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FUNDAMENTAL STUDY ON DYNAMIC TORSIONAL RESPONSES OF R/C SPACE STRUCTURES

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SUMMARY

Shaking table tests have been performed to obtain fundamental information regarding the dynamic torsional behavior of RC space structures with the shear walls. Ten specimens were tested and variables considered were ratios of hoop reinforcement for columns and types of base motion input. Each specimen was subjected to a series of scaled earthquake recorded motions or sinusoidal motions at base. Failure progresses, close to the past observed damages of actual buildings, were obtained on the specimens designed based on the RC Code before 1971. On the other hand, considerable level of resistance against gravity load has been found on the specimens designed based on the currently revised Code even after shaking table tests. Measured time histories regarding response displacements have been compared with calculated ones.

INTRODUCTION

In the past earthquakes in Japan, the columns of many reinforced concrete buildings in which shear walls eccentrically located were seriously damaged (Ref.1-3). Most of those buildings had been designed based on the R/C Code before 1971 (Ref.4). The objective of this paper is to obtain the fundamental information regarding the dynamic torsional behavior of RC space structure, with focus on the effects of hoop reinforcement amount of columns upon the overall behavior of the structure. Shaking table tests have been performed with 1/10 scaled specimens of single-story single-bay space structure. Ten specimens were tested and variable considered were ratios of hoop reinforcement for columns (pw: 0.1%, 0.32% and 0.64%), degree of eccentricity by shear walls and types of base motion input. After shaking table tests, vertical loading tests were conducted to estimate the vertical load carrying capacity of the building after earthquake attack.

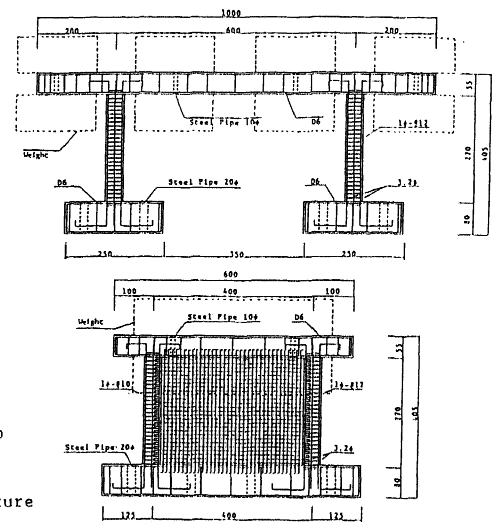
EXPERIMENTAL PROGRAM

Test Structures Ten specimens of single story 1/10 scaled model, were employed in this study. Parameters considered here were three, the first is the eccentricity of the structure, the second is the quantity of hoop reinforcement for columns and the last is a kind of base motion input. A list of all specimens is shown in Table 1. The specimens with no eccentricity, totaled four, have the same four columns while the specimens with eccentricity, totaled six, have a shear wall integrated into the frame in one side in

Table 1 A List of All Specimens

Specimen	Base Motion Input	Figure	Column bc×Dc=45 ×45 (mm)		Wall t=10 (mm)
			pg(%)	pw(%)	wph(%)
RE－C1	Earth-quake Waves	Frame	3.18	0.10	－
RE－W1 RE－W3 RE－W6		Frame + Wall		0.10 0.32 0.64	0.79
RS－C1 RS－C3 RS－C6	Sinu-soidal Waves	Frame		0.10 0.32 0.64	－ － －
RS－W1 RS－W3 RS－W6		Frame + Wall		0.10 0.32 0.64	0.79

Axial Load σ =11.1kg/cm²



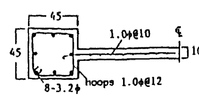


Fig.1 Details of Test Structure

the loading direction. Ten specimens were divided into three catagories in respect with the supply of hoop reinforcement in columns. Four specimens identified by '1', three by '3', the rest three by '6' were respectively supplied with the hoop reinforcement ratios of 0.1% based on RC Code before 1971, o.32% based on RC Code after 1971 and 0.64% the double of the RC Code after 1971. A test structure with dimension is illustrated in Fig.1. The allowable and ultimate strengths of columns and walls for the specimens with eccentricity are shown in Table 2. Test structures were cast into metal form by the same batch of concrete. River sand of 3.2 mm maximum size was used for concrete. The reinforcement used in the fabrication of test structures were of the diameter 1.0 and 3.2 mm and their yield strengths were 3150 and 4800 kg/cm^2 respectively. Weight of steel plates for inertia forces were attached to the concrete slab as illustrated in Fig.1. Total weight of concrete and steel plates was 900 kg.

Base Motions Six specimens were subjected to a series of sinusoidal base motios and four specimens were subjected to a series of scaled recorded earthquake (1940 El Centro EW) base motions. Response spectra of this recorded earthquake base motion are illustrated in Fig.2. Maximum base

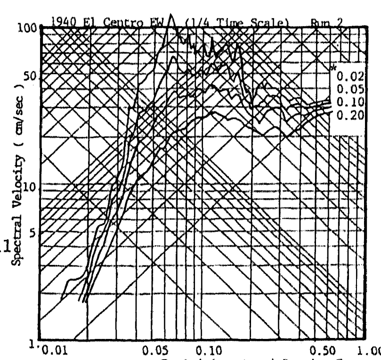


Fig.2 Response Spectra

Table 2 Allowable Strength and Ultimate Strength of Column and Wall

Specimen	Allowable Strength (kg)					Ultimate Strength (kg)					P*** (kg)
	Column		Wall	Summation		Column		Wall	Summation		
	F*	S**		F	S	F	S		F	S	
RS－W1	293	214	1099	1685	1529	319	248	2848	3486	3344	4087
RS－W3		240	1153	1739	1633		312		3486	4028	4165
RS－W6		431	1295	1881	1917		372		3486	3592	3811

* F ; Flexural Strength , ** S ; Shear Strength
*** Ultimate Strength obtained by using actual strength of material

Table 3 Maximum Base Accelerations at Each Input Level

Specimen	Run 1	Run 2	Run 3	Run 4	Run 5
RE－C1	586* 6.44**	854 13.1	1139 24.8	1312 32.6	[1379] 32.9
RE－W1	563 8.50	697 13.8	1046 25.8	[1283] 34.2	1117 33.9
RE－W3	599 9.25	670 14.8	960 25.4	1267 32.5	[1313] 32.2
RE－W6	－	981 12.4	1122 26.3	1252 30.9	1402 38.5

* Maximum Acceleration (cm/sec²)
** Spectrum Intensity (cm)
[ ] at Maximum Base Shear

Specimen	Run 1	Run 2	Run 3	Run 4	Run 5
RS－C1	332	420	[573]	689	859
RS－C3	217	[550]	665	737	853
RS－C6	209	509	[597]	686	746
RS－W1	345	[432]	580	651	922
RS－W3	239	477	[805]	835	835
RS－W6	－	459	[550]	699	1161

[ ] at Maximum Base Shear (cm/sec²)

accelerations at each input level are tabulated in Table 3. Spectrum intensity for earthquake base motions are also included in this table.

EXPERIMENTAL RESULTS AND DISCUSSIONS

Failure Progress Failure progresses are described in two parts, first, for specimens with 0.10% hoop reinforcement for columns, secondly, for specimens for 0.32% and 0.64% hoop reinforcement for columns (Photo:(a),(b),(c)).

(1) For specimens with 0.10% hoop reinforcement for columns: Shear cracks in columns for specimens, with no eccentricity, subjected to earthquake base motions were observed in the shaked direction. These cracks apparently enlarged and lead to shear failure. As the columns for specimens with eccentricity were torsionally and biaxially loaded, cover concrete failed at the first stage and bidirectional shear failure was observed at last stage. These shear failure patterns were close to the actual damages (Hachinohe Library). Diagonal cracks were also observed into the walls for torsional load. The columns for specimens subjected to sinusoidal base motions were considerably damaged than those for specimens subjected to earthquake base motions. At the last stage, the core concrete of columns regardless of the degree of eccentricity were crashed so severely that the weight of steel plates could not be supported. These damages were similar to those for Mutsu City Hall.

(2) For specimens with 0.32% and 0.64% hoop reinforcement for columns: A number of shear and flexural cracks were observed at the top and bottom end of the columns for all specimens with o.32% and 0.64% hoop reinforcement, regardless of the degree of eccentricity of the specimens. Although the cover concrete was peeled out core concrete remained uncrashed so that the weight could be supported even at the final stage.

Initial Periods and Modes Observed fundamental natural periods shown in Table 4 were obtained from the small amplitude free vibration test conducted between strong base motion tests. All values were similar regardless of degree of eccentricity. Calculated values are also included in this table. These values were obtained from the equation of motion derived in terms of relative displacement to the base and rotational angle in horizontal plane of slab. Good agreements are found between observed and calculated values for all specimens. Calculated modes for specimen with eccentricity are illustrated in Fig.3. In the first mode, location of rotational center was close to shear wall so that displacement was concentrated to the column in the opposite side of the wall.

Table 4 Initial Periods

Specimen	Tcal (sec)	Texp (sec)	Exp./Cal.
RE−C1	0.051	0.058	1.14
RE−W1	0.048	0.053	1.10
RE−W3	0.048	0.052	1.08
RE−W6	0.047	0.050	1.06
RS−C1	0.050	0.054	1.08
RS−C3	0.048	0.053	1.10
RS−C6	0.048	0.052	1.08
RS−W1	0.049	0.054	1.10
RS−W3	0.047	0.050	1.06
RS−W6	0.049	0.051	1.04

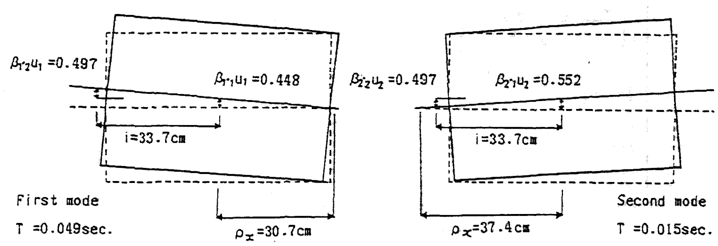


Fig.3 Calculated Modes for Specimens with Eccentricity

Maximum Base Shear The observed and calculated maximum base shears are presented in Table 5. Maximum base shear was obtained as follows.

$$\text{Maximum Base Shear} = M\ (\ddot{x}i+\ddot{x}o) \quad (1)$$

M : total mass of steel plate and concrete slab weight

$\ddot{x}i+\ddot{x}o$: absolute acceleration at mass center in loading direction

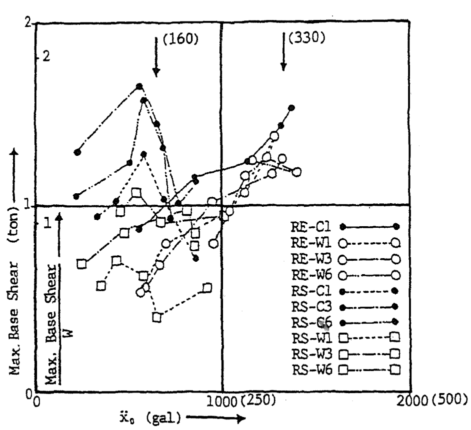
Calculated values P_1, P_2 as shown in Table 5 were obtained by assuming that P_1, P_2 are the summation of column strength (shear or flexural) and wall shear strength, and this summation value minus the horizontal rotational effect of the slab, respectively. It is found that the observed maximum base shears of the specimens with no eccentricity in sinusoidal base motion tests are similar to the corresponding calculated values while those of specimens with eccentricity are similar to the calculated values including the above twisting moment effects of the slab, less than the strength of the specimens with no eccentricity. On the other hand, these values of the specimens in earthquake base motions are considerably higher than the corresponding calculated values. This seems to be due to the higher mode of the coupling between translational and rotational vibrations.

Table 5 Observed and Calculated Maximum Base Shear

Specimen		Maximum Base Shear			Deformation Angle R **
		P (kg)	P_1(kg) (P/P_1)	P_2(kg) (P/P_2)	
RE－C1	S*	1532	1148(1.33)	1148(1.33)	1/26
RE－W1	S	1383	3740(0.37)	749(1.85)	1/37
RE－W3	F	1382	4165(0.33)	951(1.45)	1/40
RE－W6	F	1258	4165(0.30)	951(1.32)	1/20
RS－C1	S	1274	1076(1.18)	1076(1.18)	1/26
RS－C3	F	1667	1396(1.19)	1396(1.19)	1/29
RS－C6	F	1575	1396(1.13)	1396(1.13)	1/32
RS－W1	S	713	4087(0.17)	788(0.90)	1/45
RS－W3	F	998	4165(0.24)	951(1.05)	1/33
RS－W6	F	1080	3811(0.28)	951(1.14)	1/28

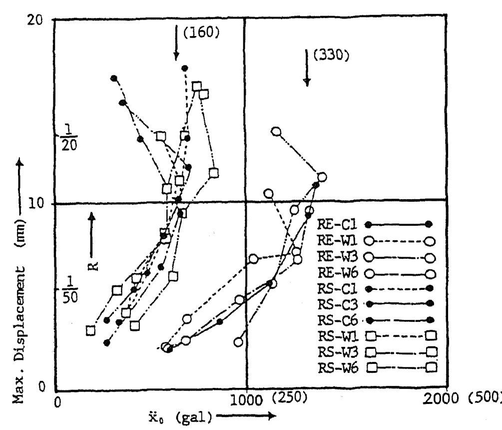
* Failure Mode S ; Shear Failue F ; Flexural Failue
** at Maximum Base Shear

Magnification Factors Figures 4 and 5 show the transitions of the maximum base shear and the maximum displacement measured regarding the base acceleration level at each run. The values of the maximum base shear is the total weight multiplied by the maximum acceleration measured at the mass center of slab, which is the same as in the last item just discussed. The maximum displacements are those at the columns line of the structure in loading direction. It is found that the base shear reach its peak at the base acceleration level of 500 gal in the sinusoidal tests. From this value, the deformations increase up to the 1.8 cm at final stage, in spite of the decrease of base acceleration level. On the other hand, the base shear increases up to the level of 1200-1300 gals regarding base acceleration. The inverted ratio of these values (1200-1300/500=2.5) shows that of destructive power effects of both the sinusoidal and earthquake waves on buildings although the values 500 gals, 1200-1300 gals should be multiplied by one-fourth because of the lack of attached weight placed on the specimens in comparison to a full-size space structure.



(a) Base Shear

Fig.4 Transitions of Maximum Base Shear



(b) Displacement

Fig.5 Transitions of Maximum Displacement

Dynamic Response Analysis The inelastic dynamic response analysis was performed to obtain time history of specimen with eccentricity at each input level for both type of base motions. A specimen was replaced by two degrees of freedom system, with lateral displacement in loading direction and rotation at the center of rigidity. Restoring force characteristics was assumed as origin oriented model with bilinear envelope curve having circle yield line in biaxial direction. A step-by-step numerical integration procedure is used to solve the equations of motion. The equation of motion can be written as follows.

$$[M]\{\Delta\ddot{x}\}+[C]\{\Delta\dot{x}\}+[KH]\{\Delta x\}=-[M]\{\Delta\ddot{x}o\} \qquad (2)$$

$[M]$: diagonal mass matrix
$[C]$: damping matrix
$[KH]$: structural stiffness matrix
$\{\Delta\ddot{x}\},\{\Delta\dot{x}\},\{\Delta x\}$: relative incremental acceleration, velocity and displacement vector respectively.
$\{\Delta\ddot{x}o\}$: base acceleration vector

The implicit form of the Newmark Beta method with $\beta=1/4$ a increment time T=0.005 sec. is chosen in this study. A damping matrix proportional to just the initial stiffness matrix is used. Calculated and observed time histories of lateral displacement are illustrated in Fig.6. Good agreements are found at from low to high input levels.

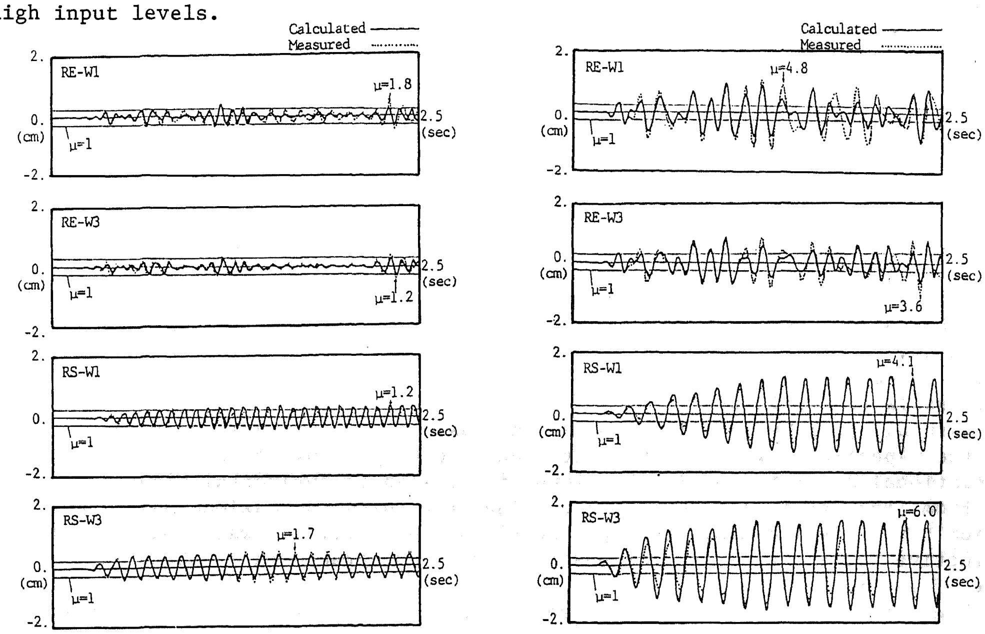


Fig.6 Time Histories of Lateral Displacement

Vertical Loading Test In order to investigate the safety against gravity load after earthquake attack, statically applied vertical loading tests for two specimens (RS-C3, RS-W3), with 0.32% hoop reinforcement in columns, were performed after shaking table test. In these specimens, complete crash down was not observed different from the specimens with poor hoop reinforcement. Test results are shown in Fig.7 and Photo (d). The vertical load carrying capacity of non-eccentric and eccentric specimens were nearly equal and two-third of the cummulative compression strengths respectively. The decrease of the vertical strength in the latter specimens was due to the fact that those columns were subjected to torsionally and biaxially motions during the shaking table test.

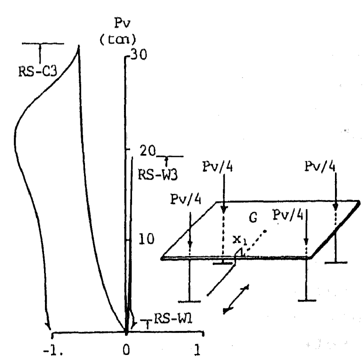
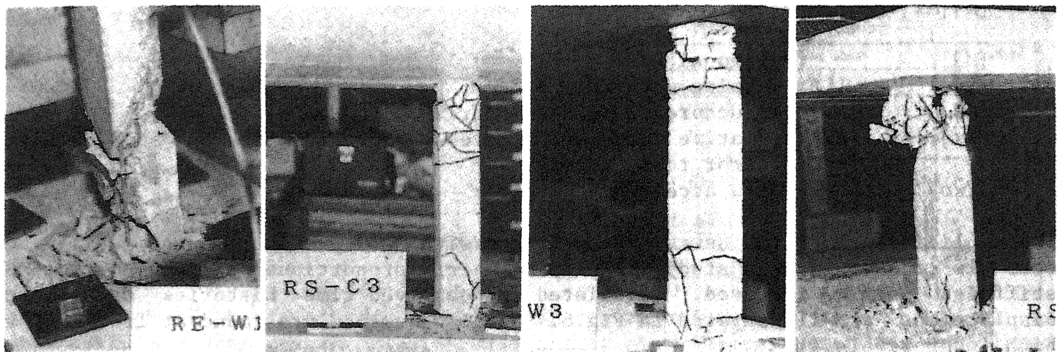


Fig.7 Vertical Load versus Lateral Displacement



(a)RE-W1 (b)RS-C3 (c)RS-W3 (d)RS-C3

Photographs of Specimens after Test

CONCLUSIONS

This paper presents the shaking table test conducted to investigate the dynamic torsional behavior of reinforced concrete space frame structure, when subjected to strong base motions. Based on the experimental and analytical results, the following conclusions can be made.

(1) Failure progresses, close to the past observed damages of actual buildings were obtained on the specimens with 0.10% hoop reinforcement for columns. On the other hand, considerable level of resistance against gravity load has been found on the specimens having 0.32% hoop reinforcement for columns, even with considerable eccentricity.

(2) Natural periods as well as modes and maximum response values of shear force for the specimens with eccentricity can be estimated by considering both the translational and rotational vibrations of the slab in horizontal planes.

(3) From the trend of the response values regarding the maximum base shear and maximum top displacement to input acceleration level, it was found that the destructive power effects of sinusoidal waves on structures is about 2.5 times those of earthquake waves.

(4) The inelastic dynamic response analysis was performed to obtain the time history of specimens with eccentricity at each input level for both types of base motions. Good agreements were found between the measured and calculated results.

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