SHAKING TABLE TESTS ON 3-STORY BRACED AND UNBRACED STEEL FRAMES

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SUMMARY

A series of shaking table tests were performed on three-story moment resistant (unbraced) steel frames and braced steel frames. Two sets of moment resistant frames, each of which was designed to have a different story shear strength, were subjected to earthquake acceleration records to examine the response shear forces and the damage concentration. Braced frames with X-type braces were also subjected to the acceleration records. The results of response displacements and story shear forces were used to verify the analytical procedure where the inelastic behavior of the structural elements were presumed.

INTRODUCTION

It is widely recognised that buildings ordinarily designed in accordance with current code procedures may undergo considerable inelastic deformations under severe earthquake excitations. In design procedures, recent computer algoriths have potential to predict the inelastic behavior precisely. However, experiments are futhermore important, because precise computer analyses require more accurate informations on structural behavior to complete the analytical models used in calculation procedures, and because the global response behavior of whole systems which are assembled from structural elements must be verified by experiments. In the past, the considerable amounts of shaking table tests were carried out and reported in the literatures. But among them only a few tests have been performed on multi-story frames (Ref. 1,2,3). Therefore, it should be necessary to accumlate further experimental data on multi-story frames. In this scope, a series of shaking table tests were performed on three-story moment resistant (unbraced) steel frames and three-story braced steel frames with X-type braces. The objective of this paper is to describe these test results focusing on the influence of the design profile of story shear strength on response behavior in the moment resistant frames, the dynamic stability of braced frames after the buckling of braces, and the overall story shear response in unbraced and braced frames. Furthermore, the prediction of response behavior by an analytical model was discussed in comparison with the test results obtained.

TEST STRUCTURES

The test structures, shown in Fig.1(a) and (b), were fabricated from the rectangular rods which were shaped out of the mild steel plate (JIS 41 steel having a mill test yield point of $2.4t/cm^2$). Two identical planar frames were spaced 800 mm apart, and were connected at each column top by rolled

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angle beams. The dimensions of the columns and brace members are summarized in Table 1. The sections of the girders are identical through all test structures and larger than any column. As recognized by Fig.1 and Table 1, the test structures are miniature models, say, one-tenth scale models of ordinary structures. The test frames are recognized as code AE, AH, BE, BH, CE and CH. Frame AE and AH were identically designed to have a uniform design profile of story shear strength, while Frame AE and AH were designed to have the design profile recommended as the Ai coefficient in Building Standard Law of Japan. Braced Frame CE have almost same story shear strength as Frame AE and AH in total. Brace members were so proportioned to carry three-quaters of shear strength in each story. The profiles of story shear strength are sketched in Fig.2. The brace members, the slenderness rarios of which are approximately 340, were stretched to keep a half of the tensile yield strength at installation. Piles of steel plates weighing about 590 kg/ floor were placed on the connecting angles between the planar frames. The combinations of these weights and stiffnesses of frames determine the nutural periods, which were measured by the resonance vibration tests done prior to the inelastic response tests. The structural properties of the test frames are summarized in Table 2.

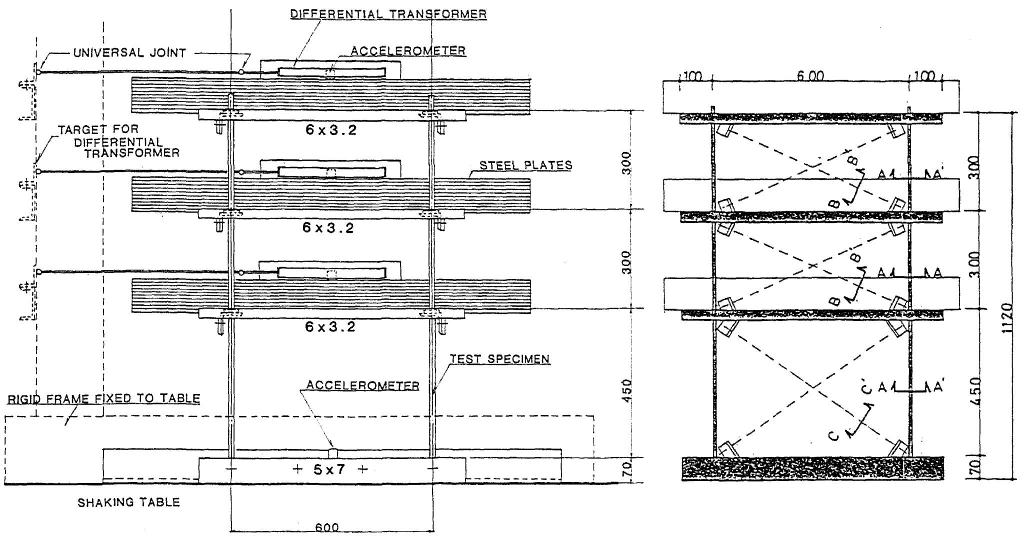
Table 1 Dimensions of Columns and Braces

TEST FRAMES	1ST STORY	2nd STORY	3RD STORY
	COLUMN bxd (cm)	COLUMN bxd (cm)	COLUMN bxd (cm)
AE, AH	3.0 x 1.2	3.0 x 1.2	3.0 x 1.2
BE, BH	3.0 x 1.2	1.5 x 1.2	1.1 x 1.2
CE, CH	1.5 x 0.9	1.5 x 0.9	1.5 x 0.9
	BRACE bxd (cm)	BRACE bxd (cm)	BRACE bxd (cm)
CE, CH	0.45 x 0.23	0.6 x 0.23	0.6 x 0.23

Table 2 Structural Properties of Structures

Frame and Test Code	Input Exitation*	Weight of Each Floor		Vertical Strength Distribution		Summary of Resonance Vibration Tests				
		Fl.	Weight (kgw)	St.	Yield Stort Shear Coeff	Mode	Natural Period (sec)	Participation Factor 1 Fl.	2 Fl.	3 Fl.
AE	El Centro NS max. 576 gal	1	590	1	0.38	1	0.39	0.86	1.01	1.11
		2	583	2	0.91	2	0.098	0.13	0.02	-0.12
		3	585	3	1.81	3	0.057	0.02	-0.04	0.02
AH	Hachinohe EW max. 333 gal	Same as AE		Same as AE		Same as AE				
BE	El Centro NS max. 530 gal	1	598	1	0.38	1	0.42	0.72	0.98	1.19
		2	582	2	0.45	2	0.14	0.20	0.08	-0.20
		3	584	3	0.67	3	0.086	0.06	-0.07	0.03
BH	Hachinohe EW max. 358 gal	Same as BE		Same as BE		Same as BE				
CE	EL Centro NS max. 879 gal	1	588	1	0.42	1	0.10	0.67	0.98	1.20
		2	581	2	0.97	2	0.034	0.22	0.06	-0.19
		3	584	3	1.94	3	—	—	—	—
CH	Hachinohe EW max. 611 gal	Same as CE		Same as CE		Same as CE				

* Original duration 10.24sec is o condensed to 5.12sec



(a) Moment Resistant Frame (b) Braced Frame

Fig.1 Elevation of Test Frames

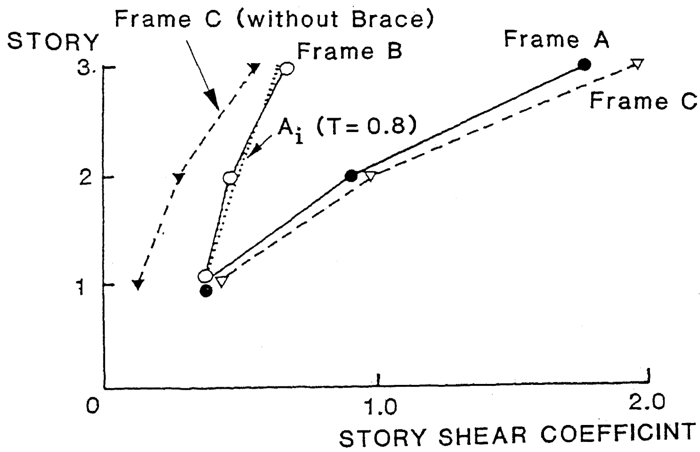


Fig.2 Design Profiles of Shear Strength

INSTRUMENTATION

Relative displacements in the direction of motion were measured by differential transformers at each floor level to the column base. Absolute accelerations at each floor were measured by accelerometers. Strains at the several sections of a column were measured by strain gages pasted on the column surfaces to know the inception of yielding and to examine the story shears calculated from the measured acceleration data. The data of a channel were sampled at a rate of 100 readings per second and recorded temporarily in digital form on a magnetic disk. After completion of each test run the recorded data were transfered to a magnetic tape.

TEST PROCEDURE

The shaking table used during these tests is installed at the Chiba Experiment Station, Institute of Industrial Science (the square table of 1.5 m × 2.0 m, the maximum driven capacity of 4.7 ton-g, the maximum stroke of 75 mm and the maximum carring weight of 5.0 ton). The motion of the table was controlled by the computer to produce the scaled acceleration wave of either the NS component of the 1940 El Centro record (the Imperial Valley Earthquake), or the 1968 Hachinohe record (the Tokachi-oki Earthquake). But the original records were condensed into a half of the original duration, because the fundamental periods were equal to or less than 0.4 second which is considered smaller as the period of a three-story steel frame. The peak value of the acceleration used for each test is shown in Table 2.

TEST RESULTS

As typical measured response waveforms, sets of the absolute acceleration record of the first floor and the relative story dispalcement record to the column base of the third floor in Frame AE and CE are shown in Figs.3 and

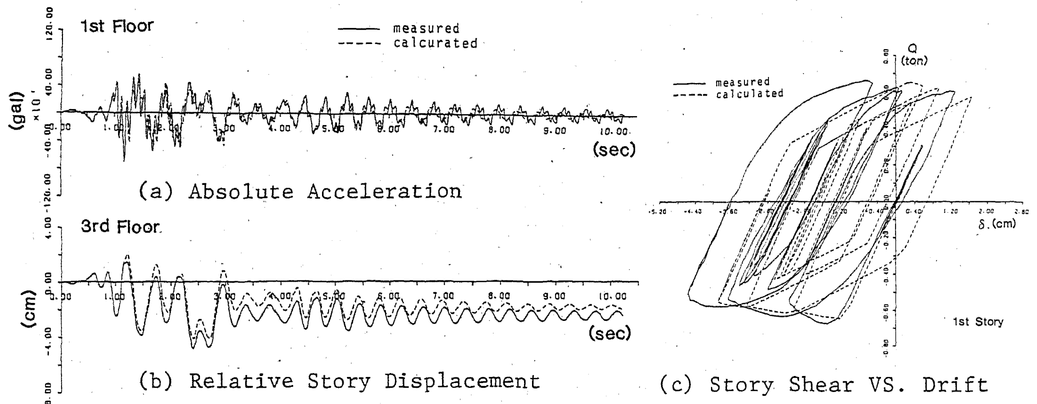


Fig.3 Measured and Calculated Responses

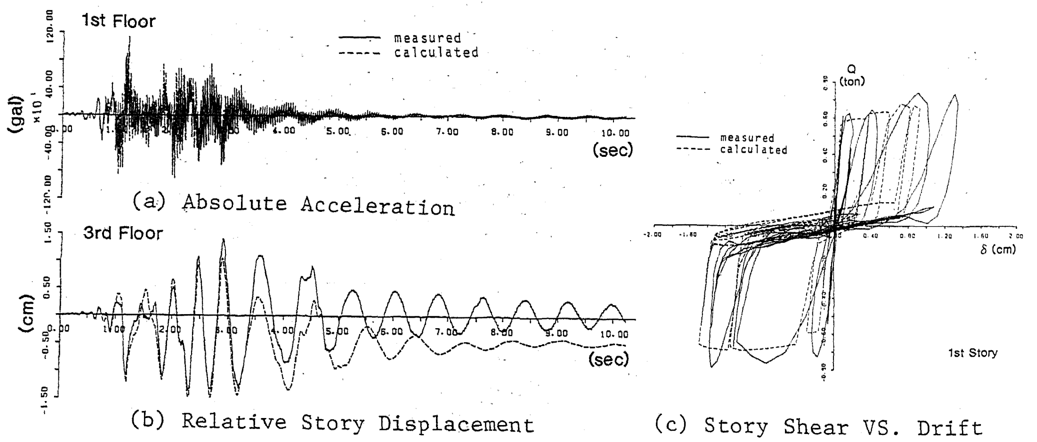


Fig.4 Measured and Calculated Responses

4, respectively. From these data, the story shear force and the story drift in a story were calculated to develop the story shear Q vs. the story drift δ hysteresis loops as shown in Figs.3(c) and 4(c).

DISCUSSIONS OF TEST RESULTS

In the moment resistant frames, AE and AH, which were endowed with the uniform story shear strength, the yielding of the columns was observed only in the first story. That resulted in the considerable permanent displacement of the first story after tests. The hysteresis loop in Fig.3(c) shows this evidence. In the moment resistant frames, BE and BH, which were provided with the degraded strength along the height, the yielding was observed in the second story as well as in the first story. However, a large amount of the absorbed energy was provided by the inelastic deformation of the columns in the first story as described in Fig.6. Compared to Fig.5, an effect of the improved shear strength distribution was noticed. But it must be noted that the incident of yielding is very sensitive to the shear strength distribution and the input ground motion. In the braced frame, CE, all braces were buckled, while the columns remained in the elastic range. The hysteresis loops in Fig.4(c) become squeezed after the buckling of the braces. During the test of CH, the rupture of a brace occured. The rupture of the braces in a side frame caused a rotational vibration due to the unbalanced resistance, which resulted in the complete collapse of the first story.

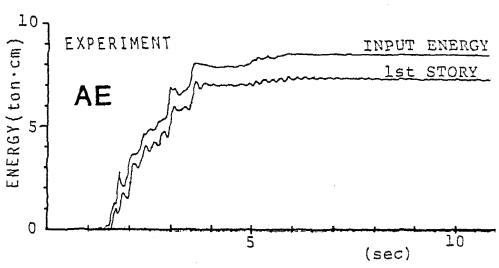


Fig.5 Input and Absorbed Energy

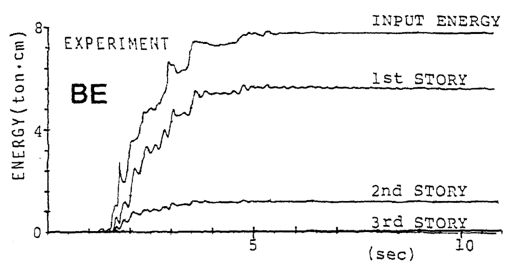


Fig.6 Input and Absorbed Energy

After processing the absolute acceleration data, the story shear force of each story was obtained. A set of the three story shear force, Q_1, Q_2 and Q_3 take a point in the three dimensional space (Q_1, Q_2, Q_3). The point is time-dependent and then it describes a locus. To represent the locus a set of three projections of the locus is used. Figs.7, 8 and 9 show such projections of the loci in AE, BE and CE tests, respectively. In the figures, the sections of the yield surfaces which were determined by the yielding of the columns and the braces are also described. The dashed line shows the projection of the locus direction of the point (Q_1, Q_2, Q_3) in the first mode vibration obtained by an elastic analysis. The intersection of a curve and a yield surface means the inception of the yielding. It is explicit that the yielding always took place at the first story in the AE test. But in the BE test the intersecting occurred at the vicinity of the corners of the yield surfaces. This evidence shows the improvement in the design profile of the

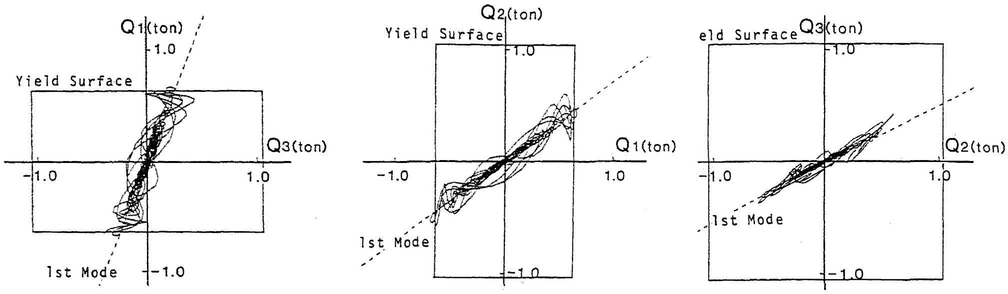


Fig.7 Projections of Response Shear Forces in AE

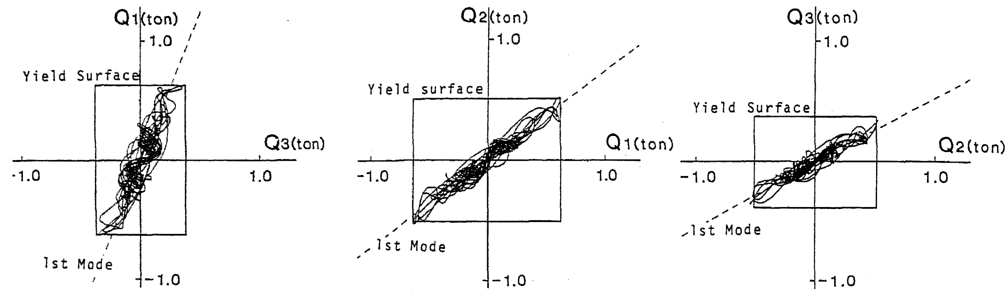


Fig.8 Projections of Response Shear Forces in BE

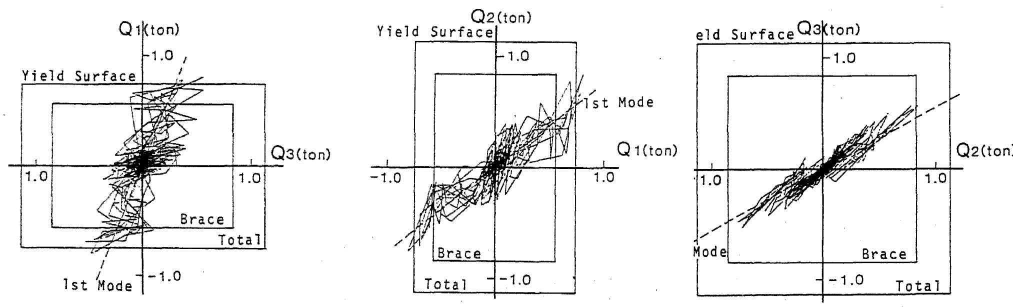


Fig.9 Projections of Response Shear Forces in CE

story strength, since that a projection curves meet the corners of the yield surface shows the simulaneous yielding at both two stories. From the viewpoint to avoid the damage concentration due to yielding, the simultaneous yielding at all stories is most desirable. Then, it can be stated that the Ai coefficient of Building Standard Law aims at the optimum design.

It is worthy to note that even in the inelastic vibration the response behavior is strongly controlled by the first mode vibration both in the moment resistant frames and in the braced frames. It suggests a practical design procedure in evaluating the ultimate strength of a structure under the earthquake excitation.

FORMULATION OF ANALYTICAL MODELS

The test results described are used to develop an analytical model for computer analyses. The test structures were so-called weak-column type structures. Therefore, only the inelastic behavior of the columns was considered in formulating the analytical model. The key analytical model is shown as a hysteretic rule of the story shear Q vs. story drift δ relationship in Fig.10. In the rule, a bi-linear type model is assumed for the first cycle, and thereafter a tri-linear type model is assumed. The elastic stiffness k in the model was derived from the elastic stiffness of the columns, and the elastic limit Q_Y corresponded to the formulation of plastic hinges at the tops and the bottoms of the columns. In other parameters, Q_B was considered the two-thirds of Q_B in order to express the Bauschinger's effect in the hystersis loops. The parameters, γ_1 and γ_2 were determined after some analyses. Table 3 shows the calculation results of the trial analyses where several values were assigned to γ_1 and γ_2. The values of the total input energy calculated were not so sensitive to these parameters, but the story drifts were extremely sensitive. Among those trial values, 0.3 and 0.03 were selected for γ_1 and γ_2, respectively. The P-Δ effect must be considered in the analysis. The P-Δ effect causes a negative slope which was evaluated from the floor weight sustained and the clear height of the columns.

The hysteretic rule for brace members was presumed to be a simple slip model as shown in Fig.11. To describe the rule of braced frames the models in Figs.10 and 11 must be combined. Some results of the analyses using the above models are shown by the dashed curves in Figs.3 and 4. Comparison between the test results and the analytical results was made at the peak values of the acceleration waveforms and the displacement waveforms. These peak values are summarized in Table 4.

CONCLUDING REMARKS

1. The evidence that the yielding only in a few stories causes the damage concentration due to large plastic deformation was observed in the inelastic response tests.

2. The design profile after the Ai coefficient of Building Standard Law of Japan may reduce the damage concentration. However, some concentration

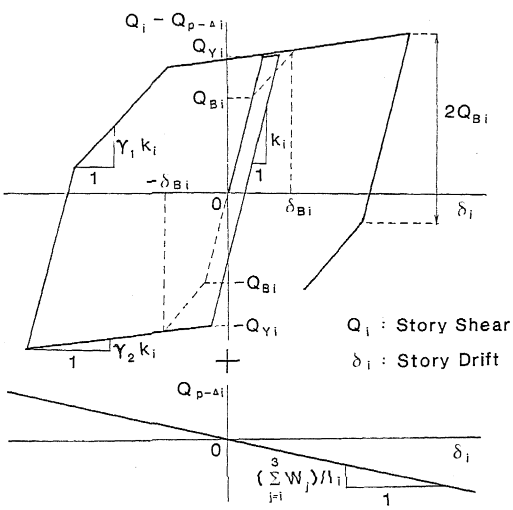


Fig.10 Hysteretic Rule of Q VS. δ

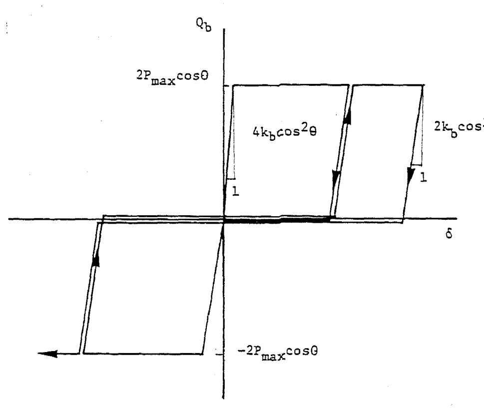


Fig.11 Hysteretic Rule of Braces

Table 3 Correlation of Measured and Computed Responses

(1) Total Input Energy

Frame and Test Code			AE	AH	BE	BH
Experiment			8.52 t·cm	8.47 t·cm	7.73 t·cm	9.73 t·cm
Tri-linear	γ_1 = 0.3	γ_2 = 0.0	7.75 (0.91)	8.15 (0.96)	7.49 (0.97)	9.08 (0.93)
	0.5		7.57 (0.89)	7.68 (0.91)	7.58 (0.98)	8.60 (0.88)
	0.7		7.39 (0.87)	7.32 (0.86)	7.72 (1.00)	8.51 (0.87)
Bi-linear	——	γ_2 = 0.0	7.31 (0.86)	7.24 (0.85)	7.64 (0.99)	8.26 (0.85)
Tri-linear	γ_1 = 0.3	γ_2 = 0.03	8.06 (0.95)	8.49 (1.00)	7.55 (0.98)	9.34 (0.96)
	0.5		8.05 (0.94)	7.83 (0.92)	7.72 (1.00)	8.70 (0.89)
	0.7		7.88 (0.92)	7.57 (0.89)	8.03 (1.04)	8.26 (0.85)
Bi-linear	——	γ_2 = 0.03	7.59 (0.89)	7.40 (0.87)	8.28 (1.07)	7.88 (0.81)

(2) 1st Story Drift

Frame and Test Code			AE	AH	BE	BH
Experiment			4.57 cm	4.33 cm	2.88 cm	4.26 cm
Tri-linear	γ_1 = 0.3	γ_2 = 0.0	4.49 (0.98)	4.81 (1.11)	3.40 (1.18)	4.63 (1.09)
	0.5		6.70 (1.47)	6.22 (1.44)	2.25 (0.78)	6.90 (1.62)
	0.7		5.42 (1.19)	5.36 (1.24)	5.80 (2.01)	2.77 (0.65)
Bi-linear	——	γ_2 = 0.0	6.63 (1.45)	5.52 (1.27)	6.02 (2.09)	4.65 (1.09)
Tri-linear	γ_1 = 0.3	γ_2 = 0.03	3.88 (0.85)	4.34 (1.00)	2.67 (0.93)	4.48 (1.05)
	0.5		4.48 (0.98)	5.27 (1.22)	2.37 (0.82)	5.84 (1.37)
	0.7		4.09 (0.89)	4.78 (1.10)	2.97 (1.03)	3.64 (0.85)
Bi-linear	——	γ_2 = 0.03	4.15 (0.91)	5.00 (1.16)	2.57 (0.89)	4.99 (1.17)

Table 4 Measured Peaks of Acceleration and Displacement Responses

Frame and Test Code	Floor	Acceleration (gal)	Displacement (cm)
AE	1	691 (1.02)	4.57 (0.85)
	2	415 (1.05)	4.83 (0.84)
	3	723 (0.97)	4.87 (0.85)
AH	1	453 (1.39)	4.33 (1.00)
	2	405 (1.02)	4.49 (1.00)
	3	483 (1.26)	4.56 (1.01)
BE	1	495 (1.47)	2.88 (0.91)
	2	488 (1.19)	3.58 (0.94)
	3	600 (1.10)	3.73 (0.97)
BH	1	572 (1.26)	4.26 (1.05)
	2	523 (1.00)	4.74 (0.98)
	3	649 (1.02)	4.88 (0.99)
CE	1	892 (1.28)	1.38 (1.08)
	2	513 (1.41)	1.41 (1.05)
	3	947 (1.24)	1.40 (1.06)
CH	1	994 (1.07)	4.81 (1.78)
	2	626 (1.51)	5.20 (1.64)
	3	1161 (1.01)	5.03 (1.70)

Parenthesized Values are the Ratios of Computed to Measured

must be considered, since the inception of yielding at a story depend on an excited vibration behavior.

3. Even in the inelastic vibration the response behavior is strongly controlled by the first mode vibration in the test structures.

4. A tri-linear type hysteretic rule of the story shear vs. story drift is avaiable to predict the response behavior of the test structures. The rigorously determined rule must be used in order to predict the response displacement precisely.

5. More rigorous models must be developed to express the behavior of the braced structures.

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