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Seismic behavior of pile foundation structure combined with soil-cement mixing wall using permanent pile based on centrifuge shaking table tests

Yuki Tomiyasu[1], K. Watanabe[2]

[1] Technical Research Institute, Obayashi Corporation, 4-640 Shimokiyoto, Kiyose, Tokyo 204-8558, Japan.
[2] Department of Civil Engineering, Faculty of Engineering, Aichi Institute of Technology,
1247 Yachigusa, Yakusa, Toyota City, Aichi, 470-0392, Japan

ABSTRACT

Soil-cement mixing wall (SMW) is commonly utilized as temporary structure during excavation process. In this research, lateral resistance of pile foundation structure combined with SMW was investigated aiming to utilize SMW as permanent pile and rationalize the foundation structure. Specifically, centrifuge shaking table tests and the simulation analyses were performed. Consequently, the effect of SMW to reduce pile stress during earth quakes was verified. This paper reports the test results and the discussion on the analysis method to evaluate pile stress.

Keywords: pile foundation; soil-cement mixing wall; centrifuge shaking table test; response displacement method

1 INTRODUCTION

Soil-cement mixing wall (SMW) is constructed by mixing cement milk with the in-situ soil in column shapes and inserting core members, such as H-section steel, into the columns. SMW is commonly utilized as earth retaining wall and impervious wall during excavation process. If SMW were also utilized as permanent pile and its bearing capacity and lateral resistance were considered, the design and the construction period of foundation structure could be rationalized.

In previous studies on SMW or similar structure to SMW, bearing capacity or lateral resistance was investigated mostly based on full-scale load tests of individual columns (e.g., Voottipruex, *et al.* 2011). However, characteristic of dynamic response of the whole structure including SMW has not been reported sufficiently (Watanabe, *et al.* 2017). In this study, the effect of lateral resistance of SMW on dynamic response of pile foundation was verified by centrifuge shaking table tests. Additionally, the test results of pile stress were simulated by a simple numerical analysis method to investigate a valid method to design foundation structure combined with SMW.

2 CENTRIFUGE SHAKIING TABLE TEST

2.1 Test models

For the centrifuge shaking table tests, ground model and two structure models were prepared in a shear soil container as shown in Fig. 1. The ground was composed of a single layer of dry sand and prepared by pluviation method to achieve the target relative density of 60%. No.7 silica sand was used for the material,

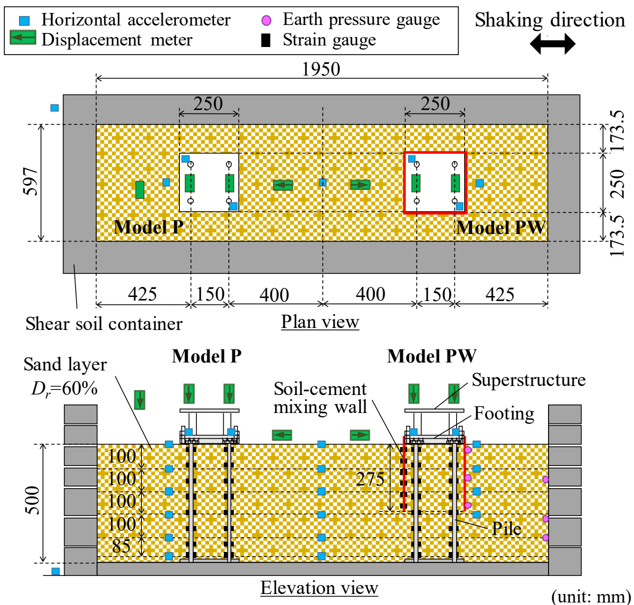


Fig. 1. Test models for centrifuge shaking table tests.

Table 1. Specifications of structure models.

		1/50 scale	1/1 scale
Superstructure (steel)	Mass	8.58 kg	1070 t
	Natural frequency	556 Hz	11.1 Hz
Footing (steel)	Mass	7.46 kg	933 t
Pile (stainless-steel pipe)	Length	500 mm	25.0 m
	Outer diameter	20.0 mm	1.00 m
	Thickness	0.50 mm	0.025 m
SMW (aluminum plate)	Length	275 mm	13.75 m
	Thickness	1.50 mm	0.075 m

Table 2. Test cases and input waves.

Case	Input wave	Maximum acceleration m/s^2
S1	Sine wave (2.4Hz in real scale)	1.0
S2		3.1
S3		3.9
S4		7.0
R1	Seismic wave (RINKAI wave)	0.5
R2		2.4
R3		3.2
R4		4.4
R5		6.0

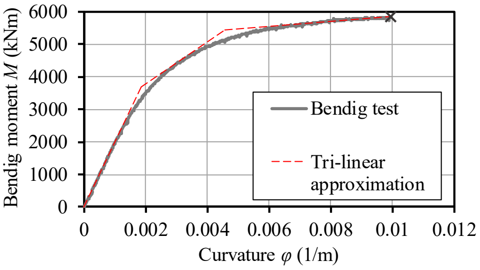


Fig. 2. Bending test result of a pile model.

which is G_s = 2.645 in real specific gravity and D_{50} = 0.19 mm in average particle size. As for structures, one of the two, "Model P", simulates a simple pile foundation structure and the other one, "Model PW", simulates a pile foundation structure combined with SMWs. Table 1 shows the specifications of structure models. The piles were modeled by stainless-steel pipes and the SMWs were modeled by aluminum plates to have equivalent bending rigidity to the real structures. The pile heads and the heads of SMWs were rigidly jointed to the footings. Two types of waves, sine wave and "RINKAI wave", were input to these models under 50G centrifuge acceleration where the scaling law was satisfied with the model scale of 1/50. RINKAI wave is a synthesized wave for prospective Minami-Kanto Earthquake in Japan. As shown in Table 2, each wave was gradually amplified at every test case. The followings were measured during the tests: acceleration of the ground and the footings, displacement of the ground surface and the structures, strain of the piles and the SMWs and earth pressure acting on the vertical SMW to the shaking direction (out-plane wall). Note that the values illustrated hereinafter have been converted from model-scale to full-scale. Additionally, a bending test of the same pipe as the pile models was performed to obtain the relationship between bending moment (M) and curvature (φ) of the pile models. Fig. 2 shows the test result and its approximation by tri-linear modeling. This approximation was adopted for the pile stress calculation of the centrifuge shaking table tests.

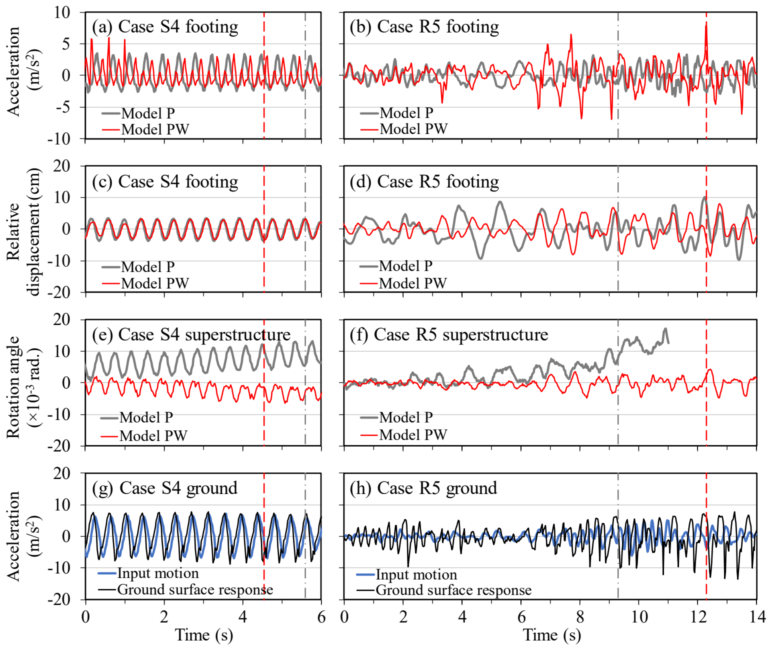


Fig. 3. Time histories of responses in Case S4 and R5.

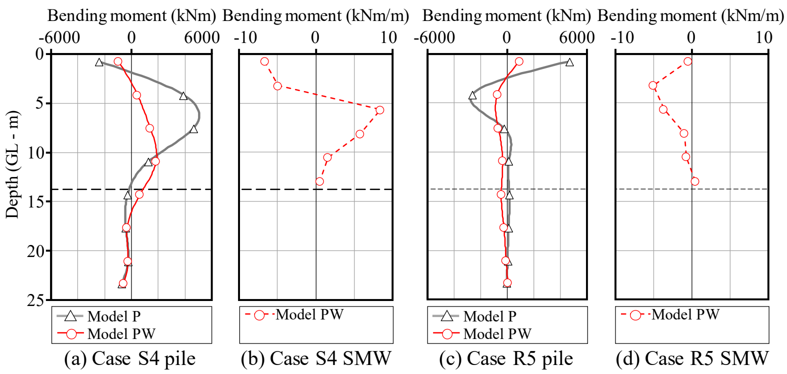


Fig. 4. Bending moment distribution of the piles and the SMW.

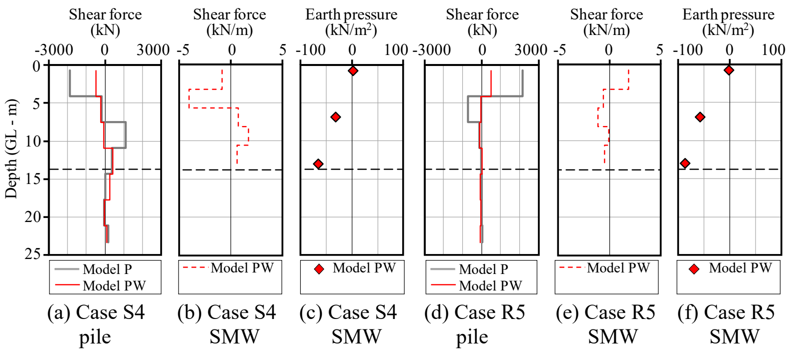


Fig. 5. Shear force distribution of the piles and the SMW and earth pressure distribution acting on the SMW.

2.2 Test results

Fig. 3 shows time histories of responses in Case S4 and R5 as the typical results. Relative displacement of footings, shown in Fig. 3(c) and 3(d), was calculated by second-order integration of lateral acceleration difference between the footing and the ground surface. Rotation angle of structures, shown in Fig. 3(e) and 3(f), was calculated from vertical displacement of both ends of each superstructure. As Fig. 3(a) and 3(b) indicate, response acceleration of Model PW comprised more short period components than that of Model P. As a reason for this, it is inferred that predominant period of ground-structure system was shortened by utilizing SMWs. Fig. 3(e) and 3(f) indicate that residual rotation was generated in both structures; however, the residual otation angle of Model P was larger than that of Model PW. Vertical dot lines in Fig.3 illustrate the time when the maximum bending moment was generated in the piles of each structure. The maximum shear force was generated at virtually the same time as them. It can be seen from these lines that pile stress was maximized at the peak time of footing acceleration, footing relative displacement or structure rotation angle.

Fig. 4 and Fig. 5 show distributions of bending moment and shear force of the piles and the out-plane wall at the time when the maximum stress was generated in the piles of each structure. These figures illustrate the results of one pile for each structure in Case S4 and R5 as representatives. Fig. 5 also shows dynamic earth pressure distributions acting on the out-plane wall observed at the same time. According to Fig. 4(a)-(d), 5(a), 5(b), 5(d) and 5(e), both bending moment and shear force of the pile of Model PW were smaller than those of Model P. Furthermore, the stress distributions of the out-plane wall indicate similar tendency to that of the pile. From these points of view, it can be said that the pile stress was reduced by utilizing SMWs. Additionally, Fig. 5(c) and 5(f) indicate that the larger the depth is, the higher dynamic earth pressure acted on the out-plane wall. It was confirmed from this tendency that the lateral subgrade reaction depends on the confining pressure.

3 ANALYTICAL STUDY ON PILE STRESS

3.1 Analysis models

The test results of pile stress were simulated by two-dimensional response displacement method. Fig. 6 shows the analysis models for the simulation. The piles, the SMWs and the footing of each structure were modeled as beam elements. The properties of their material are shown in Table 3; the footings were regarded as rigid bodies. The pile models were provided the tri-linear M-φ relationship illustrated in Fig. 2. The SMWs, out-plane wall and in-plane wall, were regarded as elastic bodies and provided the bending rigidities calculated from their width and thickness. Displacement of the in-plane wall was constrained to that of the out-plane walls at each depth. In the analyses, these models were subjected to external force: lateral inertia force of the superstructure and forced displacement of the ground. The former was applied to the center of the footing and calculated by multiplying the footing acceleration by total mass of the structure and the footing. The latter was applied to the piles and the out-plane walls through ground springs and calculated by second-order integration of ground acceleration. Each external force was calculated from

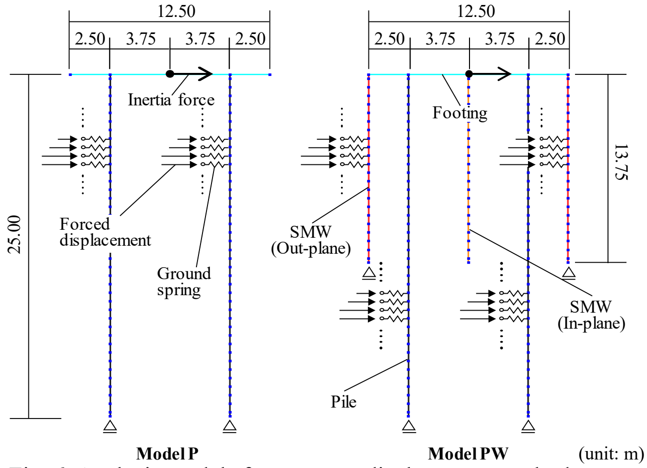


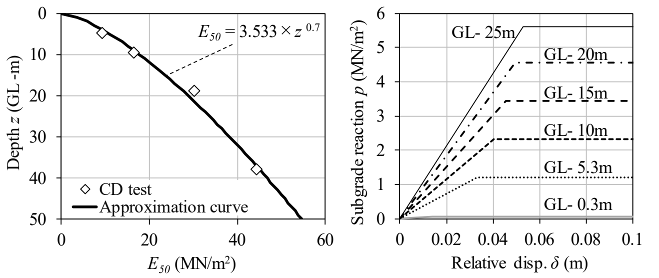
Fig. 6. Analysis models for response displacement method.

Table 3. Material properties of the structure models.

Structural element	Material	Poisson's ratio	Young's modulus kN/m²	Shear modulus kN/m²
Pile	Stainless steel	0.30	2.0×10^8	7.7×10^7
SMW	Aluminum	0.33	7.0×10^7	2.6×10^7

Table 4. Results of consolidated drained triaxial tests.

Confining pressure σ_3 kN/m²	Compression strength $(\sigma_1$-$\sigma_3)_{max}$ kN/m²	Deformation modulus E_{50} kN/m²	Cohesion c kN/m²	Angle of shear resistance φ °
25	90.2	9200	2.9	38.8
50	182.7	16300		
100	354.6	30100		
200	680.8	44200		



(a) Distribution of E_{50} (b) Characteristic of ground springs

Fig. 7. Evaluation of ground spring characteristics.

the test result observed at the time when the maximum bending moment was generated in the piles of each structure in each test case.

As suggested from Fig. 5(c) and 5(f), the subgrade reaction might have been associated with the confining pressure. Accordingly, the following approach was adopted to evaluate the ground spring characteristics.

Firstly, consolidated drained triaxial tests were performed using specimens which have the same condition as the ground model of the centrifuge shaking table tests, No.7 silica sand with 60% relative density. Table 4 shows the results and Fig. 7(a) shows the approximation of the relationship between deformation modulus (E_{50}) and depth (z). Secondly, the coefficients of lateral subgrade reaction were evaluated from Eq. (1) proposed by AIJ (2001).

$$k_{h0} = \alpha \cdot \xi \cdot E_0 \cdot \overline{B}^{-3/4} \quad (1)$$

Here, k_{h0} is the coefficient of lateral subgrade reaction in the state of initial rigidity (kN/m^3), α is the constant decided according to the evaluation method of E_0 (1/m), ξ is the coefficient representing the effect of pile group, E_0 is the deformation modulus (kN/m^2) and $\bar{B}$ is the non-dimensional pile diameter (the pile diameter in cm). The approximation curve of $E_{50} - z$ relationship shown in Fig. 7(a) was adopted to evaluate E_0. Finally, the capacity of subgrade reaction was evaluated from Eq. (2) proposed by Broms (1964).

$$p_y / \gamma B = \kappa \cdot K_p \cdot z / B \tag{2}$$

Here, p_y is the lateral capacity of subgrade reaction (kN/m^2), γ is the unit weight (kN/m^3), B is the pile diameter (m), κ is the coefficient representing the effect of pile group, K_p is the coefficient of passive earth pressure and z is the depth (m). Eq. (3), based on earth pressure theory, was applied to evaluate K_p.

$$K_p = \tan^2(\pi/4 + \phi/2) \tag{3}$$

Fig. 7 (b) shows the ground spring characteristics at several depths evaluated by this approach.

3.2 Analysis results

Fig. 8-11 show the results of the simulation analyses. These figures illustrate the results of two piles for each structure in Case S3 and R3 as representatives. Note that this analytical study did not target the test cases with large amplitude of input waves in which the pile stress approached to the ultimate value and the pile strain grew rapidly. Fig. 8-11 demonstrate that the analyses reproduce the test results of pile stress in general; however, they are not in good agreement in some test cases shown in Fig. 8(c), 8(e), 10(c), 10(e), 11(b) and 11(d). In these cases, the test results of two piles showed different distributions and the analysis results corresponded to one of them. This tendency can be attributed to the residual rotation behavior, mentioned in section 2.2 (Fig. 3(e) and 3(f)). Residual rotation generally induces uneven axial force on piles. The bending moment and the shear force of the piles should have been affected by this fluctuation of the axial force. In these analyses, however, the fluctuation of the axial force caused by the rotation behavior was not considered. In other words, we should incorporate vertical subgrade reaction, as well as vertical external force, to the analysis model in the case rotation behavior excels. Further studies would be required to determine more accurate method to evaluate pile stress.

4 CONCLUSION

In this study, centrifuge shaking table tests were performed to investigate dynamic response of pile foundation combined with SMW using permanent pile. As a result, it was seen that the pile stress during earthquakes reduces by utilizing SMW.

Additionally, the test results were simulated by a simple response displacement method adopting previously proposed evaluation of lateral subgrade reaction. The analyses generally reproduced the test results of pile stress. On the other hand, the necessity was pointed out to incorporate vertical components to the analysis model in the case rotation behavior induces the fluctuation of axial force on piles.

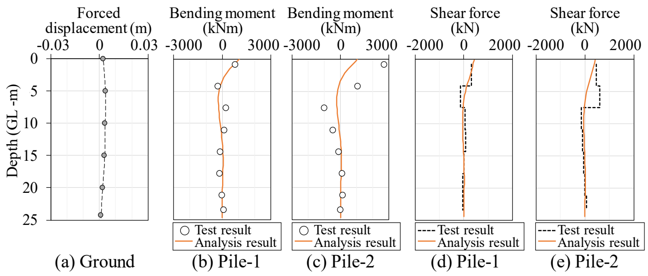


Fig. 8. Analysis results of pile stress (Case S3, Model P).

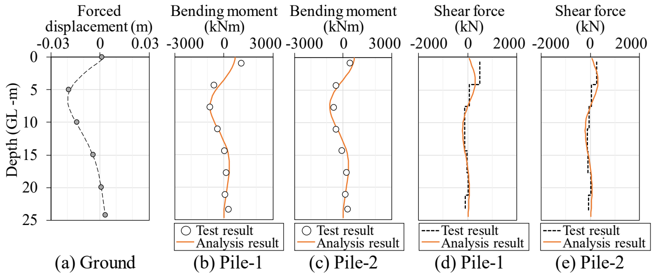


Fig. 9. Analysis results of pile stress (Case S3, Model PW).

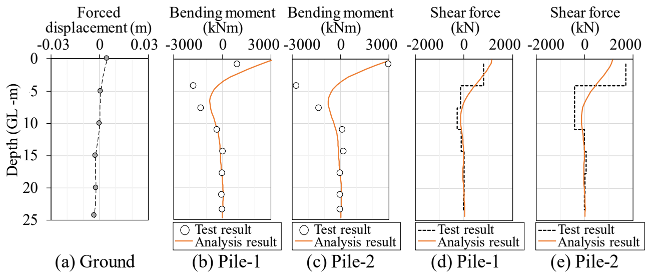


Fig. 10. Analysis results of pile stress (Case R3, Model P).

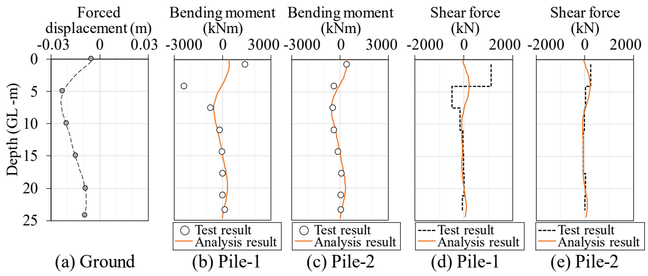


Fig. 11. Analysis results of pile stress (Case R3, Model PW).

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