Seismic Damage Control of Underground Structures Associated with Reduced Stiffness of Soil Foundation

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Seismic Damage Control of Underground Structures associated with Reduced Stiffness of Soil Foundation

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ABSTRACT

Liquefaction can significantly increase the deformation of soil around underground structures but at the same time, the stiffness and internal stress of soil are also dramatically reduced. Therefore, it is questionable that if it is really essential for the structure to accommodate all the free-field ground deformations as the conventional design approach currently requests. In the present study, a typical subway tunnel is subjected to earthquake excitations using dynamic Finite Element method coupled with soil and pore water considering the nonlinearity of both structure and soil foundation. The simulation is carried out in both drained and perfectly undrained conditions to clarify the effect of liquefaction on seismic response of the structures. The computational results imply that the required ductility of RC ducts can be moderated provided that the liquefaction risk is high. Furthermore, the effect of sheet piling on various types of tunnel damages such as uplift or shear deformation is also investigated. It is found that installation of sheet piles can cause the structure to suffer more damage due to the increase of relative stiffness between the structure and the soil.

INTRODUCTION

Liquefaction is a typical phenomenon in soft alluvial deposits in coastal areas on the occasion of strong earthquakes. It can cause severe damage to underground structures due to large deformation of soil, uplift or even floatation of structures. The relationship between the damage to structure and soil liquefaction was detected as early as in 1964 Niigata Earthquake and Alaska Earthquake (Hall and O'Rouke, 1991). Such damages to small pipelines or even large underground structures which were embedded in liquefiable soils were also found in many other earthquakes since then (Schmidt and Hashash, 1998 and Wang *et al.*, 2001). Therefore, various preventive countermeasures have been studied and developed so far to eliminate or alleviate damages to underground structures specially the uplift. Basically, they can be classified into three groups; soil improvement, drainage and suppression of shear deformations. Amid these strategies, installation of sheet pile walls is known to be effective and economical in reducing the uplift (Kimura *et al.*, 1995), however, its efficiency in other kinds of damage mitigation of underground structures such as controlling the shear deformation response is rarely studied. Besides, more accurate analyses which can well cover the nonlinearity of soil, structure and their interaction are quite necessary now. In most of the previous researches regarding soil–underground structure interaction, however, either nonlinearities of structure were neglected or soil was modeled with very simplified assumptions.

On the other hand, seismic design actions for underground ducts are generally characterized in terms of forced displacement and/or mean strains imposed on the structure. The rational and practical approach is to implicitly consider the interaction of underground RC with surrounding grounds. First, free-field ground deformations due to a seismic event are estimated and second, the underground RC is designed to accommodate these deformations through fictitious soil spring. This approach is satisfactory especially when lower levels of shaking are anticipated or the underground facility is in a stiff medium such as rocks (Hashash *et al.* 2001).

In this paper, a subway tunnel having interaction with surrounding soil is analyzed using coupled dynamic Finite Element method. In order to determine the effect of liquefaction on seismic response of structures, both drained and perfectly undrained states of pore water are discussed. Liquefaction may greatly increase in the deformation of soil around structures, but at the same time, the stiffness and internal stress of soil are dramatically reduced, too. A question is raised, what is the resultant of the both kinematics in RC nonlinearity? Furthermore, the effect of sheet piling on various types of tunnel damages such as uplift, shear deformation or cracking pattern is also investigated. It is found that the installation of sheet piles can change the damage pattern of the structure which should be considered in the design procedure of sheet piles.

NONLINEAR CONSTITUTIVE MODELS

Constitutive Model for Reinforced Concrete. A reinforced concrete material model has been constructed by combining constitutive laws for cracked concrete and that for reinforcement. The fixed multi-directional smeared crack constitutive equations (Maekawa *et al*, 2003) are used as the relations of spatially averaged stresses and strains. Crack spacing or the density and diameter of reinforcing bars are implicitly taken into account in smeared and joint interface elements no matter how large they are.

The constitutive equations satisfy uniqueness for compression, tension and shear of cracked concrete. The bond performance between concrete and reinforcing bars is taken into account in terms of tension stiffening and the space-averaged stress-strain relation of reinforcement is assumed to represent the localized plasticity of steel. The hysteresis rule of reinforcement is formulated based upon Kato's model for a bare bar under reversed cyclic loads. This RC in-plane constitutive modeling has been verified by member-based and structural-oriented experiments. Herein, the authors skip the details of the RC modeling and refer to Maekawa *et al.*, 1997, Maekawa and An, 2000 and Kato, 1979.

Constitutive Model for Soil. A nonlinear path-dependent constitutive model of soil which can predict the nonlinear response of layered soil under earthquake excitation is essential to simulate the behavior of the entire RC–soil system properly. Here, the multi-yield surface plasticity concept (Maekawa *et al*, 2003) is applied to formulate the stress-strain relationship of the soil following Masing's rule for the shear hysteresis (Masing, 1926). The authors also use the framework of elasto-plastic and damaging concrete modeling to formulate the soil nonlinearity as follows.

The basic idea of this method is rather simple. First, the total stress applied on soil particle assembly, denoted by σ_{ij}, can be decomposed of deviatoric shear stresses (s_{ij}) and compressive mean pressure (p) as,

$$\sigma_{ij} = s_{ij} + p\delta_{ij}$$

where δ_{ij} is Kronecker's delta symbol.

Soil is idealized as an assembly of finite numbers of elasto-perfectly plastic components, which are conceptually connected in parallel. Each component is given different yield strength, so all components yield at different total shear strains, which results in a gradual internal yielding. Thus, the nonlinear behavior appears naturally as a combined response of all components. Hence, the authors propose the total shear stress carried by soil particles being expressed with regard to an integral of each component stress as,

$$s_{ij} = \sum_{m=1}^{n} s_{ij}^{m}(\varepsilon_{kl}, \varepsilon_{kl}^{m}, G^{m}, F^{m})$$

$$ds_{ij}^{m} = 2G_0^{m} de_{eij}^{m} = 2G_0^{m}(de_{ij} - de_{pij}^{m})$$

$$de_{pij}^{m} = \frac{s_{ij}^{m}}{2F^{m}} df$$

$$df = \frac{s_{kl}^{m} de_{kl}}{F^{m}} = \frac{s_{kl}^{m} d\varepsilon_{kl}}{F^{m}}$$

where G_o^{m} is the initial shear stiffness of the m-th component, and F^{m} is the yield strength of the m-th one. These component parameters can be uniquely decided from the shear stress-strain relation (Maki *et al.*, 2005).

In general, the volumetric components may fluctuate and affect the shear strength and stiffness of soil skeleton. In fact, the shear strength of soil may decay when increasing pore water pressure brings reduced confining stress to soil particle skeletons. The multi-yield surface plastic envelope may inflate or contract according to the confinement stress as shown in Figure 1. It can be formulated by summing up the linear relation of the shear strength and the confinement stress as,

$$F^{m} = \chi F_{ini}^{m}$$

$$\chi = \frac{(c - I_1' \tan\phi)}{S_u}$$

$$I_1' = \frac{(\sigma_1' + \sigma_2' + \sigma_3')}{3}$$

where, F_{ini} is the multi-surface plastic envelope, □χ is the confinement index, (c,ϕ) are cohesive stress and frictional angel, and S_u is the maximum shear strength.

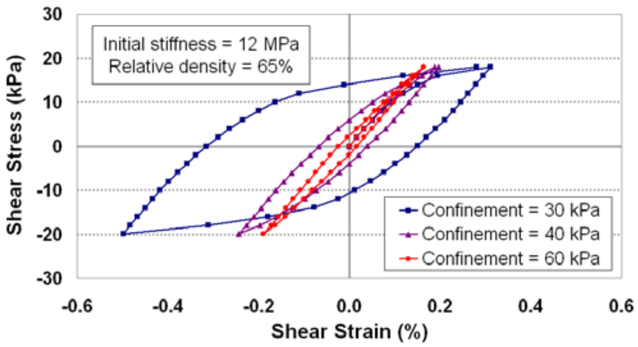


Figure 1. Soil model under drained cyclic shear loading

For simulation of the pore-water pressure and related softening of soil stiffness in shear, the volumetric nonlinearity of soil skeleton has to be taken into account. The authors simply divide the dilatancy into two components according to the microscopic events of soil particles. One is the consolidation or negative dilation as unrecoverable plasticity denoted by ε_{vc}. The other is the positive dilatancy associated with alternate shear stress due to the overriding of soil particles, which is denoted by ε_{vd} as,

$$p = 3K_0(\varepsilon_0 - \varepsilon_v), \quad \varepsilon_v = \varepsilon_{vc} + \varepsilon_{vd}$$

where K_0 is the initial volumetric bulk stiffness of soil particles assembly.

The volumetric irreversible contraction of particle will cause increasing pore pressure under hardly undrained states, which may lead to liquefaction. According to experiments of sandy soils, the following formulae are adopted as,

$$\varepsilon_{vc} = \varepsilon_{v,\lim}\left\{1 - \exp\left(-2(J_{2p} + J_{2p,ini})\right)\right\} - \varepsilon_{vc,ini}$$

which is represented by the accumulated shear of soil skeleton denoted by J_{2p} (Maekawa *et al*, 1997, Maki *et al.*, 2005) and $\varepsilon_{v,lim}$ is the intrinsic volumetric compacting strain corresponding to the minimum void ratio as,

$$\varepsilon_{v,\lim} = 0.1\left(\log_{10} I_1'^{0.6} + 1.0\right)$$

$$\varepsilon_{vc,ini} = \varepsilon_{v,\lim}\left\{1 - \exp(-2J_{2p,ini})\right\}$$

If the relative density of soil is assumed to be D_r, the following relation can be used to inversely decide $J_{2p,ini}$, which is a constant corresponding to the initial compactness of soil particles as,

$$D_r(\%) = \frac{\varepsilon_{vc,ini}}{\varepsilon_{v,\lim}} = \left\{1 - \exp(-2J_{2p,ini})\right\}$$

The shear provoked dilatancy which is path-independent and defined by the updated shear strain intensity denoted by J_{2s} as below,

$$\varepsilon_{vd} = \eta \frac{(a\,J_{2s})^2}{1 + (a\,J_{2s})^2}$$

$$J_{2s} = \sqrt{\frac{1}{2} e_{ij} e_{ij}}, \quad \eta = \frac{0.015\left(\varepsilon_{vc} + \varepsilon_{v,ini}\right)}{\varepsilon_{v,\lim}}, \; a = 25.0$$

According to the elasto-plastic and continuum damaging model of concrete (Maekawa *et al*, 2003), equivalent plasticity can be represented in general form with respect to the elastic scalar function as,

$$J_{2p} = \int dJ_{2p}^{m}, \quad dJ_{2p}^{m} \equiv \frac{1}{2}\bar{s}_{kl}^{m} d\varepsilon_{kl}$$

Then, the dilatancy factor can be defined in each component with different plastic range. Within this scheme, the liquefaction induced nonlinearity and cyclic dilatancy evolution can be consistently computed. Figure 2 shows the pure shear stress-strain relation and the corresponding pore pressure of undrained soil. Shear stiffness decay and cyclic mobility can be seen.

The overall experimental verification of the interaction analysis with soil and RC ducts was reported in (JSCE, 2002).

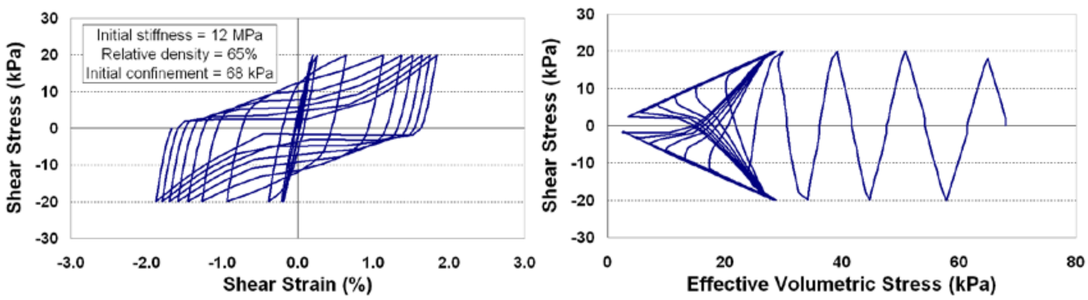
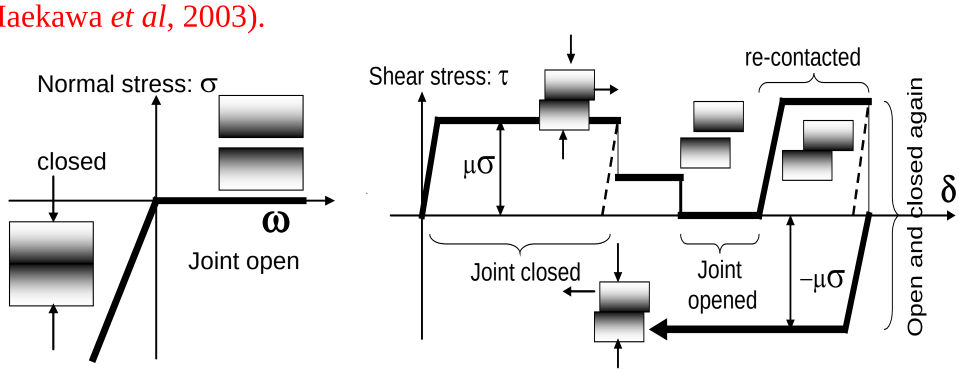


Figure 2. Soil model under undrained cyclic shear loading

Constitutive Model for Interface between RC and Soil. In this paper, the linear elastic model which assumes a bilinear relation for the opening/closure mode is employed to model the interfacial kinematics. The normal stress is zero in case of separation, which means no stress is transferred between the soil and the structure when the interface is open. On the other hand, the contact stiffness in closure mode is assigned a large value to ensure that no overlap is allowed, as shown in Figure 3(a). For shear sliding mode, shear stress–slip relation is assumed to be linear-plasticity as shown in Figure 3(b). The contact may slide if the applied shear stress exceeds the frictional shear strength, which is assumed to follow the Coulomb law. To apply this model, the initial condition of the soil–structure interface must be simulated to represent the actual static earth pressure. This is achieved by applying the natural gravity action of the soil mass alone before applying the dynamic action of the base rock (Maekawa *et al*, 2003).



(a) Normal response (b) Shear response

Figure 3. Normal and shear response of linear elastic interface model (Maekawa *et al*, 2008)

FINITE ELEMENT MODEL

Model Properties. To investigate the seismic behaviors of underground RC ducts, a typical subway tunnel section is modeled whose structural dimensions are shown in Figure 4. The center column to mainly support the dead weight of soil overlay has a rectangular cross section of 0.60 × 0.80 *m* and is idealized as firmly fixed to the slabs. The clear distance between two adjacent columns along the line is 3 *m*. The tunnel is stiffened with 45° haunches at the corners and has a longitudinal reinforcement ratio of 1.1% for side walls and slabs, 1.6% for the column, and web reinforcement ratio of 0.2% for all elements as shown in Figure 4.

The soil deposit is assumed to be loose sand with a friction angle of 30º and thickness of 15 *m* which is located on a 5-meter-thick layer of non-liquefiable soil which again lies on the bedrock as shown in Figure 4. The details of material property for reinforcing bars, concrete, interface joint and non-liquefiable soil layer are shown in Table 1.

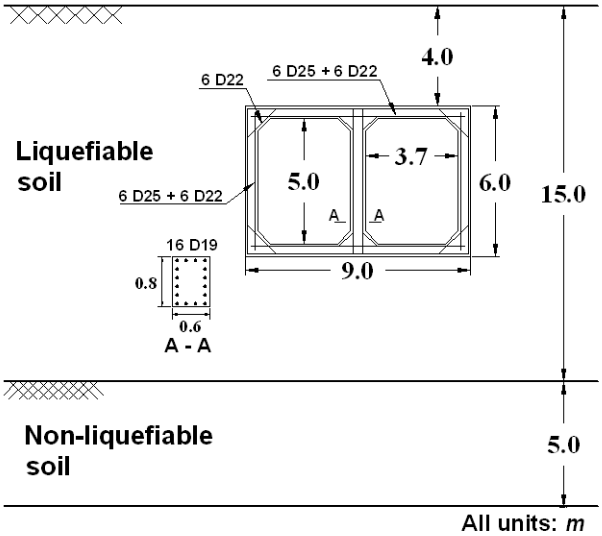


Figure 4. Soil-structure system geometry

Table 1. Material property

Non-liquefiable layer		Reinforced Concrete		Interface	
G_0	105 MPa	f'_c	24 MPa	Normal modulus	108 kPa
SPT *N*-value	15	Unit weight	24 kN/m^3	Shear modulus	103 kPa
Dry unit weight	16 kN/m^3	Poisson ratio	0.18	Friction angle	21º
Friction angle	40º	E_{steel}	2.0x10^5 MPa	Cohesion	0
Cohesion	100 kPa	f_y	240 MPa		
D_r	75 %				

By assuming the plane strain condition, the finite element mesh used in the analysis is composed of eight-node isoparametric two-dimensional elements for both RC and soil. The RC-soil interfacial elements are placed at an interface in between the soil and the RC elements. Since the angle of internal friction of the model sand is 30°, the friction angle of the interface is obtained using the relation $\delta = \tan^{-1}[(2/3) \tan\phi]$, which is about 21°. Totally, 7303 nodes and 2352 elements are arranged in the dynamic model. The north-south component of the rock base acceleration measured at 1995 Kobe earthquake, which is scale-adjusted to 0.3g based on the measurement at Kobe meteorological observatory, is used as the input bed rock motion in the seismic analysis. It shows a high horizontal ground acceleration with a short period as shown in Figure 5.

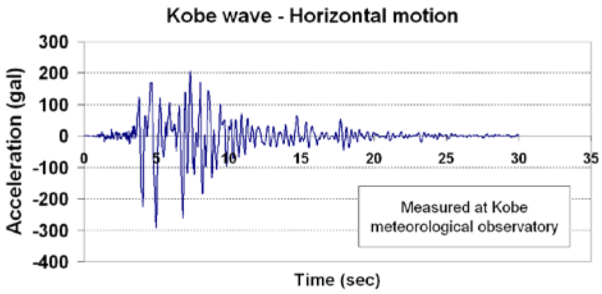


Figure 5. Input earthquake motion

Boundary Conditions. The boundary between the soil deposit and the bedrock is simply assumed to be fixed and would act as the bottom boundary of the analyzed domain at which the earthquake motion is imposed. The ground surface is assumed to be flat and free of loadings and the underground water level is assumed to locate up to the ground surface level when the soil is saturated.

The seismic behavior of the soil deposit in the far fields of the underground structure should assume the response of a free field. In the shaking table tests of soil-structures, a laminar shear box may be used to simulate the quasi-far-field boundary (Towhata, 2008). Here, quasi-far-field elements with a length of 10 *m* are placed at each extreme side of the analysis domain. The stiffness and unit weight of these elements are increased 100 times with respect to adjacent soil elements of the domain. As the far-field mode of seismic motion is simple shear, the length of the boundary condition (about the half of the domain height) is selected so that the bending deformation mode would not occur. In addition, confinement independent soil elements are used in the quasi-far-field zone in order to prevent the edge collapse in analysis. This boundary allows the harmonized horizontal and vertical displacements similar to the case of laminar shear box.

Analytical Approach. The seismic analyses of the soil-structure system require that an initial stress field in equilibrium be obtained beforehand (Maekawa *et al*, 2003). Therefore, an initial static drained analysis was firstly performed to determine the initial stress field and static earth pressure on the duct. This static stress field is then used as the initial condition for the subsequent dynamic run with the input excitation. The geological and construction history or path-dependence of the soil-structure system is not perfectly considered. But, the authors consider that these initial stress states may not be serious because of the high inelastic plasticity which is induced to soil under large ground motions.

Then, in order to investigate the effect of soil liquefaction on the damage of underground RC ducts, several models with and without the duct are analyzed in both drained and undrained states of pore water with various initial stiffness which is increased from 12 MPa to 230 MPa as shown in Table 2. The structure is assumed to be located inside the soil at a depth of 4 *m* without any change in the mesh property of the remaining soil elements.

Table 2. Material property for first soil layer

SPT *N*-value	G_0 (MPa)	D_r (%)
1	12	25
3	29	28
5	44	32
10	76	40
15	105	48
20	132	56
25	158	65
30	182	75
40	230	80

In order that the effect of the of sheet piling on the soil-structure response can be understood, an analysis is carried out with two sheet piles with a length of 18 *m* which are assumed to be installed at 0.5 *m* away from the both sides of the tunnel. The sheet pile is assumed to be made of steel with a Young modulus of 2.0×10^5 MPa and Poison’s ratio of 0.3 using elastic elements. The thickness of the sheet pile is assumed to be 0.12 *m* so that its stiffness would be close to the sheet piles which are practically used. The sheet pile and the liquefiable soil (G_0 = 35 MPa) are assumed to be perfectly bonded. i.e., no interface element is used between them. The results are compared with those without considering sheet piles.

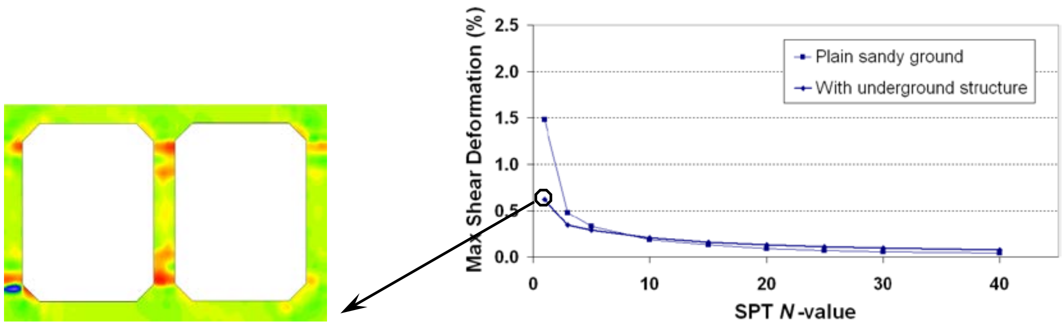
It should be pointed out that a fully undrained condition is assumed for saturated soil elements during the seismic action which could be an extreme case but still a reasonably clear assumption, because the required time for drainage of a several-meter-thick sand layer is 10-30 minute which is much longer than the duration time of earthquake loading (Towhata, 2008).

NUMERICAL RESULTS

Effect of liquefaction. Figure 6 shows the maximum shear deformation response spectrum of the RC duct and the soil volume which replaces it in the plain sandy grounds. It can be observed that the on-going practical design approach based on the free-field ground deformations can well predict the deformational demand on the underground RC structures in somehow stiffer soil mediums. In soft layers of soil which consists of loosely deposited sand, however, the design based on the free-field ground deformations would result in an overestimated deformational demand on the structural members as shown in Figure 6(a).

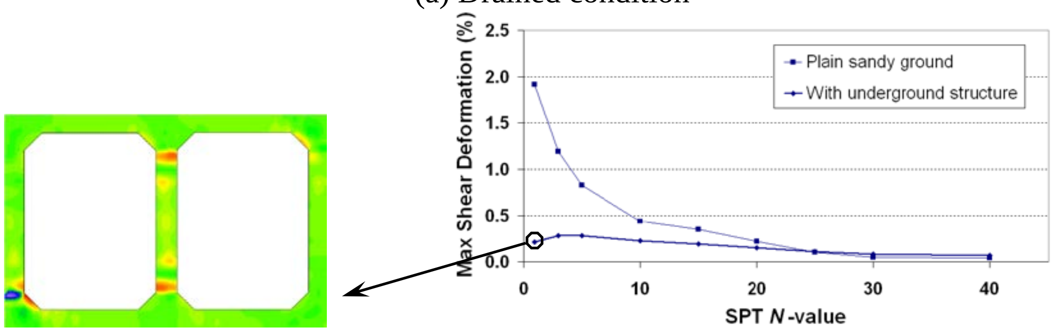
Besides, liquefaction may significantly bring about increased soil deformation which indicates that large soil strains with associated large degradation of the shear stiffness have developed within the ground. Hence, the underground RC duct which is located in the regions where the underground water level is high and designed without considering the interaction with the surrounding ground results in large amount of web reinforcement or large dimensions of structural elements for ductility demand.

However, because of the deterioration of the surrounding soil stiffness which takes place in liquefied soil, the deformation demand on the RC duct would dramatically decrease resulting in less damage to the tunnel as shown in Figure 6(b). Therefore, that large amount of reinforcement or thick structural elements is not necessary. Finally, it should be pointed out that some countermeasures have to be considered to reduce the uplift of underground structure in liquefiable soils. Here, we investigate the effect of sheet piling as a widely used method in reducing the uplift.



Strain state of tunnel at maximum shear deformation

(a) Drained condition



Strain state of tunnel at maximum shear deformation

(b) Undrained condition

Figure 6. Effect of soil stiffness on shear deformation

Effect of sheet piling. Many researchers have concluded that the effect of sheet piling in reducing the uplift of underground structures due to earthquake induced liquefaction is very obvious (Kimura *et al.*, 1995, Towhata, 2008, and Liu and Song, 2006). According to the finite element results in the present research, the uplift of the tunnel can significantly reduce when sheet piles with the length of are considered as shown in Figure 7. The deformed mesh after the earthquake motion for the cases with and without sheet piles is compared in Figure 8. It is clear that the floating of the subway tunnel is accompanied by the inward movement of liquefied soil under the structure which can push it upward. The sheet piles can reduce or even prevent such soil movement and therefore the uplift of the underground structure is much smaller.

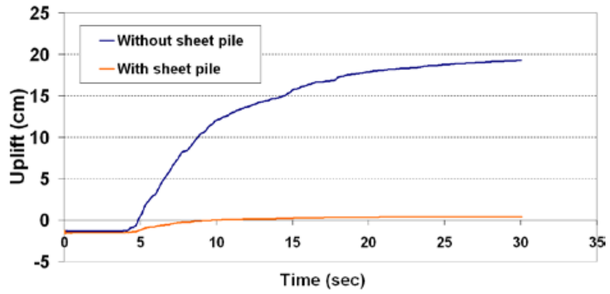
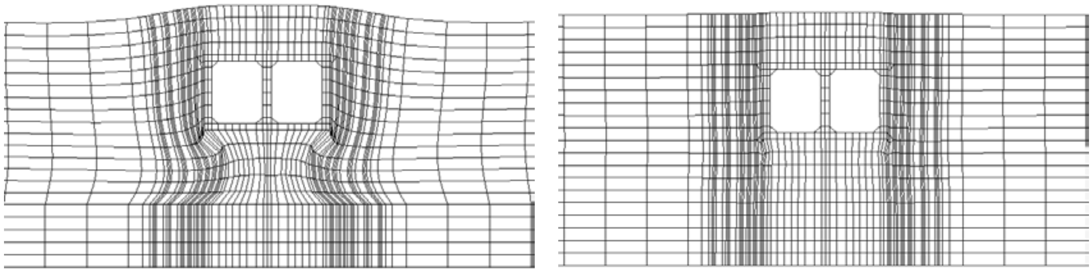


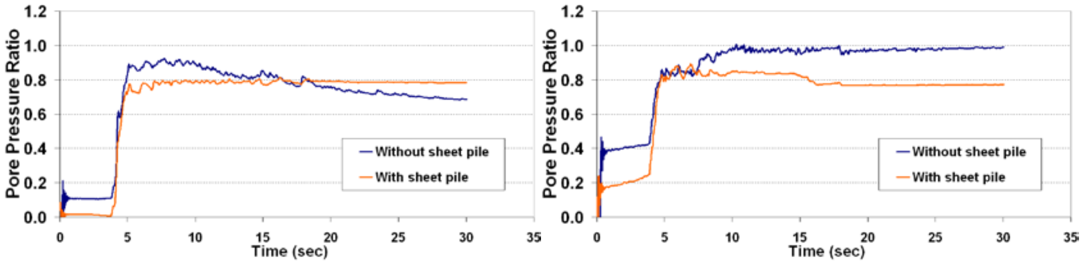
Figure 7. Effect of sheet pile in reducing uplift



(a) Without sheet pile (b) With sheet pile

Figure 8. Part of the deformed mesh (enlarged 5 times)

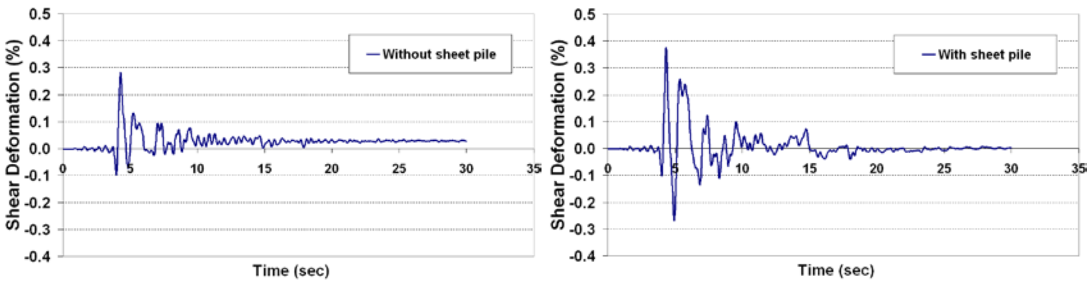
Liu and Song (2006) propose that cutoff walls cannot always inhibit the development of excess pore water pressure. In comparison with the case without cutoff walls, the magnitude of excess pore pressure of the enclosed liquefiable soils depends on the soil properties and earthquake magnitude. The excess pore pressure responses for both of the above-mentioned cases at the location 2.5 *m* below of the centerline of the centerline of the tunnel are shown in Figure 9(a). The excess pore pressures are expressed in terms of the ratio of excess pore pressure to the initial effective overburden pressure. It can be understood that although the excess pore pressure in both cases develops almost similarly at the beginning of the seismic motion, it would decrease while the uplift of the tunnel takes place for the case without sheet pile. In fact, the flotation of the underground structure would cause larger shear deformation of soil which contributes to the lowering of excess pore pressure. The sheet piles alleviate the floatation so that the excess pore pressure remains nearly constant. Furthermore, the excess pore pressure 1.5 m above the structure in the case with sheet piles is smaller than that in the case without sheet piles as shown in Figure 9(b); however, it is clear that the soil could also liquefy with sheet piles installed.



(a) 2.5 m below the tunnel (b) 1.5 m above the tunnel

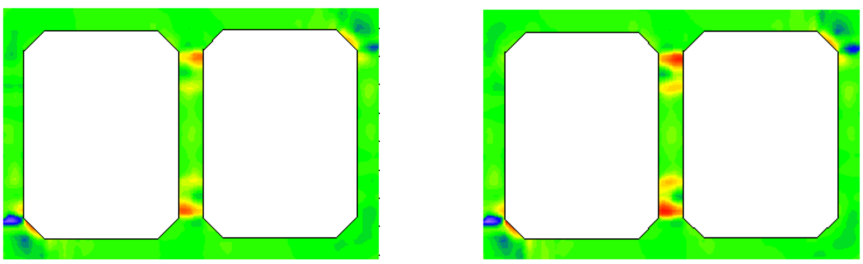
Figure 9. The development of excess pore pressure

The shear deformation of the underground structure, on the other hand, has obviously increased when sheet piles are installed, as shown in Figure 10. The maximum shear deformation of the tunnel, for example, would rise from 0.28 % in the no sheet pile case to 0.37 % in the case with sheet piles. This is believed to be due to the increase of relative stiffness between the structure and the soil. In fact, the relative stiffness between the structure and the surrounding soil has the most significant influence on the distortion of the structure due to racking deformations (Wang, 1993). When there is no sheet pile, the structure is rather stiff relative to the medium and does not deform so much. However, by restricting shear deformations in the enclosed soil, sheet piles can recover the soil stiffness degradation and thus more deformation demand would be applied to the underground structure. As a result, more damage would occur at the structural elements in the case with sheet piles as it can be observed in Figure 11.



(a) Without sheet pile (b) With sheet pile

Figure 10. Shear strain response of the tunnel



(a) Without sheet pile (b) With sheet pile

Figure 11. Shear strain state of the tunnel at the maximum shear deformation

CONCLUSION

In this paper, the nonlinear seismic response of an underground structure in dry and saturated liquefiable soils was investigated. From the numerical analyses, it can be concluded that in soft layers of soil which consists of loosely deposited sand, the design based on the large free-field ground deformations would result in high deformation demand on the structural elements which in turn requires large amount of reinforcement or large dimensions of structural elements. This situation would become even more severe in the regions where the underground water level is high because liquefaction may occur, which significantly increases the ground deformations. However, liquefaction would deteriorate the surrounding soil stiffness and thus the deformation demand on the tunnel would consequently decrease. Considering this issue could lead to a more optimum, economical and rational design of the RC underground structures.

Furthermore, it can be said that although installation of sheet piles can drastically alleviate the uplift of underground structures, it can cause the structure to suffer more damage due to the increase of relative stiffness between the structure and the soil. Hence, a rational design of sheet piling should consider both of its positive and negative effects on the underground structure response.

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