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EARTHQUAKE SIMULATOR TESTING OF A CONCENTRICALLY BRASED STEEL STRUCTURE

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SUMMARY

A six-story concentrically braced dual steel system (CBDS) was subjected to a total of twenty earthquake ground motions with effective peak accelerations of up to 0.40g on the earthquake simulator at the University of California at Berkeley. The design of the CBDS complied with the lateral force requirements of the 1985 UBC. The maximum strength of the CBDS was 2.4 times greater than its nominal yielding strength; however, the response modification factor currently adopted by the ATC for CBDSs (=6) was 67% greater than that measured during the testing program.

INTRODUCTION

The Joint Technical Co-ordinating Committee for the U.S.-Japan Cooperative Research Program Utilizing Large Scale Testing Facilities selected a six-story, two bay by two bay, steel framed office building with a composite steel metal deck and a lightweight concrete floor system as the test structure for intensive investigation (Ref. 1), under simulated earthquake loading. A full-scale structure was constructed and pseudo-dynamically tested in the Large Size Structures Laboratory of the Building Research Institute (BRI) in Tsukuba, Japan (Refs. 2,3).

The plan view and braced frame elevation of the full-scale CBDS is shown in Fig. 1. The structure, 49.21 × 52.49 ft (15.0 × 16.0 m) in plan and 73.43 ft (21.5 m) high, consisted of three frames parallel to the loading direction; two ductile moment-resisting space frames (DMRSFs) on Grid Lines A and C and an concentrically braced frame on Grid Line B. In the transverse direction there were three frames: two X-braced frames on Grid Lines 1 and 3 and an unbraced frame on Grid Line 2.

Two reduced-scale models were tested at the University of California at Berkeley: concentric bracing (Refs. 4,5) was incorporated into the first model and eccentric bracing (Refs. 6,7) into the second model. Only the test results pertaining to the first model are discussed below due to the length limitations on this paper.

CBDS DESIGN REVIEW

The design of the original full-scale CBDS was based upon the 1979 UBC and the 1981 Japanese Aseismic Design Code (JADC). The design gravity loads for the full-scale CBDS are presented in Table 1. Although the design loads do not represent the minimum quantities specified in the USA or Japan, the total gravity load was appropriate for both countries. The seismic forces were evaluated using the 1981 JADC. The base shear coefficient specified by the JADC for the CBDS was significantly larger than the UBC coefficient. However, by making different assumptions regarding site conditions and assigning twice the UBC designated level of lateral force to the DMRSF (that is, 50% of the design lateral force), a base shear coefficient of 0.197 at the service load level was chosen (Ref. 2). The design reactive weight selected by the design group did not include the floor live loads (currently ignored by the U.S. seismic regulations for office buildings) or the weight of the perimeter walls. If these loads had been included, the resultant lateral load resisting system would have been too strong to be suitably damaged in the BRI testing facility.

The original design satisfied the 1985 UBC (Ref. 8) requirements for a dual system provided that the effective length factor for the braces was taken as 0.7; this assumption was consistent with welded connection used in the full-scale structure. The requirement that the DMRSFs resist 25% of the design base shear was also satisfied.

DESIGN AND CONSTRUCTION OF THE MODEL CBDS

A primary objective of the studies at the University of California was to design, construct and test the largest possible model of the full-scale CBDS that could be accommodated on Berkeley's earthquake simulator. Considering a number of factors that included the weight and size limitations of the earthquake simulator, the most suitable model was determined to be an artificial mass simulation model with a length scale factor of 0.305; this model satisfied similitude with regard to geometric and loading parameters The mass density similitude requirement was satisfied by fastening lead ballast to the roof and floor slabs in such a manner that it did not increase the stiffness of the model CBDS (Refs. 4,5).

EXPERIMENTAL RESULTS

General To compare the experimental results presented below with the provisions of the current U.S. earthquake-resistant design regulations, a number of factors must be noted (Ref. 4), these include :

(1) the design base shear coefficient at working stress levels of 0.197 was significantly larger than that required by the 1985 UBC and the 1984 ATC (Ref. 9); therefore, the design, elastic stiffness, elastic strength and maximum strength of this CBDS cannot be considered as being representative of these seismic regulations;

(2) the as-tested reactive weight (W_{a-t}) of the CBDS was chosen to be 82% of its design reactive weight; as a result the nominal yielding strength of the CBDS was equal $0.3W_{a-t}$ (Refs. 4,5);

(3) the CBDS was a bare steel structure and the important effects of the interaction of structural and non-structural components could not be considered.

Testing Program The model CBDS was subjected to twenty simulated ground motions. The 1978 Miyagi-Ken-Oki (MKO) earthquake record was used as the input displacement signal for the majority of the tests; the MKO command signal was time-scaled in accordance with the similitude laws and the peak acceleration for each test was scaled to different levels to simulate different limit states of response. Only the results of the collapse level test with a peak input acceleration of 65%g (MKO-65) are presented below.

Dynamic Characteristics of the Model Prior to earthquake simulator testing, flexibility tests and free and forced vibration tests were undertaken to measure the dynamic characteristics of the model CBDS. Table 2 presents the natural periods and damping ratios of the first three modes of the model CBDS together with those of the full-scale CBDS.

MKO-65 Test Results The effective peak acceleration (EPA) of the MKO-65 Test was 0.40g and equal to to the maximum EPA adopted by ATC for regions of high seismic risk. During the MKO-65 Test, the braces in the bottom five stories buckled (either in-plane and/or out-of-plane); one brace in the fifth story ruptured at midspan and one in the fourth story ruptured at its lower end. As a result of brace rupture, a maximum inter-story drift index of 1.9%, exceeding the maximum UBC and ATC value of 1.5%, was developed in the fifth story during this test. The lateral displacement, inertia force and story shear profiles over the height of the CBDS at the times of minimum and maximum base shear are shown in Fig. 2; the inertia force profiles reflect the formation of a soft fifth story. The maximum base shear coefficient ($=V_b/W_{a-t}$) of 0.73 was more than six times the UBC base shear coefficient (=0.113) for this collapse limit state earthquake and 2.4 times the nominal yielding strength of the CBDS ($= 0.3W_{a-t}$).

Story Shear and Inter-story Drift The total story shear envelope (V^{TOTAL}), the envelope of the story shear resisted by the concentric braces (V^{BRACE}) and the envelope of the story shear resisted by the DMRSF (V^{DMRSF}) for the first, third and fifth stories of the CBDS are shown in Fig. 3. The DMRSF remained elastic at a fifth inter-story drift index approaching 1.5%; this fact emphasizes the lack of stiffness compatibility between the braced frame and the DMRSF. Of the three stories, only the V^{BRACE} envelope in the fifth story shows signs of strength deterioration; this observation is consistent with the rupture of the concentric braces in the fifth story. However, the total fifth story shear and the inter-story drift relationship clearly shows that as a result of significant strength of the DMRSF, the fifth story shear resistance remain stable, that is, non-decreasing, following brace buckling and rupture. Stable story shear envelopes are mandatory for the sound performance of all structures during severe earthquake shaking.

Response Modification Factors In earthquake-resistant design the structure linear elastic strength demand can be reduced if the ductile behavior of the structural components is provided. ATC (Ref. 9) establishes a linear elastic design response spectrum (LEDRS) for 5% damping and then uses a *response modification factor* (R) to reduce the LEDRS to the minimum required design base shear coefficient. If the increase in damping due to inelastic behavior is neglected, the response modification factor can be considered to be the product of a reduction in the required elastic strength due to ductility ($R_\mu \equiv$ ductility factor) and a strength factor (R_S) defined as (Refs. 6,7) :

$$R_S = \frac{\text{Maximum Strength Ratio}}{C_y} = \frac{(\text{Overstrength } +1) \times C_y}{C_y}$$

The actual response modification factor (R) can therefore be represented as :

$$R = R_\mu \times R_S \,.$$

The total reduction from the MKO-65 LERS to the nominal yielding strength of $0.3W_{a-t}$ was by a factor of 3.6 ($\equiv$ R) with a strength factor equal to 2.4 and a ductility factor equal to 1.5. The ATC response modification factor for a CBDS of 6 exceeds the experimentally measured value of 3.6. As the model CBDS was detailed more conservatively and constructed more stringently than a typical building, the maximum achievable reduction factors for full-scale CBDSs are likely to be significantly less than three, assuming that current analysis and design procedures are used.

Concentric Brace Proportioning The local buckling and rupture of the braces in the CBDS clearly indicated that it is necessary to limit the P_{cr}/P_y and B/t ratios to values significantly less than those maximum values adopted at present (Ref. 10). The ATC requirement that concentric braces have a compressive strength equal to at least 50% of their required tensile strength was insufficient to prevent significant brace strength deterioration under repeated yielding reversals. On the basis of the data provided by these tests; the authors suggest that the following limits apply to the proportioning of tubular concentric braces in regions of seismic risk :

$$\frac{B}{t} \leq 18 \quad \text{and} \quad \frac{P_{cr}}{P_y} \geq 0.8 \ \text{or} \ \frac{kl}{r} \leq 0.63\ C_c$$

CONCLUSIONS

(1) The CBDS can provide sufficient elastic stiffness to avoid structural and non-structural damage during minor earthquake shaking. However, the strength and stiffness of a CBDS are

prone to degrade during severe earthquake shaking; this is a direct result of brace buckling and rupture under repeated yielding reversals. Therefore, the ability of a CBDS to respond successfully to long duration, severe earthquake ground motions, such as those measured during the 1985 Chilean earthquake, is extremely questionable.

(2) The measured response modification factor for the model CBDS was 3.6, that is, 60% of that currently adopted by the ATC. There would appear to be an urgent need to reassess the applicability of the empirical ATC 'R' values.

(3) For tubular braces in concentrically braced frames, their compressive strength should be equal to at least 80% of their required tensile strength and their width-to-thickness ratio should be limited to 18.

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REFERENCES

1. U.S.-Japan Planning Group, Cooperative Research Program Utilizing Large-Scale Testing Facilities, "Recommendations for a US-Japan Cooperative Research Program Utilizing Large Scale Facilities," Report No. UCB/EERC-79/26, Earthquake Engineering Research Center, University of California, Berkeley, CA, 1979.

2. Yamanouchi, H. and Midorikawa, M., "Design of the Full-Scale Six-Story Steel Building," The Third Joint Technical Coordinating Committee, U.S.-Japan Cooperative Research Program Utilizing Large Scale Test Facilities, Tsukuba, Japan, July, 1982.

3. Foutch, D.A., Yamanouchi, H., Midorikawa, M., and Nishiyama, I., "Construction of Full-Scale Six-Story Steel Test Structure, "The Fourth Joint Technical Coordinating Committee, U.S.-Japan Cooperative Research Program Utilizing Large Scale Test Facilities, June, 1983, Tsukuba, Japan.

4. Uang C.M. and Bertero V.V., "Earthquake Simulation Tests and Associated Studies of a 0.3-Scale Model of a Six-Story Concentric Braced Steel Structure," Report No. UCB/EERC-86/10, Earthquake Engineering Research Center, University of California, Berkeley, CA, December, 1986.

5. Uang, C.M. and Bertero, V.V., "Earthquake Simulator Testing of a Concentrically Chevron Braced Steel Dual System," Paper submitted for publication in the ASCE Journal of Structural Engineering

6. Whittaker, A.S., Uang, C.M. and Bertero, V.V., "Earthquake Simulation Tests and Associated Studies of a 0.3-Scale Model of a Six-Story Eccentrically Braced Steel Structure," Report No. UCB/EERC-87/02, Earthquake Engineering Research Center, University of California, Berkeley, June 1987.

7. Whittaker, A.S., Uang, C.M. and Bertero, V.V., "Experimental Behavior of a Dual Steel System," Paper to be published in the ASCE Journal of Structural Engineering

8. International Conference of Building Officials., "Uniform Building Code" 1985 Edition, Whittier, California.

9. Applied Technology Council, "Tentative Provisions for the Development of Seismic Regulations for Buildings," ATC 3-06, National Bureau of Standards, U.S. Department of Commerce, Washington, D.C., 1984.

10. American Institute of Steel Construction, "Specification for the Design, Fabrication and Erection of Structural Steel for Buildings," New York, N.Y., 1980.

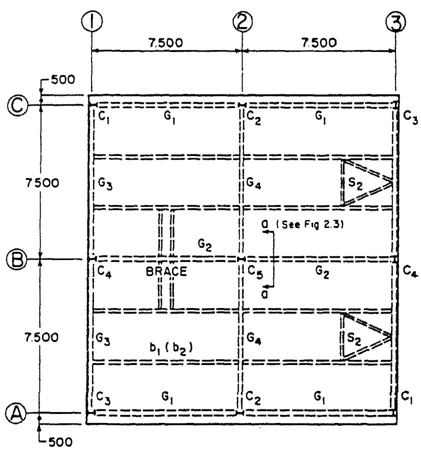
Load Type	Dead Load		Live Load	
	Floor (psf)	Roof (psf)	Floor (psf)	Roof (psf)
Metal Deck	6	6		
3.5" Lightweight Concrete	39	39		
Ceiling & Floor Finishes	10			
Ceiling & Roofing		20		
Partitions	20			
Structural Steel & Fireproofing	15	10		
Total	90	75	60	20

Exterior Wall Weight = 30 psf

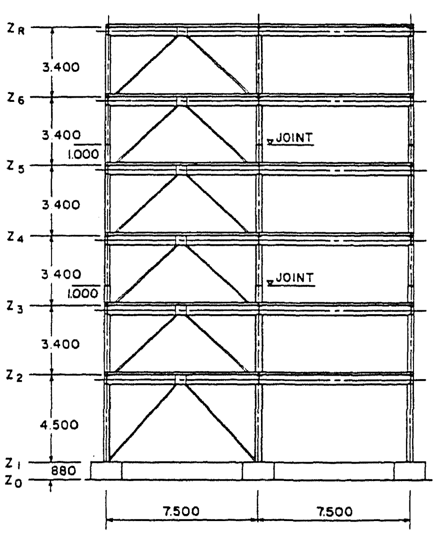
Table 1. Gravity Design Loads

		Free Vibration			Forced Vibration		
	Mode	1st	2nd	3rd	1st	2nd	3rd
Full-scale CBDS	T_i (sec)	0.60	-	-	0.61	0.22	0.13
	ξ_i (%)	0.4	-	-	0.5	0.5	0.5
Model CBDS	T_i (sec)	0.62	0.22	0.12	0.62	0.22	0.12
	ξ_i (%)	1.3	0.7	0.5	1.6	0.7	0.6

Table 2. Dynamic Characteristics of the CBDSs



Plan



Braced Frame Elevation

Fig. 1 Full-Scale CBDS Plan and Elevation (mm)

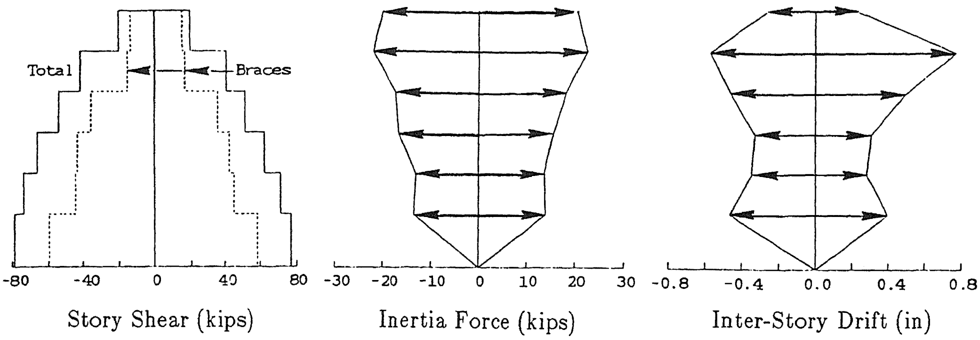


Fig. 2 MKO-65 Response Profiles

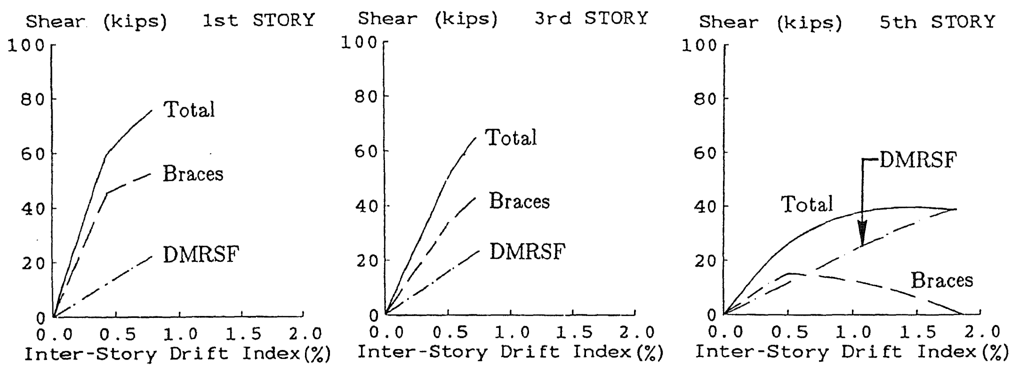


Fig. 3 CBDS Strength - Deformation Envelopes