Research Article

Comparison on the Horizontal Behaviors of Lattice-Shaped Diaphragm Wall and Pile Group under Static and Seismic Loads

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Received 19 November 2015; Revised 28 April 2016; Accepted 3 May 2016

Academic Editor: Salvatore Russo

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Lattice-shaped diaphragm wall (hereafter referring to LSDW) is a new type of bridge foundation, and the relevant investigation on its horizontal behaviors is scant. This paper is devoted to the numerical study of the comparison on the static and seismic responses of LSDW and pile group under similar material quantity in soft soil. It can be found that the horizontal bearing capacity of LSDW is considerably larger than that of pile group, and the deformation pattern of LSDW basically appears to be an overall toppling while pile group clearly shows a local bending deformation pattern during the static loading process. The acceleration response and the acceleration amplification effects of LSDW are slightly greater than that of pile group due to the existing of soil core and the difference on the ability of energy dissipation. The horizontal displacement response of pile group is close to that of LSDW at first and becomes stronger than that of LSDW due to the generation of plastic soil deformation near the pile-soil interface at last. The pile body may be broken in larger potential than LSDW especially when its horizontal displacement is notable. Compared with pile group, LSDW can be a good option for being served as a lateral bearing or an earthquake-proof foundation in soft soil.

1. Introduction

The diaphragm wall industry started in Italy in the 1940s and spread throughout the world in many decades [1]. Progress has been made in design, equipment, and technology to fulfill the needs of varied underground constructions, and its application has been broadened into many engineering areas. In recent years, diaphragm wall is gradually applied to bridge engineering being served as the foundation directly [2]. Lattice-shaped diaphragm wall (hereafter referring to LSDW), shown in Figure 1, is a new type of bridge foundation composed of a cap and diaphragm walls [3]. The diaphragm walls under the cap are connected with rigid joints that form a rectangular frame or a frame with multichambers in horizontal section [4]. With the properties of high construction efficiency, low cost, small noise, and great rigidity, LSDW is especially suitable for being used as the large-span bridge foundations.

The construction detail of wall elements installation of an LSDW is illustrated in Figure 2. The wall elements of an LSDW are constructed in turn and can be divided into many segments, and the wall element constructed at the beginning can be termed as the preceding wall element while the subsequently constructed wall element is termed as the following wall element as shown in Figure 2. It is worthwhile to note that both of the construction processes of the preceding wall element and the following wall element consist of four steps. Among these steps, Steps 1 and 2 are mainly completed by an excavator. The excavator used in the construction system of LSDW has the advantages of high efficiency, low noise, and small occupation of the construction area. The preceding and the following wall elements are connected by preset joints to form an integrated wall segment, and then an LSDW can be constructed by combining multiwall segments arranged in different directions. More details about the construction procedures and joints arrangement of LSDWs with different

FIGURE 1: Structural layout of LSDW.

■ Construction of the preceding wall elements

■ Construction of the following wall elements

FIGURE 2: Construction process of wall elements for LSDW.

FIGURE 3: Construction procedures and joint arrangement for LSDW.

chamber numbers can be found in Figure 3. As a new type of bridge foundation, LSDW is proven to be a potential new solution to the settlement control of substructure for a high-speed railway in soft soil. Wu et al. [2] performed a study by comparing the vertical behavior of lattice-shaped diaphragm wall and pile group under similar material quantity in soft soil. It turns out to be that the bearing capacity will be improved and the settlement will be reduced for a practical bridge foundation if it can adopt LSDW instead of pile group in soft soil.

A bridge foundation constructed in soft soil, say in marine soil deposits, is subjected to not only vertical load but also large lateral load [5, 6]. Usually, the critical lateral forces on the foundation are due to the bending deflection of superstructure or water waves which cannot be neglected. Meanwhile, the dynamic response of a bridge foundation during an earthquake can deeply affect the overall response of the supported structure and can be a crucial factor to the safety and stability of the system. As a conventional technology, pile group has always been considered as a foundation type with large lateral bearing capacity [7] and be an effective earthquake-proof foundation. Therefore, an investigation on the horizontal behaviors of LSDW under static and seismic loads in soft soil compared with pile group should be expected undoubtedly.

LSDW is a new type of bridge foundation, and the comparison of its horizontal behavior with pile group has not yet been conducted in any other literatures. Compared with pile group, the concrete consuming quantity of LSDW may be larger than that of pile group, but its steel consuming quantity is less than that of pile group. Meanwhile, the cross-sectional area of LSDW is commonly smaller than that of pile group [8]. Therefore, the occupied construction area of LSDW is generally less than that of pile group. Although the construction cost of pile group may be slightly lower than that of LSDW in many cases, the situation can be contrary in some cases especially when they are constructed in an urban area due to the fact that the area occupation of LSDW is much less than that of pile group. In addition, the construction cost of LSDW may become less than that of pile group in most

cases with the development of its construction technology. Considering these, the comparison between LSDWs with different chambers and pile group in this paper is under the assumption that the construction cost of unit material quantity is basically similar for LSDW and pile group.

In order to investigate the static and seismic response of LSDW in soft soil, three models (pile group, LSDWs with a single chamber and two chambers) under similar material quantity are studied from a series of numerical analyses based on finite difference method (FDM) in this paper. Special attention is given to the comparison on the static bearing capacity and the seismic response of three models. The corresponding conclusions can be useful to the further application of LSDW served in bridge foundation engineering in soft soil.

2. Numerical Modeling

This paper attempts to compare the behaviors of LSDW with single (hereinafter for SCW) and LSDW with two chambers (hereinafter for TCW) as well as pile group subjected to horizontal static and dynamic loads in soft soil. In order to facilitate the analysis, the model dimension and mesh generating condition of SCW and TCW and pile group used in the static analysis are totally the same as those in the dynamic analysis. The structure dimensions and corresponding monitoring points used in the following numerical analysis of three models are plotted in Figure 4. Note that the material usages for three models are closed to each other and the detailed parameters of three prototype models are listed in Table 1.

In order to simplify the analysis, a typical soft soil site is selected as the bed soil in this paper. The soil profile consists of two types of soil, silty clay and fine sand. In which, silty clay (ranging from 0 to 16.5 m depth) can be treated as the soft soil layer and fine sand (ranging from 16.5 to 82.5 m depth) is the support layer, as illustrated in Figure 5. The calculating range of bed soil, 145 m (length) $\times$ 105 m (width) $\times$ 82.5 (height), is designed to be large enough so that the boundary effect can be neglected for three models. Note that

FIGURE 4: Cross-sectional dimensions and monitoring points of three models.

FIGURE 5: Soil layers distribution and computational boundary condition.

TABLE 1: Parameters of three models.

Items	Pile group	SCW	TCW
Cross sectional area (m ²)	21.21	26.24	28.16
Depth (m)	33	33	33
Wall thickness/pile diameter (m)	1.5	0.8	0.8
Cap thickness (m)	2	2	2
Material usage of cap (m ³)	304.5	162	132
Material usage of wall or pile (m ³)	702	865.92	929.28
Total material usage (m ³)	1004.29	1027.92	1061.28

the dynamic boundary condition will be introduced in the following section. Additionally, the detailed information of wall/pile body and bed soil can be found in Table 2.

Based on the information of structure and bed soil provided in Figures 4 and 5 as well as Tables 1 and 2, the numerical models can be established by FLAC^{3D} as shown in Figure 6. In the analysis, the component materials of pile and walls are reinforced concrete which can be treated as elastic material before the generation of structural failure. In fact, the pile is commonly modeled as an elastic solid in numerical analyses [9–11]. Therefore, the elastic model is

used for modeling piles and walls in this paper. In addition, soil nonlinear behavior should be taken into account to numerically simulate the dynamic behavior of pile under strong excitation. In this study, the Mohr-Coulomb constitutive model is applied for reproducing perfect plastic behavior and soil nonlinearity is considered by applying local damping which will be described in the following section. In order to simulate the structure-soil interactions, interface elements are installed between the contact surface of soil and structure, and the related parameters are listed in Table 3. In Table 3, k_n is the normal stiffness, k_s is the shear stiffness, c_c is the interfacial cohesion, and φ_c is the interfacial friction angle. It is worth noting that the Coulomb frictional law was also used for the interface modeling, which is defined by the following linear Coulomb shear-strength criterion to limit the shear force acting at an interface node:

$$F_{s,\max} = c_a A + \tan \varphi (F_n - uA), \quad (1)$$

where $F_{s,\max}$ is the ultimate shear force acting on the pile-soil interface; c_a is the cohesion of the pile-soil interface; φ is the angle of friction; F_n is the normal force on the interface; u is the pore pressure which is not considered in this analysis; A is the contact area associated with an interface node.

TABLE 2: Static and dynamic parameters of structure and bed soil.

Items	Density (kg/m ³)	Cohesion (kPa)	Friction angle (°)	Poisson's ratio	Elasticity modulus (MPa)	Damping ratio
Wall/pile	2500	—	—	0.2	30000	0.05
Silty clay	1600	15	14	0.4	1.28	0.05
Fine sand	1800	2	33	0.33	24.28	0.05

FIGURE 6: Numerical models and mesh.

TABLE 3: Parameters of the interface element.

Items	k_n (N/m ³)	k_s (N/m ³)	c_c (kPa)	φ_c (°)
Silty clay	1×10^8	1.18×10^6	9.75	9.1
Fine sand	1×10^8	8.9×10^6	1.3	21.45

To determine the initial stress field, the unit density of structure is initially considered to be identical to that of the soil body. The initial displacement field is then returned to zero, and the excess unit density of the structure is overlain on the initial stress field of the structure and bed soil can be accurately simulated. The initial stress is defined by

$$\begin{aligned}\sigma_s &= \gamma z, \\ \sigma_h &= K_0 \gamma z,\end{aligned}\quad (2)$$

where γ is the unit density of soil, σ_s is the initial vertical stress, σ_h is the initial horizontal stress, and K_0 is the horizontal pressure coefficient, depending on the friction angle φ as predicted by Jacky's equation $K_0 = 1 - \sin \varphi$ [12].

3. Static Analysis

After the completion of numerical modeling, the static analysis can be undertaken by loading on the left side of the cap for three models at the positive direction of x -axis (see Figure 4). The load-deflection curves of three models under different loading levels can be illustrated in Figure 7. It can be observed from Figure 7 that all the load-deflection curves of three models show a slowly changing trend in the entire loading process. The horizontal displacement of pile group is orderly larger than that of TCW and SCW under the same load during the entire loading process. Meanwhile, the

FIGURE 7: Load-deflection curves of three models.

differential horizontal displacements among the three models ascend with the increment of loads. Currently, there is still no specific determination regulation for the ultimate horizontal bearing capacity of LSDW that can be referenced. However, for a foundation, like a bucket foundation [13] whose load-deflection curve changes slowly, its bearing capacity can be determined by the displacement. Therefore, the horizontal bearing capacity of LSDW should also be determined by the displacement considering its structural layout is close to a bucket to some extent.

FIGURE 8: Unit shear forces of three models under different loads.

Figure 7 demonstrates that the load-deflection curves of SCW and TCW are consistently running above that of pile group, which indicates that the bearing capacity of SCW and TCW should be larger than that of pile group. In order to facilitate the comparison, a uniform standard should be made to determine the horizontal ultimate bearing capacity of three models. Therefore, the corresponding load under a 0.1 m deflection of the cap is considered as the ultimate horizontal bearing capacity for three models in this paper. It can be observed that the ultimate horizontal bearing capacities of pile group, SCW and TCW, are approximately 26.1, 27.2 and 28.2 MN, respectively. By conversion, the ultimate bearing

capacities of SCW and TCW can be improved to 104.21% and 108.04%, respectively, on the basis of pile group. Under the condition of similar material quantity (see Table 1), the horizontal bearing capacity of LSDWs (SCW and TCW) is still larger than that of pile group considerably.

The shear force distribution of three models under different loadings is illustrated in Figure 8, in which the shear force of pile group refers to monitoring point of P2 (see Figure 4(a)) and the shear forces of SCW and TCW refer to monitoring points of W2 and W4; see Figures 4(b) and 4(c). It can be seen that the shear force at an upper location of pile body is much larger than that of SCW and TCW, while the shear

FIGURE 9: Bending moment of three models under different loads.

force at middle and lower locations of pile body is slightly less than that of SCW and TCW. Therefore, the upper part of pile group is more likely to have deflection deformation compared with SCW and TCW. Figure 9 shows the bending moment of pile group, SCW and TCW under multiple levels of loadings, respectively. It is observed that the bending moments at the middle and the upper locations of pile/wall body are relatively large especially when they are under a high level of loadings. Because the sectional structure-soil contact area at the right side of pile group is orderly smaller than that of TCW and SCW, a relatively large bending deflection can be aroused for pile group compared with SCW and TCW when they are

under the same load. The bending moment of pile group is clearly larger than that of TCW and SCW in sequence, and the maximum moment is more than twice that of LSDWs (SCW and TCW).

In order to investigate the overall deformation situations of three models, the magnified deformation graphs of three models (the magnification factor is 50 times) under 30 MN load are demonstrated in Figure 10. Apparently, the overall rigidity of TCW and SCW is substantially larger than that of pile group; meanwhile, the bending moment of TCW and SCW is smaller than that of pile group under the same load as mentioned above. Therefore, the bending deflection of pile

FIGURE 10: Deformation graphs of three models (magnification factor: 50 times).

group is larger than that of LSDWs. It can be observed from Figure 10 that the deformation pattern of SCW and TCW basically appears to be an overall toppling while pile group clearly shows a partial bending deformation.

4. Seismic Analysis

In this paper, the three-dimensional dynamic analysis of three models is processed with $FLAC^{3D}$. The calculation is based on

the explicit finite difference scheme to solve the full equations of motion, using lumped grid point masses derived from the real density of surrounding zones (rather than fictitious masses used for static solution). This formulation can be coupled to the structural element model, thus permitting analysis of soil-structure interaction brought about by ground shaking. $FLAC^{3D}$ contains an optional form of damping, hysteretic damping, that incorporates strain-dependent damping ratio and secant modulus functions [14], allowing direct

comparisons between the equivalent-linear method [15] and the fully nonlinear method. The characteristics of the fully nonlinear method adopted in FLAC^{3D} can be concluded as follows.

- (1) The method follows any prescribed nonlinear constitutive relation. If a hysteretic-type model is used, and no extra damping is specified, then the damping and tangent modulus are appropriate to the level of excitation at each point in time and space, since these parameters are embodied in the constitutive model. By default, if Rayleigh or local damping is used, the associated damping coefficients remain constant throughout shaking and throughout the grid.
- (2) Using a nonlinear material law, interference and mixing of different frequency components occur naturally. Irreversible displacements and other permanent changes are modeled automatically. A proper plasticity formulation is used in all the built-in models, whereby plastic strain increments are related to stresses.
- (3) Both shear and compressional waves are propagated together in a single simulation, and the material responds to the combined effect of both components. For strong motion, the coupling effect can be very important. For example, normal stress may be reduced dynamically, thus causing the shearing strength to be reduced, in a frictional material.

4.1. Dynamic Input, Boundary, and Damp. Amplitudes of seismic waves are known to increase significantly as they pass through soft soil layers near the Earth's surface [16]. Therefore, a relatively safe seismic wave with normal acceleration amplitude is suggested to investigate the dynamic response of structure in soft soil [17]. In this paper, a seismic wave derived from an earthquake record in Tianjin city of China with peak acceleration of 1.47 m/s^2 and duration time of 10 seconds (as shown in Figure 11(a)) is applied as the input seismic loads in the dynamic analysis. It can be inferred from the Fourier amplitude distribution (Figure 11(b)) that the dominant frequency of the input wave concentrates in the range of 1 to 2 Hz.

The boundary conditions at the sides of the model must account for the free-field motion that would exist in the absence of the structure. For soils with high material damping, this condition can be obtained with a relatively small distance [18]. In this paper, free-field boundaries are applied to the model, as shown in Figure 12. Meanwhile, an absorbing quiet boundary [19] was applied along the base of the model to prevent the reflection of outward propagating waves back into the model.

One restriction when applying velocity or acceleration input to model boundaries is that these boundary conditions cannot be applied along the same boundary as a quiet (viscous) boundary condition since the effect of the quiet boundary would be nullified. To input seismic motion at a quiet boundary, a stress boundary condition is used (i.e., a velocity record is transformed into a stress record and applied

to a quiet boundary). A velocity wave may be converted to an applied stress using the formula:

$$\sigma_s = -2(\rho C_s) v_s, \quad (3)$$

where σ_s is applied shear stress; ρ is mass density; C_s is speed of s-wave propagation through medium; v_s is input shear particle velocity; C_s is given by

$$C_s = \sqrt{\frac{G}{\rho}}. \quad (4)$$

According to formulas (3) and (4), the input seismic wave of shear stress acting on the quiet boundary can be obtained from the velocity time history after baseline correction and filtering (Figure 11(c)), as illustrated in Figure 11(d). For a dynamic analysis, the damping in the numerical simulation should reproduce in magnitude and form the energy losses in the natural system when subjected to a dynamic loading. In this paper, local damping is applied for dynamic simulations and the local damping coefficient, α_L , can be obtained by the expression:

$$\alpha_L = \pi D, \quad (5)$$

where D is the damping ratio of the material, which can be found in Table 2.

4.2. Horizontal Displacement Response. During the dynamic loading process, the horizontal displacement time histories of the cap for three models can be drawn in Figure 13, respectively.

It can be seen that the horizontal displacement time histories of three models basically coincide with each other during the first three seconds, and the maximum displacement is approximately 0.13 m. After the third second, a difference among the horizontal displacements of three models can be found gradually. The horizontal displacement time history line of pile group is continuously running below that of SCW and TCW from the third second to the completion of loading. When the dynamic loading is completed, the horizontal displacements of the cap for SCW and TCW almost go back to zero while that for pile group still remains at a relatively large value. During the dynamic soil-structural interaction (viz, SSI), nonlinear behavior of soil can be aroused when the seismic wave is relatively large. The soil may separate or slide from the structure considering the interfacial situation. It can be inferred that plastic soil deformation may occur near the pile-soil interface during the second half of the dynamic loading process. The main reason is that the shear forces acting on the pile heads are significantly larger than that acting on the wall heads which is demonstrated in Figure 8. In other words, the force working on the soil around pile heads is greater than that around wall heads. Therefore, plastic soil deformation is easier to happen for pile group compared with LSDWs.

In order to investigate the overall deformation situation of three models during the dynamic loading process, the horizontal deformation of wall/pile body at different seconds

FIGURE 11: Tianjin seismic wave.

(in which 2, 4, and 7 s are selected due to the displacement of structure in these seconds being relatively large; see Figure 13) are illustrated in Figure 14. Note that the horizontal displacement of three models is enlarged by 50 times to highlight their deformation.

Based on Figure 14, it can be drawn that SCW and TCW almost show an overall bending deformation pattern even when their horizontal displacement is relatively large; see Figures 14(a) and 14(b). But for pile group, its deformation pattern appears to be a local bending deformation, especially when the horizontal displacement is large enough. In Figure 14(c), the horizontal displacement of the upper part of pile body in silty clay layer is obviously larger than that of the lower part of pile body in fine sand layer. The

deformation pattern of three models during the dynamic loading process indicates that the pile body may be broken in larger potential than LSDWs (SCW and TCW), especially when the horizontal displacement is notable.

4.3. Acceleration Response. In Figure 15, the acceleration time history curves at a different depth for three models are demonstrated in detail. It can be found that (1) the acceleration at lower locations is smaller than that at upper locations for three models which indicates that the structure has amplification effect on the acceleration of seismic wave from below upward; (2) at different seconds, the maximum absolute value of acceleration for pile group is basically smaller than that for SCW and TCW in sequence, and the

FIGURE 12: Dynamic boundary condition.

FIGURE 13: Horizontal displacement time histories of cap for three models.

differences are obvious at the upper locations of pile/wall body. The amplification effect of pile group is orderly smaller than that of SCW and TCW.

In order to investigate the amplification effect of three models acting on the input seismic wave in further detail, the amplification factors of structural peak acceleration at different depths based on the peak acceleration of input seismic wave are illustrated in Figure 16. It can be observed that the amplification factors of structural peak acceleration develop from below upward, and the growth speeds in fine sand layer (16.5~33 m depth) are smaller than that in silty clay layer (0~16.5 m) which indicates that the amplification effect of structure in silty clay layer is stronger than that in fine sand layer. Meanwhile, Figure 16 shows that the amplification factor of pile group is smaller than that of SCW and TCW

in sequence at different depths, and the amplification factor of SCW is close to that of TCW. Namely, the amplification factor of peak acceleration for pile group is smaller than that of LSDWs.

Figure 17 demonstrates the peak acceleration of inner soil points (including S_1 , S_2 , and S_3 ; see Figure 4) at different depths based on the peak acceleration of input seismic wave. It can be observed that (1) all the amplification factors of peak acceleration for inner soil points grow from below upward, and the growth speed in fine sand layer (16.5~33 m depth) is much smaller than that in silty clay layer (0~16.5 m); (2) the differences among the amplification factors of inner soil for three models at lower locations are obviously smaller than that at upper locations. In other words, the silty clay layer has great influence on the amplification factors of inner

FIGURE 14: Deformation of wall/pile body at different seconds (magnification factor: 50 times).

soil, especially for LSDWs (SCW and TCW); (3) compared with pile group, the inner soil points (S_2 and S_3) of LSDWs are surrounded by walls, and the energy cannot easily be dissipated or absorbed when the seismic wave arrives at the inner core of LSDWs. Therefore, energy aggregation may occur and the acceleration can be aroused rapidly for the soil core of LSDWs. This may be one of the reasons to explain why the structure peak acceleration of LSDWs is larger than that of pile group.

4.4. Dynamic Bending Moment. The bending moment time histories at a different depth for three models are drawn in Figure 18. It is observed that the varying trend of bending moment time histories is similar to the input seismic wave (see Figure 11), and the bending moment at upper depths is larger than that at the lower depths. It can also be observed that the peak bending moment of pile group is basically larger than that of TCW and SCW in sequence at different

depths during the dynamic loading. At 31.2 m depth, the peak bending moment of pile group is close to that of SCW and TCW, but it is almost twice larger than that of SCW and TCW at 1.5 and 18 m depth.

Figure 19 displays the peak bending moment (PBM) of three models at different depths. It can be found that the peak bending at the upper location is much greater than that at a lower location. Namely, the structure is under greater moment in silty clay than that in fine sand. In addition, the peak bending moment of pile group is larger than that of SCW and TCW in sequence in the whole depth range. Because the sectional structure-soil contact area at the right side of pile group is orderly smaller than that of TCW and SCW, a relatively large bending deflection can be aroused for pile group compared with SCW and TCW during the dynamic loading process. As observed above, the acceleration amplification effect of pile group is generally smaller than that of LSDWs (SCW and TCW), but the horizontal displacement

FIGURE 15: Acceleration time histories at different depth for three models.

response of pile group is not smaller than that of LSDWs at the first several seconds. One of the reasons is that the moment of pile group is larger than that of LSDW, so that the horizontal displacement of pile group is close to that of LSDW even when the acceleration of pile group is smaller than LSDW. Meanwhile, because the moment of pile group at the upper location is much larger than that at the lower location and the moment of pile group at lower location is close to that of LSDW, a relative large bending deflection of pile group may be generated during the dynamic loading compared with LSDW as shown in Figure 14.

4.5. Dynamic p - y Curves. Hysteretic loops for structure-soil reactions, that is, the dynamic p - y curves, of three models in different soil layers due to dynamic loads are shown in Figure 20. In general, the development and the shape

of dynamic p - y curves can partially represent the ability of energy absorption [20]. It can be seen from Figure 20 that both of the hysteretic loops of pile group in silty clay (Figure 20(a)) and in fine sand (Figure 20(d)) are larger than the corresponding hysteretic loops of SCW and TCW, respectively. Meanwhile, the varying range of relative displacement for pile group is larger than that for SCW and TCW, too. Therefore, pile group should have a stronger ability of energy absorption than that of LSDW. Note that the hysteretic loops of pile group are not occlusive compared with LSDW which indicates that plastic soil deformation may be generated for pile group during the dynamic loading process which is also observed from Figure 13 in Section 4.2.

In general, the dynamic acceleration response of LSDWs (SCW and TCW) is slightly greater than that of pile group. The reasons can be as follows: (1) compared with pile

FIGURE 16: Amplification factor of structural peak acceleration.

FIGURE 17: Peak acceleration of inner soil.

group, LSDW has stronger amplification effect on acceleration because its soil core can accumulate seismic energy as observed from Figure 17 in Section 4.3; (2) meanwhile, the overall rigidity of pile group is obviously smaller than that of LSDW, and pile group can be treated as a flexible structure compared with LSDW. Therefore, the ability to absorb energy for pile group is greater than that for LSDW. Nevertheless, the moment response of pile group is much larger than that of LSDW. As a result, the horizontal displacement response of pile group is close to that of LSDW in the first half of dynamic loading process but becomes stronger than that of LSDW due to the generation of plastic soil deformation in

the last half. Meanwhile, the pile body of pile group may be broken in larger potential than that of LSDW (SCW and TCW), especially when the horizontal displacement is notable. Therefore, LSDW can be a good option for being served as an earthquake-proof foundation in soft soil compared with pile group.

5. Conclusion

Comparisons on horizontal responses of LSDW and pile group under static and seismic loads in soft soil are presented in this paper. The main conclusions are as follows.

FIGURE 18: Bending moment at different depth for three models.

FIGURE 19: Peak bending moment (PBM) of three models.

FIGURE 20: Dynamic p - y curves of three models in different soil layers.

- (1) In static loading process, the horizontal displacement of pile group is orderly larger than that of TCW and SCW under the same load and the horizontal bearing capacity of LSDWs (SCW and TCW) is considerably larger than that of pile group; the deformation pattern of SCW and TCW basically appears to be an overall toppling, while that of pile group clearly shows a local bending deformation.
- (2) The dynamic acceleration response of LSDW is slightly greater than that of pile group, and the amplification factor of peak acceleration for pile group is smaller than that for LSDWs. For the soil core of LSDW, its acceleration can be aroused rapidly during the dynamic loading process.
- (3) The hysteretic loops of pile group and its varying range of relative displacement are larger than those for LSDWs. Meanwhile, the hysteretic loops of pile group are not occlusive compared with LSDWs which indicates that plastic soil deformation may be generated for pile group because the shear force acting on the pile heads is larger than that on the wall heads.
- (4) The horizontal displacement response of pile group is close to that of LSDW at first, and it becomes stronger than that of LSDW due to the generation of plastic soil deformation near the pile-soil interface at last. The pile body may be broken in larger potential than LSDWs (SCW and TCW), especially when the horizontal displacement is notable.
- (5) Compared with pile group, LSDW can be a good option for being served as a lateral bearing or an earthquake-proof foundation in soft soil. The conclusions made by this paper are based on the numerical results which should be validated by a field test or a model test. What has been described can serve as a basis for an expanded study.

Competing Interests

The authors declare that they have no competing interests.

Acknowledgments

This research is supported by the National Nature Science Foundation of China (nos. 41172260, 41372292, and 41530639), the Program for Changjiang Scholars and Innovative Research Team in University (no. IRT13092), and the Doctoral Fund of Southwest University of Science and Technology (15zx7152).

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