The 2018 World Congress on
Advances in Civil, Environmental, & Materials Research (ACEM18)
Songdo Convensia, Incheon, Korea, August 27 - 31, 2018

Keynote Paper

Safety Assessment of Coastal Infrastructure via Vibration Monitoring and Simplified Model

*Jeong-Tae Kim[1)], So-Young Lee[2)] and Thanh-Canh Huynh[3)]

[1),2),3)]*Department of Ocean Engineering, Pukyong National University, 45 Yongso-ro, Nam-gu, Busan, Korea*
[1)] *idis@pknu.ac.kr*

ABSTRACT

In this study, safety of upright breakwater system is assessed by using in-situ measurements and a simplified mass-spring-dashpot model allowing a few degrees of freedom. Firstly, a vibration-based method is designed to monitor dynamic characteristics and to identify a simplified analytical model from the limited in-situ measurement condition of the caisson breakwater. Secondly, acceleration signals recorded from in-situ tests on a real upright breakwater are analyzed to acquire modal responses under water level change. Thirdly, the structural identification is performed to obtain the simplified model representing the measured dynamic characteristics of the caisson structure. The structural parameters of the caisson unit are estimated from the model fine-tuning process based on the theory of modal sensitivity. Finally, the simplified model is utilized to predict vibration responses of the target caisson system under water level variation.

1. INTRODUCTION

The upright breakwater is inevitably damaged in the lifetime since the repeated wave action causes local and global variations of geometric and boundary properties, being deviated from the as-built state. Recently, the structural safety of the caisson breakwater has become more important issue due to the extreme event like tsunami attack (Oumeraci et al. 2001). The vibration-based structural health monitoring (SHM) has been adopted as a promising way to ensure the as-built design and to estimate the structural integrity of civil infrastructure. Focused on coastal structures, researchers have attempted to estimate the structural performance of gravity-type breakwaters using vibration responses (Gao et al. 1988, Lamberti and Martinelli 1998, Huynh et al. 2013, Lee et al. 2015). Despite of the above-mentioned works, so far, just a little

---

[1)] Professor
[2)] Post-doctoral researcher
[3)] Post-doctoral researcher

attention has been made on the application of the vibration-based SHM techniques to breakwater systems. That might be due to multiple reasons such as the limitation of geometric accessibility, the requirement of excitation source, and the difficulty of vibration signal analysis due to ambient wave-induced noises. So there still remains: (1) to practically monitor vibration responses under the limited condition (e.g., a few sensors or wave-induced ambient excitation), (2) to accurately estimate dynamic characteristics from the vibration measurement data, and (3) to assess structural integrity and performance by the vibration measurement-oriented simplified model.

2. VIBRATION-BASED STRUCTURAL IDENTIFICATION METHOD

To assess the safety of an existing upright breakwater, we should have a reliable representation of its as-of-now status which may include the design, the loading history, and the response data. The vibration-based structural identification of upright breakwater is illustrated in Fig. 1. The scheme includes majorly two phases: firstly, vibration responses are measured and dynamic characteristics are extracted from the target structure (phase A); and secondly, an analytical model which has equivalent dynamic properties is numerically identified to represent the target structure (phase B). A vibration monitoring approach is designed to experimentally acquire dynamic characteristics of the caisson breakwater system. The vibration monitoring scheme is shown in Fig. 2.

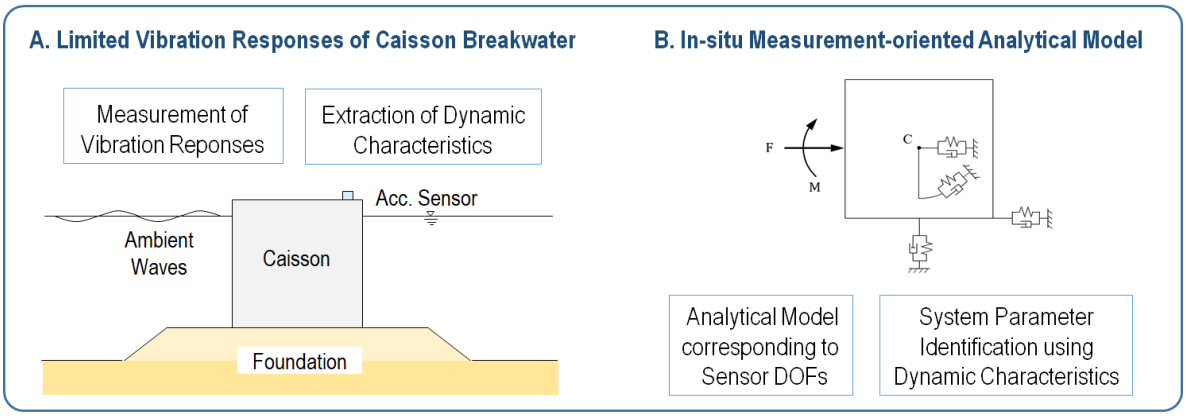


Fig. 1 Schematic of vibration-based structural identification of upright breakwater

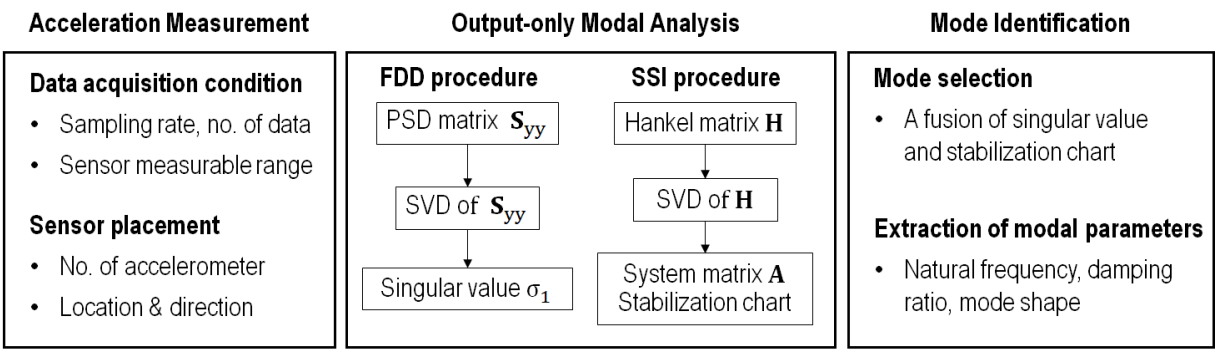
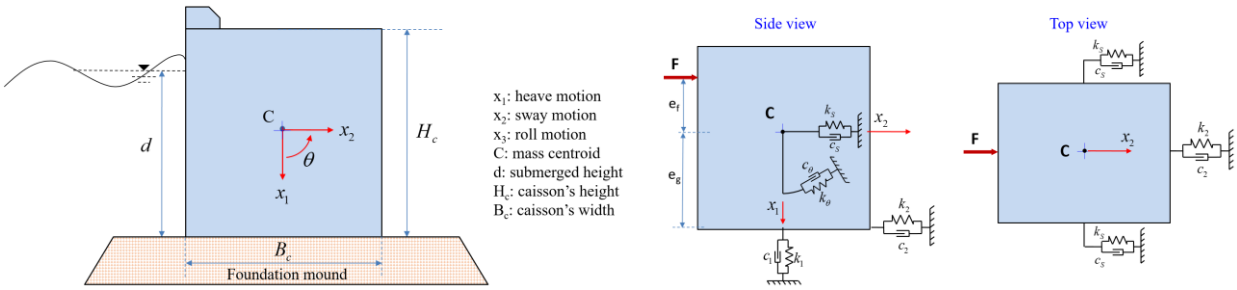


Fig. 2 Vibration monitoring and modal analysis scheme for caisson breakwater

The analytical model is designed to represent the target upright breakwater based on the in-situ vibration measurement. The measurement-based analytical model is called as "simplified model". Vibration responses of the breakwater can be limitedly monitored via a few sensors placed on top of the caisson. A simplified model of the caisson breakwater can be designed by considering the in-plane motion normal to the axis of caisson array, as shown in Fig. 3. The out-of-plane motions of the caisson are neglected since the wave action is basically perpendicular to the axis of caisson array. The caisson is considered as rigid body since its deformation is much smaller than the deformation of the surrounding water and foundation.



(a) Caisson-foundation system with 3 DOFs (b) Analytical mass-spring model

Fig. 3 Simplified model of caisson breakwater with limited vibration data

As shown in Fig. 3(a), an analytical mass-spring model of the caisson-foundation system is designed based on limited degrees-of-freedom (DOFs), which are corresponding to locations and orientations of sensors. The caisson unit has 3 DOFs including heave (x_1), sway (x_2), and roll (θ) around mass centroid. As shown in Fig. 3(b), the analytical mass-spring model of an isolated caisson has been adopted on the basis of a few existing conceptual models (Smirnov and Moroz 1983, Goda 1994, Vink 1997, Huynh *et al.* 2013). In the analytical model, the caisson is treated as a rigid body on elastic half-space foundation which can be represented by springs and dashpots in heave, sway, and roll. In the caisson system, each caisson unit has its own degrees of freedom and linked to the adjacent units.

By equating to the equilibrium condition of the free-body diagram of the breakwater system, the governing equation can be formed in the matrix form, as follows:

$$\begin{bmatrix} m_1 & 0 & 0 \\ 0 & m_2 & 0 \\ 0 & 0 & m_\theta \end{bmatrix} \begin{Bmatrix} \ddot{x}_1 \\ \ddot{x}_2 \\ \ddot{x}_\theta \end{Bmatrix} + \begin{bmatrix} c_1 & 0 & 0 \\ 0 & c_2 + 2c_s & -e_g c_2 \\ 0 & -e_g c_2 & c_\theta + e_g^2 c_2 \end{bmatrix} \begin{Bmatrix} \dot{x}_1 \\ \dot{x}_2 \\ \dot{x}_\theta \end{Bmatrix} + \begin{bmatrix} k_1 & 0 & 0 \\ 0 & k_2 + 2k_s & -e_g k_2 \\ 0 & -e_g k_2 & k_\theta + e_g^2 k_2 \end{bmatrix} \begin{Bmatrix} x_1 \\ x_2 \\ x_\theta \end{Bmatrix} = \begin{Bmatrix} F_1(t) \\ F_2(t) \\ M(t) \end{Bmatrix} \quad (1)$$

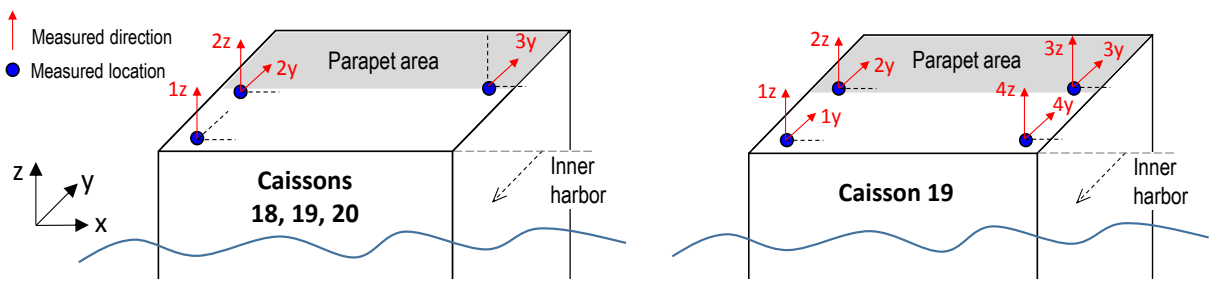
where m_1, m_2, m_θ represents the total masses for the heave, sway, and roll motions, respectively; $F_1(t)$, $F_2(t)$, and $M(t)$ are three components of the impact force F corresponding to the heave, sway, and roll motions; e_f is eccentricity of the impact force F and e_g represents the level of the mass centroid of the caisson from its bottom.

3. VIBRATION MONITORING ON REAL UPRIGHT BREAKWATER

The vibration monitoring scheme, shown in Fig. 4, was evaluated by field tests on a real upright breakwater. The Oryuk-do breakwater is located in Busan, Korea. The breakwater system has a total length of 1,004 *m* consisting of 50 caisson units, as identified in Fig. 4. Among those, three adjacent caisson units #18~#20 were selected for vibration tests under ambient (wave-induced excitation) condition and water level (tide) change. More detailed information of the breakwater can be found in Yi *et al.* (2013).

Fig. 4 The Oryuk-do breakwater of the port of Busan, Korea

The first experiment was carried out for the three caisson units #18, #19, and #20. In each caisson unit, four accelerometers were installed at three sensor locations shown in Fig. 5(a). As listed in Table 1, Sample A was recorded in March 3, 2011 for one hour with the sampling frequency of 50 Hz. The incident wave height and period were not recorded at the time of the experiment.



(a) Experiment in 2011 (b) Experiment in 2014

Fig. 5 Sensor layouts on target caissons

Table 1. Six experimental samples recorded from the Oryuk-do breakwater

Sample	Date	Caisson Units	Water Level (Tide)
A	03-03-2011	18, 19, 20	22.40 *m*
B	04-16-2014	19	23.43 *m*
C	04-16-2014	19	23.22 *m*
D	04-16-2014	19	22.85 *m*
E	04-16-2014	19	22.54 *m*
F	04-16-2014	19	22.19 *m*

For Sample A, vibration signals were measured from Caisson Units #18, #19 and #20. Figure 6 shows vibration responses of Caisson #18 measured by accelerometers 2y (*Acc. 2y*) and 2z (*Acc. 2z*). The proposed modal analysis scheme was implemented into the real caisson breakwater. By adopting the fusion of the frequency-domain FDD method and the time-domain SSI method, as shown in Fig. 2. Figure 7(a) shows the singular values decomposed from the PSDs of Caisson #18 from Sample A. The singular value chart obtained from the acceleration signals indicates the first and second resonance frequencies at 1.5137 Hz and 2.6733 Hz, respectively. Next, the SSI method extracts modal parameters by using stabilization charts, as shown in Fig. 7(b). The stabilization chart obtained from acceleration signals indicates the first and second resonance frequencies at 1.4969 Hz and 2.6448 Hz, respectively.

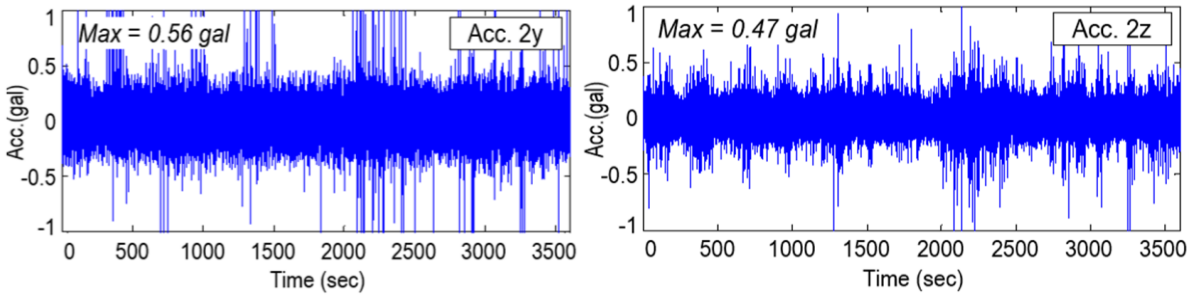
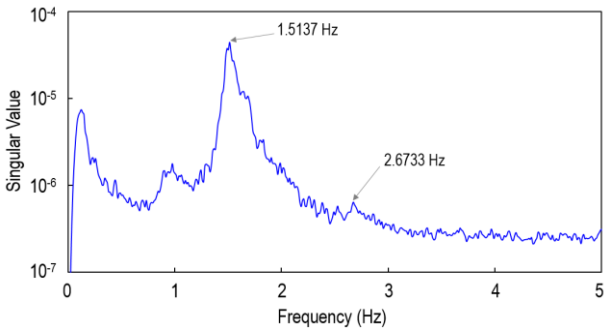
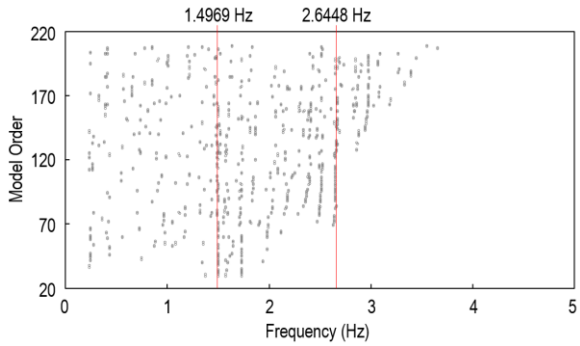


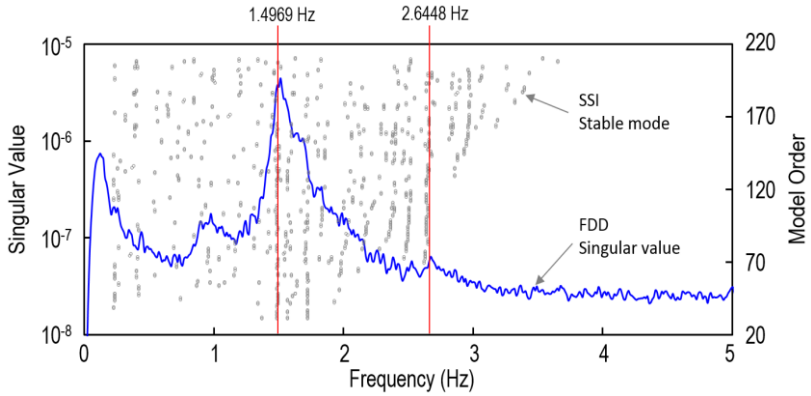
Fig. 6 Acceleration signals of Sample A for Caisson #18 (in 2011)



(a) Singular value chart of FDD method



(b) Stabilization chart of SSI method

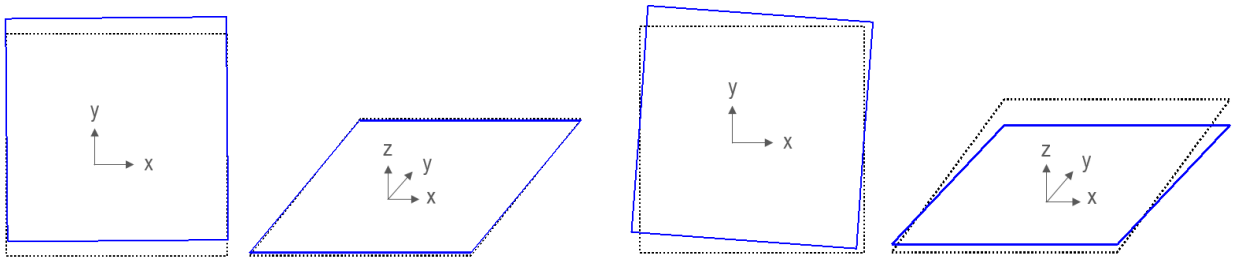


(c) Fusion of FDD and SSI methods

Fig. 7 Modal parameters extracted by the combined FDD and SSI methods: Caisson #18 from Sample A

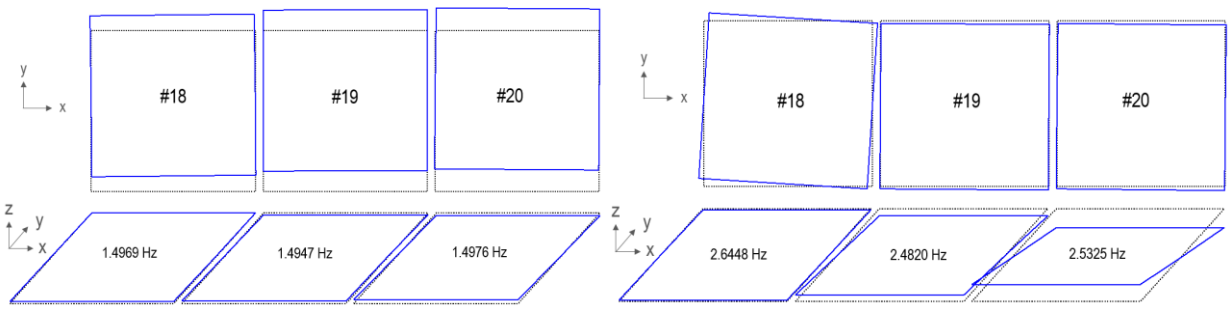
From the combined use of the FDD and SSI methods, modal parameters were extracted for Caisson #18 from Sample A, as shown in Fig. 8. Also, relative modal responses of the three adjacent caisson units were also estimated from Sample A, as shown in Fig. 9. As outlined in Table 2, natural frequencies were extracted by the combined FDD and SSI methods. For the three caissons, the first natural frequency (Mode 1) was around 1.5 Hz while the second one (Mode 2) was about 2.5 Hz.

To form the mode shapes, vibration responses of the caisson array were analyzed at the same time. It is observed that Mode 1 is the sway-dominant motion while Mode 2 is the coupled sway and roll motion. On the sway and roll motions, Caisson #18 was the smallest and Caisson #20 was the largest among the three caissons. On assuming that the three caissons were under the same ambient wave-induced excitation, the modal displacement of Caisson #20 was the largest, which may imply the healthy state of Caisson #20 was worse than other two caissons.



(1) Mode 1 (1.4969 Hz) (2) Mode 2 (2.6448 Hz)

Fig. 8 Mode shapes identified by the combined FDD and SSI methods: Caisson #18 from Sample A



(a) Mode 1 (b) Mode 2

Fig. 9 Experimental mode shapes of 3 consecutive caissons extracted from Sample A

Table 2. Modal parameters of 3 consecutive caissons extracted from Sample A

Modal Parameter		Caisson #18	Caisson #19	Caisson #20
Frequency (Hz)	Mode 1	1.4969	1.4947	1.4976
	Mode 2	2.6448	2.4820	2.5325
Damping Ratio (%)	Mode 1	2.0649	2.4989	2.4336
	Mode 2	2.8902	1.0256	0.4659

4. VIBRATION-BASED STRUCTURAL IDENTIFICATION OF TARGET CAISSON BREAKWATER

Choosing appropriate structural parameters is an important step in the model-updating procedure. All parameters related to structural geometries, material properties, and boundary conditions can be potential choices for adjustment in the model-updating procedure. Physically, the structural parameters which are uncertain in the updating model due to the lack of information on structural properties should be selected. At the same time, the structural parameters which are relatively sensitive to vibration responses should be selected (Huynh *et al.* 2013).

As described previously in Fig. 3, the simplified model of the isolated caisson has three degrees of freedom for which at least three model-updating parameters should be selected. Firstly, the vertical subgrade reaction modulus b_1 was selected to control the vertical motion of the spring k_1 and also the rotational motion of the spring k_θ. Secondly, the horizontal subgrade reaction modulus b_2 was selected to control the horizontal motion of the spring k_2. Finally, the interlocking stiffness k_s was selected to control the relative horizontal motion of the target caisson with respect to its adjacent caisson units.

The geometrical parameters of Caisson 19 were selected as: B_c = 20 m, H_c = 20.78 m, L_c = 20 m, d = 16.32 m, and e_g = 10.39 m. The material properties of Caisson 19 were selected as ρ_{conc} = 2400 kg/m^3, ρ_{sand} = 1800 kg/m^3, ρ_s = 2000 kg/m^3, υ_s = 0.3. The initial values of the sand foundation modulus were selected as $b_1 = 1\times10^7$ N/m^3 for the heave motion, and $b_2 = 0.6\times10^7$ N/m^3 for the sway motion (Bowles 1996, Vink 1997). From those structural and geometrical information, the mass and stiffness parameters of the initial simplified model was computed as: $m_1 = 1.778\times10^7$ kg, $m_2 = 2.357\times10^7$ kg, $m_\theta = 163.351\times10^7$ kg-m^2, $k_1 = 4\times10^9$ N/m, $k_2 = 2.4\times10^9$ N/m, $k_\theta = 133.333\times10^9$ N-m^2/m, and $k_s = 2.4\times10^9$ N/m. Next, the modal sensitivities of the three model-updating parameters with respect to the two identified modes were calculated as plotted in Fig. 10. From the sensitivity analysis, it is found that the vertical subgrade modulus of the foundation was the most sensitive to the two modes, while the interlocking stiffness of that was the least sensitive to these modes. The horizontal subgrade modulus has higher sensitivity to Mode 2 while the other parameters have higher sensitivities to Mode 1.

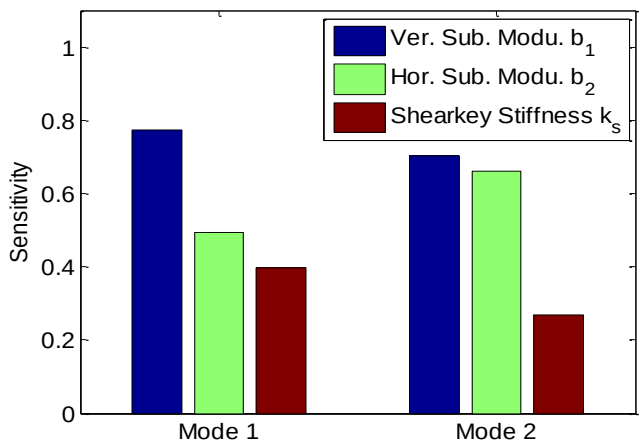


Fig. 10 Modal sensitivities of model-updating parameters

Figure 11 shows the convergence of the two natural frequencies during the six iterations. As compared to the experimental natural frequencies, the frequency difference between the simplified model and the real caisson reduced from 26.73 % to 0.019% for Mode 1 and from 28.90 to 0.022% for Mode 2 after the final iteration. Note that eigenvalue problems were performed to calculate the natural frequencies of the simplified model. Relative changes of the model-updating parameters during the updating process are shown in Fig. 12. The structural parameters at the final iteration were regarded as the baseline parameters. The updated values of model-updating parameters, and stiffness matrix [K] of the final simplified model are listed in Table 3. It is noted that only the stiffness parameters were updated in this study since the mass parameters were assumed to be able to obtain accurately from the as-built design of the caisson.

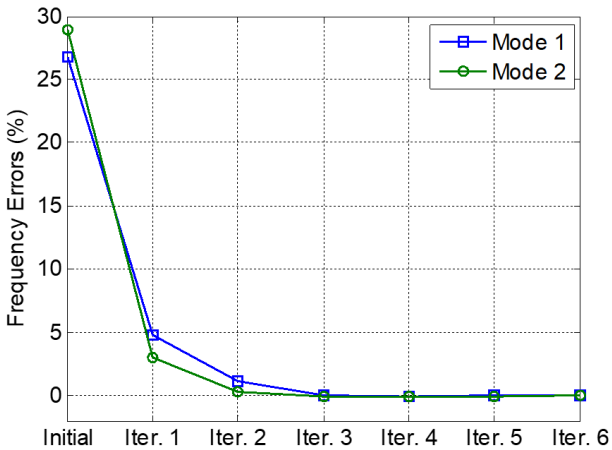


Fig. 11 Convergences of natural frequencies

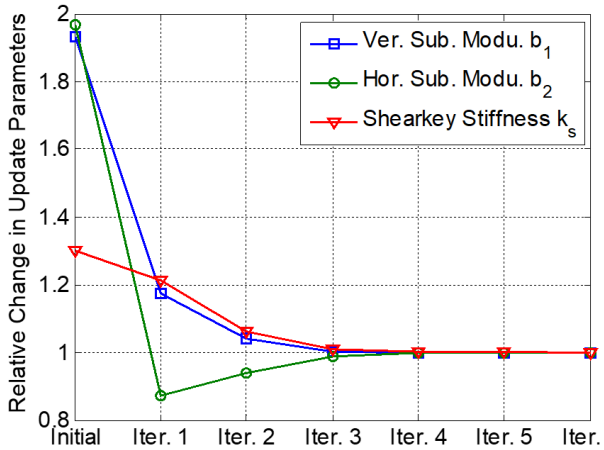


Fig. 12 Changes in model parameters

Table 3 Identified structural parameters of Caisson 19's simplified model

	Initial model	Updated model
Updating parameter	$b_1 = 1.0\times10^7$ N/m^3 $b_2 = 0.6\times10^7$ N/m^3 $k_s = 2.4\times10^9$ N/m	$b_1 = 0.568\times10^7$ N/m^3 $b_2 = 0.341\times10^7$ N/m^3 $k_s = 1.634\times10^9$ N/m
Mass matrix [M]	$10^7\times\begin{bmatrix}1.778 & 0 & 0\\ 0 & 2.357 & 0\\ 0 & 0 & 163.351\end{bmatrix}$	$10^7\times\begin{bmatrix}1.778 & 0 & 0\\ 0 & 2.357 & 0\\ 0 & 0 & 163.351\end{bmatrix}$
Stiffness matrix [K]	$10^9\times\begin{bmatrix}4 & 0 & 0\\ 0 & 7.2 & -24.936\\ 0 & -24.936 & 392.418\end{bmatrix}$	$10^9\times\begin{bmatrix}2.272 & 0 & 0\\ 0 & 4.635 & -14.164\\ 0 & -14.164 & 222.894\end{bmatrix}$

The accuracy of the simplified model identified for Caisson 19 was evaluated as follows. Firstly, a forced vibration analysis was performed on the simplified model to simulate acceleration responses at a few locations corresponding to the field sensors installed on the Oryuk-do breakwater. Modal parameters were extracted from the

acceleration responses of the simplified model. The analyzed modal responses of the simplified model were compared with those of the field test. Secondly, modal responses of the simplified model were simulated for the water level variation, corresponding to the field experiments.

Vibration signals of the simplified model were recorded from the two locations, as shown in Fig. 13, corresponding to the real sensor locations on Caisson 19 in the 2011 experiment. The y and z-directional displacements and accelerations of the two acquisition locations were computed using those of the mass centroid, as shown in Fig. 14. It is noted that the y-directional vibration responses of two locations were close to each other since the caisson was treated as the rigid body.

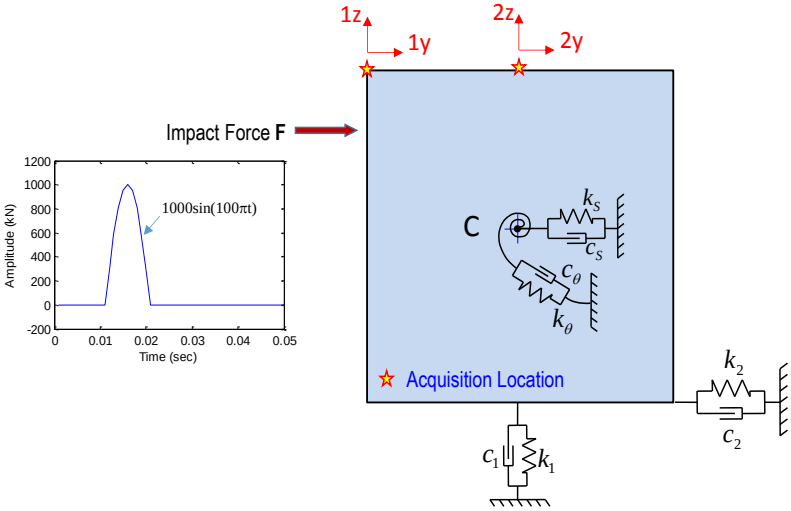
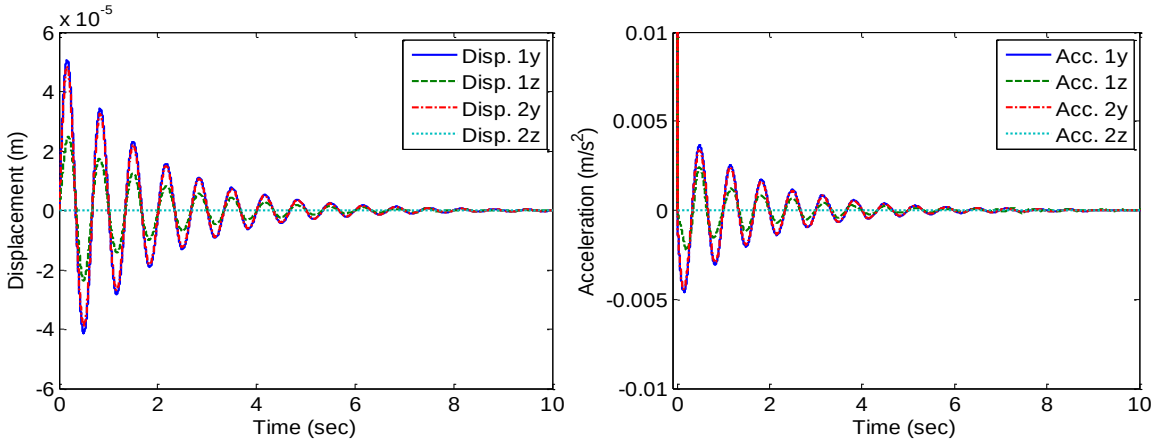


Fig. 13 Setup of forced-vibration test for Caisson 19's simplified model



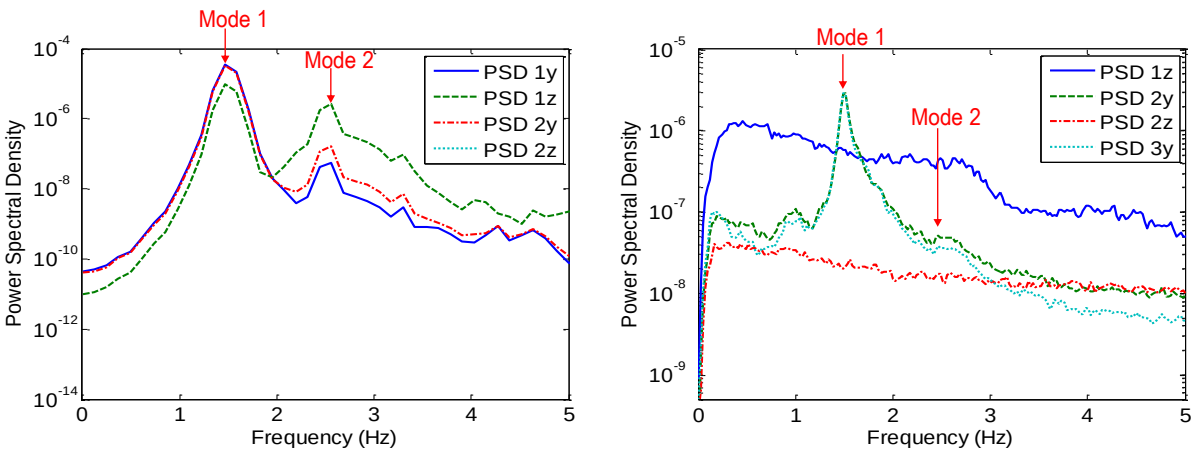
(a) Displacement (b) Acceleration

Fig. 14 Vibration responses at two acquisition locations on simplified model of Caisson 19

As plotted in Fig. 15(a), the PSDs were computed from the four acceleration signals of the simplified model of Caisson 19. In the figure, there exist two clear peaks which indicate two vibration modes of the simplified model. As also plotted in Fig. 15(b), the corresponding PSDs were computed from the experimental acceleration signals of Caisson 19. It is observed that the PSDs obtained from the simplified model have consistent patterns as compared to those obtained from the 2011 ambient test. It is also

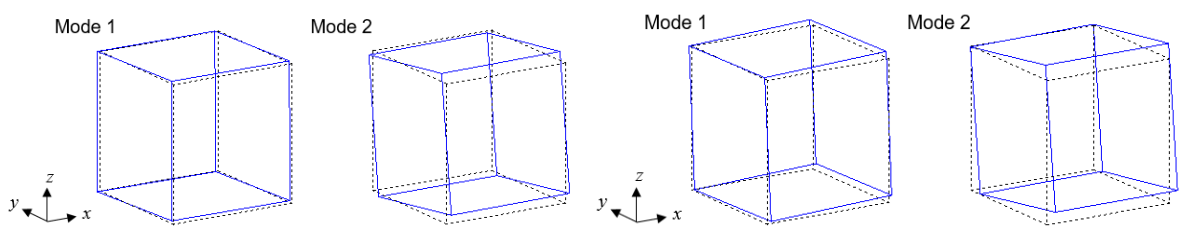
observed that the z-directional PSDs from the simplified model indicate two modes, which were absent from the ones obtained from the experiment. This phenomenon may be caused by the difference in excitation conditions. While the 2011 experiment was performed with the wave-induced ambient excitation, the simplified model was excited by the impact force.

Next, the SSI method was used to extract the modal parameters from the accelerations computed from the simplified model. Figure 16 shows two extracted mode shapes from simplified model, compared to the experimental results. It is found that the mode shapes of two vibration modes of the simplified model were well-matched to those measured from Caisson 19 in 2011. The two extracted natural frequencies of the simplified model were compared to the experimental ones as listed in Table 4. The frequency errors between two models were very small as 0.19 % for Mode 1 and 0.24 % for Mode 2.



(a) Simplified model (b) Experiment in 2011 (Test A)

Fig. 15 Power spectral densities of Caisson 19's simplified model and the 2011 experiment



(a) Simplified model (b) 2011 experiment (Test A)

Fig. 16 Comparison of Caisson 19's mode shapes: simplified model vs 2011 experiment

Table 4 Natural frequencies of Caisson 19's simplified model extracted by SSI method

Mode	Natural Frequencies of Caisson 19		
	Simplified Model	Experiment in 2011	Errors
Mode 1	1.4975 Hz	1.4947 Hz	0.19%
Mode 2	2.4879 Hz	2.4820 Hz	0.24%

5. CONCLUSION

A vibration-based structural identification was performed for a real upright breakwater by using in-situ vibration measurements and a simplified model representing the target structural system. A vibration-based structural identification method was designed for the upright breakwater. Acceleration signals recorded from in-situ tests on a real breakwater were analyzed by the output-only modal analysis method and a few reliable vibration modes of the real breakwater were extracted. The simplified model was utilized to predict vibration responses of the target caisson system.

ACKNOWLEDGEMENT

This research was supported by a grant(18CTAP-C142999-01) from Technology Advancement Research Program (TARP) funded by Ministry of Land, Infrastructure and Transport of Korean government.

REFERENCES

Gao, M., Dai, G. Y., and Yang, J. H. (1988), “Dynamic studies on caisson-type breakwaters”, *Proceedings of 21st Conference on Coastal Engineering*, Torremolinos, Spain, 2469-2478.

Goda, Y. (1994), “Dynamic response of upright breakwater to impulsive force of breaking waves”, *Coastal Engineering*, **22**, 135-158.

Huynh, T.C., Lee, S.Y., Kim, J.T., Park, W.S., and Han, S.H. (2013), “Simplified planar model for damage estimation of interlocked caisson system”, *Smart Structures and Systems*, **12**(3-4), 441-463.

Lamberti, A., and Martinelli, L. (1998) “Prototype measurements of the dynamic response of caisson breakwaters,” *Proc. 26th ICCE, ASCE,* Copenhagen, Denmark.

Lee, S.Y., Huynh, T.C., and Kim, J.T. (2015), “Structural identification of gravity-type caisson structure via vibration feature analysis”, *Smart Structures and Systems,* **15**(2), 259-281.

Oumeraci, H., Kortenhaus, H., Allsop, W., De Groot, M., Crouch, R., Vrijling, H., and Voortman, H. (2001), *Probabilistic design tools for vertical breakwaters*, Swets & Zeitlinger B.V., Lisse, Netherlands.

Smirnov, G.N., and Moroz, L.R. (1983), “Oscillations of gravity protective structures of a vertical wall type”, *IAHR Proceedings of 20th Congress*, Moscow, 216-219.

Stubbs, N., and Kim, J.T. (1996), “Damage localization in structures without baseline modal parameters”, *AIAA Journal*, **34**(8), 1644-1649.

Vink, H.A.Th. (1997), Wave Impacts on Vertical Breakwaters, *Master’s thesis*, Faculty of Civil Engineering, Delft University of Technology, The Netherlands.

Yi, J.H., Park, W.S., Lee, S.Y., Huynh, T.C, Kim, J.T., Seo, C.K. (2013), “Evaluation of vibration characteristics of an existing harbor caisson structure using tugboat impact tests and modal analysis”, *International Journal of Distributed Sensor Networks*, **2013**(Article ID 806482), 11pages.