Section F

ECCENTRICALLY BRACED FRAMES

G.K. Sidwell*

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

1. CONTENTS

2. INTRODUCTION

This paper considers the provisions to be made when designing eccentrically braced frames to meet the philosophy of design embodied in two possible means of compliance to the NZ Loadings Code NZS 1900 Chapter 8, viz. NZS 4203: 1976 (1) and the SDPP (2) recommendations.

Eccentrically braced frames should be considered as ductile structures with the inelastic deformation and energy absorption being confined to the regions of joint eccentricity, where it is possible to ensure that these actions may take place without adversely affecting the strength and overall stability of the structure.

A capacity design approach that considers the overall collapse mechanism provides a means of determining the actions within the other members of the frames while the shear zones are yielding. Since those procedures are based on considerations of strength of members one vie another only the strength method of design is considered appropriate for these structures.

3. BACKGROUND

It has long been realised that in most structures it is uneconomic to design against the adverse effects of seismic activity and prevent any inelastic deformation. When inelastic deformation is permitted it is possible to design structures to lower force levels. The extent to which these force levels can be lowered depends upon the characteristics of the hysteresis loops in the inelastic regions.

Eccentrically braced frames seek to exploit inelastic behaviour of steel deforming in shear. Considerable research into the characteristics of I sections subject to cyclic shearing forces and eccentrically braced Z frames, has been carried out by Roeder and Popov (3), (4).

The other highly desirable characteristic of these structures is that deformations in moderate seismic events are lessened with consequential reduction in the potential for secondary damage.

While there are several possibilities for the configurations of eccentrically braced frames, this paper considers only eccentric K bracing and eccentric Z bracing where the eccentricity is in the beam elements. These are shown in figures 1 and 2.

4. REQUIREMENTS OF EXISTING CODES

4.1 NZS 4203: Code of Practice for General Structural Design and Design Loads for Buildings

Clause 4203: 1.3 specifies design load combinations for the strength method of design to be considered in the design of all components of the structure. Those of primary concern are

$$U = 1.4D + 1.7\,L_R \qquad 1.3.2.3\text{-}(1)$$

$$U = D + 1.3\,L_R + E \qquad 1.3.2.3\text{-}(4)$$

and

$$U = 0.9D + E \qquad 1.3.2.3\text{-}(5)$$

The seismic forces E are determined by loadings based on the seismic design coefficient C_d; clause 4203: 3.4.2 as C_d = CISMR where M = 0.8 for steel structures. Eccentrically braced frames are not considered in the structural classifications of table 5. However, it is recommended that $0.8 \leqslant S \leqslant 1.0$

---

* Brickell, Moss & Partners, Wellington.

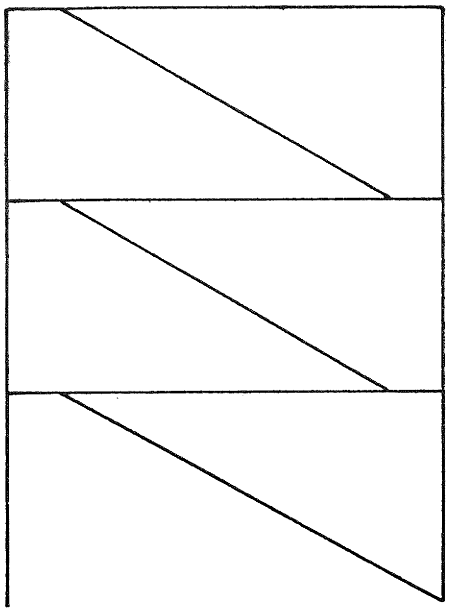


FIG 1
ECCENTRIC Z FRAME

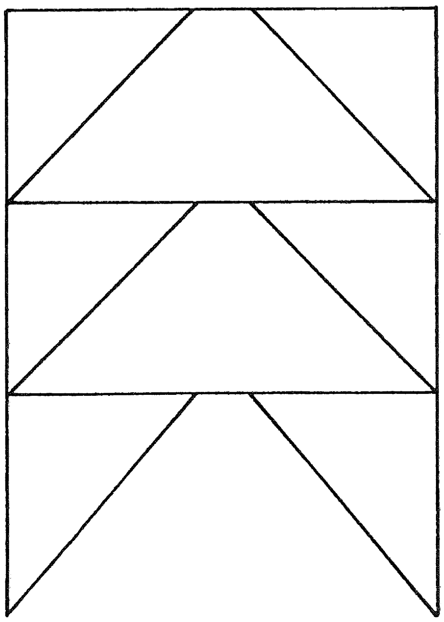


FIG 2
ECCENTRIC K FRAME

and the range can be evaluated on the basis of the number of framed bays in a manner akin to that for moment resistant frames.

4.2 Recommendations for the seismic design of petrochemical plants

Similar load combinations to those of NZS 4203 are prescribed by these recommendations (2) but the seismic coefficient is based on the considerations of a ductility factor μ.

It is recommended that this should be taken as 4 for the frame types considered in this paper.

4.3 Code for Design of Steel Structures NZS: 3404

Section 14 of this code determines the basis requirements of the strength method of design to be applied to all members for all load combinations.

Notwithstanding that the capacity analysis procedures are similar to a plastic analysis it is not considered necessary to invoke the stability provisions of clause 10.7

5. DESIGN AND DETAILING OF THE SHEAR ZONES

Since the basis for the overall design philosophy is that these zones shall yield in shear in the event of the design earthquake the shear forces provide the primary design values. Thus if P_d and V_d are the design values for the axial load and shear on the shear zone, then

$$\left(\frac{P_d}{A.F_y}\right)^2 + \left(\frac{V_d}{V_p}\right)^2 \leq 1.0 \qquad 4\text{-(A)}$$

where $V_p = 0.55.d.t_w.F_y$

This basic design relationship is derived from 3402: 12.4.1.

In order to ensure ductile behaviour Roeder and Popov recommend that the eccentricity (e) of the shear span be restricted within the following bounds -

$$\frac{2\ M_{pf}}{1.3\ V_p} \leq e \leq \frac{2\ M_{pf}}{1.1\ V_p}$$

where $M_{pf} = B.T.F_y\ (d - T)$.

The upper limit ensures that the link zone will yield in shear before flexure while the lower limit is to prevent the flanges from contributing any appreciable shear strength to the link zone (3).

To control the tendency for local buckling effects the following additional criteria shall apply:

5.1 Web stiffners shall be provided within the link zone at centres not exceeding 24 t_w.(5)(6)

5.2 The flange outstand to thickness ratio shall not exceed 7.5

Recent research by Hjelmstad, Malley and Popov (8)(9)(10) has produced evidence that the above limits may be relaxed provided detailed assessment of the member's ductility demand is made. Accordingly it is recommended that literature reviews be undertaken by any designer considering these frames as the whole subject is currently the subject of extensive research.

6. EVALUATION OF ACTIONS FOR CAPACITY DESIGN

6.1 Method of analysis

The values of the actions within the other component members of an eccentrically braced frame can be determined by a rational analysis of the collapse mechanism, the overstrength capacities of the link zones and the imposed gravitational loads. These other components of an eccentrically braced frame to be considered are:

6.1.1 Those portions of the beam elements not within the link zone

6.1.2 The diagonal braces

6.1.3 The columns

6.1.4 The connections of the frame

Figure 3 indicates the collapse configuration for an eccentric K frame. The various possible collapse mechanisms for an eccentric Z braced frame are indicated in figures 4 to 6. Figure 7 derives the condition that determines whether the beam element of an eccentric Z frame has a shear link at the top of the lower brace or the bottom of the upper brace.

6.2 Over capacity shear strength

The over capacity shear strength of

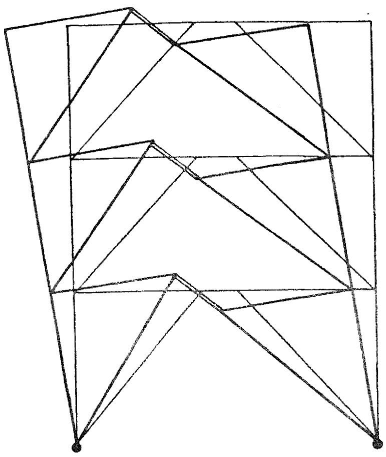


FIG 3
COLLAPSE MECHANISM ECCENTRIC K FRAME

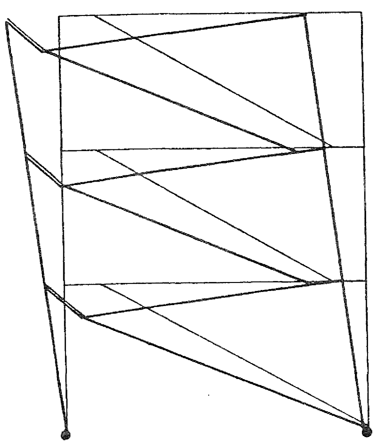


FIG 4
COLLAPSE MECHANISM DIAGONALS IN TENSION

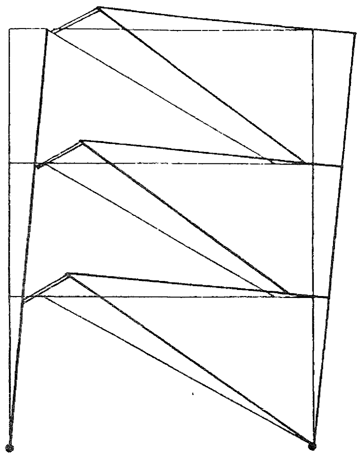


FIG 5
COLLAPSE MECHANISM-NO SIGNIFICANT VERTICAL LOADING

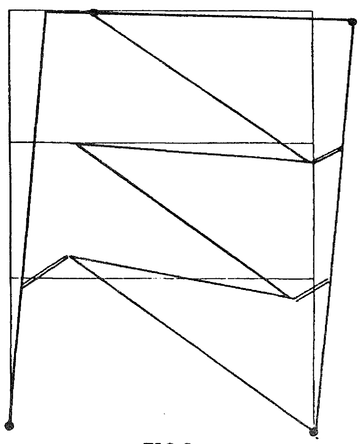


FIG 6
COLLAPSE MECHANISM - SIGNIFICANT VERTICAL LOADING

the link zones may be determined from equation 4-(A) where the actual section and material properties are substituted in the equation. The material properties used should consider the effects of strain hardening as well as the fact that the actual strength will exceed the specified yield strength by some amount.

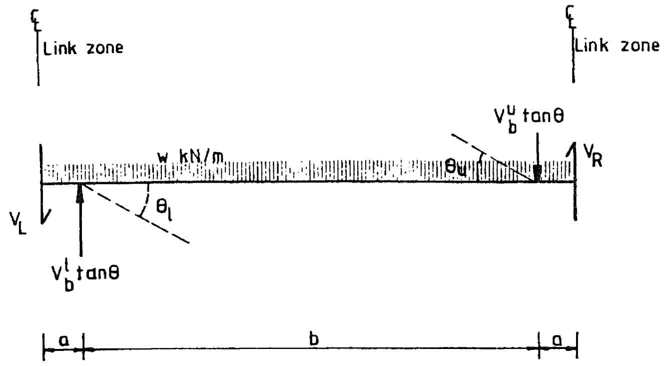
In the absence of any more reliable data it is suggested:

6.2.1 The actual material yield stress (F_{ya}) be considered to be 35 percent greater than the nominal yield stress F_y

6.2.2 The overstrength yield stress (F_{yo}) be considered to be elevated 15 percent above the actual material yield stress by the effects of strain hardening.

The effects of the shear strength of any concrete diaphragm system should be considered as a potential source of overstrength. However, it is considered that slabs of normal thicknesses will suffer extensive local damage when strain hardened strengths of the steel section is achieved that the shear strength of the slab will

EQUILIBRIUM OF BEAM ELEMENT OF ECCENTRIC Z WHEN DIAGONALS IN COMPRESSION



$$V_L = V_b^l \tan\theta_l \frac{b+a}{b+2a} - V_b^u \tan\theta_u \frac{a}{b+2a} - w\,\frac{b+2a}{2}$$

$$V_R = V_b^u \tan\theta_u \frac{b+a}{b+2a} - V_b^l \tan\theta_l \frac{a}{b+2a} + w\,\frac{b+2a}{2}$$

for the case of $\theta_u = \theta_l$
with $W = w(b+2a)$
& $V_b^l = V_b^u + \Delta V_b$

it can be shown $\tan\theta > \frac{W}{\Delta V_b}$ will result in a hinge zone at the left end

& $\tan\theta < \frac{W}{\Delta V_b}$ will result in a hinge zone at the right end

FIG 7

have deteriorated to such an extent that it can be ignored.

It is necessary to ensure the cracking strength of the diaphragm is not significant and to develop some special details in the event that it is.

7. PROTECTED ELEMENTS

7.1 Beams

Those portions of the beams that are not within the link zone are subject to moments and either axial tensions or compressions.

Since these elements are contiguous with the link zones their actual material yield stress will be the same as for those areas. Consequentially in comparing their strength with the capacity demand it is suggested that this actual yield stress should be used in determining the capabilities of the member.

In some instances it may be difficult to establish satisfactory performance of these elements without providing additional translational support and possibly reducing the imposed moment by means of a moment connection between the beam and brace. It will probably be necessary to provide lateral restraints as determined by the provisions of NZS 3404: 10.9.

In instances where a concrete diaphragm is present then clearly this has a beneficial effect in assisting the resisting of any compression loads.

7.2 Columns and braces

These elements are not contiguous with the link zones and hence may not have the same actual yield stress. Thus the assessment of their strength should be based on the nominal yield stress or a slightly conservative assessment of their probably yield stress.

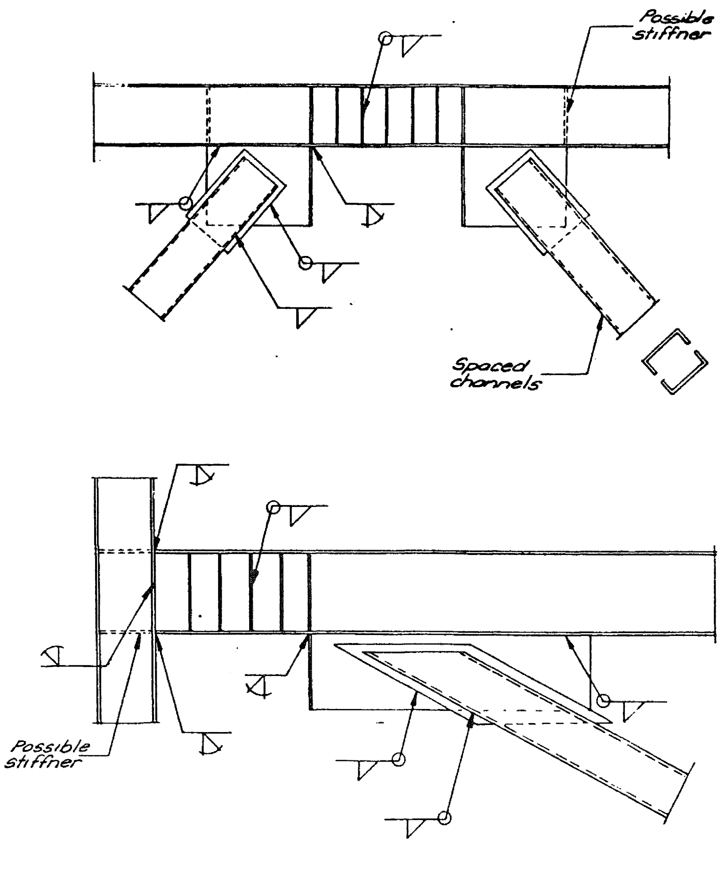
It should be noted, however, that for fairly typical brace slenderness ratios of 70-100 buckling dominates the strength of the member and that a 25 percent increase in material yield only results in an 18 percent to 10 percent increase in strength.

7.3 Connections

Roeder and Popov noted adverse performance of conventionally detailed bearing type bolted connections. They recommend that the allowable bearing stress should be reduced to F_y (4).

Considerable care must be taken to avoid eccentricity about the minor axis as the moments associated with axial loads can significantly detract from the load carrying capacity of the member.

Indicative details for the connections of these frames and the associated detailing are shown in figure 8.



INDICATIVE DETAILS

FIG 8

7.3.1 Eccentric Z beam-column

This connection is required to develop the full shear capacity of the web and the plastic moment capacity of the flanges.

Roeder and Popov (4) consider that a fully butt welded flange connection with either butt or fillet welded webs is required for this connection. An end plate connection may suffice but the very high shear forces probably preclude the use of shims and hence reduce their practicality.

7.3.2 Eccentric K beam-column

This connection needs only to connect the axial forces in the beam, brace and column to each other. Thus both welding and bolted connections can be detailed for these locations.

7.3.3 Brace-beam connection

This connection connects the axial forces in the two members and may in addition be required to transfer some moment. Both bolted and welded forms can be readily used.

8. DISCUSSION OF ECCENTRIC K AND Z FRAMES

As mentioned previously Roeder and Popov have studied extensively the eccentric Z frame and have made a recommendation that the brace should be designed for 1.5 to 1.0 times the code force. Since the beam overstrength is at least 1.35 x 1.15 = 1.55, very little margin for the effects of gravitational loads is included in their recommendation.

A capacity based analysis indicates that the effects of gravity are such that greater protection to the brace is required in order to ensure that buckling will not occur.

For normal gravitational loadings it would appear that the eccentric K frame has significant advantages over the eccentric Z when a capacity approach is required for the design. Basically this arises from the much lower gravitational shears in the shear link which consequently reduces total overstrength of the shear link. Also the ratio of axial load in the beam to the shear in the link is lower in the K frame and thus strength and stability problems with this element are reduced.

It would also appear that the failure mechanism is more predictable for the K frame. In eccentric Z frames a brace may be fused by either an upper or lower link zone. Which link actually fuses the brace depends on the gravity load on both beams as well as the variation in storey shears.

Further Popov and Roeder studies of eccentric Z frames have all been based on prototypes that have a moment connection of the beam to the column. There seems very little need for this form of connection to be used on an eccentric K frame.

9. REFERENCES

(1) NZS 4203: 1976 "General Structural Design and Design Loads for Buildings", Standards Association of New Zealand, Wellington.

(2) NZ Ministry of Works and Development, "Seismic Design of Petrochemical Plants - Volume 1: Recommendations and Volume 2: Commentary", March 1981.

(3) Roeder, Charles W. and Popov, Egor P., "Cyclic Shear Yielding of Wide-Flange Beams", Proc. ASCE, Vol. 104, No. EM4, Aug. 1978, pp. 763-780.

(4) Roeder, Charles W. and Popov, Egor P., "Eccentrically Braced Steel Frames for Earthquakes", Proc. ASCE, Vol. 104, No. ST3, March 1978, pp. 391-412.

(5) Popov, E.P., "Recent Research on Eccentrically Braced Steel Frames", SEAOC Conference, September 1981.

(6) Popov, E.P., "An Update on Eccentric Seismic Bracing", Eng. Jnl., AISC, Vol. 17, No. 3, 1980.

(7) Trahair, N.S., "The Behaviour and Design of Steel Structures", Methuen of Australia, 1977, 320 p.

(8) Hjelmstad, Keith D. and Popov, Egor P., "Cyclic Behavior and Design of Link Beams", Proc. ASCE, Jour. of Struct. Engineering, Vol. 109, No. 10, Oct. 1983, pp. 2387-2403.

(9) Hjelmstad, Keith D. and Popov, Egor P., "Characteristics of Eccentrically Braced Frames", Proc. ASCE, Jour. of Struct. Engineering, Vol. 110, No. 2, Feb. 1984, pp. 340-353.

(10) Malley, James O. and Popov, Egor, P., "Shear Links in Eccentrically Braced Frames", Proc. ASCE, Jour. of Struct. Engineering, Vol. 110, No. 9, Sept. 1984, pp. 2275-2295.

10. NOTATION

A	crossectional area of section
B	breadth of flange
C_d	Seismic Design Coefficient
D	Dead load effect
d	depth (overall flanges)
E	seismic load effect
e	eccentricity in shear link
F_y	nominal material yield stress
F_{ya}	actual yield stress
F_{yo}	overstrength yield stress
L_R	reduced live load
M	materials factor
M_{pf}	plastic moment resistance due to flange action only
P_a	design axial load
S	structural type factor
T	section flange thickness
T_w	section web thickness
V_a	design shear in link zone
V_p	plastic capacity of link zone
μ	ductility factor