ESTIMATION OF STORY PARAMETERS FROM MICROTREMOR RECORDS USING MODAL RESPONSES OF SUBSTRUCTURES

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Abstract

A method to estimate the stiffness coefficient and the damping factor (story parameters) of each story of a structure using microtremor records including soil-structure interaction effects is proposed. For the substructure system extracted from the structure, the absolute amplification functions (modal response) of the generalized coordinates fixed at the bottom are calculated using the horizontal records of each floor and the rocking motion observed on the base. The story parameters are identified so that the modal response is equal to the absolute magnification function of one mass system with the modal frequency and the damping factor. In the formulation, it is assumed that the structure is a shear type, and the horizontal motion at each floor and the rocking motion are observed. In the calculation of the modal responses, the complex Fourier amplitude or the ratio to a reference point are used. To confirm the validity of the proposed procedure, the method is applied to the numerical model of a 6-storied structure, and to the 5-storied steel frame structure that have been used in a shake table test.

Keywords: SSI, Story parameter, Microtremor, Modal analysis, Substructure

1. Introduction

Microtremor records have been utilized in the health monitoring of structures and the damage detection after big earthquakes [1]. It is preferable to establish a method to extract not only the mode factors such as frequency or damping but also the story parameters. In estimation of the story parameters, if the structure is supported on a soft soil, soil-structure interaction (SSI) effects must be eliminated. Luco [2], Todorovska [3], Kawakami [4] have studied methods to find dynamic characteristic of the structures in case of missing of rocking motions.

Kashima and Kitagawa [5] identified the structural stiffness, modal damping and sway-rocking soil springs by comparing observed data with the numerical responses of an assumed lumped multi-mass model. There is a possibility, however, that the estimated values of the soil springs may have influence on the accuracy of the structure parameters.

There have been presented several identifying methods that do not require estimation of the soil springs. Luco [2] presented a method to estimate structural properties from a magnification function of the relative displacement of the superstructure including horizontal and rocking motions, and Ishibashi and Naito [6] proposed a similar procedure using the absolute magnification function. Kawashima et al. [7] and Kasai et al. [8] showed that the responses of a multi-mass model may be calculated by the sum of the products of the mode participation functions, the effective input motion [9] and the response of each mode, and tried to identify the mode parameters. However, these methods are not applicable to evaluation of the story parameters.

Kawashima et al. [10] showed another prospective procedure eliminating SSI effects by utilizing the subspace method which was based on the input-output relation of the observation system. Although the procedure becomes a powerful tool in the determination of modal parameters such as natural frequencies and damping factors, further analyses [11] are required to identify the story parameters of the structure.

By applying the above-mentioned method [7,8] to substructure systems which are made by isolating the structure at any level, the modal response of the substructure is obtained. By isolating the whole structure successively from top to bottom, the story parameters can be identified from the top story in series. This method has the advantage of making the most use of easily obtainable absolute motions recorded at every story and the base rocking motion, and of not requiring the identification or estimation of the soil springs. Moreover, as the modal responses can be calculated by using the Fourier spectrum ratios to a reference point, simultaneous observation is not necessarily required as far as the motion is stationary such as microtremors.

2. Fundamental equations

2.1 Equation of motion of soil-structure system

Fig.1 shows a soil-structure system. The super structure is a n-lumped mass shear type model, and m_i, H_i and $[k_i + ik_i']$ are the i-th mass, the i-th floor height and the i-th story complex stiffness coefficient, respectively. It is supposed that the system is subjected to the horizontal ground motion $x_g e^{i\omega t}$, and then $X_0 e^{i\omega t}$, $\theta_0 e^{i\omega t}$ and $x_i e^{i\omega t}$ are the horizontal motion at the top of base, the rocking motion at the base and the relative displacement of the i-th mass, respectively, in which i, ω and t are the imaginary unit, the circular frequency and the time, respectively. The equation of motion of the system in frequency domain is given as

$$-\omega^2[M]\{x\} + ([K] + i[K'])\{x\} = \omega^2[M](\{1\}X_0 + \{H\}\theta_0), \quad (1)$$

$$X_0 = x_g + x_{0,} \quad (2)$$

where $[M]$: the mass matrix, $([K] + i[K'])$: the complex stiffness matrix, $\{x\}$: the displacement vector, $\{H\}$: the height vector, $\{1\}$: the unit vector, x_0: the relative displacement of the base to the ground motion. In above equations the time factor $e^{i\omega t}$ is omitted, and it follows hereafter. And the right side of Eq. (1) is so called the effective earthquake forces [12].

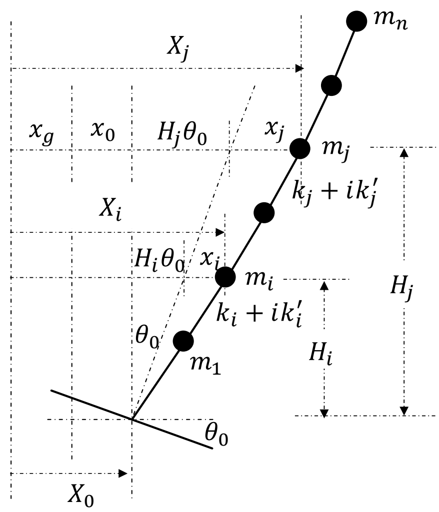

Fig. 1 – Soil-structure system

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The absolute displacement vector $\{X\}$ which consists of X_i may be formulated as

$$\{X\} = \{1\}X_0 + \{x\} + \{H\}\theta_0, \tag{3}$$

and the complex stiffness coefficient of the i-th story is represented as

$$k_i + ik_i' = k_i(1 + i2h_{m,i}), \tag{4}$$

$$h_{m,i} = k_i'/(2k_i), \tag{5}$$

where $h_{m,i}$ is material damping factor, and k_i and $h_{m,i}$ are target variables to be identified.

2.2 Equation of motion of substructure system

Fig.2 shows the j-th substructure system which is made by isolating the structure at the foot of the j-th story. The equation of motion of the model can be formulated as

$$-\omega^2[M_j]\{x_j\} + ([K_j] + i[K_j'])\{x_j\} = \omega^2[M_j](\{1\}X_{j-1} + \{H_j\}\theta_0), \tag{6}$$

where, the right side of Eq. (6) corresponds to the effective earthquake forces of Eq. (1). $[\blacksquare_j]$ and $\{\blacksquare_j\}$ denote a matrix with size of (n-j+1) × (n-j+1) and a vector with size of (n-j+1), respectively. By referring Fig.2 $\{X_j\}$ will be shown to be given as

$$\{X_j\} = \{1\}X_{j-1} + \{x_j\} + \{H_j\}\theta_0, \tag{7}$$

The components of $\{X_j\}$ which is denoted by $X_{j,i}$ and $\{H_j\}$ by $H_{j,i}$ are given as follows,

$$X_{j,i} = X_{i+j-1} \quad (i = 1 \sim n - j + 1), \tag{8}$$

$$H_{j,i} = H_{i+j-1} - H_{j-1} \quad (i = 1 \sim n - j + 1), \text{ where } H_0 = 0. \tag{9}$$

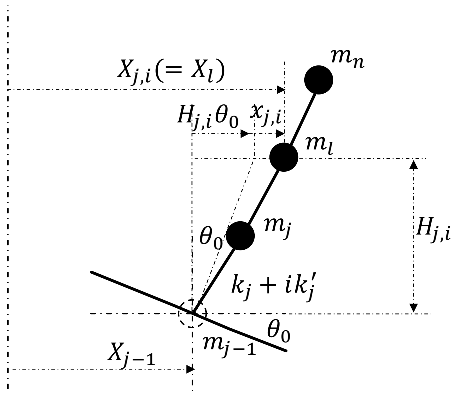

Fig. 2 – j-th Substructure system

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2.3 Modal response of base fixed substructure model

From the eigenvalue equation that is obtained by setting the right side of Eq. (6) equal to 0 with $[K_j'] = 0$, [*n*-*j*+1] th natural circular frequencies ${}_s\omega_j$ and eigenvectors $\{{}_su_j\}$ can be obtained, where the subscript of the left side (s) denotes the mode number. Then using the generalized coordinate ${}_sq_j$, $\{x_j\}$ can be expanded as

$$\{x_j\} = \sum_{s=1}^{n-j+1} {}_sq_j \{{}_su_j\}. \tag{10}$$

Substituting Eq.(10) into Eq. (6), and multiplying by $\{{}_su_j\}$ from the left side, ${}_sq_j$ is obtained considering the orthogonality of the eigenvectors. Further by using Eq. (11) and Eq. (12), Eq. (7) can be expressed as Eq. (13).

$$\{1\} = \sum_{s=1}^{n-j+1} {}_s\beta_j \{{}_su_j\}, \tag{11}$$

$$\{H_j\} = \sum_{s=1}^{n-j+1} {}_s\beta_j \{{}_su_j\} \, {}_sH_{e,j}, \tag{12}$$

$$\{X_j\} = \sum_{s=1}^{n-j+1} {}_s\beta_j \{{}_su_j\} \, {}_sG_{FX,j}^{(C)} (X_{j-1} + {}_sH_{e,j}\,\theta_0), \tag{13}$$

$${}_sG_{FX,j}^{(C)} = \frac{(1 + i2\,{}_sh_{m,j})\,{}_s\omega_j^2}{(1 + i2\,{}_sh_{m,j})\,{}_s\omega_j^2 - \omega^2}. \tag{14}$$

In above equations, ${}_s\beta_j$, ${}_s\omega_j$, ${}_sh_{m,j}$, ${}_sH_{e,j}$ denote the participation factor, the natural circular frequency, the modal damping and the equivalent height [8, 13] respectively, and these may be calculated by following formulas,

$${}_s\beta_j = \{{}_su_j\}^T[M_j]\{1\}/\{{}_su_j\}^T[M_j]\{{}_su_j\}, \tag{15}$$

$${}_s\omega_j^2 = {}_sK_{e,j}/{}_sM_{e,j}, \tag{16}$$

$${}_sh_{m,j} = {}_sK'_{e,j}/(2\,{}_sK_{e,j}), \tag{17}$$

$${}_sH_{e,j} = \{{}_su_j\}^T[M_j]\{H_j\}/\{{}_su_j\}^T[M_j]\{1\}. \tag{18}$$

Where, ${}_sM_{e,j}$, ${}_sK_{e,j}$, ${}_sK'_{e,j}$ are the equivalent mass, the equivalent stiffness coefficient and the equivalent imaginary stiffness coefficient,

$${}_sM_{e,j} = {}_s\beta_j^2\{{}_su_j\}^T[M_j]\{{}_su_j\}, \tag{19}$$

$${}_sK_{e,j} = {}_s\beta_j^2\{{}_su_j\}^T[K_j]\{{}_su_j\}, \tag{20}$$

$${}_sK'_{e,j} = {}_s\beta_j^2\{{}_su_j\}^T[K_j']\{{}_su_j\}. \tag{21}$$

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It is noted that Eq. (29) gives an approximate value in case of non-proportional damping.

${}_sG^{(C)}_{FX,j}$ in Eq. (13) and Eq. (14) is the absolute modal response function of the base fixed j-th substructure of the s-th mode. The $\left(X_{j-1} + {}_sH_{e,j}\,\theta_0\right)$ in the right side of Eq. (13) corresponds to the effective input motion to the substructure.

Multiplying Eq. (13) by ${}_s\beta_j\{{}_su_j\}^T[M_j]$ from the left side, and considering the orthogonality of the eigenvectors, we obtain

$$ {}_sG^{(O)}_{FX,j} = \frac{{}_s\beta_j\,\{{}_su_j\}^T[M_j]\{X_j\}}{{}_sM_{e,j}\left(X_{j-1} + {}_sH_{e,j}\,\theta_0\right)}, \tag{22} $$

where, the right superscript (o) in ${}_sG^{(O)}_{FX,j}$ shows that Eq. (22) is evaluated using the observed records such as $\{X_j\}$, X_{j-1} and θ_0. In what follows, ${}_sG^{(O)}_{FX,j}$ will be called as the observed modal response, and ${}_sG^{(C)}_{FX,j}$ is called as the theoretical modal response. Now, dividing the numerator and the denominator of the right side of Eq. (22) by X_A that is the observed record at a reference point A, ${}_sG^{(O)}_{FX,j}$ may be expressed as

$$ {}_sG^{(O)}_{FX,j} = \frac{{}_s\beta_j\,\{{}_su_j\}^T[M_j]\dfrac{\{X_j\}}{X_A}}{{}_sM_{e,j}\left(\dfrac{X_{j-1}}{X_A} + {}_sH_{e,j}\,\dfrac{\theta_0}{X_A}\right)}, \tag{23} $$

Eq. (23) shows that the Fourier spectrum ratio can be used in place of the amplitude itself, and simultaneous observational records at all points are not necessarily in case of the stationary motion such as microtremors.

2.4 Treatment of viscous damping

In case of showing the viscous damping, the equation of motion is obtained by replacing $[K']$ in Eq. (1) with $\omega[C]$, thus

$$ -\omega^2[M]\{x\} + ([K] + i\omega[C])\{x\} = \omega^2[M](\{1\}X_0 + \{H\}\theta_0), \tag{24} $$

and the complex stiffness coefficient of the *i*-th story is given as

$$ k_i + i\omega c_i = k_i + i2\omega h_{v,i}\sqrt{m_i k_i}, \tag{25} $$

$$ h_{v,i} = c_i/\left(2\sqrt{m_i k_i}\right), \tag{26} $$

where $h_{v,i}$ is viscous damping factor. Thus Eq. (14) can be rewritten as

$$ {}_sG^{(C)}_{FX,j} = \frac{\left\{1 + i2\,{}_sh_{v,j}\left(\dfrac{\omega}{{}_s\omega_j}\right)\right\}\,{}_s\omega_j^2}{\left\{1 + i2\,{}_sh_{v,j}\left(\dfrac{\omega}{{}_s\omega_j}\right)\right\}\,{}_s\omega_j^2 - \omega^2}, \tag{27} $$

where

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$$ {}_s h_{v,j} = {}_s C_{e,j} / (2\sqrt{{}_s M_{e,j}\ {}_s K_{e,j}}) , \tag{28} $$

$$ {}_s C_{e,j} = {}_s \beta_j^2 \{{}_s u_j\}^T [C_j] \{{}_s u_j\}. \tag{29} $$

It is noted that Eq. (29) gives an approximate value in case of non-proportional damping.

3. Method of search

Eq. (30) shows the objective function to identify the story parameters,

$$ {}_s J(k_j, h_j) = \frac{1}{m} \sum_{s=1}^{l} \sum_{k=1}^{m} \left\{ \left| {}_s G_{FX,j}^{(C)}(\omega_k) \right| - \left| {}_s G_{FX,j}^{(O)}(\omega_k) \right| \right\}^2 , \tag{30} $$

where corresponding to the damping type, Eq. (14) or Eq. (27) is used as ${}_s G_{FX,j}^{(C)}(\omega_k)$, and to the amplitude type of floor responses (the Fourier amplitude itself or the amplitude ratio), Eq. (22) or Eq. (23) is used as ${}_s G_{FX,j}^{(O)}(\omega_k)$. And ω_k, k, m and l are the discretized circular frequencies, the number of the frequencies, the maximum number of the frequencies and the maximum mode number considered. In the case that m_i and H_i are known and, k_i and $h_i(h_{m,i}\ or\ h_{v,i})$ are the target variables, the 1-st mode is enough to be considered and then l is set to 1.

Searching is done by using the hybrid method [14] with the genetic algorithms and the simulated annealing. Where the parameter values are as follows: the maximum generation number = 100, the population = 30, the crossover rate = 0.7, the mutation rate = 0.01, and moreover the dynamic shift of the mutation rate and the elite selection are applied. The parameters of the cooling function $[T_k = T_0 exp(-ck^a)]$ are set as follows : T_0 = 100, a = 0.5, c = 1.0, k = 10. On these conditions, trials are done 10 times, then the average value is set to the identified one.

4. Examination by using numerical model

4.1 Numerical model and frequency response

Table 1 shows the constants of the shear type structure for long span direction, being made from the pilot design [15] of a 6-storied reinforced concrete apartment with the standard floor plan 35.4m × 9.32m. The material damping whose factors are distributed from 2% to 8% vertically (non-proportional damping) is assumed. Fig. 3 shows the sway-rocking model who has a rigid foundation and soil springs as shown by Eq. (31) and Eq. (32)

$$ K_H = K_{H0} + i\omega C_H, \tag{31} $$

$$ K_R = K_{R0} + i\omega C_R, \tag{32} $$

where, K_H : the horizontal spring, K_{H0} : the static value of K_H, C_H : the horizontal viscous damping coefficient, K_R : the rocking spring, K_{R0} : the static value of K_R, C_R : the rocking viscous damping coefficient. And the values of these parameters are shown in Table 2.

The equation of motions subjected to the sinusoidal ground motion is shown as

$$ -\omega^2 \sum_{i=1}^{n} m_i X_i - \omega^2 m_0 X_0 + K_H X_0 = 0, \tag{33} $$

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Table 1 – Constants of numerical model

Mass no.	Height (m)	H_i (m)	m_i (ton)	k_i (kN/m)	$h_{m,i}$
6	17.70	2.85	478	2.22×10^6	0.02
5	14.85	2.85	500	2.42×10^6	0.03
4	12.00	2.85	500	2.47×10^6	0.04
3	9.15	2.85	520	2.58×10^6	0.06
2	6.30	2.85	520	2.36×10^6	0.07
1	3.45	3.45	520	3.05×10^6	0.08

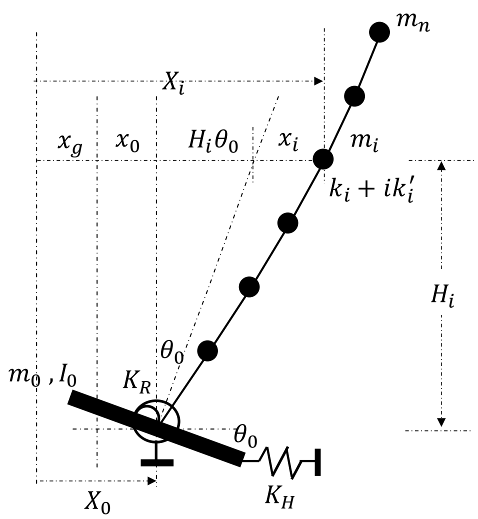

Fig. 3 – Sway-rocking model with foundation

Table 2 – Soil spring coefficients

K_H	K_{H0} (kN/m)	3.05×10^6
	C_H (kN·s/m)	3.13×10^4
K_R	K_{R0} (kN·m/rad)	1.55×10^9
	C_R (kN·m·s/rad)	6.88×10^6

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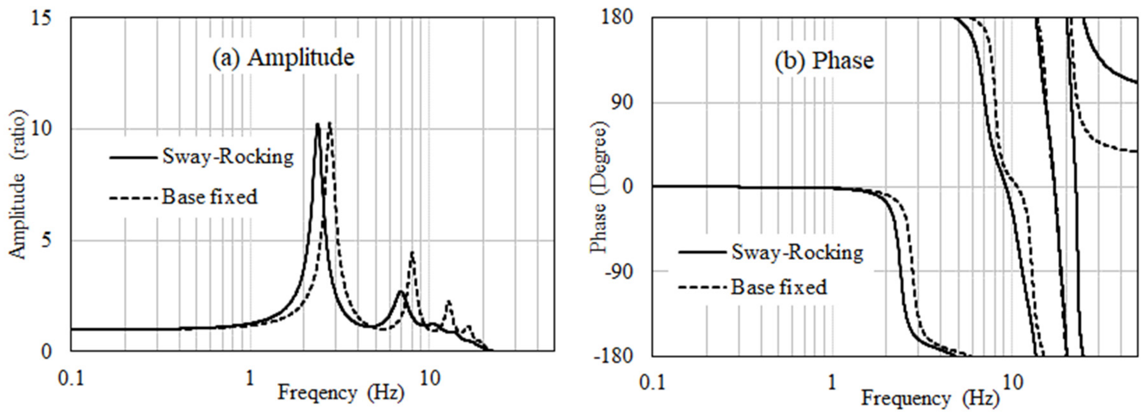

Fig. 4 – Frequency response function of X_6/X_g (case of numerical model)

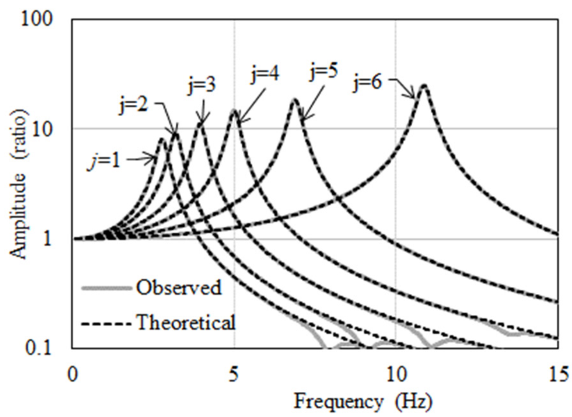

Fig. 5 – Modal responses (case of numerical model)

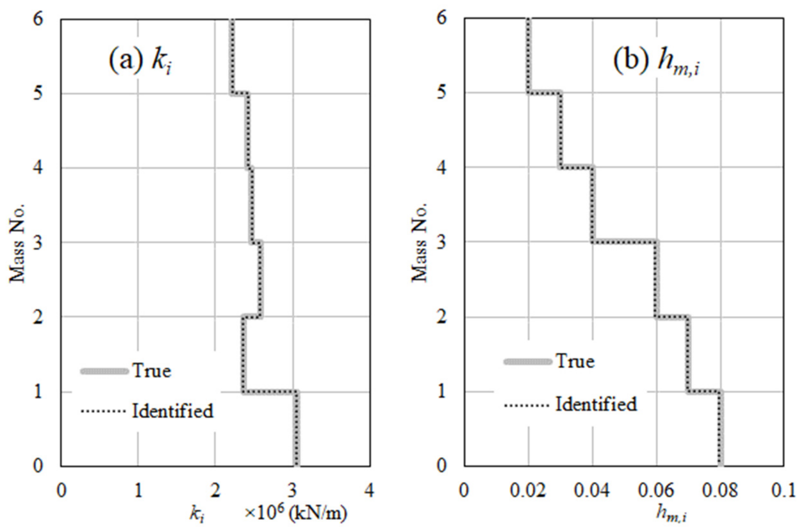

Fig. 6 – Identified results (case of numerical model)

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$$-\omega^2 \sum_{i=1}^{n} m_i H_i X_i - \omega^2 I_0 \theta_0 + K_R \theta_0 = 0, \tag{34}$$

where m_0 and I_0 are the mass of foundation and the rotary inertia mass, and those values are 795(ton) and $8.31\times 10^4 (ton \cdot m^2)$ respctively. The responses of the system shown in Fig. 3 is obtained by calculating Eq. (1), Eq. (33) and Eq. (34) simultaneously. When the horizontal ground motion x_g is $1 \cdot e^{i\omega t}$, then the responses correspond to the ratios to the reference point at the free field. Fig. 4 shows the frequency response functions of the top mass (X_6/X_g) together with the base fixed case, where Fig. 4(a) is the amplitude function and Fig. 4(b) is the phase characteristics respectively. The first mode natural frequency of sway-rocking model is 2.38(Hz) and the base fixed model is 2.77(Hz), therefore SSI effects of the model are not so strong.

4.2 Search and Identified results

In the case variable m_i and H_i are known, k_i and $h_{m,i}$ are searched. The search frequency range is set to from 0.1 Hz to 15.0 Hz, and the search range of k_i is 1/3 to 3 times the identification value of the immediately above story, and similarly h_i is 1/2 to 2 times the upper story.

Fig.5 shows the amplitude functions of the modal response of the j-th substructure model (j=1~6), being compared the observed with the theoretical one. The two around the peaks are almost identical and indistinguishable. Fig.6(a) shows the identified results of k_i, and Fig.6(b) shows the identified results of $h_{m,i}$. In both variables, the identified values are in good agreement with the true values. Where, the case of viscous damping type is not shown, similar good results have been obtained.

It is remarked that if the system has strong SSI effects, in order to identify the damping factors exactly, the quasi-complex modal analysis techniques [16] must be introduced.

5. Application to shake table test results

5.1 Structure model and data processing

A steel frame structure had been used in the E-Defense shake table test [17] is considered. The specimen is a 5-storied steel frame structure with the standard floor plan of 10m x 12m, the first story height of 3.85m, and the standard story height of 3m. Table 3 shows the dimensions of the structure [8]. The test results of interest are the case (Data Storage No.: 2009-00406-011M) where the white noise with an amplitude of 100 gal was input for 327 seconds in the short span direction of the specimen in the case of without damper. It is considered that the white noise has the same effects as the microtremor because of their stationarity. And it is noted that the vibration direction of the shake table was horizontal, but the rotational motions that couldn't be ignored occurred [8].

Table 3 – Dimensions of steel frame structure (from Kasai et al. [8])

Mass No.	Height (m)	Hi (m)	m_i (ton)	k_i (kN/m)
5	15.84	2.99	150.7	1.099×10^5
4	12.85	3.00	82.1	1.101×10^5
3	9.85	3.00	83.8	1.249×10^5
2	6.85	3.00	84.1	1.305×10^5
1	3.85	3.85	86.1	1.323×10^5

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In the observation, three components of acceleration at four measuring points were measured at the shake table and on each floor, and the sampling frequency is 500 Hz. An anti-aliasing filter with a cutoff frequency of 50 Hz is applied to the original records. The waveforms obtained by sampling the filtered records at 100 Hz are used for the analysis. The horizontal waveform on the first floor is obtained by interpolating in the height direction (0.9 m) the average of the four points on the shake table. The rotational motion of the shake table is obtained by averaging the rotational motion of the both sides obtained from the vertical movements. The used horizontal waveforms of the floor higher than the first floor are the average of the left and right components in the vibration direction, being removed torsional motions.

The interval of a sample is 81.92 seconds, and the interval is shifted by 40.96 seconds in sequence, being cut out five data sets. In addition, a cosine taper with a length of 4 seconds is applied to both ends of the cut waveform. The Fourier transforming and bandpass filter (BPF) processing of these waveforms are performed, and then the ratio of the interest point to a reference point is calculated. Furthermore, the ratio obtained by averaging the data sets is used as the complex Fourier amplitude ratio of the interest point. Where, the horizontal component of the first floor is selected as the reference value.

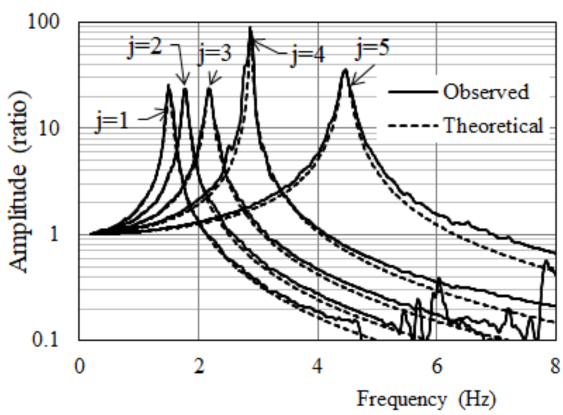

Fig. 7 – Modal responses (case of steel structure)

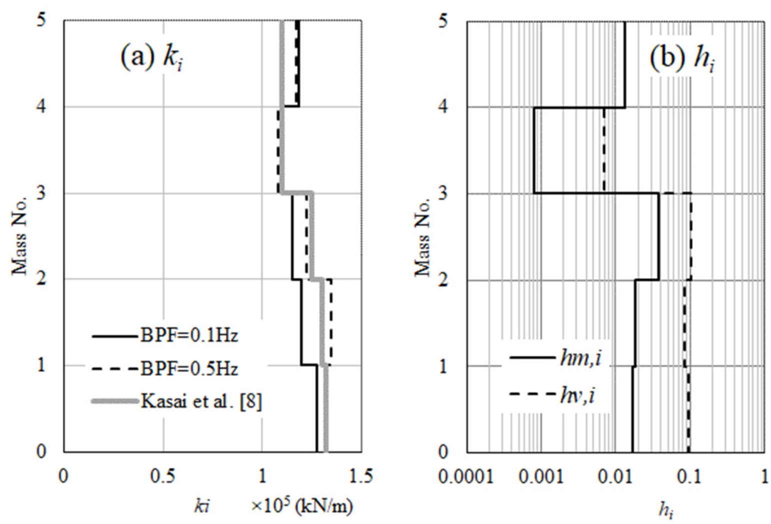

Fig. 8 – Identified results (case of steel structure)

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5.2 Search and identified results

The frequency range for the search is 0 Hz to 15 Hz, the range of k_i is from 1 / 1.5 to 1.5 times the identified value of the immediately upper layer, and the range of $h_i(h_{m,i} , h_{v,i})$ is set to from 0.0 to 0.15.

Fig. 7 shows the modal responses of the j-th substructure model (j=1~5) in the case of the material damping type and the BPF is 0.1 Hz. The observed and the theoretical values are in good agreement. Where, the case of the viscous damping type is not shown, the same results have been observed.

Fig.8(a) shows the identified result of k_i ,where the BPF is 0.1 Hz and 0.5 Hz, and the latter is in good agreement with the reference value [8]. Fig.8(b) shows the results of $h_i(h_{m,i} , h_{v,i})$ for the case where the BPF is 0.1 Hz. The values depend on BPF, and so further accumulation of identification data is required to evaluate the validity of the damping factors.

Fig.9 show the frequency response function of the top mass (X_5/X_0), being compared the observed and the calculated. Where, the calculated one is obtained by using Eq. (1) and Eq. (7), showing for the case of the material damping and of viscous damping. The observed agrees well with the calculated at the first peak frequency regardless of the damping type. On the other hand, around the second peak of Fig.9(a), the amplitude of the viscous damping type is considerably smaller than the observed value, showing that the damping type of the structure is not viscous type but material type.

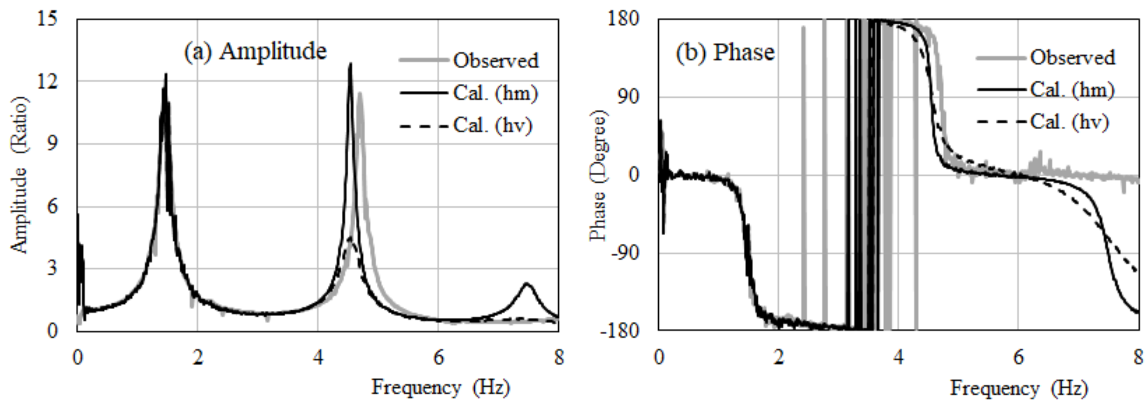

Fig. 9 – Frequency response function of X_5/X_0 (case of steel structure)

6. Conclusions

A method to identify the story parameters (stiffness coefficient and damping factor) has been proposed on conditions that the rocking motion at the base, and the horizontal motion on every floor of a shear type are observed. The findings obtained in this paper can be summarized as follows.

The modal response of a substructure model with the fixed base has been derived with use of the observed records. A criterion to minimize the difference between the observed modal response and the theoretical one has been introduced into the procedure identifying the story parameters.

The proposed method has an advantage of using easily obtainable absolute motion recorded on each story and makes the identification of the story parameters possible without identifying the soil springs. Moreover, simultaneous observation at all stories is not necessarily needed for the stationary motion such as microtremor.

Numerical results obtained by applying the proposed method to a 6-storied RC building have indicated that identified story parameters have agreed well with the true values.

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The shake table test results of a 5-storied steel frame structure have been examined. The story stiffness coefficients identified have well agreed with the reference values. Although the identified damping factors couldn't have been evaluated precisely, the damping type has been supposed to be of material but not of viscous.

References

[1] Nakamura M, Yasui Y (1999): Damage evaluation of a steel structure subjected to strong earthquake motion based on ambient vibration measurements. *Journal of Structural and Construction Engineering*, Architectural Institute of Japan, No.517, 61-68.

[2] Luco JE (1980): Soil-structure interaction and identification of structural models. *Proc. 2nd ASCE Conference on Civil Engineering and Nuclear Power*, Vol.2, 10/1/1-10/1/31.

[3] Todorovska MI (2009): Seismic interferometry of a soil-structure interaction model with coupled horizontal and rocking response. *Bulletin of the Seismological Society of America*, Vol.99, No.2A, 611-625.

[4] Kawakami H, Oyunchimeg M (2004): Wave propagation modeling analysis of earthquake records for building. *Journal of Asian Architecture and Building Engineering*, Vol.3, No.1, 33-40.

[5] Kashima T, Kitagawa Y (2006): Dynamic characteristics of a building estimated from strong motion records using evolution strategy. *Journal of Structural and Construction Engineering*, Architectural Institute of Japan, No.602, 145-152.

[6] Ishibashi T, Naito Y (1994): System identification methods of buildings considering rocking motion of the base. *Annual Report*, Vol.42, Kajima Technical Research Institute, Kajima Corporation, 149-152.

[7] Kawashima M, Okawa I, Kashima T, Iguchi M (2008): Evaluation of dynamic characteristics of soil-structure system based on recorded earthquake motions (part6) observed changes of dynamic characteristics in the BRI building. *Summaries of Technical Papers of Annual Meeting*, Architectural Institute of Japan, 271-272.

[8] Kasai K, Murata S, Kato F, Hikino T, Ooki Y (2011): Evaluation rule for vibration period, damping, and mode vector of buildings tested by a shake table with inevitable rocking motions. *Journal of Structural and Construction Engineering*, Architectural Institute of Japan, Vol.76, No.670, pp.2031-2040.

[9] Kawashima M, Iguchi M, Minowa C (2007): Extraction of effective input motions to structures based on earthquake observations and a measure for the input motions. *Journal of Structural and Construction Engineering*, Architectural Institute of Japan, No.615, 85-92.

[10] Kawashima M, Nagano M, Kashima T, Iguchi M (2010): Varying of dynamic characteristics of a building and prediction accuracy by regression equation examined with over the long term earthquake observation. *The 13th Japan Earthquake Engineering Symposium*, Tsukuba, Japan, GO14-Fri-AM-2, 1530-1537.

[11] Yoshimoto R, Mita A (2003): Online identification of structural parameters utilizing multi-input multi-output models. *Journal of Structural and Construction Engineering*, Architectural Institute of Japan, No.574, 39-44.

[12] Chopra AK, Gutierrez JA (1974): Earthquake response analysis of multistorey buildings including foundation interaction. *Earthquake Engineering and Structural Dynamics*, Vol.3, 65-77.

[13] Tajimi H (1967): Building-foundation interaction effects, Discussion by Hiroshi Tajimi. *Journal of the Engineering Mechanics Division*, ASCE, Vol.93, EM6, .294-298.

[14] Yamanaka H (2007): Inversion of surface-wave phase velocity using hybrid heuristic search method. *BUTURI-TANSA*, The Society of Exploration Geophysicists of Japan, Vol.60, No.3, 265-275.

[15] Inoue K (2009): Pilot study, 6-storied middle high building. *Seismic Response Analysis and Design of Buildings Considering Dynamic Soil-Structure Interaction*, Architectural Institute of Japan, 205-215.

[16] Yasui Y, Maeda T, Iguchi M (2019): Evaluation method of story stiffness and damping coefficients through structural Response Influenced by SSI effects, *Journal of JAEE*, Japan Association for Earthquake Engineering, Vol.19, No.5, 267-282.

[17] Kasai K, Motoui S, Ozaki H, Ishii M, Ito H, Kajiwara K, Hikino T (2013): Full-scale tests of passively- controlled 5-story steel building using E-Defense shake table. https://www.edgrd.jp/datas/

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