Seismic design of concentrically braced frames – code comparisons

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ABSTRACT

Several codes and standards provide seismic design provisions for concentrically braced steel frames. Some of these are recent, and are based on research results produced over the last decade. The requirements for Canada, Japan, New Zealand, Europe and the USA are compared. The basic design philosophy, loading and member detailing requirements are considered. Similarities and differences are summarized, and areas needing further study are identified.

INTRODUCTION

Research and experience in earthquakes has shown that ductile response of concentrically braced steel frames is feasible in spite of two characteristics that militate against their ability to absorb energy under severe seismic loading. The low redundancy compared with that of other possible structural systems limits the potential for redistribution of load following inelastic deformation, and the fact that load is resisted principally by members subject to axial force, rather than shear or bending, increases the possibility of member instability controlling the response under severe load. In recent years rational approaches to design for ductile behaviour have been incorporated in several codes and standards. In view of the appearance of these provisions for the first time, it is the objective of this paper to summarize and compare the existing requirements for concentrically braced frames and to determine the extent of uniformity and to comment on significant differences. The codes and standards considered are those of Canada (NBCC 1990 and CSA 1989), the USA (UBC 1988), New Zealand (NZS 1984, 1989), Europe (Eurocode 8, 1988 (draft)) and Japan (BSL, 1981).

Prior to this new generation of codes, the design of concentrically braced frames (CBF) for seismic load in Europe and North America was not subject to many restrictive requirements. This was particularly true of detailed member design, there being for example no limitations on slenderness ratio. The principal

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effect of the code provisions was to control the member design loads in a variety of ways. The low fundamental period of the stiff CBF system naturally leads to higher response loads, and high values of the structural system coefficients were assigned due to the limited ductility of CBF. These coefficients usually considered some structural system effects, such as the difference between tension-only and tension-compression bracing. In addition, higher loads for brace members were recommended in some codes to enhance the elastic response and reduce the ductility demand (SEAOC 1976).

Research on the physical behaviour of bracing members (for example the work of Jain, Goel and Hanson (1980), Popov and Black (1981) and Astaneh, Goel and Hanson (1986)), has provided a greatly improved understanding of detailing necessary to achieve high ductility, and non-linear modelling of the brace behaviour has provided information on the response of different structural systems. This information provides the basis for the significant changes incorporated in some recent editions of codes and standards.

Codes differ in the extent of detail specified. NZS (1989) for example provides, in Part 1, the design philosophy and general requirements, and in Part 2 gives very detailed means of compliance with Part 1. On the other hand, Eurocode 8 gives loading and performance requirements, and does little to specify the means of achieving these requirements. Others, such as UBC (1988) and CSA (1989) give specifications for some of the major ductile details. These different approaches leave some comparisons made below incomplete. Many of the seismic provisions described are not applicable in regions of low seismic risk, and the discussion relates only to the most severe seismic regions.

In the following comparisons, two aspects are of primary interest: first the basic philosophy used in the design approach, and the resulting specification of loading, and secondly, the detailed design requirements for members and connections considered necessary to achieve the implied level of available ductility.

CLASSIFICATION OF CBF AND PRINCIPLES OF DESIGN

Canadian design (NBCC 1990) classifies CBF as 'ductile', 'nominally ductile' or in a third category for which no special provisions are made for ductile behaviour, which can be called 'nominally elastic'. Detailed provisions for the first two of these are given in CSA (1989) and impose limitations on overall brace slenderness ratio, brace cross-section width-to-thickness ratios, stitch fasteners for built-up members, framing configurations, and loading on beams and columns. The expected behaviour is that ductility is provided by braces yielding in tension or in flexure under compression; other members of the structure remain essentially elastic, except for beam flexure under certain circumstances. The braced frame is considered in isolation, that is, there are no differences between a CBF alone and one participating in a dual framing system. The base shear magnitude for the nominally elastic CBF is double that of the ductile CBF.

BSL (1981) of Japan classifies brace members in four categories, those with excellent, good, fair and poor ductility. The highest base shear coefficient is 1.43 times the lowest. In providing interpretative material, the commentary on the BSL (Ishiyama, 1985) classifies braced frames according to three parameters: the brace overall slenderness ratio as in the four categories of the BSL, the extent to which the CBF participates in a dual system with a moment resisting frame (MRF), and the ductile classification of the MRF in the dual system. Many different combinations of the three parameters are possible leading to a total of 28 different loading categories.

The New Zealand standard NZS (1989) requires that CBF be designed so that "energy is dissipated through compression and/or flexural yielding of the braces, whereas the beams and columns shall remain elastic." In addition the Standard provides for energy dissipation in tension braces in tension-only systems. Four categories of ductilty demand are defined: 'fully ductile CBF'; 'CBF with limited ductility'; 'nominally elastic' and 'fully elastic CBF'. Members are categorized into those subject to high, low, very low and no ductility demand, and in general, as a minimum, the member categories must be used in the corresponding structure category. Seismic load is related to brace slenderness, frame configuration and number of storeys. For structures over one storey, nine different loading categories are specified and the ratio of structural coefficient for the least ductile to the most ductile CBF is 2.82, but for similar brace slenderness ratios the value is 1.27.

Eurocode 8 was available only in early draft form and consequently it is possible that significant changes from those discussed herein will be made. The draft contains provisions for three categories of frame, 'diagonal bracings' (defined as tension-only), 'V-bracing', which includes Chevron braces, and 'non-dissipative' systems such as K-bracing, where braces intersect within the column height. Diagonal tension-compression bracing is not mentioned. Energy dissipation is to occur in braces only. Design load depends upon the above configurations and on the degree of structural regularity, which is defined both in plan and elevation. The ratio of proposed design load for the non-dissipative system and for the most highly ductile CBF system is 4, and between the highest and lowest dissipative systems is 2.27.

UBC (1988) gives requirements that are almost identical with those of SEAOC (1988). Except for light framed bearing wall systems with tension-only bracing, the UBC provisions apply to all CBF in US seismic zones 2, 3 and 4, and thus frames with lesser ductility are not recognized as a separate category. However the braces in Chevron systems are subject to special treatment as outlined below, and K-braces are forbidden in severe seismic zones. When combined with a MRF incorporating fully ductile details, the dual system structural coefficient is reduced to 80% of the CBF alone.

Most of the codes reviewed incorporate special provisions for Chevron braced frames in view of the problems anticipated in their response, as outlined for example by Nordenson (1984) and Khatib et al. (1988). Thus Eurocode (1988), CSA (1989) and NZS (1989) assign them to a different category from diagonal braced frames, requiring respectively 2, 1.5 and 1.33 times the base shear of the latter, as well as imposing other detailing requirements. UBC (1988) assigns the same base shear to all CBF but requires Chevron brace members to be designed for 1.5 times the calculated member load. Since the maximum force that can be delivered by the braces to the columns must be considered in column design this is almost the same as imposing a higher load on the complete structure. Only the earlier BSL (1981) appears not to treat Chevron bracing differently from diagonal bracing.

All the above design methods are based on the brace acting as the principal, or often the only, energy dissipating element, unless the CBF is part of a dual system. The classification of CBF (and thus the design base shear) varies from a single category to many categories related principally to brace slenderness. The latter allows a close gradation in design load with variation in the main parameters. Dual action of a CBF with a ductile MRF results in a reduced design load (compared with the CBF alone) in Japan, Europe and the USA. In Canada the design load in a dual system is equal to that for the least ductile component of the system. In New Zealand, while NZS (1989) comments on the improved inelastic performance of typical dual systems, NZS (1984) suggests a procedure that uses the structural load coefficients corresponding to the systems acting separately.

Some restrictions are imposed on the bracing systems by requirements in UBC (1988) that neither tension nor compression braces in any planar braced frame should carry more than 70% of the total shear in any storey. A similar restriction exists in CSA (1989) for ductile CBF, and in NZS (1989) the difference between the seismic shear carried by tension and compression braces must not exceed 10%. Eurocode (1988) imposes a limit on tension-only bracing so that the difference in resistance in opposite directions of loading does not exceed 10% of the average of these resistances.

Among the codes considered there is substantial uniformity in the understanding of the behaviour of ductile MRF, and in the codification of requirements for ductile behaviour of MRF. A comparison of the structural system loading coefficient for the CBF with that for the ductile MRF is therefore of interest, and is shown in Table 1. Chevron bracing is excluded, since this system is not considered ductile by some codes, or is assigned a significantly higher design load in others.

Table 1. Comparison of CBF and MRF Design Loads

	Ductile CBF/ Ductile MRF
Canada	1.33
Japan	1.25-1.40
New Zealand (>1 storey)	1.38-1.88
Europe	1.25-1.50
USA	1.50

In nearly all cases some restrictions related to height (or number of storeys) are placed on CBF in the more severe seismic regions. These may prohibit use of low ductility CBF (Canada, NZ) or prohibit the system altogether (NZ); increase the design load (Canada); require a dual structural system (USA), or require special analysis (NZ) or special analysis and governmental approval (Japan). These criteria are too numerous to detail here, but in the comparisons made, it is assumed that the height is less than the value that triggers these special provisions.

Slenderness ratio

Compression braces in the most ductile category of CBF are subject to the slenderness ratio limits given in Table 2 where the slenderness parameter $\lambda = (KL/r)\sqrt{(F_y/\pi^2 E)}$ is shown. The very low slenderness specified in New Zealand and Japan, where three ranges are utilized, influences other aspects of design and should be born in mind when comparing these most ductile systems with those of the other codes.

Table 2. Limitations on brace slenderness (λ)

	Most ductile	Other categories
Canada	1.35	1.35-2.47
Japan	0.35	0.35-0.63 & 0.63-1.41
New Zealand	0.45	0.45-0.90 & 0.90-1.35
Europe	1.50	1.50
USA	1.35	1.35

Width-thickness ratios

Limiting values of these ratios for the most ductile braces are given in Table 3. Wide differences are apparent, reflecting the fact that this is an active subject of research and while there is awareness of the severe curvatures that occur at hinges in buckling braces, there is not yet sufficient information or a concensus on appropriate width-thickness limitations.

Compression strength reduction factor

Due to residual curvatures and the Bauschinger effect, a buckled strut has a reduced compressive resistance on second and subsequent loadings to its compressive ultimate strength, even after the yield load in tension has been applied. For this reason SEAOC (1988) recommends a reduction factor, equivalent to $1/(1+0.35\lambda)$, be applied to the brace compressive resistance. This factor is used in UBC (1988), for nearly all braces, in CSA (1989) for the most ductile category, and in NZS (1989) for Chevron braces. For braces and columns the latter also limits the ratio of factored design force to yield load to 0.5 and 0.7 for the two most ductile of the member categories.

Built-up members

The individual components of built-up members are susceptible to local buckling following overall buckling under cyclic load (Goel and Aslani 1989), and

Table 3. Width-thickness limits for ductile braces ($b\sqrt{F_y}/t$)

Origin[1]	Flanges	Webs	RHS	Angle
Canada	116	670	336	115
New Zealand	136	512	350	136
Europe	169	598	567	N.A.
USA[2]	249	664	624	200

1 Japanese values not available.
2 These are specified in UBC (1988); more stringent requirements are under consideration.

so some limitations have been placed on component slenderness ratios. UBC (1988) limits the component KL/r of a brace to 0.75 of that of the overall member, and CSA (1989) uses 0.5. These codes, as well as NZS (1989), specify design loads for intermediate connections in these members.

BRACE CONNECTIONS

In all codes considered, the basic design load for brace connections in ductile CBF is the brace tensile yield load. This is modified in several ways. Overstrength factors are incorporated by Eurocode (1988) and NZS (1989). For the former this is 1.20 and for the latter range from 1.35 to 1.70, depending on steel grade and ductile category of the brace. CSA (1989) requires the factored connection resistance to exceed the (unfactored) brace yield load, which is equivalent to imposition of an overstrength factor of between 1.11 and 1.33.

BSL (1981) requires the connection "not to fail before braces yield," and UBC (1988) requires connection design for the brace tensile strength. The latter restricts the net to gross area ratio in bolted connections, and CSA (1989) includes an impact factor for tension-only bracing. Three codes, CSA, NZS and UBC, give an upper limit on the connection design load, applicable when braces are over-designed. These are respectively the nominally elastic response load, the fully elastic value, and for UBC an estimate of the maximum response load in the brace (which is related to the elastic response value), but limited also to the "maximum force that can be transferred to the brace by the system."

COLUMNS AND BEAMS

CSA (1989), NZS (1989) and UBC (1988) require columns to be designed for forces corresponding to brace yield loads, with NZS including the brace overstrength factor only at the level of the column considered. In each case the load need not exceed the maximum code specified response load, this being defined as for the brace, above, and each being related to the elastic response value of the column load. Eurocode (1988) defines the column (and beam) design load as the product of 1.20, the calculated load, and an additional factor equal to the ratio of brace resistance to brace calculated load. The minimum value of this factor in all floors is to be used. The column is therefore designed for a load 20% greater than that at first brace yield. BSL (1981) makes no specific mention of column or beam design loads.

NZS (1989) requires beams that "transfer axial load developed in the braces" to be designed to develop brace overstrength. CSA (1989) specifies that beams shall be designed to resist the forces due to brace yielding, and "redistributed loads due to brace buckling or yielding shall be considered." Again, such loads need not exceed the nominally elastic load if the beam participates in the lateral load resisting system.

CONCLUSIONS

The most recent of the codes considered have many similar features reflecting recent research results, and commentary material indicates the influence of the SEAOC Lateral Force Recommendations in some of these standards. The New Zealand standard is the most comprehensive of those considered, and provides a complete, rational approach using capacity design. The other codes are more succinct and make use of many capacity design principles, although not as uniformly as NZS (1989). Areas of similarity are many, whereas significant differences also exist, largely reflecting subjects of current research interest. The Japanese regulations date from 1981, and since considerable study of braced frames has taken place since then, much of it in Japan, it is probable that BSL (1981) no longer reflects all aspects of Japanese CBF design.

The concensus of the recent codes can be summarized as follows:

- CBF design treats the brace as the energy dissipating element; its slenderness ratio is restricted, and seismic design load increases with this ratio; the design load may be from 1.25 to 1.88 of that for a DMRF; when part of a dual structural system, the design load can be reduced
- seismic shears should be resisted partly by members in tension and partly by those in compression, and the proportion carried by each should be balanced
- due to the anticipated poor response of Chevron braced frames, special design requirements are required
- brace connections must be designed to carry the tensile yield load of the brace, and the forces induced in other members by this load must be considered in their design
- brace cross-section elements must conform to severely restricted width-thickness ratios to permit high inelastic strains.

Not all the codes subscribe to all these features. The main differences and other considerations not embraced in the above are:

- the widely different local buckling requirements (Table 3)
- categories of brace slenderness: is one or several appropriate?
- overstrength factors are not used in all codes: should they be and if so what magnitude?
- the upper limit on brace connection design load based on the elastic response value could be exceeded during inelastic response
- approaches to the design of tension-only bracing differ significantly
- beams not active in the lateral load resisting system during elastic response may be called upon to carry substantial axial loads during

inelastic response: should they be considered explicitly? (see CSA 1989)
- the accumulation of column loads due to yielding braces: there is a very small probability of simultaneous yielding; a load combination rule is needed
- the benefits assigned to a CBF acting in a dual system differ considerably.

ACKNOWLEDGEMENT

The authors acknowledge with appreciation the contribution of Vadiraj Channagiri in accumulating data for this paper.

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