Section E

CONCENTRICALLY BRACED FRAMES

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This paper is the result of deliberations of the Society's Study Group for the Seismic Design of STEEL STRUCTURES

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2. INTRODUCTION

Concentrically braced frames have been a traditional form of construction for many years. Diagonal bracing provides an efficient and economical way of resisting lateral loads. Braced frames tend to be stiffer than moment-resisting frames, and combinations of these two systems are also possible and commonly used overseas. A great many arrangements of bracing are possible and only a few common types are discussed here.

3. PREVIOUS RESEARCH

Some cross-braced frames have been badly damaged during strong earthquakes, such as those described by Tanaka et al (1). Many other failures have also been reported, but it would appear that the principal causes are designing to too low a value of lateral force, and inadequate design of connections. Adherance to the philosophy of current N.Z. design codes codes should prevent these failures. In particular a relatively high level of seismic force is required and capacity design is required to be carried out. (2,3,4).

Wakabayashi et al (5) have given an example of the typically punched hysteresis loops that are obtained with slender diagonal members under cyclic loading. The pinched loops are undesirable, because less energy is absorbed and so greater response will occur. In addition with every increasing inelastic extension of the diagonals, the frame can sway freely until the brace pulls tight under tension, possibly imparting an impact load to the structure. Where the frame has negligible bending stiffness, because of essentially pinned joints, once yielding occurs in a particular level, it is likely that almost all plastic deformations will be concentrated at that level. For these reasons N.Z. codes have specified high levels of design force for braced frames and restricted the number of storeys.

Popov and Black (6) studied the inelastic buckling of steel struts under reversing loading and noted the low stiffness when the member straightened out under tensile loading from its buckled condition. The strength in compression was found to be considerably diminished after the first cycle, either due to prior tensile yielding or residual curvature remaining after prior buckling. They showed that although struts with a slenderness ratio KL/r of 120 gave pinched hysteresis loops, those with a slenderness of 80 were less pinched, and those with a slenderness of 40 provided nearly full hysteresis loops. In addition they found that struts with the same slenderness, and different end conditions had slightly differently shaped loops; but these had essentially the same enclosed area. The overall buckling behaviour was largely determined by the effective slenderness ratio, independant of the sectional geometry, provided this complied with the AISC specification (7). The compressive strength may be considerably reduced, both after the initial tensile yielding because of a reduction in modulus due to the Bauschinger effect, and after previous buckling due to residual curvature.

The tests of Black et al (8) showed that for struts with a slenderness of 35 only 30% reduction in compressive strength occurred after 16 cycles, whereas with a slenderness of 90 a reduction of 70% occurred after 16 cycles.

Nordenson (9) has given the background to the provision for the earthquake-resistant design of concentrically-braced steel frames for possible inclusion in the SEAOC design rules (10). He suggested that a concentrically X-braced frame with diagonals that have a slenderness of 80 should provide hysteresis loops that enclose about 75% of the area enclosed by the loops of a moment-resisting frame of equivalent strength. He recommended that diagonal braces which act as primary load-resisting elements must have a KL/r less than $1420/\sqrt{F_y}$ except where the strength of the brace exceeds 2 to 3.75 times the storey design shear in which case KL/r should not exceed $2130/\sqrt{F_y}$.

Aoyama (11) has indicated that mild-steel braces having a slenderness less than 57.1 are regarded as having the same ductility as moment-resisting frames.

Jain et al (12,13) studied the behav-

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iour of mild-steel struts and stated that with a slenderness less than 40, they would retain more than 70% of their original buckling strength.

The behaviour of V-braced frames (often called K-braced frames) may be significantly different from X-braced frames. The diagonals form a V or inverted V, which means that the storey shear strength is largely restricted to the strength of the diagonals in compression, unless the beam has significant bending strength. Hence the seismic resistance of frame with negligible bending strength and slender V braces is likely to be vastly inferior to the same frame with pairs of braces between beam-column joints. V-braced moment-resisting frames are nevertheless a popular form of construction. A design process based on the elastic analysis of code lateral forces will tend to give smaller beam sizes for V-braced frames compared to X-braced frames.

Cheng (14) has shown that V-braced systems give the lightest design. Where the diagonals have a slenderness of 40 or less the behaviour should be quite good, but when that slenderness is exceeded the compressive strength of the diagonal (and hence the shear strength of the frame) at high lateral displacements is less than 30% of the initial compressive strength of the diagonal.

Toyama (15) has presented the results of experimental and theoretical studies into the response of frames stiffened by bent bracing. This appears to have some advantages, by eliminating the sudden deterioration of strength due to buckling, and thereby giving well-formed hysteresis loops which do not degrade.

Little research has been published on the capacity design of concentrically braced frames. Usually overseas codes require the connections of diagonal braces to be designed for a higher design force for the brace. This does not necessarily ensure yielding in the member before failure of the connection. It is understood that the SEAOC code, when revised will indirectly include significant overstrength requirements on the bracing connections above the specified member strength.

Astaneh-Asl et al (16,17,18) have shown the importance of detailing the end conditions of braces to accommodate the assumed boundary condition. Connections designed to act as pins should have sufficient flexibility to prevent fracture, and those assumed to be fixed should allow the formation of plastic hinges. Where X-braces are fastened together at their intersection, to provide mutual restraint, this connection should allow for the plastic deformation that may occur. Where gusset plates are used at the ends of double angle diagonals, it was observed that in post-buckling deformations, the rotation of the plastic hinges in the gusset plate took place about an axis normal to the axis of the member. It was found that the portion of the gusset plate connected to the angles had to be allowed to rotate freely about the axis of the plastic hinges, otherwise early fracture occurred in the gusset plates. It was found that a free length of gusset plate beyond the angles equal in length to at least twice the thickness of the gusset plate should be provided. They found that the stitch welding required by the AISC specification was inadequate for double angles. They recommended that the stitch welds should be designed for an eccentric force equal to half the yield capacity of a single angle. Where double angles were connected by bolts they recommended that they should be designed as friction-type connections.

Goel and Hanson (19) have studied the behaviour of moment-resisting frames which have been stiffened by light X bracing. They found that the braced frames showed reduced lateral displacements and inelastic activity in the columns and girders, while the axial forces in the columns are substantially increased, as compared to the corresponding unbraced frames.

Anderson (20) investigated the behaviour of three 10-storey three-bay structures, having moment-resisting frames and V-bracing, designed to UBC code (21) requirements. It was found that where all floors were braced, lateral displacements were effectively controlled. The addition of a truss across the top floor was effective in reducing the inelastic deformation of the columns. These structures were designed so that the frame, without bracing, was capable of resisting 25% of the code loading.

Goel (22) also studied V-braced moment-resisting frames under combined horizontal and vertical ground motion. It was found that the vertical component of ground motion could cause increases in the column axial forces and in the ductility requirements for columns, beams and bracing members.

4. DESIGN RULES

4.1 Arrangement of Braces

Braces shall be placed at all levels in all frames assumed to resist seismic action. Braces shall be arranged in pairs, so that for every brace acting in tension at a particular instance, there will be another brace acting in compression.

4.2 Number of Mass Levels

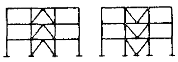
For frames with negligible moment-resistance there shall be no more than three mass levels supported by this form of construction.

For moment-resisting braced frames designed and detailed so that they would have sufficient strength to resist at least 25% of the seismic forces specified by NZS 4203, without the bracing, there may be no more than five mass levels supported by this form of construction, provided adequate special studies are made to verify the seismic performance.

4.3 Strength of Members

The members shall be designed in accordance with NZS 4203 or SDPP (4) except that the S and μ factors respectively

TABLE 1

	Bracing Slenderness	$\frac{KL}{r}\sqrt{\frac{F_y}{250}}$	< 40		41-80		81-135	
		Number of Storeys	S	μ	S	μ	S	μ
	Pairs of braces between beam-column joints	1	1.4	5.0	1.7	3.9	2.0	3.0
		2	1.5	4.5	1.9	3.5	2.5	2.4
		3	1.6	4.0	2.0	3.0	3.0	1.8
	V-bracing	1	1.8	3.7	2.5	2.4	4.0	1.5
		2	2.0	3.0	3.0	1.8	5.0	1.2
		3	2.2	2.7	4.0	1.5	6.0	1.0

shall be used as given in Table 1, for frames which have negligible bending strength. Structures using threaded rods must be designed using $S = 6$ or $\mu = 1$.

Beams in V-braced frames shall be designed to carry all their tributary gravity loads, without relying on the vertical support provided by the bracing, unless the bracing is designed to carry all of the beam gravity load in addition to the forces from the lateral loads.

The time frame of this study group has not permitted the calculation of the response of braced moment-resisting frames when designed to N.Z. codes. These are allowed in overseas codes and should perform satisfactorily if designed using factors intermediate between those for braced frames and moment-resisting frames. This form of construction should be allowed where special studies establish a satisfactory level of design force.

4.4 Capacity Design

The connections to any diagonal shall be designed using an overstrength factor of 1.35. i.e. the strength method design forces shall be 1.35 times the specified area of the member times the specified yield stress.

The members framing into any diagonal shall be designed to carry the above design force.

For the design of columns it may be assumed that only one of the diagonals from any three levels is yielding at any one time. i.e. the force from one diagonal shall be calculated using an overstrength factor of 1.35 and the other two using the dependable strength reduction factor (currently 1.00).

4.5 Detailing

4.5.1 Cross-Sections

The width-to-thickness ratios of all members shall comply with Table 1 of Section C - Beam Design (23). Diagonals designed with S of less than 2 or μ more than two shall be put in category one. Those with S between 2 to 6 or μ from 1 to 2 shall be put in category two. Those with S equal to 6 or μ equal to 1 shall be put in category three.

4.5.2 Net areas

The net area of brace members shall satisfy the following:

$A_n \geq A_g$ when $S \leq 4$ or $\mu \geq 1.5$

$A_n \geq 0.92\, A_g$ when $6 > S > 4$ NZS 4203

or $1 < \mu < 1.5$ SDPP

Threaded rods or bolts etc shall not be used where the thread area is less than the shank or gross area unless the frame is designed to respond elastically, i.e. $S = 6$ at present or $\mu = 1$.

4.5.3 Gusset plates

The gusset plate should be made long enough, so that there is a clear length of gusset plate, beyond the end of the member, equal in length to at least twice the thickness of the gusset plate.

4.5.4 Built-up members

Members shall comply with Ch. 6.7 of AS1250:1981 (24). (This already incorporates the recommendations Astaneh-Asl et al.) In addition any intermediate connections shall be designed for an eccentric force equal to half the yield strength of a single member.

The end connections and connections at the centre of X-bracing should be detailed so that a plastic hinge can form at these positions with buckling taking place about the axis with maximum slenderness.

5. FURTHER RESEARCH

Time history analyses of braced frames, which have been designed in accordance with these recommendations, when excited by various earthquake records are required to verify that the level of seismic design force gives satisfactory behaviour. These analyses would also verify the capacity design procedures.

Concentrically-braced moment-resisting frames appear to be accepted in other countries such as Japan or U.S.A. with little restriction on the number of storeys etc. Although slender bracing may have relatively poorer ductility, when carefully detailed and conservatively designed, the performance should be satisfactory and construction still economical.

For braced moment-resisting frames further research is required to determine design factors, such as the structural type factor S for NZS 4203 or the ductility capability factor μ for SDPP.

In addition further work is required to establish whether there is a need to restrict the number of storeys with the various forms of braced frames.

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7. NOTATION

A_g = the gross cross-sectional area

A_n = the net cross-sectional area

F_y = the specified yield stress MPa

K = the ratio of the effective length of a member in axial compression to its actual length

L = the actual centre-to-centre length of a member

r = the radius of gyration of a member about its minor principal axis

S = the structural type factor of NZS 4203

μ = the ductility capability factor of SDPP(4)