IMPROVING THE SEISMIC RESPONSE OF BRACED FRAMES

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SYNOPSIS

The energy absorbing capacity and ductility of steel braced frames can be increased by the introduction of bending stresses through eccentric connections. The benefits of stiff elements to minimize drift with its consequent damage and increased stability can be combined with the ductility of the moment frame in a very economical construction. Design procedures are suggested.

Traditionally, the design of braced frame structures of structural steel has been similar in approach to the design of a truss. Columns act as chord members of the truss and the horizontal floor beams act as the vertical web members of the truss. Both columns and beams must carry the vertical loads, which in many braced frame structures is the major part of the load. Diagonal members, either X-bracing, K-bracing or other similar arrangements, as shown in Figure 1, are then added to the rectangular grid of columns and beams. Normally, the diagonal members are located concentrically at the joints to eliminate bending stresses and so achieve the most efficient use of the material.

Although such a system can easily resist equivalent static lateral forces specified in most building codes, and can do so with less material and labor than other alternate systems, its performance under major earthquake loadings that require inelastic action may be questioned. The inelastic action will generally be confined to yielding or buckling of the relatively slender diagonal members. These members are usually of minimal area since they carry little or no vertical load stress, and the energy absorbing capacity of these members and consequently of the entire structure can be very limited. In order to partially allow for this effect, some building codes such as the 1976 Uniform Building Code(1) and the Structural Engineers Association of California 1974 Recommendations (2) require that the design for these members shall be for 1.25 times the forces as determined for the building. This only partially compensates for the lack of ductility and energy absorption for the system.

The philosophy of design of braced frames as described here purposely introduces flexural stresses in beams to increase the energy absorbing capability of the system. This can be achieved by locating diagonal members with significant eccentricities, in the beam-column joint as shown in Figure 2 or providing stiff moment resisting beam spans without bracing between traditionally braced bays as shown in Figure 3.

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Considering the method illustrated in Figure 2, various arrangements of the diagonal bracing pattern can be adopted, depending on the requirements for windows, doors, or utility openings in the wall. All arrangements have one common factor: The diagonals are eccentrically connected either at the beam column joint or some place along the beam.

From tests on steel joints and beam column assemblages[3], it has been found that it takes a length along the beam of about the depth of the beam to form a hinge. Since the eccentricity of the diagonal to the column creates a restrained condition at each end of the eccentricity with a point of inflection between, the diagonal should be located at least twice the member depth from the column face as shown in Figure 2(b) and two or three times the depth of the beam between diagonals as shown in Figure 2(c). The design procedure should be approximately as follows. From the vertical loadings and the lateral earthquake loadings, the member axial stresses, bending moments and shears are determined and preliminary member sizes are calculated in accordance with local code requirements. The effect of the eccentric bending moment must be included in the beam sizing and the effect of the fixity moment of the beam where it joins the column should be included in the column sizing. These will be the minimum sizes to satisfy code requirements or project criteria. Since the energy absorption will be determined by hinging of the beam, it must be the element that reaches yield first. From the preliminary beam size, its plastic hinging capacity and the dimensions of the eccentrictiy a maximum beam shear can be calculated. The beam web, if not strong enough to carry that shear, should be reinforced so that yielding will occur in bending as a hinge and not in shear. From this shear and the moment capacity of the beam, the maximum diagonal stress can be determined and the diagonal member size increased so that it will not buckle or yield in tension before the beam hinge forms. The diagonal member should be chosen so that it also can act in a ductile manner without brittle failure. While theoretically this may not be necessary, in actuality many of the factors in the system are somewhat unknown at present - actual hinging capacity of beam, fixity of diagonal ends due to rotation and bending of beam, etc. The shear in the beam and the plastic hinge moment of the beam at the face of the column must now be introduced into the design of the column so that the column will not fail even where the maximum plastic moment capacity of the beam is developed.

Not only is it essential to consider the energy absorbing capacity of the system, but it is also necessary to provide diagonal bracing members which will perform in a ductile manner. Traditional bracing systems of angles or channels with bolted connections to gusset plates lack that ductility for tension loadings since the reduced section at the bolt holes provides a stress riser or notch effect. These members traditionally fail prematurely without inelastic performance. Connections and details should be such that ductile performance is possible should the diagonal members exceed their elastic capacity. Examples of proper connections are welded connections exceeding the member capacity, upset threads for rods, etc.

This procedure effectively forces failure into the beam, protecting the column and preventing buckling of the diagonals which in turn would greatly damage partitions that enclose the bracing wall. After a major earthquake the columns should be sound and the repairs made to the beams which are damaged - probably the easiest member of the system to repair.

The same procedures can be used where it is more convenient to introduce the eccentricity somewhere along the beam as in Figure 2(c) to accommodate required openings.

Another method of introducing energy absorption and ductility into braced frames is illustrated in Figure 3. More traditional systems of braced frames that use concentric connections can be combined with unbraced bays if very heavy and stiff girders can be used. The girders must be stiff enough to materially reduce the overturning forces "A" in Figure 3 and strong enough to develop plastic hinges of enough strength to relieve the column stresses in the design maximum earthquake. In principle, this may be similar to the design of coupled shear walls as described by Pauley(4). This system is not as efficient as the one described earlier and will be difficult to achieve in low buildings. It may be combined with the eccentric joint shear to meet the requirements of some architectural arrangements.

The principles discussed here have been employed in the design of two steel frame structures in San Francisco, one 28 stories and the other 35 stories in height with very economical results. Unfortunately, neither was finally constructed because of other non-structural reasons. The concept has provoked some research at the University of California Earthquake Engineering Research Center where tests are being completed. A preliminary report on the progress of these tests has been presented(5). They indicate that the concept is very promising.

REFERENCES

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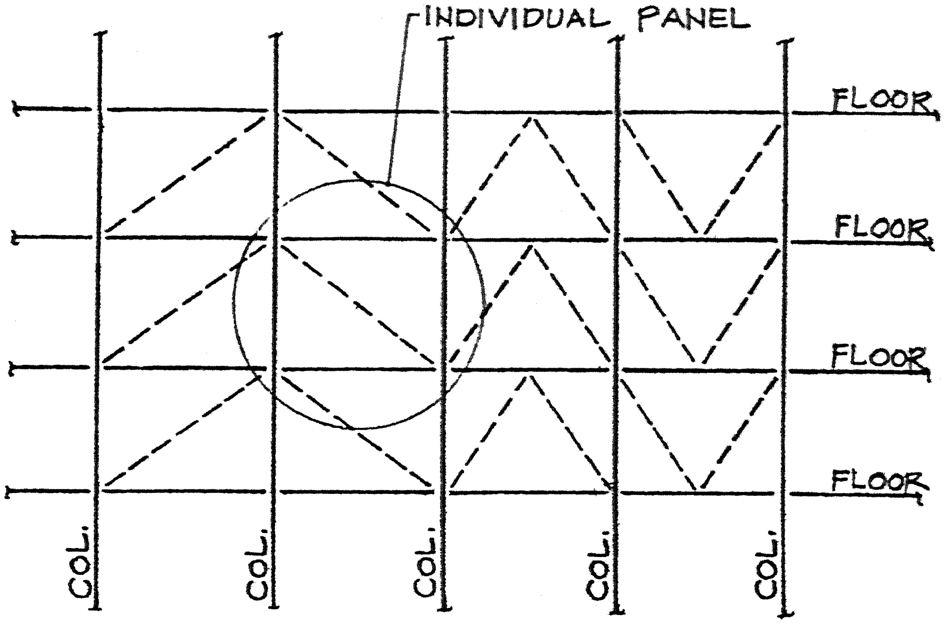
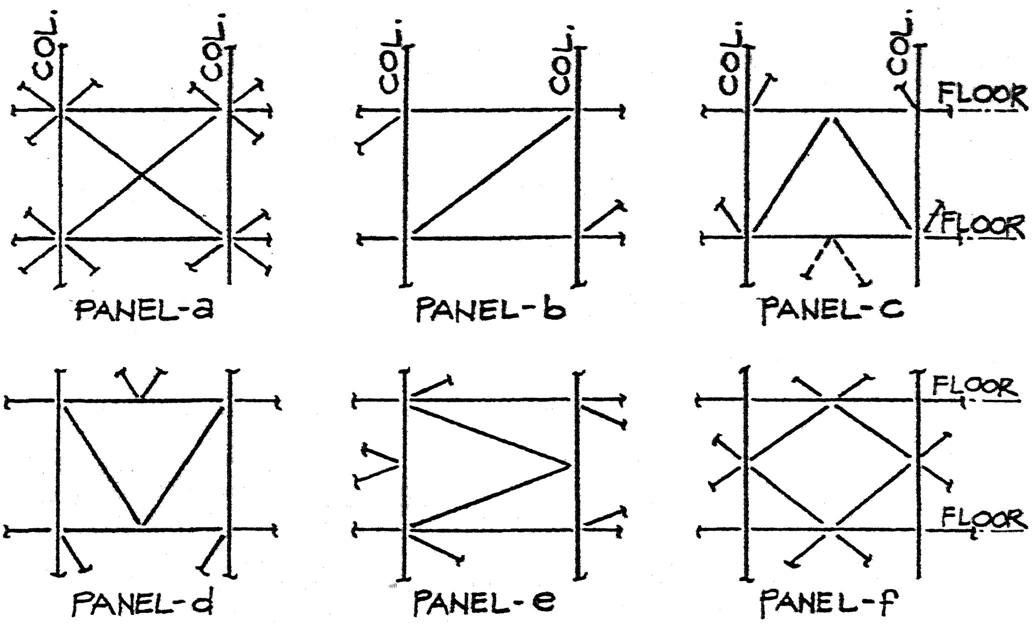
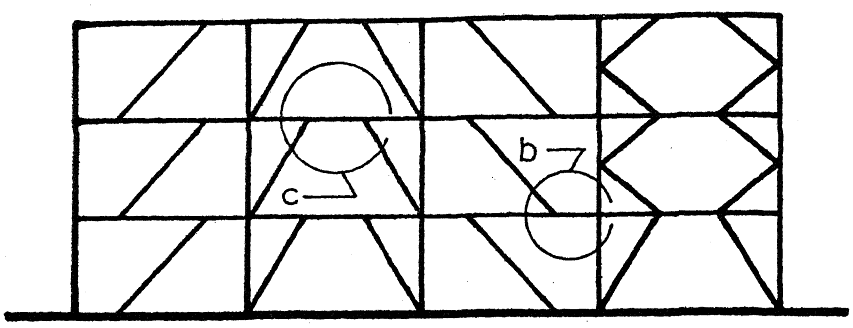


PORTION OF BRACING BENT.
(a)



FRAMING VARIATIONS OF INDIVIDUAL PANELS–CONCENTRIC
(b)

FIGURE 1



(a.) ELEVATION OF BRACING BENT SHOWING ALTERNATE ARRANGEMENTS OF DIAGONAL MEMBERS.

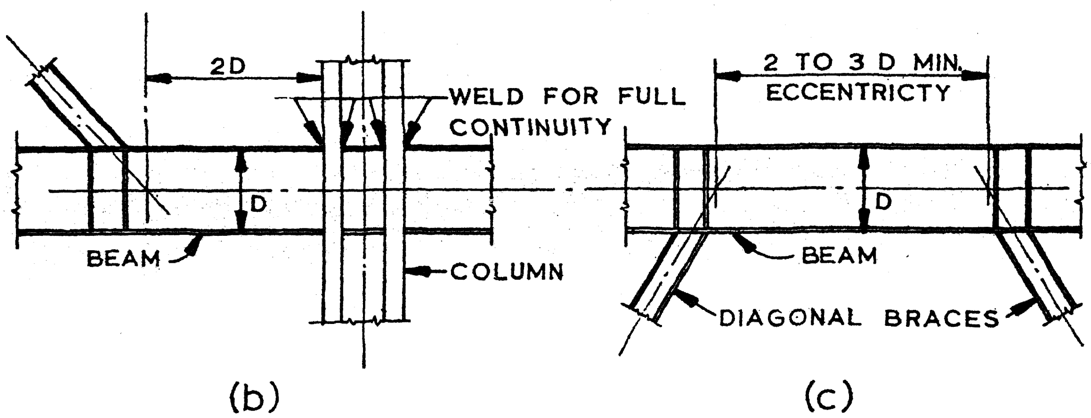


FIGURE 2

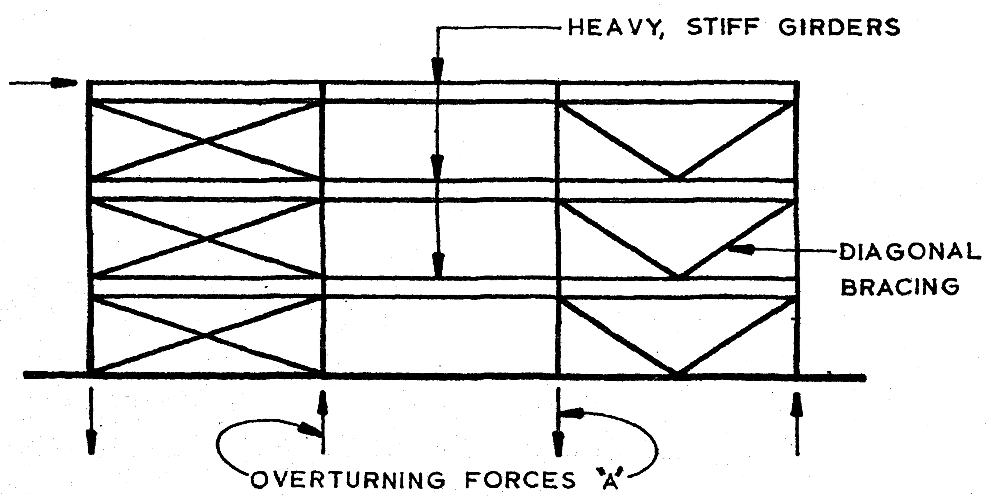


FIGURE 3