NOTES ON THE SEISMIC DESIGN OF STEEL CONCENTRICALLY BRACED FRAMES

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SUMMARY

The background to provisions for the earthquake resistant design of Concentrically Braced Steel Frames (CBF) currently under review for inclusion in the Recommended Lateral Force Requirement of the Structural Engineers Association of California (SEAOC) is presented. The results of recent research on the inelastic cyclic behavior of steel struts and of complete CBF and KBF assemblies is reviewed insofar as these guided the development of the CBF provisions and prompted the questions raised regarding the behavior of KBF's. A discussion of some features of the inelastic cyclic behavior of K-Braced Steel Frames (KBF) which distinguish it from that of CBF's is also included.

INTRODUCTION

The hierarchy of structural systems as delineated in the table of R factors of the ATC-3-06 (Ref. 1) and of K factors in the 1982 UBC. (Ref.2) place CBF's on a par with or slightly below reinforced concrete shear walls. Ductile Moment Resisting Frames (DMRF) are favored because of their more reliable ductility and energy dissipating capacity. CBF's especially those with very slender braces (e.g. tie-rods) are faulted for the relatively poor inelastic cyclic response that results from the cyclic buckling and tensile yielding of the braces under severe seismic excitation. Nevertheless CBF's have the advantage over DMRF's of limiting story drifts at moderate levels of excitation and thereby reducing the potential for often costly "non-structural" damage.

In an effort to combine the better parts of DMRF and CBF behavior recent efforts have led to the development and increasing use of Eccentrically Braced Frames (EBF). Combining as they do the stiffness of braced frames with the stable hysteretic response of flexural or shear yielding beam links, these structure should in many cases supplant the use of CBF's. Nonetheless until CBF's are proscribed in favor of EBF's, code provisions incorporating the results of recent research on the response of individual struts as well as complete CBF assemblies subjected to cyclic axial loading are in order.

In studying the inelastic behavior of CBF's it is important to establish which parts of the system are to undergo inelastic excursions and which are to remain essentially elastic. The components of a typical CBF are: the diagonal brace; the beams and columns included within the braced bent; the collector beam(s) called on to drag loads to the braced bent; and the brace-beam and/or brace-column, beam-column and column-base connections (see Fig. 1).

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In a strictly Concentrically Braced Frame, that is a CBF wherein at both ends of a brace the center lines of the brace, beam and column essentially intersect, a mechanism will in most cases first form as the tension brace yields and the compression brace buckles. If the cyclic tensile yielding and buckling of the brace is to be selected as the principal locus of inelastic action in CBF's then the remaining elements of the system should be designed to develop that mode.

This paper will first briefly review currently available information on the elastic cyclic behavior of stress struts and list those parameters that most significantly effect that behavior. A summary and discussion of the CBF code provisions follows and in closing some hypotheses are offered regarding the inelastic cyclic behavior of KBF's.

INELASTIC CYCLIC BEHAVIOR OF STEEL STRUTS

It has been recognized for some time that the inelastic cyclic behavior of steel struts, as represented schematically in Fig. 2, is largely determined by the buckling behavior of the strut. Two important characteristics may be noted.

1. In each cycle of response there is a range of deformation characterized by low stiffness where the strut, under tensile loading, straightens out from its buckled condition.
2. With repeated cycles of loading the buckling capacity of the brace, P_{yc}, is considerably diminished either due to prior tensile yielding or residual curvature imparted by prior buckling.

As a result the typical hysteresis curve for cyclically loaded struts has what has been termed a "pinched" look when compared to those of cantilever stubs or DMRF's. Since the capacity of a structure to dissipate seismic energy in the inelastic range is indicated by the area enclosed by these curves it is apparent that a structure dependent on the inelastic action of a brace to supply that capacity is at a disadvantage when compared to DMRF's. In cases where very slender bracing (Kl/r greater than 200) is used, this problem is particularly severe since with the ever increasing inelastic extension of the brace and little resistance from the compression brace the frame will sway freely in either direction until the tensile brace straightens and "catches" thereby imparting what is essentially an impact load on the structure.

On the basis of the results of recent research (Ref. 3,4,5 and 6 among others) it is evident that the inelastic behavior of steel struts is determined by the following parameters.

1. Slenderness or Kl/r Ratio As may be seen in Fig. 3 the hysteresis curves for cyclically loaded struts "fill out" with increasing Kl/r.

   Furthermore where it is possible to provide restraints that reduce the K the resulting behavior is essentially identical to a strut with an equivalent l/r (see Fig. 4). Where members of different sectional geometry but equal Kl/r's are used the hysteresis curves are also essentially identical. Finally, as can be seen in Fig. 5 the reduction in buckling capacity of a strut due to prior buckling increases with increasing Kl/r, though for a Kl/r greater than 90 the reduction in buckling capacity remains essentially constant (with P_{yc} after 2 cycles equal to about half the initial buckling load).

2. Sectional Geometry of Strut. If the overall buckling behavior of the struts is to be largely determined by the Kl/r ratio, lateral-torsional buckling and local buckling of outstanding elements should be prevented through proper selection of the section (e.g. complying with Section 1.9 of the AISC Specifications). Where built-up members are used the stitching should effectively restrain the parts and limit their individual Kl/r between stitches to less than the overall member's.

3. Baushinger and Residual Curvature Effects. As noted above the buckling capacity may be considerably reduced subsequent to the initial yielding or buckling of the strut. In the case where a strut is initially yielded in tension this is due to the reduction in the elastic modulus due to the Baushinger effect. A similar problem exists for members (e.g. pipes and tubes) composed of materials with less distinct proportional limits. In the case where a previously buckled strut is re-compressed the reduction in buckling capacity is mostly due to a residual curvature in the member.

4. Boundary Conditions. As noted in Ref. 7 it is important that the end connections of braces be detailed to accomodate the assumed boundary conditions. Where braces are assumed pinned-pinned sufficient flexibility in the gusset should be allowed to prevent fracture. Where a fixed condition is assumed the detailing should allow for the formation of a plastic hinge. Where a plastic hinge is to form within the span of a built-up member the stitching within that zone must be particularly secure. Finally in cases where X-bracing is used and the braces are attached at the center to provide mutual lateral restraint such connections should allow for the hinging that may occur.

DESIGN OF CONCENTRICALLY BRACED FRAMES

In recognition of the relatively poor ductility and energy dissipaing capacity of CBF's recent earthquake resistant design codes have provided as follows:

1. The 1982 UBC (Ref. 2), for a dual system prescribes a K factor of 0.80. With a typical 10 story structure the base shear factor works out to 0.104 (CS=0.13, I=1.0, Z=1.0). The code further requires that members in a braced frame be design for 1.25 times the design shears, giving a base shear factor of 0.13. The braces will buckle at about 1.7/1.33 times this value, or a base shear of 0.166W.

2. The ATC-3-06 (Ref. 1) for a similar case requires an R factor of 6. With the same structure and conditions the ATC base shear factor C_s works out to 0.169. Combined with the required capacity reduction factor this amounts to a base shear of 0.169/0.9 = 0.188 W at the point of incipient brace buckling. The ATC code further requires that for Seismic Performance Categories C and D the brace be selected such that F_a/F_t= 0.50. With A36 steel this corresponds to a Kl/r of 115.

3. The recently revised Japanese code (Ref. 8) provides for the full range of relative stiffnesses of brace and DMRF in the assignment of story shear factors. The matrix from which is drawn the Structural

Characteristic Factor D_s varies with member ductility, relative share of story shear between brace and frame and brace slenderness. Braced frames with braces having Kl/r less than 57.1 (for A36 steel) are accorded the same D_s factor as DMRF's or dual systems where the DMRF resists over 70% of the story shear.

The CBF provisions prepared for considerations by the SEAOC Seismology Committee are directly applicable only to strict CBF's as defined above. Provisions for KBF's, as yet incomplete, may differ somewhat to allow for the problems outlined in the next section of this paper. The format and content resembles the UBC and ATC provisions. No effort was made to alter the approach along the lines of the Japanese code, though these are in many respects better, since that would require a more extensive overhaul of the rest of the code.

The specific requirements as enumerated below, are best understood by referring to Fig. 1 and 6 which show two typical CBF configurations. The principal elements of the requirements are as follows.

1. Diagonal braces which act as primary lateral load resisting elements must have a Kl/r less than $540/\sqrt{F_y}$ (for A36 steel Kl/r is then 90 or less). The value of K may be taken as 1.0 or equal to 1.3 times the theorectical K. This last factor is similar to that used in the AISC Specifications Table C1.8.1. An exception is granted where the strength of the brace exceeds 2-3.75 times the design story shear (depending on the structural system, and R factor, used). In such cases the Kl/r is limited to $810/\sqrt{F_y}$ (135 for A36). The precise numbers are of course subject to review. The intent is to enhance the energy dissipating capacity of the system while not adversely affecting the sizing of the other members which, as described below, must develop the tensile capacity of the bracing member.

2. Braces are to be provided in tension/compression pairs in any framing line and at each story (see Fig. 1a). This will insure symmetrical response, by opposing the assymetrical halves of the brace's own response.

3. The remaining members of the braced frame system must either develop the story shear capacity (usually determined by the tensile capacity of the brace) or 2-3.75 (again related to the R factor) times the design story shear. This will not only impact the column design, which must resist the overturning, but also both the braced bent and collector beams. In cases such as that shown in Fig. 1b the beam must act as a strut to complete the truss once the compression brace is buckled. Similarly in Fig. 1a the collector beam must drag the full, not just half of the tributary story shear of the braced frame line. This demand on the beams could be reduced perhaps by recognizing that for Kl/r's less than 80-90 the compression capacity of the brace, after several cycles, stabilizes at about 30-50% of the initial buckling load (see Fig. 5) so that 50-70% of the tensile yield load applied will be balanced and the beams can be designed accordingly.

4. Provisions are also included to limit the Kl/r or parts of built-up members between stitches to 3/4 of that of the whole members, (referencing AISC Section 1.9 for stiffened and unstiffened compression

elements) to require that connections develop the forces calculated by the provisions of paragraph 3 above and to set a limit on the ratio of effective net to gross area at bolted connections. Absent from the provisions are guidelines for the design of brace connections to allow for rotation, elastic or inelastic, that will occur, as discussed above. At present the knowledge available is insufficient to develop code requirements.

The effect of these provisions will, it is hoped, be to rationalize the design so that each requirement follows from and indeed obliges the designer to think in terms of the assumed collapse mechanism. Though not envisioned at this time it may be possible to reduce somewhat the seismic design shears for which CBF's are designed (i.e. increase the R factors) in recognition of the improved ductility and energy dissipating capacity. To do so though it may be necessary to apply stricter limits on Kl/r, reducing the limiting value to about 60-80. Such a direction would be in keeping with a philosophy that regards improved post-elastic behavior as more beneficial than increased elastic strength.

K-BRACED FRAMES

The behavior of KBF's, (both in the K or "chevron" and inverted K or "V" configurations) as different from CBF's, has as yet received little attention. Nevertheless if one takes into consideration the reduction in the buckling capacity of a strut after several cycles (see Fig. 5) and the probable collapse mechanism of KBF's (see Fig. 7) it is evident that:

1. The tensile capacity of the brace cannot be developed since for a given load direction, once the compression brace has buckled and unloads, the beam, unless it is unusually strong, will form one or several plastic hinges (depending on its end restraint conditions) and so allow a collapse mechanism to form without reaching the tensile brace's capacity. The story shear capacity of the system is therefore determined by the buckling capacity of the brace and therefore the remaining members of the system need only develop that capacity.

2. Since with repeated cyclic loading the buckling capacity of the brace is reduced to 30-50% of the initial capacity, the braced frame after one or more cycles into the inelastic range will retain roughly 30-50% of its original strength. This reduction may of course be lessened if the frame containing the braces has some lateral capacity afforded by the beam-column connections.

It is unclear, given the present paucity of research information, whether the earthquake resistant capacity of KBF's is necessarily diminished by these factors when compared to CBF's, since it is difficult to gage the energy dissipating capacity of the system and since under actual earthquake excitation the ductility demand may not be as severe as is imposed in cyclic loading tests of individual struts. Nonetheless it is apparent that as the system stabilizes at the lower buckling capacity, the area enclosed by the presumed hysteresis loops (see Fig. 7) is roughly 30-50% of what would be enclosed by the hysteresis loops of a DMRF of equivalent strength. A CBF of equivalent strength with braces having a Kl/r around 80 should exhibit hysteresis loops that enclose roughly 3/4 the area enclosed by the loops of a DMRF of equivalent strength.

In order to possibly improve the performance of KBF's several approaches are possible:

1. Stricter limits on the Kl/r ratio would result in a lesser reduction of the buckling capacity. Struts with a Kl/r less than 40 will retain 70-80% of their original buckling capacity. (see Fig.5)

2. KBF might be considered as a special case of Eccentrically Braced Frames (EBF) and the beam detailed as a flexural link. If braces with Kl/r's around 80 are used, the corresponding EBF would need only be designed for 50-70% of the design story shear. Some drift limitations should most likely be included when considering the EBF phase of the system.

Clearly it is as of yet not evident whether the problems presented above are necessarily severe. Before any conclusions can be drawn, and before appropriate code provisions can be developed, further study is undoubtably called for.

CONCLUSION

The background to code provisions for the design of Concentrically Braced Frames as well as the essential details of those provisions have been presented. The proposed requirements, intended for inclusion in the Recommended Lateral Force Requirements of the Structural Engineers Association of California are currently under review. Some notes regarding cyclic inelastic behavior of K-braced frames have also been offered and it is argued that the behavior of KBF's is sufficiently different from that of CBF's to possibly warrant an independent approach to their design.

ACKNOWLEDGEMENTS

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1. ATC-3 (Applied Technology Council) Tentative Provisions for the Development of Seismic Regulations for Buildings National Bureau of Standards, U. S. Dept. of Commerce, Washington, D. C., June, 1978.

2. Uniform Building Code International Conference of Building Officials, Whittier, Calif., 1982.

3. Jain, A. K. and Goel, S. C., "Cyclic End Moments and Buckling in Steel Members", Proceedings, 2nd U. S. National Conference on Earthquake Engineering" Stanford University, CA, August 1979.

4. Black, G. R., Wenger, W. A., and Popov, E. P., "Inelastic Buckling of Steel Struts Under Cyclic Load Reversals", EERC Report 80-40, University of California, Berkeley, October 1980.

5. Popov, E. P., "Inelastic Behavior of Steel Braces Under Cyclic Loading," Proceedings, 2nd U. S. National Conference on Earthquake Engineering, Stanford University, CA., August, 1979.

6. Wakabayashi, M., Nonaka, T., Nakamura, T., Morino S., and Yoshida, N., "Experimental Studies on the Behavior of Steel Bars Under Repeated Axial Loading," Disaster Prevention Research Institute Annals, Kyoto University, No. 16B, 1973, pp. 113-125.

7. Astaneh-Asl, A., Goel, S. C., and Hanson, R. D., "Behavior of Steel Diagonal Bracing," Preprint 81-522, ASCE Convention, St. Louis, Missouri, October 26-31, 1981.

8. Aoyama, Hiroyaki, "Outline of Earthquake Provisions in the Recently Revised Japanese Building Code," Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 14, No. 2, June 1981.

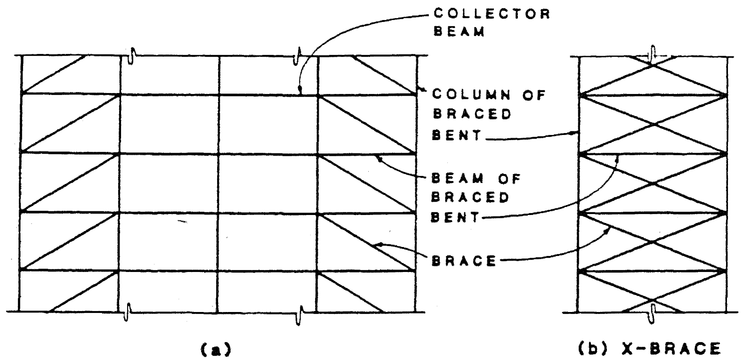

FIG.1 TYPICAL CBF'S

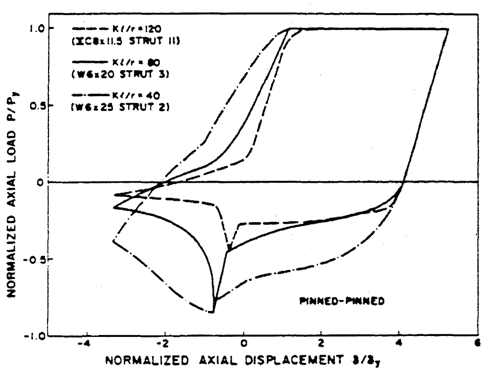

FIG. 31 HYSTERETIC ENVELOPES FOR STRUTS WITH DIFFERENT SLENDERNESS RATIOS

Fig. 3 (Ref. 4)

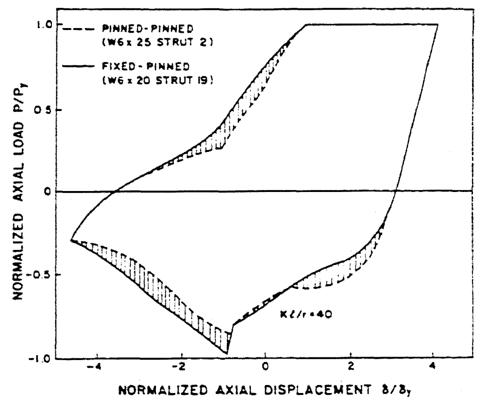

FIG. 37 COMPARISON OF HYSTERETIC ENVELOPES FOR STRUTS WITH DIFFERENT BOUNDARY CONDITIONS

Fig. 4 (Ref. 4)

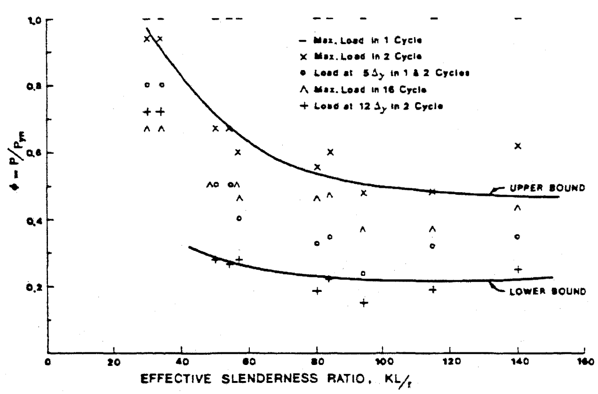

Figure 3 - Reduction in Compressive Loads

Fig. 5 (Ref. 3)

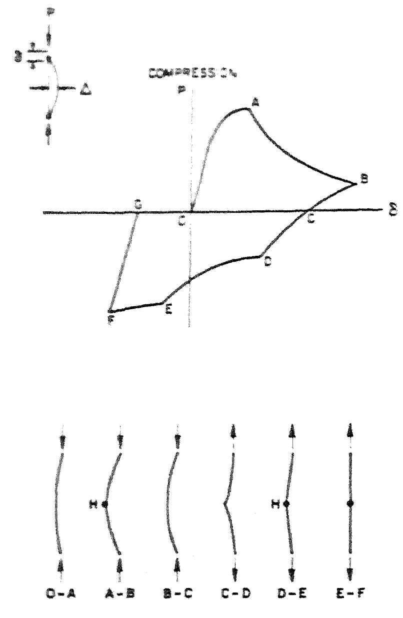

FIGURE 1 - TYPICAL LOAD DEFORMATION RELATIONSHIP OF A SLENDER BAR

Fig. 2a (Ref. 4)

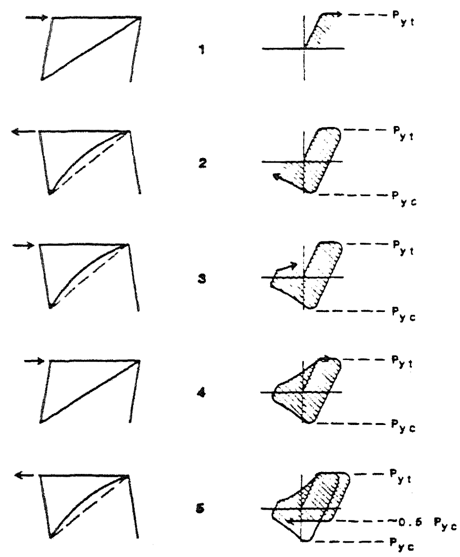

FIG.2b DIAGONAL BRACE SCHEMATIC HYSTERESIS

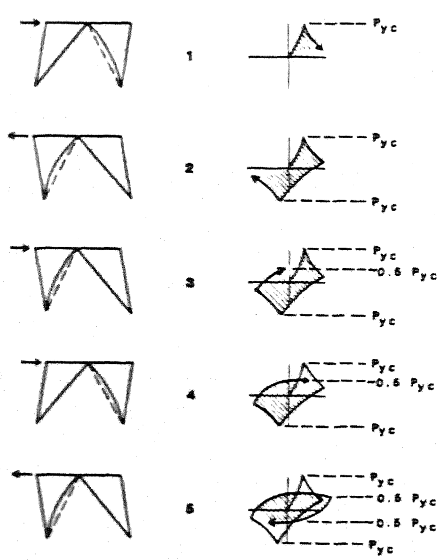

FIG.7 K-BRACED FRAME SCHEMATIC HYSTERESIS

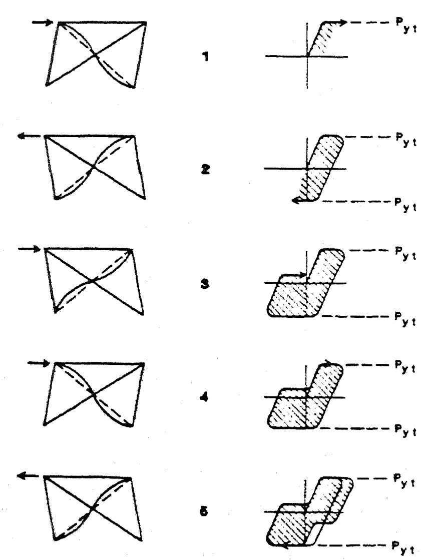

FIG.6 X-BRACED FRAME (STRICT CBF) SCHEMATIC HYSTERESIS