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Torsional vibration of a building with stepped heights

Y. Fukada, T. Kobayashi, M. Adachi, M. Nagata & Y. Hayashi
Kajima Corporation, Tokyo, Japan

ABSTRUCT: Torsion control design of a highrise building with stepped heights which was anticipated to cause torsional deformations during earthquakes was performed. Vibration characteristics of the building were observed by the vibration tests after its completion. The differences between these observed vibration characteristics and the analyzed ones of the design stage have been explained by the simulation analysis for the condition at the time of tests.

1 INTRODUCTION

Construction work of new City Hall Complex of Tokyo which consists of three buildings, that is, the City Hall Tower #1, #2 and Assembly Hall, were completed in March, 1991. The City Hall Tower #2 is a high-rise building whose height is 163.3m with 34 stories above ground and 3 stories underground and has an unique feature of stepped heights in its longitudinal direction as shown in Fig.1.

The torsional deformation in a building with stepped heights like this will be excited during earthquakes because the masses of upper setback stories act as the eccentric lateral forces to the lower portion and the lateral rigidities of higher frames tend to become smaller than those of lower ones by the canti-lever type bending deformation. To minimize this torsional deformation is very important problem to be solved for the structural design of this type of building.

Presented in this paper are the result of comparative studies to minimize the torsional deformation, the result of vibraion tests which were performed after the completion of the structure to verify the vibration characteristics with torsion and the result of simulation analysis to prove the properties of the results from the vibration tests.

2 STUDY ON TORSION CONTROL DESIGN

2.1 Objective structure

The objective structure for a study on the torsion control structural design has been modified from the Tokyo City Hall Tower #2 building as shown in Fig.2 and Fig.3.

To find a counterplan to control the torsional deformations caused by the building shape, many cases with different distributions of the sectinal properties and arrangement of the super-beam which has a height of one story have been analyzed.
Typical two cases selected from these studied caces are presented as follows.

Fig.1 New Tokyo City Hall Tower #1 and #2

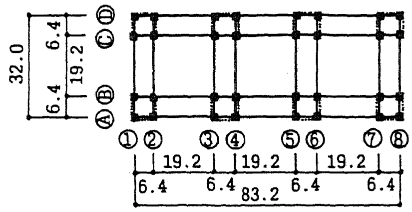


Fig.2 Framing plan of objective structure

-Case A- This structure was consciously designed not to control the torsion. Fig.4 shows the transverse framing sections, and column and beam members are listed in Table 1 and 2 respetively. The sections of column were decided to keep almost the same axial stress ratios in every columns by long-term load to the allowable stress as about 35%. Thus the maximum plate thicknesses of columns are 40mm for the lowest frames, 60mm for the middle ones and 80mm for the highest ones.

-Case B- This case consists of the modified sections of beams and braces in the transverse direction to Case A and are employed several super-beams in this direction to *minimize the differences between the lateral* rigidities of frames with varying heights, that is, the highest frames have two levels of supre-beams, the middle frames have one level of super-beam and the lowest frames have no super-beam while they have the same sections of columns and beams in the longitudinal direction to Case A as shown in Fig.4. Table 3 indicates the beam and brace members of the transverse direction for the Case B.

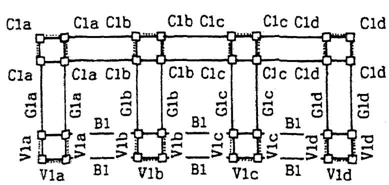
2.2 Static behavior

Static analysis has been carried out by the rigorous 3-D frame analysis method in which bending, shearing and axial deformations for column, bending and shearing deformations for beam, axial deformation for brace and shearing deformation for joint panel between column and beam are taken into account. The lateral static shear forces corresponding to the design earthquake shear force are applied. Story drifts (inter-story displacements) of each case are shown in Fig.5 where dotted(L), bold(M) and broken(H) lines correspond to the lowest end frame, midst or center of framing plan and highest end frame respectively.
Torsional deformation in the Case B becomes considerably smaller than the Case A and in the Case B the differences of the story drift at the 20th story where the super-beams are installed is very small.

To grasp the behaviors mentioned above, static analysis for the elementary 2-D or

Table 1. Member section of column

Code	Member section
C1a	□-800x800x(25-40)
C1b	□-800x800x(25-60)
C1c,d	□-800x800x(28-80)



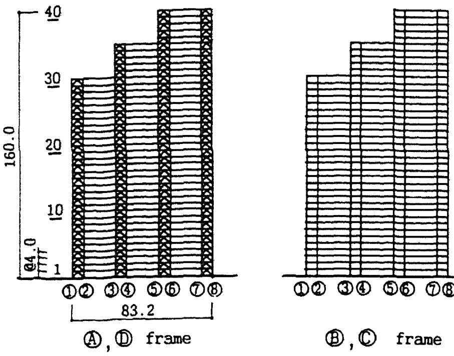


Fig.3 Longitudinal framing section of objective structure

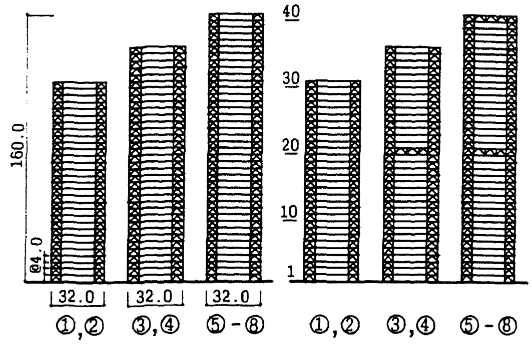


Fig.4 Transverse framing section of objective structure

Table 2. Member section of beam and brace for Case A

Code	Member section
G1a	BH-1000x(450-300)x19x32
G1b	BH-1000x(450-300)x19x36
G1c	BH-1000x(500-300)x19x40
G1d	BH-1000x(550-350)x19x40
B1	BH-1000x(450-300)x19x32
V1a-d	RH-350 x 350 x12x19

Table 3. Member section of beam and brace for Case B

Code	Member section
G1a	BH-1000x(450-250)x 19 x 28
G1b	BH-1000x(450-300)x(19-40)x(28-40)
G1c	BH-1000x(450-300)x(19-40)x(32-40)
G1d	BH-1000x(450-350)x(19-40)x(36-40)
V1a,b	RH-350 x 350 x 12 x 19
V1c,d	RH-350 x 350 x(12-19)x(19-22)

plane frames which are taken apart from the original 3-D structures has been executed against the lateral load equal to the burden of weight of each 2-D frame, i.e., 1G lateral force. Fig.6 shows the lateral displacements of each case which indicate the degree of lateral rigidity of each frame to its weight. In the Case A, the displacements are becoming larger according to their heights higher. It means the lateral rigidities go dowm as their heights. On the other hand, in the Case B which has super-beams in the middle and highest frames, top displacements become nearly equal to each other. Therefore, when the lateral rigidities of frames having different heights are to make almost the same, the torsional deformations of the original 3-D structure can be reduced considerably against static lateral forces.

2.3 Dynamic response

The time-history earthquake response analysis for each 3-D model has been conducted. The input waves applied to the transverse direction are EL CENTRO 1940 (NS), TAFT 1952 (EW), SENDAI 038 1978 (EW) and HACHINOHE 1968 (NS) with the maximum velocity of 30 cm/sec.
The damping factor is assumed as 1% of the critical damping for the 1st vibration. Fig.7 shows the maximum story drifts against the SENDAI (EW) wave whose influence on the torsion is the largest among the input waves. While the story drifts and the torsional deformations are large in the Case A, those of the Case B are evidently reduced.
The maximum response torsional rotation angles of each floor diaphram against HACHINOHE and SENDAI waves are shown in Fig.8. The maximum torsinal rotations of the Case B are reduced to 40% to 70% compared with Case A.

As the result of the case studies on the torsional control for the structure with the stepped heights when subjected to earthquake forces, the torsional deformations can be reduced by making the lateral rigidities of the plane frames with varying height close each other. However, it is difficult to make them close by ajusting merely the sectional properties of the structural members. So the adoption of the super-beams is very effective to achive this purpose as shown here. The Case B was slightly modified to adopt in the actual structural design of the new Tokyo City Hall Tower #2 building.

3 VIBRATION TESTS

3.1 Testing methods

Forced vibration test by a generator, free vibration after the man-power excitation and micro-tremor mesurement were performed.
The arrangement of the transducers are shown in Fig.9.

The construction work of the floor-slab

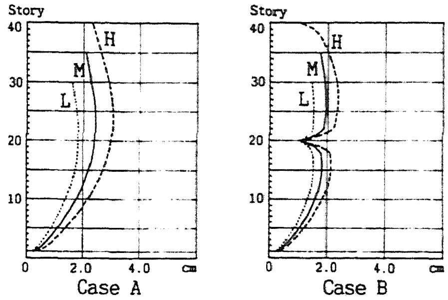


Fig.5 Story drift by static lateral force

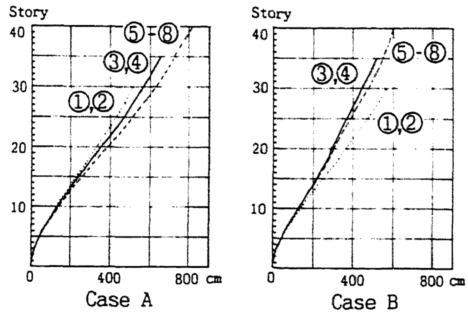


Fig.6 Lateral disp. of elementary frames

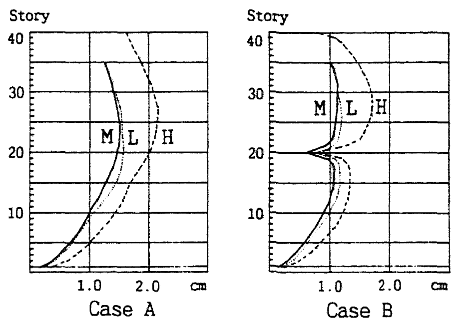
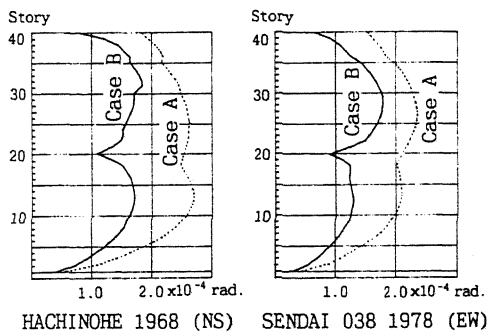


Fig.7 Max. respnse story drift (SENDAI 038)



HACHINOHE 1968 (NS) SENDAI 038 1978 (EW)

Fig.8 Max. respnse torsional rotation angle concrete and of the pre-cast concrete curtain walls (hereinafter called PCCW) was almost finished but the installation of the ceilings and of the partition walls was completed partially at that time. Since there was no live loads in addition to the incompletion of these finishings, the total weight of this building at the time of the vibration tests are guessed to be about 85% of the design value.

The resonant curves at every measuring points were derived from the response for the constant excitation force by the generator with a maximum excitation force of 3 tonsf which was installed on the 33rd floor as shown in Fig. 9. Fig.10 shows the resonant curves for the normarized unit excitation force of 1 tonf, from which it is found the phase curves are fallen into disorder and show no good preciseness in the longer period range because of the lack of the excitation power of the generator. So the detailed study on these frequency ranges should be discussed from the result of the man-power excitation and the micro-tremor measurement.

Free vibrations were observed after sudden stop of the excitation caused by about twenty people's synchronized weight shifting until the vibration amplitudes grew enough.

Micro-tremor measurements were also performed for 30 minuites at the midnight after each forced vibration test. The natural vibration periods and mode shapes were derived from the peaks of the transfer functions which were calculated as the ratios of cross-spectra to the basic measuring point of the 2nd floor.

3.2 Test results

Table 4 indicates the natural vibration periods and the damping factors obtained from the tests. Fig.11 also shows the vibration mode shapes observed. The damping factors of the 1st mode of the longitudinal direction and the 1st and 2nd modes of the transverse direction were derived from the free vibration decaying waves and the others were from the resonant curves by the $1/\sqrt{2}$ method.

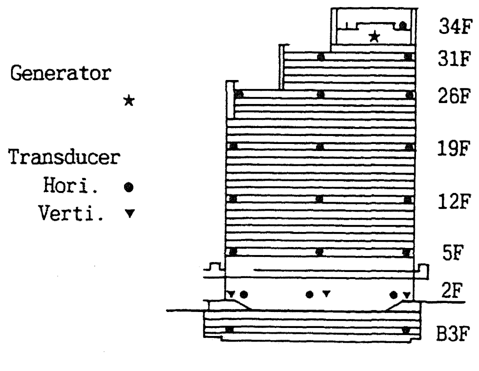


Fig.9 Arrangement of measuring points

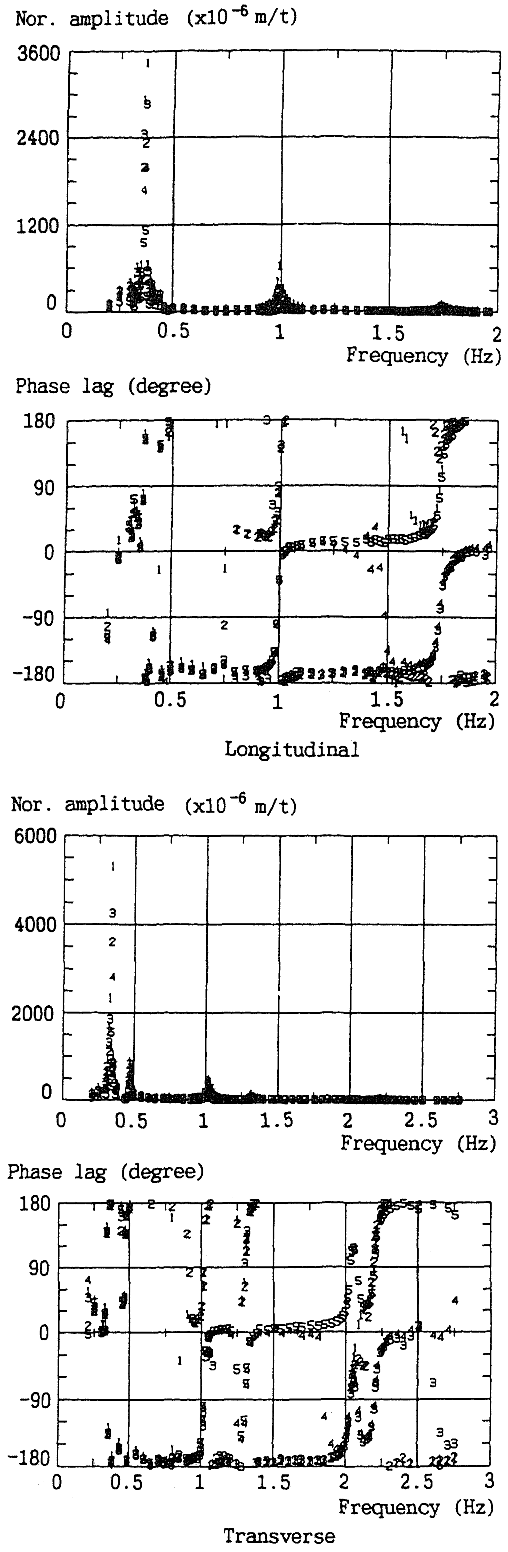


Fig.10 Displacement resonant curve

The 1st vibration mode shapes of the longitudinal direction and the 1st and 2nd modes of the transverse direction were from the micro-tremor measurements and the others were from the forced vibration test by the generator. In the longitudinal direction the 1st, 2nd and 3rd vibrations of the swaying component were observed but the torsional one was not observed because this direction is nealy symmetry in its shape.In the transverse direction the 1st, 3rd and 5th vibrations of the swaying component increasing the nodes one by one in the vertical direction of the building and the 2nd, 4th and 6th vibrations of the torsional component vibrating with the reverse phase deformations in the north- and the south-side of the building were observed.

4 SIMULATION ANALYSIS OF TEST RESULT

4.1 Dynamic analysis of the design stage

The equivalent simplified model which were derived from the deformations and forces by the rigorous 3-D analysis, was employed to perform the analyses for the earthquake resistant design of the Tower #2 building. This model consists of several bending and shearing deformation elements and bending, shearing and axial deformation elements with boundary beams substituted for the original structure. The natural vibration periods analyzed by this equivalent model for the design are shown in Table 5.

4.2 Additional rigidity by curtain wall

The observed vibration periods are considerably shorter than those from the design analysis. The causes are anticipated that the building weight was lighter than the design value, and that the non-structural elements, especialy the PCCW acted as the additional rigidities to the main structure because of the small vibration amplitudes of the tests. Therefore, the simulation analysis on the natural vibration characteristics under the conditions of the time of the tests have been performed using the frame analysis model just as the same as the design analysis
The rocking mechanism of the PCCW was adopted to go free from the deformations of the frames when subjected to the lateral forces. However, additional rigidities by the PCCW to the structural elements of the analysis models were calculated, provided that the rocking or lifting-up deformation of the PCCW did not occur when the vibration tests were conducted. The estimation methods for these additional rigidities are shown in Fig.12.
It is necessary that the lateral forces or axial forces acted to PCCW overcome the vertical forces by the gravity in the PCCW to begin the rocking or lifting-up. The axial, bending and shearing deformations of the PCCW caused by these forces can be calculated as

Table 4. Natural vibration period and damping factor from the test

Direction	Order	Period (sec.)	Damping factor	Notes
Longi.	1	2.73	1.82	Sway 1st
	2	1.00	0.92	Sway 2nd
	3	0.58	1.00	Sway 3rd
Trans.	1	2.92	2.04	Sway 1st
	2	2.16	0.94	Tor. 1st
	3	0.98	1.30	Sway 2nd
	4	0.76	0.98	Tor. 2nd
	5	0.50	1.10	Sway 3rd
	6	0.46	0.89	Tor. 3rd

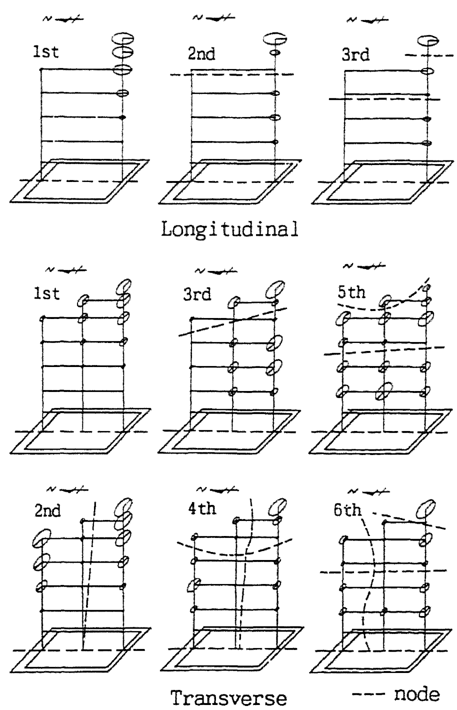


Fig.11 Vibration mode shapes

Table 5. Natural vibration period from the design analysis

Direction	Order	Period (sec.)	Notes
Longi.	1	4.21	Sway 1st
	2	1.33	Sway 2nd
	3	0.70	Sway 3rd
Trans.	1	3.98	Sway 1st
	2	3.07	Tor. 1st
	3	1.13	Sway 2nd
	4	0.99	Tor. 2nd
	5	0.57	Sway 3rd
	6	0.51	Tor. 3rd

shown in Fig.13. Thus, the total deformations of PCCW to start rocking and lifting-up are obtained by the summation of the above values as δ_T=38μm and δ_N=15μm respectively. On the other hand, those values at the time of the test are estimated as 13μm and 1μm from the horizontal displacement of 400 to 500μm of the top floor at the free vibration test. Consequently, neither rocking or lifting-up of the PCCW could occur at time of the tests.

4.3 Natural vibration periods

The natural vibration periods analyzed by the equivalent design analysis model to take into account of the actual weight and additional rigidity, are listed in Table 6 with the test results. The values by the analysis show good agreements to those by the tests.
The minimum input earthquake velocities which will start the rocking and lifting-up of the PCCWs are estimated as 0.09 to 0.12 cm/sec and 0.7 to 0.8 cm/sec respectively from the result of the earthquake response analysis. Therefore these additional rigidities will be expected to disappear for small earthquakes.

5 CONCLUSIONS

The followings are concluded:

1. Provided that the plane frames with different heights in a setback building are given almost the same lateral rigidities to their own weights, the torsional deformations during earthquakes can be reduced.
2. The natural vibration periods obtained from the vibration tests were considerablely shoter than those of the design analysis. These values, however, can be explained by the simulation analysis which is taken into account of the actual weight and the additional rigidity by non-structural elements.
3. These additional rigidities have been proved to disappear when the vibration amplitudes become larger. Therefore, the building will show the dynamic characteristics which were anticipated at the design stage.

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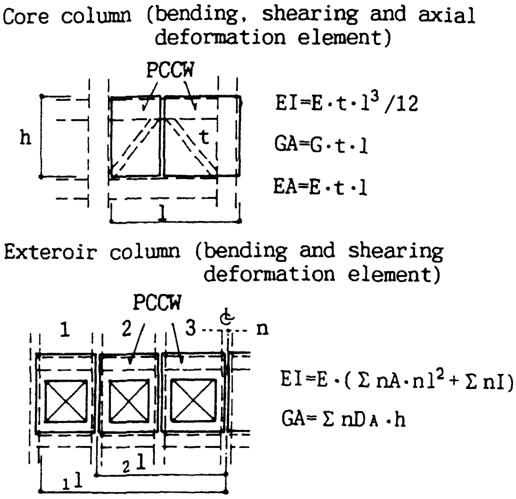

Fig.12 Estimation method of additonal rigidities

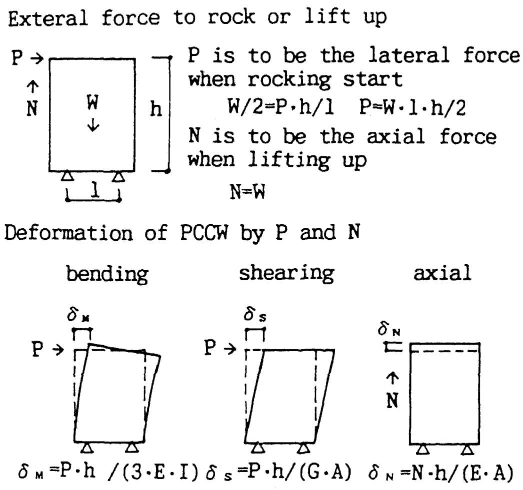

Fig.13 Axial, bending and shearing deformation of the PCCW

Table 6. Natural vibration period from the simulation analysis and vibration test

Direction	Order	period (sec.) test	period (sec.) analysis
Longi.	1	2.73	2.92
	2	1.00	0.93
	3	0.58	0.47
Trans.	1	2.92	2.95
	2	2.16	2.16
	3	0.98	0.83
	4	0.76	0.71
	5	0.50	0.43
	6	0.46	0.37