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EARTHQUAKE RESISTANT DESIGN OF DUCTILE BRACED STRUCTURES

Subhash C. GOEL[1] and Xiaodong TANG[2]

[1]Professor, Department of Civil Engineering, The University of Michigan, Ann Arbor, Michigan, U.S.A.
[2]Formerly Doctoral Student, Department of Civil Engineering, The University of Michigan, Ann Arbor, Michigan, U.S.A.

SUMMARY

Problems of early brace failures on seismic response of braced structures having ductile moment resisting frames designed by current code procedures are presented in this paper. It is shown that seismic design of concentrically braced structures should be based on providing adequate ductility of bracing members rather than specifying increased design forces.

INTRODUCTION

In current design practice concentrically braced structures are not considerded as ductile structures. The non-ductile behavior of these structures mainly results from early cracking and fracture of bracing members due to local buckling in regions of plastic hinges which form during cyclic post-buckling deformations. Building codes recognize this fact and attempt to take care of this problem by specifying increased design forces for braced frames in general. This practice may not necessarily result in safe structures, however, since the fracture life of less slender bracing members may be shorter than that of more slender members with compact sections. This paper deals with studying the seismic behavior of braced structures designed by current code procedures versus a suggested approach where the bracing members are designed for improved ductility but smaller design forces.

The structures selected for this study are similar to the six-story, full-scale test structure of the U.S.-Japan Cooperative Earthquake Research Program. In Phase 1 this structure was designed as a dual system with inverted V-bracing in one bay of the middle frame. The structure was designed jointly to represent the design practice in the two countries (Ref. 1). W sections of A36 steel were used for beams and columns, and square tubes of A500 grade B were used for the bracing members which were directly welded to the flanges of the girders. During the inelastic test NS component of the 1978 Miyagi-Ken-Oki earthquake with peak acceleration scaled to 500 gal, approximately 0.5g, was used to represent a severe earthquake motion. Severe overall and local buckling of the tubular bracing members was observed in several stories very early in the response (Ref. 2). Complete fracture at mid length in one brace in the third story and 50% - 70% fracture in both braces in the second story had occurred shortly after 11 seconds of the planned 20 second test. The test was stopped at this point as the story drifts began to increase, even though considerable strength and ductility still remained in the moment frames.

Structure F5 Structure F5 is designed according to the UBC 1982 by using the allowable stress design procedure of the current AISC Specification (Refs. 3,4). The floor plan, dimensions and gravity loads are kept the same as those of the U.S.-Japan structure. The girders and columns are connected with full moment connections. The minimum lateral design force is computed as V = 0.113W, where W = 1356 kips (total dead weight of the structure). The bracing system is designed for 1.25 times this force and the moment frames without the bracing for 50 percent of it. For more details reference should be made to the original report by the authors (Ref. 5). The resulting member sizes of the braced and unbraced frames are shown in Fig. 1.

The response of the structure is computed for the same ground motion as used in the inelastic test of the U.S.-Japan structure. The results are presented in Fig. 2. Figure 2 shows that extensive yielding in beams and columns as well as buckling in the bracing members occurred. It is also noticed that both bracing members in second story and one in the third story fractured which resulted in very large story drifts in those stories. The maximum drift in the second story is close to 30 cm. Inspite of the back-up moment frames capable of resisting 50% of the design lateral force early failure of bracing members caused large story drifts. It is questionable whether the columns could remain stable under such large story drifts and associated plastic rotations.

The fracture criterion for the rectangular tube bracing members used in the analysis is rather simple and crude which is based on recent tests at The University of Michigan (Ref. 6). The detailed derivation of the criterion is given in the original report by Tang and Goel (Ref. 5). It is empirical in nature and can be expressed as follows:

$$N_f = C\,(KL/r)(b/d)/[(b-2t)/t]^2 \qquad KL/r > 60$$

where,
N_f = number of equivalent cycles to failure
C = 262, an empirical constant
KL/r = effective slenderness ratio of the member
b/d = width-to-depth ratio of the section, and
$(b-2t)/t$ = clear width-thickness ratio of the flanges.

It should be pointed out that the width-thickness ratio of tubulular bracing members used in Structure F5 meet the limit specified by the AISC Plastic Design criteria (Ref. 4).

Structure F6 The Structure F6 is designed according to SEAOC Draft 1985 (Ref. 7). The minimum lateral design force is computed as V = 0.118W, which is quite close to that given by UBC for Structure F5. However, the bracing system had to be designed for twice this force instead of 1.25 times in case of UBC. The resulting member sizes are shown in Fig. 3, where considerably larger sizes for the braced frame members can be noted.

The response of Structure F6 to the same groung motion as used for F5 is shown in Fig. 4. Figure 4 shows quite severe inelastic deformation experienced by the braced frame including fracture of braces in the lower two stories. This resulted in maximum drift in the first story close to 60 cm and the roof displacement of about 65 cm. The response of this structure is even worse than that of the UBC designed structure F5 where smaller design forces were used for the bracing system indicating that increasing the design forces for the bracing systems may not be the way to go.

Structure F7 Early fracture of bracing members in Structures F5 and F6 is the most undesirable aspect of their computed response. Early fractures of bracing members in some stories led to very large story drifts inspite of stiffness and strength of the moment frames. It can be noticed from the proposed fracture

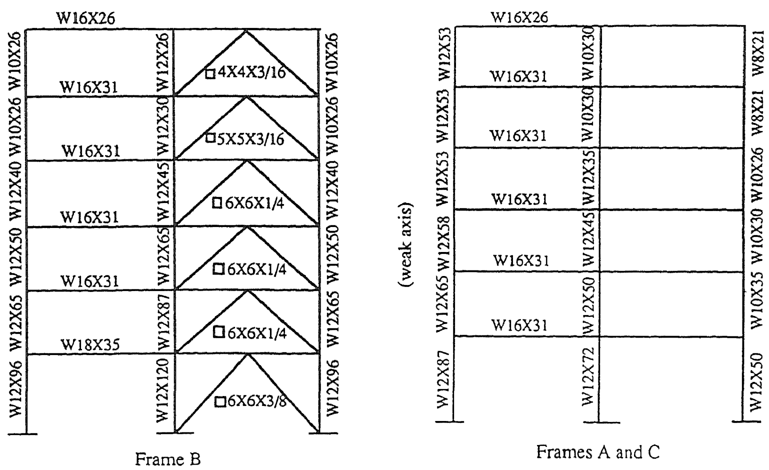

Figure 1 Member Sizes of Structure F5

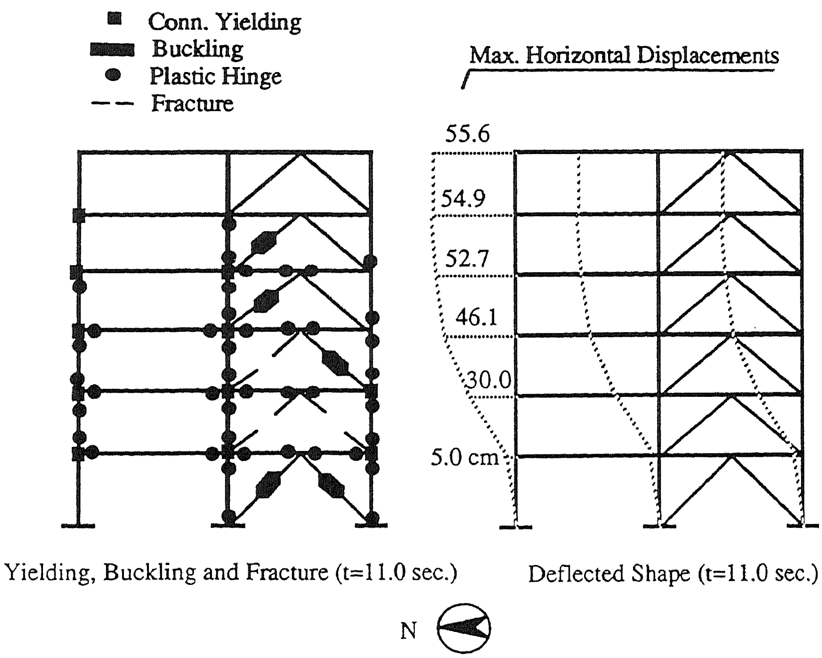

Figure 2 Yielding, Buckling, Fracture and Deflected Shape of Structure F5 - Miyagi-Ken-Oki Earthquake

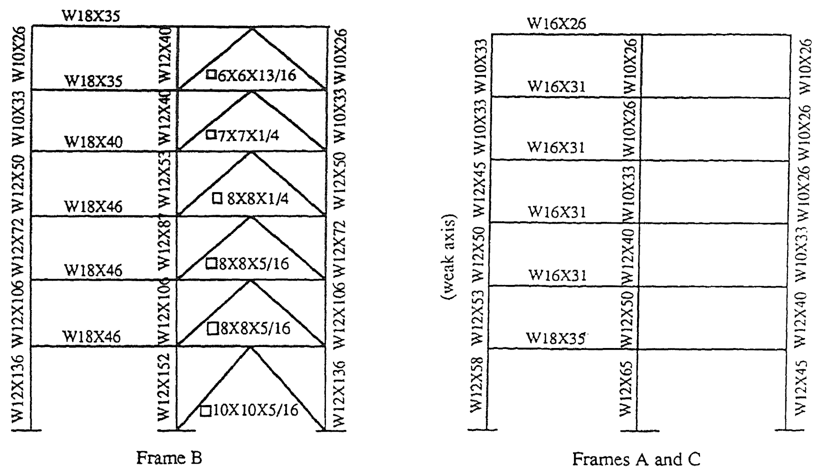

Figure 3 Member Sizes of Structure F6

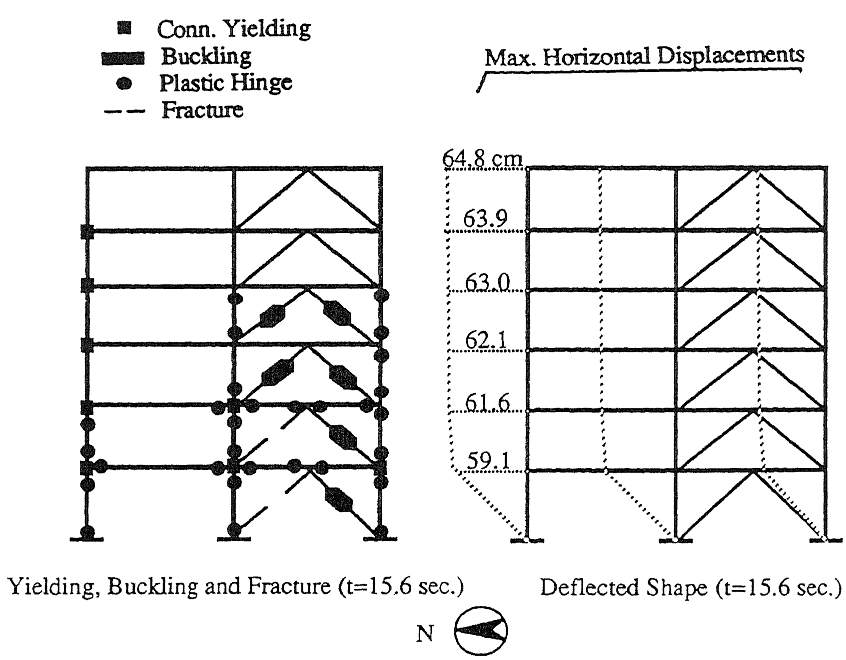

Figure 4 Yielding, Buckling, Fracture and Deflected Shape of Structure F6 - Miyagi-Ken-Oki Earthquake

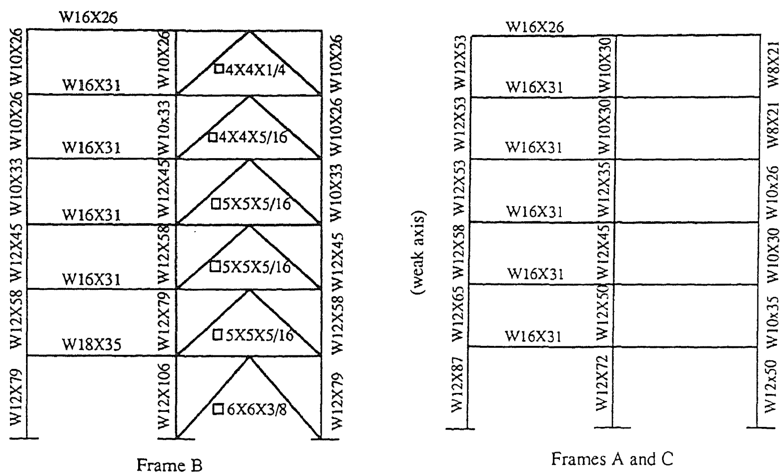

Figure 5 Member Sizes of Structure F7

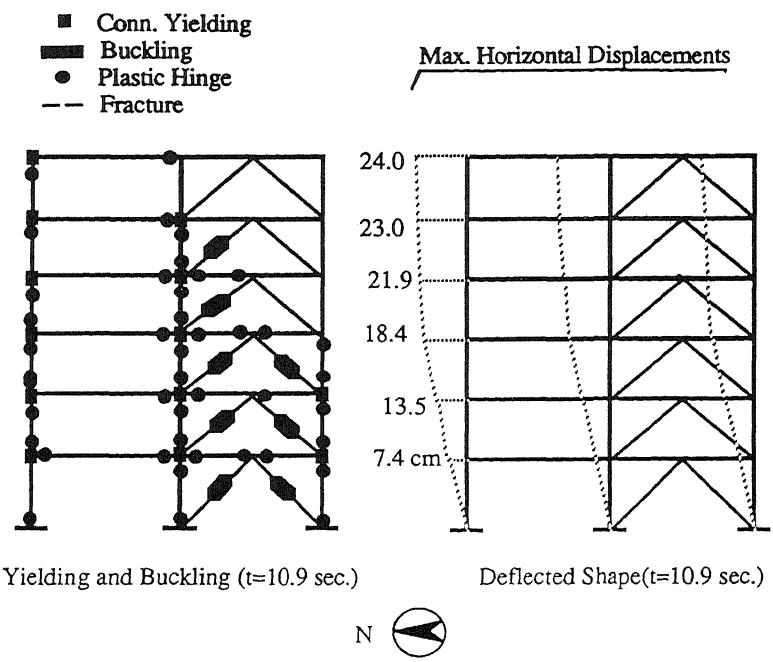

Figure 6 Yielding, Buckling and Deflected Shape of Structure F7
- Miyagi-Ken-Oki Earthquake

criterion that the fracture life of the tubular bracing members could be much improved by lowering the allowable width-thickness ratio. Thus, the width-thickness ratio of bracing members in Structure F7 was limited to $95/F_y$, which is half of that allowed in plastic design according to the current AISC Specification. By doing so the fracture life of the bracing members could be expected to be approximately four times of those used in the previous two structures. In view of this the UBC lateral force factor of 1.25 for the bracing system was deleted and the structure was designed as per the normal UBC method with more compact sections for the bracing members. The resulting member sizes are shown in Fig. 5. The unbraced frames A and C are the same as in structure F5 but the braced frame B is significantly lighter.

The response of Structure F7 was computed for the same ground motion and the results are shown in Fig. 6. It can be seen that significant yielding and buckling occurred but no early brace fractures were encounterd. The horizontal floor displacements are significantly smaller than those in Structure F5 or F6. Overall, the response of this structure can be considered to be much more acceptable than the response of Structure F5 or F6.

CONCLUSION

Early brace failures during severe earthquakes can be expected to occur in concentrically braced structures designed by current code procedures. Large story drifts caused by early brace failures can place undesirably large ductility demands on the beams and columns of the back-up moment frames. Therefore, earthquake resistant design of concentrically braced structures should emphasize ductility of bracing members rather than increasing the design forces in order to compensate for lack of it.

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