NUMERICAL STUDY OF EARTH PRESSURE ACTING ON SEMI-UNDERGROUND STRUCTURES DURING AND AFTER EARTHQUAKES

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Semi-underground structures are often constructed for storing water. Most parts of their bodies are placed underground and the roofs are covered with thin or no soil layers. The current earthquake design standard for semi-underground structures mainly considers the earth pressure, which is proportional to ground acceleration and displacement. Detailed mechanisms of active and passive earth pressures are not considered in the current design method. In this paper, we study the dynamic interaction mechanism between ground and side wall of semi-underground structures during earthquakes based on FEM analysis. Earth pressures during and after the earthquake, which result from shear-failures in the surrounding soil, are especially examined. The results show that effects of soil failure enlarge the earth pressure. Passive earth pressure only acts on the shallow part of the wall, whereas active earth pressure acts on the deeper part as well, during earthquake. The residual earth pressure acts after earthquake, which is intermediate between the maximum and minimum values during earthquake.

Key Words: semi-underground structure, soil-structure interaction, earth pressure, shear-failures, FEM analysis

1. INTRODUCTION

Water supply facilities are among the indispensable lifelines for daily life and socio-economic activities. The seismic design of these structures is very important to maintain serviceability during and after large earthquakes. There are different structural types in water supply facilities, namely above-ground, underground, and semi-underground structures, as illustrated in Figs.1(a), (b), and (c). Water reservoir tanks installed on hills are representative examples of above-ground structures (Fig.1(a)). Note that in this paper the standard retaining walls are categorized as above-ground structures. Buried pipelines, vertical shafts and utility corridors are distinguished as underground structures (Fig.1(b)). Semi-underground structures, for example, water reservoirs, are often constructed having most parts of the structure placed underground and the roofs are covered with thin or no soil layers (Fig.1(c)). The difference between the semi-underground structures and the standard retaining wall is in the rigidity of the walls against surrounding grounds.

During earthquakes, the above-ground structures are mainly affected by their inertia forces; this is known as “inertia interaction.” The seismic force is generally given by the product of mass and total of ground and structural response acceleration. In static design procedure, such as the “seismic coefficient method” proposed by Sano1), the seismic force is assumed as the product of the weight and seismic coefficient. For the design of ground slope and/or retaining wall, the weight of soil block separated by the slip failure plane at the stationary state is used; this is called the “Mononobe-Okabe method” as proposed by Okabe, Mononobe, and Matsuo2). The reaction force acting on the retaining wall is restricted to the active earth pressure because the wall is easily deformed by the movement of soil block. Note that the reaction force gets larger as the overburden soil thickness increases3), because the active pressure is proportional to the overburden pressure (Fig.2(a)).
Koseki et al. 5) proposed the “Modified Mononobe – Okabe method” in which the failure plane was determined at the instant when the failure first occurs, based on the stability analyses of damaged retaining walls during the 1995 Kobe earthquake. This method is widely used for the static design of retaining walls 6), 7).

The underground structures are predominantly influenced by the surrounding soil due to “kinematic interaction.” Previous studies have shown that the structural deformation depends on the difference in rigidity and weight between the ground and the structure 8). Static and dynamic earth pressures acting on the underground structure are the main external forces for structural design. Static analyses are often used to evaluate the earth pressure during earthquake. One of these methods is “ground response acceleration method,” which models the soil–structure interaction (SSI) using static FEM, where the inertia force is given by the individual dynamic ground response analyses 9). Another method is the “seismic deformation method” in which structural beam and soil spring are often used for modeling, and the maximum displacements of free-field ground are inputted statically to the soil spring 10). In this method, the resulting reaction force against the structure decreases as the overburden soil thickness increases (Fig.2(b)). Note that the failure plane in the ground is not assumed, whereas the stiffness of soil spring is reduced in accordance with its plasticity 11). Inertia force, which is calculated based on the difference between structure and soil weights, is also considered.

A few studies have been performed for semi-underground structures that are stiffer than the surrounding soil. Sitar et al. 12) studied U-shaped retaining structures with varying stiffness, buried in cohesionless soil. They performed centrifuge model experiments and numerical simulations using ground motion records from the Kobe earthquake, and concluded that the post-earthquake earth pressures acting on both the side walls of stiff retaining structures were larger than the pressures at rest. Lohrasb and Anne 13) studied seismic earth pressures on vertically flexible underground structures. They conducted a series of shaking table experiments at the E-Defense facility, Japan. The results indicated that lateral earth pressures accumulated as the shaking time increases. In addition, the pressures were correlated with the fill soil settlement. The depth distribution of the maximum pressure was explained using the analytical solution for flexible structures suggested by Veletsos and Younan 14), 15). Inoko et al. 16) examined the effect of surrounding soil stiffness on semi-underground structure. However, standard design method for semi-ground structures have not been developed yet 17).

In this paper, we studied the mechanism of dynamic interaction between ground and side wall of semi-underground structures based on a two-dimensional FEM analysis using a simplified model. The elastic-perfectly plastic constitutive model with Mohr-Coulomb failure criteria was adopted. The semi-underground structure was assumed to be stiffer and lighter than the surrounding soil. We examined non-linear SSI, focusing on the stress state of the surrounding soil and the earth pressure acting against the side wall, during and after earthquake. The following effects were investigated: soil failure, thickness of surface layer, depth of structure, and period and amplitude of the input motion.

2. MODEL ANALYZED

The semi-underground structure and surrounding ground were analyzed using the computer program TDAP III 18), which is based on a two-dimensional plane-strain dynamic FEM. The goal is to show the mechanism of the earth pressure acting on the side wall of the structure, during and after earthquake, through the examination of stress state in the surrounding soil. Figure 3(a) shows the model analyzed.
Fig. 2  The reaction force assumed in static design methods.

(a) Seismic coefficient method for retaining wall

(b) Seismic deformation method for shaft

Fig. 3  Analysis model.

(a) Finite element mesh

(b) Model for connection between rigid structure and soil elements
The top of the model is a free surface, whereas the sides and the bottom are viscous boundaries. The width of the main area including a 10m-wide structure is 30 m, accompanied by 40m-wide side areas to eliminate the boundary effects. The values of surface layer thickness D and the structural depth d depend on the case. The side walls and bottom slab of the structure consist of rigid beams whose stiffness is 1000 times more than 1 m thick RC wall. The structural beams are connected to the modeled soil using the normal and shear springs, as shown in Fig.3(b). In order to make the mechanism simple for later discussions, the rigidity of the shear springs on the walls are set to small values (~ 10^{-8}) to simulate slippage between the walls and soil, whereas the rigidity of other shear and normal springs are set to large values (~ 10^{12}) to prevent slippage, separation, and invasion. It has been confirmed from the analysis results that the purpose has been achieved by setting these spring rigidities. The dynamic analysis calculates only the increments from the initial state of elements given by a static analysis in which a static distributed load simulating the weight of the structure is applied on the bottom slab, as shown Fig.3(b). The assumed weight of the structure is identical to that of soil (1.7 tf/m^2) to make the surrounding soil an initial condition of one-dimensional compression. Note that the mass of the structure in the following dynamic analysis can be given independently of the initial state.

Analysis was performed on 11 cases as listed in Table 1. The properties of surface layer are set in order to simulate a simple ground, which is categorized into “ground Type II” in the seismic design standard for water supply facilities in Japan^{19), 20}. The surface layer thickness, D, is set to 10 m for the basic case and varies between 7.5 m and 20 m for comparison. The shear-wave velocity of surrounding soil, Vs, is 151.15 m/s, calculated using the relation: \( V_s^2 = \frac{G}{\rho} \), where shear modulus \( G \) is 3.903×10^4

### Table 1 Specifications Applied to the Analysis.

| Case                        | Common parameters | Soil material | Structure | Ground | Input motion |
|-----------------------------|-------------------|---------------|-----------|--------|--------------|
|                             |                   | non-linear model | Cohesion C (kPa) | Friction angle \( \phi \) (degree) | Unit volume weight \( d \) (m) | Beam stiffness | Surface layer thickness D (m) | Natural period of ground (s) | Period of motion T (s) | Peak ground acceleration (g) |
| Li-D10-d05-T1.0-0.3g        | Unit volume weight \( 1.7 \) (tf/m^2) | Linear | - | - | 0.85 | 5 | Young's modulus 7.5 | 10.0 | 0.264 | 0.264 | 0.3 |
| Li-D10-d05-T1.0-0.3g        |                   |               |               |       |       |       |                        |               |                |                |                |
| MC-D7.5-d05-T1.0-0.3g       | Shear modulus \( G \) \( 3.903 \times 10^4 \) (kN/m^2) |               |               |       |       |       |                        |               |                |                |                |
| MC-D10-d05-T1.0-0.3g        |                   |               |               |       |       |       |                        |               |                |                |                |
| MC-D15-d05-T1.0-0.3g        | Poisson’s ratio \( \nu \) | 0.33 | Elastic-perfectly plastic model with Mohr-Coulomb failure criterion | 5 | 30 | 0.0 |                        |               |                |                |                |
| MC-D20-d05-T1.0-0.3g        | Damping ratio | 3 | (%) |               |               |       |                        |               |                |                |                |
| MC-D20-d05-T1.0-0.3g        | Shear wave velocity \( V_s \) | 151.5 | (m/s) |               |               |       |                        |               |                |                |                |
| MC-D20-d05-T1.0-0.3g        |                   |               |               |       |       |       |                        |               |                |                |               |
kN/m². The natural period of the ground is 0.264 s for the cases of \( D = 10 \) m calculated by the quarter-wavelength law. Following the relation: \( K_n = \sigma / \sigma_v = v / (1 - v) \), where \( \sigma \) is horizontal stress, \( \sigma_v \) is vertical stresses, and \( K_n \) is the earth pressure at rest, the Poisson’s ratio \((v)\) is set to 0.33 using \( K_n = 0.5 \).

Elastic-perfectly plastic constitutive model is assumed for non-linear analyses. Mohr-Coulomb criteria with friction angle, \( \phi \), of 30 degrees, is only adopted to simulate the shear failure of soil for later simple discussions. To keep the calculation stable, cohesion, \( C \), of 3 kPa in the failure criteria and 3% of damping factor²¹ set at 1 and 10 Hz based on Rayleigh damping, are introduced. Note that large damping ratio may give inadequate solutions during convergence calculation for nonlinear constitutive model. The damping values used has been properly confirmed.

Linear soil cases were also calculated to examine the effect of structure mass and to compare the soil failure cases, in which we adopted a non-mass case “Li-D10-d05-T1.0-0.3g” and a half-mass case “Li-D10-d05-T1.0-0.3g-M50%” whose structural densities were 0% and 50%, respectively, of the soil density.

The structural depth, \( d \), is set to 5 m to model typical water reservoirs, except in two cases “MC-D15-d10-T1.0-0.3g” and “MC-D20-d10-T1.0-0.3g,” which have thick surface layers. Horizontal motion is applied through the bottom viscous boundary and its amplitude is adjusted to make the surface ground response acceleration attain the following values: 0.1 g for the case “MC-D10-d05-T1.0-0.1g,” 0.5 g for the case “MC-D10-d05-T1.0-0.5g,” and 0.3 g for all the other cases. Note that the damping factor has little effect on the results because the amplitude of input motion is adjusted by the surface ground response.

**Figure 4** shows the depth-wise distribution of response acceleration in the surface layer for all cases. The period of input motion is set to 0.267 s, which is the natural period of the ground, to make it vibrate with its first mode. Exception is the case “MC-D10-d05-T0.5-0.3g” where the period of input motion is 0.132 s. The input waveform consists of 8-cycles of sine wave accompanied with 3-cycles of increasing and decreasing sections, as shown in **Fig.5**. In this study, we define four important instants as follows: \( T_{ini} \) representing the state before shaking, \( T_{pas} \) for passive state on right area of surrounding soil, \( T_{act} \) for active state on the right area of surrounding soil, and \( T_{aft} \) for the state after shaking (**Fig.5**). At \( T_{pas} \), the right area of surrounding soil (except the deep part as explained later) attached to the right wall of the semi-underground structure, is compressed laterally, where \( \sigma \) increases, and passive earth pressure acts against the right wall. In contrast, the left area is simultaneously tensioned laterally where \( \sigma \) decreases.

![Depth-wise distributions of \( \sigma_x \) at the four instants for linear non-mass case.](image)

**Fig.6** Distribution of horizontal stress \( \sigma_x \) for linear cases.
and active earth pressure acts against the left wall. At $T_{act}$, reversal of the tensile and compression areas take place as illustrated in Fig.6(a) and Fig.8(a).

3. STRESS DISTRIBUTION DURING AND AFTER SHAKING

Linear soil cases were investigated to examine the effect of structural mass and to compare with the soil failure cases, as shown in Fig.6. Figure 6(a) shows spatial distributions of horizontal stress ($\sigma_x$), normalized by the initial values ($\sigma_{0i}$), at the four important instants for the non-mass case “Li-D10-d05-T1.0-0.3g.” The red and blue colors represent the active ($\sigma_x < \sigma_{0i}$) and passive ($\sigma_x > \sigma_{0i}$) areas, respectively. The distribution pattern at $T_{act}$ is that at $T_{pas}$ flipped horizontally. The stress value after shaking is restored to the initial value because it is a linear case. Figure6(b) shows the distributions of $\sigma_x$ with depth at the four instants, where $\sigma_x$ should be earth pressure acting on the right wall of the structure and its extended deeper plane. The black solid line represents the stress distribution of $\sigma_x$ at $T_{ini}$, which marks the boundary between the passive state on the right and active state on the left. The gray dashed lines, representing the theoretical active and passive earth pressure, are shown as auxiliaries to compare $\sigma_x$ with the non-linear cases. Note that the theoretical active earth pressure near the ground surface is negative because of cohesion effect, which is given for stability of calculation. The blue solid and dashed lines are the stress distributions at $T_{pas}$ for the non-mass case “Li-D10-d05-T1.0-0.3g” and half-mass case “Li-D10-d05-T1.0-0.3g-M50%,” respectively. The red solid and dashed lines are the stress distributions at $T_{act}$ for these cases.

Differences between the stresses at $T_{pas}$ and $T_{act}$ (shown as blue and red lines) and the stress at $T_{ini}$ (shown as black line) are due to the dynamic SSI. It is shown that the effects of dynamic SSI are larger in the non-mass case than in the half-mass case.

As we focus on the mechanism of earth pressure during earthquake in this paper, only non-mass condition, giving largest effects of dynamic SSI, is adopted for the non-linear cases. The stress in the shallow part becomes large at $T_{pas}$, and exceeds the theoretical passive earth pressure corresponding to $\phi=30$ degrees and $C=3$ kPa, because the soil maintains linearity in this case. These values of $\phi$ and $C$ are used hereafter. At $T_{act}$, the stress is smaller than the theoretical active value. The stress at $T_{act}$ is restored to the initial value at $T_{ini}$.

To understand the mechanism for the above-mentioned findings, Fig.7 shows the schematic deformation of 2D-FEM mesh at $T_{pas}$, the instant when maximum displacement of the surrounding soil appears leftward. If the structure is heavy and similar to the soil’s weight, the structure moves with the surrounding soil as shown by the thick red dashed line, resulting in a small earth pressure. In the non-mass case, the structure moves insignificantly as compared to the surrounding soil because the inertia force of the structure is not acting. Thus, the horizontal stress is the largest in the non-mass case. The structural stiffness affects the distribution of passive and active earth pressure on the structural wall. Stiff structure keeps its form, while the surrounding soil deforms in a sinusoidal shape. This difference in deformations makes the upper-part of the stress passive and the lower-part active at $T_{pas}$, as shown in Fig.7. Peaks of the horizontal stress at GL-5 m in Fig.6(b) should be the effect of discontinuity due to the structural bottom corner.

Figure 8(a) shows the spatial distribution of normalized horizontal stress for a non-linear soil case “MC-D10-d05-T1.0-0.3g,” which is the basic case for the following comparison. The area in active state (shown in red) at $T_{act}$ is smaller than that of the linear case shown in Fig.6(a). The distribution at $T_{pas}$ is that at $T_{act}$ flipped horizontally. The stress values at $T_{act}$ do not return to the initial values and have residual stresses, which are passive in the shallow area and active in the deep area. Figure 8(b) plots the distributions of $\sigma_x$ with depth. Note that soil failure causes larger earth pressure than that of the linear case.
shown in Fig.6(b). It is shown that the horizontal stress is restricted along the theoretical lines of passive and active earth pressure. The horizontal stress at $T_{\text{aff}}$ is approximately the median values at $T_{\text{pas}}$ and $T_{\text{act}}$.

Figure 9(a) shows deformation in the right area of the structure at the four important instants for the basic case “MC-D10-d05-T1.0-0.3g.” The horizontal and vertical axis represent the distance from the right wall and the depth from the ground level, respectively. The black dashed lines and solid lines show the displacements at $T_{\text{ini}}$ and $T_{\text{aff}}$, respectively. The blue and red lines are the displacement at $T_{\text{pas}}$ and $T_{\text{act}}$, respectively. These displacements are scaled up 250 times using a fixed point at the bottom of the right wall, where the depth is -5 m. The ground surface near the structure sinks at $T_{\text{pas}}$, $T_{\text{act}}$, and $T_{\text{aff}}$, whose maximum value is about 6.4 mm at $T_{\text{aff}}$. In the area except near the wall, the displacement distributions at $T_{\text{pas}}$ and $T_{\text{act}}$ is in the shape of a quarter sine curve, which is understood as the first-mode response of surface layer.

The active and passive failure zones at $T_{\text{pas}}$ and $T_{\text{act}}$ are shown by red and blue colors, respectively, in Fig.9(b). In the area attached to the right wall, the passive failure zone at $T_{\text{pas}}$, shown by blue color, is limited to the shallow part, whereas the active failure zones at $T_{\text{act}}$, shown in red color, expand into deeper area. The failure zones at $T_{\text{pas}}$ are similar to those at $T_{\text{act}}$ flipped horizontally.

We discuss deformations and stresses of the four elements, named P1, P2, P3, and P4, which are in “near (X=0.125 m) and shallow (GL-0.625 m),” “far (X=9.375 m) and shallow,” “near and deep(GL-2.625 m),” and “far and deep” locations from the right wall and ground level, respectively, as shown in Figs.9(a) and (b). Figure 9(c) show the deformations of the elements at the four instants. For the element P2 and P4, which are in the far area from the right wall, simple shear deformation is predominant. These deformations can be understood as the response of free surface ground against horizontal ground motion.

In the area near the wall, the elements P1 and P3 are compressed vertically and stretched horizontally at $T_{\text{pas}}$, $T_{\text{act}}$, and $T_{\text{aff}}$. These deformations correspond to the active state and result in the ground surface settlement. Note that the deformation in active state is even seen at $T_{\text{pas}}$. It implies that plastic strain in active state grows more than that in passive state. The volumetric strains $(\varepsilon_1+\varepsilon_2)$ of these elements shown in the figure are about 0.03%. These strain values may result in only 1.5 mm of ground surface settlement for 5 m thickness of ground layer, which is small in comparison with the total settlement, about 6.4 mm, as shown in Fig.9 (a). It suggests that the main cause of ground settlement should not be the volumetric
Fig. 9 State in the right area of the structural wall at the four important instants for the basic case “MC-D10-d05-T1.0-0.3g.”

Fig. 10 Horizontal stress and strain of the two focused elements “P1” and “P3” touching the structural wall for the basic case “MC-D10-d05-T1.0-0.3g.”
strain but shear deformation of the ground.

**Figure 10(a)** shows the hysteresis of horizontal stress ($\sigma$) and strain ($\epsilon$) relationship for the elements P1 and P3. The horizontal chain line separates the active and passive states where the horizontal stress is smaller and larger than the initial stress $\sigma_{it}$, respectively. If active failure occurs, the strain decreases (moves leftward in the figure) without clear stress change, whereas the passive failure increases the plastic strain (moves rightward).

Only active failure occurs for the element P3, whereas both active and passive failures are seen for the element P1. However, the plastic strain due to active failure is predominant for the element P1, because the hysteresis moves leftward. Horizontal stresses at $T_{pas}$ shown by white squares, which are approximately the median values at $T_{pas}$ and $T_{act}$, remain after the earthquake, because neutral stress in shake is shifted by passive and active failure, which is similar to the behavior of back stress. The depth distribution of residual stresses at $T_{act}$ is also shown in [Fig.8(b)].

**Figure 10(b)** shows the Mohr’s stress circles of the elements P1 and P3. Open circles indicate the vertical stresses and filled circles are horizontal stresses. We explain the basic behavior of the stresses using the stress history of the element P1 as an example. The vertical stress moves little from the initial stress shown by dashed circle. Small decrease from the initial one in horizontal stress results in active failure, shown by the red circle. Larger increase in the horizontal stress is necessary for passive failure, because the failure line is far, as shown by the blue circle. It means that the active failure easily occurs in comparison with the passive failure. It is the reason why plastic strain in active state is predominant, though both active and passive failures occur for the element P1, in the shallow area.

In the deeper area where the element P3 is located, the initial vertical and horizontal stresses are larger than those in shallower area. However, the fluctuations of the horizontal stress due to earthquake motion became smaller, making the failure hard to occur in deeper areas. This is the reason why only active failure occurs for the element P3. The distribution of active and passive failure zones, shown in [Fig.9(b)], can be understood from the basic behavior of stresses mentioned above.

Lohrasb and Anne set up a hypothesis that “lateral residual pressures are accumulated and locked into the system and that an increase in soil density during shaking may translate into an increase in lateral pressures.” In our simulation, however, as the volume change is not the main cause of ground settlement and strength parameters, $C$ and $\phi$, are independent of the density, it can be said that the increase in soil density is not necessarily the reason why large earth pressure remains after the earthquake as shown in [Fig.8(b)].

### 4. EFFECTS OF ANALYSIS CONDITIONS

(1) Soil and structure conditions

Four cases with varying surface layer thicknesses of $D=7.5$, 10, 15, and 20 m were examined keeping the depth of structure ($d$) fixed at 5 m. **Figures 11(a) and (b)** show the depth distribution of horizontal stress for the cases of $D=7.5$ m “MC-D7.5-d05-T1.0-0.3g” and $D=20$ m “MC-D20-d05-T1.0-0.3g,” respectively. We additionally compared the results of five cases combining structural depth $d=5$ and 10 m, with surface layer thickness $D=10$, 15, and 20 m, except the case of $d=10$ m and $D=10$ m. **Figure 12(a)** compares the depth-wise distribution of $\sigma_x$ at $T_{pas}$, where the solid lines and dashed lines represent the results with structural depth $d=5$ m and $d=10$ m, respectively. It is observed that the distribution of horizontal stresses at $T_{pas}$ has no significant difference between all the cases with the same structural depth.

**Figure 12(b)** is converted from **Fig.12(a)** to emphasize the effects of structural and soil depth. The vertical axis is normalized by structural depth. Since the tangential stress on the structural wall is zero as a result of no friction, the horizontal and vertical stresses can be simply converted to Rankine’s coefficient of earth pressure, using the ratio of $\sigma_x + C\cot\phi$ and $\sigma_y + C\cot\phi$, where $C\cot\phi$ is added to consider the cohesion. The converted value is compared with the Rankine’s coefficient of passive and active earth pressure, $K_p = (1+\sin\phi)/(1-\sin\phi)$ and $K_a = (1-\sin\phi)/(1+\sin\phi)$, are 3.0 and 0.33, respectively, when the friction angle $\phi$ is 30 degrees without cohesion. Normalization in structural depth results in the single distribution for all cases. The peak value of earth pressure agrees well with $K_p$ indicated by the blue chain line, at an area shallower than 20% of structural depth. Note that the value of this ratio is changed by the conditions; for example the amplitude of input motion, as shown later. The smallest value of earth pressure is regulated with $K_a$ shown by the red dashed line, which appears at around the structural bottom. No significant effect of surface layer thickness is observed.

(2) Input motion

To examine the effect of resonance in surface layer, case “MC-D10-d05-T0.5-0.3g” was designed. In this case, a short-period motion ($t=0.132$ s) was inputted to compare with the basic case “MC-D10-d05-T1.0-0.3g.” **Figure 13** shows the distribution of horizontal stress $\sigma_x$ with depth for comparing with **Fig.8(b)**. **Figure 14** shows the normalized comparison at $T_{pas}$ as defined in the previous section. The
**Fig. 11** Depth-wise distributions of $\sigma_x$ at the four instants for the different surface layer cases.

(a) Thin surface layer ($D=7.5$ m)  
(b) Thick surface layer ($D=20$ m)

**Fig. 12** Depth-wise distributions of earth pressure at the $T_{pas}$ for different surface layers and structural depths. Black solid lines and dashed lines are the cases of structural depth $d=5$ m and $d=10$ m, respectively, with different surface layers thickness $D$. 

Theoretical active earth pressure $\Phi=30^\circ$, $C=3$kPa

Coeficient of earth pressure at rest $\sigma_{xy}/C \cot \phi$

Rankine’s coefficient of residual earth pressure $\left(\sigma_{xy}+C \cot \phi \right) / \sigma_{xy}+C \cot \phi$
open circles and the filled circles represent the results of short-period case and the basic case, respectively. No significant difference is seen in the figure, because the amplitude of the motion is regulated by the same acceleration value on ground surface.

Additional two cases with different amplitudes of input motion, the case “MC-D10-d05-T1.0-0.1g” with 0.1 g and the case “MC-D10-d05-T1.0-0.5g” with 0.5 g, were analyzed and compared with the basic case “MC-D10-d05-T1.0-0.3g” with 0.3 g. Figures 15 (a) and (b) show the depth-wise distribution of $\sigma_x$ for the cases of 0.1 g and 0.5 g, respectively, to compare with Fig.8(b). For the 0.1 g case, only active failure occurs. Figure 16 shows the normalized comparison at $T_{pas}$ using Rankine’s coefficient and structural depth. The open triangles, open circles, and cross marks represent the result of 0.1 g, 0.3 g, and 0.5 g, respectively. The thicknesses giving the

Fig.13 Depth-wise distributions of $\sigma_x$ at the four instants for the case of short-period input motion.

Fig.14 Depth-wise distributions of Rankine’s coefficient of earth pressure at the $T_{pas}$ for different periods of input motion.

Fig.15 Depth-wise distributions of horizontal stress $\sigma_x$ at the four instants for the cases with different amplitudes.
states appear alternatively during shaking. In the area near the structural wall, active and passive failure occur. The effects of both inertia and kinematic interaction. Semi-underground structures should be governed by the non-mass and side wall of semi-underground structures, on the basis of two-dimensional FEM analysis using elastic-perfectly plastic constitutive model with Mohr-Coulomb failure criteria. The earth pressure acting on semi-underground structures should be governed by the effects of both inertia and kinematic interaction. In order to make the mechanism clear, non-mass and rigid structures were studied, which gave the maximum interaction. The spatial distributions of the horizontal stress normalized by the initial stress, \( \sigma_x/\sigma_{x0} \), and the depth-wise distribution of \( \sigma_x \) acting on the structural wall and its extended deeper plane, were discussed.

The numerical results show that shear failure in the surrounding soil affects the earth pressure a lot. In the area near the structural wall, active and passive states appear alternatively during shaking. In the shallow area, both active and passive failure occur, while in the deeper area, only active failure takes place because very large horizontal stress is required to cause passive failure against large vertical stress. When passive failure occurs in the shallow area, active failure also appears in the deeper area simultaneously. Settlement is seen on ground surface near the structure during and after shaking. This is the reason why the soil is compressed vertically and stretched horizontally with little volume change, which corresponds to the active state.

After shaking, residual earth pressure is observed, which is intermediate between the maximum and minimum values during earthquake. It can be understood that neutral stress is shifted by passive and active failures.

Parametric study has been performed considering the surface layer thickness, structural depth, and predominant period and amplitude of input motion. To understand the effect of parameters, the depth is normalized by structural depth and the horizontal stress is converted to Rankine’s coefficient of earth pressure. The values of coefficient vary between \( K_p \) and \( K_a \), Rankine’s coefficient of passive and active earth pressure, respectively. Depth-wise distribution of the coefficient is not affected by the surface soil thickness. The amplitude of input motion determines the area in which the passive earth pressure acts, whereas its period has little effect on the distribution.

In this study, we have shown a qualitative mechanism of earth pressure acting on the side wall of semi-underground structure during and after earthquake, based on the numerical simulations using the limited and simple conditions. More realistic soil and structural conditions should be studied in the future for a quantitative estimation.

The conventional design methods could not represent the complex distribution of earth pressure acting on the semi-underground structure as shown in this study. It is necessary to develop a standard design method considering the detailed mechanism of earth pressure in order to design the semi-underground structures properly.

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