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Research Article

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Posted Date: September 15th, 2021

DOI: <https://doi.org/10.21203/rs.3.rs-889520/v1>

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Abstract:

Damages to underground structures due to liquefaction of the soils caused by cyclic loads such as earthquakes have always been an important issue in geotechnical underground engineering practices. This paper presents a numerical study of the utility tunnels at different burial depths in "Coh-Liq-Coh" horizontally layered liquefiable grounds using the finite-difference program FLAC^{3D}. "Finn-Byrne" cyclic load volumetric strain increment model simulates the fluid-solid coupling of saturated sand and the increase in pore water pressure

during vibration. The numerical model was loaded using an acceleration sine wave for dynamic calculations. The numerical results showed that the burial depths have a strong influence on the liquefaction of the soil beneath the utility tunnels and on the forces and deformations of the structures. Under the numerical simulation conditions in this paper, the greater the burial depth, the greater the liquefaction of the soil beneath the structure, the greater the shear stress on the side walls and the smaller the settlement difference between the structure and the surrounding soil. In the numerical simulations in this paper, a reasonable burial depth for utility tunnels was 0.8 to 1.1 times of the structure height.

Keywords: Utility tunnel; Finite-difference; Layered liquefiable soils; Burial depth; Soil liquefaction

1. Introduction

Growing urbanization and increasing urban populations worldwide have led to a huge demand for reliable infrastructure, and the use of underground space can help cities meet these increased demands (Makdisi and Seed, 1978). Urban underground utility tunnels effectively use the underground space of the city and centralize the arrangement of multiple municipal pipelines, which not only beautifies the city's environment but also facilitates the management of pipelines. Therefore, more and more cities are building and using utility tunnels. There are many cases of soil liquefaction causing damage to underground structures in previous seismic events, such as 1995 Hyogoken-Nambu earthquake (Tokimatsu and Asaka, 1998), 2010 Chile

earthquake (Kang et al., 2013), and 2011 Great East Japan Earthquake (Chian et al., 2014). The more widely utility tunnels are used, the more their safety deserve greater attention, especially in cities located in seismically active areas. The utility tunnel is a typical segmented long-line underground structure with lighter mass and usually shallow burial depth in the project, and the main part is usually made up of small sections of 20 m-30 m connected by joints. The characteristics of utility tunnels determine that one of the most serious risks in case of soil liquefaction is uneven settlement, which can lead to damage such as joint failure and internal pipe misalignment. At the same time, the force state of utility tunnels in liquefied soils is altered, causing structural deformation and damage to weak areas, etc., which eventually make them lose their working performance.

Loose saturated sand is subjected to cyclic loads such as earthquakes, the strength and shear modulus of the soil decreases, while the pore water pressure in the soil increases sharply, the effective stress decreases, and the soil changes from solid to liquid state. In previous studies, a large number of numerical simulations and model tests have discussed the seismic performance of utility tunnels (Chen et al., 2012; Chen et al., 2010; Ding et al., 2020; Jiang et al., 2010; Tang et al., 2020), but there are fewer studies on the forces and deformations of utility tunnels in large liquefied soils.

Currently, most studies of underground structures in liquefied soils are carried out in homogeneous liquefied soils (Ling et al., 2003; Liu and Song, 2005a; Mahmoud et al., 2020; Watanabe et al., 2016; Zhuang et al., 2015). In reality, however, underground structures are

mostly built in layered soil profiles. This paper investigated urban utility tunnels in horizontally layered liquefiable grounds. The upper and lower layers of the soil profile are impermeable cohesive soils (labelled Coh) and the middle layer is a thicker liquefiable sand layer (labelled Liq), the stratigraphy is abbreviated as "Coh-Liq-Coh", as shown in **Fig. 1**. In addition, underground structures with circular or single-compartment rectangular cross-sections have been studied more frequently in the past (Chen et al., 2018; Lee et al., 2017; Liu and Song, 2005b; Miao et al., 2018; Unutmaz, 2014; Yang et al., 2004), while trunk utility tunnels are more often in the form of multi-compartment rectangles, so the structural form used in this study was a rectangular three-compartment utility tunnel.

Illustration of the "Coh-Liq-Coh" stratigraphic section.

This paper placed the rectangular three-compartment utility tunnel in a typical “Coh-Liq-Coh” layered liquefied soils, focusing on the effect of the structure on soil liquefaction under different burial depth conditions; the forces and deformations of the structure in the liquefied soil under different burial depth conditions; and finally, judging the reasonable burial depth

based on the numerical simulation results. Nonlinear large deformation analysis of continuous media using the three-dimensional explicit function finite difference program FLAC^{3D}. The constitutive model of liquefiable sand was adopted from the built-in Finn model of FLAC^{3D}. Finn model can simulate the liquefaction characteristics of loosely saturated sand, so the numerical model can simulate the properties of underground structures in liquefied soils under cyclic loading (Byrne et al., 2004; Sudevan et al., 2020; Viand and Eseller-Bayat, 2017; Wu and Hsieh, 2014).

2. Numerical model and parameter setups

2.1. Analysis conditions and procedure

In this study, a 22 m thick layered saturated soil was assumed in which the rectangular three-compartment utility tunnel is located, as shown in **Fig. 2**. The "Coh-Liq-Coh" layered liquefiable soils were overlain by a 2m thick layer of clay, the middle layer was the liquefiable sand layer where the utility tunnel was located, 15 m thick, and the lower layer was also clay, 5 m thick. The design of the utility tunnel in the numerical model was based on the utility tunnel of the Beijing Daxing International Airport trunk line. The structure has a width B of 11.6 m, a height H of 4.1 m and a longitudinal dimension of 25 m ($11.6\text{m} \times 4.1\text{m} \times 25\text{m}$), comprising a communications compartment (2.6 m wide), an electrical compartment (2.8 m wide) and an integrated compartment (4.8 m wide). The thickness of structural side walls, top slab and bottom slab is 0.45m, and the thickness of structural partition walls are 0.25m. In this paper,

the burial depths of the structure in the soil were different. The distances between the top slab of the utility tunnel and the ground surface in the five numerical simulation scenarios were 2 m, 3 m, 4 m, 5 m and 6 m respectively. The bottom of the model was a clay layer, therefore a flexible bottom boundary condition was used for inputting seismic motion at the bottom of the model. The four lateral boundaries of the model were set as free field boundaries in the dynamic analysis to simulate a semi-infinite field. The water level is assumed to be at the ground surface. Details of the numerical model, including soil models, boundary conditions, and input motions, are presented in the following sections.

Stratigraphic distribution, gridding and location of monitoring points for numerical model with burial depth of 2 m.

2.2. Soil and structural models

The upper and lower parts of the numerical model in this paper were impermeable clay layers. The Earthquake occurs suddenly and the shaking time is short, therefore it can be

assumed that the saturated sand layer is undrained during the ground shaking process and the cyclic shear strain causes a rise in pore water pressure and liquefaction of the sand.

A large amount of previous test data shows that the volumetric compressive strain in sand soils is caused by cyclic shear strain due to the coupling of the shear response with the volume response. The "Finn-Byrne" sand model used in this paper is able to reflect the coupling of shear and volume in sand under cyclic loading. Martin (Martin et al., 1975) established four-parameter equations for irrecoverable volumetric strain, ϵ_v , and cyclic shear strain, γ , based on engineering experience and laboratory data. Based on that, Byrne (Byrne, 1991) established a two-parameter equation for volume-shear coupling by dividing the irrecoverable volumetric strain, ϵ_v , and the volumetric strain increment, $\Delta\epsilon_v$, by the shear strain:

$$\frac{\Delta\epsilon_v}{\gamma} = C_1^c \exp(-C_2^c(\frac{\epsilon_v}{\gamma})) \quad (1)$$

In the first loading cycle $\epsilon_v = 0$, therefore:

$$C_1 = \frac{(\Delta\epsilon_v)_{cycle\ 1}}{\gamma} \quad (2)$$

$$C_2 = 0.4/C_1 \quad (3)$$

It can be seen that the parameter C_1 controls the amount of change in volume strain.

C_2 controls the shape of the curve of the variation of the accumulated volume strain with the number of cycles.

C_1 and C_2 are related, as shown in Equation (3), but the presence of C_2 makes the application of Equation (1) more flexible when complete experimental data are available.

Byrne gave an empirical formula for the parameter C_1 , which can be calculated from the

relative density, D_r , of the sand:

$$C_1 = 7600(D_r)^{-2.5} \quad (4)$$

In cyclic shear loading, the total volume strain increment, $\Delta\varepsilon_v$, in each half cycle is divided into two parts, one for the recoverable elastic volume strain increment, $\Delta\varepsilon_v^e$, and one for the irrecoverable plastic volume strain increment, $\Delta\varepsilon_v^p$:

$$\Delta\varepsilon_v = \Delta\varepsilon_v^e + \Delta\varepsilon_v^p \quad (5)$$

Under simple shear conditions, the elastic volume strain increment can be calculated from the change in effective stress in 1/2 cycle and the tangential modulus of the effective stress of the restrained rebound of the sand skeleton, M :

$$\Delta\varepsilon_v^e = \frac{\Delta\varepsilon_v'}{M} \quad (6)$$

Under saturated undrained conditions, $\Delta\varepsilon_v \approx 0$, so that

$$\Delta\sigma_v' = -M \Delta\varepsilon_v^p \quad (7)$$

Under conditions of no change in total stress, the increase in pore water pressure is equal to the decrease in effective stress, $\Delta u = -\Delta\sigma_v'$.

Therefore, an equation for the incremental plastic volume strain versus the incremental pore water pressure can be developed:

$$\Delta u = M \Delta\varepsilon_v^p \quad (8)$$

The pore water pressure in any cyclic shear strain, u_g , can be expressed by a simple summation of the pore water pressure increments, $u_g = \sum \Delta u$.

In this paper, sand parameters were determined by relative density, D_r , and shear wave

velocity, V_s . The relative density of loosely saturated sand in this paper was set to 30% and the shear wave velocity was set to 100 m/s (Mahmoud et al., 2020). The maximum shear module, G_0 , was determined based on the value of V_s from the elastic relationship between G_0 and v_s , $G_0 = \rho \times V_s^2$. By assuming a Poisson ratio (ν) of 0.33, the bulk modulus of soil (K_0) was determined from G_0 .

In the soil profile, the clay (non-liquefied soil) in the upper and lower layers was modelled by the Mohr-Coulomb model. Material parameters for non-liquefied soils, including elastic modulus (E_N), Poisson's ratio (ν_N), cohesion (c) and friction angle (ϕ_N), were obtained from engineering experience.

A three-dimensional numerical model was established with a longitudinal dimension of 25 m. Both the structure and the soil were simulated using solid elements. The utility tunnel was modelled using elastic model with the material params shown in **Table 1**.

Table.1 Numerical parameters of materials used in this study

<i>Sand parameters</i>		<i>Non-liquefiable parameters</i>		<i>Structure & interface parameters</i>	
$D_r\%$	30	$E_N(\text{MPa})$	40	$E_s(\text{GPa})$	30
$G_0(\text{MPa})$	15	ν_N	0.33	ν_s	0.2
$K_0(\text{MPa})$	40	$\rho_{Nd}(\text{kg/m}^3)$	1600	$\rho_{sd}(\text{kg/m}^3)$	2500
$\rho_d(\text{kg/m}^3)$	1500	$c(\text{kPa})$	30	$k_s(\text{GPa/m})$	1
ϕ°	33	ϕ_N°	24	$k_n(\text{GPa/m})$	1
$k(\text{m/s})$	1.02E-04	$k_N(\text{m/s})$	1.00E-06	δ°	22-23
n_1	0.43	n_2	0.4	T_s	0
C_1	1.54	$K_w(\text{GPa})$	2	S_s	0
C_2	0.26				

Sand parameters. D_r : relative density; G_0 : initial shear modulus at a confining pressure of 100 kPa; K_0 : initial bulk modulus at a confining pressure of 100 kPa; ρ_d : dry density; ϕ : angle of internal friction; k : soil permeability; n_1 : porosity; C_1 , C_2 liquefaction parameters.

Non-liquefiable parameters. E_N : non-liquefiable soil young modulus; ν_N : non-liquefiable poisson ratio; ρ_{Nd} : dry density; c : cohesion; ϕ_N : angle of internal friction; k_N : soil permeability; n_2 : porosity; K_w : Water modulus.

Structure & interface parameters. E_s : structure young modulus; ν_s : structure poisson ratio; ρ_{sd} : structure density; k_s and k_n : shear and normal interface stiffness; δ° : friction angle of the interface surface; T_s : tensile bond strength; S_s : interface cohesion strength.

The soil-structure interface is represented as normal and shear springs between two nodes in contact with each other. The relative deformations of the normal and shear springs are controlled by the normal stiffness (k_n) and shear stiffness (k_s) values, respectively. k_n and k_s are set to ten times the maximum equivalent stiffness of adjacent zone.

2.3. Boundary condition

The numerical simulation process was divided into a static stage under gravity and a dynamic stage under the action of a horizontal sine wave. In the first stage (self-weight static analysis), the base boundary was fixed both horizontally and vertically and the lateral boundaries were fixed in the normal direction. In the second stage (seismic analysis), free field boundaries were used laterally to represent the lateral extent of the far field, and quiet boundary was used at the bottom. The quiet boundary eliminates the input velocity or acceleration and in order to input seismic motion at the quiet boundary, the acceleration wave needs to be converted to a stress wave. In this paper a sine wave was input in the form of stress at the nodes at the

bottom of the model along the horizontal direction. The shear stress, σ_s , determined from input shear particle velocity, v_s , as:

$$\sigma_s = 2(\rho C_s)v_s \quad (9)$$

Where ρ and C_s are the mass density and speed of s-wave propagation through medium, respectively.

2.4. Input motion

The details of the input seismic wave have an important influence on the response of the soil. Even if the control params of the input seismic waves (maximum acceleration, mainshock frequency, strong seismic holding time, etc.) are exactly equivalent, the effects of seismic waves with different waveforms on the structure is different (Makdisi and Seed, 1978). However this paper focused on the impact of saturated sand liquefaction on the utility tunnel. In order to reduce the difference in results caused by the use of different forms of seismic waves, the numerical simulation used sine waves for loading. The peak acceleration of the acceleration sine wave in this paper is 0.3 g, the vibration frequency is 5 Hz, and the duration is 12 s (Zhan-Fang et al., 2021). The acceleration gradually increases from 0 to the maximum value in 0-2 s. The stable period is 2-6 s, followed by a decreasing period, and the acceleration gradually decreases to 0 in 6-12 s, as shown in **Fig. 3**.

Time history of acceleration sine waves.

3. Liquefaction characterisation of soils and structures

3.1. Seepage and displacement of the soil

In this paper, section S is chosen as the monitoring surface for acceleration, excess pore water pressure, pore pressure and effective stress of the soil. The monitoring surface S is the middle section perpendicular to the longitudinal direction in the numerical model, as shown in **Fig. 2**. The left side of the figure shows the distribution of the "Coh-Liq-Coh " stratum and the right side shows the meshing of the numerical simulation.

In this section we illustrate the typical liquefaction characteristics of layered liquefiable soils and structure using the example of the utility tunnel buried at a depth of 2 m (the top slab of the structure is at 2m below ground surface). Under cyclic shear loading, the pore water pressure in the saturated undrained sand layer increases and seepage forms, **Fig. 4(a)** shows the seepage vector at section S at the end of the dynamic loading. A certain amount of water flows upwards in the "Liq" sand layer. The pore water pressure in the non-liquefied undrained layer

remains constant during shaking, while the pore water pressure in the liquefied sand layer increases, so that the seepage vector at the soil intersection points to the side with less pore water pressure. In the saturated sand layer, the flow is high in the upper and lower parts and low in the middle. The flow in the soil directly below the structure is high and the flow on the sides is low.

Soil liquefaction results at the cross section at the midpoint of the lengthwise axis of the numerical model, (a) Seepage vector distribution; (b) grid deformation and soil displacement vectors.

Previous engineering experiences show that uneven settlement of the structure and soil caused by soil movement is the main cause of damage to underground structures. Utility tunnel is light underground structure with a total structural density less than the surrounding soil

density. **Fig. 4(b)** shows the displacement vector of the soil and the structure. During the vibration, the soil around the utility tunnel moved towards the underside of the structure bottom slab and the underground structure uplifts.

Typical soil response at points M, N and K for the structure buried at a depth of 2 m: time histories of (a) acceleration, (b) excess pore water pressure ratio, (c) pore water pressure and effective stress.

3.2. Soil liquefaction

Liquefaction is defined as the loss of shear strength of the soil. The soil gradually changes from a solid to a liquid state and the amplitude of the soil velocity and acceleration decays. **Fig. 5(a)** shows the time histories of soil acceleration at different locations. points M and N are located to the left of the structure and point K is located in the soil below the structure and the monitoring points information is shown in **Fig. 2**. Compared to the acceleration of the input seismic wave, there was a significant attenuation of the acceleration at the three points. The attenuation of the acceleration at point M was more pronounced.

In numerical calculations, the excess pore water pressure ratio represents the degree of liquefaction of the soil, with r_u equal to 1 indicating complete liquefaction and r_u above 0.7 indicating that the soil is close to liquefaction. **Fig. 5(b)** shows the time histories of the excess pore water pressure ratio (EPWPR) at points M, N and K. The EPWPR started small, increased steadily and rapidly after 2s until 4s, after which the EPWPR remained stable until the end of

the earthquake. The EPWPR stable values at points M and N were 0.8 and 0.7 respectively, and the EPWPR stable value at point K was approximately 0.5. A comparison of **Fig. 5 (a)** and **(b)** show that the increase in the excess pore water pressure ratio of the soil corresponds to a decrease in its acceleration. The EPWPR at point N increased to 0.7 at around 4 s and the acceleration at point N also decreased significantly at around 4 s. The accumulation of pore water pressure and the reduction of effective stress in the soil under cyclic load action is one of the important characteristics of soil liquefaction. In **Fig. 5(c)**, the variation of pore water pressure and effective stress at points M, N and K were consistent with the state of liquefaction exhibited by the sand in **Fig. 5 (a)** and **(b)**.

3.3. Vertical displacements of the structure

Fig. 6 shows the time histories of vertical displacement at point B in the top slab of the utility tunnel and point A in the left soil. Continuous settlement at point A for a period of 6 s from the start of the shaking, with a maximum settlement of 12.2 mm, after which the settlement remained stable. There was a small settlement of the soil at point B from the start of the shaking to 2.8 s, due to volume contraction of the loose sand beneath the structure as a result of the shaking. During liquefaction, the soil moved towards the underside of the structure's bottom slab, point B gradually uplifted to 3.8 mm and then remained stable. There was a settlement difference between the utility tunnel and the surrounding soil, as shown in **Fig. 4(a)**.

Time history of vertical displacements at points A and B at 2 m burial depth.

4. Effect of different burial depths

4.1. Effect of burial depths on soil liquefaction

It is generally accepted that the shallower the burial depth of the utility tunnel, the lower the cost of the project, provided that the design requirements are met. However, in layered liquefied soils, soil liquefaction can lead to uplift of the structure if the burial depth is too shallow. Therefore, a reasonable depth of burial of the structure needs to be determined. Five options of burial depths of 2 m, 3 m, 4 m, 5 m and 6 m were set in this paper to determine the reasonable burial depth of the utility tunnel.

Fig. 7 shows the time histories of the EPWPR of the soil at point K, 2 m below the bottom slab of the structure, for five different sets of burial depth conditions. The monitoring position of point K changed with the burial depth of the structure. At a burial depth of 2 m, the stable EPWPR of the soil at point K was approximately 0.5. When the structure was buried at 3 m, 4 m and 5 m, the EPWPR curves of the soil reached 0.65 at around 9 s, 7 s and 5 s. At a burial

depth of 6 m, the EPWPR of the soil at point K reached 0.75. This indicates that in "Coh-Liq-Coh" layered liquefied soils, the deeper the utility tunnel is buried, the greater the liquefaction of the soil beneath the structure.

Time history of excess pore water pressure at point K for structures at different burial depths.

Time history of excess pore water pressure at point C for structures at different burial depths.

Fig. 8 shows the EPWPR time histories for monitoring point C under five different burial conditions, which is located to the left of the structure, at a depth of 5 m below the surface. The soil at point C reached liquefaction at all five burial depths, indicating that the depth of burial has little effect on the liquefaction of the soil at more distant locations on the side of the structure.

Fig. 9 shows the excess pore water pressure ratio clouds for the final state of the cross-section S for four different burial depth conditions. In this paper, the liquefied sand layer was divided into three parts; zone I was the soil directly below the structure, zone II was the soil to the left and right of the structure, and zone III was the soil beneath the sides of the structure. As shown in **Fig. 9**, the extent and degree of liquefaction of soil in Zone I increased with burial depth. In zone II, the soil closer to the structure were more affected by it, and the EPWPR values

for the soils close to the sides of the structure were below 0.5 for all five burial conditions, and no liquefaction occurred. Zone III was farther away from the structure and was less influenced by it, with little variation in the extent of liquefaction.

Figure 10 Cloud plot of excess pore water pressure ratio for the final state of the cross section at the midpoint of the lengthwise axis of the model for different burial depth conditions: (a) Burial depth 2m, (b) Burial depth 3 m, (c) Burial depth 5 m, (d) Burial depth 6 m.

4.2. Effect of burial depth on vertical displacement of the structure

Differential settlement between the utility tunnel and the surrounding soil is one of the main factors causing structural damage. **Fig. 10** shows the absolute value of the difference in vertical displacement difference between point B at the top slab of the structure and point A in the soil (the position of points A and B varies with depth of burial). The settlement difference between point A and point B decreased with increasing depth of burial, the settlement difference between points A and B at a burial depth of 6 m was 30.2% of that at a burial depth of 2 m.

Fig. 10. Settlement difference between point A and point B for different burial depth conditions.

Fig. 12. Maximum drift ratios of structural side walls and partition walls for different burial depths.

Fig. 11. Vertical displacement values of soil monitoring points on the horizontal surface of the structure bottom slab for five different burial depth conditions.

Fig. 11 shows the vertical displacement values of the soil on the same level as the bottom slab of the structure for five different conditions at the end of the shaking. Starting at the left boundary of the model and ending at the right boundary of the model, monitoring points were selected at 1 m intervals. At a burial depth of 2 m, the soil on both sides settled significantly

and the settlement difference between the soil and the structure was greatest. As the burial depth of the structure increased, the settlement at the monitoring point decreased and so did the settlement difference.

4.3. Effect of burial depth on structural deformation and forces

In this paper, the deformations and forces of the structure were represented by drift ratio and shear stress. The drift ratio is the absolute difference in lateral displacement between the top and bottom of the underground structure normalized by the height of the structure. Based on Eurocode 8 (Code, 2005) and CCSDB (Standard, 2010), the drift rate should be limited to 1.0%, 0.7% and 0.4% respectively, and in this paper 0.4 % should be used as the limit, while the shear should be within the design capacity value.

The rectangular three-compartment utility tunnel has two side walls and two partition walls, all four of which need to meet the limits of the drift ratio. **Fig. 12** shows the maximum values of drift ratios for these four walls in different burial depth conditions. The maximum drift ratio of the wall at a burial depth of 5 m was 27% of that at a burial depth of 3 m. In this study, the drift ratio values in all five conditions were well below the thresholds of the adopted performance criteria.

Fig.13. Maximum shear stresses in structural side walls and partition walls for five burial depth conditions: (a) shear stress distribution on the left wall, (b) shear stress distribution on the left partition wall, (c) shear stress distribution on the right wall, (d) shear stress distribution on the right partition wall.

Fig.13 shows the average distribution of the maximum shear stresses along the left wall, left partition wall, right partition wall and right wall for the five different structural burial conditions. As the depth of burial increases, the shear stresses distributed across the walls

became greater. The distribution of shear stresses in the walls showed a roughly concave shape, with the distribution of shear stresses in the left and right walls being greater than those in the partition walls. The maximum shear stress occurred at the intersection of the wall with the top and bottom slabs. In all conditions, the maximum shear stresses were within the bearing capacity. **Fig. 14** shows the maximum shear stress curve at the base of the right wall for different burial depth conditions. Within the depth range set in this paper, the maximum shear stresses increased linearly with increasing the buried depth.

Fig.14. Maximum value of shear stress at the bottom of the right wall for five burial depth conditions.

5. Conclusion

This paper used the finite-difference software FLAC^{3D} to investigate a rectangular three-compartment utility tunnel in typical "Coh-Liq-Coh" layered liquefiable soils with different burial depths. The model was loaded using a sine wave with a peak acceleration of 0.3g to simulate a seismic wave. The "Finn-Byrne" cyclic load-carrying strain incremental model was

used for liquefiable sands and the Mohr-Coulomb elastoplastic constitutive model was used for non-liquefiable soils. This paper investigated the effect of structures on soil liquefaction under different burial depth conditions and the forces and deformations of structures in liquefied soils. Vertical displacement, drift ratio and shear stress were used to calculate the dynamic response of the underground structure.

Earthquakes cause liquefaction of layered liquefiable soils. The soil on both sides of the structure moved towards the underside of the structure bottom slab, causing uneven settlement of the structure and the soil, creating a convex rise in the ground with a high centre and low sides, which in turn causes structural damage.

The different burial depths had a greater impact on the soil beneath the structure than the soil on either side of the structure. As the depth of burial increased, the extent of liquefaction of the soil beneath the structure increased and the degree of liquefaction became higher. Lateral soils further away from the structure were largely unaffected by changes in the depth of burial of the structure. When the structure was buried at a shallow depth, the settlement of the soil to the left and right of the structure and its differential settlement with the structure were both large, while both gradually decreased with increasing depth of burial. The drift ratios and shear stresses of the structure were within the limits and the drift ratios were much less than their limit value. The shear stresses in the wall increased linearly with depth of burial and could cause shear damage to the utility tunnel during more intense seismic activity. The above conclusions on the effect of burial depths on the structure were based on the burial depths (2m to 6m) set in

the numerical simulations in this paper. In previous studies, for deeply buried structures, the dynamic response of the structure eventually decreases as the depth of burial increases.

A comparison of the five different burial depth options showed that burial depth had a significant effect on the underground structure; a reasonable burial depth for the utility tunnel in this paper was 0.8 to 1.1 times the height of the structure. The effects of seismic features, the proportional relationship between the depth of burial and the dimensions of the underground structure, and the thickness of the liquefiable and non-liquefiable layers on the underground structure need to be further investigated.

Acknowledgements

The authors would like to thank the National Natural Science Foundation of China (No. 51578023) for funding the work presented in this paper.

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