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THE AMPLITUDE DEPENDENT DYNAMIC CHARACTERISTICS OF AN EXISTING BUILDING BEFORE AND AFTER SEISMIC RETROFIT

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SUMMARY

Amplitude dependency of the dynamic characteristics of an existing 9-story RC building with uni-axial eccentricity is addressed in the paper, based on the various kind of observations and vibration tests such as earthquake observation, micro tremor observation and forced vibration tests using various exiting forces like excitor machine, man induced load and the air gun type impactor, obtained these 35 years after its completion.

The building was damaged during the 1978 Miyagi-ken Oki earthquake. During this earthquake, the maximum acceleration recorded at 9th floor exceeded 1G, 1040 cm/s^2, and the deflection angle of the building was of the order of 10^{-2} radian. The seismic retrofit work was performed from year 2000 to 2001, and it was the first large scaled reinforcement work. The vibration tests were performed before and after the retrofit work. After the retrofit work, the earthquake observation records were obtained for several earthquakes with different amplitude level. The dynamic characteristics of the building are comparatively discussed for three terms in the lifetime of the building, namely, Term-1, comprising the years after completion until before the Miyagi-ken Oki earthquake, this was the most severe earthquake for the building; Term-2, comprising the years after the Miyagi-ken Oki earthquake until before the retrofit work; and Term-3, after the retrofit work.

The data obtained through the observations and tests make possible to discuss about the amplitude dependency of the dynamic characteristics of the building, the evaluation of the degree of damage before the retrofit work and the reinforcement effects after the retrofit. The damage detection technique is also addressed using the tests results before the retrofit work by comparing the vibration characteristics based on the structural model which explain well those after the completion. The remaining stiffness coefficients were calculated for each portion of the structural elements. Then, the seismic strengthening effect is discussed using the results after the retrofit.

It was confirmed from the vibration tests that the torsional motion was reduced by the retrofit work. Through the damage evaluation of the building before the retrofit work, based on the system

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identification, it was found that the remaining stiffness coefficients of each portion of the structural elements were consistent with the damage pattern during the 1978 Miyagi-ken Oki earthquake. It was confirmed by the system identification of the building after the retrofit work that the remaining stiffness at the reinforced portions became higher than before the retrofit work. Through the forced vibration tests, for a deflection angle of the order of 10^{-5} radian, the global stiffness shows its highest value in Term-1, and was reduced by 60% (remaining stiffness equal to 40%) in Term-2 due to the severe damage caused by the 1978 Miyagi-ken Oki earthquake. The global stiffness was increased by 1.5 times after the retrofit work (Term-3). Namely stiffness recovered up to 60% compared to that in Term-1. It was also found that the same trend of the earthquake observations was recognized in the tests with deflection angle of the order of up to 10^{-3} radian.

INTRODUCTION

The dynamic characteristics of RC buildings changes depending on the amplitude of the deflection of the structure. Different types of observations and vibration tests have been performed to an existing 9-story RC building for 35 years after its completion in 1969 up to date, as shown in Table 1. The building was severely damaged during the 1978 Miyagi-ken Oki earthquake (Shiga et al., 1981) [1]. During this earthquake, the maximum acceleration recorded at 9th floor exceeded 1G, 1040 cm/s^2, but the repair work in 2001 [2] was the first large scaled retrofit. On this building, some forced vibration tests were performed after the completion times (Shiga et al.,1973) [3]. Continuous earthquake observation and also microtremor observations have been performed. Vibration tests by applying impulsive loads to the ground were also performed using the Air Gun Impactor (Ali and Motosaka, 2001)[4]. These observations made it possible to investigate the change of vibration characteristics of the building for these 35 years.

Table 1 Historical change of fundamental dominant frequencies

Year,Dat	Transversal (TR	Longitudinal (LN	Exciting force
1969	2.28Hz	2.45Hz	Forced vibration
1970/09/14	1.85Hz (79gal)	2.00Hz (103gal)	Earthquake (1970.9.14)
1971	2.08Hz	2.10Hz	Forced vibration(60.22kg . m)
1974		2.13Hz	Microtremor
1978/02/20	1.18Hz (421gal)	1.18Hz (298gal)	Earthquake(Miyagi-Ken Oki)
1978/06/12	0.89Hz (1040gal)	0.98Hz (523gal)	Miyagi-Ken Oki earthquake
1995	1.44Hz	1.53Hz	Man-Power excitation
1996	1.46Hz	1.51Hz	Microtremor
1998	1.36Hz	1.56Hz	Air Gun Impactor
1998/09/15	1.03Hz (190gal)	1.12Hz (379gal)	Miyagi-Ken Nanbu earthquake
1998	1.41Hz	1.50Hz	Microtremor
Before	1.325Hz	1.40Hz	Forced vibration
Before	1.48Hz	1.54Hz	Microtremor
2001/01	1.72Hz	1.75Hz	Microtremor
After	1.65Hz	1.725Hz	Forced vibration
After	1.74Hz	1.85Hz	Microtremor
2003/05/26	1.19Hz (231gal)	1.29Hz (264gal)	Earthquake(Miyagi-Ken Oki)
2003/07/26	1.37Hz (98gal)	1.36Hz (102gal)	Miyagi-Ken Hokubu Earthquake
2003/09/26	1.44Hz (29gal)	1.55Hz (22gal)	Tokachi-Oki Earthquake
2003/09/26	1.48Hz	1.53Hz	Earthquake(Tokachi-Oki)

The seismic retrofit work of the building was performed in year 2000 and 2001. The authors performed the forced vibration tests before and after the strengthening work in order to evaluate the damage of the building through identification of the dynamic characteristics of the building before the repair work, and to confirm the reinforcement effects after the repair work. The relation between the change of dynamic characteristics and structural damage degree and/or strengthening was discussed in Motosaka et al., 2002[5]. After the retrofit work, the building was shaken by several earthquakes with different amplitude level. This paper describes the amplitude dependent dynamic characteristics of the building.

After a description of the building, the retrofit work is briefly addressed. Then the forced vibration tests before and after the retrofit work are described. Then, the evaluation of damage degree before the reparation and strengthening effects are described through a system identification technique to evaluate the damage degree of each block divided portion of the structure before the retrofit and the reinforcement effect due to the repair work based on the forced vibration tests. Finally, the amplitude dependent dynamic characteristics are addressed. In this case, the term of 35 years is divided into the three characteristic terms as shown in Fig.1; 1) Term-1: comprising the years after completion until before of 1978 Miyagi-ken Oki earthquake. 2)Term-2: after the 1978 Miyagi-ken Oki earthquake and before retrofit work, 3) Term-3: after the retrofit work up to date.

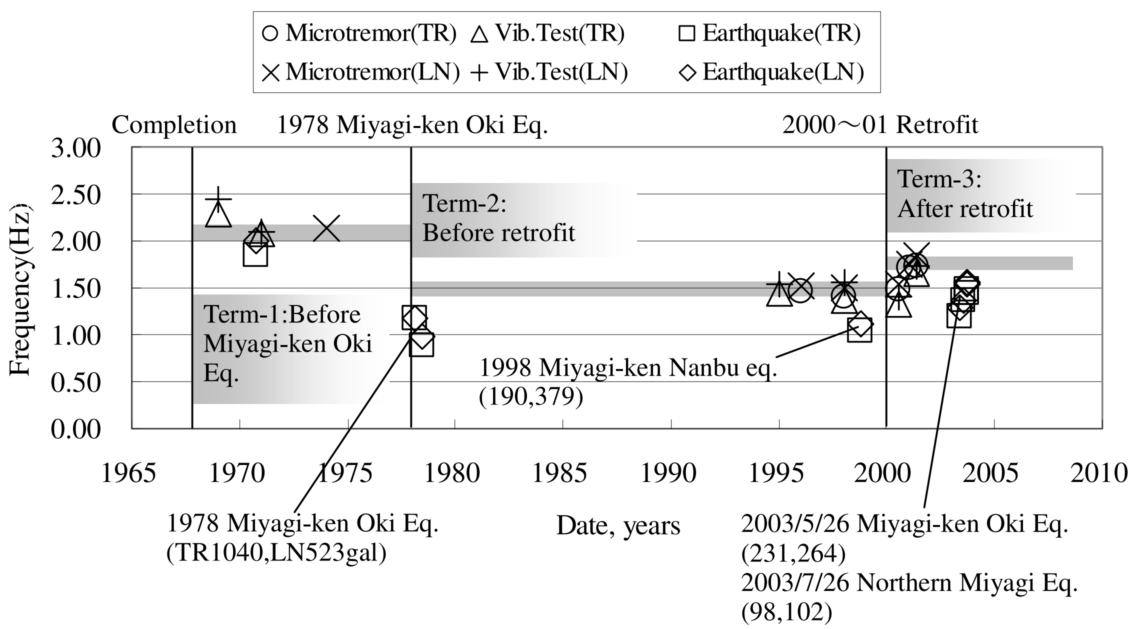


Fig.1 Historical change of fundamental dominant frequencies

DESCRIPTION OF THE BUILDING AND ITS RETROFIT WORK

Building description

The objective building for architectural and civil engineering departments of Tohoku University, is a 9 story RC structure with pile foundation, consisting of a 9 story higher portion and a two story lower extended portion as shown in Photo 1. The plans of 1st floor (1F) and 3rd floor (3F) are shown in Fig.2. It is noted that the floor designation in this paper is not European style but Japanese/American style.

The seismic resistant elements of the building comprise two side shear walls and core walls besides the moment resistant frames. The building has uni-axial eccentricity in the longitudinal direction.

The building was constructed at the slope site as shown in the original topography map together with the two soil profile data (Fig.3).

Photo 1 Overall view of the building from the north-west direction

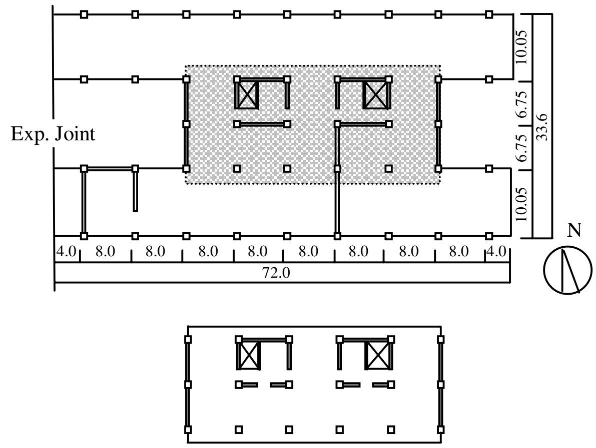


Fig.2 Plans at the 1st floor (top) and the 3rd floor (bottom)

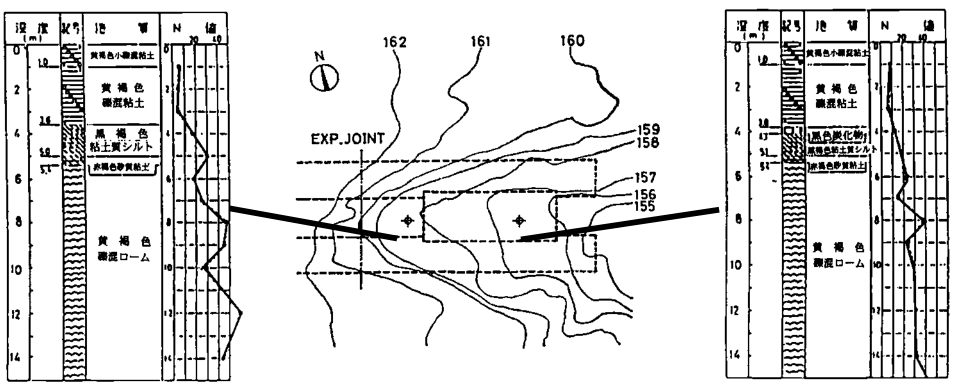


Fig.3 Original topography at the construction site and soil profiles

Seismic diagnosis and retrofit

Before the retrofit work, the seismic diagnosis was performed. Table 2 shows the result. The *Is* value, index for seismic resistance (refer to e.g. homepage of Takenaka Corporation for seismic diagnosis of existing building) at several floors were less than 0.6. The repair work was planed to upgrade the value, based on the four retrofits shown in Fig.4. Namely, the first retrofit was the replace of the two side walls. High strength concrete (Fc = 24MPa) was used. Second, steel braces were installed from 3rd to 8th floors to reduce torsional motions. Third, the boundary beams connected to the shear walls at the two cores were reinforced by steel plate wrapping. Fourth, the floor slabs between the side walls and the core walls were reinforced.

Through the retrofit, the *Is* values at all floors exceeded 0.6 as shown in Table 3 indicating the result of the diagnosis after the retrofit work.

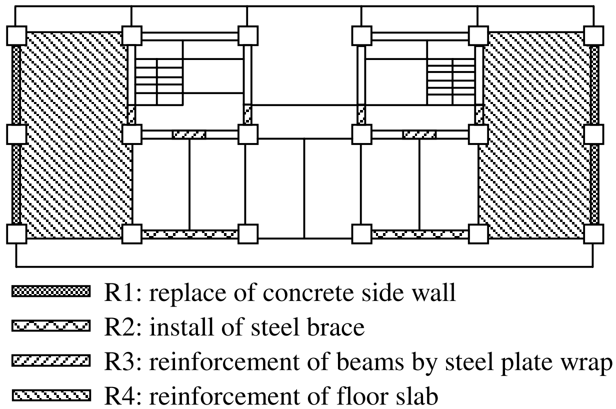


Fig.4 Outline of the retrofit

Table 2 *Is* values* before the reinforcement

	1fl	2fl	3fl	4fl	5fl	6fl	7fl	8fl	9fl
Trans	0.56	0.61	0.54	0.57	0.68	0.69	0.72	0.79	1.04
Long.	0.55	0.94	0.91	0.55	0.56	0.57	0.63	0.74	0.76

Table 3 *Is* values* after the reinforcement

	1fl	2fl	3fl	4fl	5fl	6fl	7fl	8fl	9fl
Trans	0.75	0.88	0.84	1.01	1.02	1.05	1.13	1.33	1.32
Long.	0.63	1.12	1.45	0.88	0.84	0.93	1.02	1.20	1.07

*Results of the secondary seismic diagnosis

DESCRIPTION OF VIBRATION TESTS

Excitation force and sensor location

The same sinusoidal excitation forces from 0.5 Hz to 10 Hz (Fig.5) were applied in the transversal (TR) and longitudinal (LN) directions at the tests before and after the retrofit work using the unbalanced mass-type exciter with the maximum force of 3 tons. The exciter was placed at center of roof floor (RF). The frequency increment was 0.05Hz for the range from 1 Hz to 3Hz and 0.025Hz at around the resonant frequencies. In other range, the increment was 0.1 Hz.

Fig.6 shows the sensor location for TR excitation. Micrometers for the microtremor observation were used as sensors. The four sensors were placed vertically to examine the vibration modes. At the three floors, three sensors were placed to examine the torsional motions. The rocking motion is evaluated from the two vertical sensors of the first floor. The simultaneous signals from the sensors were acquitted to a digital data recorder. As for the data processing, the resonance and the phase lag curves are calculated by using the cross correlation analysis. It is noted that the microtremor observations were performed immediately after the forced vibration tests.

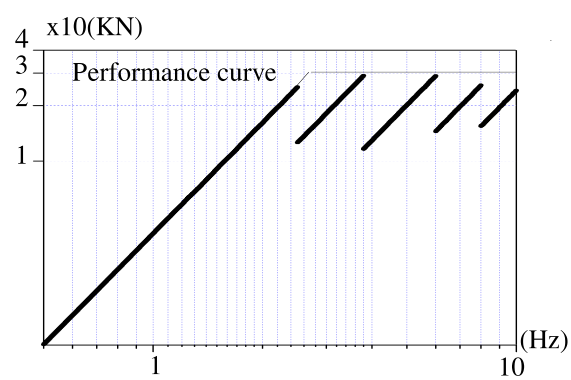


Fig.5 Exciting force-frequency relation

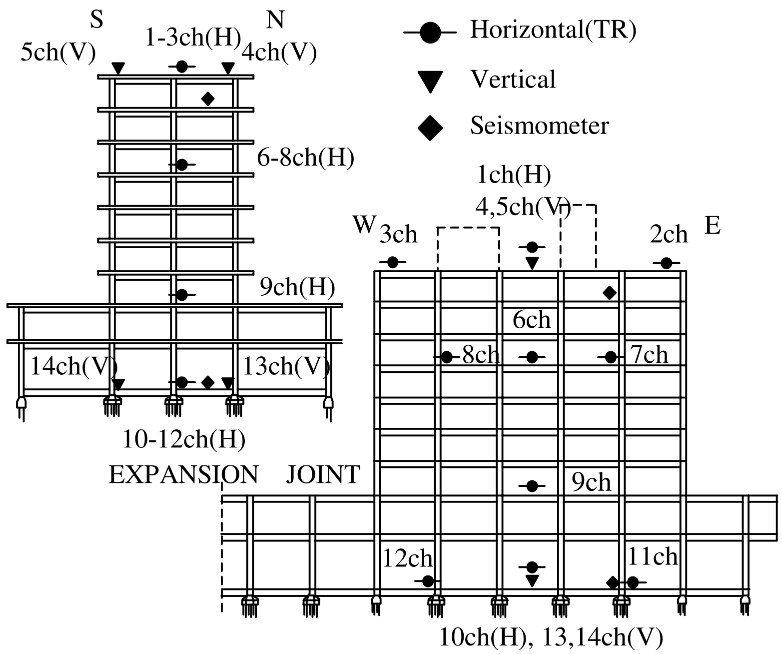


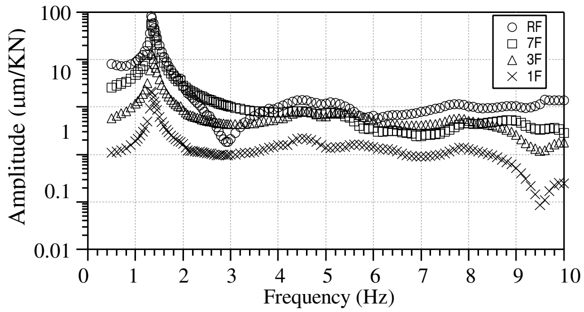
Fig.6 Sensor location for TR excitation

Test results

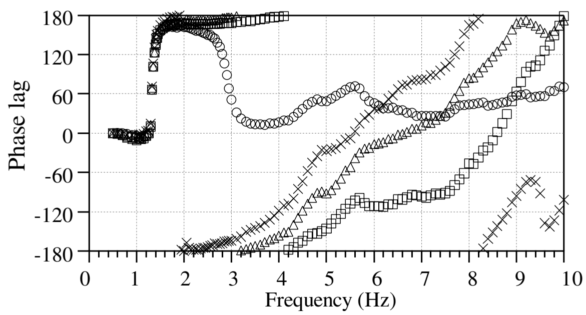
Fig.7 shows the resonance and the phase lag curves at the center of 4 floors for TR excitation of the test before the retrofit work. Fig.8 shows the comparison of the resonance curves for the tests before and after the retrofit work. Fig.8(a) is the resonance curves at the center of the roof slab for LN direction and Fig.8(b) is for the TR direction. The vertical axis shows the amplitudes for the unit excitation force, 1tonf. It is recognized that the resonant frequency becomes higher and the amplitude become smaller due to the strengthening in the both excitation directions. In the LN excitation, fundamental frequency changed from 1.40 Hz to 1.725Hz. The corresponding resonance modes are shown in Figs.9 (a) and (b) for the two tests. It is recognized that the torsional component is relatively large as shown in Fig.9(a) but becomes fairly small in Fig.9(b) due to the retrofit, installing the steel braces to depress the torsional motion. In the TR excitation the fundamental frequency changed from 1.325 Hz to 1.65 Hz. Note that the fundamental frequency for the LN excitation was 2.1 Hz just after the completion of the building, and 2.0 Hz for the TR excitation.

The resonant frequencies of the two tests are summarized in Table 4. In this table, the corresponding dominant frequencies obtained from microtremor observations are comparatively tabulated. It is found that the dominant frequencies are slightly higher in the microtremor cases than the forced vibration test cases. This is due to the difference in amplitude level. It is noted that the damping factors of the fundamental modes in the two tests, before and after the work, are 5% in both directions.

Table 5 shows the sway-rocking ratios based on the two forced vibration tests before and after the retrofit work. Those from the microtremor observations are also tabulated. The three swaying ratios are indicated for the TR excitations using the records at the three measuring points at the ground floor. The swaying ratios at the east side are lager compared to the west side. This is due to the soil condition in Fig.3. The sway-rocking ratios, summation of swaying ratio and rocking ratio, were 5-7% before the retrofit work, but those became 12-15% after the work due to the increased stiffness of the building.

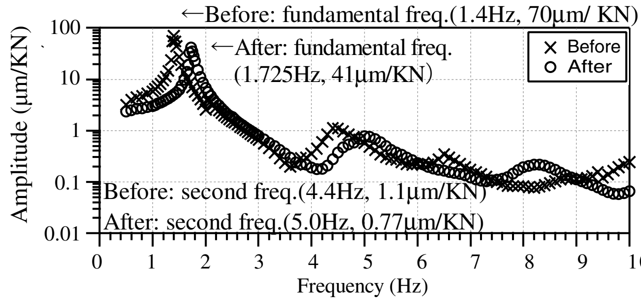


(a) Resonance curves

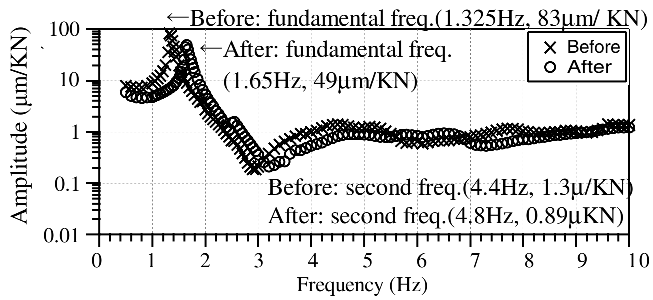


(b) Phase lag curves

Fig.7 Resonance and phase lag curves for TR excitation

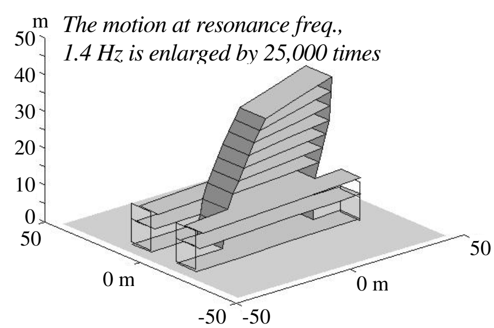


(a) LN excitation

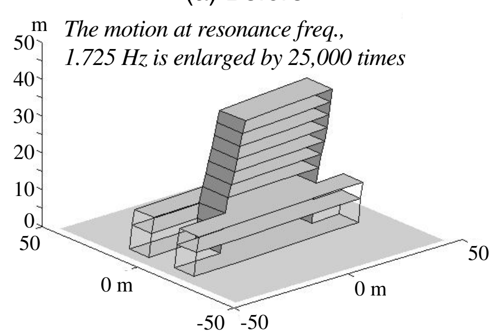


(b) TR excitation

Fig.8 Comparison of resonance curves at center of roof floor for the two tests



(a) Before



(b) After

Fig.9 Comparison of fundamental modes of the two tests for LN excitation

Table 4 Comparison of Natural frequencies

Direction	Mode	Forced Vibration		Microtremor		
		Before	After	Before	Jan,2001	After
LN	Translation 1st	1.4Hz	1.725Hz	1.54Hz	1.75Hz	1.85Hz
	Translation 2nd	4.4Hz	5.00Hz			
	Torsion 1st	2.05Hz	2.55Hz	2.20Hz	2.60Hz	2.61Hz
TR	Translation 1st	1.325Hz	1.65Hz	1.48Hz	1.72Hz	1.74Hz
	Translation 2nd	4.4Hz	4.8Hz			
	Torsion 1st	2.05Hz	2.50Hz	2.19Hz	2.60Hz	2.61Hz

Table 5 Comparison of swaying-rocking ratios

	Direction	Observation Point	Forced Vibration		Microtremor	
			Before	After	Before	After
Sway Ratio	LN	Center	2.20%	4.80%	3.40%	5.20%
	TR	Eastside	2.90%	5.90%	3.80%	6.10%
		Center	2.40%	4.70%	3.10%	5.20%
		Westside	2.20%	4.50%	3.10%	5.30%
Rocking Ratio	LN	Center	3.50%	7.70%	5.10%	8.40%
	TR	Center	4.30%	9.40%	6.70%	10.60%

DETECTION OF DAMAGE AND REINFORCEMENT EFFECTS

Methodology of system identification

Damage evaluation of the building before the retrofit was investigated based on the vibration test by comparing the vibration characteristics after the building was completed. Then the strengthening effects were evaluated based on the vibration test after the retrofit work. Damage degree was evaluated as the remaining stiffness to the stiffness at the completion time. The latter was evaluated based on the structural model which explains well the vibration test at the completion time. The system identification is performed to minimize the sum of residue squares in the following form.

$$J = \sum_i (\mathbf{y}^*(f_i) - \mathbf{x}^*(f_i))^T \mathbf{W}(\mathbf{y}(f_i) - \mathbf{x}(f_i)) \qquad (1)$$

where $\mathbf{x}(f_i)$ denotes a vector of complex resonance curves obtained by the test. $\mathbf{y}(f_i)$ denotes a vector of those by the identification analysis. **W** denotes the weighting matrix. The symbol * denotes complex conjugate. To solve the minimization problem, the least squares method (Marquardt method) was used.

The system identification procedures are shown in Fig.10. Analytical model comprising frame models of LN and TR directions and the block division was made and applied as shown in Fig.11.

In the vertical direction, the 9 stories are divided into three blocks, namely, 1-2 flr., 3-6 fls., and 7-9 fls. The stiffness remaining coefficient $\square$ is assumed for each block division. In each floor, structural elements are divided into six blocks, namely, core walls in the LN direction, core walls in the TR directions, eastside wall, west side wall, columns and beams in the LN direction and columns and beams in the TR directions. Thus the 18 parameters for stiffness evaluation are considered. Regarding damping factors, modal damping factors obtained from the test are used and these are included from the identification parameters. Then a reduced model with 2 horizontal degrees of freedom and one torsional degree of freedom at each floor is made to calculate the resonance curves. Other details are referred to Motosaka et al., 2002. [5]

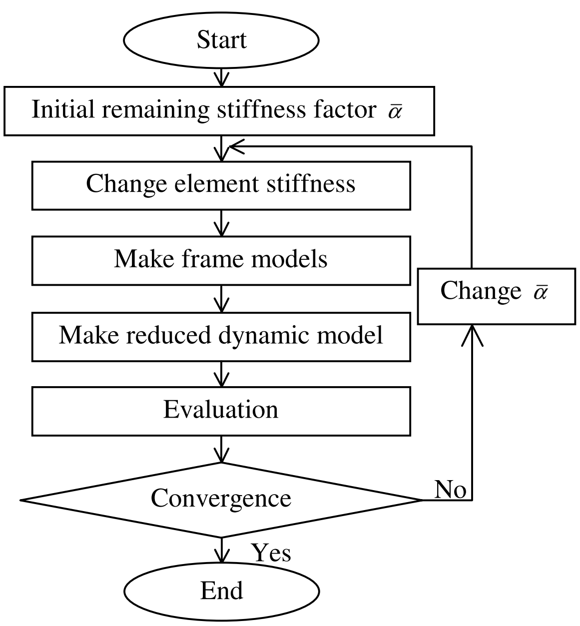


Fig.10 Flow chart of system identification

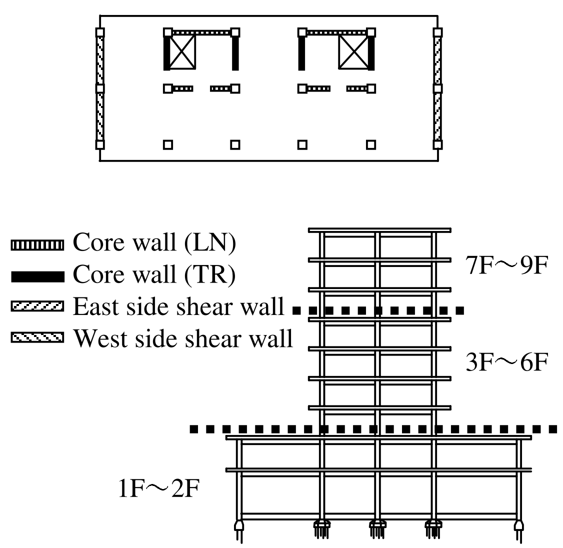


Fig.11 Block division of the building

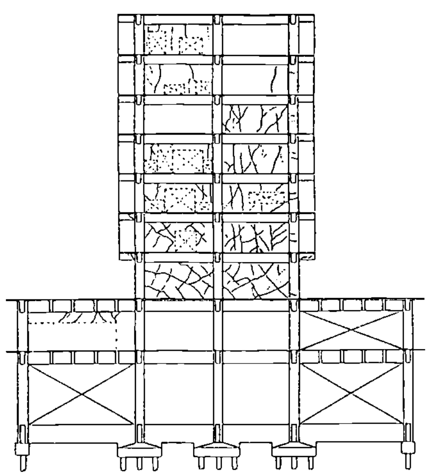
Damage evaluation before the retrofit

The damage evaluation was done by comparing the vibration characteristics based on the structural model which explain well those characteristics after the completion. Considering that the fundamental frequency for the LN excitation was 2.1 Hz, and 2.0 Hz for the TR excitation just after the completion of the building, the remaining stiffness was determined such that the fundamental frequency for LN excitation became 1.40Hz and 1.325 Hz for the TR excitation.

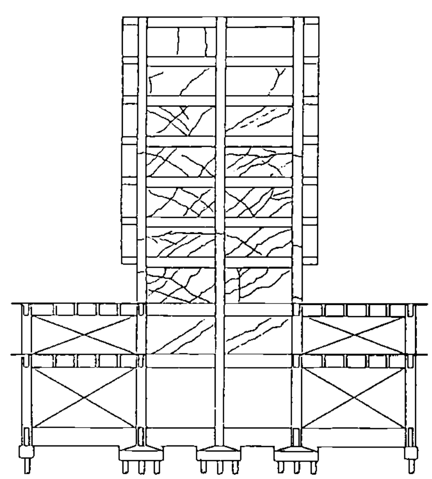
The results of the system identification are shown in Table 6. The calculated resonance and phase lag curves are compared with the corresponding test results for the case of TR excitation as shown in Fig.13(a). It is found that the remaining stiffness is smallest at the block of 3rd to 6th floor meaning that the portion is damaged more than the other portions. The damage of columns and beams at the higher portion (3rd to 9th floor) in the TR direction seems to be larger compared to the LN direction. This may be explained by the fact that deflection level was larger in the TR direction compared to LN direction during the 1978 Miyagi-ken Oki earthquake. It is also found that damage of west side shear wall is larger compared to the east wall. This is consistent with the damage extent judging from the state of shear cracks as shown in Fig.12.

Table 6 Identified remaining stiffness coefficients before the reinforcement

	Columns, Beams		Core walls		Side Shear wall	
Floor	LN	TR	LN	TR	West	East
7 ~ 9F	0.41	0.22	0.23	0.22	0.38	0.59
3 ~ 6F	0.29	0.21	0.24	0.22	0.21	0.41
1 ~ 2F	0.62	0.61	0.68	0.61	0.72	0.74



(a) West shear wall



(b) East shear wall

Fig.12 Shear crack distribution due to the 1978 Miyagi-ken Oki earthquake

Evaluation of Reinforcement effects

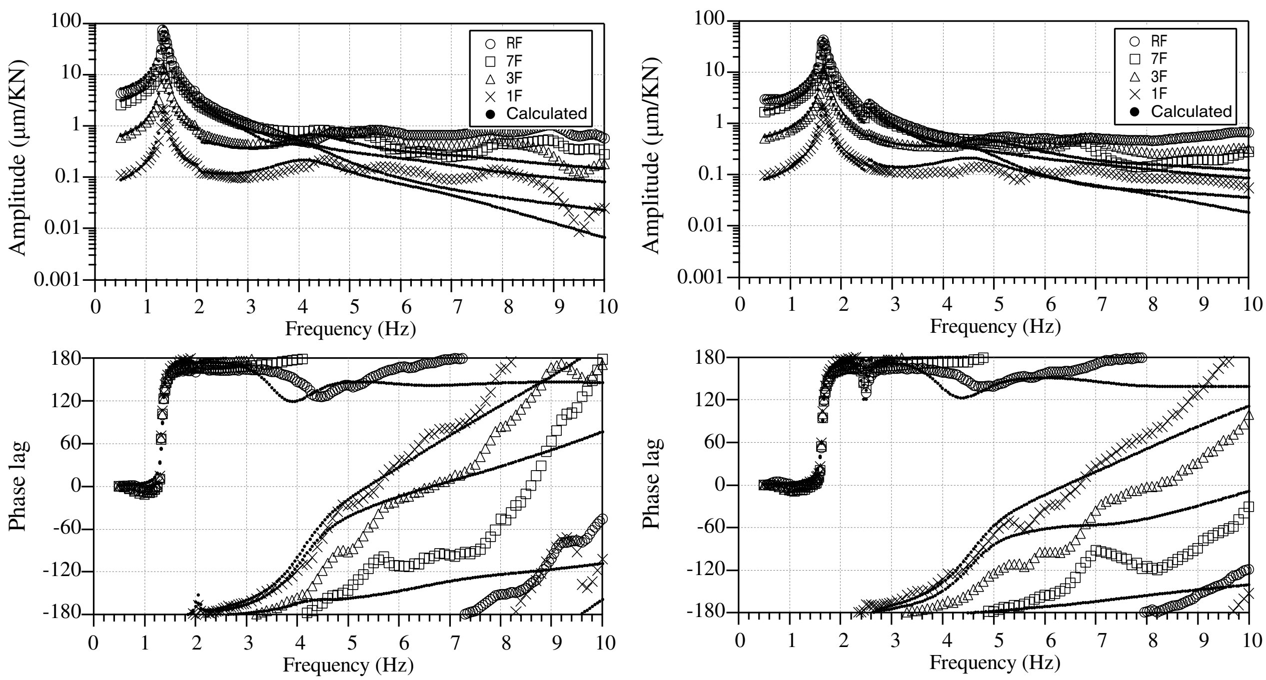
Reinforcement effects were evaluated as the remaining stiffness coefficient for each block, as explained before. The system identification procedure was performed based on a similar procedure. The installed braces were taken into account in making the frame model of the LN direction. In this identification analysis, the stiffness remaining factors at the block for 1-2 fls. was fixed to the values shown in Table 7, because retrofit work was not performed for these portions.

The results of the system identification are shown in Table 7. The calculated resonance and phase lag curves are compared with the corresponding test results for the Transverse direction in Fig.13.

It is confirmed that the remaining coefficients at the reinforced portions become higher. The stiffness recovery of the side shear walls is remarkable, and the coefficients are now almost the same among them.

Table 7 Identified remaining stiffness coefficients after the reinforcement

	Columns, Beams		Core walls		Side Shear wall	
Floor	LN	TR	LN	TR	West	East
7 ~ 9F	0.48	0.59	0.42	0.55	0.79	0.79
3 ~ 6F	0.35	0.59	0.41	0.49	0.77	0.78
1 ~ 2F	0.62	0.61	0.68	0.61	0.72	0.74



(a) Before the retrofit work (TR excitation)

(b) After the retrofit work (TR excitation)

Fig.13 Comparison of calculated and tested resonance and phase lag curves

Amplitude Dependency based on Earthquake Observation

Earthquake observation records obtained at the building are shown in Fig.14 for some of the major records. The observation records for the 1970/9/12 earthquake, the 1978/2/20 earthquake as well as those for the 1978/6/12 earthquake are used for investigating the dynamic characteristics of the building in Term-1 (since completion until the 1978/6/12 Miyagi-ken Oki earthquake). The investigated records for Term-2 (after 1978 Miyagi-ken Oki earthquake but before retrofit work) are those for 10 earthquakes. The obsernation records were obtained for the 1998/9/15 Miyagi-ken Nanbu earthquake, a shallow inland earthquake. Those observations forTerm-3 (after the retrofit work up to date) are about 50. After the retrofit work, the observation records were obtained for the 2003/5/26 Miyagi-ken Oki earthquake, the 2003/7/26 Northern Miyagi earthquake, and the 2003/9/26 Tokachi Oki earthquake.

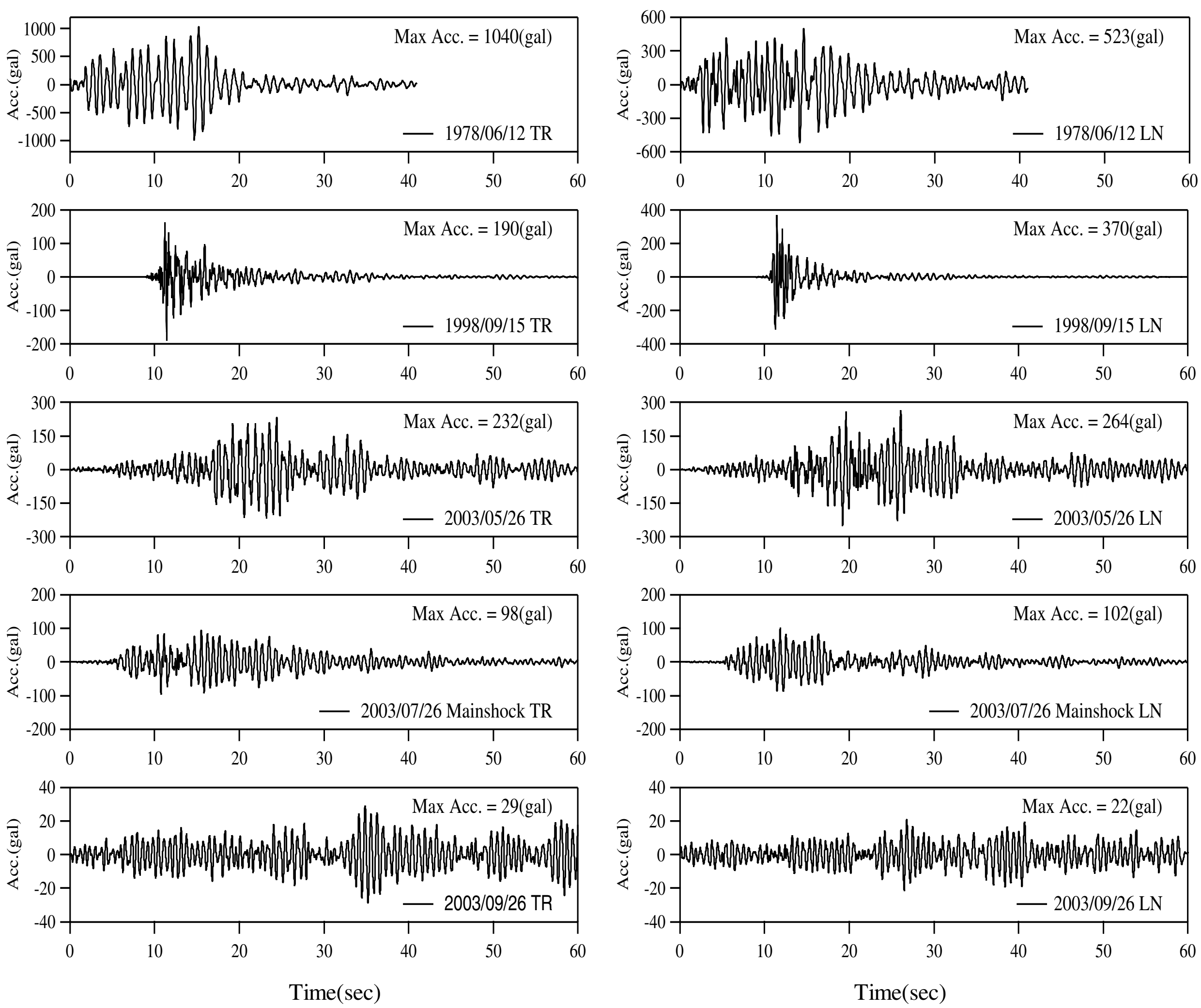


Fig.14 Some of the earthquake observation records obtained at the building

In order to discuss the amplitude dependency of dynamic characteristics of the building, the relation between the dominant period and average deflection angle was investigated using the earthquake observation records at 9th floor. The results are shown in Fig.15 for TR and LN directions. The deflection angle was calculated as the apparent value by dividing the maximum displacement by the height from ground to the seismometer. To calculate the dominant period, the Parzen Widow with 0.2 Hz was applied to the Fourier spectral of the record. In this figure, the fundamental periods obtained from forced vibration tests before and after the retrofit work are plotted for the corresponding deflection angles (the symbol ◆ for before and the symbol ● for after the retrofit). For each Period, the approximated curve is determined and plotted in the figures. The fundamental periods for the microtremor observations are also plotted in the figure. It is noted that the deflection angle order is about 10^{-2} radian during the 1978 Miyagi-ken Oki earthquake.

Fig.16 shows the force-deflection relations for the large amplitude range and the small amplitude range. It is found from these figures that the global stiffness in the small amplitude range shows highest values in Term-1 and are reduced by 60% (the remaining stiffness is 40%) in Term-2 due to the severe damage caused by the 1978 Miyagi-ken Oki earthquake. Then, the stiffness is increased 1.5 times after the retrofit work (Term-3) in the deflection angle order of 10^{-3} radian. This is noticed from the earthquake observation records of Term-3 compared to those in Term-2. Namely, stiffness is recovered up to 60% compared to that in Term-1. It is also that the slope of the approximated curves of the relation between the fundamental period and deflection is steeper before the retrofit work compared to the curve after the retrofit work, especially in the TR direction. This means that the amplitude dependency in stiffness is larger before the retrofit.

It is also noted that the fundamental periods for the vibration tests before and after the retrofit work are almost same as those for small earthquakes.

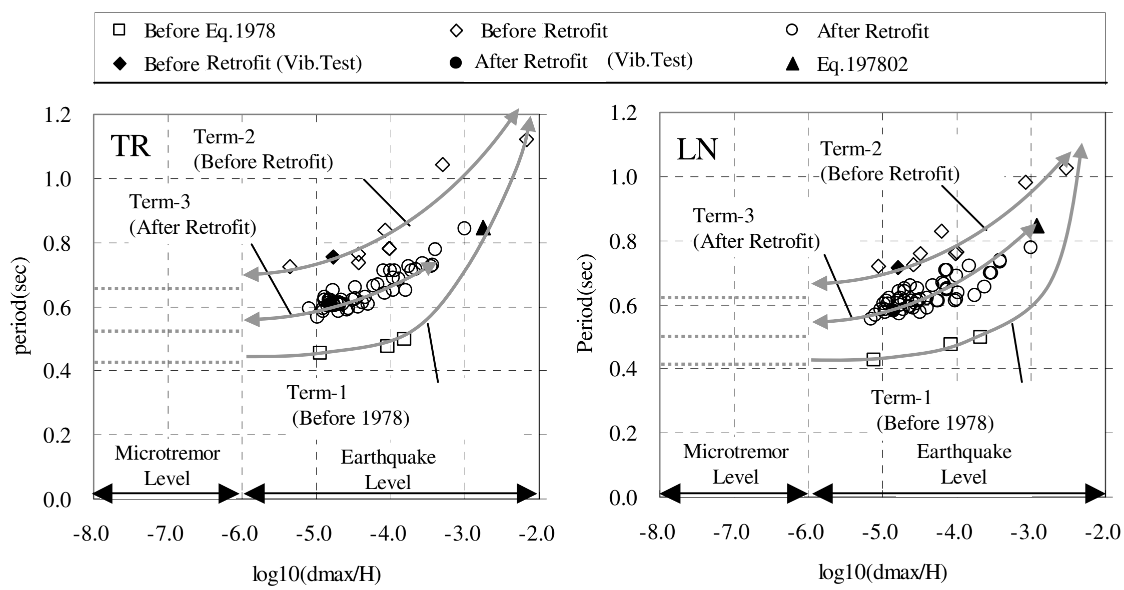


Fig.15 The relation between the dominant period and average deflection angle

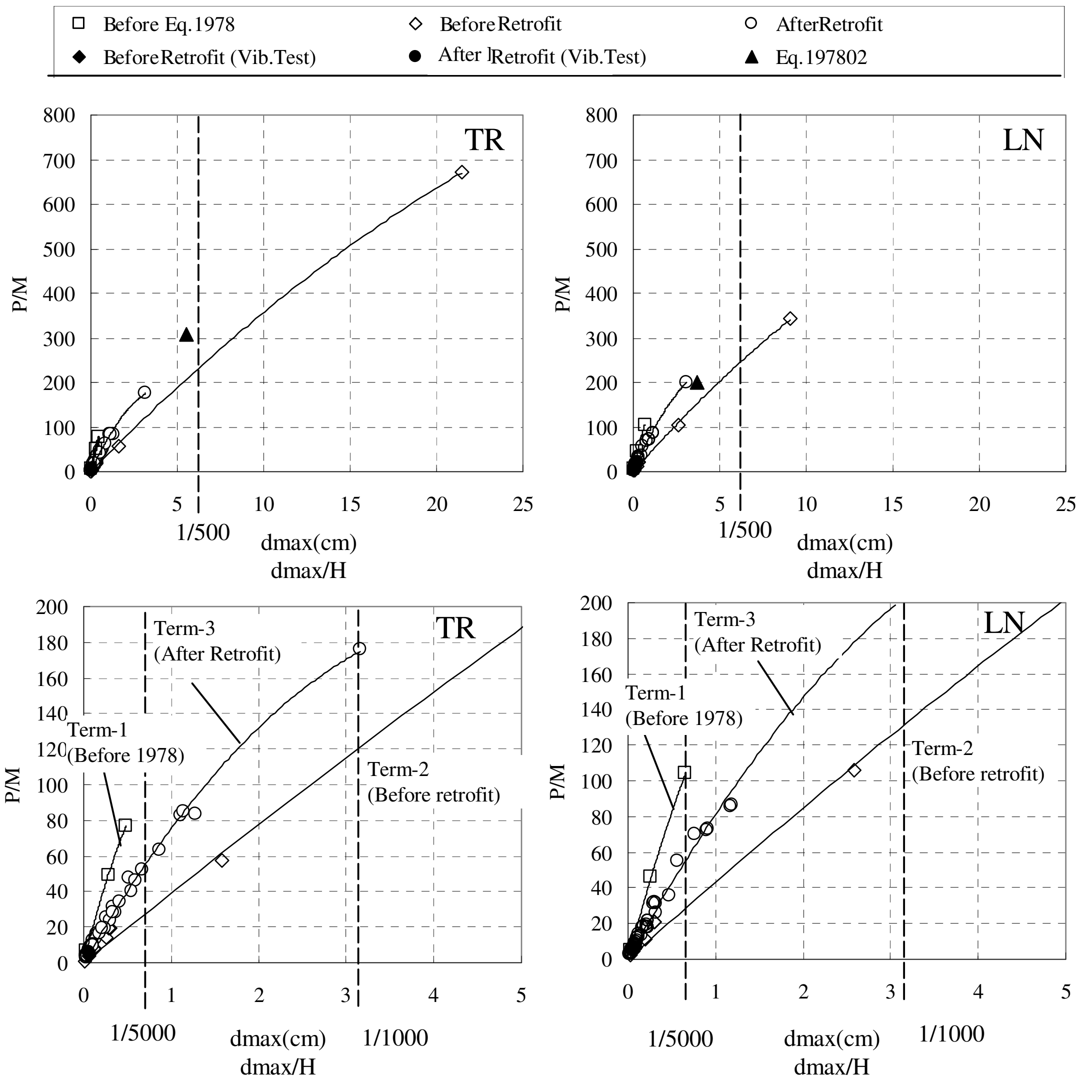


Fig.16 The relation between the force normalized by mass and maximum displacement (dmax), average deflection angle (dmax/H) (Bottom figures are expansion figures of top figures)

CONCLUSIONS

The amplitude dependent dynamic characteristics of an existing 9-story RC building are comparatively discussed based on the various types of observations and vibration tests for the divided three terms for these 35 years after it completion, Term-1, since completion of the building until the 1978 Miyagi-ken Oki earthquake, this is the most severe earthquake for the building; Term-2, before the retrofit work but after the severe earthquake; and Term-3, after the retrofit work. These observation/test data make possible to discuss the amplitude dependency of dynamic characteristics and also to evaluate the damage degree before the retrofit work as well as the reinforcement effects after the retrofit. The damage detection technique is also addressed using the test results before the retrofit and comparing them with the vibration characteristics of the building just after completion, based on the structural model. Also, the seismic strengthening effect is then discussed using the results after the retrofit, based on the different deflection levels detected from microtremor observation, earthquake observation and vibration tests. The obtained findings are as follows;

1) It was confirmed from the vibration tests that the torsional motion was reduced by the retrofit work.
2) It was found from the damage evaluation of the building before the retrofit, by the system identification, that the stiffness remaining coefficients of each portion of the structural elements were consistent with the damage pattern during the 1978 Miyagi-ken Oki earthquake.
3) It was confirmed from the system identification for the building after the retrofit that the remaining stiffness at the reinforced portions became higher than before retrofit.
4) It was found, from the forced vibration test, that global stiffness in the small amplitude range is highest in Term-1 and is reduced by 60% (the remaining stiffness is 40%) in Term-2 due to the severe damage caused by the 1978 Miyagi-ken Oki earthquake. It was also found that the global stiffness was increased 1.5 times after the retrofit work (Term-3) for the deflection angle order of 10^{-5} radian. Namely, stiffness was recovered up to 60% compared to that in Term-1.
5) It was found from the earthquake observation records for the deflection angle order of up to 10^{-3} radian, that the same trend as the forced vibration test was recognized. It is noted that the deflection angle order is about 10^{-2} level during the 1978 Miyagi-ken Oki earthquake.

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