EARTHQUAKE DAMAGE EVALUATION OF AN 8-STORY STEEL-REINFORCED CONCRETE BUILDING USING S_a-S_d CURVES

SRC 建物の建物性能曲線による地震損傷評価

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A seismic performance curve shows the relationship between the representative displacement (S_d) and the equivalent base shear force coefficient (S_a) of a structure. The seismic capacity of a building can be assessed in a straightforward manner using a simple S_a-S_d performance skeleton curve. In this study, earthquake response records were used to obtain S_a-S_d curves for a steel-reinforced concrete (SRC) building and assess the changes in the seismic performance of the building that occur during an earthquake.

This paper compares the seismic performance curves of an 8-story SRC building for 27 strong earthquakes that occurred in Japan between 1998 and 2012. The results demonstrate the practicability of using S_a-S_d curves to evaluate the damage to real buildings using data collected during strong earthquakes.

Keywords: Fundamental frequency, Performance curve, SRC building, Strong earthquake records, Wavelet transform technique

固有周波数, 性能曲線, 鉄筋コンクリート, 強震記録, ウェーブレット変換

1. Introduction

Rapid and quantitative evaluation of earthquake damage is important for seismic retrofit and avoidance of secondary hazards caused by aftershocks. Recent advances in the technology used in accelerometers have made good-quality measurement data available for this purpose. The data recorded by seismic accelerometers installed in buildings have been used in damage evaluation. In addition, damage evaluation methods have been developed to identify both the location and the degree of damage. In general, earthquake damage detection methods are based on monitoring changes in the dynamic characteristics (vibration frequency and modal shapes) of structures¹⁾.

However, it is difficult to understand how the fundamental frequency and seismic capacity of a structure change during an earthquake using methods based on dynamic characteristics. The fundamental frequency does not always reliably reflect the damage condition of a building due to a specific earthquake. Changes in the fundamental frequency are influenced by many factors, such as soil-structure interaction (SSI) effects²⁾ and the

local weather³⁾ (high temperature, heavy rainfall, and wind). A reduction in the fundamental frequency has been observed even in cases when no damage could be detected with a vibration test³⁾. Furthermore, modal shapes are not reliable for use in damage detection¹⁾.

In recent years, a new type of earthquake damage detection method based on wavelet technology and wave propagation has been developed by Maria I. Todorovska and Mihailo D. Trifunac^{4), 6), 7)}. In this method, a building is modeled as a layered shear beam, and structural damage can be detected from the distribution of shear and torsional wave velocities, which can be directly related to the structural rigidity along the building's height. The advantages of this method, including its robustness and insensitivity to SSI effects and local weather, over methods based on monitoring changes in the dynamic characteristics have been well described⁸⁾.

An earthquake damage evaluation method based on the S_a-S_d curve (seismic performance curve) concept was developed recently. This method uses wavelet transform

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technology (WTT) to extract the fundamental mode responses of a structure⁹⁾. The effectiveness of the $S_a - S_d$ curve approach to assessing the damage caused by large structural deformations has been confirmed in a shaking table test. The measured performance curve largely agreed with the computed curve calculated using WTT¹⁰⁾. In contrast to traditional dynamic characteristics-based methods, the $S_a - S_d$ curve method was found to be able to detect changes in the fundamental frequency from small responses to the maximum response and to identify the seismic capacity points of the measured building.

However, there has been little applied research on how to use $S_a - S_d$ curves to evaluate the seismic capacity of real steel-reinforced concrete (SRC) buildings. In the paper, we present an evaluation strategy for using $S_a - S_d$ curves to evaluate the seismic capacity of real buildings and the damage suffered by such buildings in real earthquakes. We also describe how to use $S_a - S_d$ curves to explain and evaluate changes in the fundamental frequency of a real building—the Building Research Institute (BRI) annex building (an 8-story SRC building) using earthquake response measurement data.

2. The case study

2.1 BRI ANNEX building

The BRI annex building is an eight-story SRC building with a base floor underground (B1F). The building is supported by a gravity foundation and connected to the main building through a nonstructural passageway, as shown in Figure 1(a). The height of the building (from the ground level to the 8th floor) is 28 m, and

the depth of the underground portion is 8.5 m. An instrument cabin consisting of a steel structure is located on the 8th floor. The building is instrumented with 11 accelerometers, each of which can record accelerations in 3 directions: east–west (taken here to be the X direction), north–south (the Y direction), and up–down (the Z direction). The locations of the measurement points in the building are shown in Figures 1(b) and 1(c).

The responses of the BRI annex building to seismic activity have been monitored since it was built in 1998. Kashima et al.^{11), 12)} carried out a series of studies on the SSI effects, ground motion and dynamic characteristics of the building, using earthquake response measurement data. The findings from these studies can be summarized as follows: (a) the rocking effect (8–11%) is much larger than the swaying effect (2–3%), (b) the fundamental frequency of the building decreased to 70% of its initial value between 1998 and 2005, and (c) the decrease in the fundamental frequency was due to degradation of the superstructure’s stiffness. However, the research did not explain the reason for the degradation of the superstructure’s stiffness, such whether the building experienced structural damage during earthquakes that occurred between 1998 and 2005. In this research, we try to use $S_a - S_d$ curves to answer this question.

2.2 Earthquake response records

A total of 1,239 earthquake records were collected for the annex building between 1998 and 2012. During the same period, 141 earthquakes with peak ground accelerations (PGA) greater than 10 Gal and Japanese Meteorological Agency intensities (IJMA) greater than 1 degree occurred in Japan. In this paper,

Fig. 1 Measurement points in the building

Table 1 Strong earthquakes that occurred in Japan between 1998 and 2012

Time	Epicenter	Latitude	Longitude	Depth (Km)	M (Degree)	Dist. (Km)	PGA (gal)	IJMA (Degree)	Fundamental response	
									E-W	N-S
1998/06/24 23:52	S Ibaraki Pref.	140.1017	36.1500	68	4.7	3	19.3	2.6	rank5	rank5
1998/08/29 08:46	Tokyo Bay	140.0317	35.6283	65	5.2	56	10.1	2.1	rank5	rank5
1999/03/26 08:31	N Ibaraki Pref.	140.6183	36.4467	59	5.0	60	46.3	3.2	rank5	rank5
1999/04/25 21:27	N Ibaraki Pref.	140.6250	36.4517	59	5.2	61	36.9	3.1	rank5+rank6	rank5
2000/04/10 06:30	S Ibaraki Pref.	140.0633	36.1867	55	4.7	6	49.4	3.2	rank6	rank6
2000/06/03 17:54	NE Chiba Pref.	140.7483	35.6850	48	6.1	79	16.1	2.7	rank6	rank5+rank6
2000/07/21 03:39	Off Ibaraki Pref.	141.1217	36.5250	49	6.4	104	37.9	3.4	rank6	rank6
2003/05/26 18:24	Off Miyagi Pref.	141.6533	38.8167	72	7.1	330	29.2	3.2	rank6	rank6
2003/09/20 12:54	S Chiba Pref.	140.3033	35.2150	70	5.8	104	13.7	2.8	rank6	rank6
2004/10/06 23:40	S Ibaraki Pref.	140.0917	35.9850	66	5.7	17	54.5	3.8	rank6	rank6
2004/10/23 17:56	Chuetsu, Niigata Pref.	138.8700	37.2883	13	6.8	168	30.9	3.4	rank6	rank6
2004/10/23 18:34	Chuetsu, Niigata Pref.	138.9317	37.3033	14	6.5	165	18.1	3.1	rank6	rank6
2005/04/11 07:22	NE Chiba Pref.	140.6200	35.7267	52	6.1	67	25.9	3.3	rank6	rank6
2005/08/16 11:46	Off Miyagi Pref.	142.2783	38.1500	42	7.2	298	29.8	3.3	rank6	rank6
2005/10/19 20:44	Off Ibaraki Pref.	141.0417	36.3817	48	6.3	91	40.2	3.5	rank6	rank6
2007/07/16 10:13	Off Jochuetsu, Niigata Pref.	138.6083	37.5567	17	6.8	205	19.3	3.6	rank6	rank6
2008/05/08 01:45	Off Ibaraki Pref.	141.6067	36.2267	51	7.0	138	49.6	3.6	rank6	rank6
2008/06/14 08:43	S Inland Iwate Pref.	140.8800	39.0283	8	7.2	330	26.2	3.4	rank6	rank6
2009/08/09 19:55	S Off Tokaido	138.4033	33.1267	333	6.8	368	28.4	3.2	rank6	rank6
2010/03/14 17:08	Off Fukushima Pref.	141.8167	37.7233	40	6.7	235	12.0	2.5	rank6	rank6
2010/06/13 12:32	Off Fukushima Pref.	141.7950	37.3950	40	6.2	208	15.9	2.5	rank6	rank6
2010/11/05 19:14	S Ibaraki Pref.	139.8417	36.0617	45	4.6	22	20.6	2.5	rank6	rank6
2011/03/11 14:46	Off Sanriku	142.8600	38.1033	24	9.0	330	279.3	5.3	rank6	rank6+rank7
2011/03/11 15:15	Off Ibaraki Pref.	141.2650	36.1083	43	7.6	107	151.1	4.7	rank6	rank6
2011/03/19 18:56	N Ibaraki Pref.	140.5700	36.7833	5	6.1	85	80.8	3.9	rank6	rank6
2011/04/11 17:16	Hama-dori, Fukushima Pref.	140.6717	36.9450	6	7.0	105	118.1	4.6	rank6+rank7	rank6+rank7
2012/03/14 21:05	E Off Chiba Pref.	140.9317	35.7467	15	6.1	88	33.3	3.3	rank6+rank7	rank6+rank7

the 27 strongest of these earthquakes (see Table 1) were selected for use in the analysis. The sampling frequency of the accelerometers in the building is 100 Hz, and displacements were calculated using the Fourier integration method in this paper.

2.2.1 Calculation of the rocking motion

In general, the vertical motions of two points are used to calculate the rocking angle^{(11), (13)}. However, in this research, three measurement points in the basement were not located on the translational axes, so the vector method, which makes use of three points, was employed to calculate the rocking angle.

As shown in Figure 1(c), *BFN*, *BFE*, and *BFS* are the three points on the base mat. We can calculate the following two vectors:

$$\vec{p} = \overline{BFN} - \overline{BFE} \quad (1a)$$

$$\vec{q} = \overline{BFS} - \overline{BFE} \quad (1b)$$

The normal vector of the base mat $\vec{n} = (x_n, y_n, z_n)$ can be calculated as follows:

$$\vec{n} = \vec{p} \times \vec{q} \quad (2)$$

The normal vector $\vec{n}$ can be normalized to a length of 1. The rocking angle α is sufficiently small that $\cos \alpha \approx 1$, so $|z_n| \approx 1$. We can easily obtain the rocking angle in the E-W direction, $\alpha_x \approx x_n$, and the rocking angle in the N-S direction, $\alpha_y \approx y_n$.

2.2.2 Calculation of the deformation of the superstructure

The lateral fundamental-mode deformation of each floor can be calculated as follows (with α , u_i , and u_0 extracted using WWT):

$$R_{ri} = H_i \times \alpha \quad (3a)$$

$$q_{i1} = u_i - u_0 - R_{ri} \quad (3b)$$

Fig. 2 MDOF model simplified to an SDOF model

where R_{ri} is the rocking displacement of the i th floor, H_i is the height of the i th floor, α is the rocking angle, u_i is the measured lateral displacement of the i th floor, u_0 is the measured displacement of the base floor, and q_{i1} is the deformation of the i th floor for the fundamental mode, as shown in Figure 2(b).

In fact, only the motions of the first floor (u_1), the second floor (u_2), the fifth floor (u_5) and the roof floor (u_8) were measured. It is reasonably assumed that the fundamental mode is dominant, so we need to calculate the motions of the other floors as follows.

$$u_3 = \frac{h_2}{h_2+h_3+h_4}(u_5 - u_2) + u_2 \quad (4a)$$

$$u_4 = \frac{h_2+h_3}{h_2+h_3+h_4}(u_5 - u_2) + u_2 \quad (4b)$$

$$u_6 = \frac{h_5}{h_5+h_6+h_7}(u_8 - u_5) + u_5 \quad (4c)$$

$$u_7 = \frac{h_5+h_6}{h_5+h_6+h_7}(u_8 - u_5) + u_5 \quad (4d)$$

where h_i is the height of the i^{th} floor. The properties and motions of the MDOF system are summarized in Table 2.

Table 2 Masses, fundamental displacements and heights corresponding to floors of the MDOF system

i	Mass m_i (10^6kg)	u_i (m)	q_{i1} (m)	H_i (m)
8	1.12	u_8	q_{81}	34
7	0.74	u_7	q_{71}	28.9
6	0.77	u_6	q_{61}	25.2
5	0.93	u_5	q_{51}	21.5
4	0.78	u_4	q_{41}	17.8
3	0.78	u_3	q_{31}	14.1
2	0.84	u_2	q_{21}	10.3
1	1.26	u_1	q_{11}	6

3. Methodology and evaluation strategy

3.1 Seismic performance curve ($S_a - S_d$ curve)¹⁴⁾

For the BRI annex building, the representative displacement S_d was calculated using the deformation of the superstructure (Equation (5a)). The base shear force coefficient S_a was calculated using Equation (5b). The base shear force Q_B was calculated using Equation (7).

$$S_d = \Delta_1 \cdot \frac{\sum m_i}{M_e} \quad (5a)$$

$$S_a = \frac{Q_B}{M_e} \quad (5b)$$

$$\Delta_1 = \frac{\sum m_i q_{i1}}{\sum m_i} \quad (6)$$

$$Q_B = \sum m_i \cdot \ddot{u}_i \quad (7)$$

$$M_e = \frac{(\sum m_i q_{i1max})^2}{\sum m_i q_{i1max}^2} \quad (8)$$

Fig. 3 Fourier spectrum of each component of the roof floor displacement for the 2011 off the Pacific Coast of Tohoku Earthquake in N-S direction

M_e is the equivalent mass corresponding to the fundamental mode. We assumed that M_e remained unchanged during a single earthquake and can be calculated (using equation (8)) from the maximum response points (q_{i1max}) of the fundamental mode response. Mass ratios $M_e/\sum m_i$ in the range of 0.72 to 0.76 were calculated for the earthquakes considered in this paper.

The relationship between S_a and S_d (for the peak response points, for which the damping effect is zero) can be expressed as follows, with ω_f denoting the fundamental circular frequency of the superstructure.

$$S_a = \omega_f^2 \cdot S_d \quad (9)$$

The fundamental response (q_{i1} ; see Figure 2(b)) is calculated using the WTT technique. KOICHI KUSUNOKI & MASAOMI TESHIGAWARA⁹⁾ have presented the method for using WTT to calculate the fundamental response. As Figure 3 shows, the original response can be decomposed into components that contain different frequency range signals). Rank6 and Rank7 reflect the fundamental response of the building to the earthquake that occurred at 2011/03/11/14:46. This fundamental response can be used to calculate the $S_a - S_d$ curve (equation (5)). A performance skeleton curve that reflects changes in the seismic performance from small responses to the maximum response can then be obtained¹⁰⁾, as shown in Figure 4.

3.2 Damage evaluation strategy using the $S_a - S_d$ curve

One of the objectives of this research was to use the $S_a - S_d$ curve to determine whether the building was damaged in past earthquakes. $S_a - S_d$ curves for the building for strong earthquakes were calculated using WTT and SDOF model theory (Figure 6 and Figure 7). However, there are two problems associated with using $S_a - S_d$ curves for damage detection: (1) it is difficult to determine the initial stiffness of the $S_a - S_d$ curve and (2) the velocity effect (the damping force) is contained in the $S_a - S_d$ curve. To overcome these problems, we established the following analysis strategy.

Fig. 4 Performance curve using Rank6 and Rank7 for Figure 3 as the fundamental response

Fig. 5 Damage assessment using the S_a-S_d curve for the 2011 earthquake off the Pacific Coast of Tohoku Earthquake in N-S direction

Table 3 Deformation level for the S_a-S_d curve of the BRI annex building

Level (i)	1	2
D_{ai} (rad)	$\frac{1}{3200}$	$\frac{1}{200}$
Height of the building (m)	28	
S_{di} (10^{-3} m)	5.6	89.1

Step 1: Calculate the performance skeleton curve (S_a-S_d curve) using earthquake response measurement data collected during the earthquake, according to the method described in section 3.1.

Step 2: Define the two deformation levels shown in Table 3, $1/3200$ and $1/200$, where D_{ai} is the drift angle of the superstructure and S_{di} is the corresponding representative displacement (calculated using equation (5a)). The minimum acceleration recorded by the accelerometers in the building was $1 \text{ Gal}^{[10]}$; therefore, the drift angle $1/3200 \text{ rad}$ ($S_{d1}=0.0056 \text{ m}$ for the building) is accurate and sufficiently small for the first level. The second-level $1/200 \text{ rad}$ is larger than the maximum response for the earthquake shown in Figure 8 (for example, the maximum S_d is 0.065 m , which is less than $S_{d2}=0.0891 \text{ m}$).

Step 3: Select the peak response points inside or outside the

deformation level 1 (S_{d1}) as the “initial points”—for example, N1 and P1 in Figure 5. The maximum peak response points (N2 and P2 in Figure 5) are selected as the “final points”. The fundamental frequencies of the “initial points” and the “final points” are compared to assess whether damage occurred. As Figure 5(c) shows, the fundamental frequency decreased from approximately 1.35 Hz to approximately 1.1 Hz , which means that earthquake damage occurred.

In this research, some large earthquakes (E-W direction: X01–X11, N-S direction: Y01–Y04, YA–YD and Y08–Y11; see Figure 8) with larger S_d values than those of the neighborhood earthquakes were studied using the damage evaluation strategy mentioned above.

4. Results and analysis

Using the S_a-S_d curve evaluation strategy described in section 3.2, S_a-S_d curves for several strong earthquakes were developed and are shown in Figure 6 (E-W direction) and Figure 7 (N-S direction). The fundamental frequencies calculated using the “initial points” (for example, N1 and P1 defined in Figure 5) and “final points” (for example, N2 and P2 defined in Figure 5) of the S_a-S_d curve (where “Positive direction” refers to the points in the positive direction and “Negative direction” refers to the

Fig. 6 Comparison of S_a-S_d curves in the E-W direction

Fig. 7 Comparison of S_a-S_d curves in the N-S direction

points in the negative direction) are summarized in Figure 9. For smaller earthquakes whose maximum representative displacement S_d is less than S_{d1} (for example, X01–X04, Y01–Y03, YA, and YB), we calculated the fundamental frequency using the maximum response points only. For larger earthquakes (such as X05–X11, Y04, YC, YD, and Y08–Y11), we used the points (“initial points” and “final points”) described previously. However, for the earthquake X09 or Y09, which was an aftershock that occurred approximately 30 minutes after the earthquake X08 or Y08, only the maximum response points were used to calculate the fundamental frequency.

For the E–W direction, we found that earthquake damage might have occurred in earthquake X08 (X09 was the aftershock of X08). The fundamental frequency decreased from approximately 1.30 Hz to 1.09 Hz (see Figure 9 (a), X08 and X09), as shown by the $S_a - S_d$ curve and Figure 6 (c) (X08 and X09). We also found that no severe earthquake damage occurred in the other earthquakes considered, such as X01–X07, X10, and X11.

For the N–S direction, we found that earthquake damage occurred in earthquake Y08 (the fundamental frequency decreased from approximately 1.35 Hz to 1.05 Hz (see Figure 9(b), Y08 and Y09 and Figure 7(c), Y08 and Y09). We also found that no severe earthquake damage occurred in the other earthquakes considered, such as Y01–Y04, YA–YD, Y10, and Y11.

Overall, the fundamental frequency decreased from approximately 1.9 Hz to approximately 1 Hz (a decrease of approximately 47.4%), but the decrease in the fundamental

frequency caused by earthquake damage was only approximately 11.1% in the E–W direction and 16.32% in the N–S direction. For some smaller earthquakes (X01–X07, X10, and X11; Y01–Y04, YA–YD, Y10, and Y11), the $S_a - S_d$ curves (Figures 6(a) and (b) and Figures 7 (a) and (b)) are linear. However, the $S_a - S_d$ curves for these earthquakes show that the seismic capacity of the superstructure decreased substantially, as shown in Figure 9. The fundamental frequency decreased from approximately 1.9 Hz to approximately 1.36 Hz in the E–W and 1.30 Hz in the N–S direction between 1998 and 2010 (as Figure 6 (a)(b) and Figure 7(a)(b) show, the $S_a - S_d$ curves are linear), which is consistent with the results calculated by Kashima¹¹⁾. We conclude that the decrease in the superstructure stiffness (the fundamental frequency) over this time period was not caused by severe earthquake damage but other unknown reasons such as material aging.

5. Conclusions

The seismic performance curve ($S_a - S_d$ curve) concept was used to evaluate earthquake damage in a real SRC building using the evaluation strategy described in this paper. SSI effects (rocking motion and swaying motion) were eliminated in the calculation of the representative displacement S_d , and fundamental responses were extracted using wavelet transform technology. The conclusions reached can be summarized as follows.

- (1) Changes in $S_a - S_d$ curves (which reflect changes in

Fig. 8 S_d for 27 strong earthquakes

Fig. 9 Evaluation of S_a-S_d curves

seismic capacity) can be observed using measurement data. The fundamental frequency of the BRI annex building decreased from approximately 1.90 Hz to approximately 1 Hz between 1998 and 2012. This decrease was mainly caused by a decrease in the seismic capacity of the superstructure. The contribution of earthquake damage to the decrease in the fundamental frequency was approximately 11.1% in the E-W direction and 16.32% in the N-S direction. This damage occurred during the earthquake that occurred in 2011 off the Pacific Coast of Tohoku. Although no severe earthquake damage occurred between 1998 and 2010, the fundamental frequency of the superstructure decreased substantially (the superstructure stiffness decreased to approximately 50% of its initial value).

(2) The damage evaluation method based on the S_a-S_d curve concept is simple and practicable. A simplified S_a-S_d curve can help us understand changes that occur in the seismic capacity of a building in real time. Nevertheless, even though linearity and nonlinearity in S_a-S_d curves can be observed, the initial stiffness of the curve is difficult to establish numerically. Therefore, the damage evaluation strategy presented in this paper is considerably appropriate for cases in which the story drift angle is sufficiently large (for example, when the story drift angle of the building is greater than $1/3200$ rad).

The evaluation method described in this paper needs to be adapted to other types of buildings, and its accuracy needs to be refined in the future. The building considered in this study is an

8-story SRC building for which the applicability of the SDOF model is reasonable. For high-rise buildings and irregular buildings, much more complex mechanisms (such as high-mode effects, torsional effects) will influence the earthquake responses. Therefore, the applicability of the S_a-S_d curve method should be evaluated for these types of buildings. In addition, it is possible to determine the damage points using the S_a-S_d curve, and the residual displacements also need to be studied further in the future.

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和文要約

1. はじめに

建築構造物の地震時の損傷評価は、余震での二次災害の防止、建物の効率的な維持管理という観点から重要である。既往の研究では、伝達関数の分析に基づく固有振動数や固有モード形といった構造物の動特性の変化に着目した損傷評価法が主流である¹⁾。しかし、これらの手法では地震中の固有振動数の変化を捉えることができず、構造物の損傷レベルを適切に判定することが難しい。

勅使川原、楠ら⁹⁾¹⁰⁾は、限界耐力計算等で用いられている建物の代表的な荷重変形関係である建物性能曲線に着目し、地震観測記録から建物性能曲線を精度よく算定する方法について検討している。本手法を用いて建物性能曲線を表示することで、地震中の建物の動特性の変化を把握することができ、迅速な残余耐震性能評価が可能となることが期待される。本研究では、SRC造8階建建築物の実観測記録を用いて建物性能曲線を算定し、性能曲線を用いた損傷評価の可能性について検討する。特に、動的相互作用や高次モードの影響の排除する方法や、有意な建物性能曲線を算出する方法、時系列的に見た建物性能曲線の変動から構造物の損傷を評価する方法について検討している。

2. 計測概要

対象建物は、8階建SRC造建築物である¹¹⁾¹²⁾。1998年の竣工当初から強震観測が行われており、鹿島ら¹¹⁾¹²⁾の研究により、ロッキング挙動がスウェイ挙動よりも大きいこと、固有振動数は2005年までに竣工時の70%程度まで低下していること、その低下の要因は上部構造の剛性の変化であることが示されている。本研究では、2012年までに観測された強震記録を分析し、時系列での動特性の変化だけでなく、地震による損傷の有無についても明らかにする。

解析に用いる観測記録は、2012年までに記録された1239個の記録のうち、比較的大きな応答を示す27個の記録とする。ロッキング挙動の影響を取り除くため、最下階床位置の3点の計測記録から2次元的なロッキング応答を算定し、それを上部構造の応答記録から取り除いた。

3. 性能曲線の算定方法

代表加速度 S_a 、代表変位 S_d は、加速度記録で得られる変位分布が建物の代表的なモードであると仮定することで、式(7)のように得られる。応答記録に含まれている高次モードやノイズの影響を排除するため、ウェーブレット変換を用いる。ウェーブレット変換は、記録を複数の周波数範囲（ランク）の成分に分割することができるため、主要な成分を含むランクのみを用いることで高次モードやノイズの影響を排除できる。

得られた性能曲線は、多折れ線にモデル化することで構造物の動特性の変化を把握する。本論文では、計測機器の精度や、SRC造構造物の一般的な挙動に配慮して、層間変形角レベルで、 $R=1/3200$ までの応答を初期応答と仮定し、初期応答の平均剛性と、最大層間変形角時の剛性を比較することで損傷レベルを判定する方法を提案する。

4. 解析結果

建物性能曲線を算定した結果の一例を Fig. 6,7 に示す。また、各地震での初期剛性と最大層間変形角時剛性を比較した結果を Fig.9 に示す。E-W 方向、N-S 方向とも、X08 地震時に損傷が生じたこと

が確認でき、その他の地震では損傷は確認されない。竣工時から2012年までに、固有振動数が初期固有振動数 1.9Hz から 1.0Hz まで低下（約 47%の低下）しているが、そのうち地震時の損傷によるものは E-W 方向で 11.1%、N-S 方向で 16.3%と考えられる。建物性能曲線からは、上記以外の地震時の損傷は確認されないことから、竣工当時から固有振動数の低下は、経年劣化等の要因が主要因となっていると考えられる。

5. まとめ

(1) 建物性能曲線から、固有振動数が 1.9Hz から 1.0Hz まで低下していることが確認された。また、そのうち地震時の損傷によるものが約 11%であることも確認された。

(2) 建物性能曲線を用いることで、容易に建物の状態を把握することができることが確認された。ただし、初期応答とみなす変形レベルの設定には、より詳細な検討が必要である。

本対象建物は比較的低層の、整形建物であるため、適用性が高かった。今後は、高層建物や非整形建物、高次モードやねじれモードの卓越する複雑な建物に対する適用可能性について検討する。

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