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Structural Performance Estimation of Gravity-type Caisson Structure by Vibration-based System Identification

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ABSTRACT

In this study, the structural performance of the gravity-type caisson structure is assessed from vibration-based system identification. To achieve the objective, the following approaches are implemented. Firstly, a structural identification algorithm based on the use of vibration characteristics is outlined. Structural stiffness and damping are selected as two key parameters to be identified for model update. Secondly, experimental vibration analysis of a lab-scaled gravity-type caisson structure is described. Vibration tests on 2-D wave flume are performed for a few scenarios such as water-level changes and foundation damage. Experimental modal parameters are extracted by frequency domain decomposition method. Finally, vibration-based structural integrity is assessed for the tested caisson system. A simplified model is formulated to represent the tested structure with the limited degrees-of-freedom that corresponds to the sensors' degrees-of-freedom. Structural parameters of the simplified model are identified by tuning the natural frequencies and modal damping factors.

1. INTRODUCTION

Recent years, the structural integrity of harbor caisson structures becomes more important issue due to severe environmental phenomena and extreme events like typhoon or ship collision. Damage in gravity-type caisson structures is inevitable due to local failures or global instability problems which are mostly attributed to foundation-structure interface (Oumeraci, 1994; Lee et al., 2009). For example, the sliding at the foundation-structure interface is the primary damage that leads the overall global failure. Also, the cavity in foundation mound is the typical damage that leads the local failure

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which makes harbor caissons weaker against extreme loads such as storm waves (Goda, 1994; Takahashi et al., 2000).

The overall failures such as sliding, overturning and settlement affect to structural stability; meanwhile, the local failures cause unexpected water flow and reduction of load-carrying capacity. Noting that the progress of the local failures can be lead to the overall failures, all damages of vertical breakwaters should be assessed to secure the structural integrity of the caisson system.

Many researchers have studied vibration-based structural health monitoring techniques in the field of civil engineering (Adams et al., 1978; Stubbs and Osegueda, 1990; Li et al., 2004; Jang et al., 2012). Also, many researchers have worked on developing structural identification methods such as the modal sensitivity method, modal flexibility method, genetic algorithm, neural network, etc. (Aktan et al., 1997; Kim et al., 2003; Yun et al., 2009). Despite those research efforts, only a few research efforts have been made to gravity-type caisson structures, including vibration response analyses of soil-structure or fluid-soil-structure interactions (Yang et al., 2001; Kim et al., 2005; Lee et al., 2013; Yi et al., 2013). For gravity-type caisson structures, several key research needs are existed to analyze their vibration characteristics via the limited accessibility and to estimate the structural performance via the change in the vibration characteristics which is sensitive to the occurrence in local and global damage in the system.

In this study, the structural performance of the gravity-type caisson structure is assessed from vibration-based system identification. To achieve the objective, the following approaches are implemented. Firstly, a structural identification algorithm based on the use of vibration characteristics is outlined. Structural stiffness and damping are selected as two key parameters to be identified for model update. Secondly, experimental vibration analysis of a lab-scaled gravity-type caisson structure is described. Vibration tests on 2-D wave flume are performed for a few scenarios such as water-level changes and foundation damage. Experimental modal parameters are extracted by frequency domain decomposition method. Finally, vibration-based structural integrity is assessed for the tested caisson system. A simplified model is formulated to represent the tested structure with the limited degrees-of-freedom that corresponds to the sensors' degrees-of-freedom. Structural parameters of the simplified model are identified by tuning the natural frequencies and modal damping factors.

2. VIBRATION-BASED STRUCTURAL IDENTIFICATION ALGORITHM

Vibration Response

Dynamic motion of a structure can be defined in terms of displacement, velocity, and acceleration responses. The acceleration response depends on structural dynamic characteristics such as mass, stiffness, and damping which can be defined as

$$\ddot{u}_t = [\mathrm{M}]^{-1}\left(\{F\} - \dot{u}_t[\mathrm{C}] - u_t[\mathrm{K}]\right) \quad (1)$$

where u_t, $\dot{u}_t$, and $\ddot{u}_t$ represent the displacement, velocity, and acceleration vectors, respectively; [M], [K] and [C] represent the mass matrix, stiffness matrix, and damping matrix, respectively; and $\{F\}$ is the vector of external force. The acceleration response provides an understanding of the dynamic characteristics that represent the structural behaviors. Accelerometer is utilized to measure acceleration responses of a structure. The most widespread type of accelerometers is the piezoelectric accelerometer. For this accelerometer, the piezoelectric elements act as a spring and connect the base of the accelerometer to a seismic mass. When an input is introduced to the base of the accelerometer, a force is created on piezoelectric material which is proportional to the applied acceleration and the size of the seismic mass. The piezoelectric accelerometer provides high-fidelity acceleration due to its high-sensitivity.

For the forced response of a damped structural system, the general receptance FRF can be simplified in a complex form as follows:

$$\left[\Omega(\omega)\right] = \left[\mathrm{K} + i\omega\mathrm{C} - \omega^2\mathrm{M}\right]^{-1} \tag{2}$$

in which the receptance $\Omega(\omega)$ is defined as force to response ratio in frequency domain, so that Eq. (1) can be interpreted as Eq. (2). Assuming the damping matrix is proportional to the mass matrix, the proportional damping matrix should be the following form:

$$\left[\mathrm{C}\right] = \beta\left[\mathrm{K}\right] + \gamma\left[\mathrm{M}\right] \tag{3}$$

in which β and γ are the stiffness and mass damping coefficients. Due to the orthogonality conditions of mass and stiffness matrices, the damped system have eigenvalues as follows:

$$\bar{\omega}_i^2 = \omega_i\sqrt{1-\xi_i^2} \tag{4a}$$

$$\xi_i = \beta\omega_i / 2 + \gamma / 2\omega_i \tag{4b}$$

where ξ_i is the critical-damping ratio; ω_i is the undamped natural frequency; and $\bar{\omega}_i$ is the damped natural frequency. If the damping ratios (e.g., ξ_i and ξ_j) corresponding to two specific undamped natural frequencies (e.g., ω_i and ω_j) are known, the two Rayleigh damping factors (i.e., β and γ) can be evaluated.

<u>Structural Identification</u>

Stubbs and Kim (1996) proposed a system identification method to identify a realistic theoretical model of a structure by using vibration modal parameters. Suppose p_j^* is an unknown structural parameter (i.e., mass, stiffness, or damping parameters) of the j^{th} member of a structure for which modal parameters (i.e., natural frequencies and damping coefficients) are known for *M* vibration modes. Also, suppose p_j is a corresponding known structural parameter of the j^{th} member of an analytical model for which the corresponding set of modal parameters are known for the same *M* vibration modes.

Relative to the target structure, the structural parameter of the j^{th} member of the analytical model is defined as follows:

$$p_j^* = p_j + \delta p_j \tag{5}$$

in which δp_j is the variation of the structural parameter of the j^{th} member which result in changes of modal parameters. The sensitivity S_{ij} of the i^{th} damped eigenvalue $\bar{\omega}_i^2$ with respect to the j^{th} structural parameter p_j is defined as follows:

$$S_{ij} = \frac{\delta \overline{\omega}_i^2}{\delta p_j} \frac{p_j}{\overline{\omega}_i^2} \quad (6)$$

in which the damped eigenvalue is defined as $\overline{\omega}_i^2 = \omega_i^2 (1 - \xi_i^2)$. Since the sensitivity S_{ij} is related to the combination of the undamped natural frequency and damping coefficient, a simplified approach is needed to analyze the sensitivity with respect to each parameter for the practical purpose.

3. EXPERIMENTAL VIBRATION ANALYSIS OF GRAVITY-TYPE CAISSON STRUCTURE

Vibration Tests on Lab-scale Caisson System

As shown in Fig. 1, a lab-scale caisson model is selected as the target structure. Four main subsystems include caisson body, cap concrete, two adjacent blocks, and foundation mound. The caisson has 34 cm of height and 34 cm of width; it also includes the empty inner space with vertical wall of 5 cm thickness. Height of the cap concrete is maximum 10 cm. Bottom dimension of the concrete is the same as that of the caisson. Due to the limited width of the 2-D wave flume, two adjacent caissons were replaced by adopting concrete blocks which have 6 cm thickness to make the main caisson body fitted into the side wall of the wave flume. Each caisson body has shear keys for interlocking between adjacent caisson bodies.

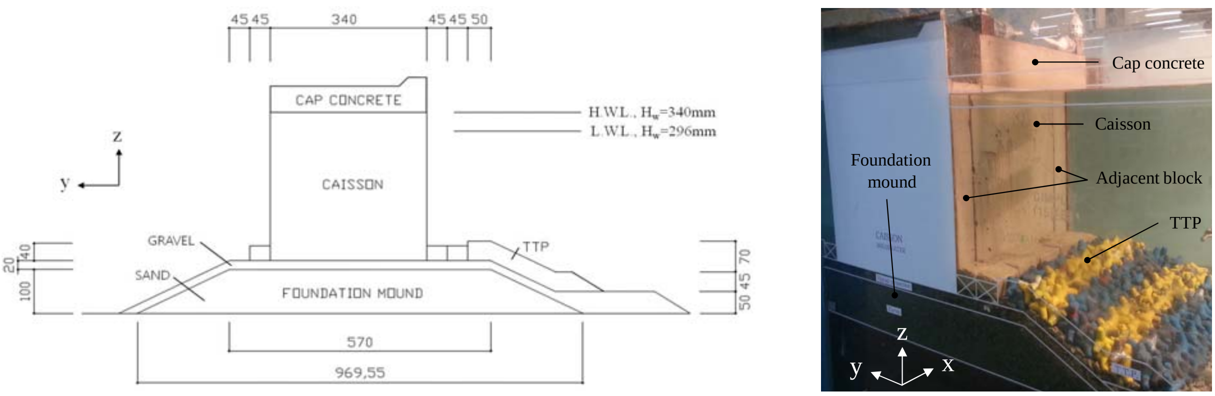


Fig. 1 Test setup in the 2-D wave flume

Vibration responses were measured by a data acquisition system which consists of accelerometers (PCB 393B04 model which has $\pm$ 5g of measureable range and 1 V/g of sensitivity), a signal conditioner, a DAQ card, and a laptop. As shown in Fig. 2, the accelerometers were installed on the cap concrete to measure the vibration response of the caisson.

In the previous study by Lee et al. (2012), mode shapes of the caisson system were extracted from the numerical modal analysis of caisson system. For their mode shapes, deformation of caisson is much smaller than deformation of foundation mound.

According to the previous studies by Ming et al. (1988) and Lee et al. (2012), only first one or two modes could be extracted from the field tests and lab tests for the caisson breakwater. It was also reported that the rigid body motion is sensitive to the variation of foundation integrity (Lee et al., 2012; Huynh et al., 2013). In this study, the rigid body motion is selected as the target motion. Also, the limited accessibility and possibility of extracting modal parameters at the on-site field area was considered for the installation of the sensors for the lab test. To extract the rigid body motion, sensors were installed along with y-and z-axes for four points. Acceleration signals were acquired for 20 seconds at 500Hz of sampling rates. Impact excitations which simulate ship collisions are implemented by using a rubber hammer applied on the front wall of the caisson. The impact was manually applied by a test implementer.

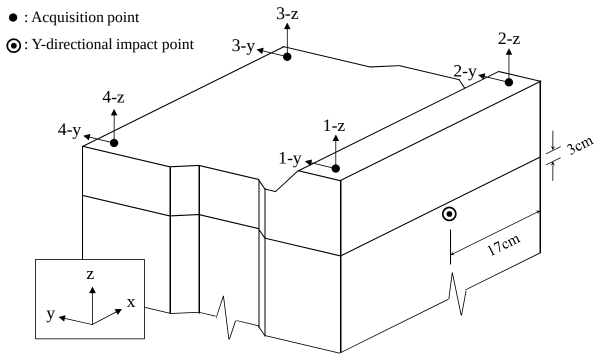
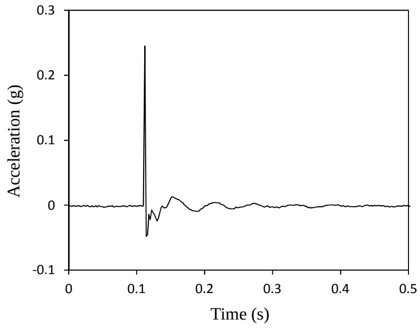


Fig. 2 Orientation of sensors and excitation point

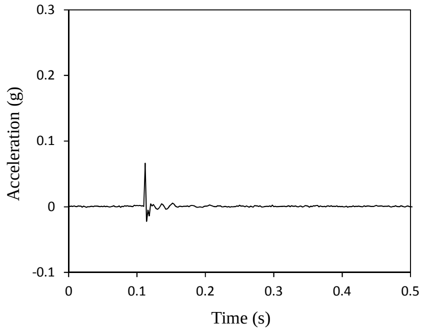
To examine the changes in modal parameters due to water-level variation, two water-levels in 2-D wave flume were designed. Tidal condition at the field area was considered for the designed water-levels. As shown in Fig. 1, two water-levels, high water level (H.W.L.) and low water level (L.W.L) were selected on the basis of tidal conditions in Korea. The variation in depth between water-levels is 44mm.

Next, to examine the changes in modal parameters due to foundation damage, damage was inflicted into the foundation mound. The occurrence of the foundation damage is typically caused by dislocation of wave-dissipating blocks and subsequent scouring induced by repeated extreme wave actions. The damage was simulated by removing part of armor blocks and TTP blocks from the foundation mound and the 18.7% of caisson's bottom area was lost.

Acceleration responses of the test caisson excited by hammer impact were measured by the vibration measurement system. Figure 3 shows the y-directional and z-directional vibration responses measured at the acquisition point 1. Note that the y-direction corresponds to wave incident direction. The maximum y-directional acceleration was about 0.25g with short impulse duration of about 0.004s; meanwhile the maximum z-directional acceleration was less than 0.1g.



(a) Y-direction



(b) Z-direction

Fig. 3 Acceleration responses at acquisition point 1 of test caisson system excited by impact

<u>Modal Parameter of Test Caisson System</u>

The modal parameters of the test caisson system under three different cases are extracted by FDD method. The singular value matrix was extracted as sketched in Fig. 4. The first two modes are extracted for the analysis of modal parameters of the undamaged caisson model. Two modes are observed at about 17 Hz and 42 Hz. Extracted natural frequencies and damping coefficients for the three cases are summarized in Table 1.

From the result of three cases, it is observed that the natural frequencies of the test caisson system are increased by water-level decrease but decreased by occurrence of the foundation damage. It is also observed that the damping ratio of the test caisson system are decreased by water-level decrease but increased by damage occurrence. The amount of the variation of the natural frequencies and damping ratios are larger in mode 1 than in mode 2.

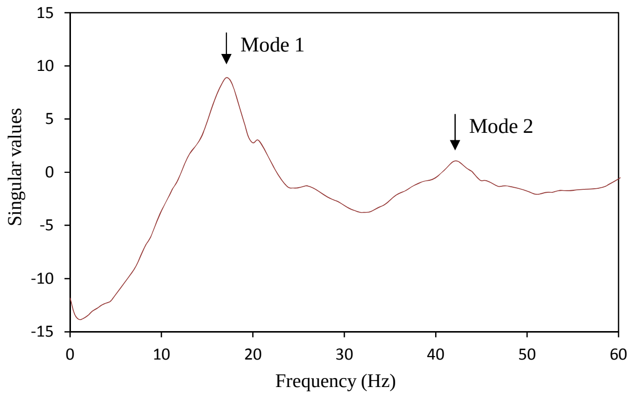


Fig. 4 Singular values of FDD method for undamaged caisson system: “Case: H.W.L. and undamaged”

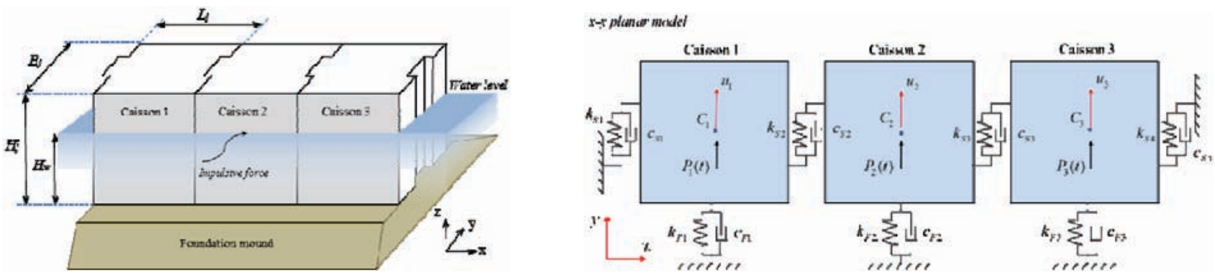
Table 1 Modal parameters of test caisson system for three cases: 1) H.W.L. and undamaged, 2) L.W.L. and undamaged, and 3) H.W.L. and damaged.

Test case	Natural frequency (Hz)		Damping ratio (Hz)	
	Mode 1	Mode 2	Mode 1	Mode 2
H.W.L. and undamaged	17.33	41.99	6.89	2.74
L.W.L. and undamaged	19.78	44.68	5.26	2.64
H.W.L. and damaged	15.39	42.24	12.08	2.89

4. VIBRATION-BASED STRUCTURAL IDENTIFICATION OF LAB-SCALE CAISSON SYSTEM

4.1 Simplified Model of Caisson System

As shown in Fig. 5(a), the caisson system is subjected to an impulsive breaking wave force that results in forced vibration responses. Since the wave action is usually perpendicular to the caisson array axis (i.e., x-direction), the vibration in the impact direction (i.e., y-direction) is relatively larger than other directions (Lee et al., 2011 and 2012; Yoon et al., 2012). Therefore, only the sway motion of caissons (i.e., y-direction) is taken into account in this study. Based on a few existing simplified models (Smirnov and Moroz, 1983; Marinski and Oumeraci, 1992, Goda, 1994; Oumeraci and Kortenhaus, 1994; Vink, 1997), a planar model of three interlocked caissons is proposed as shown in Fig. 5(b). In the simplified model, caissons are treated as rigid bodies on elastic half-space foundations which can be described via the horizontal springs and dashpots. To represent the condition of interlocking mechanism, springs and dashpots are also simulated between adjacent caissons unit.



(a) Caisson system of three units system (b) Simplified model of caisson

Fig. 5 A caisson system of three units and its simplified model

By equating to the equilibrium conditions of the free-body diagrams of caissons (see Fig. 5(b)), the sway motion can be formulated in matrix form as:

$$[\mathrm{M}]\{\ddot{u}\} + [\mathrm{C}]\{\dot{u}\} + [\mathrm{K}]\{u\} = \{\mathrm{F}\} \quad (7)$$

where m_j is the total horizontal mass of the j^{th} caisson; k_{Fj} and c_{Fj} separately represent the horizontal spring and dashpot of the j^{th} caisson's foundation (j=1-3); k_{Fj} and c_{Fj}, respectively, represent the horizontal spring and dashpot of the k^{th} shear-key connection (k=1-4); $\ddot{u}_j, \dot{u}_j$ and u_j are, respectively, the horizontal acceleration, velocity and displacement of the j^{th} caisson; and $P_j(t)$ is the external force placed at the center of gravity of the j^{th} caisson.

When the caisson is oscillated by an impact load, the surrounding media (i.e., soil and water) are forced to move with the structure. Therefore, the total horizontal mass of the j^{th} caisson (m_j) includes not only the mass of the caisson (m^{cai}_j) but also the horizontal hydrodynamic (m^{hyd}_j) and the horizontal geodynamic masses (m^{geo}_j) as follows:

$$m_j = m_j^{cai} + m_j^{hyd} + m_j^{geo} \tag{8}$$

For calculating the horizontal hydrodynamic mass, the equation presented by Oumeraci and Kortenhaus (1994) is used:

$$m_j^{hyd} = 0.543 L_j \rho_w H_w{}^2 \tag{9}$$

in which the quantities L_j and H_w represent the j^{th} caisson's length and the water level, as shown in Fig. 5; and the quantity ρ_w is the mass density of sea water.
According to Richart et al. (1970), the horizontal geodynamic mass can be computed as:

$$m_j^{geo} = 0.76 \rho_s \left(\frac{B_j L_j}{\pi} \right)^{3/2} / (2 - \nu) \tag{10}$$

where ρ_s and ν are respectively the mass density and Poisson's ratio of the foundation soil; and B_j is the j^{th} caisson's width, as sketched in Fig. 4.
For the stiffness parameter, it is commonly accepted in geotechnical engineering that the horizontal spring constant (k_{Fj}) of the elastic foundation is the function of the horizontal modulus of subgrade reaction *(b)* as, the j^{th} caisson width (B_j) and length (L_j), follows:

$$k_{Fj} = b L_j B_j \tag{11}$$

The modulus of subgrade reaction of various soil types, which has the unit of pressure per length, can be found in literature by Bowles (1996). The same formulas have also been adopted by Goda (1994) and Vink (1997).

Unlike the foundation mound, the theoretical basis for determination of the shear-keys' stiffness is weaker since it depends on the linking capacity between contacted units in the real caisson breakwater (Oumeraci et al., 2001). Normally, caisson segments are designed with the uniform linking capacity, where k_{S2} = k_{S3}. Since the rest of caisson array is not represented in the planar model, the stiffness of the last shear-keys (i.e., k_{S1} and k_{S4}) is smaller than that of the middle shear-keys (i.e., k_{S2} and k_{S3}). This condition can be expressed as:

$$k_{S1} = k_{S4} = pk_{F1} \text{ and } k_{S2} = k_{S3} = qk_{F2} \tag{12}$$

The Rayleigh damping, which is often used in the dynamic mathematical model, is used to simulate the energy dissipation in the caisson system. The Rayleigh damping is assumed to be proportional to the mass and stiffness matrices (Wilson, 2004):

$$[C] = \alpha [M] + \beta [K] \tag{13}$$

in which α is the mass-proportional damping coefficient; and β is the stiffness-proportional damping coefficient. Due to the orthogonality conditions of the mass and stiffness matrices, this equation can be rewritten as:

$$\xi_n = \frac{1}{2\omega_n}\alpha + \frac{\omega_n}{2}\beta \tag{14}$$

where ξ_n is the critical-damping ratio for mode *n*; and ω_n is the n^{th} natural frequency. If the damping ratios corresponding to two specific frequencies (e.g., ω_i and ω_j) are known, the two Rayleigh damping factors (i.e., α and β) can be evaluated from the following equation:

$$\begin{bmatrix} \xi_i \\ \xi_j \end{bmatrix} = \frac{1}{2}\begin{bmatrix} \frac{1}{\omega_i} & \omega_i \\ \frac{1}{\omega_j} & \omega_j \end{bmatrix}\begin{bmatrix} \alpha \\ \beta \end{bmatrix} \tag{15}$$

When damping ratios for both frequencies are set to an equal value, $\xi_i = \xi_j = \xi$, the Rayleigh damping factors are calculated as (Wilson, 2004):

$$\beta = \frac{2\xi}{\omega_i + \omega_j} \quad \text{and} \quad \alpha = \omega_i \omega_j \beta \tag{16}$$

4.2 Structural Identifications of Caisson System

Case: H.W.L. and undamaged

Motion of the test structure is initially established by using geometry of the test structure and the experimental modal parameters. The total horizontal masses are computed with 0.34m of the water height as m_1=m_3=22.95kg and m_2=138.06kg by using Eqs. (8) – (10). The total horizontal mass includes the mass of concrete, the mass of water filled, the added masses of sea water, and the added mass of foundation soil. The diagonal component of the M matrix consists of those three mass parameters.

The modulus of subgrade reaction of the foundation mound, *b*, is selected as 9.6MN/m which is equivalent with that of medium dense sand (Bowles, 1996). By using Eq. (11), the spring constants of the foundation mound are calculated as $k_{F1} = k_{F3}$ =1.824x10^{5}N/m and k_{F2} =1.110x10^{6}N/m. By assuming stiffness ratios as *p*=1, and *q*=0.5 for the initial model, the stiffness of the middle and last shear-keys are obtained as $k_{S1} = k_{S4}$ =1.824x10^{5}N/m and $k_{S2} = k_{S3}$ =5.549x10^{5}N/m, respectively.

For calculating the damping parameter, the experimental modal parameters which includes the first two natural frequencies (ω_1=17.33Hz, ω_2=41.99Hz) and the damping ratio (ξ_1=6.89%, ξ_2=2.74%) are utilized as the initial values to calculated the two Rayleigh damping coefficients (see Eq. (13)).

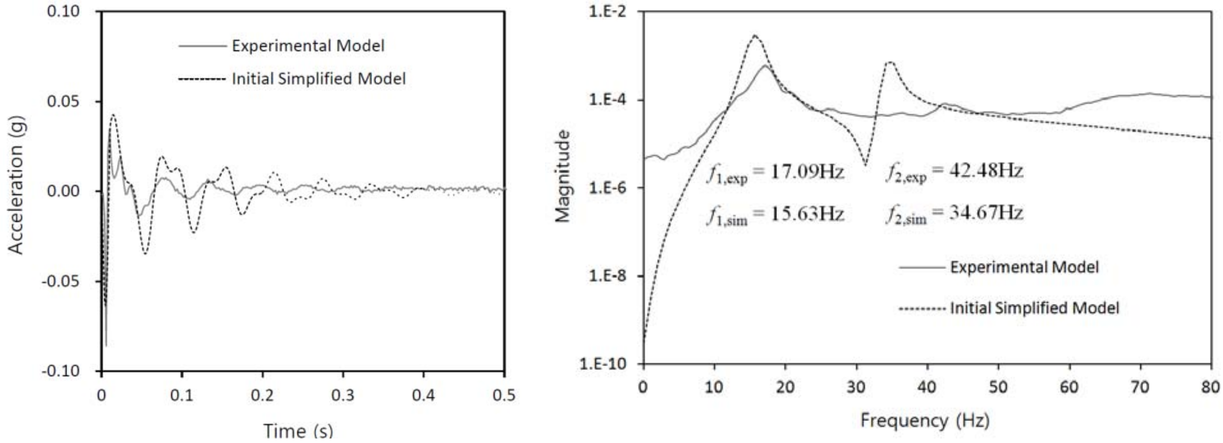
Calculated K matrix and C matrix are summarized with the stiffness ratio in Table 2. To solve the equations of motion, the Runge-Kutta scheme supported in Matlab 7.9.0.529 (R2009b) is utilized. In the calculation of vibration responses, the time interval is set as 0.001 second.

To examine accuracy of the simplified model, acceleration response and power spectral density are compared to that of experiment. The vibration response of initial model is shown with those of experimental results in Fig. 6. In the comparison of acceleration response to those of experiment, the differences in dissipation and oscillating period are observed. For the power spectral density, it is observed that the dissipation of corresponding peaks is quite different.

To improve the accuracy of the simplified model in the peak frequency, the stiffness parameters are updated by using trial and error method. The stiffness ratios, *p* and *q*, of Eq. (12) controlled to adjust the peak frequencies of power spectral density. As the result of the stiffness parameter update, the stiffness ratios are selected as *p*=1.5 and *q*=0.8, so that the stiffness of the shear-keys is updated as $k_{S1} = k_{S4} =$2.736x10^5N/m and $k_{S2} = k_{S3} =$8.880x10^5N/m, respectively. Calculated K matrix and C matrix with the updated stiffness parameters are summarized in Table 2. The vibration response of the stiffness updated model is shown in Fig. 7. In the comparison of acceleration response to those of experiment, the differences in dissipation are still observed. On the other hand, it is still observed that the dissipation of corresponding peaks is quite different.

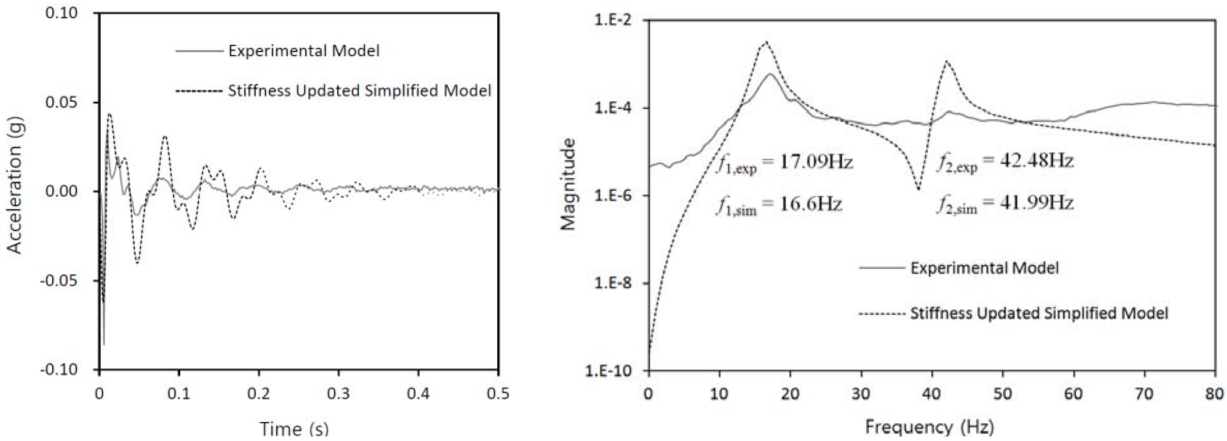
To improve the accuracy of the simplified model in the dissipation of the peaks, damping parameters are updated by using trial and error method. The damping parameters, ξ_1 and ξ_2, are used as updating parameter to adjust the peak dissipation of power spectral density. As the result of the damping parameter update, the damping ratios are selected as ξ_1=15.17%, ξ_2=10.98%, respectively. Calculated K matrix and C matrix with the updated damping parameters are summarized in Table 2. The vibration response of damping updated model is shown in Fig. 8. In the comparison of the acceleration response to those of experiment, it is observed that the acceleration signals are well-matched. Also, in the result of power spectral density, they are well-matched in both peak frequencies and peak dissipations.

To examine the accuracy of the updated simplified model, similarity of the vibration responses of simplified model to those of the experimental test are estimated by correlation coefficient (CC) of acceleration signal and root mean squared deviation (RMSD) of power spectral density. Table 3 shows the estimated accuracy of the simplified models with respect to the parameter updates. For the results in acceleration response, correlations between those of the simplified model and experimental test model are gradually increased by each parameter updates. For the results in power spectral density, deviation between those of the simplified model and experimental test model is rather increased after stiffness parameter update, but decreased evidently after damping parameter update.



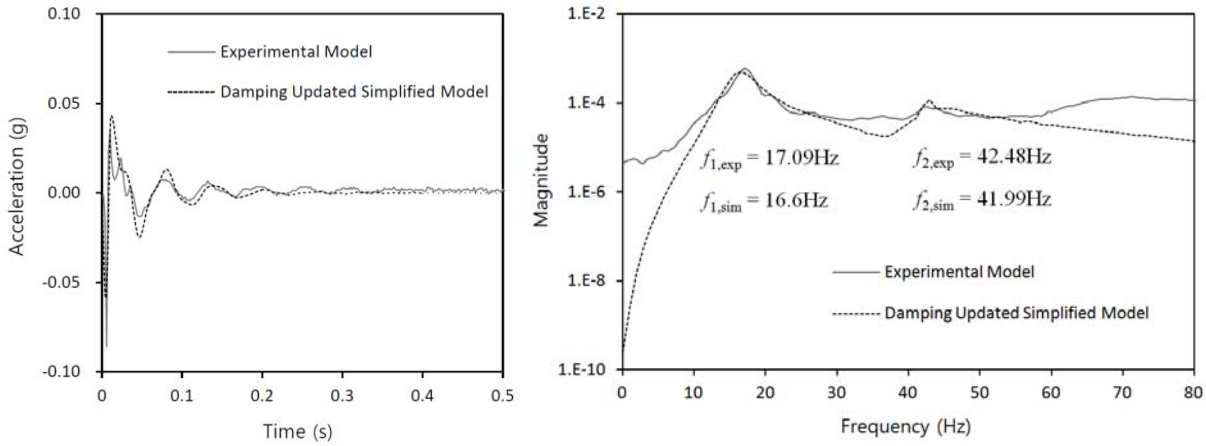
(a) Acceleration response (b) Power spectral density

Fig. 6 Vibration responses of initial simplified model: “Case: H.W.L. and undamaged”



(a) Acceleration response (b) Power spectral density

Fig. 6 Vibration responses of stiffness updated simplified model: “Case: H.W.L. and undamaged”



(a) Acceleration response (b) Power spectral density

Fig. 8 Vibration responses of damping updated simplified model: “Case: H.W.L. and undamaged”

Table 2. Components of equation of motion for the caisson system and stiffness ratio: "Case: H.W.L. and undamaged"

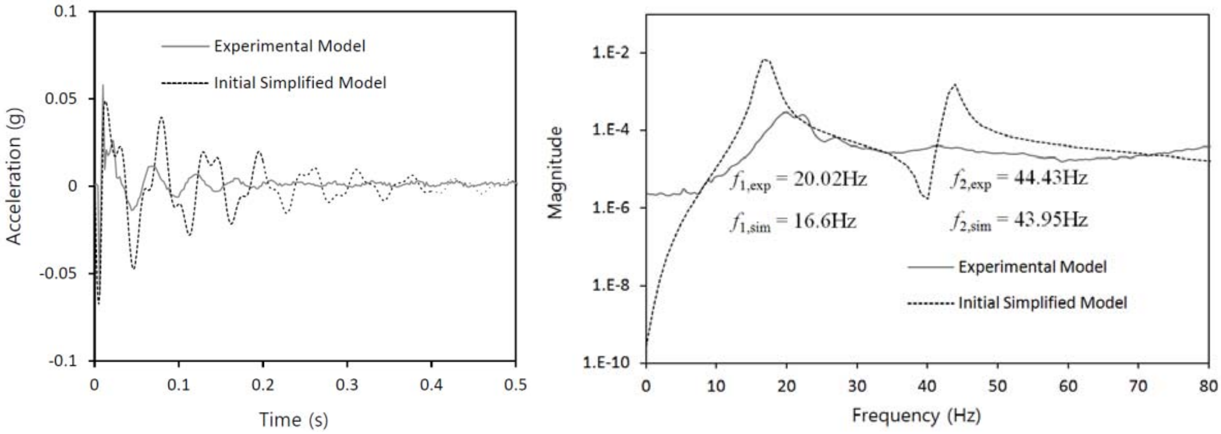
	Stiffness ratio	K matrix	C matrix
Initial	$k_{S1}=k_{S4}=k_{F1}$ $k_{S2}=k_{S3}=0.5k_{F2}$	$\begin{bmatrix} 0.92 & -0.55 & 0 \\ -0.55 & 2.22 & -0.55 \\ 0 & -0.55 & 0.92 \end{bmatrix} \times 10^6$	$\begin{bmatrix} 0.34 & 0.01 & 0 \\ 0.01 & 2.07 & 0.01 \\ 0 & 0.01 & 0.34 \end{bmatrix} \times 10^3$
Stiffness updated	$k_{S1}=k_{S4}=1.5k_{F1}$ $k_{S2}=k_{S3}=0.8k_{F2}$	$\begin{bmatrix} 1.34 & -0.89 & 0 \\ -0.89 & 2.89 & -0.89 \\ 0 & -0.89 & 1.34 \end{bmatrix} \times 10^6$	$\begin{bmatrix} 0.33 & 0.01 & 0 \\ 0.01 & 2.06 & 0.01 \\ 0 & 0.01 & 0.33 \end{bmatrix} \times 10^3$
Damping updated	$k_{S1}=k_{S4}=1.5k_{F1}$ $k_{S2}=k_{S3}=0.8k_{F2}$	$\begin{bmatrix} 1.34 & -0.89 & 0 \\ -0.89 & 2.89 & -0.89 \\ 0 & -0.89 & 1.34 \end{bmatrix} \times 10^6$	$\begin{bmatrix} 1.22 & -0.38 & 0 \\ -0.38 & 5.10 & -0.38 \\ 0 & -0.38 & 1.22 \end{bmatrix} \times 10^3$

Table 3. Accuracy of simplified model with respect to model updates: "Case: H.W.L. and undamaged"

	CC of acceleration signal	RMSD of PSD
Initial	0.6041	0.1046
Stiffness updated	0.6399	0.1147
Damping updated	0.7068	0.0449

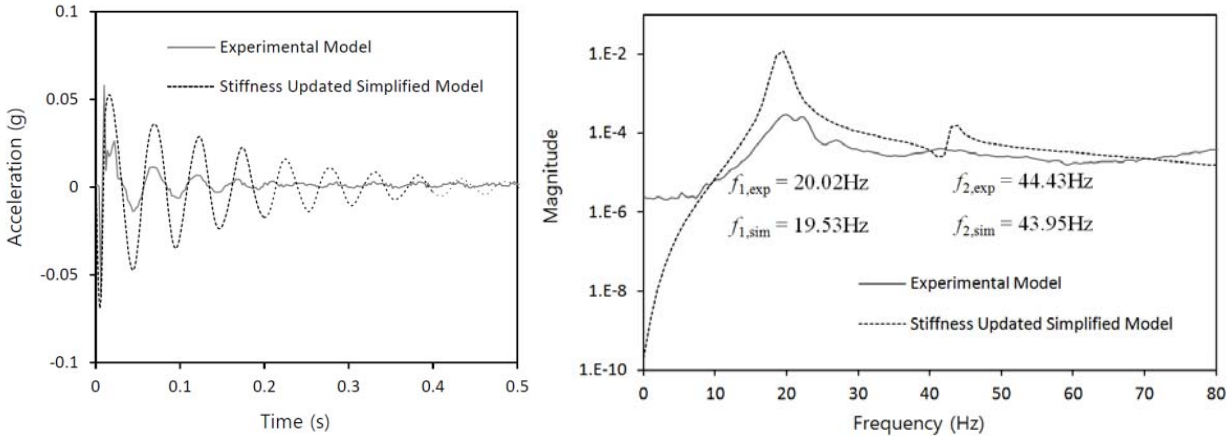
<u>Case: L.W.L. and undamaged</u>

In the identification of the structural parameters for the caisson system with 0.296m of water height (L.W.L.), the damping updated simplified model for "case: H.W.L. and undamaged" is employed and it is assumed that the water level is known. The total horizontal masses, therefore, are computed with 0.296m of water level for the diagonal components as m_1=m_3=21.25kg and m_2=127.72kg by using Eqs. (8) – (10). About the stiffness parameters, the same stiffness ratios to the damping updated model for "Case H.W.L. and undamaged". For the damping parameters, experimental modal analysis of "Case: L.W.L. and undamaged" (ω_1=19.78Hz, ω_2=44.68Hz, ξ_1=5.26%, ξ_2=2.64%) are applied as the initial values to calculate the damping parameters. The model update was performed in the same way described previously. Figure 9 shows vibration responses of initial simplified model for "Case: L.W.L. and undamaged". Figure 10 shows vibration responses of stiffness updated simplified model for "Case: L.W.L. and undamaged". Figure 11 shows vibration responses of damping updated simplified model for "Case: L.W.L. and undamaged". Table 5 shows the accuracy of simplified model with respect to model updates: "Case: L.W.L. and undamaged".



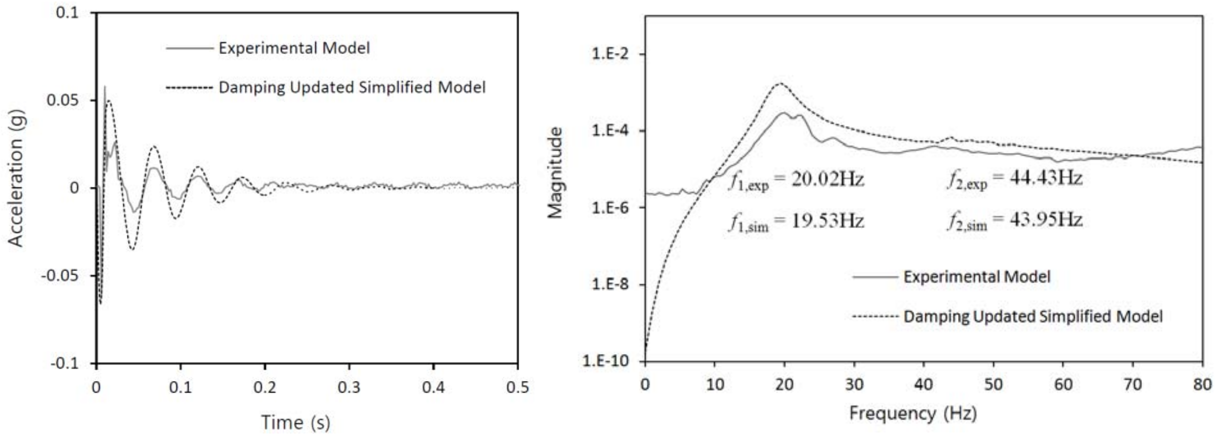
(a) Acceleration response (b) Power spectral density

Fig. 9 Vibration responses of initial simplified model: “Case: L.W.L. and undamaged”



(a) Acceleration response (b) Power spectral density

Fig. 10 Vibration responses of stiffness updated simplified model: “Case: L.W.L. and undamaged”



(a) Acceleration response (b) Power spectral density

Fig. 11 Vibration responses of damping updated simplified model: “Case: L.W.L. and undamaged”

Table 4. Components of equation of motion for the caisson system and stiffness ratio: "Case: L.W.L. and undamaged"

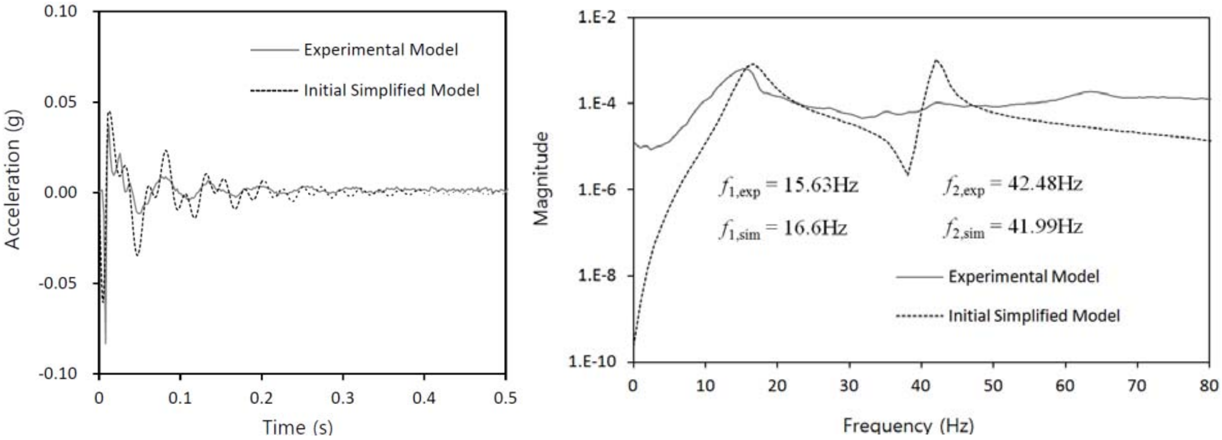
	Stiffness ratio	K matrix	C matrix
Initial	$k_{S1}=k_{S4}=1.5k_{F1}$ $k_{S2}=k_{S3}=0.8k_{F2}$	$\begin{bmatrix} 1.34 & -0.89 & 0 \\ -0.89 & 2.89 & -0.89 \\ 0 & -0.89 & 1.34 \end{bmatrix} \times 10^6$	$\begin{bmatrix} 1.27 & -0.59 & 0 \\ -0.59 & 3.54 & 0.59 \\ 0 & -0.59 & 1.27 \end{bmatrix} \times 10^3$
Stiffness updated	$k_{S1}=k_{S4}=5.76k_{F1}$ $k_{S2}=k_{S3}=0.43k_{F2}$	$\begin{bmatrix} 1.49 & -0.40 & 0 \\ -0.40 & 2.13 & -0.40 \\ 0 & -0.40 & 1.49 \end{bmatrix} \times 10^6$	$\begin{bmatrix} 0.30 & -0.02 & 0 \\ -0.02 & 1.49 & -0.02 \\ 0 & -0.02 & 0.30 \end{bmatrix} \times 10^3$
Damping updated	$k_{S1}=k_{S4}=5.76k_{F1}$ $k_{S2}=k_{S3}=0.43k_{F2}$	$\begin{bmatrix} 1.49 & -0.40 & 0 \\ -0.40 & 2.13 & -0.40 \\ 0 & -0.40 & 1.49 \end{bmatrix} \times 10^6$	$\begin{bmatrix} 1.19 & -0.20 & 0 \\ -0.20 & 3.81 & -0.20 \\ 0 & -0.20 & 1.19 \end{bmatrix} \times 10^3$

Table 5. Accuracy of simplified model with respect to model updates: "Case: L.W.L. and undamaged"

	CC of acceleration signal	RMSD of PSD
Initial	0.5194	0.1654
Stiffness updated	0.6871	0.1284
Damping updated	0.7579	0.0910

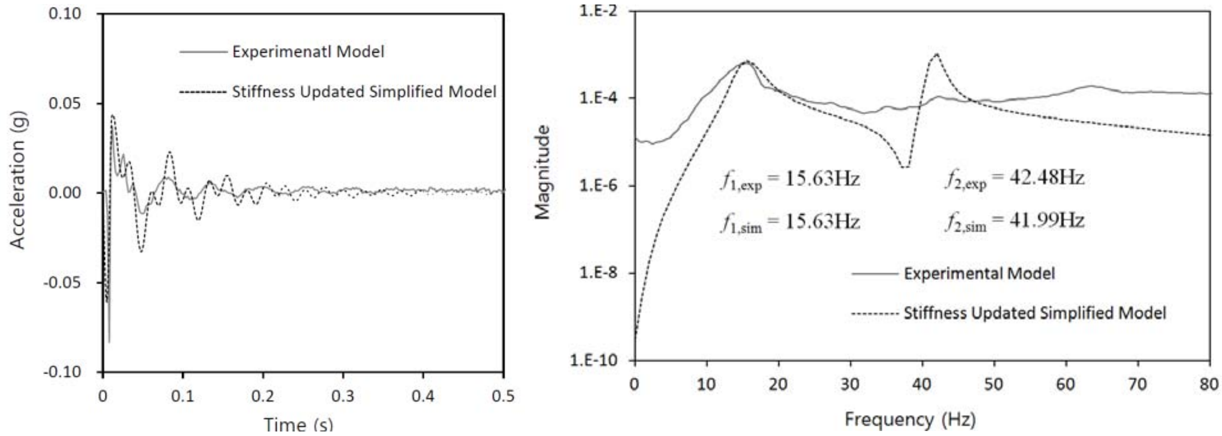
Case: H.W.L. and damaged

In the identification of the structural parameters for the caisson system with 0.34m of water height (H.W.L.), the damping updated simplified model for "case: H.W.L. and undamaged" is employed and it is assumed that the water level is known. The total horizontal masses, therefore, are computed with 0.34m of water level for the diagonal components as $m_1=m_3=22.95$kg and $m_2=138.06$kg by using Eqs. (8) – (10). About the stiffness parameters, the same stiffness ratios to the damping updated model for "Case H.W.L. and undamaged". For the damping parameters, experimental modal analysis of "Case: H.W.L. and damaged" ($\omega_1=15.39$Hz, $\omega_2=42.24$Hz, $\xi_1=12.08\%$, $\xi_2=2.89\%$) are applied as the initial values to calculate the damping parameters. The model update was performed in the same way described previously. Figure 12 shows vibration responses of initial simplified model for "Case: H.W.L. and damaged". Figure 13 shows vibration responses of stiffness updated simplified model for "Case: H.W.L. and damaged". Figure 14 shows vibration responses of damping updated simplified model for "Case: H.W.L. and damaged". Table 7 shows the accuracy of simplified model with respect to model updates: "Case: H.W.L. and damaged".



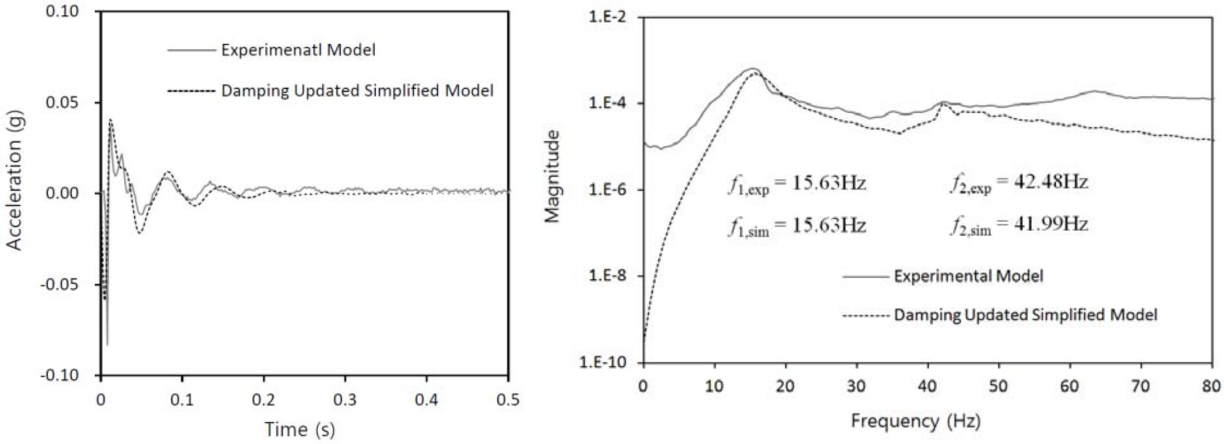
(a) Acceleration response (b) Power spectral density

Fig. 12 Vibration responses of initial simplified model: “Case: H.W.L. and damaged”



(a) Acceleration response (b) Power spectral density

Fig. 13 Vibration responses of stiffness updated simplified model: “Case: H.W.L. and damaged”



(a) Acceleration response (b) Power spectral density

Fig. 14 Vibration responses of damping updated simplified model: “Case: H.W.L. and damaged”

Table 6. Components of equation of motion for the caisson system and stiffness ratio: "Case: H.W.L. and damaged"

	Stiffness ratio	K matrix	C matrix
Initial	$k_{S1}=k_{S4}=1.5k_{F1}$ $k_{S2}=k_{S3}=0.8k_{F2}$	$\begin{bmatrix}1.21 & -0.80 & 0\\ -0.80 & 2.60 & -0.80\\ 0 & -0.80 & 1.21\end{bmatrix}\times 10^6$	$\begin{bmatrix}0.42 & 0.15 & 0\\ 0.15 & 3.4 & 0.15\\ 0 & 0.15 & 0.42\end{bmatrix}\times 10^3$
Stiffness updated	$k_{S1}=k_{S4}=1.22k_{F1}$ $k_{S2}=k_{S3}=0.73k_{F2}$	$\begin{bmatrix}1.31 & -0.90 & 0\\ -0.90 & 2.80 & -0.90\\ 0 & -0.90 & 1.31\end{bmatrix}\times 10^6$	$\begin{bmatrix}0.41 & 0.17 & 0\\ 0.17 & 3.40 & 0.17\\ 0 & 0.17 & 0.4\end{bmatrix}\times 10^3$
Damping updated	$k_{S1}=k_{S4}=1.22k_{F1}$ $k_{S2}=k_{S3}=0.73k_{F2}$	$\begin{bmatrix}1.31 & -0.90 & 0\\ -0.90 & 2.80 & -0.90\\ 0 & -0.90 & 1.31\end{bmatrix}\times 10^6$	$\begin{bmatrix}1.53 & -0.68 & 0\\ -0.68 & 5.14 & -0.68\\ 0 & -0.68 & 1.53\end{bmatrix}\times 10^3$

Table 7. Accuracy of simplified model with respect to model updates: "Case: H.W.L. and damaged"

	CC of acceleration signal	RMSD of PSD
Initial	0.5770	0.1294
Stiffness updated	0.5602	0.1260
Damping updated	0.5978	0.0870

5. CONCLISION

In this study, the structural performance of the gravity-type caisson structure was assessed from vibration-based system identification. To achieve the objective, the following approaches were implemented. Firstly, a structural identification algorithm based on the use of vibration characteristics was outlined. Structural stiffness and damping were selected as two key parameters to be identified for model update. Secondly, experimental vibration analysis of a lab-scaled gravity-type caisson structure was described. Vibration tests on 2-D wave flume are performed for a few scenarios such as water-level changes and foundation damage. Experimental modal parameters were extracted by frequency domain decomposition method. Finally, vibration-based structural integrity was assessed for the tested caisson system. A simplified model is formulated to represent the tested structure with the limited degrees-of-freedom that corresponds to the sensors' degrees-of-freedom. Structural parameters of the simplified model were identified by two phases: 1) K matrix update by tuning of stiffness ratios and 2) C matrix update by tuning of damping factors. Tuning of the model was performed based on the power spectral density. The accuracy of the updated simplified model was examined by the indicators such as correlation coefficient and root mean squared deviation and the structural parameters of experimental model were well identified by simplified model.

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