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Numerical methods for estimating seismic retrofit effect of SG-Wall method

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ABSTRACT: A new technology (SG-Wall) was proposed for the target of seismic retrofit and front-water-depth-increase for quay walls. It is a combination of stabilization technique of dredged material and geosynthetics. In order to verify its seismic retrofit effect, a 1/24 model test utilizing an underwater shake table was conducted. In this paper, two methods were proposed for evaluating the seismic stability of marine structure with SG-Wall. One method is a simple method based on the Newmark method and considers the characteristic of earthquake wave. And the other one is dynamic effective stress-based FEM, which uses a new subloading Cam-clay model and soil-water coupled analysis technology. It is found that both methods are not sophisticated enough for practical design, and further study should be continued. Although two types of SG-Wall method (caisson type and sheet pile type) were proposed and tested in shake table tests, only the caisson type was considered here.

1 INTRODUCTION

As Japan is an earthquake-prone country, and many marine structures suffered damages during the 1995 Hyogoken-Nambu earthquake, it is important for civil engineers to increase the seismic-resistance of marine structure during reconstruction. Besides that, some existing quay walls are required to increase the front-water-depth for larger ships. For above objectives, a new technology, SG-Wall, which is the combination of stabilization technique (S) of dredged material and geogrid (G) for quay wall (Wall), was newly proposed by our research associate (Ichii et al., 2006).

Two types of SG-Wall methods, namely, caisson type and sheet pile type, were developed. A systematic experimental study by 1G underwater shake table test was carried out, and the results showed that both types were useful for seismic retrofitting of quay wall. Detail descriptions can be referred to Ichii, et al. (2006).

In this paper, two methods were proposed for evaluating the seismic retrofit effect of the SG-Wall. One is a simple method for estimating the residual displacement of caisson, which is based on Newmark method and considers the characteristic of earthquake wave. The other one is a dynamic effective stress-based FEM, which uses a new subloading Cam-clay model (Ye et al., 2005) and soil-water coupled analysis technology. Following the shake table test conditions, the quay wall without SG-Wall (Case 1) and with SG-Wall (Case 2) were analyzed, and the results were compared carefully.

2 A SIMPLE METHOD FOR ESTIMATING RESIDUAL DISPLACEMENT OF CAISSON

2.1 *Shape characteristic of wave*

This method is based on the Newmark method (Newmark, 1959) and considers the shape characteristic of wave. Most research works, such as Makdisi and Seed (1978), were based on the assumption that the wave shape is rectangular. The actual shape of earthquake wave, however, seems more like a combination of individual triangle wave. Therefore, the wave is simplified as isosceles triangles in this study, as shown in Figure 1(a).

As shown in Figure 1, three assumptions are made in analyzing the shape characteristic of wave. (1) Only the one side of the accelerations (Seaward inertia) will cause residual displacement; (2) The magnitude of acceleration below a certain threshold value (denoted as limit acceleration) will not produce residual deformation; (3) The gradient and the surpassing area of a single peak are obtained by average. The accumulated surpassing time T, surpassing area A_a and number of surpassing peak m can be obtained when the limit acceleration is

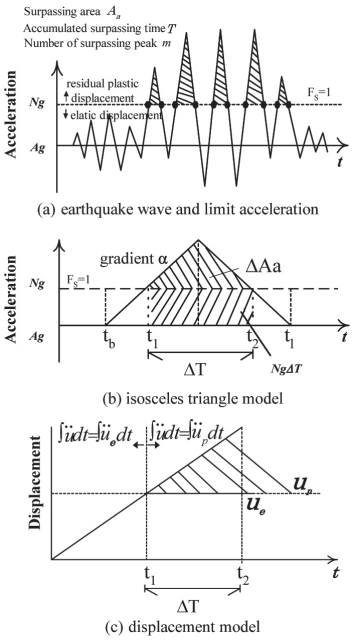

Figure 1. Earthquake simplification and threshold acceleration.

Table 1. Input waves in shake table test and its characteristics.

Input wave name	Max acce. A (gal)	Aa (gal. sec)	m	T (sec)	α (m/s^4)
Hachinohe (0.5 times)	85	0.014	0	0	0
Hachinohe (1.0 times)	180	0.006	1	0.008	868.75
Hachinohe (1.5 times)	260	0.014	1	0.016	557.81
Hachinohe (2.0 times)	335	0.014	2	0.024	808.33
L2 (0.5 times)	175	0.006	2	0.012	1094.44
L2 (1.0 times)	360	0.04	30	0.284	859.94
PortIsland (0.5 times)	200	0.004	2	0.012	1127.77
PortIsland (1.0 times)	430	0.034	12	0.134	799.28
PortIsland (1.5 times)	670	0.054	12	0.194	725.68
PortIsland (2.0 times)	926	0.062	14	0.24	811.8
sine wave (10.8 Hz)	200	0.242	20	0.384	392.25
sine wave (20.16 Hz)	190	0.104	19	0.162	825.33
sine wave (20.16 Hz)	390	0.268	20	0.35	679.18
sine wave (20.16 Hz)	600	0.314	20	0.408	836.21
sine wave (20.16 Hz)	800	0.332	20	0.428	1052.49
sine wave (10.8 Hz)	300	0.422	20	0.578	354.4
sine wave (10.8 Hz)	630	0.558	20	0.742	496.94

appointed. Then the average accumulated surpassing time ΔT, average surpassing area ΔA_a and gradient α can be obtained.

The waves used in the shake table test and its characteristics are shown in Table 1.

2.2 *Analysis model for caisson type SG-Walll*

The analysis model for caisson type SG-Wall method is shown in Figure 2. The spring and dashpot are utilized to model the interaction between caisson and surrounding ground.

The definition of displacement and the kinematics equations are given out as follows.

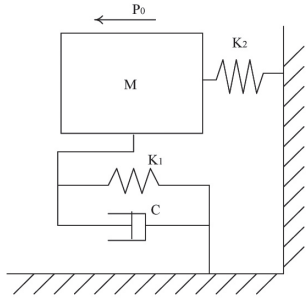

Figure 2. Analysis model for caisson type SG-Wall.

$$u = u_c + u_g \tag{1}$$

$$u_c = u_{ce} + u_{cp} \tag{2}$$

$$\ddot{u} = \ddot{u}_c + \ddot{u}_g = (1 + R)\ddot{u}_g \tag{3}$$

$$p = M\ddot{u} + k_1 u_{ce} + \dot{u}_c + k_2 u_c \tag{4}$$

where, u is the displacement of caisson, u_c is the relative displacement between caisson and the ground, u_g is the displacement of ground, u_{ce} and u_{cp} are elastic and plastic part respectively, R is an amplifier. Other symbols can refer to Figure 2.

By integrating Eq. (4) with time, the residual displacement of caisson can be obtained.

$$U_f = \frac{A_a + NgT}{\frac{4Ng}{\alpha}K_1 + \frac{T}{2}K_2 + \frac{24\,\mathrm{m}^2}{\alpha T}\left(\frac{\alpha T}{6\,\mathrm{m}} + Ng\right)C} \cdot \frac{p_0}{g} \tag{5}$$

2.3 *Analysis results*

The threshold acceleration is 156 gal, which is calculated with a safety factor for rotation $Fs = 1.0$. The estimated displacement by the Eq. (5) and the observed residual displacement were shown in Figure 3. The parameters of spring and dashpot for above estimation are summarized in Table 2.

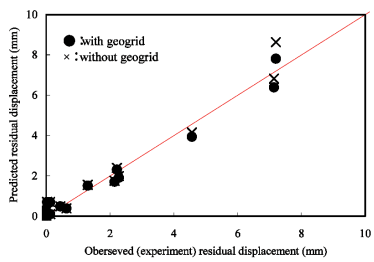

Figure 3. Estimated displacement and observed displacement.

Table 2. Parameters for spring and dashpot.

Parameter	Unit	Value
Soil spring (K1)	kN/m/m	0
Viscous coefficient(C)	kN · s/m	0.25
Geogrid spring (K2)	kN/m/m	10

From Figure 3, it can be concluded that the proposed simple method can estimate the residual displacement of caisson. And it also shows that the constraint effect of geogrid can reduce the residual displacement of caisson only when the magnitude of displacement is large.

3 DYNAMIC FEM SIMULATION

The dynamic effective stress-based FEM program DGPILE-2D, which is developed from DGPILE-3D (Zhang and Kimura. 2002), were used in this study. A new subloading Cam-clay model (Ye et al., 2005) was applied to sand, which takes into consideration the density dependency property of sand. And a u-p format soil-water coupled scheme is also adopted. The u-p formulation has been developed during last two decades, such as the works of Oka et al. (1994) and Zienkiewicz and Shiomi (1984).

3.1 *FEM model*

The meshes for Case 1 and Case 2 are shown in Figure 4. Scales of the meshes are the same as those in shake table testes.

The initial stress state was considered as a gravity field. The traction boundary conditions were set as followings.

Bottom: X and Y fixed

Sides: X fixed, Y free

As to the drained conditions, the sea bottom and the ground surface were permeable while the other boundaries were impermeable.

3.2 *Dynamic fem results*

A comparison between Case 1 and Case 2 will be conducted to reveal the seismic stability of SG-Wall method. Due to the fact that under, only Case 2 test has been carried out, In FEM analysis, at first, the parameters were calibrated by the try-and-error method with the test result of 10.8 Hz 600 gal sinusoidal wave input for Case 2 (with SG-Wall). The parameters are shown in Table 3. Then, using the same parameters, Case 1 (without SG-Wall) were analyzed.

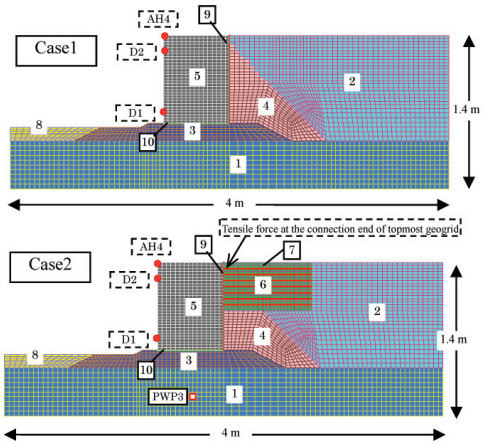

Figure 4. Meshes for Case 1 and Case 2.

The caissons in both cases inclined seaward after shaking, as shown in Figure 5. The angle of inclination in Case 2, however, was approximately half of that in Case 1. This is to say, the SG-Wall method can reduce the rocking behaviour of caisson during earthquake.

The acceleration responses at the top of caisson (AH4) and inclination response of the caisson (D2-D1) obtained from FEM and shake table test are shown in Figure 6 and 7. The calculated acceleration agreed well with the test results. And the gradual seaward-moving tendency of inclination also agreed with the

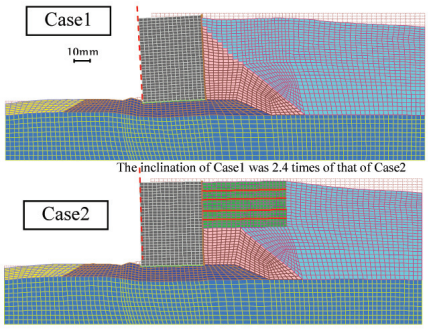

Figure 5. Deformation after shaking.

Table 3. Parameters used in dynamic FEM.

No	Soil name	γ (ton/m^3)	e_0	ν	λ	κ	M^*	a	k (m/sec)	
1	SCP-A	1.9	0.8	0.3	0.040	0.004	1.11	500	1.0×10^{-4}	Sub-Cam-clay
2	SCP-B	1.9	0.8	0.3	0.040	0.004	1.11	500	1.0×10^{-4}	
3	Foundation rubble	2.1	1.2	0.3	0.006	0.002	1.58	800	1.0×10^{-3}	
4	Backfill rubble	2.1	1.2	0.3	0.006	0.002	1.58	800	1.0×10^{-3}	
5	Caisson	2.1	–	0.2	$E = 2.5 \times 10^6$ kPa				–	Elastic
6	Stabilized soil	1.45	1.6	0.4	$E = 2.5 \times 10^4$ kPa, C = 80 kPa, $\phi = 0°$				1.0×10^{-8}	Modified D-P
7	Geogrid	$E = 0.88 \times 10^5$ kPa, $A = 3.8 \times 10^{-4}$ m^2, $I = 0.45 \times 10^{-10}$ m^4								Elastic beam
8	Front soil	1.9	0.8	0.3	0.040	0.004	1.11	500	1.0×10^{-4}	Sub-Cam-clay
9	Rear joint	$k_s = 3.38 \times 104^4$ kN/m, $k_n = 1.0 \times 10^8$ kN/m, C = 0 kPa, $\varphi = 15°$								Goodman joint
10	Bottom joint	$k_s = 3.38 \times 10^4$ kN/m, $k_n = 1.0 \times 10^8$ kN/m, C = 0 kPa, $\varphi = 30°$								

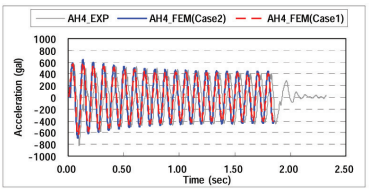

Figure 6. Acceleration response at the top of caisson.

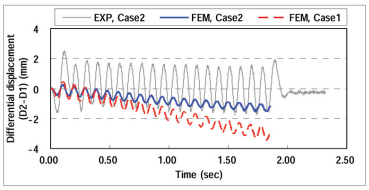

Figure 7. Inclination response of caisson.

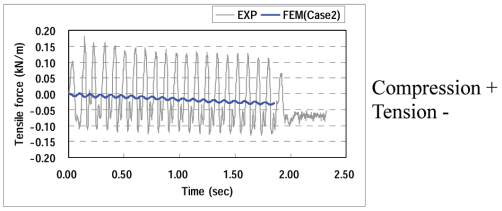

Figure 8. Tensile force at the connection end of topmost.

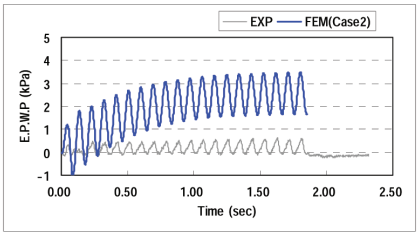

Figure 9. Excess pore water pressure under caisson.

test. Accordingly, it can be said that the parameters is acceptable.

The tensile forces at the connection end of the topmost geogrid are shown in Figure 8. Comparing with the test results of –0.1~0.2 kN/m, the predicted value was much smaller. The difference should be studied in the future.

Figure 9 shows the excess pore water pressures at the ground beneath the caisson. The predicted result was much larger than the observed one. It is due to that the proposed subloading Cam-clay model cannot properly describe the volumetric stationary and stiffness changes during cyclic loading.

4 CONCLUSIONS

In this study, two methods for evaluating the aseismic ability of caisson type SG-Wall were proposed. Their performances were verified by comparing with underwater shake table tests. Some conclusions can be obtained as following.

(1) The proposed simple method can estimate the residual displacement of caisson using a group of identical parameters.
(2) The dynamic effective stress FEM (DG-PILE2D) revealed that the SG-Wall method can reduce the rocking of caisson effectively. However, it cannot correctly predict the tensile force of geogrid and the excess pore water of the ground. This is due to that the proposed subloading Camclay model cannot properly describe the volumetric stationary and stiffness change during cyclic loading, despite it can take into consideration the density dependency
(3) Further study should focus on the improvement of the constitutive model.
(4) Although, this study is a preliminary effort for evaluating the seismic stability for SG-Wall, the methods will be useful for building the design method of SG-Wall as well as other marine structures that utilize geosynthetics.

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