defined as the horizontal distance between the surface trace and the bedrock fault tip at the start of deformation, as indicated in the insert. The triangular shear zone above the fault tip is also indicated in the insert. The number neighbors on fault rupture strands denoting the development of the faults in sequence. D is the diameter of the tunnel.

similar. However, two distinct scarps were present on the ground.

The results of tunnel deformation observation in sequence are presented in Fig. 6. The results showed that when the tunnel was located at $FW2D$, there was no perceptible shifting or deformation of tunnel after ground uplifting. Where the tunnel was located at $FW1D$, the tunnel was distinctly shifted with a 3 cm vertical displacement (see also Fig. 5c, the initial position of the tunnel is in red, the position after uplift is in yellow). The triangular shear zone is enlarged in case that the tunnel is located inside the triangular shear zone.

3.2. Comparison between sandbox experiment and numerical analysis

The results from numerical analysis were then compared with the results from the sandbox experiments (Fig. 7). Fig. 7a compared the zones of plastic shear strain concentration with the fault zones from the sandbox experiments without underground tunnel. The locations of the plastic shear strain concentration zones correspond to the locations of the fault zones in sandbox experiments with similar ground surface affected areas.

When the tunnel was located at $FW2D$, the development of concentrated plastic strain zones was limited by

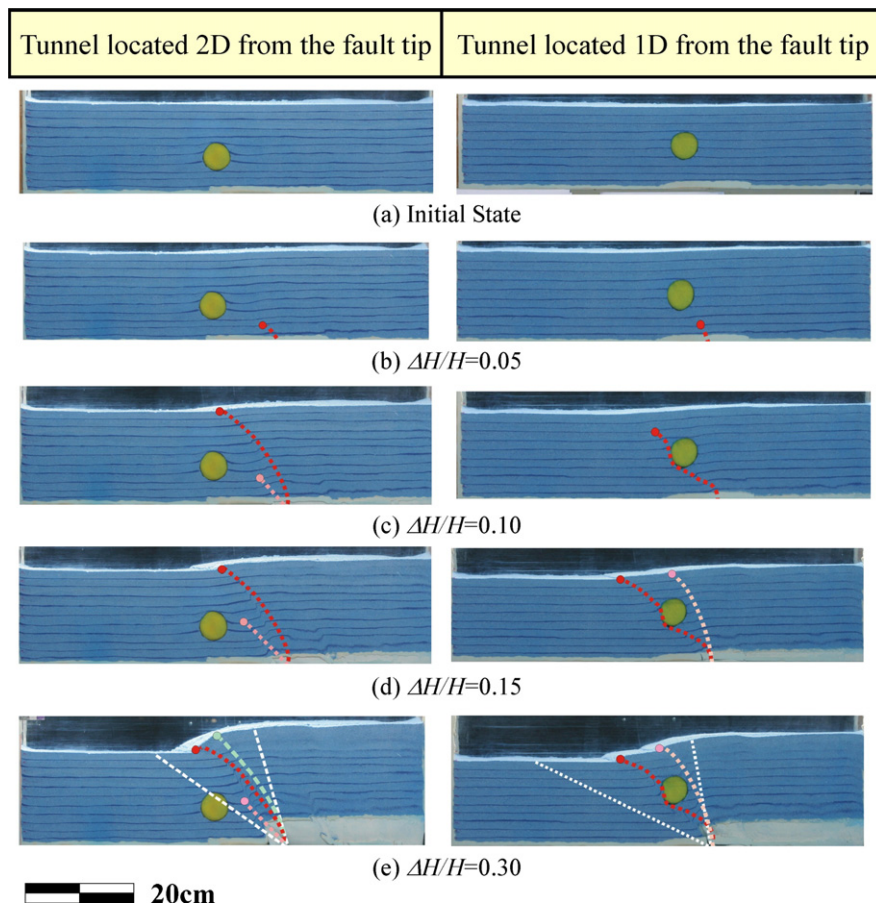


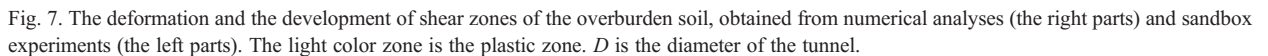
Fig. 6. Fault development subjected to varied uplifting in sequence. The right part is a tunnel located 1D from the fault tip; the left part is a tunnel located 2D from the fault tip. D is the diameter of the tunnel. Dotted lines indicate the fault slip surfaces, and white dotted lines indicate the extent of triangular shear zones involved in all stages.

the underground tunnel, and the concentrated plastic strain zones occurred in the footwall at the area between the fault and the tunnel (Fig. 7b). The affected ground surface area was similar to those resulting from sandbox experiments. The distortion of the mesh was severe, indicating that distortion of the soil within the triangle zone was more severe than that of the soil without the tunnel. There was no conspicuous deformation or shifting of the tunnel due to ground uplifting.

In the case where the tunnel was located at $FWID$, the first concentrated plastic strain zone developed in an upward direction while the ground was uplifted. When the first concentrated plastic strain zone encountered the tunnel, it was hampered and bent by the underground tunnel, then it passed directly underneath the tunnel. The second concentrated plastic strain zone developed upward toward the hanging wall; as development of the first concentrated plastic strain zone reached the ground surface, and it passed through the upper-right side of the

tunnel (Fig. 7c). Therefore, there exhibit two fault scarps on the ground. During ground uplift, the tunnel was distinctly shifted with a 3 cm vertical shift. In addition, the tunnel was deformed and rotated due to squeezing during uplifting.

Comparison of the deformations of the tunnel obtained both from sandbox experiments and numerical analyses of the $FWID$ case revealed that both of the deformed tunnels had similar elliptical shapes (Fig. 8, r_0 and r are the radius of the tunnel before and after thrusting, respectively). Actually, the deformations of the tunnel in numerical analyses were smaller than those of the sandbox experiments. It might be a result of the elastic assumption of the tunnel lining adopted in numerical analyses. For easy comparison of the circumferential deformation, the elastic modulus of the carton paper had been reduced to one-tenth of the original value in order to amplify the deformation of the numerical analysis. The apex of the tunnel crown was



The detail of deformation patterns around the tunnel both from sandbox experiments and numerical analyses are further compared (Fig. 9). The location of the displaced tunnel and the deformed configuration of the

Remarkably, if the material around the tunnel wall was separated allowably from the wall, the numerical analysis indicated that cavities in the invert and the crown could be yielded (Fig. 9a). This phenomenon highlights that the occurrences of cavities should also be of concern in the practical design.

In summary, when the tunnel was situated inside the triangular shear zone (deducing from free-field conditions, i.e. without underground tunnels) the

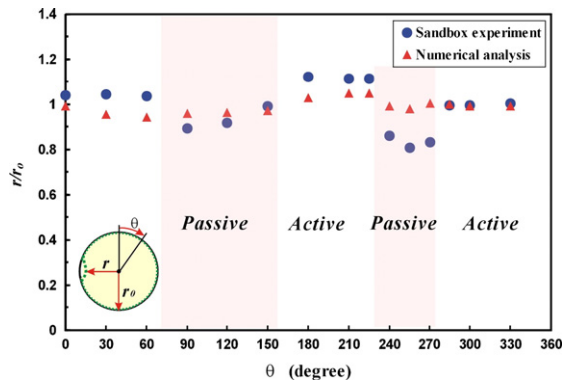


Fig. 8. Comparison of the tunnel deformations obtained by the experimental study and the numerical analyses (case *FWID*). D is the diameter of the tunnel. The apex of the tunnel crown was taken to be 0° , and following clockwise rotations as indicated in the inset. r_0 and r are the radius of tunnel before and after fault thrusting, respectively.

overburden soil might develop greater fault zones or even bifurcate during propagation of the bedrock thrust fault, rendering greater deformation and earth pressure on the tunnel. When the tunnel was situated outside the triangular shear zone under free-field, it might be safe.

The study by Lin et al. (2006a) on overburden deformation behavior due to fault movement indicated that major parameters affecting the fault zone included fault dip angle, Young's module, and angle of dilation. Fur-

thermore, increasing Young's modulus of the geologic formation might possibly lead to development of back-thrust. In this regard, in conducting whole-scale numerical analysis in this study, emphasis was placed on several soil parameters. The main purpose lay in unravelling the distributions of danger factors within overburden soils and the distributions of loading states on P – M diagrams of the tunnel linings through varied properties of soil materials.

4. Deformation of overburden strata and tunnel — full-scale numerical model

4.1. Deformation process and failure modes

Fig. 10 illustrates the predicted deformation of an *FWID* case in sequence when the basement is thrust upward in an inclined angle of 60° . The process of deformation is shown from Fig. 10a to d. As in the small-scale model experiments, a bifurcated shear zone occurred, extended to the ground surface, and was followed by an increase in width (Fig. 10, left parts). The right part of Fig. 10 shows the distributions of danger factors. It indicates that a much wider zone, around the shear zone, and near the ground surface, was close to a failure state with D_f exceeding 0.7. Detailed examination of the stress paths of the soil revealed that the soil around the ground surface failed due to tensile stress,

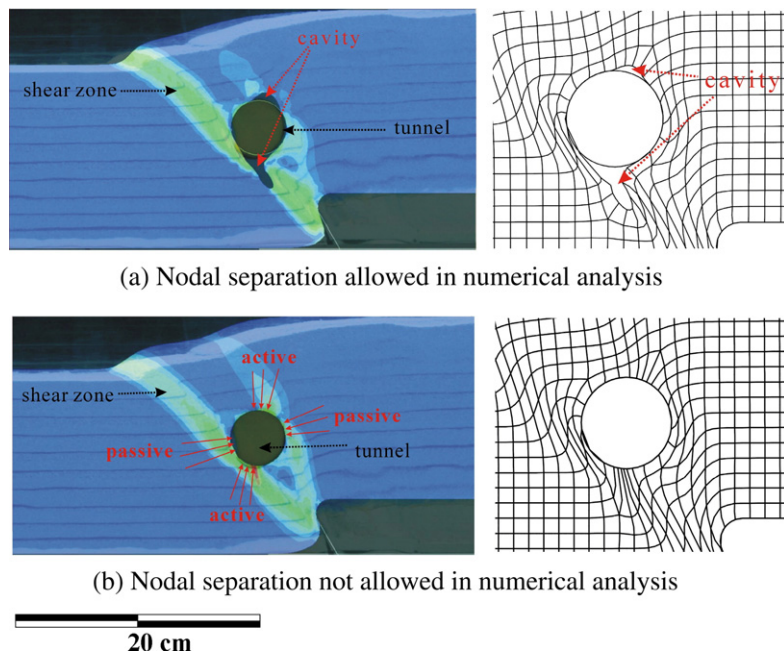


Fig. 9. Comparison of the deformations obtained by the experimental study and the numerical analyses (case *FWID*). D is the diameter of the tunnel.

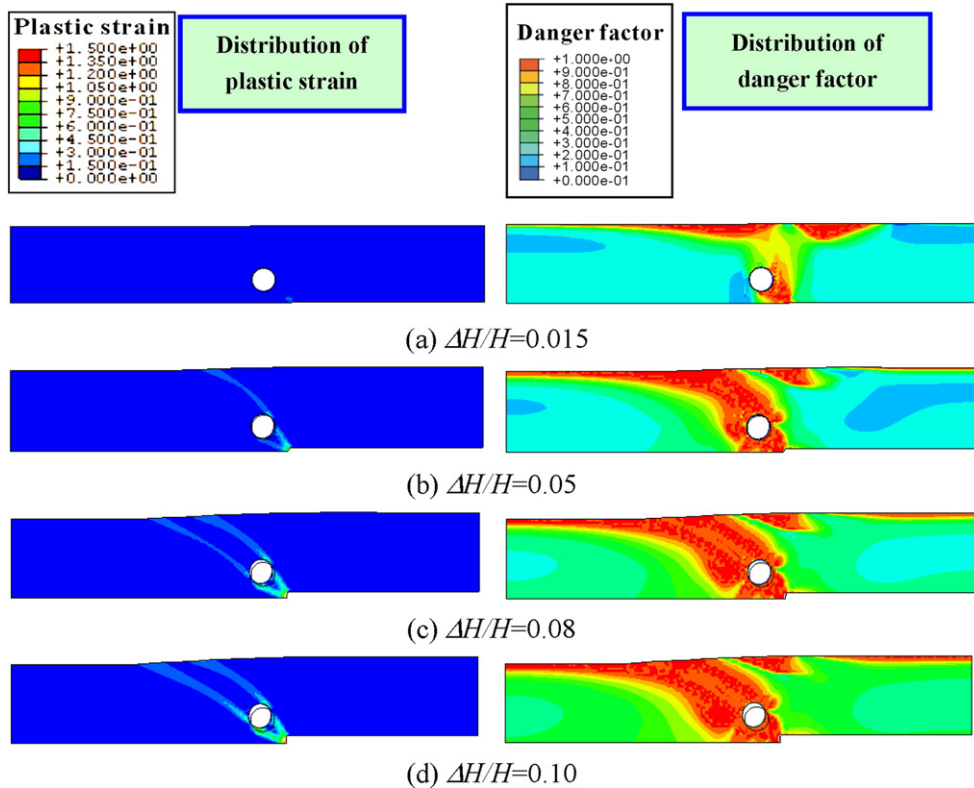


Fig. 10. Development of shear zone upon consecutive bedrock slipping (case *FWID*). D is the diameter of the tunnel. The left parts are distributions of plastic strain; the right parts are distributions of danger factors.

and the failure of soil within the shear zone was due to shearing. This result coincides with the observation that tensile cracks often appear on the ground surface, especially near the thrust-fault trace, after coseismic deformation (Lee et al., 2001a, 2004; Chen et al., 2004).

4.2. Effect of tunnel location relative to thrust faulting

The location of a tunnel will affect the displacement and the deformation of the ground as well as the development of a shear zone within the overburden soil stratum. Fig. 11 illustrates the influence of tunnel location on the above-mentioned behavior. When the tunnel is located in the range of the footwall and not very close to the potential shear zone (Fig. 11a, b, and c), the displacement and deformation of the tunnel is relatively insignificant. Meanwhile, the development of the shear zone is not or is only slightly affected by the existence and the location of the tunnel.

When the tunnel is located in the range of the hanging wall and not very close to the potential shear zone (Fig. 11e and f), the tunnel is also insignificantly deformed and the development of the shear zone is only slightly

affected. However, the tunnel is substantially displaced according to the slide direction of the underlying bedrock. For the two-dimensional analysis (Fig. 11e and f), the stress applied to the lining might not cause it to fail; however, in a three-dimensional sense, it can be foreseen that this large displacement may cause longitudinal deformation and cause the tunnel to move and to fail along the longitudinal direction (Kennedy et al., 1977; Wang and Yeh, 1985).

When the tunnel is located near the boundaries between the hanging wall and the footwall and thus inside the shear zone (Fig. 11d), the one potential major fault zone interacts with the existence of the tunnel and causes development of two fault zones, as described in Figs. 5, 6 and 7.

Remarkably, for the case shown in Fig. 11c, the deformation and displacement of the tunnel is insignificantly visible, but the soil around the tunnel is close to failure state (Fig. 11c, right part), and the stress applied to the lining may cause it to fail. Therefore, for such cases, the induced axial force and moment in the lining should be compared to the capacity of the lining to check the soundness of the lining.

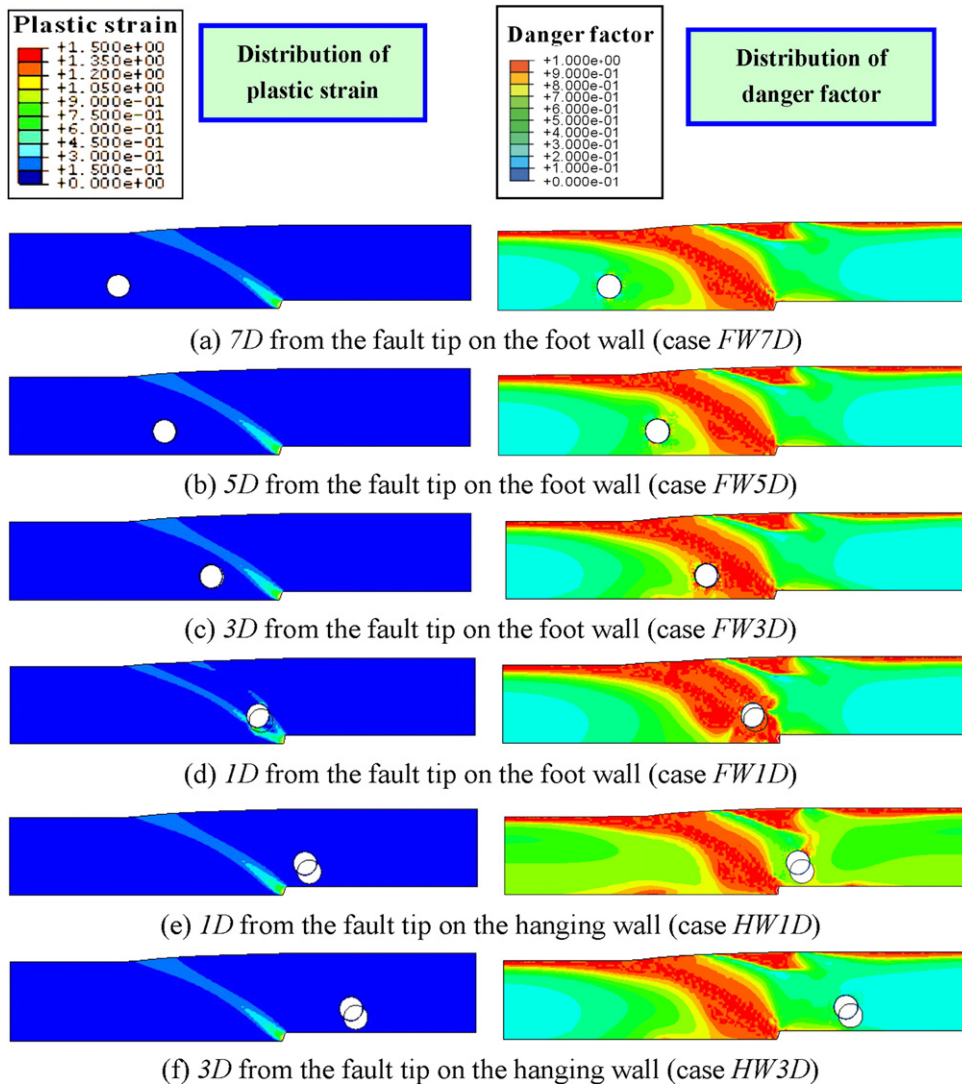


Fig. 11. Configurations of shear zones corresponding to various tunnel locations, medium stiff soil. The left parts are distributions of plastic strain; the right parts are distributions of danger factors. D is the diameter of the tunnel.

4.3. Effect of soil stiffness

When Young's modulus of soil increased from 19 MPa (representing the typical medium stiff soil; the cases shown in Fig. 11) to 190 MPa (representing the typical stiff soil), and the resulted deformation is shown in Fig. 12. The increase of soil stiffness, when comparing Fig. 12 with Fig. 11, does not seem to affect the displacement and the deformation of the tunnel. For a stiffer soil, the development of the fault zone tends to be wider and toward the footwall, as compared in Figs. 11 and 12. Furthermore, as indicated by the right figures (the distributions of danger factors), the zone in which the soil is close to failure

(e.g., D_f exceeding 0.7) is significantly enlarged due to the increase of soil stiffness. Importantly, the soil near the tunnel is closer to a failure state, as depicted in Fig. 12 and compared to Fig. 11. As such, the loads and the soundness of the linings require further examination.

4.4. Development of back-thrust in stiff soil

It has been found that an increase in soil stiffness enables the development of back-thrust faulting (Lin et al., 2006a). Although the left figures in Fig. 12 do not show the development of back-thrust faulting, the right figures do indicate the halfway development of back-thrust, when the soil is quite close to a failure state.

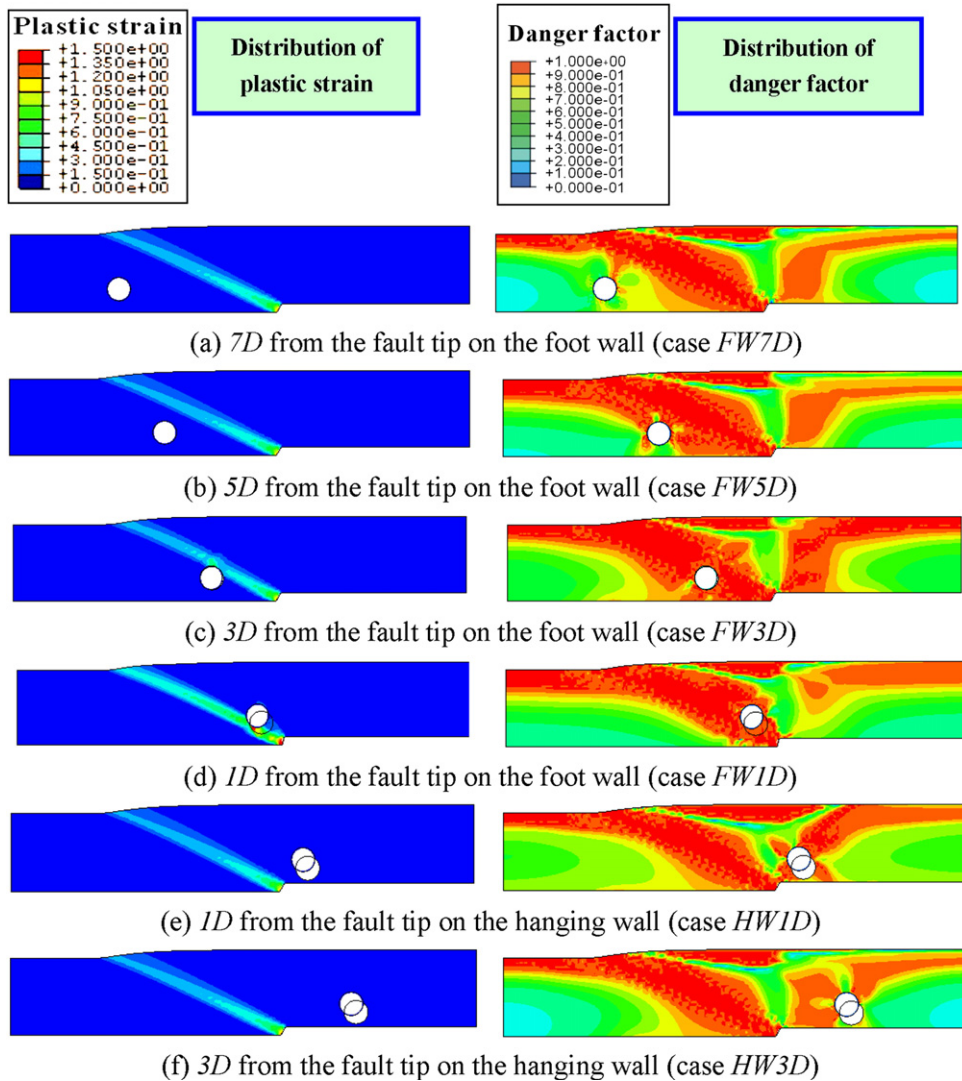


Fig. 12. Configurations of shear zones corresponding to various tunnel locations, stiff soil. The left parts are distributions of plastic strain; the right parts are distributions of danger factors. D is the diameter of the tunnel.

The location of the tunnel seems to affect the development of back-thrust. When the tunnel is located within the shear zone (Fig. 12c), major deformation occurs in the shear zone, which seems to ease the occurrence of back-thrust, when comparing Fig. 12c with Fig. 12a and b. At the other extreme, if the tunnel is located in the potential location of the back-thrust (Fig. 12e), the development of the back-thrust becomes more imminent, when compared to other conditions (Fig. 12).

4.5. Effect of dilation angle

Although the strength parameter or the deformation modulus of sand could be measured, another parameter,

the dilation angle of plastic flow rule ψ , was difficult to constrain by simple experiments and thus required numerical analysis to evaluate. With the other properties of the sand being the same, the dilation angle ψ for the numerical analysis was accordingly determined as 6° , by adjusting the numerical results to have the same W with the free-field experimental results without a tunnel (Lin et al., 2006a). Since the dilation angle in a full-scale model will vary from site to site depending on soil type and in-situ state, a parametric study for dilation angle ranging from 30° to 6° was accordingly conducted. The dilation angle of the soil was decreased from 30° to 6° . When comparing Fig. 13 with Fig. 11, a smaller ψ leads to a smaller W , a narrower fault zone, a smaller degree of plastic deformation and a lower surface slope of fault

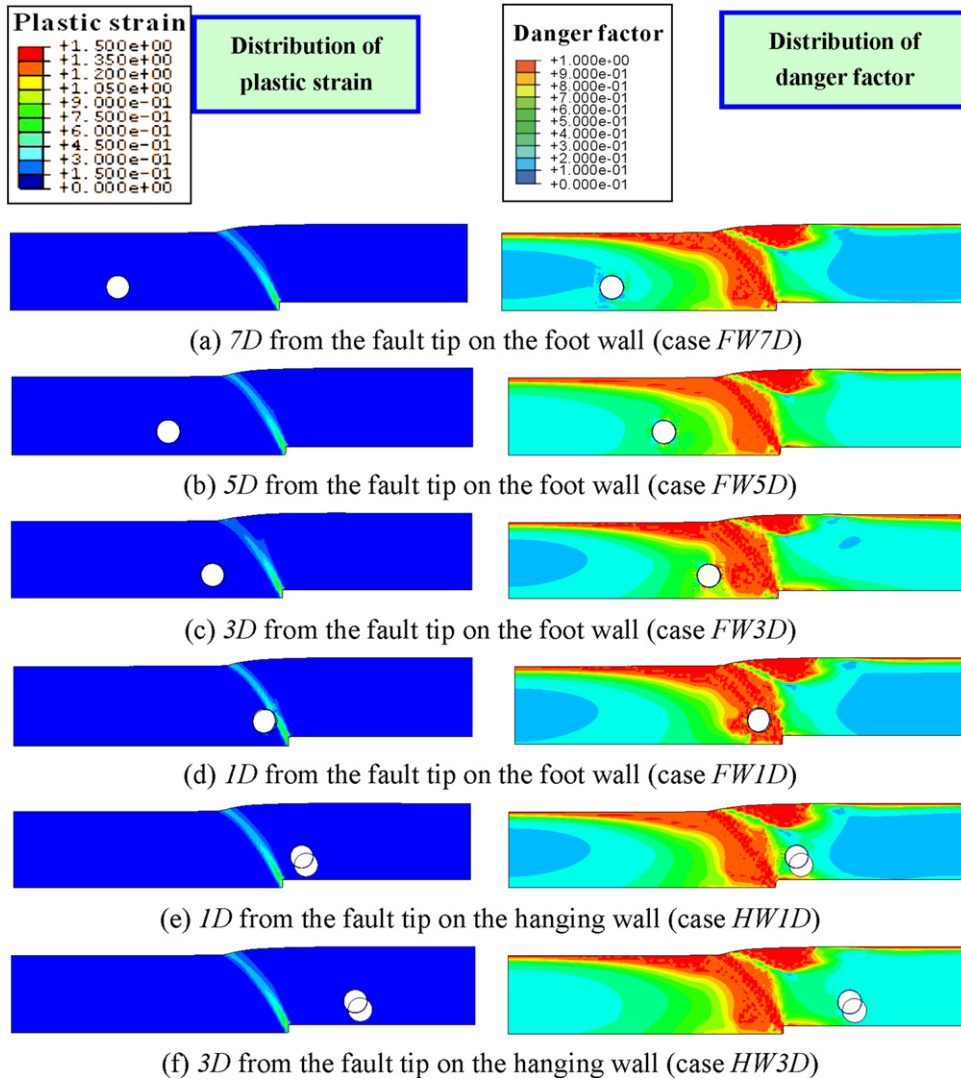


Fig. 13. Configurations of shear zones corresponding to various tunnel locations. The dilation angle $\psi=6^\circ$. The left parts are distributions of plastic strain; the right parts are distributions of danger factors. D is the diameter of the tunnel.

scarp. An associated flow rule, in which a dilation angle equals a friction angle, was conservatively selected for the base case, since it would lead to a larger shear zone and a higher load on the tunnel lining.

5. Soundness of the lining — full-scale numerical model

Based on the typical configuration of lining used in engineering practice, the corresponding P – M curves (axial load capacity versus moment capacity) for linings with or without joints were calculated and shown in Fig. 14. Linings, which are assembled by segments and hence have joints, tend to have less resistance to moment

and axial force due to the existence of joints. The red color curves represented the designed P – M diagram of beam elements with rigidity reduction. The blue color curves represented the designed P – M diagram of beam elements without rigidity reduction. The axial load and moment at various points located within the linings could be obtained from numerical analysis and were also plotted in the figure (Fig. 14) containing P – M curves so that the load state of every beam constituting the whole ring could be compared to the capacity. Hence, it could be identified whether and where the lining would fail or not. When the stress was located to the right of the P – M curve, the load exceeded the capacity and hence the lining failed (Fig. 14). In reality, the lining material

would yield plastically after reaching the limit state of P – M curve. Therefore, the results of the analysis serve as an indication for the potentiality of failure instead of the actual configuration of failure (Fig. 14).

For a soil with medium stiffness ($E=19$ MPa), the lining may fail for cases $FW1D$, $FW3D$, and $FW5D$ (Fig. 14a). Also, for case $HW1D$, the lining with joints may fail. In general, it can be seen that greater load will be induced: (1) when the tunnel is closer to the fault tip; and (2) when the tunnel is located inside the triangular shear zone and at downward near hanging wall side rather than at upward hanging wall side.

In the case of $HW3D$, the tunnel was almost unaffected by thrust faulting, judging from the distributions of danger factors (right part, Fig. 14f). Therefore, the load states of case $HW3D$ can serve as indicators of the initial load states on segmental lining before thrusting. It can be seen that the maximum moment (~ 1800 kN m) induced by thrust faulting in case $FW1D$ is about nine times larger than the original value (~ 200 kN m) and the maximum axial load (~ 1800 kN) induced by thrust faulting in case $FW1D$ is only about two times larger than the original value (~ 900 kN). This result conforms that the loading imposed on a lining induced by the

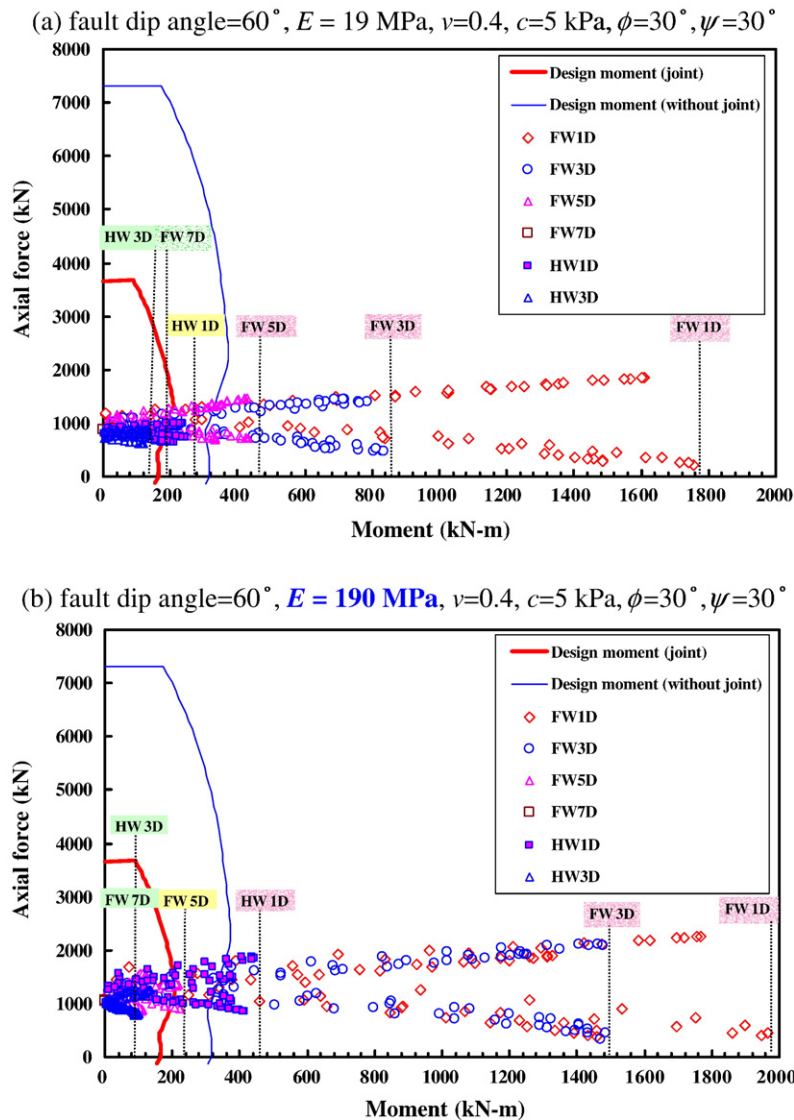


Fig. 14. The P – M curves of the lining and the load states of the lining. A complete circular ring was formed by seventy-four elements. The curves in red color represent load capacity of beam elements with rigidity reduction. The curves in blue color represented load capacity of beam elements without rigidity reduction. Square boxes denoted with cases name plus dotted lines indicate the maximum moments for varied cases, respectively.

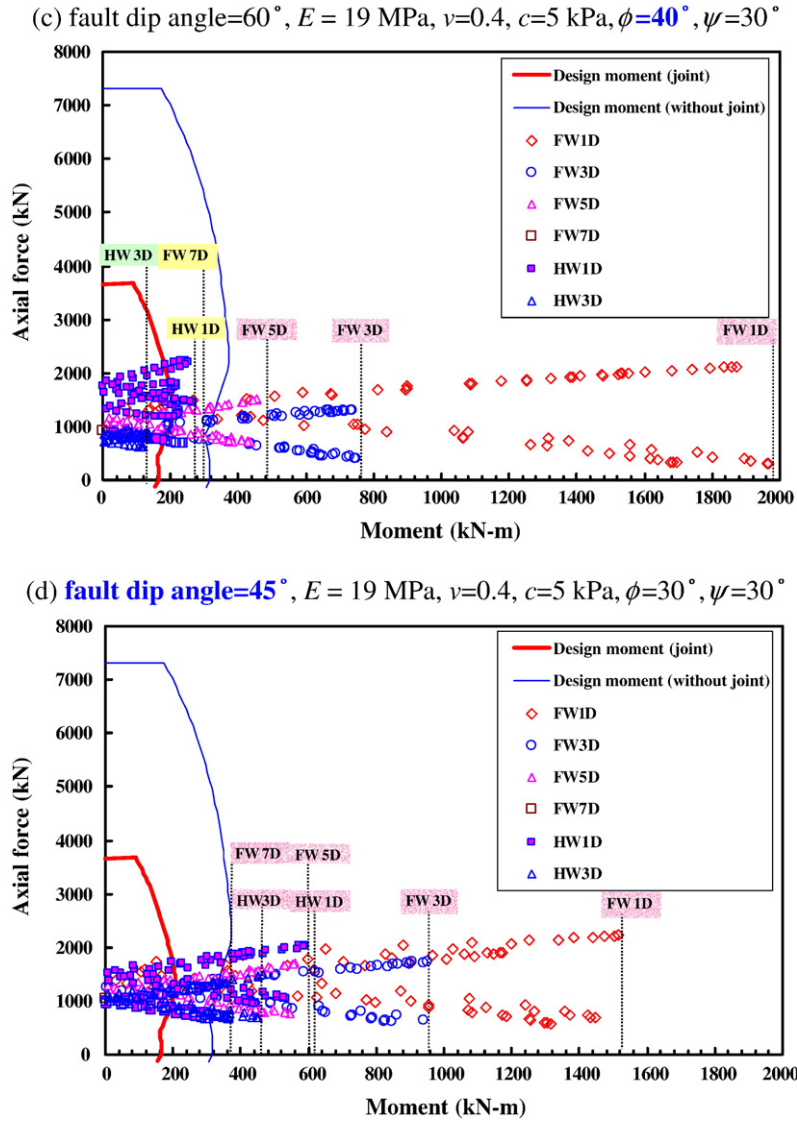


Fig. 14 (continued).

deformed soil around the tunnel is roughly an antisymmetric loading condition (Fig. 8).

For the purpose of understanding the influence of factors, including Young's modulus E , Poisson ratio ν , cohesive strength c , frictional angle of soil ϕ , fault dip angle α , ..., etc., a systematic parametric study was conducted using the inputs summarized in Table 3.

5.1. Effect of soil stiffness

The cases where the soil has medium or higher stiffness are compared in Fig. 14a and b. Apparently, much higher load was imposed on the lining by the stiffer soil. Overall, for a given amount of thrusting, an increase in

soil stiffness is indeed undesirable from a safety point of view since it not only induces large failure in the soil (including additional back-thrusting) but also in the lining.

5.2. Effect of frictional angle

The cases of soil with lower or higher frictional angles (30° and 40°) are compared in Fig. 14a and c. Soil with a greater frictional angle inherently has greater shear strength and hence a "thinner" shear zone will form when this soil is subjected to thrusting. However, comparing Fig. 14c to a, it indicated that the greater frictional angle seemed to result in a slightly greater load in the lining for

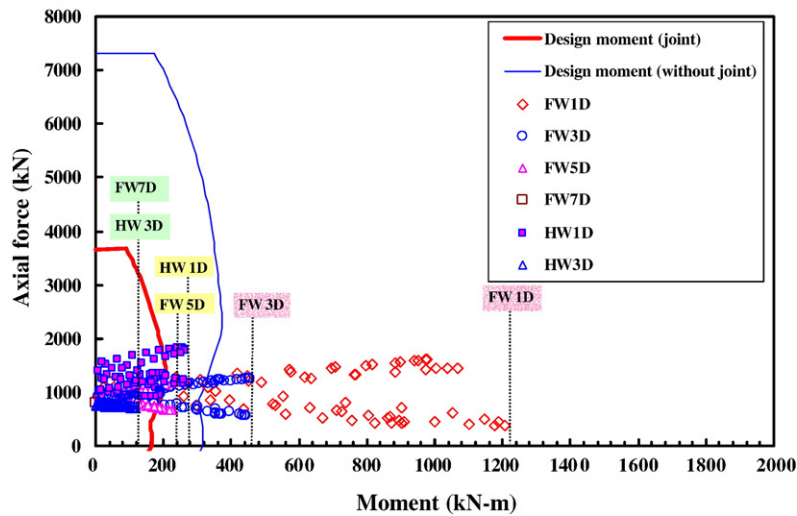
(e) fault dip angle = 60° , $E = 19 \text{ MPa}$, $\nu = 0.4$, $c = 5 \text{ kPa}$, $\phi = 30^\circ$, $\psi = 6^\circ$ 

Fig. 14 (continued).

tunnels located in the footwall and near the fault (cases *FW1D* and *FW3D*) as well as much greater moments for tunnels located in the hanging wall and very close to the fault tip, especially for case *HW1D*. A possible interpretation of such phenomena is that the smaller shear zone represents a larger degree of “strain accommodation” in and around the shear zone, and leads to a greater load on the lining in the near-fault region.

5.3. Effect of fault dip angle

The cases of faults with higher or lower dip angles (60° and 45°) on the loading state are compared in Fig. 14a and d. The fault dip angle was converted into the dip angle of the induced shear zone as well, so that the relative distance of the tunnel to the shear zone

would be altered in the meantime. Comparing Fig. 14d to a, a flatter dip angle (45° compared to 60°) resulted in a closer distance of the tunnel to the shear zone located within the footwall so that the loads for cases *FW1D* and *FW3D* might increase somewhat.

Remarkably, the load state for tunnels located on the hanging wall far away from the fault (cases *HW3D* and *FW7D*) also increased. A detailed check found that a flatter fault dip angle situation would lead to development of back-thrust, so that the loads for tunnels located on the hanging wall would also be influenced. In case of faults with a smaller fault dip angle, tunnels should be relocated farther away from the fault tip line.

5.4. Effect of dilation angle

The cases of soil with lower or higher dilation angles (6° and 30°) on the loading state are compared in Fig. 14e and a. The soil with a smaller dilation angle (of 6° compared to 30°) resulted in a farther distance of the tunnel to the shear zone in the footwall so that the load states for cases *FW1D* and *FW5D* may decrease somewhat.

The influences of other factors, such as Poisson ratio ν and cohesive strength c , have also been studied and were found to be insignificant.

6. Conclusion and discussion

For underground tunnels sited in various locations, the deformations of overburden soil and tunnel lining by thrust faulting were studied. It was found that the

Table 3
Summary of the material properties for full-scale numerical analysis

Purpose of analyses	Control parameter					
	α ($^\circ$)	E (MPa)	ν	c (kPa)	ϕ ($^\circ$)	ψ ($^\circ$)
Influence of dip angle	45, 60	19	0.4	5	30	30
Influence of Young's modulus	60	19, 190	0.4	5	30	30
Influence of Poisson ratio	60	19	0.3, 0.4	5	30	30
Influence of cohesion	60	19	0.4	5, 50	30	30
Influence of friction angle	60	19	0.4	5	30, 40	30
Influence of dilation angle	60	19	0.4	5	30	6, 30

development of a shear zone in the soil is influenced by the existence and the location of an underground tunnel. A major or bifurcated fault zone may develop and extend along the slide direction of the thrust fault, and will induce the failure of the lining, especially for tunnels inside the shear zone. Furthermore, the potential occurrence of a back-thrust shear zone will also significantly affect the stress in the soil and the load in the lining. The properties of the soil, especially the stiffness, affect the development of shear zones, back-thrusts, and the safety of the lining as well.

Last but not least, the use of a danger factor, which has better capability than contours of plastic strain to illustrate the interaction between soil and tunnel, is helpful in evaluating how far the soil is away from a failure state. Safety of the segmental linings for combined axial load and flexure moment are examined by confirming that the points of loads are located inside the P – M curves. The influence of local controls (horizontal distance of the tunnel to the fault tip, fault dip, soil properties) on the safety of the tunnel thus can be identified through full-scale numerical analyses.

6.1. Strategy for hazard prevention

Based on the deformation characteristics found in this study, possible strategies for hazard prevention relating to the deformation induced by thrust faulting are as follows:

- (1) Provided that the potential shear zone is recognizable and the selection of the tunnel location is admissible, the tunnel should be located within the overburden in the footwall rather than in the hanging wall, thus avoiding the uplifting induced longitudinal failure and as much farther from the triangular shear zone as possible. If the tunnel is located in the hanging wall or very close to the shear zone or the back-thrust shear zone, failure is deemed as inevitable;
- (2) If the tunnel must be located close to the shear zones, the reduction of soil stiffness around the shear zone can apparently ease the magnitude of lining load and hence increase the safety; and
- (3) An artificial cavity between the tunnel and the shear zone can accommodate the deformation induced by fault thrusting, insulate against the passage of deformation from the shear zone to the tunnel, decrease the amount of load induced, and hence prevent the tunnel from failure. This idea is so far possible rather than feasible or practical; further assessments and cautious design are necessary.

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