SEISMIC CODE DEVELOPMENT FOR STEEL STRUCTURES

by

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SYNOPSIS

This paper discusses revisions proposed by the Structural Engineers Association of California for the seismic design of steel structures. Several of the new provisions are drastic departures from current practice and are expected to have a considerable impact on design. The proposed changes are discussed from the perspectives of the practising engineer, the researcher, and the committee that drafted the new provisions.

INTRODUCTION

The Structural Engineers Association of California (SEAOC) has published for many years a booklet called "Recommended Lateral Force Requirements and Commentary" which is customarily referred to as the SEAOC Blue Book. Although this booklet is not an enforceable code, it is considered a model document that is taken over verbatim by many code writing bodies and agencies in the United States and elsewhere. The unique aspect of this booklet is that it is written by practising engineers rather than code writers or government officials. Thus, its function is to provide guidance by engineers to engineers. This makes a powerful and dynamic document since its content is governed by engineering conscience and knowledge and not by material interests or political decision processes.

This booklet is being updated continuously and has undergone many changes during the last few decades to keep up with new knowledge developed through research or practical experience or lessons learned from earthquakes. The last official edition was published in 1980. Since this time the Seismology Committee of SEAOC has devoted its efforts to a major revision of this booklet which has recently been published in a draft form under the title "Tentative Lateral Force Requirements, October, 1985". The revisions are drastic, indeed, at least in two areas. One is in general design requirements, including identification of base shear, period determination, treatment of irregularities, and site-soil effects. The other area of drastic changes is in design requirements for steel structures. The new provisions are at this time only tentative but are expected to be published officially and with only few changes in the next Blue Book edition in October 1986.

The purpose of this paper is to discuss a few of the relevant new provisions and their research background as well as their design implications. Since several of the new provisions are drastic departures from current practice, they are expected to have a considerable impact on design practice and may affect the choice of structural systems from a structural and economic viewpoint. Because of the controversial nature of some of the new provisions, arguments for the provisions as well as personal reservations are expressed in this paper. In the discussion, presently employed seismic requirements are referred to as "existing provisions" and the new proposed requirements are referred to as "proposed provisions".

SEISMIC DESIGN LOADS

For a great majority of structures a static force procedure similar to the one used presently is suggested. However, the base shear equation in the proposed provisions is modified and reads as follows:

$$V = ZICW/R_w \qquad (1)$$

The individual terms of this equation are discussed in the plenary paper by Popov 1986. In concept, although not literally, the product ZCW represents a conservative estimate of the elastic force demand on the structure and the term R_w/I reduces this elastic force demand to a working stress design force level. Since R_w is as large as 12 and I is usually unity, the design force level is usually a small fraction of the elastic force demand.

When this semi-empirical equation is related to theory, the parameter R_w becomes a matter of much controversy. In its present form this parameter depends only on the type of structural system and is independent of the natural period of the structure. Time history analyses of nonlinear single and multi-degree of freedom systems indicate clearly that a period independent R_w will lead to designs in which short period structures ($T \sim 0.5$ seconds and smaller) are subjected to much higher ductility demands than intermediate and long period structures. Thus, implicitly,

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less safety appears to be provided for short period structures. There may be good reasons to do so since low rise structures may have more reserve strength due to the absence of high column loads and larger participation of nonstructural elements. It is questionable whether these arguments are adequate to justify the much higher ductility demands indicated by analysis. In the writer's opinion they are not and the period dependence of R_w should be considered.

Dependent on the site soil coefficient S and new period equations, there will be differences in the seismic design forces between the existing and proposed provisions. However, in most cases the differences will be small except for structures with very long periods for which the proposed provisions place an upper limit of 0.03 on the base shear coefficient in seismic zone 4.

The static force procedure may be used for regular structures under 240 feet (73m) in height and for irregular structures less than 4 stories or 50 feet (15m) in height. The proposed provisions address the issue of irregularity in plan and elevation in detail and classify them according to geometry, eccentricity, and stiffness and mass discontinuities.

A dynamic force procedure may be used for any structure as an alternative to the static force procedure but is required for structures of more than 240 feet (73m) in height and for most irregular structures of more than 3 stories or more than 50 feet (15 m) in height. Dynamic analysis may be based either on elastic response spectrum analysis together with modal superposition or on time history analysis.

DESIGN PHILOSOPHY FOR STEEL STRUCTURES

In the development of design criteria attention was focused on the concept that non-ductile failure modes should be avoided through proper member strength design and inelastic deformations should be concentrated in regions in which ductility can be provided through proper detailing. For steel structures this implies that emphasis should be on the prevention of column buckling and connection failures. In concept this simply means that design should assure that the buckling strength of vertical load carrying columns and the strength of connections will not be called upon in a severe earthquake.

Implementation of this concept was the subject of much discussion in the drafting of the proposed provisions. It requires the identification of the maximum force (for strength design) and/or deformation (for detailing for ductility) that elements may experience in an earthquake. This could be achieved approximately by deforming the structure under the seismic load pattern to a story drift level that is expected in a severe earthquake. Such story drift levels could be estimated with the same degree of judgement as the R_w factors. The analysis to be performed would have to be an inelastic analysis that takes into account the strength of the individual elements and the foundation.

To simplify the design approach, the Committee decided against the need for inelastic analysis and for an elastic estimate of the maximum force demand. This force demand was set at $3R_w/8$ times the seismic design force, which corresponds to the use of an R_w value of 2.67 for all structural systems. Thus, whenever reference is made to a maximum force demand, the proposed provisions refer to $3R_w/8$ times the seismic design force. In addition, in connections the force demand is limited by the strength of the connected members, and in columns the axial force demand is limited by the maximum force that can be transferred to the column by the other elements of the structure.

There are several reservations that can be expressed to this approach. Firstly, in a severe earthquake in which significant inelastic deformations are expected it is conceptually more suitable to estimate member force demands from inelastic rather than elastic analysis. And, secondly, the proposed provision for force demand based on elastic analysis necessitated the addition of several exceptions to account for the limiting conditions expressed at the bottom of the previous paragraph. The problems are illustrated in conceptual lateral load-deflection diagrams shown in Fig. 1. For both cases (a) and (b) the force demand based on elastic analysis is the same whereas inelastic analysis gives vastly different results. For case (a) an exception in the proposed provisions permits a reduction in the force demand to the true mechanism load but this would require an inelastic analysis. So why not use an inelastic analysis in the first place?

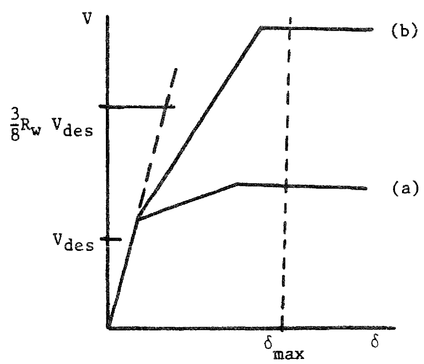


FIG. 1 - STRENGTH DEMAND BASED ON ELASTIC AND INELASTIC ANALYSIS

The resistance to inelastic analysis makes it also impossible to evaluate rationally the P-Δ effect that may become important in flexible frame structures at large inelastic story drifts. This is illustrated in Fig. 2 which shows conceptual lateral load-dflection diagrams without (solid lines) and with (dashed lines) P-Δ effect. The P-Δ effect may cause negative structural stiffness and consequently drifting of the seismic response and amplification of lateral deflection to the point of dynamic instability. If inelastic analysis would be performed, the P-Δ effect could be assessed at maximum expected story drifts and mitigating measures could be taken if this effect becomes too large.

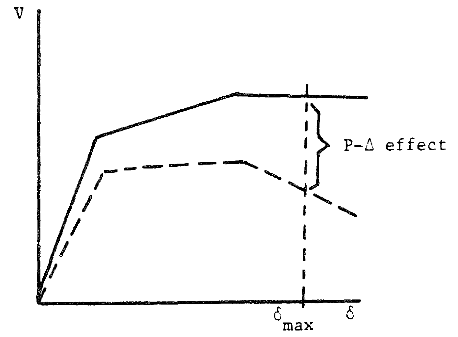


FIG. 2 - P-Δ EFFECT IN FLEXIBLE STRUCTURES

Despite these reservations to the resistance to incorporate inelastic analysis in the design process, the fact that the proposed provisions address the issue of maximum force demand will lead to considerable design improvements. In the existing provisions this demand is considered only in beam-to-column connections of ductile moment resisting frames where the connection strength must be sufficient to resist the bending strength of the connected beam. In the proposed provisions the force demand must be considered also in beam-to-column connections of ordinary moment frames and in brace connections of braced frames. The latter may require a considerable increase in connection strength compared to present practice.

The impact of design for maximum force demand may be largest in vertical load carrying columns. Particularly in slender cross-braced frames this may lead to a considerable increase in column sizes. For such frames the story shear capacities are often much larger than the design story shear forces since brace size is governed by compression and tension capacity exceeds the design force considerably. These large story shears cause overturning moments that must be resisted by the columns in addition to gravity loads. In the existing provisions only overturning moments due to 1.25 times the design story shear forces need to be considered whereas in the proposed provisions the column axial strength must be sufficient to resist gravity loads plus overturning due to the smaller of the story shear capacities or $3R_w/8$ times the seismic design forces.

The second concept of the design philosophy is to provide ductility through detailing requirements in the regions in which inelastic deformations are expected. This concept is not novel but has led to additional provisions for beams and joints in moment resisting frames, bracing members in braced frames, and link beams in eccentrically braced frames. A few of the proposed provisions are discussed in the following sections.

SPECIAL MOMENT RESISTING SPACE FRAMES (SMRSFs)

The proposed provisions for these frames are similar to the existing provisions for ductile moment resisting frames. Some differences exist in compactness and lateral bracing requirements for flexural elements (not discussed here) beam-to-column connection design requirements (see Popov 1986), and continuity plate design (see Popov 1986). Column design is affected by the previously discussed column axial strength requirement (design for maximum force demand) and a new simplified strong column-weak girder concept that must be followed if under any design load combination the axial stress in the column exceeds 40% of the yield stress.

The major change in the seismic behaviour of special moment resisting frame structures is believed to result from a new provision on beam-column joint shear strength. This new provision will also have an economic impact since it will result in considerably reduced demands for joint doubler plates. Both the existing and proposed shear design provisions are illustrated in Fig. 3.

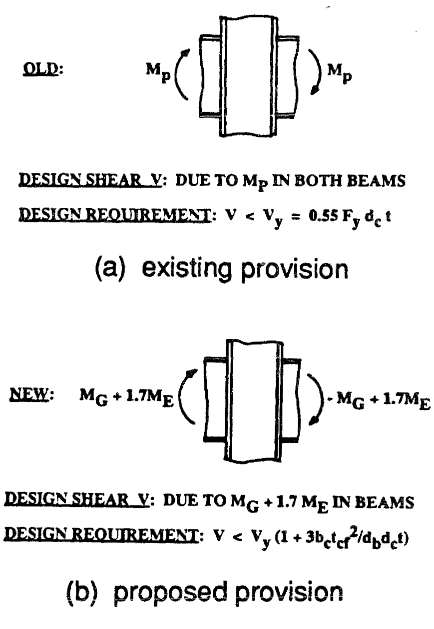


FIG. 3 - SHEAR DESIGN CRITERIA FOR BEAM-COLUMN JOINTS

Presently it is recommended that joint panel zones have sufficient shear strength to permit development of the plastic moment capacity in both beams framing into a joint. This provision is deemed to be too restrictive for several reasons. Firstly, it often necessitates the addition of very thick doubler plates and the restrained welding needed to connect these thick plates to the column may lead to severe distortions of the column flanges. Secondly, joint panel zones have been proven to be very ductile elements when subjected to large cyclic shear deformations (see Fig. 4). Thirdly, the actual shear strength of a beam-column joint may be considerably larger than that

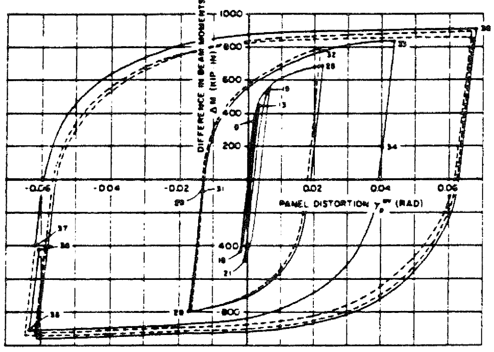


FIG. 4 - HYSTERETIC BEHAVIOUR OF A JOINT IN SHEAR (KRAWINKLER, BERTERO, POPOV 1971)

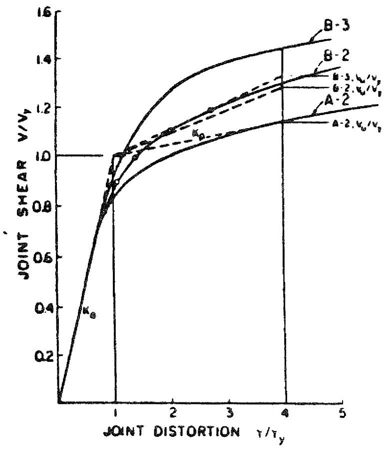


FIG. 5 - JOINT SHEAR FORCE - DISTORTION DIAGRAMS (KRAWINKLER 1978)

predicted by the AISC equation $V_y = 0.55\ f_y d_c t$, particularly for joints in columns with thick flanges (see Fig. 5). All three reasons justify a relaxation of the presently employed design provision.

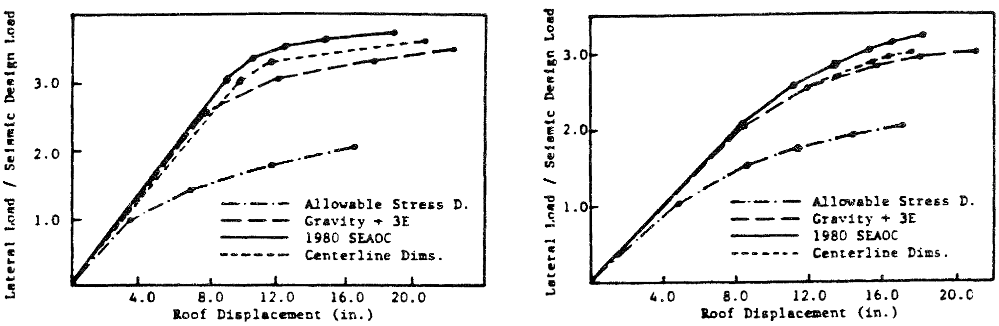
The new provision states (see Fig. 3 (b)) that a joint needs to have only sufficient shear strength to resist gravity load moments plus 1.7 times the code seismic design moments. In addition, the joint shear strength may be computed from a more refined equation that includes the beneficial effects of column flanges on the joint shear strength. This equation (shown in Fig. 3(b)) is based on a joint model in which the flexural resistance of the column flanges is represented by springs at the four joint corners (Krawinkler 1978). Experiments have shown that the AISC shear strength equation (V_y in Fig. 3(a)) predicts gross yielding in the panel zone reasonably well but that joints exhibit considerable reserve strength beyond panel zone yielding if the panel zone is bounded by thick column flanges. This is shown in the solid lines in Fig. 5 which represent experimental results for three different joint configurations. The strength values V_u predicted from the new equation, which may be associated with considerable inelastc joint distortions, are also shown in this figure.

The impact of this new provision on design, fabrication, and seismic performance may be very large. In a series of typical structures designed by the writer it was found that in many cases no doubler plates were required in joints, and when they were required their thicknesses were small compared to those obtained from present design practice.

The consequence of the new provision on seismic response may be a drastic relocation of inelastic regions in frame structures. With the existing provision most of the inelastic deformations are expected to be concentrated in plastic hinge regions of beams. With the proposed provision it is quite likely that most of the inelastic deformations will be shear distortions in joints. This can be illustrated with the following examples. A few years ago the writer performed a joint parameter study on two typical structures, one a seven story perimeter frame structure and the other a ten story all moment resisting frame structure. These structures were subjected to inelastic static and dynamic analyses, in the latter case using an amplified Taft record as input motion. A few results of this study are shown in Figs. 6 to 8.

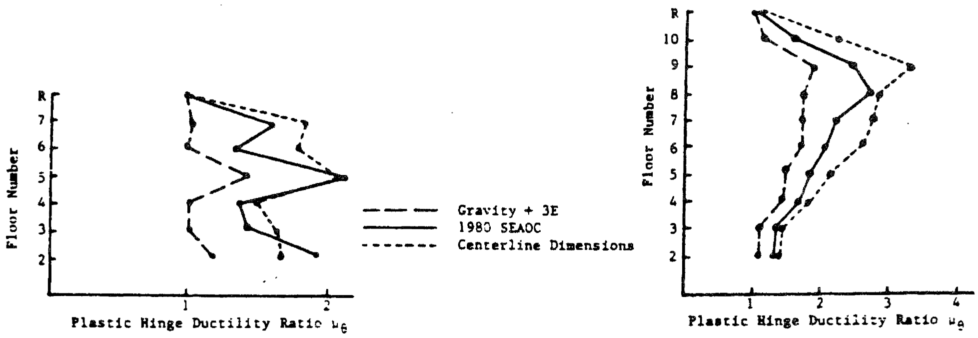
A redesign of these structures according to the proposed provision has shown that the differences between using allowable stress design (for ordinary moment frames) and the proposed provision are small. Thus, the results shown in Fig. 6 to 8 for the "Allowable Stress Design" analyses are similar to those expected from the use of the proposed provision.

A comparison of results between the "Allowable Stress Design" and "1980 SEAOC" (existing provisions) analyses illustrates the effects of the proposed provision on structure strength and seismic ductility demands in beams and joints. Figure 6 shows the results of static analysis using a triangular lateral load pattern. It is evident that the proposed provision will lead to a considerable decrease in structure strength because of the weak joints that yield before the beams attain their plastic moment capacities. Figure 7 shows no results for beam rotation ductility ratios for "Allowable Stress Design" because in none of the beams a plastic hinge was developed in the seismic analysis. Similar results are expected with joints designed according to the proposed provision. Figure 8 shows that the ductility demands in the "Allowable Stress Design" joints can be substantial although the sample structures did not undergo very large inelastic displacements in the postulated earthquake. It is interesting to note, however, that the structures with the weak Allowable Stress Design joints did not undergo larger displacements than the structures with the strong 1980 SEAOC joints.



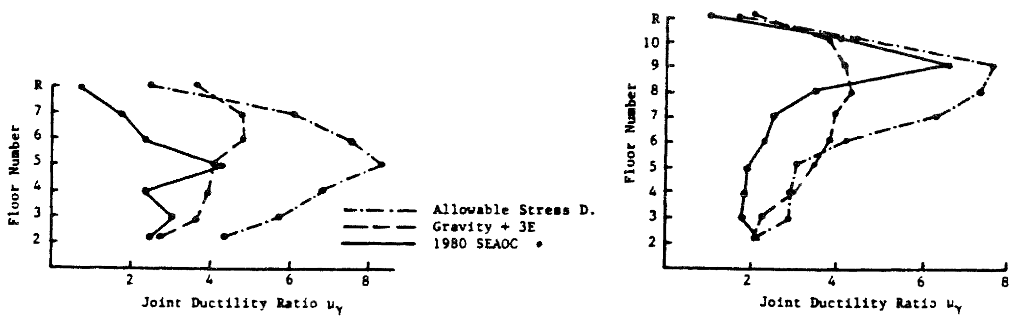
(a) Seven Story Frame (b) Ten Story Frame

FIG. 6 - LATERAL LOAD VERSUS ROOF DISPLACEMENT DIAGRAMS



(a) Seven Story Frame (b) Ten Story Frame

FIG. 7 - BEAM ROTATION DUCTILITY RATIOS



(a) Seven Story Frame (b) Ten Story Frame

FIG. 8 - JOINT DISTORTION DUCTILITY RATIOS

The drastic changes in seismic frame behaviour caused by the proposed provision may necessitate a rethinking of design concepts for SMRSFs. Most of us are accustomed to visualize energy dissipation in an earthquake as beam hinging, particularly if the strong column - weak girder concept is followed. The beam hinge concept as well as the strong column - weak girder concept may lose their importance if the joints are the weak links in the structure. If a plastic hinge cannot develop in the beam, do we need to be overly concerned about the compactness of beam sections and lateral bracing of beams? If a plastic hinge cannot develop in the column why then do we need the strong column - weak girder concept?

There are also problems that may be caused by joints that are very weak in shear. If these joints have to account for all the energy dissipation through inelastic deformations they will have to undergo very large inelastic shear distortions in a severe earthquake. This may cause problems outside the joint at the corners where beam flanges are usually welded to the column flanges. In past experiments (Krawinkler et al. 1971) fracture at these welds was caused by high curvatures at the joint corners. Also, as Fig. 6 shows, the lateral resistance of frames may be reduced considerably by weak joints. This will lead to an increase in the P-Δ effect that may adversely affect the stability of the structure.

For these reasons the writer believes that the shear strength requirements for joints have been relaxed too much. It can easily be shown that for joints in columns with thick flanges the proposed provision requires a smaller shear resistance than that obtained from allowable stress design. More often than not, joints will be so weak that plastic hinges in beams cannot be developed. Would it not be better to share inelastic deformations between beams and joints in a more balanced manner? This could be achieved by increasing the shear force that must be resisted by a joint. Referring to Fig. 3(b), the writer would recommend that the design shear force be that caused by gravity load moments plus at least two or preferably three times the seismic design moments. The latter was one of the options used in the analysis study whose results are shown in Figs. 6 to 8.

BRACED FRAMES

More than any other structural system, the proposed provisions will affect the design of braced frames in comparison to present practice. Presently, elements and connections of the bracing system are designed for 1.25 times the seismic design forces, without regard to relative member and connection strength. In the proposed provisions the braces, despite their relatively poor post-buckling performance, are considered to be the energy dissipating elements in which inelastic deformations should be concentrated. To achieve this objective all other elements, i.e. columns, brace connections, and preferably the horizontal elements of the bracing system (beams), should be designed for the maximum force demand as was discussed earlier in this paper.

The demand on braces to perform the function of energy dissipating elements made it necessary to place certain restrictions on brace configurations. Because of the poor post-buckling behaviour of axially loaded elements it was deemed necessary to require the presence of tension braces in every braced frame unit and in both directions of loading. In addition, slenderness limitations of $L/r < 720/\sqrt{F_y}$ are placed on all bracing members. The argument for this provision is that braces with small L/r will dissipate more energy because in the post-buckling range they will undergo cyclic inelastic bending which slender braces will not. Very slender braces have almost no stiffness when they straighten out from a buckled configuration and pick up stiffness rapidly once they are straightened. For a braced frame this rapid increase in stiffness may cause impact loading under dynamic excitation. This impact may overstress the brace or more likely will cause failure at the brace connection.

Unfortunately, the choice of a brace with small L/r is not always ideal either. Such a brace will undergo cyclic inelastic bending in the post-buckling range and the curvatures associated with this bending may be large enough to cause local buckling. This local buckling may be severe and may lead to localized kinking of plate elements which in turn may cause crack propagation and fracture. Such fractures have been observed rather early in tests of tubular bracing members. There is an evident need to look more carefully at the width-thickness limitations for bracing members, in particular tubular members. Also, heavy bracing members may cause large overturning moments and consequently large axial forces in the supporting columns.

The proposed provisions call also for a cyclic strength reduction factor to be applied to the code allowable compressive stress of braces. This factor is given as $1/[1 + (KL/r/2C_c)]$ and corresponds to the application of a seismic load multiplier that varies from 1.0 for $KL/r = 0$ to 1.5 for $KL/r = C_c$. Thus, slender braces get penalized heavily.

The argument made for this provision is that under load reversals the compressive strength of slender braces deteriorates faster than that of stocky braces. In the writer's opinion this observation is not always evident and it remains to be seen whether this provision is necessary and beneficial to the seismic behaviour. All this time the only evident consequences are a complication of the member design process and an increase in story shear capacity which in turn will increase the axial force demand on the columns.

The design of brace connections must be based now on the previously discussed maximum force demand which is given by the smaller of the yield strength of the brace and $3R_w/8$ times the seismic design load. In addition, for bolted connections fracture across a bolt line (net section) should be avoided in order to develop this maximum force in the brace. The force that can be transferred across a net section may be estimated as $A_e/F_u/1.2$, where the factor 1.2 is included because fracture at the net section may occur at a stress smaller than the tensile strength F_u because of stress concentrations at bolt holes. Relating this net section capacity to the maximum force demand results in a provision for the required effective net section (A_e) at bolt holes. For instance, if the maximum force demand is equal to the brace yield strength A_gF_y, the ratio A_e/A_g must be at least $1.2F_y/F_u$. This provision may necessitate reinforcement of the brace at the net section.

Frames with chevron bracing are given special attention in the proposed provisions because of two problems peculiar to these bracing systems. Firstly, once one of the braces buckle, large vertical deflections will occur in the floor system at the joint within the beam span (see Fig. 9). Secondly, the post-buckling strength of the bracing system may deteriorate significantly because the tensile force that can be developed in the tension brace will exceed the decreasing post-buckling strength of the compression brace

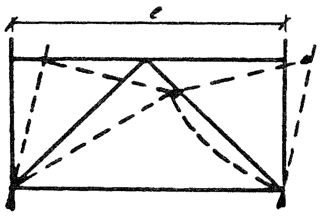


FIG. 9 - CHEVRON BRACING IN POST-BUCKLING STAGE

only by the additional force caused by hinging of the floor beams. This additional force will be small unless the beam is of substantial size and has a moment connection to the column. Also, once a brace buckles it will not straighten out fully under load reversal, the opposite brace will buckle at load reversal and the story shear resistance of the bracing system will be governed by the post-buckling behaviour of two opposite braces. The consequences will be a deteriorating story shear-story drift relationship as shown in Fig. 10. The only reason why the relationship shown in this figure does not exhibit more rapid deterioration is that in the test structure, from which this relationship was obtained, the chevron bracing was surrounded by a strong moment resisting frame that provided a considerable portion of the shear resistance once the bracing system had buckled.

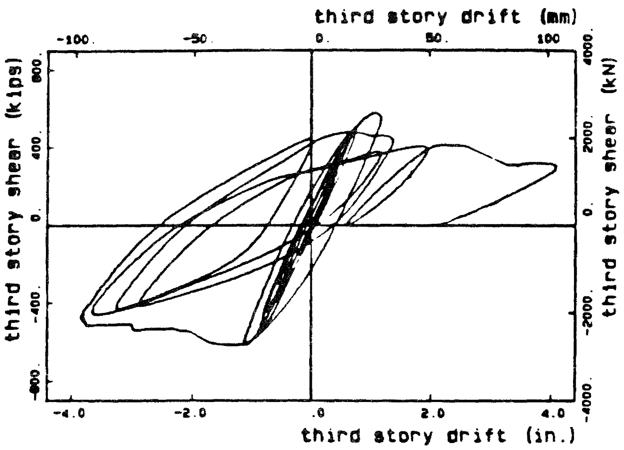


FIG. 10 - STORY SHEAR-STORY DRIFT DIAGRAM FOR A FRAME STRUCTURE WITH CHEVRON BRACING (WALLACE AND KRAWINKLER 1985)

A number of options were considered for this type of frame, ranging from prohibition, to more stringent limitations on slenderness ratios, to increased design load levels. The third option was finally selected and the proposed provisions state that these frames have to be designed for 1.5 times the seismic design loads of other braced frames. This appears to be a large increase but in the opinion of several committee members is barely adequate. Here the previously discussed cyclic strength reduction factor is certainly helpful since it will lead to a substantial further increase in lateral load resistance if slender braces are used.

CONCLUDING REMARKS

The proposed SEAOC provisions are much more detailed than the existing ones. In general they are more stringent with a few exceptions as, for instance, the shear design provision for beam-column joints. A concern is that in certain aspects the new provisions may be too complex and may cause difficulties in implementation. The new provisions address all important issues of seismic steel design and, if implemented properly, should improve the seismic behaviour of steel structures considerably.

This paper has addressed only a few of the new provisions. But those are the ones that are believed to have the greatest impact on future designs. Not discussed here are eccentrically braced frames which received considerable attention in the new provisions.

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