

# ON CALIFORNIA STRUCTURAL STEEL SEISMIC DESIGN

Egor P. Popov<sup>(1)</sup>

Presented at the Pacific Structural Steel Conference, Auckland, August 1986

## SYNOPSIS

A number of new code developments, largely initiated in California, are taking place in the USA for the seismic design of steel structures. The principal ones are reviewed and commented upon in the paper. Key experimental support for some of the changes is indicated. Major attention is directed to the three main types of steel construction: moment resisting frames, concentrically braced steel frames, and, the relatively new method for seismic design, eccentric bracing. Some of the proposed and possible practical improvements in moment-resisting connections are given: the reasons for some concern over the use of concentrically braced frames for severe seismic applications are discussed; and a brief overview on the application of eccentrically braced steel frames is presented. The paper concludes with a few remarks on future trends and needs in structural steel seismic design.

## INTRODUCTION

In recent years a great many structural steel buildings have been built in California, especially in metropolitan areas. At the same time important new developments in design, nurtured by research plus intensive code updating of seismic provisions spearheaded by California structural engineers, has taken place. As part of a major U.S.-Japan cooperative steel structure research project a full-size building model was tested pseudo-dynamically at Tsukuba, Japan (Yamanouchi et al. 1984, B.R.I. Steel Group 1985). Under the same U.S.-Japan cooperative research program corroborative subassemblage experiments together with 0.3-scale shaking table replica tests of the Tsukuba building at Berkeley (Bertero 1986) provided much valuable information. The behaviour of concentrically braced frames (CBFs) and eccentrically braced frames (EBFs) was extensively studied in this program. In the Tsukuba full-size test program important investigations were also conducted on the behaviour of non-structural elements consisting of exterior panel cladding and interior partitions (Ito and Wang 1985). Simultaneously industry motivated research on moment-resisting connections was carried out at Berkeley (Popov et al. 1985) and the behaviour of concentric braces was investigated at the University of Michigan (Astaneh-Asl et al. 1982, Gugerli and Goel 1982).

Independently of the above efforts, the Seismology Committee of the Structural Engineers Association of California (SEAOC) undertook a complete revision of the lateral force requirements for building design in regions of high seismicity. This work, adopting many of the features of the Tentative Provisions for the Development of Seismic Regulations for Buildings (ATC 1978), is now being circulated among the membership in the form of Tentative Lateral

Force Requirements (SEAOC 1985). With some revisions this work will be published by SEAOC in final form by the end of 1986 and, as in the past, with a few minor modifications should be accepted in October 1986 for inclusion in the 1988 Uniform Building Code (UBC).

In December 1985 the Building Seismic Safety Council (BSSC) issued a tentative version of an updated ATC document as an NEHRP (National Earthquake Hazard Reduction Program) Recommended Provisions for the Development of Seismic Regulations for New Buildings. With minor revisions this work will shortly be generally available from FEMA (Federal Emergency Management Agency) through their publication series.

Again independently from the above activity, the American Institute of Steel Construction (AISC) has for several years been intensively working on developing Load and Resistance Factor Design (LRFD) Specification for Structural Steel Buildings including seismic provisions. This LRFD Specification (AISC 1985) is based upon the philosophy of limit states of strength and serviceability combined with first-order probability analyses. This approach is the prototype for a new generation of structural design codes in steel.

Strong effort is being made to coordinate the proposed seismic requirements to be issued shortly by the four groups: AISC, BSSC, SEAOC, and UBC. The design requirements in some of these documents are based on allowable stresses, whereas others recognize inelastic behaviour directly. Therefore great care is necessary to correlate the provisions. One can foresee that before long these design specifications will be written on a common basis.

In order to set the stage for the new developments in seismic design of MRFs (moment-resisting frames), CBFs, and EBFs, some relevant observations are given on the evolution of structural steel fabrication

(1) Department of Civil Engineering, University of California, Berkeley, CA 94720, U.S.A.

in the next section. This is followed by a general discussion of some aspects of the seismic code development and provisions. Then, after discussing MRFs, CBFs, and EBFs, the author's opinions on future trends and needs in structural steel seismic design are expressed.

#### EVOLUTION OF STRUCTURAL STEEL FABRICATION

The 1906 San Francisco earthquake established an outstanding reputation for structural steel. To this day one measures the excellence of seismic behaviour in other materials to that of steel. It is important to note, however, the great difference in the manner in which modern steel buildings are constructed resulting in totally different structures than formerly. In the immediate pre- and post-1906 San Francisco earthquake era and through the early '60s, connections were riveted. Wind bracing connections, which were directly adopted for seismic design of moment-resisting frames, were usually detailed per AISC recommendations such as those shown in Fig. 1 (AISC 1934). In these excellent connections the steel is used in an optimum manner, either in tension along the rolling direction, or shear. A slight connection flexibility caused by the bending of column flanges and attachment details is not critical. A limited amount of flexibility in such connections is probably even beneficial during a possible seismic event.

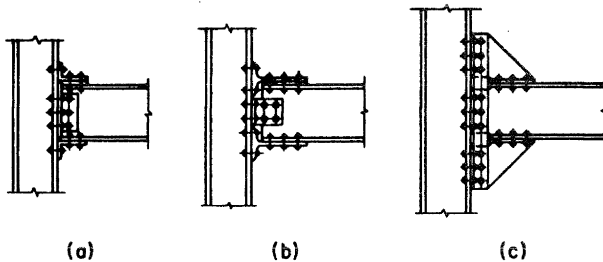


FIG. 1 - TYPICAL WIND BRACING CONNECTIONS CIRCA 1930s (AISC 1934)

From the mid-'40s shop welding became increasingly popular, and by the mid-'50s, when high-strength bolts were introduced, the fabricating industry rapidly adopted the use of welding and high-strength bolting to the exclusion of riveting. For reasons of cost, typical seismic moment connections in general use today on the West Coast heavily rely on field welding resulting in rigid joints whose ductility may be limited. Moreover modern structures are often less redundant and the joined steel elements are much thicker than formerly. As a consequence thick welded connections, which are common, are susceptible to large shrinkage stresses from cooling. The ductility of such assemblies must be carefully ascertained. "In seismic resistant design it is mandatory that the design engineer carefully design every detail." (Wyllie 1985)

It should further be noted that no significant number of modern steel buildings has been subjected to severe earthquakes. One must depend almost entirely on available field observations and experimental research for design and the development of codes.

Some aspects pertaining to code issues are discussed next.

#### ESSENTIALS FOR SEISMIC CODE DEVELOPMENT

It is generally recognized that the basic function of a building code is to provide minimum requirements for safeguarding against major failures and loss of life. Some of the user-sponsored codes, such as California's Administrative code for schools and hospitals, are concerned with minimizing damage as well as protecting occupants. Such damage control is generally not considered in earthquake provisions of the usual building codes, and is not discussed herein. The main objectives in a general seismic code are based on the following philosophy (SEAOC 1976):

1. A building must resist a minor shake without damage;
2. In moderate earthquakes some nonstructural damage is permissible;
3. During a major earthquake, a building must not collapse, but both some structural and nonstructural damage may occur.

This approach appears to have been adopted from the beginning and the seismic code provisions continue to be refined with new knowledge.

In order to accomplish these ends for the usual service conditions, a structure must possess sufficient strength and stiffness. This is achieved by an elastic limit state. It is also possible to design a structure to respond elastically in resisting severe seismic loadings, but is economically not feasible. Therefore an alternative approach is generally adopted. For severe seismic loading a structure must be able to absorb and dissipate earthquake energy input. This requires the structure to resist an appropriate level of cyclic lateral forces and be capable of sustaining large inelastic deformations, i.e. a structure must be ductile. For an economical seismic-resistant design some of the joints and members are expected to behave inelastically in the event of a major earthquake. This provides a mechanism for energy dissipation, effectively damping the motion of a frame.

The great importance of frame ductility in seismic design is brought out in Fig. 2. Here, a very strong earthquake is assumed to have a site acceleration of 0.33g. For this condition the upper curve gives the response spectrum for elastic oscillators with different periods of vibration. In this diagram the ductility ratio  $\mu_\delta$  is defined as the ratio of the system deflection to the deflection at yield. For the upper curve,  $\mu_\delta = 1$ . When, due to inelastic (plastic) deformations, the system deflections increase, the values for  $\mu_\delta$  also increase, and the curves progressively drop down. The base shear coefficient C for which a system must be designed dramatically changes with ductility. For example, for an elastic system with a natural period of 2 sec the value of C corresponds to the upper arrow in the figure, whereas the lower arrow defines the value of C for a system with  $\mu_\delta = 6$ . The economic consequences of this are dramatically apparent.

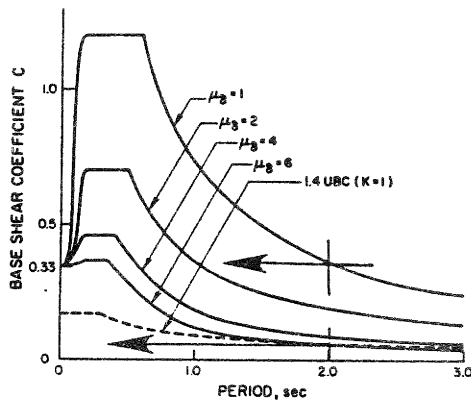


FIG. 2 - BASE SHEAR COEFFICIENT SPECTRA FOR SINGLE-DEGREE-OF-FREEDOM SYSTEMS WITH DIFFERENT DUCTILITIES (AFTER VELETSOS AND NEWMARK)

For comparison a dashed curve for a widely used seismic code (UBC 1985) based on elastic concepts is also shown in Fig. 2. This curve corresponds to the code elastic design requirements multiplied by a factor of 1.4 to approximate the limit of elastic action, i.e. the beginning of yield. For the earthquake considered the discrepancy between this curve and the upper one is very large and can be reconciled only if a structure is capable of deforming inelastically several times more than that occurring at first yield. Assuming that ductility alone is responsible for the reduction in lateral forces, only buildings of over twenty stories with a period greater than 2 sec and meeting a severe ductility requirement of 6 would survive. Therefore in seismically active regions direct use of code specified response spectra sometimes may be hazardous. For this reason, geologically foreseeable maximum credible earthquake site accelerations on the order of 0.5g have sometimes been used for design in San Francisco (Merovich and Nicoletti 1982). On the other hand the usual response spectra (contrast this with the response spectra given for the Mexico earthquake in the companion paper in this Bulletin) favour tall buildings by indicating a decreasing design base shear with increasing period, i.e. with increasing building height. Therefore, in some geographic locations wind, rather than seismic requirement, controls the design of tall buildings.

#### SOME NEW BASIC SEISMIC CODE PROVISIONS

The new basic seismic requirements pertaining to the lateral load magnitudes and frame ductility assessment for different steel framing systems are drawn from the tentative provisions of SEAOC (1985). These are strongly related to the ATC (1978) and BSSC (1985) recommendations, and are serving as the prototype for the ongoing code work by the AISC and UBC. Except for keeping a mixture of elastic and capacity methods in SEAOC and UBC, or adopting the limit states design approach in AISC and BSSC (formerly ATC), the essential features of these seismic provisions are alike. It can be hoped that all of these bodies soon not only will adopt the member ultimate strength design approach but will go one step further and proceed to the next logical step of capacity design for realistic seismic

loads.

For the allowable stress design approach in a static design procedure, SEAOC gives the following formula for the total design shear  $V$  for regions of high seismicity.

$$V = \frac{ZICW}{R_w} \quad (1)$$

where  $Z$  is the seismic zone factor, either 0.3 or 0.4 depending on the zone,  $I$  is the importance factor, taken as one for standard occupancy structures and as 1.25 for essential or hazardous facilities;  $C$  is the coefficient given below by Eq. 2;  $W$  is the total applicable gravity load; and  $R_w$  is the seismic response modification coefficient.

The numerator of Eq. 1 without  $I$  can be considered representative of an elastic response spectrum, whereas  $R_w$  is the reduction coefficient for obtaining a code design spectrum. The coefficients for  $R_s$  given in Table 1 are patterned after those given for  $R_s$  by ATC (1978) but have been increased about one and one-half times to account for the difference in elastic and ultimate member strength design bases. These values include considerations of ductility as well as system ultimate capacity over and above the calculated nominal strength. Note that  $R_s$  are larger for dual systems thereby recognizing the advantages of such systems in seismic design.

Table 1 - SEISMIC RESPONSE MODIFICATION COEFFICIENT  $R_w$  FOR STRUCTURAL STEEL SYSTEMS PER TENTATIVE SEAOC (1985) RECOMMENDATIONS

BASIC SYSTEM	LATERAL SYSTEM	$R_w$	$H^+$
Bearing Walls	Tension Braces	4	65
	Gravity Load Braces	6	160
Building Frame Systems	CBFs	8	160
	EBFs	10	240
MRF Systems	Ordinary MRFs	6	160
	Special MRFs	12	N.L.*
Dual Systems	CBFs with SMRFs	10	N.L.
	EBFs with SMRFs	12	N.L.

<sup>+</sup>  $H$  is height limit for high seismicity zones in feet.

\* N.L. means no height limit.

The coefficient  $C$  is determined from the following formula:

$$C = \frac{1.25 S}{T^{2/3}} \quad (2)$$

where the site coefficient  $S$  assumes a value of one for rock-like foundations, a value of 1.2 for dense and stiff soils, and is 1.5 for clays or loose sands. The value of  $C$  need not exceed 2.75, nor 2.25, for clays or loose sands. The ratio  $C/R_w$

should not be less than 0.075.

The fundamental period  $T$  of vibration for any structure in seconds can be determined using Rayleigh's formula. For braced frames it can be more simply approximated by the following relation:

$$T = \frac{0.05h_n}{D^{1/2}} \quad (3)$$

where  $h_n$  is the height of a structure in feet, and the effective dimension  $D$  of a building is given as:

$$D = D_{s \max} \left[ \sum \frac{D_s^2}{D_{s \max}^2} \right] \quad (4)$$

where  $D_s$  are the widths of braced frame segments in the direction parallel to the applied force, and  $D_{s \max}$  is the widest braced frame. All dimensions are in feet. For steel MRFs the fundamental period can be found using the following relation:

$$T = 0.035 h_3^{3/4} \quad (5)$$

Instead of using the above formulas the tentative SEAOC (1985) requirements allow the use of design based on the elastic dynamic lateral force procedure using response spectrum analyses. This approach is in wide current use for design of large buildings. ATC (1978) normalized response spectra are recommended for such analyses. Development of specific site response spectra is encouraged for major structures. Time history analyses are also permitted but are seldom used in practice. Dynamic analyses are mandatory for structures with specified types of vertical and/or plan irregularities. These issues are not pursued in this paper. Instead attention is directed to a number of new design developments in widely used steel structural systems: MRFs, CBFs, and EBFs.

MOMENT-RESISTING FRAMES (MRFs)

In the design of steel buildings, moment-resisting structural steel frames are more widely used than any other type. This kind of framing results in no obstructions between the columns, allowing maximum freedom for interior planning and fenestration. In a majority of conventional buildings with numerous columns and beams,

the framing is highly redundant, an important advantage in seismic design. In California construction the connections of beams to columns in such frames are generally made in the field. This contrasts with typical Japanese practice, where they shop-weld the beams to the columns and, in the field, connections are bolted in the middle of the span. In either case the lateral integrity of such frames depends on having beam-column joints not only of sufficient strength but also possessing good ductility. This is a challenging task since welded connections are rigid and must be carefully detailed to obtain the requisite ductility.

Moment connections are basic to the whole field of steel fabrication, and in practice continuous changes take place. Many of the changes are driven by problems of economy.

In designing MRFs for a seismic environment two important problems pertain to the beam-column joints, and one to column splices. The one relates to the beam-column connection itself; the other to the panel zone of the column for beams connected to column flanges. No such problem develops in connecting the beams to column webs, but the beam end connection is critical. These problems are discussed below.

Beam-to-Column Flange Connections

The two basic problems involved in the beam-to-column flange connections are illustrated in the next two figures. If the column flange is relatively thin and no continuity or stiffener plates are used on the column web the condition shown in Fig. 3 can develop. The usual assumption of uniform stress distribution along the column flange tends to cause deformations incompatible with those of the attached beam. Additional stresses are needed to close the gap resulting in actual stresses approximately as shown in Fig. 3(f). Therefore one can anticipate flange failures initiating at the middle of the column face. Column web continuity plates (stiffeners) greatly remedy this condition. Before proceeding with this problem further, some comments on the second basic problem occurring at a joint follow.

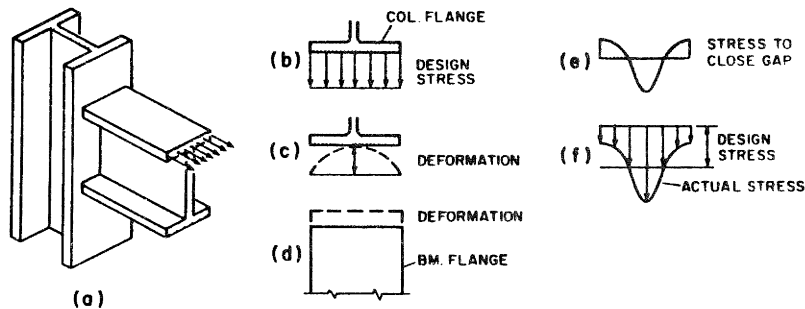


Fig. 3 - BEAM FLANGE STRESS DISTRIBUTION AT A COLUMN WITH THIN FLANGES AND NO STIFFENERS (AFTER BOUWKAMP)

A typical shear force diagram for a column for an interior beam-column joint is shown in Fig. 4 (Krawinkler et al. 1971). From this diagram it is evident that a large panel shear in the column web accompanied by deformations can readily develop. An exaggerated deformation for this condition is shown in Fig. 5. This deformation contributes to the story drift of a frame, and gives rise to regions of high curvature near the beam flange welds which may deleteriously affect these critical welds. The importance of this problem and its remedy by web reinforcement with doubler plates, as well as the problem of the beam-column connection itself, recently received renewed attention, and some new provisions are now included in the tentative SEAOC (1985) recommendations. The supporting evidence for some of these items is discussed below. Re-examination of this problem is particularly important at this time since perimeter framing is now being used with increasing frequency for taller buildings. In such design there is a tendency to make columns from beam sections such as W36s. Compared to the W14 sections used in conventional MRFs, the panel zones of the deeper sections are larger and thinner.

Likewise the flanges are often considerably thinner than in the W14 sections. On the other hand the efficiency of tube construction employing perimeter framing is architecturally and economically very attractive.

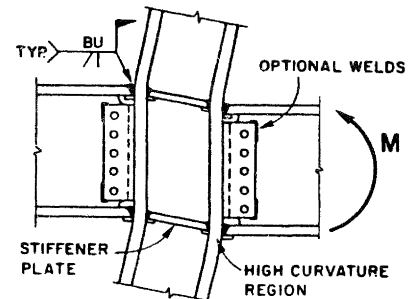


FIG. 5 - EXAGGERATED BEAM-COLUMN JOINT DEFORMATION

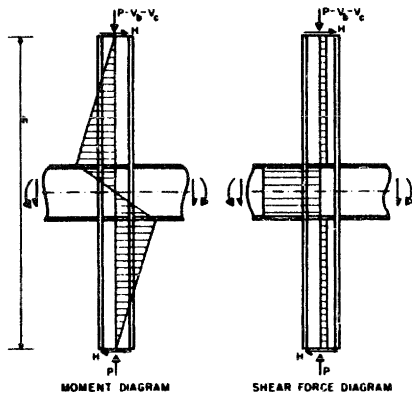


FIG. 4 - MOMENT AND SHEAR DIAGRAMS FOR INTERIOR COLUMN

Experimental Support and Code Provisions

The experimental support for the extensive use of welded flanges and bolted webs for seismic connections stems largely from the Popov and Stephen (1972) tests for a specific project using W18 x 50 and W24 x 76 members. The details for the W24 x 76 connection details are shown in Fig. 6. Note that 7 rather than the customary 6 high-strength 1-in. bolts were used in the bolted connection. Since the column stubs were thoroughly bolted along the outer flange to a substantial steel box, deformation of the panel zone was inhibited in the tests. The hysteretic loops for two cantilever beam tests are shown in Fig. 7. Similar experiments were performed with six other specimens. Based on this work a consensus was reached that beam connections with welded flanges and bolted webs were satisfactory in seismic design.

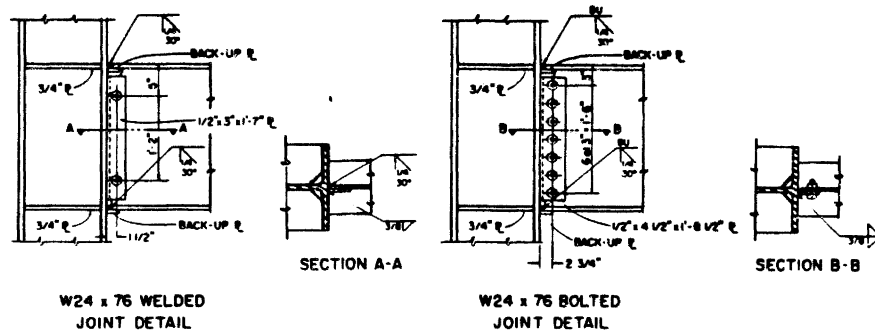


FIG. 6 - MOMENT-RESISTING CONNECTIONS FOR W24 x 76 BEAM

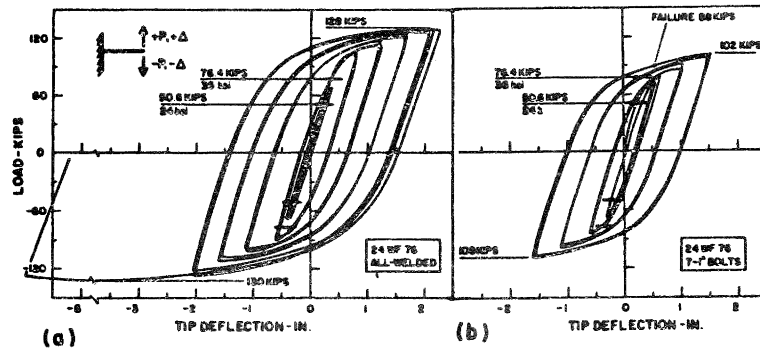


FIG. 7 - HYSTERETIC LOOPS FOR (a) ALL-WELDED AND (b) WELDED FLANGES/BOLTED WEB CONNECTIONS

This problem was re-studied (Popov et al. 1985) in connection with a perimeter framing project for a 47-story building in San Francisco. Since the columns and beams of the prototype structure involved 36-in. (0.91 m) deep members, because of the capacity of the test equipment, the specimens were made half-scale. Even then the replicas required 18-in. (0.46 m) deep members. Some seven specimens of the type shown in Fig. 8 were cyclically tested in the manner schematically illustrated in Fig. 9. An axial force P simulating the gravity and overturning forces were applied to each specimen by means of a 4,000 kip (18 GN) universal testing machine. The tip

cantilever loads  $V_1$  were applied using hydraulic actuators. Two of the specimens were shown in Fig. 8; four specimens, in addition to having doubler plates on the panel zone, also had continuity plates at the level of the beam flanges; one specimen, while having the continuity plates, had no doubler.

Cyclic experiments on these specimens showed the unquestionable advantage of continuity plates. This can be readily seen from Fig. 10 where a set of hysteretic loops for Specimen 2 having continuity plates shows a much greater capacity to dissipate energy than in the case for Specimen 1 which

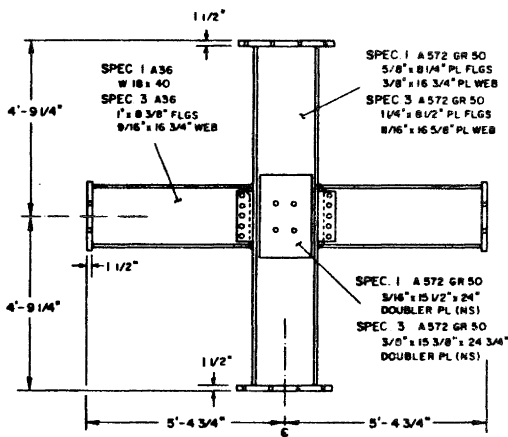


FIG. 8 - GENERAL CONFIGURATION OF SPECIMENS

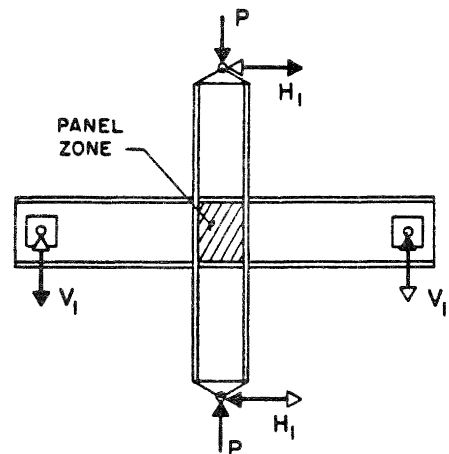


FIG. 9 - SCHEMATIC TEST SET-UP

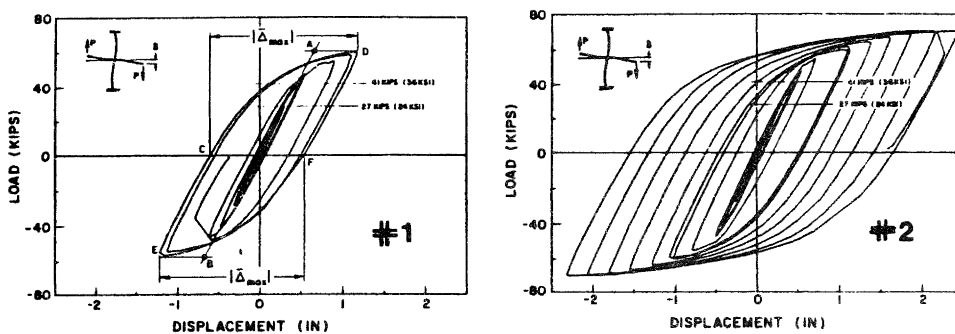


FIG. 10 - LOAD-DISPLACEMENT HYSTERETIC LOOPS FOR SPECIMENS 1 AND 2

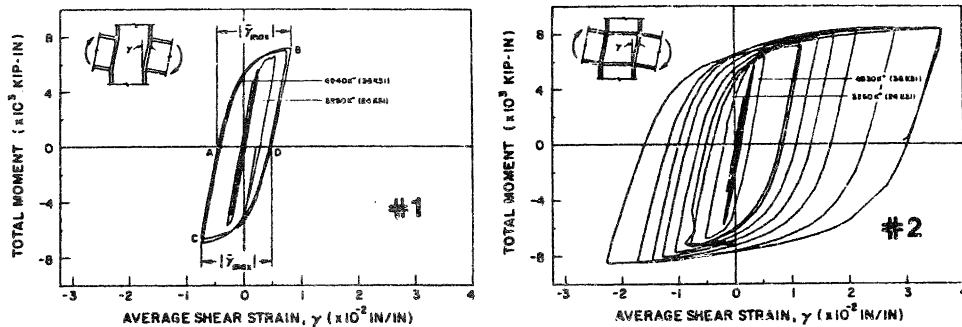


FIG. 11 - MOMENT-PANEL SHEAR STRAIN HYSTERETIC LOOPS FOR SPECIMENS 1 AND 2

had only a doubler plate. It is also evident that Specimen 2 was much more ductile than Specimen 1. The ultimate inelastic rotation for Specimen 2 caused by the panel zone shear deformation shown in Fig. 11 together with the inelastic beam rotation was 0.053 rad, representative of good inelastic behaviour. On the other hand Specimen 1, also shown in Fig. 11, attained a total inelastic rotation of only 0.018 rad. Moreover when Specimen 6 with no doubler plate is compared (not shown) with a similar Specimen 4 with a doubler plate, a substantially lower capacity is observed.

In both of these series of experiments the plastic moduli of the flanges alone were greater than 70% of the corresponding plastic moduli of the total girder section. The behaviour of these connections under cyclic load was considered to be satisfactory although in advanced stages of loading a small amount of bolt slippage was observed in all bolted web connections. Therefore it was considered by SEAOC (1985) to be appropriate to continue permitting the use of such connections for girders of comparable or larger flange geometry. However, if a girder proportionally has a larger web than the test specimens, at least 20% of plastic web capacity should be developed by means of welding the shear tab to the girder web. This welding is to be applied in addition to the required high-strength bolts. It is believed that for such cases this small amount of welding would inhibit bolt slip.

The 1985 SEAOC recommendation for determining the column flange thickness when column stiffeners can be dispensed with are more stringent than the provisions given in the 1980 AISC specifications. The beam tension flange force  $P_{bf}$  in comparison with the AISC Eq. 1.15-3 is increased by an empirical factor of 1.8 to account for the beam flange strain-hardening effect leading to the following relation:

$$P_{bf} = 1.8F_{yb} b t_b \quad (6)$$

where  $F_{yb}$  is the yield strength of the beam flange,  $b$  is the beam flange width, and  $t_b$  is the thickness of the beam flange. Since in determining the column flange thickness,  $P_{bf}$  enters the relevant AISC equation as  $\sqrt{P_{bf}}$ , the critical flange

thickness is increased only 34%. This recommendation is in accord with the test results (Popov et al. 1985). The same experiments suggest the use of slightly thicker stiffeners for columns with thin flanges than is currently specified.

The 1985 SEAOC tentative provisions drastically reduce the required shear strength of joints. Previously the relevant moment for calculating the panel shear for beams framing on both sides of the column was  $2M_p$  where  $M_p$  is the plastic beam moment. Now this moment is to be determined as that caused by the gravity loads plus 1.7 times the code seismic design forces. Further, the joint shear strength  $V$  is calculated using a modified equation:

$$V = 0.55F_y d_c t \left[ 1 + \frac{3b_c t_{cf}^2}{d_b d_c t} \right] \quad (7)$$

where as before  $F_y$  is the yield strength of the material,  $d_c$  is the column depth,  $t$  is the total thickness of the joint including doubler plates,  $b_c$  is the width of the column flange,  $t_{cf}$  is the thickness of the column, and  $d_b$  is the depth of the beam.

Equation 7 is a refinement of an earlier proposal (Krawinkler et al. 1971, Krawinkler 1978). The expression in the square brackets recognizes the contribution of column flanges in resisting shear. Such column flanges form a frame around the panel zone and inelastic hinges must form in these column flanges in regions of high curvature (see Fig. 5) along with the yielding process in the column web. Therefore the concept of including flange size parameters in Eq. 7 appears to be sound. However the resulting reduction in the required panel zone thickness using this equation as well as greatly reduced beam moments for calculating the panel zone shear may lead to serious problems. It has been shown (Manheim and Popov, 1983) that frames having column panel zones of inadequate capacity can form plastic shear hinges, thereby significantly reducing frame capacity. Moreover the panel zone deformations can substantially contribute to the story drift. The author believes that further experimental and analytical research in this

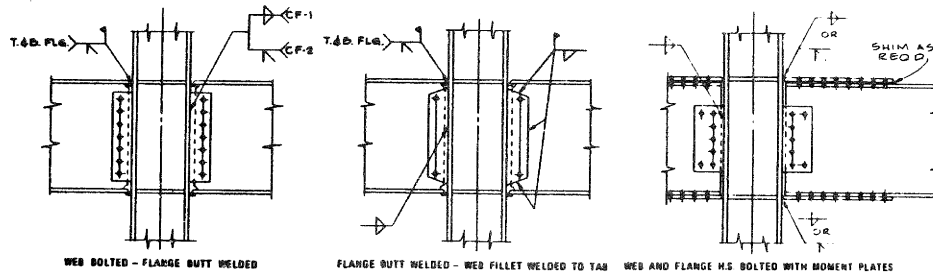


Fig. 12 - MOMENT CONNECTIONS: BEAM-TO-COLUMN FLANGE (COURTESY STEEL COMMITTEE OF CALIFORNIA)

general area is essential.

Some representative beam-to-column flange connections in general use in California are shown in Fig. 12. Although the type shown in Fig. 12(a) is preferred, the proposed requirements for seismic design will, as previously noted, require in some cases field welds along the shear tab to develop 20% of the beam web plastic moment. The connection illustrated in Fig. 12(b) is somewhat costlier than that shown in Fig. 12(a), since it is more time consuming for the erection process. Costs are about double for the connection shown in Fig. 12(c), and it is used only if no field welding is possible.

Several typical moment-resisting joints for connecting beams to column webs used in California are shown in Fig. 13. These connections are less reliable than those to column flanges for severe cyclically reversing loads. Popov and Pinkey (1968) reported rather erratic behaviour of such connections under cyclic loading. The attained ductility of these connections was significantly smaller than that of connections to column flanges. Often cracks propagated either along the heat-affected zone of the flange welds or along the stiffener plates causing premature failure. These cracks initiate in the regions of high stress concentration at the juncture of the beam flange with the stiffener plate. Similar observations were made on large connections in monotonic tests by Rentschler et al. (1980). More recently Lee and Lu (1985) again demonstrated that the behaviour of such connections in severe cyclic tests is poor.

The suggestions by Driscoll and Beedle (1982) for improving the ductility of beam-to-column web connections appear to be inconclusive. Because of the uncertainty regarding the cyclic behaviour of these connections, Tsai and Popov (1986) have recently made two alternative designs and carried out cyclic experiments. In one design the best current fabrication practice was adopted. In the other a novel approach for resolving the problem was attempted. The cyclic tests carried out with the best conventional design showed good behaviour, and that with the alternative new design performed outstandingly. Some highlights from this investigation are given below.

In a good deal of previous experimental research the continuity plates welded to the column web as well as to the column flanges generally have been made thicker than the beam flanges by a very modest amount. This is possible under laboratory conditions. In reality, however, beam depths may slightly under-run or over-run their normal dimensions. Therefore, in order to eliminate field problems, the fabricating shops prefer to make continuity plates thicker than required by calculations. Good practice suggests having the top continuity plate at least 1/8 in. (3.2 mm) thicker than the beam flange. Allowing for mill beam depth tolerances the bottom continuity plate is made 1/4 in. (6.4 mm) thicker than the beam flange. These conditions alter the behaviour of a joint, and, with good welding, are likely to produce a reasonably satisfactory connection.

Even better behaviour of a beam-to-column web connection can be achieved by

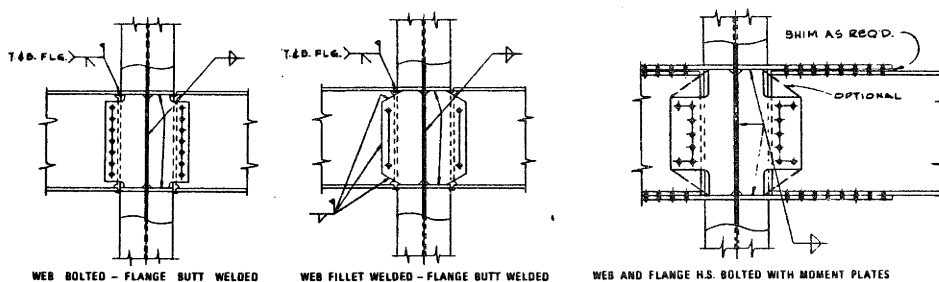


FIG. 13 - MOMENT CONNECTIONS: BEAM-TO-COLUMN WEB (COURTESY STEEL COMMITTEE OF CALIFORNIA)

modifying the above detail. By using the detail discussed above and reinforcing the beam flange welds with small ribs the connection is dramatically improved. This unconventional detail is shown in Fig. 14. Here two small ribs are welded to the top flange and its continuity plate, and two similar ribs are attached at the bottom. In both instances the ribs straddle the flange welds. Since full penetration flange welds project above the top of the continuity plates, a small smooth notch on the bottom of each rib is required. These crescent-shaped notches were made by grinding to allow placement of continuous welds along the ribs. Note that there are no welds at the tops of the bottom ribs. The three small welds along the shear tab effectively prevented bolt slippage.

Hysteretic loops for the two recent experiments are shown in Fig. 15. Specimen 1, with the ribs, showed an exceptionally good ductility of 5.2 when measured from the initial position to the point of maximum load. The comparable ductility for Specimen 2 was 4.0. If instead the ductilities are measured from the preceding zero load intercept, they are, respectively, 10.6 and 6.5. However it should be remarked that Specimen 2 abruptly

failed, whereas the test with Specimen 1 was terminated before any indication of imminent failure. Further research on these connections is in progress at Berkeley.

Column Flange and Web Splices

Column splices are encountered in structural steel construction in all but small buildings. In seismic design these splices must have adequate capacity to resist lateral forces caused by an earthquake. The tentative SEAOC (1985) recommendations are rather explicit as to how the splices should be made. Besides requiring the column splices to resist the required axial forces and moments, specific provisions are spelled out for partial penetration welds. Columns subjected to net tension must be designed for 150% of the code-specified strength, and should be no less than 50% of the flange area of the smaller column. Further, SEAOC requires that partial penetration welds should not be within 3 ft (0.9 m) of a beam-column joint. In reality designers try to locate column splices not only away from beam-column joints, but to have them near column inflection points. This issue requires further comment.

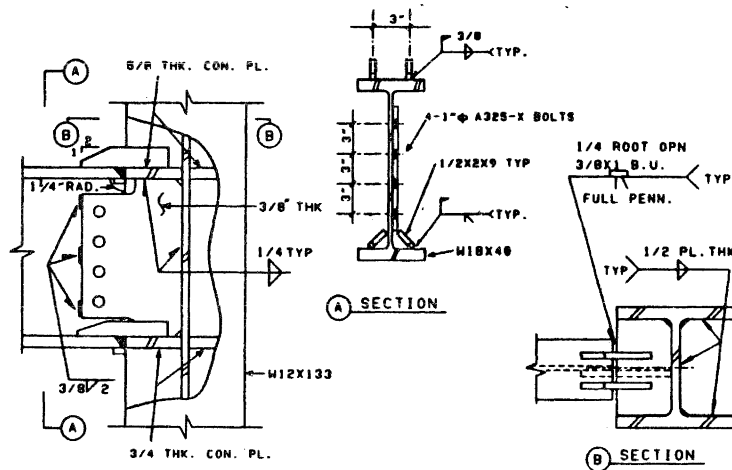


FIG. 14 - DETAILS FOR SPECIAL BEAM-TO-COLUMN WEB CONNECTION

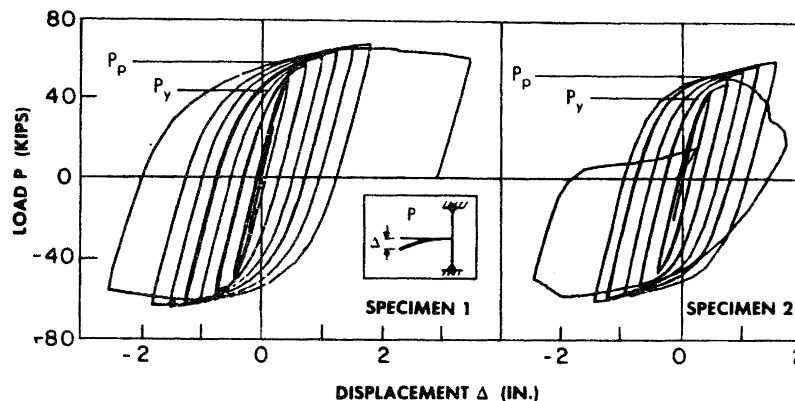


FIG. 15 - HYSTERETIC LOOPS FOR BEAM-TO-COLUMN WEB CONNECTIONS

A simplistic adherence to trigonometric solutions in locating inflection points in columns based on static lateral loading patterns may be grossly misleading. As pointed out by Paulay (1983) column inflection points during an earthquake may experience an extraordinary shift in location. Therefore the New Zealand Code (1982) for concrete structures deserves serious consideration for adoption for steel frames. This suggestion is reinforced by the brittle character of welded splices of large sections in tension. Some limited experimental results on the behaviour of full and

partial penetration welds in heavy sections are shown in Fig. 16 (Popov and Stephen 1977). The ductility of tension specimens with identical partial penetration welds in both flanges was very poor. The specimen with full penetration welds behaved much better.

Representative details of flange and web column splices commonly used in California are shown in Fig. 17. Some further investigations of this problem are in progress in Berkeley.

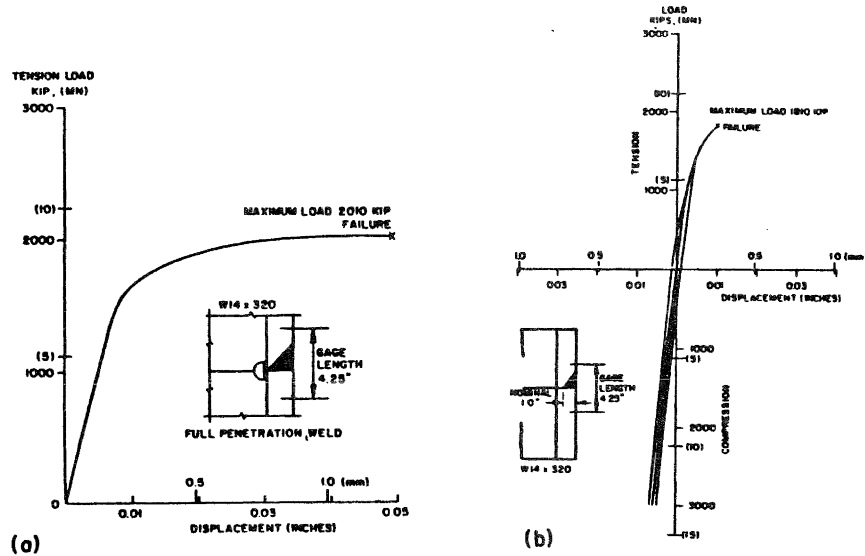


FIG. 16 - DUCTILITY COMPARISONS FOR LARGE WELDED TENSION SPLICES

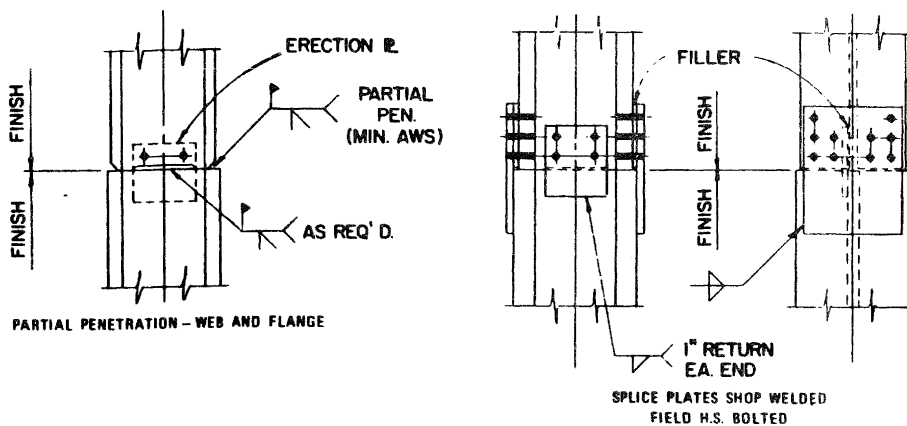


FIG. 17 - TYPICAL FLANGE AND WEB COLUMN SPLICES (COURTESY STEEL COMMITTEE OF CALIFORNIA)

### CONCENTRICALLY BRACED FRAMES (CBFs)

Moment-resisting framing is basic in steel design. However it becomes uneconomical for controlling story drift at service loads, particularly along narrow widths of buildings, and diagonal bracing or concrete shear walls become necessary. Here attention will be confined to the conventional CBFs which are vertical trusses cantilevered from the foundations or lower stories for the purpose of resisting lateral forces. In such framing the columns act as chords and floor beams and diagonal braces act as web members. As can be noted from Table 1, the tentative SEAOC (1985) recommendations permit the use of such framing with some limitations on building height. When used in conjunction with special MRFs, discussed in the previous sections, such framing, referred to as a dual system, can be used for buildings of any height. In the opinion of the author, CBFs alone without a backup from MRFs should be used only for one and two-story buildings and minor construction.

SEAOC (1985) provides a number of guidelines for the design of CBFs. Before discussing these, however, it is well to review the behaviour of the critical element of such systems - the brace. In seismic applications a brace may be subjected to severe cyclic load reversals. Experimental evidence on the behaviour of such braces is very limited. The conventional codes are concerned primarily with the ultimate static capacity of struts with an appropriate factor of safety. Under cyclic loads such members behave very differently. An example of the hysteretic behaviour of a strut is shown in Fig. 18 (Popov 1979, Popov and Black 1981). The initial maximum compressive load is in excellent agreement with predictions given

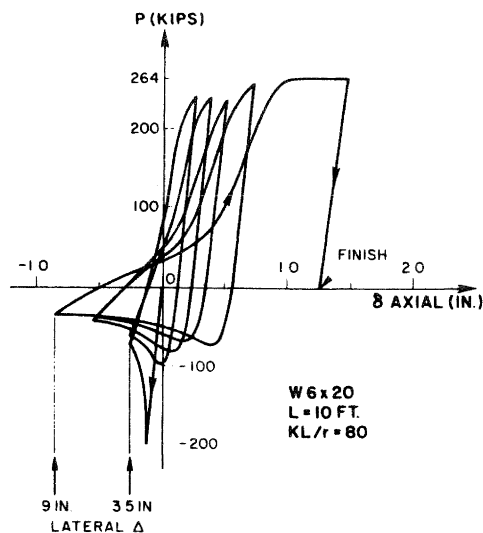


FIG. 18 - AXIAL FORCE-DISPLACEMENT HYSTERETIC LOOPS FOR A STRUT

by conventional codes such as the AISC (1980). On cycling, the tensile capacity is retained. However a dramatic decrease in the compressive load-carrying capacity occurs during consecutive cycles. Even after just one severe cycle the compressive capacity is only half as large as it was initially. Moreover because of the Bauschinger effect even straining a strut initially in tension may reduce its capacity by as much as 25% in compression, Fig. 19 (Popov 1979).

The above observations explain the poor behaviour of a single diagonal strut serving as a brace in the simple frame shown in Fig. 20(a) (Ballio and Zandonini 1985). Even for an X-braced frame, shown in Fig. 20(b), the hysteretic loops significantly degrade during cyclic loading. Both of these frames had hinges at all joints. The behaviour of CBFs is substantially improved if used as part of a dual system (Maison and Popov 1980). Hysteretic loops of a test on such a dual system are shown in Fig. 21. The attained ductility is only moderate.

As the result of such evidence, SEAOC (1985) recommends that braces in regions of high seismicity be designed for compressive stresses at code loads whether acting in tension or compression. Moreover the braces should be so oriented that for either lateral force direction the compression and tension capacities of the braces are within 10% of each other. SEAOC (1985) further recommends that, except for buildings not over two stories and in roof structures, K-braces on one side of a column be prohibited. Special admonitions are also expressed with regards to V-braces. Except for roof stories and penthouses, V-bracing

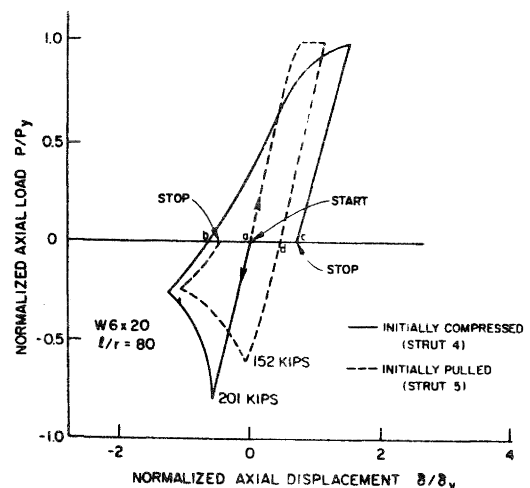


FIG. 19 - LOSS OF COMPRESSIVE CAPACITY OF STRUT INITIALLY STRAINED INTO TENSILE YIELD

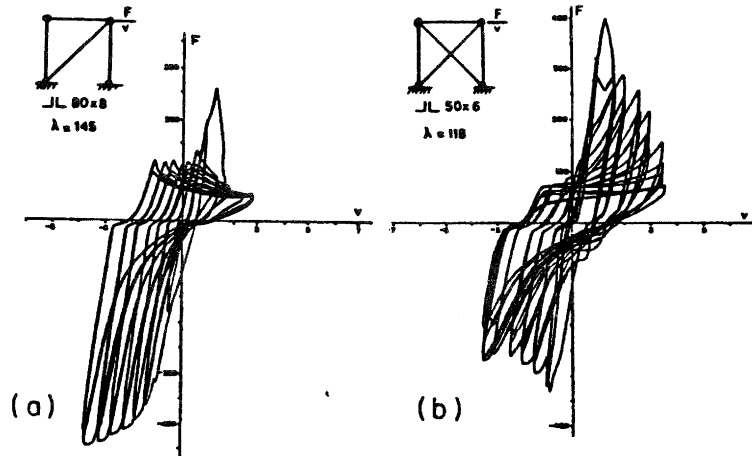


FIG. 20 - HYSTERETIC LOOPS FOR CBFs HAVING (a) SINGLE AND (b) DOUBLE DIAGONAL BRACES (COURTESY BALLIO AND PLUMIER)

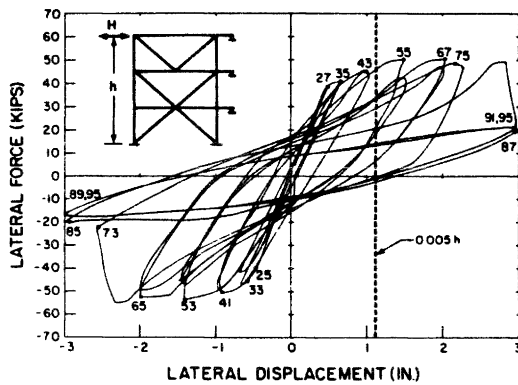
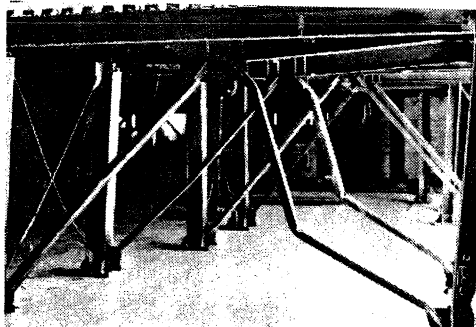
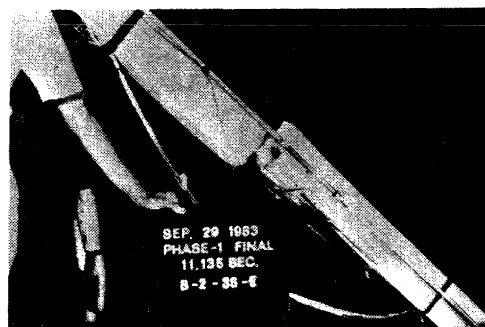


FIG. 21 - HYSTERETIC LOOPS FOR DUAL MRF AND CBF SYSTEM

is limited to carry no more than 10% of the story shear in any story of structures over 120 ft (36 m) in height. Moreover, when V-braces intersect a beam from below, the beam must be capable of supporting all tributary loads as if the bracing were not present. The necessity for the last requirement is dramatically illustrated by a photograph from a Tsukuba cyclic experiment shown in Fig. 22(a). After one of the braces buckles it carries a negligible axial force, whereas the tension brace remains sound. This causes an imbalance of vertical forces at the joint and the floor beam is severely pulled down at the center. Deterioration of tubular braces such as in Fig. 22(b) can be greatly delayed by concrete infill (Bridge 1976).



(a)



(b)

FIG. 22 - TSUKUBA CYCLIC TESTS ON CBFs (COURTESY MAKOTO WATABE)

In CBFs, for a successful bracing system, the connections should be designed for the full strength of the member. Experiments as well as field experience clearly show that failures can occur at bolt holes in the brace as intended. Therefore bolted connections must be reinforced with welded-on plates to maintain the gross section of a brace. The tentative SEAOC (1985) recommendations may be somewhat inadequate on this point. In general, the preferred welded connection should be designed such that its centroid coincides with the applied force in the brace (Astaneh-Asl et al. 1982, Gugerli and Goel 1982).

Since it is recognized that the brace carrying capacity in CBFs is reduced during cyclic loading, different proposals have been advanced to overcome this problem. In all these schemes the braces are designed not to buckle, as, at a predetermined force controlled slip at an end brace connection or yielding of an axial element takes place. To achieve this various ingenious devices are becoming available. Pall (1982) allows a controlled slip along breaklining inserts to take place at one end of a brace, whereas Scholl (1984) uses viscoelastic material in a similar manner, and Fitzgerald (1985) employs the same concept using Belleville spring washers. Koller (1985) absorbs energy in a long yielding circular rod enclosed in a tube. An explanation for the basis is developed by Feodos'ev (1950). Some of these devices are patented. As an alternative, one can employ EBFs to accomplish the same end.

#### ECCENTRICALLY BRACED FRAMES (EBFs)

Usually in conventional design an attempt is made to have the centerlines of braces intersect with the centerlines of columns and beams at a point. This results in conventional CBFs. However it is possible to offset the brace centerlines such that they would not intersect at floor levels at a common point with those of the columns and/or beams. Such framing generates EBFs. The main characteristic of such framing is that the axial forces induced in the braces are principally transmitted either to a column or another brace through shear and bending in a floor beam segment called a link. At service loads EBFs provide an excellent laterally stiff structural system, whereas during a severe earthquake the links yield and effectively dissipate the input energy. Since the ultimate capacity of the links can be accurately estimated, the braces can be designed so as not to buckle thereby obviating the principal disadvantage encountered with CBFs. The behaviour and preliminary design of EBFs are discussed in the companion paper in this Bulletin. Here the discussion is limited to some remarks on a few selected types which at this writing are favoured by the author.

Whereas EBFs can be arranged in many different ways, the three types shown in Fig. 23 have a number of advantages. The bracing shown in Fig. 23(a) avoids the need for critical beam flange welding in the field, and joints at columns can be bolted (see Fig. 12 on p. 28). Of course, in

the latter, the advantages of a dual system are lost. The V-bracing shown in Fig. 23(b) has an advantageous kinematic mechanism in the plastic range of frame behaviour. The single diagonal bracing shown in Fig. 23(c) is well suited for narrow bays. The nominally concentric joints at columns can be bolted as suggested for the frame in Fig. 23(a).

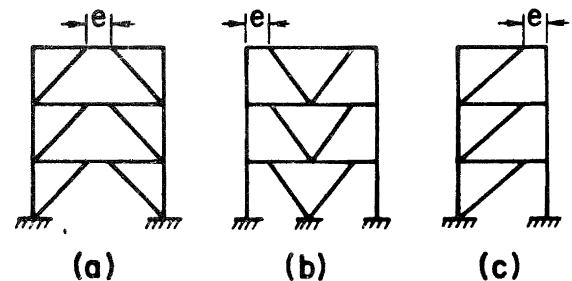


FIG. 23 - RECOMMENDED EBF TYPES

In general the links should be made as short as possible consistent with the required kinematics of inelastic deformation. Braces should be conservatively chosen such that, together with the beam segment outside the link, enough strength is developed to cause the link to function in the strain-hardening range. The links must be laterally braced at the ends, and the beam segment outside the link acting as a beam-column may require lateral bracing to prevent lateral torsional buckling. Lastly, it is well to have braces at a steep angle in order to reduce the axial force in the beam-column referred to above. Experimental evidence as well as analytical studies indicate that by implementing the above recommendations, excellent seismic-resistant framing is achieved.

#### FUTURE TRENDS AND NEEDS

Much progress is being made in the design, fabrication, and erection of steel buildings in seismically active regions. Further research, however, is needed to make new advances. For achieving this the following directions seem worthy of consideration.

1. Designs of the future should be based on the actual capacity of members and realistic seismic forces. For steel this means full recognition of strain-hardening properties under cyclic loads. Design based on elastic spectra with constant  $R_y$  reduction factors for each system requires further examination.

2. Cyclic experiments supported by analytical studies on large members having thick material is greatly needed. Such studies must include consideration of welds and welding processes.

3. Greater interaction between analysts, designers, fabricators, and erectors is most desirable, particularly with regards to connections.

4. Steel designers must provide leadership in developing mixed construction. Reduced costs and speed of construction may be gained in pursuing this direction.

Readers with an interest in California seismic design provisions are urged to study the forthcoming SEAOC Lateral Force Requirements and accompanying Commentary. No attempt at comprehensiveness was made in this paper.

#### ACKNOWLEDGEMENTS

The author is indebted to the National Science Foundation (current grant ECE-8418487), the American Iron and Steel Institute, and the New Zealand Heavy Engineering Research Association for providing the opportunity for developing this paper and for their partial support of this effort. The opinions expressed are those of the author and do not necessarily reflect the views of the sponsors or SEAOC. In the above organizations, Dr. S.C. Liu, A. Kuentz, and G.C. Clifton, respectively, were particularly helpful.

#### REFERENCES

- AISC (American Institute of Steel Construction). 1934. Steel construction manual. New York, N.Y. 2d ed., p.127.
- AISC. 1980. Manual of steel construction. Chicago, IL.
- AISC. 1985. Proposed (LRFD) load and resistance factor design specifications for structural steel buildings. Chicago, IL.
- ATC (Applied Technology Council). 1978. Tentative provisions for the development of seismic regulations for buildings. NBS Special Publication 510. U.S. Department of Commerce, Washington, D.C.
- Astaneh-Asl, A., Goel, S.C., and Hanson, R.D. 1982. Cyclic behaviour of double angle bracing members with gusset plates. Report No. UMEE 82R7, University of Michigan, Ann Arbor, MI.
- Ballio, G. and Plumier, A. 1985. Définition d'une méthode expérimental de la résistance antisismique d'éléments de structure en acier. Construction Métallique. (3)21-35.
- Bertero, V.V. et al. 1986. Analytical and experimental studies conducted on the response of 0.3 scale models of 6-story steel structures to earthquake ground motions. Proceedings 3rd U.S. National Conference on Earthquake Engineering. Charleston, S.C.
- B.R.I. (Building Research Institute, Tsukuba, Japan) Steel Group. 1985. Full scale tests on a six-story steel building. U.S.-Japan Sixth Joint Technical Coordinating Committee Meeting. Maui, Hawaii.
- Bridge, R.Q. 1976. Concrete filled steel tubular columns. Research Report No. R283. University of Sydney, Australia.
- BSSC (Building Seismic Safety Council). 1985. NEHRP recommended provisions for the development of seismic regulations for new buildings. Washington, D.C.
- Driscoll, G.C. and Beedle, L.S. 1982. Suggestions for avoiding beam-to-column connection failure. AISC Engineering Journal. 19(1) 16-19.
- Feodos'ev, V.I. 1950. Selected problems and questions in strength of materials (in Russian). Technico-Theoretical Literature. Moscow, U.S.S.R. 188-192.
- Fitzgerald, R.F. 1985. Slotted bolted connections in seismic resistant bracing systems. T.F. Fitzgerald & Associates, San Francisco, CA.
- Gugerli, H. and Goel, S.C. 1982. Inelastic cyclic behaviour of steel bracing members. Report No. UMEE R1. University of Michigan, Ann Arbor, MI.
- Itoh, H. and Wang, M.L. 1985. Nonstructural element test phase. U.S.-Japan Sixth Joint Technical Coordinating Committee Meeting. Maui, Hawaii.
- Koller, K. 1981. SaintJohn beam-column. U.S.A. Patent No. 4,281,487.
- Krawinkler, H. 1978. Shear in beam-column joints in seismic design of steel frames. AISC Engineering Journal. 15(3)82-91.
- Krawinkler, H., Bertero, V.V. and Popov, E.P. 1971. Inelastic behaviour of steel beam-to-column subassemblages. Report No. EERC 71-7. University of California, Berkeley, CA.
- Lee, S.J. and Lu, L.W. 1985. Cyclic tests of full-scale composite beam-to-column joints. U.S.-Japan Sixth Joint Technical Coordinating Committee Meeting. Maui, Hawaii.
- Maison, B.F. and Popov, E.P. 1980. Cyclic response prediction for braced steel frames. Jour. Struct. Div. ASCE. 106(ST7) 1401-1416.
- Merovich, A.T. and Nicoletti, J.F. 1982. Eccentric bracing in tall buildings. Jour. Struct. Div., ASCE. 108(6) 2066-2080.
- Manheim, D.N. and Popov, E.P. 1983. Plastic shear hinges in steel frames. Jour. Struct. Engrg., ASCE. 109(10) 2404-2419.
- Pall, A.S. and Marsh, C. 1982. Response of friction damped braced frames. Jour. Struct. Engrg., ASCE. 108(6) 1313-1323.
- Paulay, T. 1983. Deterministic seismic design procedures for reinforced concrete buildings. Engineering Structures. 5(1) 79-86.
- Popov, E.P. 1979. Inelastic behaviour of steel braces under cyclic loading. Proceedings 2d U.S. National Conference on Earthquake Engineering. Earthquake Engineering Research Institute. Stanford, CA. 923-932.
- Popov, E.P., Amin, N.R., Louis, J.J.C., and Stephen, R.M. 1985. Cyclic behaviour of large beam-to-column assemblies. Earthquake Spectra. 1(2) 203-238.
- Popov, E.P. and Black, R.G. 1981. Steel struts under severe cyclic loadings. Jour. Struct. Div., ASCE. 107(ST9) 1857-1881.

Popov, E.P. and Pinkey, R.B. 1968. Behaviour of steel building connections subjected to inelastic strain reversals. Bulletins 13 & 14. American Iron and Steel Institute. New York, N.Y.

Popov, E.P. and Stephen, R.M. 1972. Cyclic loading of full-size steel connections. Bulletin 21. American Iron and Steel Institute. New York, N.Y.

Popov, E.P. and Stephen, R.M. 1977. Tensile capacity of partial penetration welds. Jour. Struct. Div. ASCE. 103 (ST9) 1721-1729.

Rentschler, G.P., Chen, W.F., and Driscoll, G.S. 1980. Tests on beam-to-column web moment connections. Jour. Struct. Div. ASCE. 106 (ST5) 1005-1022.

SEAOC (Structural Engineers Association of California). 1976. Recommended lateral force requirements. Seismology Committee. San Francisco, CA.

SEAOC. 1985. Tentative lateral force requirements. Seismology Committee. Los Angeles/Sacramento/San Francisco, CA.

Scholl, R.E. 1984. Brace dampers: an alternative structural system for improving the earthquake performance of buildings. Proceedings 8th World Conference on Earthquake Engineering. San Francisco, CA.

Standards Association of New Zealand. 1982. Code of practice for the design of concrete structures. NZS 3104. Wellington, N.Z.

Tsai, K.-C. and Popov, E.P., 1986. Alternative designs and tests of beam-to-column web seismic moment connections. Report No. UCB/EERC-86/05. University of California, Berkeley, CA.

UBC (Uniform Building Code). 1985. International conference of building officials. Whittier. CA.

Wyllie, L.A. Jr. 1985. The 1985 international engineering symposium on structural steel committee, AISC. Chicago, IL. 7-1 to 7-14.

Yamanouchi, H. et al. 1984. Experimental results on a K-braced steel structure under seismic loading utilizing full-scale six-storey test structures - U.S.-Japan Cooperative Research Program. Proceedings, 1984 Annual Meeting. Structural Stability Research Council. San Francisco, CA.