論文 / 著書情報
Article / Book Information

Title(English)	Vibration-based Damage Detection of a High-rise Steel Building before and after the E-defense Shaking Table Test
Authors(English)	Kenji Kanazawa, Haruyuki Kitamura, Daiki Sato, Masashi Morimoto, Masaru Ono, Takuya Nagae
Citation(English)	5th World Conference on Structural Control and Monitoring, , ,
Pub. date	2010, 7

VIBRATION-BASED DAMAGE DETECTION OF A HIGH-RISE STEEL BUILDING BEFORE AND AFTER THE E-DEFENSE SHAKING TABLE TEST

Kenji. Kanazawa
Central Research Institute of Electric Power Industry (CRIEPI), Chiba 270-1194, JAPAN
kanazawa@criepi.denken.or.jp

Haruyuki. Kiramura, Daiki.Sato, Masashi Morimoto, Masaru Ono
Tokyo University of Science, Chiba278-5810, JAPAN
kita-h@rs.noda.tus.ac.jp

Takuya Nagae
National Research Institute for Earth Science and Disaster Prevention, Hyogo 673-0515, JAPAN
nagae@bosai.go.jp

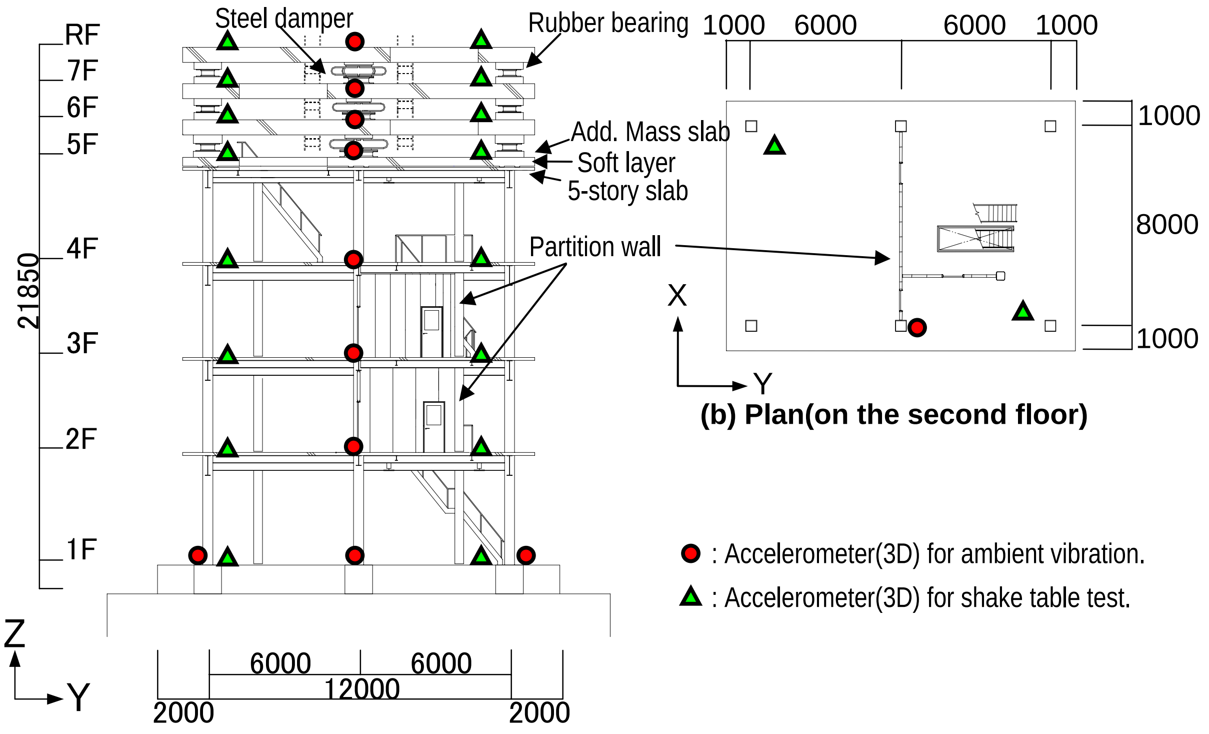
Abstract

Changes in modal properties of a high-rise steel building specimen on the full-scale shaking table test are evaluated from early stage of the construction to the test, which are expected to employ as damage index of structural health monitoring. The shaking table test is conducted for investigating structural performance of a ordinary high-rise building subjected to earthquakes which has long duration and large velocity amplitude, where the lower four stories of the real-size building is used as the specimen, and on the above portion a multiple-layer mass-spring system is installed to adjust and elongate the fundamental period of the specimen. The changes in the modal properties are evaluated from continuous ambient vibration test and shaking table test, which correspond to damage detections using ambient vibration and earthquake observation for the real building. The results are summarized as follows: (1) Changes in natural frequency due to construction works and seismic experience can be detected clearly. (2) Of these changes, the amount of the changes after weak seismic experiences without severe damage is larger than that after occurring severe damage: e.g., damages at several edges of the lower-flange of the girders. Thus a discrimination scheme of the changes between non-severe damage state and severe damage state must be needed in the future problem.

Introduction

Accurate and quick damage detection of a quake-stricken building helps early recovery of urban facilities after a huge earthquake. Visual inspection has been widely employed for judging whether the building can be continuously used or not; however it might be difficult to find damage in large or complex structures just after a huge earthquake. As an alternative to the visual inspection, vibration-based damage detection (VBDD) is promising for seismic diagnosis of the building, where the modal properties before and after the earthquake are compared as damage indicators (Retter, 1993). A lot of VBDD techniques including sensor technology, data processing and system identification have been proposed in the last two decades, and review papers have already been published in the scientific journals (e.g. Salawu, 1997; Housner et al., 1997). Practical examples of the VBDD have been also reported, where the modal properties are successively evaluated to grasp the present structural state (e.g. Kanazawa, 2005; Clinton, 2006; Kanazawa et al., 2006; Ookuma, et al., 2008).

Such research activity for supporting the VBDD is relatively high: however, the damage criterion for evaluating the extent of damage occurred from modal properties is not well-established yet, which is necessary to judge by VBDD whether a quake-stricken building can be continuously used or not. To develop the damage criterion, we need a complete set of lifetime records before and after seismic experiences. *Real* data are most desirable, that is, a complete set of *real*-vibration records on a *really* used and *really* damaged building experienced *real*-earthquakes. However, it is not likely that we get the *real* data, because there is very little chance that the building we set up vibration measuring equipment is damaged by earthquake motions.



(b) Plan(on the second floor)

(a) Section

Figure 1. The high-rise steel building specimen, Unit :mm.

To gather useful records for the development on the damage criterion, the authors have conducted ambient vibration based-damage detection of a full-scale test specimen on the shaking table “E-defense” under the cooperation of the Hyogo Earthquake Engineering Research Center, the National Research Institute for Earth science and Disaster prevention (NIED). With use of the E-defense, many types of shaking table tests have been conducted by using real-size or closely real-size structures (Nakashima, 2008). Most of the specimens are set up as almost entire structure system, and given realistic ultimate seismic loads: therefore at the end of the shaking table test the specimen could get similar damage state to a real building. Also, the ambient vibration level of a full-scale specimen is the same as the real building, therefore, such results on a full-scale specimen can be considered to be the same as those of the real building.

Motivated by what mentioned above, the authors have measured and reported the results on the ambient vibration test for a collapse test of four-story steel moment frame (Kanazawa and Kirita, et al., 2008), and for three full-scale bridge structure tests (Nagata, 2009a; Nagata, 2009b), and so on. In this paper, we will introduce an example on the high-rise building specimen, where changes in modal properties due to construction process or seismic damage are investigated effectively utilizing the chance of the full-scale shaking table test on the E-defense project.

Outline of the shaking table test and specimen

By the middle of this century, huge oceanic earthquakes are likely to hit the Pacific coast of Japan, which have periodically occurred with the interval of a few hundred years. Earthquake of such type causes ground motion of long-period, long-duration and large velocity. The shaking table test conducted by E-defense to investigate the actual structural performance of a general high-rise building subjected to such oceanic earthquakes is one theme of the research project promoted by the Ministry of Education, Culture, Sports and Technology in Japan (Nagae, et al., 2009).

The specimen of a steel structure was designed with the prototype of a 21-story steel moment building, and shown in Figure 1. The specimen is the lower four stories of the prototype steel building and on the above portion a multi-layer mass-spring system is set to adjust and elongate the fundamental period of the specimen. The lower portion of the specimen is one-bay and one-bay steel frame with four concrete slabs on the 2nd to 5th floors, and the basement is rigidly fixed on the shaking table. The two kinds of real partitions are installed on the 2nd and the 3rd floor to investigate their seismic performances and damage states, which are autoclaved-lightweight-concrete (ALC) wall on the 2nd floor, and light-gauge steel grid with plaster panel on the 3rd floor, respectively. As shown in Figure 1, the 5th floor slab composed of three layers of the 5-story slab, the soft layer and the additional mass slab: the lower layer of the 5-story slab is the 5th floor concrete slab itself; the upper layer of the additional mass layer is an thick concrete slab whose weight is substitute for the 6th- and 7th-floor weight of the 21-story prototype building; the middle layer of the soft layer is made of expanded polystyrene foam to block out the load transmission between the upper and the lower slabs. The effect of the “block out” will be discussed later in Figure 4. The upper portion of the mass-spring system, on the other hand, is composed of three mass slab, natural rubber bearings (NRBs), and U-shaped steel dampers. The specifications of the mass-spring system was designed so that the fundamental frequency of the entire specimen is set to 0.46 Hz (2.19 s).

The shaking table test was conducted on March 17 to 21, 2008, as shown in Table 1. Two kinds of tests were conducted alternately in the series of the whole tests, which are the seismic performance excitation and the system identification excitation. In the seismic performance excitation several artificial earthquake inputs were employed for investigating the structural performance against huge oceanic earthquake envisioned in the near future: e.g., the Higashi-Oogijima wave, the San-no-maru-wave. An observed earthquake input, the El Centro wave, which has been widely used in seismic design of the high-rise building, was also employed for investigating the behavior against a typical seismic design wave. Of these inputs the San-no-maru wave affects the seismic performance of the specimen, which has the longest-duration and the highest intensity in the long-period range. The system identification excitation was also conducted before and after each seismic performance excitation, from which dynamic behavior of the specimen is evaluated in the elastic range.

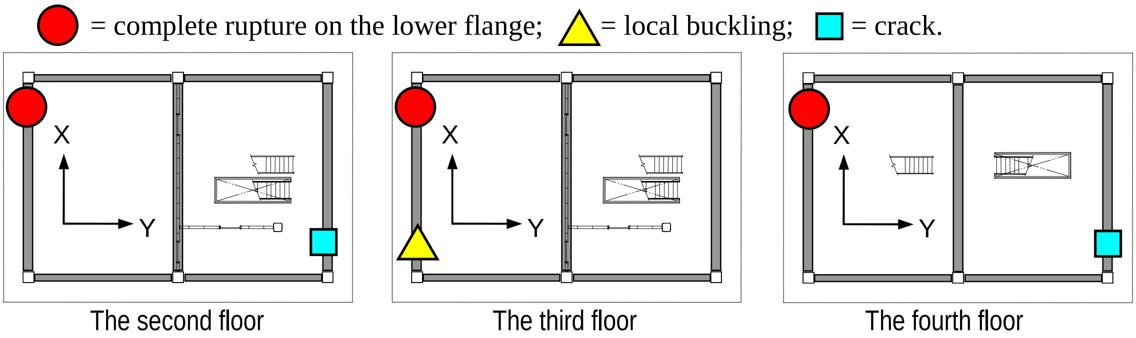
The schedules for the shaking table test and the state of the specimen are as follows. On the first day the excitations for the level-1 earthquake motions were conducted to investigate whether the specimen behave within the elastic range. Here, the level-1 earthquake motions is defined as the small-amplitude input in seismic-resistant design, which can occur once or more within the in-service period of a building, and against such earthquake motions the building must behave elastically. In the test the peak ground velocity (PGV) was set to less than 25 cm/s as usual in Japan. By visual inspection after the first day, we found no damage in the main structural members, but some hairline cracks appeared on the concrete slabs of the 2nd to 4th floor. We also confirmed that the relations between force and displacement were strictly linear on all the stories during the excitations for the level-1 earthquake motions. The facts indicate that no damage occurred in the first day.

On the second to third days the excitations for the level-2 earthquake motions were conducted to investigate the structural performance beyond elastic limit. Here, the level-2 earthquake motions is defined as the large-amplitude input in seismic-resistant design, which may hit a building with low possibility, and against such earthquake motions the building must not collapse but may behave elasto-plastically. In the test the PGV was set to more than 50 cm/s as usual in Japan. By visual inspection within the second to third days, we cannot find remarkable damage, but some yield lines appeared slightly on the surface of steel girders. According to the records against the level-2 earthquake motions, the relation between force and displacement became slightly in-elastic. The facts indicate that some steel members behaved beyond their elastic limits; however, they caused no severe damage.

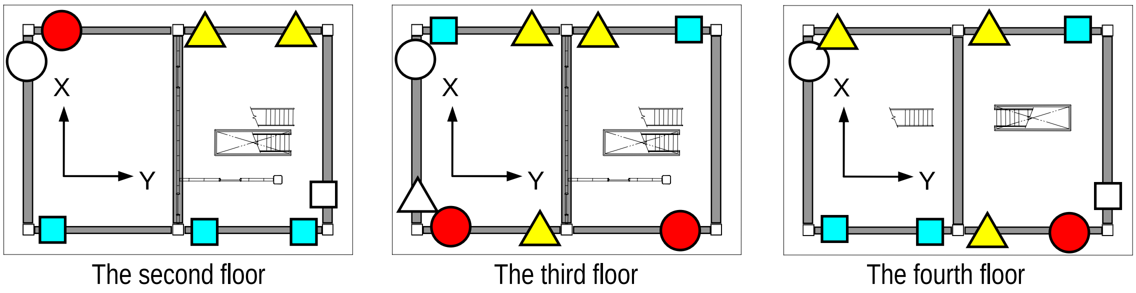
Table 1. Primary shaking table tests and the specimen states

Date: March 2008	Time	**[Abbr.]**Input wave(Input Direction) Target amplitude of the table input and its duration.	Specimen state. Six States:ST1 to ST6
17	13:35	**[WH1]** White noise excitation(XY), PGA = 100cm/s^2, Duration=250s.	No damage. **ST1** All steel members behaved in the elastic range. Hairline cracks appeared on the concrete slabs on the 2 to 4 floors. **ST2**
	14:04	**[WH2]** White noise excitation(XY), PGA = 200cm/s^2, Duration=250s.	
	14:33	**[WH3]** White noise excitation(XY), PGA = 300cm/s^2, Duration=250s.	
	19:03	**[HGO1]** Artificial earthquake of the Higashi-Oogijima wave(XY), PGV = 25cm/s, Duration=100s.	
	19:39	**[SAN1]** Artificial earthquake of the San-no-maru wave(XY), PGV = 20cm/s, Duration=300s.	
	20:11	**[EL1]** Observed earthquake of the El Centro wave(XY), PGV = 25cm/s, Duration=50s. **ST1**	
18	11:00	**[WH1]** and **[WH2]**	Almost no damage. Some steel members behaved in-elastically (very slightly). Cracks appeared on the slabs.
	15:47	**[EL2]** The El Centro wave(XY), PGV = 50cm/s	
19	10:59	**[WH1]** and **[WH2]**	Almost no damage. Some steel members behaved in-elastically. Cracks appeared on the slabs. **ST3**
	15:02	**[HGO2]** The Higashi-Oogijima wave(XY), PGV = 50cm/s	
21	11:35	**[WH1]** and **[WH2]**	
	15:00	**[SAN2-1]** The San-no-maru wave(XY),PGV=50	*Serve damage appeared in the X-axis girders as shown in Fig.2(a). **ST4**
	19:28	**[WH1]** and **[WH2]**	
	19:58	**[SAN2-2]** The San-no-maru wave(Y only),PGV=50	**ST5**
	20:05	**[WH1]** and **[WH2]**	
	20:44	**[SAN2-3]** The San-no-maru wave(Y only),PGV=50, which is the same case as the SAN2-2.	*Serve damage appeared in the Y-axis girders as shown in Fig.2(b). **ST6**
	20:56	**[WH1]** and **[WH2]**	

On the fourth day the long-duration excitations were repeatedly conducted for investigating the structural performance and damage state against huge oceanic earthquakes. In the first seismic excitation of the SAN2-1, several parts of damage appeared in the X-axis girders only, as shown in Figure 2-(a). The damage were located at the edges on the outer X-axis girders from the 2nd to 4th floors, the most severe damage were in the red-circle-marked parts where the lower flanges and the webs were cut completely along the welded joint of girder-to-column connection. Those severe damage appeared in the X-axis frame, whereas in the Y-axis frame, by contrast, no remarkable damage could find by visual inspection. Thus, additional seismic excitation tests were conducted for the sole Y-axis shaking. And then, in the second sole shaking of the SAN2-3, clear damage appeared in the Y-axis girders, which can be visual



(a) The first damage state after the 1st San-nomaru-wave excitation



(b) Final damage state after the 3rd San-nomaru-wave excitation

Figure 2. Damaged locations of girders

contact as shown in Figure 2-(b). It is notable, as shown in Figures 2-(a) and (b), that the damage were distributed uniformly from the 2nd to 4th floors: The fact shows that the distribution of the seismic energy absorption appeared homogeneous in the lower portion of the prototype, therefore, the specimen is considered to belong better-designed category against low-cycle fatigue problem.

In the paper, we labeled six structural states from “ST1” to “ST6” with referring to the seismic experiences of the specimen, as shown in Table 1. The ST1 state denotes the specimen is no seismic experience before the major shaking table test. From the ST2 to ST6 each boundary is determined by the typical level-2 earthquake motions.

Ambient vibration test and Modal identification

The ambient vibration test was continuously conducted from early stage of the construction to the end of the whole shaking table test, and eventually, the time span of the all record is about 53 days. Figure 3 shows the schedules of the ambient vibration test, the construction works and the shaking table test. The ambient vibration test began on January 29, 2008, when the specimen was in bare steel frame state and in the 4-story state. After the started, the 4-story prototype specimen was measured in the almost all construction works, where the structure was constructed outside the E-defense experimental facility. When the 4-story prototype moved onto the shaking table from the construction site, the ambient vibration test have been stopped shortly. After the 4-story prototype was set on the table, the ambient vibration test was restarted immediately. And then, the ambient vibration test was conducted continuously while the specimen was installed on the shaking table.

Eleven three-axial acceleration-meters were installed at the center of each floor and at the edges on the first floor, as plotted by red-circle in Figure 1. These eleven accelerometers were measured only for the ambient vibration from quiet vibration in midnight to noisy vibration under construction, thus whose measuring ranges, resolution and sampling rate are set to 25 cm/s/s, 24Bit and 200 Hz, respectively. Thus, the acceleration records of the ambient vibration test can sometimes exceed over the measuring range during the shaking table test: however, the accelerations during each shaking table test were measured by the E-defense project, where the accelerometers are located as plotted by green-triangle in Figure 1, therefore, it is notable that the all vibration records from tiny micro-tremor to strong earthquake motion were completely measured in the project.

Natural frequencies, damping factors and their mode shapes are identified by a cross spectrum based modal identification technique which is employed auto-regressive moving-average model (ARMA model) (Kanazawa, 2004; Kanazawa and Hirata, 2005). Here, the eigenvalues (proper-values) such as natural frequency and damping factors are identified from the X- and Y-components of acceleration records on the 5th floor, and then the complex modal vectors are identified from all components on the ambient vibration records. To obtain the modal property closely in time, almost all the records are divided by every 5 minutes with no overlapping; therefore, *almost all the estimates* of the modal properties are calculated by *every 5 minutes*. By using such condition, however, time-history in modal property during the shaking table test seems to be too sparse to investigate relation the changes and excitation events. Thus, only for the test period between March 17 to 22, the records are divided by every 5 minutes with one-fifth overlapping; therefore, *only in the shake test days* the estimates of modal properties are calculated by *every one minutes*. Further, using the ARMA model based-identification, we obtain both substantial eigenvalues and spurious eigenvalues: i.e., the substantial eigenvalues (True) are related to the modal properties of the building, whereas the spurious ones (False) are related to obscure the physical meaning. To choose only the substantial eigenvalues, we employ the modified Modal Assurance Criterion (MAC; Liven, 1988)-based Algorithms (Kanazawa et al., 2008).

Change in modal property in all specimen’s life

Modal property of the specimen can be well-estimated in about 53 days of almost all the specimen’s life. Figure 5 shows the vibrational amplitude of acceleration response on the 5th floor, and the first natural frequencies and their damping factors in the orthogonal two directions. Changes in natural frequencies due to the construction progress can be clearly detected, whereas changes in damping factors with construction seems to be able to detected unclearly. Here, we will discuss on change in natural frequency in detail. As an side, the acceleration response shown in Figure 4-(a) moved to high in daylight, which moved to low in midnight. The changes were caused by the construction works. And during the 4 days conducted on the shake test, the value is saturated in the limit of ambient vibration measurement.

The first natural frequencies in the X and Y direction trend to be close each other. In the bare state of only 4-sotry steel frame, on February 5, the first natural frequencies were 2.5 Hz. On February 6 to 7, after casting concrete slab twice, named as “the first and second cast”, the natural frequency decreased rapidly by about one-third of the bare steel frame state. The changes are caused by weight addition of the fresh concrete. There are more important fact than those weight-induced changes, that the frequency increased gradually within about three day after the casts. We are sure that this gradual increase caused by hardening of the concrete slabs. The similar change in natural frequencies was also observed on February 16, when the additional mass slab concrete was cast on the 5th floor, named as “the third cast”. As described the above description about the specimen, the 5th floor slab composed of three layers: the lower is the 5-story concrete slab itself; the upper is an additional mass slab whose weight is substitute for the two floor weights of the envisioned 21-story building; the middle expanded polystyrene foam layer is set to block out the load transmission between the upper and the lower slab. By composing of such three layer structure, at the third cast, the effect on the hardening of the additional mass slab cannot transmit to the 5th floor and the 4-story stated specimen, consequently, the natural frequencies cannot increase after the third cast. Similar change in frequency have been reported by the authors (Kanazawa et al., 2008).

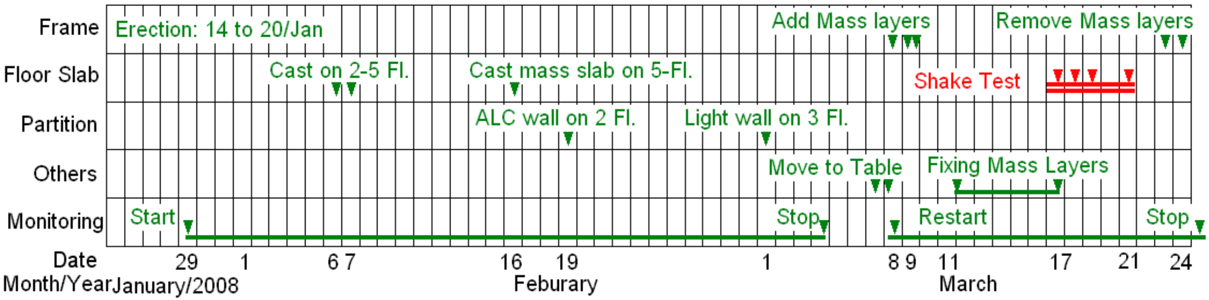
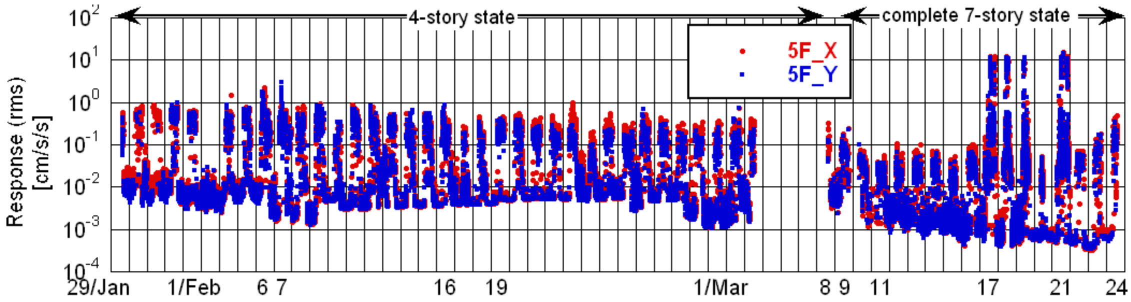
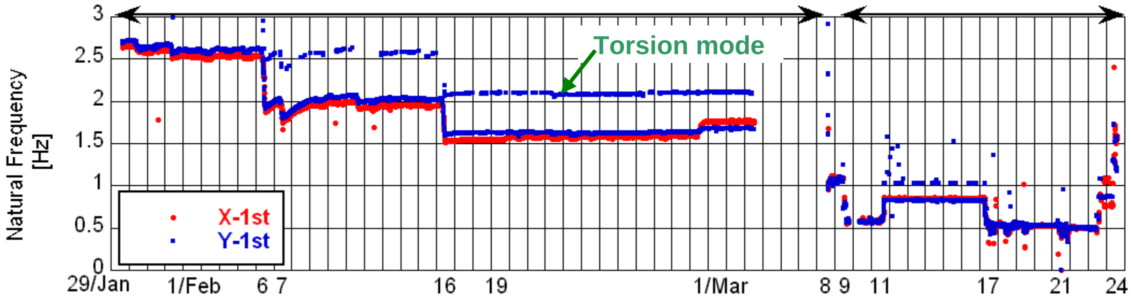


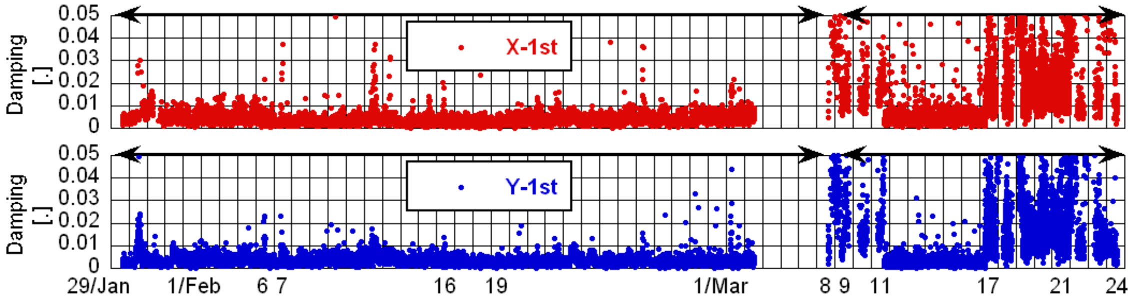
Figure 3. Schedule of Construction works and Tests.



(a) Root-means-square of acceleration response on the 5-floor.



(b) The first natural frequency.



(c) The first modal damping.

Figure 4. Time history of the response and modal property estimates.

The increase in natural frequency with hardening concrete slabs, observed here, must be an important attribute to develop the VBDD for steel structure. Because damage induced-change in stiffness of concrete is larger than that of steel. That is, damage induced-change in stiffness of concrete slabs might affect strongly change in natural frequency of a entire steel structure.

On February 19, the ALC inner partitions were installed on the 2nd floor, however, the natural frequencies did not change at all. On march 1, on the other hand, when the light partitions were installed on the 3rd floor, the natural frequency only in the X-direction increased rapidly by about 10 percent. As shown in the Figure 1-(b), the partition walls inside the building strongly act on the X-direction and the partitions installed on the 3rd floor are light in weight, whereas the partitions installed on the 2nd floor are heavy in weight.

Large changes in natural frequency appeared through March 4 to 9, because the specimen transformed from the 4-story steel prototype to the complete model with three additional mass-spring system. On March 11 to 17, while the three additional mass-spring system were rigidly fixed by steel bars, the natural frequencies remained higher than those in completion.

The shaking table test were conducted on March 17 to 21, however, the changes in frequency during test are smaller than those in the construction process. The change during the test will discussed on the latter section. After the shaking table test, while three additional mass-spring system was removed on March 23-24, the natural frequency increased to the contrary of the construction.

Modal property on the complete specimen

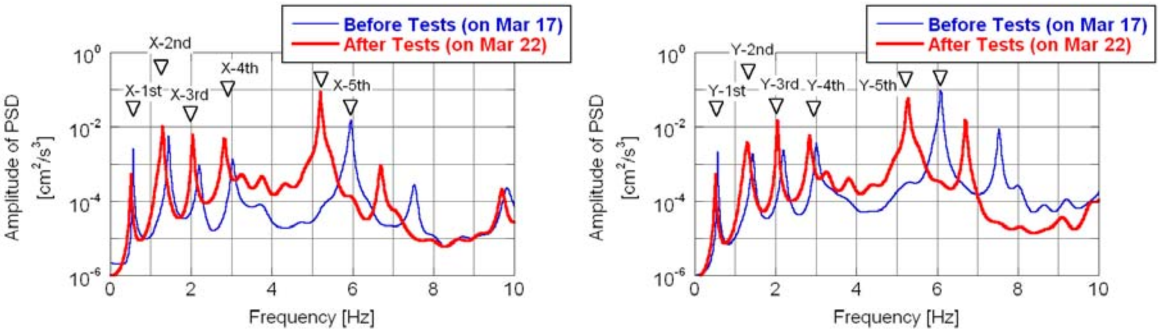
To clarify the basic feature of the recorded ambient vibration, an example of the auto power spectral density (PSD) are shown in Figure 5, where the 5th floor acceleration records were used before and after the whole shaking table test. The lowest five mode can be well-identified in our modal identification, whose resonance peaks appear sharply in the figures, named as from the X-1st to X-5th, or as from the Y-1st to T-5th. The five mode shapes are described in Figure 6. The lowest four mode shapes are seems to be equivalent to those of a four-degree-of-freedom (4DOF) system composed of the 5th to RF floors only, whereas the stories on the 2nd to 5th floor dependently oscillate in the same phase one another. The X-5th mode shape, on the other hand, has different characteristics, where the lower four stories behave in the general second mode shape of a 4DOF system, whereas the upper three floors on the 6th to RF floors cannot oscillate at all. Probably the X-5th mode cannot exist in the real flexible building without the contraction operation like the upper three stories of this specimen; however, since the deformation of the X-5th mode appears only on the lower four stories on which the damage occurred in the specimen, it is interesting for damage detection to investigate change in the 5th mode natural frequencies.

Comparing the before to the after auto-PSDs in Figure 5, we find that the frequency shift is larger in the higher mode; therefore, we apparently think that information from the higher mode is more useful for the VBDD. However, the authors do not think that is true. To investigate the fact, the frequency shift before and after the whole test are compared by using the records of construction and removal process of the specimen shown in Figure 7. Here, the 1st and 5th natural frequencies are compared with before and after the test (namely before and after damage) and with four different state of the specimen. The result shows in Figure 8, where numbers in percent terms denote the decreasing rates before and after the whole test in the same structural state and in the same mode. The decreasing rates in the 1st and 5th frequencies are similar to one another; therefore if estimate accuracies in the low and high frequencies are similar, the accuracies of the damage detection from both frequencies can be also similar to one another.

Change in natural frequency due to seismic experience

Relation between change in natural frequency and seismic experience will be discussed. Figure 9 shows time history of acceleration response and the first natural frequencies. In the figure the time history in each estimate are shown from 0 o’clock on March 17 to 24 o’clock on March 22, and the arrows point to the time of the major shaking tests. Estimates in the natural frequency seems to disappeared up to 8 o’clock on March 17, since the natural frequencies are high beyond the range of y-axes because the upper portion of the mass-spring system was fixed rigidly as shown in Figure 4 (b).

As shown in Figure 9 (b) and (c), the natural frequencies tend to decrease with the progress of the shaking table test. However, natural frequencies also fluctuated with change in acceleration amplitude. To



(a) X-component acceleration **(b) Y-component accelertion**

Figure 5. Auto power spectral densities of acceleration on the 5th floor before and after the whole shaking table test.

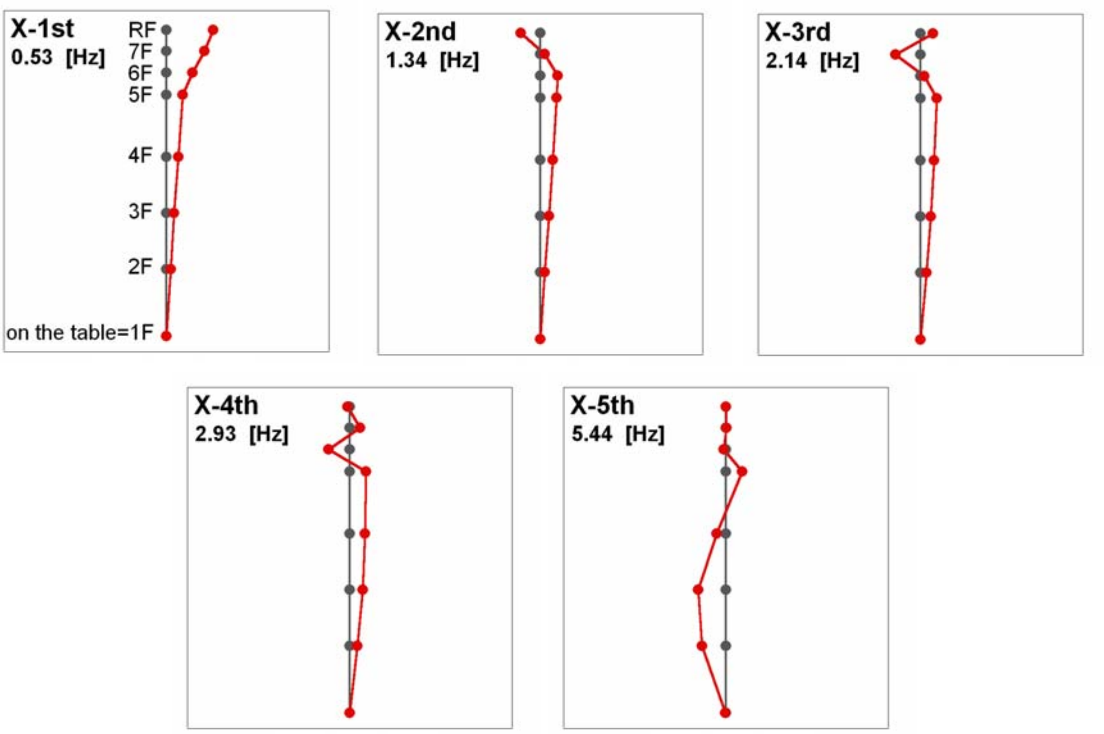


Figure 6. Examples of the mode shape in the X-direction at just completed, on March 17 before the shaking table test.

investigate the change in frequency only due to seismic experience, the five estimates in the natural frequencies are chosen so that the amplitudes of the acceleration are similar to one another. The chosen estimates indicate by the green-circle and the values of the estimates in the Figure 9 (b) and (c).

Comparing the five estimates, we found that the amount of the changes on the first excitation day is the largest of all changes, and that the changes can detectable on the fourth excitation day when the severe damage appeared in the SAN2-1 and the SAN2-3 excitations. The X-1st frequency was 0.567 Hz in no seismic experience state, which decreased to 0.533 Hz after the first excitation day; the X-1st frequency decreased by about 6.0 percent, even when all the excitations are so small that the specimen behaved

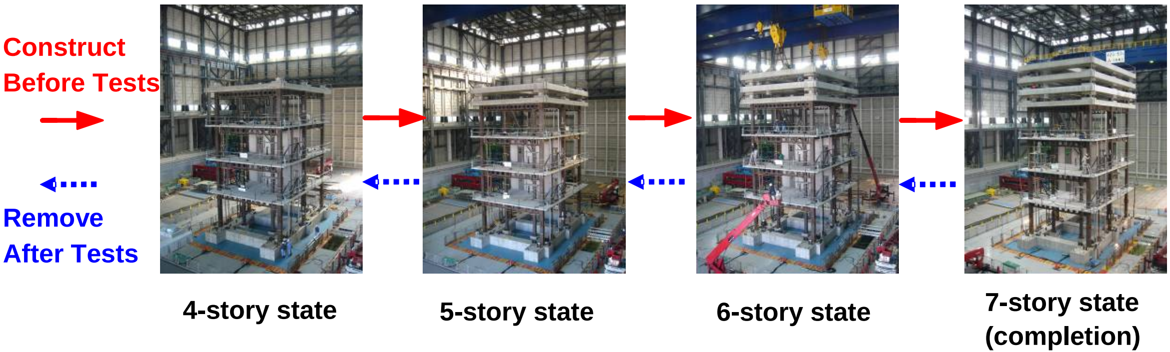
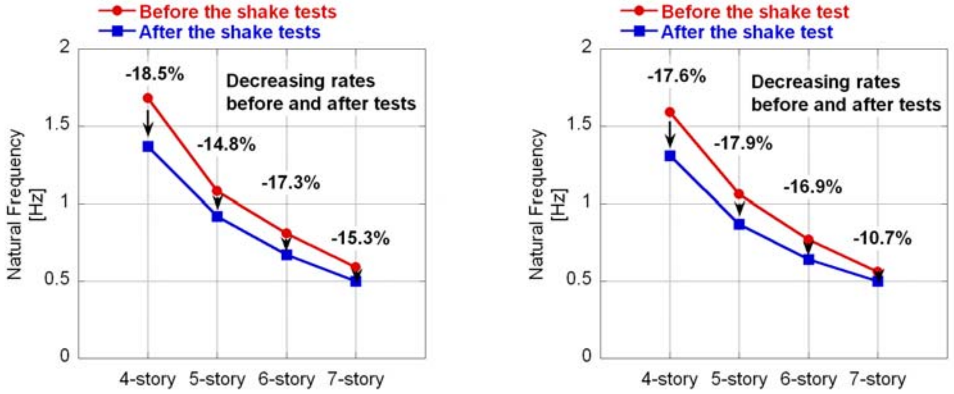
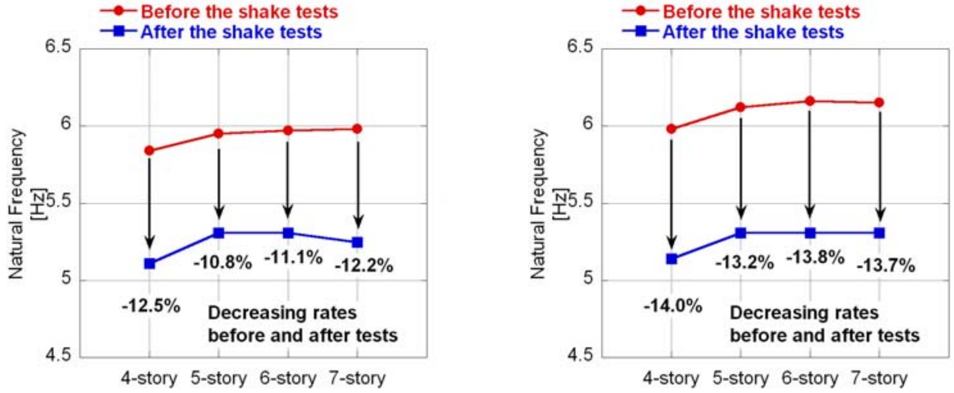


Figure 7. Construction and removal process of the specimen.



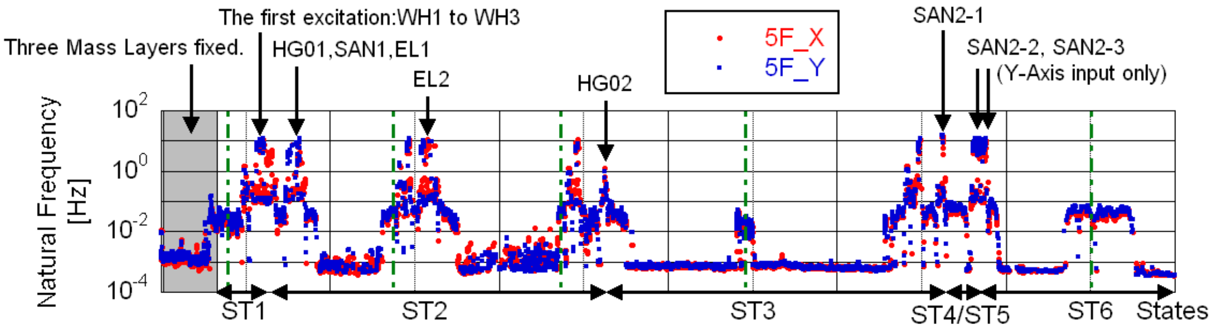
(a) The X-1st natural frequency (b) The Y-1st natural frequency



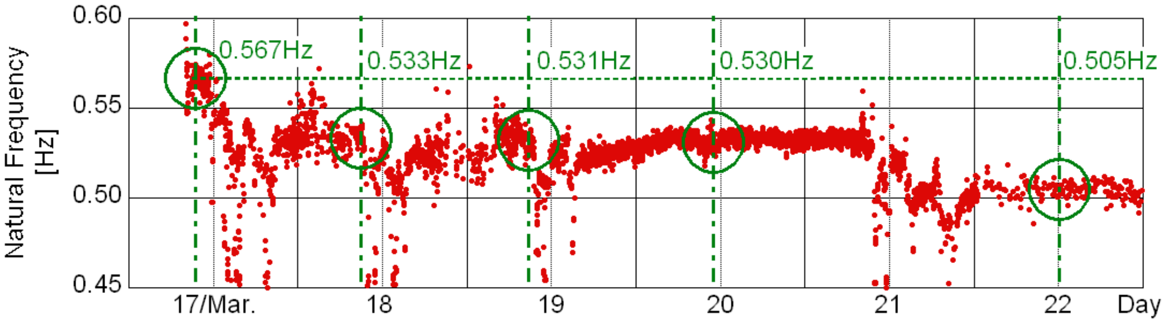
(c) The X-5th natural frequency (d) The Y-5th natural frequency

Figure 8. Changes in the natural frequencies with structural states, caused by installation of three mass layers or by seismic experience on the shaking table tests.

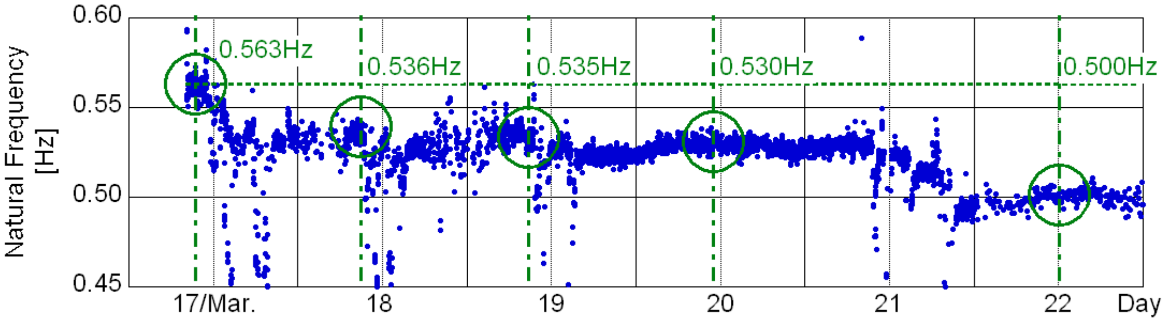
elastically. In the next two days on March 18 and 19 the X-1st frequency did not changed, even when the specimen was shaken by the level-2 earthquake inputs, which also behaved elasto-plastically. Between the fourth day the X-1st frequency decreased from 0.530 Hz to 0.505, whose decreasing rate was about 4.7 percent. The trend in the Y-1st frequency was quite similar that in the X-1st; ; the decreasing rates in the



(a) Root-means-square of acceleration response on the 5-floor.



(b)The first natural frequency in the X-direction(X-1st).



(c)The first natural frequency in the Y-direction(Y-1st).

Figure 9. Changes in the natural frequencies with progress on the shaking table tests.

Y-1st frequency was 4.8 percent in the first day, and 5.7 percent in the fourth day. As observed above, the changes in natural frequency are significant at the first excitation and at the severe-damaged excitation.

The similar frequency shifts are observed from the small-quake records. Figure 10 shows the amplitude dependency of the natural frequency, where the small dots are the result from the ambient vibration test, and the large square are the results from the system identification excitation using white noise wave. In the figure, the blue-color plots denote in the ST1 state when the specimen has no seismic experience; the green color plots denote in the ST3 state when the specimen experienced up to the level-2 earthquake and has no severe damage; the red color plots denote in the ST6 state when the specimen has several severe damage. As shown in Figure 10, we found that the frequency shifts are quite similar in both estimates from ambient vibrations and small excitations.

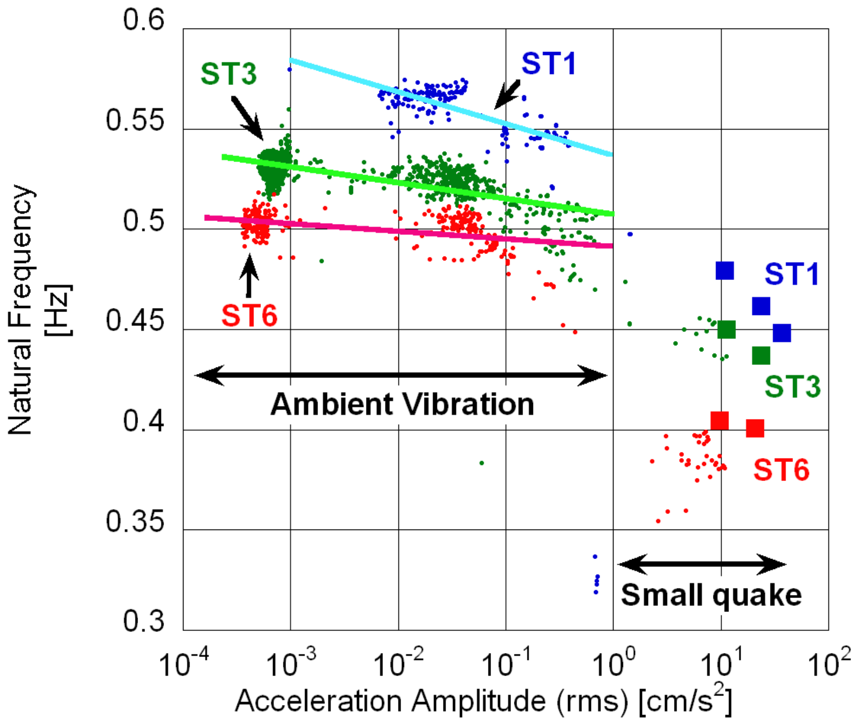


Figure 10. Changes in the amplitude dependency of the Y-1st natural frequencies, where the three seismic experience states are compared by different color plots; the dots and squares indicate estimates from ambient vibration and shake test.

The results shows that the frequency changes due to severe damage can be detectable by using the modal identification scheme. However, the results also shows that the frequency shifts within non-damaged state cannot be negligible in the VBDD. Thus a discrimination scheme of the changes between non-severe damage state and severe damage state must be developed in the future. The authors believe the changes within non-severe damage can caused by decreasing stiffness of the concrete slabs. To develop the discrimination scheme, further experimental researches are needed.

Conclusions

Vibration based damage detection (VBDD) is promising for quick diagnosis of the quake-stricken building, however, the damage criterion to evaluate the extent of damage detected from modal property change is not well-established yet, which are necessary to practical-use of the VBDD. In the paper, by utilizing the chance of the full-scale shaking table test, the modal properties of a high-rise building specimen were monitored from early stage of the construction to the end of the shaking table test. From the monitoring results, relations between modal property changes and earthquake experience were discussed. The results are summarized as follows:

1. Vibration records of the high-rise building specimen were obtained completely from tiny micro-tremor to strong earthquake motions. Modal property changes can be also well-estimated from all records. The records were obtained from the early construction stage of the building specimen, i.e., from the bare steel frame state, and the specimen experienced several earthquakes

small to large resulting in severe damage state; The record is useful to develop the damage criterion in the damage detection.

2. Changes in natural frequency during the construction works were investigated. It is notable that changes in natural frequency with hardening the concrete slab can clearly perceived from the continuous evaluation of the natural frequencies. The modal property of the steel building is largely affected by the stiffness of the concrete slabs, and the stiffness of concrete members is generally more sensitive to the seismic experience than that of steel members. Thus, further research is needed to evaluate relation between stiffness of the concrete stab and soundness of the entire steel building.
3. Changes in natural frequency during the shaking table test were investigated. The frequency changes due to severe damage can be detectable by using the modal identification scheme. However, the frequency shift within non-damaged state cannot be negligible in the VBDD. Thus, a discrimination scheme of the frequency shift between non-severe damage state and severe damage state must be developed in the future problem.

Acknowledgements

The authors gratefully thank Dr. Kazuta Hirata of Central Research Institute of Electric Power Industry for discussing the draft of the paper. This research is partially conducted on “the special project for Earthquake Disaster Mitigation in Tokyo Metropolitan Area” promoted by the Ministry of Education, Culture, Sports Science and Technology, Japan.

References

Rytter, A.(1993), “Vibrational based inspection of civil engineering structures,” Ph.D. T*hesis Department of building technology and structural engineering, University Aalborg*, Denmark.

Salawu, O.S.(1997), “Detection of structural damage through changes in frequency: a review,” *Engineering Structures*, **19**(9), 718-723.

Housner, G.W., et al.(1997), “Special issue: Structural control: Past, present and future,” *Journal of Engineering Mechanics*, ASCE,**23**(9), 897-971.

Kanazawa, K.(2005), “Natural frequency transition of a building during seismic retrofitting works”, *Proc. of the 23rd International Modal Analysis Conference*(IMAC), Paper No.67.

Clinton, J.F., S.C. Bradford, T.H. Heaton and J. Favela (2006), “The observed wander of the natural frequencies in a structure,” *Bulletin of the Seismological Society of America*, **96**(1), 237-257.

Kanazawa, K., N. Morikiyo and H. Kitamura(2006), “Dual vibration observation system for structural health monitoring,” *Proc of the 8th US National Conference on Earthquake Engineering*, Paper No.831.

Ookuma, N, Y. Etou, K. Kanazawa, K.Hirata.(2008), “Dynamic properties of a large arch dam after forty-four years of completion,” *Proc. of the 14h World Conference on Earthquake Engineering,* Beijing, China, Paper No. 05-03-0173.

Nakashima, M.(2008),”Role of large structural testing for the advancement of earthquake engineering,” *Proc. of the 14h World Conference on Earthquake Engineering,* Beijing, China, Paper No. K009.

Kanazawa, K., F. Kirita, H. Kitamura and Y. Matsuoka (2008), “Health Monitoring of 4story steel moment frame before and after the collapsed test,” *Proc. of the 14h World Conference on Earthquake Engineering,* Beijing, China, Paper No. S17-01-006.

Nagata, S., K. Kanazawa, S. Yabana, H. Ukon, K.Kajiwara(2009a), “A long-term ambient vibration monitoring on a shaking table test of a full-scale bridge, Part I: Detection of a modal parameter transition caused by flexural failure of a RC structure,” CRIEPI-Report, N08010 (in Japanese).

Nagata, S., K. Kanazawa, S. Yabana, H. Ukon, K.Kajiwara(2009b), “A long-term ambient vibration monitoring on a shaking table test of a full-scale bridge, Part II:A relationship between seismic damage levels and natural frequency reduction of RC structure,” CRIEPI-Report, N08010 (in Japanese).

Nagae, T., et al.(2009), “Development of frame test system to assess seismic performance of high rise buildings,” *Journal of structural and construction engineering,* Architectural Institute of Japan, 74(640), 1163-1171(in Japanese).

Kanazawa, K.(2004), “Application of cross spectrum based modal identification to output-only records of ambient vibration,” *Proc. of the 13th World Conference on Earthquake Engineering,* Vancouver, Canada, Paper No.3068.

Kanazawa, K. and K. Hirata(2005), “Parametric estimation of the cross spectral density”, *Journal of Sound and Vibration,* **282**(1),1-35.

Liven, N.A.J. and D.J. Ewins(1988), “Spatial correlation of mode shapes, the coordinate modal assurance criterion (COMAC),” *Proc. of the 6th International Modal Analysis Conference*(IMAC), 690-695.