A comparative study on response spectrum analysis and dynamic analysis for seismic design of pile-supported wharf

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ABSTRACT
Generally, for the response spectrum analysis for the seismic design of a pile-supported wharf, it is suggested that the virtual fixed point be utilized. The virtual fixed point is known to be useful for static-behavior evaluation, but it is not certain that the results of the response spectrum analysis utilizing the virtual fixed point are reliable. In this study, the result of the response spectrum analysis of a pile-supported wharf considering the virtual fixed point and of the dynamic analysis results not considering the virtual fixed point were compared to improve the response spectrum design method. A commercial software, MIDAS, was used for the analysis of the response spectrum, and a 3D FEM software, PLAXIS 3D was used for the dynamic analysis. The results of the response spectrum analysis of a pile-supported wharf considering the virtual fixed point and of the dynamic analysis of an actual structure differed depending on the type and depth of the seismic waves. This is because the response spectrum analysis considering the virtual fixed point does not reflect the fact that the end condition is different from the actual structure, and does not consider the ground under the virtual fixed point.

1. INTRODUCTION

Recently, due to the frequent earthquakes occurring throughout the world, the need for a reasonable seismic design method for the harbor structure is on the rise. In the case of the existing port structures, the ASD (allowable stress design) method or the USD (ultimate strength design) method have been uniformly applied. Lately, however, the performance-based seismic design method, which allows flexible design, has been applied. The performance-based design method presents a number of performance requirements and uses a variety of design methods as a method of performing the rational design. It is gradually being applied to the pile-supported wharf, one of the typical port structures. In PIANC (2001) by the International Navigation Association, four performance levels are presented, while PARI (2009) in Japan and ASCE and COPRI (2014) in the U.S. present three. In Korea, only two performance levels are still being applied to the design.

In the case of the pile-supported wharf, which is known for its excellent seismic-performance level, structures are designed according to the above performance level, and the 3D FEM (finite-element method) numerical analysis technique is also applied for seismic-performance evaluation as one of the seismic performance evaluation methods.

Donahue et al. (2005) compared the results of the numerical simulation of the SMD (strong-motion data) seismic wave with actual measurement data to investigate the interaction between the ground and the pile-supported wharf. The interaction between the pile and the ground was simulated by a spring, and the spring coefficient of the ground was calculated via pushover analysis. Shafieezedeh et al. (2012) evaluated the seismic vulnerability using 3D FEM for the worn-out pile-supported wharf designed from the 1960s to the 1970s. Similarly, Doran et al. (2015) also performed seismic-performance evaluation using 3D FEM for a worn-out pile-supported wharf where frequent earthquakes had occurred.

In most of the previous related studies, the seismic performance of the pile-supported wharf was evaluated via the pushover and time history analyses of the 3D modeling finite-element analysis. However, PIANC (2001) and Seismic Design Standard of Port and Harbor (1999) currently propose a simple response spectrum analysis method as well as dynamic analysis for the pile-supported wharf. There has been limited research worldwide, however, on the response spectrum analysis method for the seismic design of a pile-supported wharf.

Using the response spectrum analysis, the maximum response can be obtained by applying a response spectrum load to the structure. In addition, the method determines the acceleration response value by connecting the maximum response using the mode combination method. The mode combination method that is mainly used in the response spectrum analysis is the CQC (complete quadratic combination) method. Unlike the time history analysis method, it can be a relatively simple way to understand the dynamic characteristics of the structure. Generally, it is proposed that a structure be modeled by applying the virtual fixed point in the response spectrum analysis of a pile-supported wharf. The virtual fixed point is known to be a useful method of static-behavior evaluation(PIANC, 2001). It is not certain, however, that the results of the response spectrum analysis utilizing the virtual fixed point are reliable.

Therefore, the purpose of this study is to find problems by comparing the response spectrum analysis considering the virtual fixed point and the 3D dynamic analysis not considering the virtual fixed point in the seismic design of the pile-supported wharf.

2. DETERMINATION OF THE VIRTUAL FIXED POINT

The procedure for the determination of the virtual fixed point is as follows. First, a virtual surface, which is a line connecting the point corresponding to 1/2 of the slope inclination and the pile, is defined. Next, the virtual fixed point is located at the 1/ß point under the virtual surface, and the pile length below the virtual fixed point is ignored while the pile with fixed ends is designed. Here, the ß value is a characteristic value of the pile calculated using Eq.1 and Eq.2

$$ß = \sqrt[4]{\frac{K_h D}{4EI}}\,(\mathrm{cm}^{-1}) \qquad [1]$$

$$k_h = 0.15N\ (\mathrm{kgf/cm^3}) \qquad [2]$$

The N value is the average N value from SPT up to 1/ß from the virtual surface, K_h is the coefficient of the horizontal subgrade reaction, D is the diameter or width of the pile, and EI is the flexural rigidity of the pile. The method of determining the virtual fixed point is shown in Figure 1.

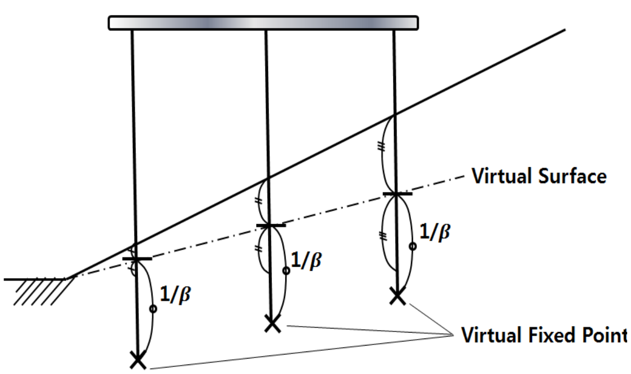


Figure 1. Determining the virtual fixed point

3. ANALYSIS METHODS AND CONDITIONS

For the response spectrum analysis and the dynamic analysis using 3D numerical analysis, the analysis model (3x3 piles) from the wharf construction site located in Pohang, Korea was selected. The distance between the piles was 5 m wide and 6 m high. In the case of the response spectrum analysis model, the structure was modeled considering the virtual fixed point, and the virtual fixed point became shorter as it went from pile 1 to pile 3, as shown in Figure 5(a). In the case of the dynamic analysis model, the entire length of the pile was modeled. To simplify the analysis model and ground properties, the ground was divided into the soil layer and the bedrock layer. The properties used in the analysis are shown in Table 1.

To carry out seismic analysis, three seismic waves were applied to the analysis. These were the long-period seismic wave (the Hachinohe earthquake), the short-period seismic wave (the Ofunato earthquake), and the artificial earthquake that reflected the characteristics of the site in Korea. According to the performance level, it is divided into the collapse prevention level and the limited serviceability level in South Korea. In this study, the collapse prevention level was applied, and the average returning period cycle was determined to be 1,000 years. Figure 2 shows the regional acceleration factor for the 1,000-year earthquake ground motion proposed in Port and Harbor Design Standard (2014) in Korea. The maximum acceleration of the seismic waves in Pohang on the earthquake disaster map is 0.13 g, as shown in Figure 2. Therefore, the maximum acceleration of the three seismic waves was adjusted to 0.13 g, as shown in Figure 3 and the response spectrum curves of those earthquakes were shown in Figure 4.

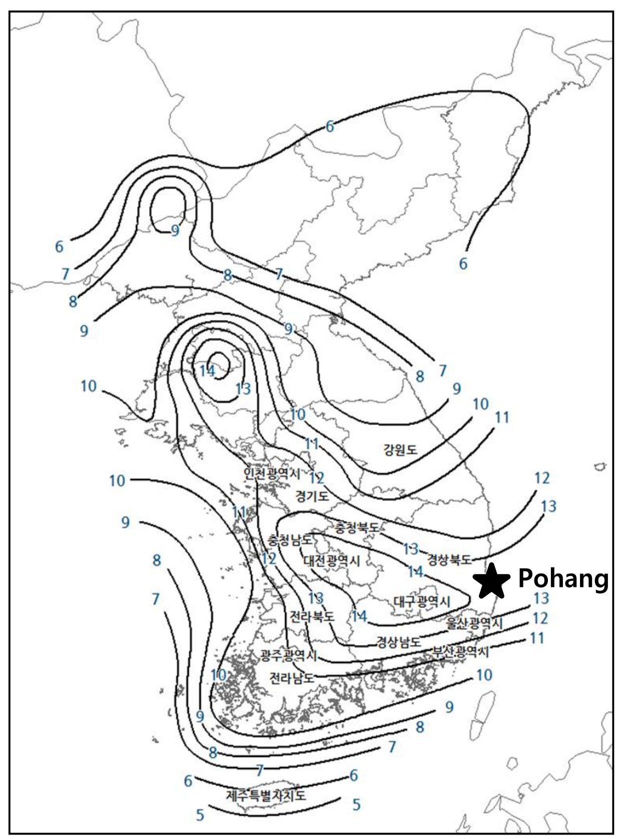
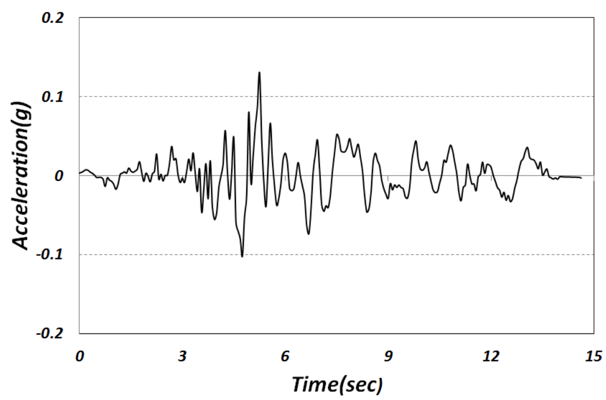


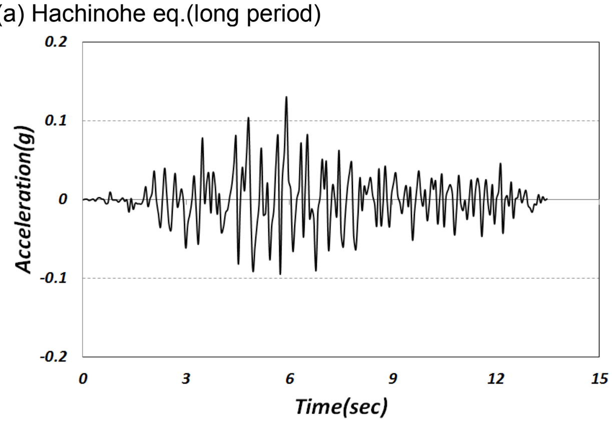
Figure 2. 1,000 years returning period earthquake hazard map (Port and harbor design standard, 2014)

Table 1. Material properties for numerical analysis

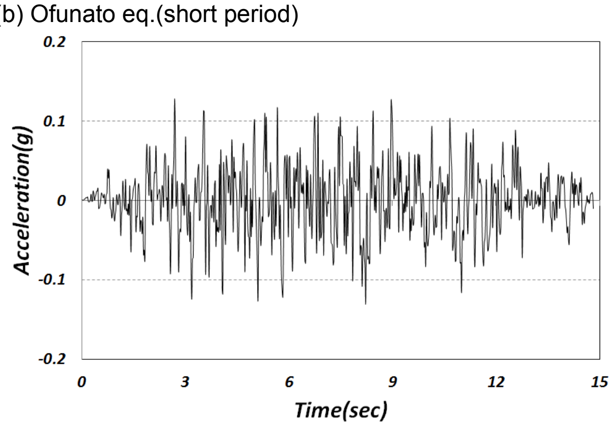
Property	γ_t (kN/m^3)	γ_{sat} (kN/m^3)	E (kN/m^3)	v	V_S (m/s)	c (kN/m^2)	Φ
Soil (sand)	18	20	3.59 x 10^5	0.34	270	0	35
Rock	25	25	7.59 x 10^6	0.23	1100	1000	40
Plate (concrete)	24.5		2.6 x 10^7	0.16		$\mathrm{D = 1m}$	
Pile (steel)	78.5		2.1 x 10^8		$\mathrm{A = 0.03958m^2}$ $\mathrm{D = 0.914m}$	$\mathrm{I = 2.051\ x\ 10^{-3}m^4}$ $\mathrm{t = 0.014m}$	



(a) Hachinohe eq.(long period)

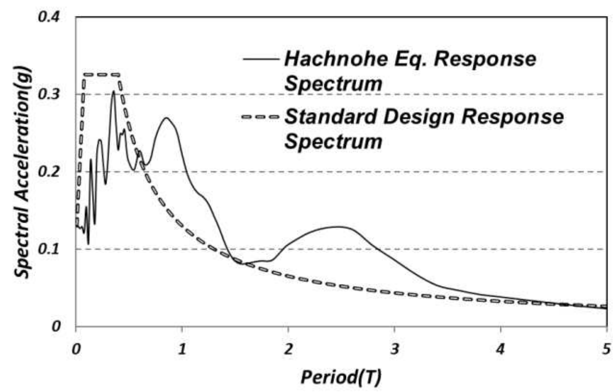


(b) Ofunato eq.(short period)

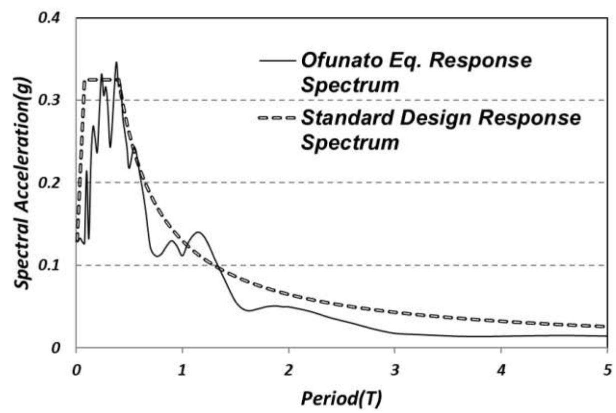


(c) Artificial eq.

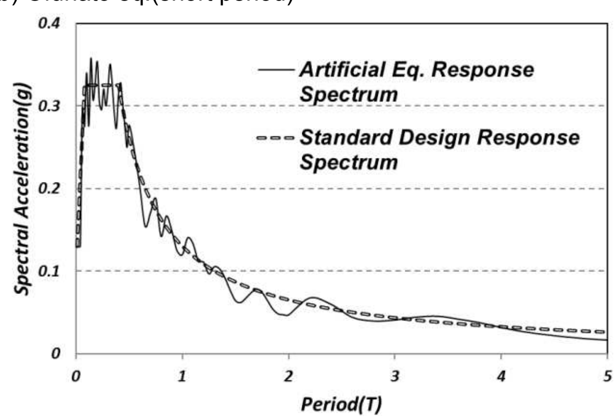
Figure 3. Input accelerations



(a) Hachinohe eq.(long period)



(b) Ofunato eq.(short period)



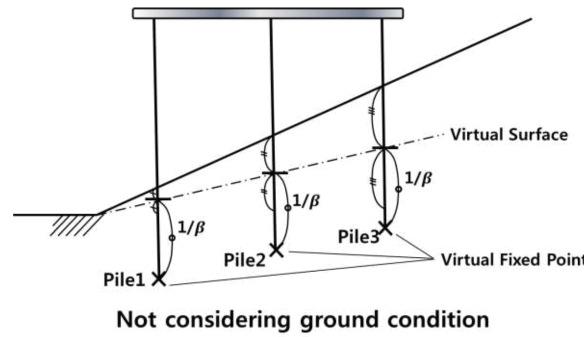
(c) Artificial eq.

Figure 4. Response spectrum curves

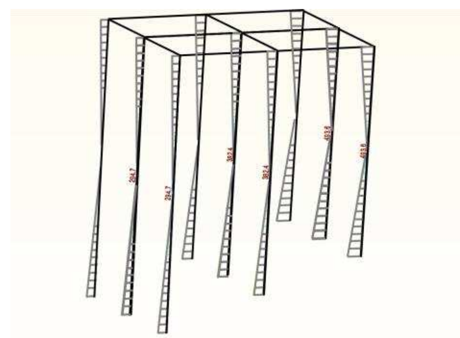
In the case of the response spectrum analysis considering the virtual fixed point, it modeled only the pile structure and not the ground, and ignored the pile length below the virtual fixed point and modeled only the upper pile, as shown in Figure 5. Therefore, in the case of response spectrum analysis, since the ground is not modeled, the ground response analysis should be performed to consider the ground amplification phenomenon during earthquake. Next, the response spectrum analysis is performed using the acceleration value derived from the ground response analysis. As response spectrum analysis is an elastic analysis method, the maximum displacement of the structure is only measured. MIDAS GEN ver.1.4 was used as a numerical analysis program. MIDAS GEN is a 3D FEM program that is widely used for structure analysis. In this study, the response spectrum analysis module provided by MIDAS GEN is utilized.

For the comparison, 3D dynamic analysis of the entire structure was performed using the PLAXIS 3D ver.1.0.0 program, considering the ground. PLAXIS 3D is a 3D FEM program that is widely used in the world today. Jeong and Kim (2013) used this program for pile and soil design, and used to design of the virtual fixed point model of a bent pile structure. Besseling and Lengkeek (2015) used the program to model the interaction between the soil and the pile. Figure 6(a) shows the pile structure of the entire structure of the model not considering the virtual fixed point. The PLAXIS 3D model consists of a 3D triangular mesh, like that shown in Figure 6(b). The adjacent part of the pile and ground can be designed with fine mesh elements to improve the accuracy of the analysis. In this analysis, the soil model that satisfies the Mohr-Coulomb characteristics of the elasto-plastic model and the contact element (boundary strength modulus) provided by PLAXIS 3D were used for the contact surfaces between the piles and the ground, and the slip between the ground and the pile was modeled. In PLAXIS 3D, the contact element was determined using the theories by Goodman et al. (1968) and Van Langen and Vermeer (1991). The analysis region was extended to where there is almost no stress and deformation. Therefore, the boundaries of the model were specified to be 100m apart from the area where the structure was constructed.

Also, as the pile was inserted in the bedrock, the analysis of the entire structure including the ground was performed, and the properties and standard cross-section were the same as in the response spectrum analysis. The groundwater level was designated to be 2 m below the surface, and the residual displacement and maximum displacement were evaluated.

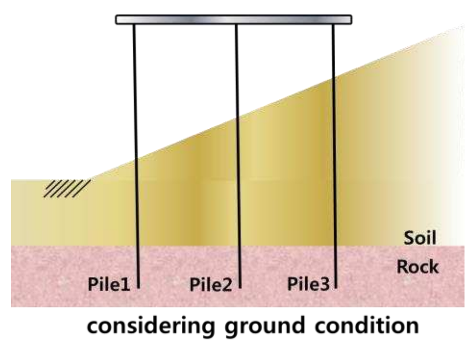


(a) Response spectrum analysis model diagram

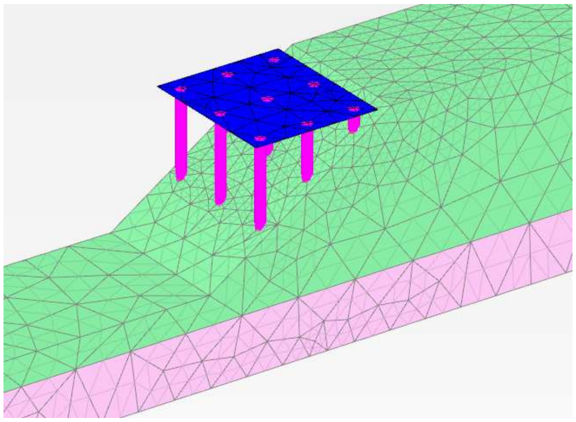


(b) Response spectrum analysis model (MIDAS GEN)

Figure 5. Response spectrum analysis model with the virtual fixed point



(a) Dynamic analysis model diagram



(b) Dynamic analysis model (PLAXIS 3D)

Figure 6. Dynamic analysis model without the virtual fixed point

4. ANALYSIS RESULTS

4.1 Sine Sweep Test

In the case of dynamic analysis model, sine sweep test was conducted to estimate the natural frequency of the soil-pile system. For the analysis, PLAXIS 3D was used. The natural frequencies of the sine waves were changed from 0 to 10 Hz, and the natural frequency of the pile was calculated by comparing the input acceleration with the pile acceleration. As a result, it was found that the resonance of the soil-pile system occurred at around 3.3Hz (natural period: 0.3s) as shown in Figure 8.

To compare the result of sweep test with the spectral acceleration value of the three seismic wave, the spectral acceleration values at 0.3s were calculated from the response spectrum curves of the seismic waves shown in Figure 4. Hachinohe seismic wave (long-period seismic wave) was 0.21g, the Ofunato seismic wave (short-period seismic wave) was 0.29g, and the artificial seismic wave was 0.32g. The resonance phenomenon of artificial seismic waves can be expected to occur the most.

4.2 Ground Response Analysis

Generally, when an earthquake occurs, the seismic wave is transmitted to the soil layer through bedrock, and the response acceleration value generally increases in the soil layer. This phenomenon is called the ground amplification phenomenon. In order to consider the ground amplification phenomenon of the pile supported wharf, the response acceleration value was calculated using the SHAKE program, which is a one-dimensional equivalent linear analysis program, and the calculated response acceleration value was applied to the response spectrum analysis.

Figure 7 shows the ground response analysis results of three seismic waves. These seismic waves were adjusted to the peak acceleration of 0.13g in bedrock. As a result of analysis, the largest ground amplification occurred with 0.28g of artificial seismic wave, 0.27g of Ofunato seismic wave, 0.25g of Hachinohe seismic wave.

4.3 Response Spectrum Analysis vs. Dynamic Analysis

4.3.1 Pile Moment Results

Figure 9(a) shows the moment time history curve of the Hachinohe seismic wave obtained by PLAXIS 3D. As the three seismic features are similar, the Hachinohe seismic wave was only shown in this paper. Comparing Figure 3(a) with Figure 9(a), it can be seen that the moment increased sharply between 4 and 6 seconds when the acceleration of Hachinohe wave became maximum in Figure 3(a). After the 6 seconds, the moment converged to a constant value.

Figure 10(a) shows the maximum moment value of the response spectrum analysis and the dynamic analysis by seismic waves. In the case of the response spectrum analysis, the maximum moment occurred in the pile 3, and in the dynamic analysis, the maximum moment occurred mostly in the pile 1. In the response spectrum model, the moment has the minimum value in the Hachinohe wave and has the maximum value in the Ofunato seismic wave. In the dynamic analysis, the moment has also a minimum value in the Hachinohe wave but has the maximum value in the artificial seismic waves. It is because that the resonance phenomenon of the soil-pile system occurring in artificial seismic waves.

The maximum moments in the response spectrum analysis and the dynamic analysis were compared in Figure 10(a). It can be seen that the difference in moment between the Hachinohe seismic wave and the Ofunato seismic wave is large, and that the artificial wave has the smallest gap. This shows that the moment difference is large according to the resonance phenomena of the soil-pile system.

From the above results, a conservative design can be expected from the use of the response spectrum analysis method because the moment value calculated by response spectrum analysis was generally larger, but it should be emphasized that the difference between the two analysis values varied depending on the resonance phenomena of the soil-pile system.

Next, the moments by depth for the response spectrum analysis and the dynamic analysis were compared. As the results of 3 seismic waves were similar, the moment value of Hachinohe seismic wave was only shown in Figure 11. Figure 11(a) shows the moment values by depth when the Hachinohe wave was applied to the response spectrum analysis, and Figure 11(b) shows the moment by depth when the Hachinohe wave was applied to the dynamic analysis.

In the response spectrum analysis, the moment values changed linearly with the depth. It varied linearly according to the depth and the maximum value was occurred near the virtual fixed point. The minimum moment was generated in pile 1, the longest pile, and the maximum moment was generated in pile 3, the shortest pile. The top deformation, however, was constant due to the constraint on the top deformation. Therefore, it seems that pile 3, which was relatively short, had the maximum moment.

In the case of the dynamic analysis, the moment values changed nonlinearly with the depth, and converged to 0 as the depth increased as shown in Figure 11. The maximum moment was generated in pile 1, and the minimum moment was generated in pile 3. This is because pile 1 is close to the coastal side, and as such, its moment value was the largest.

However, the two models tend to have similar pile moment tendency up to the virtual fixed point, but in the case of the dynamic analysis, the moment of the pile is reversed beyond the virtual fixed point. As the moment value can be maximized under the virtual fixed point, it is necessary to strictly design a pile-supported wharf considering the ground condition under the virtual fixed point.

4.3.2 Pile Displacement Results

Figure 9(b) shows the displacement time history curve of the Hachinohe seismic wave. Comparing Figure 3(a) with Figure 9(b), it can be seen that the displacement increased sharply between 4 and 6 seconds when the acceleration of Hachinohe seismic wave became maximum in Figure 3(a). After the 6 seconds, the displacement converged to a constant value.

Ferritto (1997) considered the residual displacement for the displacement criteria, and the allowable horizontal displacement of the limited serviceability level is up to 10 cm while the collapse prevention level is limited to 30 cm. As the response spectrum analysis that was performed in this study was an elastic analysis method, it was impossible to estimate the residual displacement, and as such, the maximum displacement was considered. In the case of dynamic analysis, both the maximum and residual displacements were evaluated. Figure 10(b) shows that the maximum displacement of the response spectrum analysis is relatively smaller than that of the dynamic analysis. It is because that the response spectrum analysis takes into account the virtual fixed point and therefore the displacement decreases. In the case of the dynamic analysis, the value of the maximum and residual displacements were similar.

Figure 12(a) shows the displacement values by the depth when the Hachinohe wave was applied to the response spectrum analysis. Figure 12(b) shows the displacement values by the depth when the Hachinohe wave was applied to the dynamic analysis. In the case of the response spectrum analysis considering the virtual fixed point, the displacement value decreased linearly with the depth, and the displacement values of the virtual fixed points below each pile were all zero. In the case of the dynamic analysis, a pile also existed below the virtual fixed point, and similarly, the displacement value of the lower region converged to zero. In the case of the response spectrum analysis, the point at which the displacement became zero was 10-14 m. For the dynamic analysis, on the other hand, the point at which the displacement became zero was about 15 m. However, the decrease in displacement per depth seems to have a similar tendency in general.

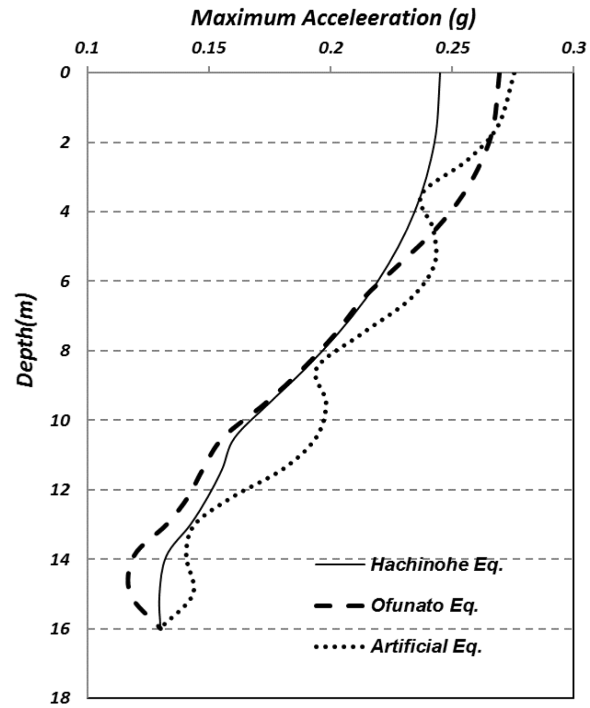


Figure 7. Ground response analysis

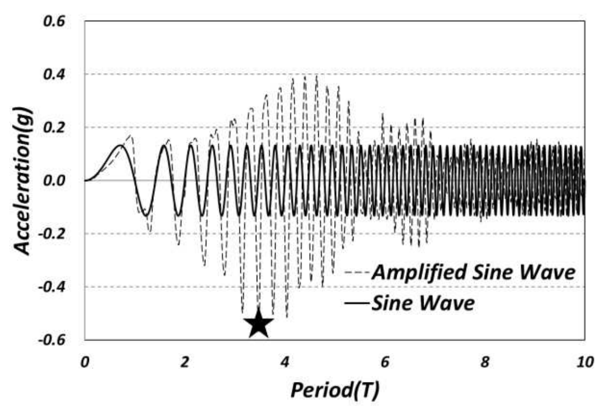
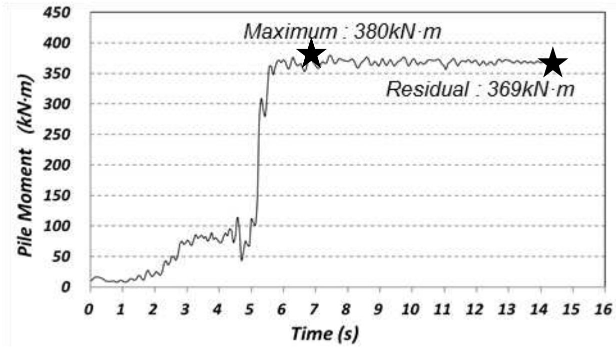
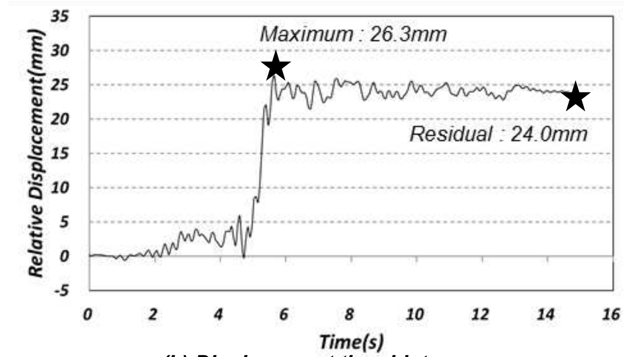


Figure 8. Sine sweep test

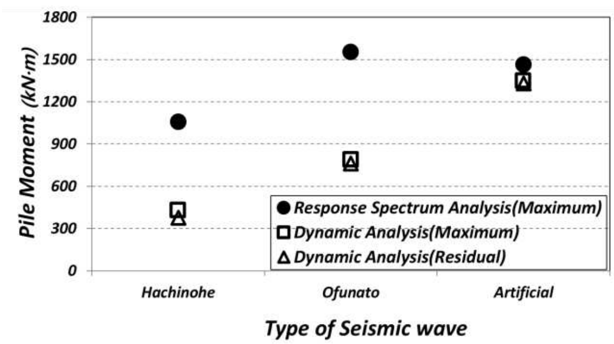


(a) Moment time history curve

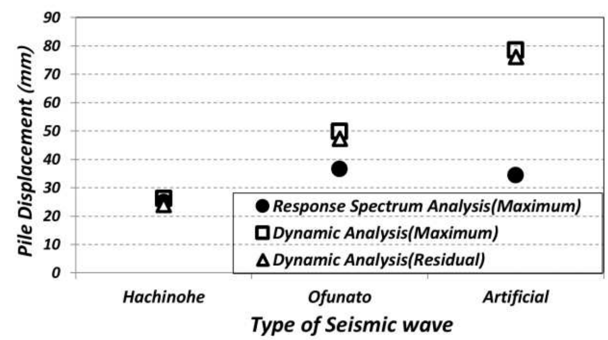


(b) Displacement time history curve

Figure 9. Pile moment and displacement time history curve (Pile1, Hachinohe eq.)

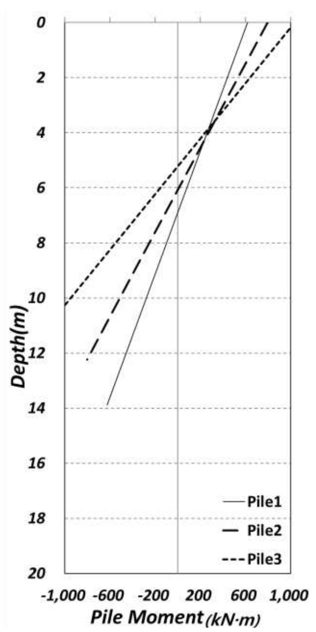


(a) Comparison of pile moment

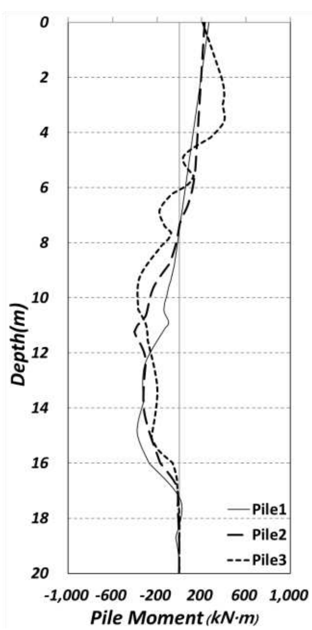


(b) Comparison of pile displacement

Figure 10. Comparison between the response spectrum analysis and the dynamic analysis results

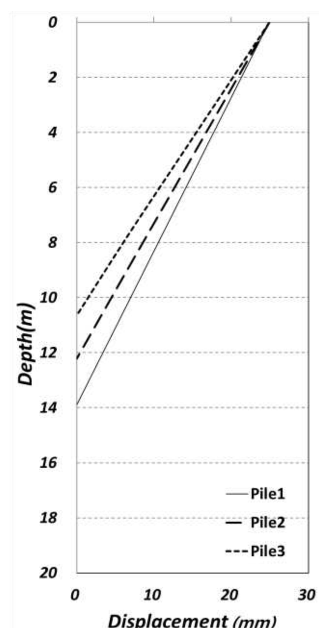


(a) Response spectrum analysis

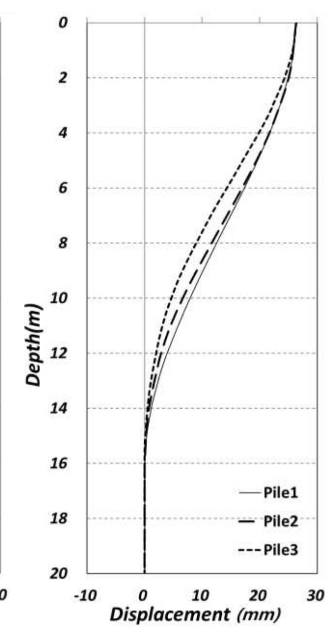


(b) Dynamic analysis

Figure 11. Comparison of pile moment depending on depth (Hachinohe eq.)



(a) Response spectrum analysis



(b) Dynamic analysis

Figure 12. Comparison of pile displacement depending on depth (Hachinohe eq.)

5. CONCLUSIONS

In this study, the differences between two analysis models were derived by comparing the response spectrum analysis results considering the virtual fixed point and the dynamic analysis results not considering the virtual fixed point. The conclusions below were drawn.

(1) A conservative design can be expected from the use of response spectrum analysis because the maximum moment value of the response spectrum analysis model was generally larger. It was seen, however, that the difference between the values obtained from the two analysis models varied depending on the resonance phenomena.
(2) The pile moments by the depth from the response spectrum analysis and the dynamic analysis showed that it is necessary to strictly design a pile-supported wharf considering the ground condition under the virtual fixed point because the moment value can be maximized under the point.
(3) The maximum displacement of the pile from the response spectrum analysis was relatively smaller than that of the dynamic analysis. The moment and displacement tendencies by the depth in the two analysis methods, however, were similar.
(4) In the case of the pile moment, the response spectrum analysis result were larger, and for the displacement, the dynamic model results were larger. The results of this study have limitations, however, because the analyses were performed as a single-stratum study for the simple estimation of the virtual fixed points. Therefore, further research on multi-layered ground analysis is necessary and the analysis results should be verified through dynamic centrifuge model tests or existing case studies.

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