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ORIGINAL RESEARCH PAPER

Assessment of European seismic design procedures for steel framed structures

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Abstract This paper assesses the fundamental approaches and main procedures adopted in the seismic design of steel frames, with emphasis on the provisions of Eurocode 8. The study covers moment-resisting as well as concentrically-braced frame configurations. Code requirements in terms of design concepts, behaviour factors, ductility considerations and capacity design verifications, are examined. The rationality and clarity of the design principles employed in Eurocode 8, especially those related to the explicit definitions of dissipative and non dissipative zones and associated capacity design criteria, are highlighted. Various requirements that differ notably from the provisions of other seismic codes are also pointed out. More importantly, several issues that can lead to unintentional departure from performance objectives or to impractical solutions, as a consequence of inherent assumptions or possible misinterpretations, are identified and a number of clarifications and modifications suggested. In particular, it is shown that the implications of stability and drift requirements as well as some capacity design checks in moment frames, together with the treatment of post-buckling response and the distribution of inelastic demand in braced frames, are areas that merit careful consideration within the design process.

Keywords Seismic design · Eurocode 8 · Steel frames

1 Introduction

Although seismic design has benefited from substantial developments in recent years, the need to offer practical and relatively unsophisticated design procedures inevitably results in various simplifications and idealisations. These assumptions can, in some cases, have advert implications on the expected seismic performance and hence on the rationale and reliability of the design approaches. It is therefore imperative that design concepts and application rules are constantly appraised and revised in light of recent research findings and improved

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understanding of seismic behaviour. To this end, this paper focuses on assessing the underlying approaches and main procedures adopted in the seismic design of steel frames, with emphasis on European design provisions.

In accordance with current seismic design practice, which in Europe is represented by Eurocode 8 (EC8) (2004), structures may be designed according to either non-dissipative or dissipative behaviour. The former, through which the structure is dimensioned to respond largely in the elastic range, is normally limited to areas of low seismicity or to structures of special use and importance. Otherwise, codes aim to achieve economical design by employing dissipative behaviour in which considerable inelastic deformations can be accommodated under significant seismic events. In the case of irregular or complex structures, detailed nonlinear dynamic analysis may be necessary. However, dissipative design of regular structures is usually performed by assigning a structural behaviour factor (i.e. force reduction or modification factor) which is used to reduce the code-specified forces resulting from idealised elastic response spectra. This is carried out in conjunction with the capacity design concept which requires an appropriate determination of the capacity of the structure based on a pre-defined plastic mechanism (often referred to as failure mode), coupled with the provision of sufficient ductility in plastic zones and adequate over-strength factors for other regions. Although the fundamental design principles of capacity design may not be purposely dissimilar in various codes, the actual procedures can often vary due to differences in behavioural assumptions and design idealisations.

This paper examines the main design approaches and behavioural aspects of typical configurations of moment-resisting and concentrically-braced frames. Although this study focuses mainly on European guidance, the discussions also refer to US provisions (AISC 1999, 2002, 2005a,b) for comparison purposes. Where appropriate, simple analytical treatments are presented in order to illustrate salient behavioural aspects and trends, and reference is also made to recent experimental observations and findings. Amongst the various aspects examined in this paper, particular emphasis is given to capacity design verifications as well as the implications of drift-related requirements in moment frames, and to the post-buckling behaviour and ductility demand in braced frames, as these represent issues that warrant cautious interpretation and consideration in the design process. Accordingly, a number of necessary clarifications and possible modifications to code procedures are put forward.

2 General considerations

2.1 Limit states and loading criteria

The European seismic code, EC8 (Eurocode 8 2004) has evolved over a number of years changing status recently from a pre-standard to a full European standard. The code explicitly adopts capacity design approaches, with its associated procedures in terms of failure mode control, force reduction and ductility requirements. One of the main merits of the code is that, in comparison with other seismic provisions, it succeeds to a large extent in maintaining a direct and unambiguous relationship between the specific design procedures and the overall capacity design concept.

There are two fundamental design levels considered in EC8, namely ‘no-collapse’ and ‘damage-limitation’, which essentially refer to ultimate and serviceability limit states, respectively, under seismic loading. The no-collapse requirement corresponds to seismic action based on a recommended probability of exceedance of 10% in 50 years, or a return period of 475 years, whilst the values associated with the damage-limitation level relate to a recommended probability of 10% in 10 years, or return period of 95 years. As expected, capacity design procedures are more directly associated with the ultimate limit state, but a number of checks are included to ensure compliance with serviceability conditions.

The code defines reference elastic response spectra (S_e) for acceleration as a function of the period of vibration (T) and the design ground acceleration (a_g) on firm ground. The elastic spectrum depends on the soil factor (S), the damping correction factor (η) and pre-defined spectral periods (T_B, T_C and T_D) which in turn depend on the soil type and seismic source characteristics. For ultimate limit state design, inelastic ductile performance is incorporated through the use of the behaviour factor (q) which in the last version of EC8 is assumed to capture also the effect of viscous damping. Essentially, to avoid performing inelastic analysis in design, the elastic spectral accelerations are divided by 'q' (excepting some modifications for $T < T_B$), to reduce the design forces in accordance with the structural configuration and expected ductility. For regular structures (satisfying a number of code-specified criteria), a simplified equivalent static approach can be adopted, based largely on the fundamental mode of vibration.

2.2 Behaviour factors

For moment frames, such as that shown in Fig. 1a, the reference behaviour factors assigned in EC8 are 4 and $5\alpha_u/\alpha_1$ for ductility classes medium (DCM) and high (DCH), respectively. The multiplier α_u/α_1 depends on the ultimate-to-first plasticity resistance ratio, which is related to the redundancy of the structure. A reasonable estimate of this value may be determined from push-over analysis, but should not exceed 1.6. In the absence of a detailed evaluation, the approximate values recommended by EC8 for moment frames are 1.1 for portal frames, 1.2 for single-bay multi-storey frames and 1.3 for multi-bay multi-storey frames.

In the case of braced frames with concentric diagonal bracing, such as those shown in Fig. 1b, c, the reference q factor in EC8 is 4, for both DCM and DCH. Frames with V or inverted-V bracing, such as that in Fig. 1d, are assigned lower q values of 2.0 and 2.5 for DCM and DCH, respectively. This type of frame has special features that are not dealt with in this study, although some comments relevant to its behaviour are made within the discussions. Also, K-braced frames (e.g. Fig. 1e), are not considered herein as they are not recommended for dissipative design. On the other hand, eccentrically-braced frames (e.g. Fig. 1f), which can combine the advantages of moment-resisting and concentrically-braced frames in terms of high ductility and stiffness, are beyond the scope of this study.

The reference behaviour factor should be considered as an upper bound even if non-linear dynamic analysis suggests higher values. For regular structures in areas of low seismicity, a 'q' of 1.5–2.0 may be adopted without applying dissipative design procedures, recognizing the presence of a minimal level of inherent over-strength and ductility. In this case, the structure would be classified as a low ductility class (DCL) for which global elastic analysis can be utilized, and the resistance of members and connections may be evaluated according to EC3 (Eurocode 3 2005) without any additional requirements.

The application of $q >$ 1.5–2.0 must be coupled with sufficient local ductility within dissipative zones. Similar to other seismic codes, EC8 recognizes the direct relationship between local buckling and rotational ductility of steel members. For dissipative elements in 'compression or bending', this requirement is ensured in EC8 by restricting the width-to-thickness (b/t) of components through the cross-section classification in EC3. For DCM and $2 < q \leqslant 4$, Class 1 or 2 cross-sections should be used, whereas for DCH and $q > 4$, only Class 1 sections should be employed in dissipative zones. EC8 is also clear regarding the intended dissipative zones in the structure. For moment frames, plastic hinges are sought at

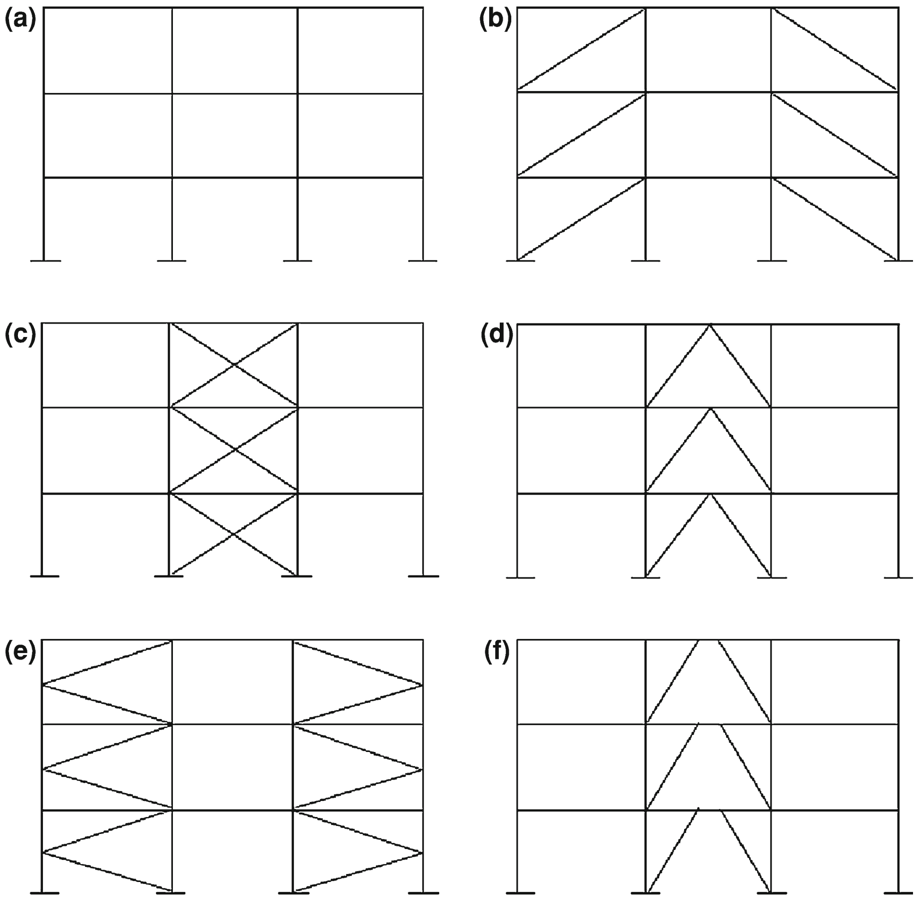


Fig. 1 Examples of frame configuration: **a** moment-resisting, **b** decoupled diagonal bracing, **c** diagonal X-bracing, **d** inverted V-bracing, **e** K-bracing, **f** eccentric-bracing

beam ends except for column bases and at the top storey. In the case of frames with diagonal braces, the dissipative zones are considered in the tension diagonals (but in both braces in the case of V-types).

The behaviour factors proposed in EC8 are summarized in Table 1 which also provides a comparison with the force reduction factors (R) suggested in US provisions (ASCE/SEI 2005). Although a direct code comparison can only be reliable if it involves the full design procedure, it is still useful to compare the level of force reduction allowed in the two codes. As indicated in Table 1, q factors in EC8 appear generally lower than R values in US provisions for similar frame configurations. The suggested R factor in US provisions for regular structures with no specific ductility considerations is 3.0, which is again larger than the equivalent values in EC8.

It is important to note that the same force-based behaviour factors (q) are also proposed as displacement amplification factors (q_{d}). This is not the case in US provisions where specific seismic drift amplification factors (C_{d}) are proposed, as indicated in Table 1. These values, which are largely based on extensive dynamic analysis and hence may implicitly account for inherent frame characteristics, are generally lower than the corresponding R factors for

Table 1 Behaviour factors in European and US Provisions

European code	Ductility class	q	q_d
Non-dissipative	DCL (detailed to EC3)	1.5–2.0	1.5–2.0
Moment frames	DCM	4.0	4.0
	DCH	$5\,\alpha_u/\alpha_1$	$5\,\alpha_u/\alpha_1$
Concentrically-braced frames (diagonal bracing)	DCM	4.0	4.0
	DCH	4.0	4.0
Concentrically-braced frames (V-bracing)	DCM	2.0	2.0
	DCH	2.5	2.5
US Provisions	**Frame type**	R	C_d
Non-dissipative	Detailing to AISC (non-seismic codes)	3.0	3.0
Moment frames	OMF	3.5	3.0
	IMF	4.5	4.0
	SMF	8.0	5.5
Concentrically-braced frames	OCBF	5.0	4.5
	SCBF	6.0	5.0

all frame types. Other differences between European and US provisions include the use of a 'system over-strength' parameter (Ω_o) in the latter (specified as 3.0 and 2.0 for moment and braced frames, respectively), as opposed to determining the level of over-strength within the capacity design procedure in the former. US provisions also consider a range of structural systems which are not directly addressed in EC8, including steel-truss moment frames, steel-plate wall forms and frames with buckling-restrained braces.

In principle, capacity design procedures based on a selected behaviour factor imply a specific pre-determined lateral load resistance, beyond which inelastic dissipative performance is ensured through an appropriate ductility level. In practice, however, the inherent design idealisations and limitations may lead to considerably different response as discussed in subsequent sections.

3 Moment-resisting frames

3.1 Capacity design of columns

In addition to a number of required verifications for the dissipative zones at beam ends, the main capacity design criterion in moment-resisting frames is related to the desirable weak-beam strong-column behaviour. A typical application rule used in seismic codes takes the following form:

$$\sum M_{Rc} \geq k \sum M_{Rb} \tag{1}$$

in which ΣM_{Rc} and ΣM_{Rb} are the sums of the design values of the moments of resistance of the columns and beams, respectively, framing at a joint. This requirement is typically waived at the top storey. A relationship of the above general form is used in AISC (2005a) for SMF (special moment frames), with due account of the material over-strength on the

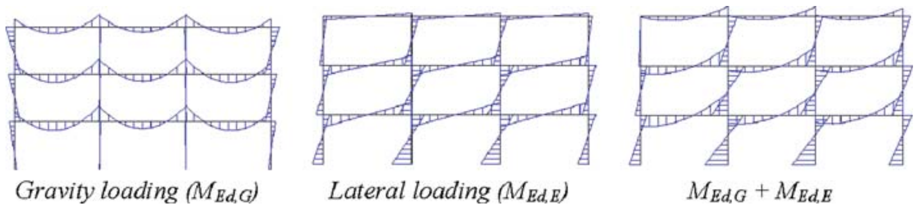


Fig. 2 Moments due to gravity and lateral loading components in the seismic situation

beam capacity (i.e. 1.1 R_y, where R_y is a multiplier to the specified minimum yield strength, varying between 1.1 and 1.5 depending on the steel grade) as well as the additional moment due to the shear amplification from the location of the plastic hinge to the column centreline. A similar approach is adopted in EC8 in which a general requirement of the form shown in Eq. (1) is employed with $k = 1.3$. For steel moment frames, a more specific requirement is included in the most recent version of the code, through which the design bending moment ($M_{\mathrm{Ed,\,col}}$) of the columns is obtained from:

$$M_{\mathrm{Ed,\,col}} = M_{\mathrm{Ed,\,G}} + 1.1\gamma_{\mathrm{ov}}\Omega M_{\mathrm{Ed,\,E}} \tag{2}$$

where $M_{\mathrm{Ed,\,G}}$ and $M_{\mathrm{Ed,\,E}}$ are the bending moments due to the gravity loads and seismic forces, respectively, as illustrated in Fig. 2 for a typical moment frame; Ω is a beam over-strength factor determined as a minimum of $\Omega_{\mathrm{i}} = M_{\mathrm{pl,\,Rd,\,i}}/M_{\mathrm{Ed,\,i}}$ of all beams in which dissipative zones are located, where $M_{\mathrm{Ed,\,i}}$ is the total design moment (i.e. gravity + lateral) in beam 'i' in the seismic design situation and $M_{\mathrm{pl,\,Rd,\,i}}$ is the corresponding plastic moment.

If the gravity loading on the beam is ignored, and adopting the recommended value of 1.25 for γ_{ov}, Eq. (2) effectively takes the same form as Eq. (1) with $k = 1.375$. However, the gravity loading can have a significant effect on the actual over-strength in the beams in certain cases. In fact, a more accurate account of this effect would necessitate a modification to the code-specified relationship of Ω_{i} to Ω_{mod}, represented as (Elghazouli 2007):

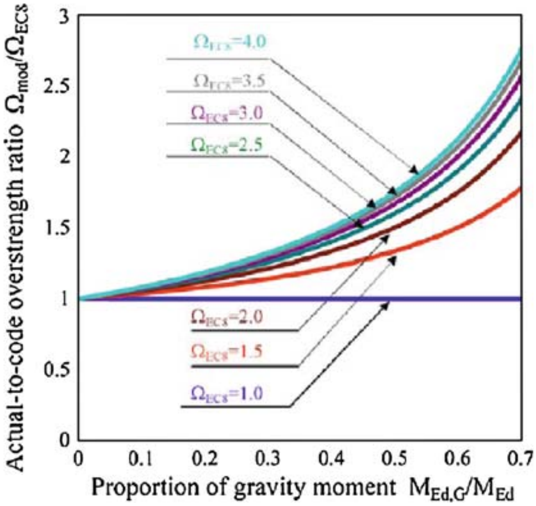
$$\Omega_{\mathrm{mod,\,i}} = \frac{M_{\mathrm{pl,\,Rd,\,i}} - M_{\mathrm{Ed,\,G,\,i}}}{M_{\mathrm{Ed,\,E,\,i}}} \tag{3}$$

As shown in Fig. 3, the actual beam over-strength (Ω_{mod}) may be up to 2 or 3 times that implied by EC8 (Ω_{EC8}) when significant gravity loading is present, except for very low values of over-strength. This problem becomes particularly pronounced in gravity-dominated frames (i.e. with large beam spans) or in low-rise configurations (since the initial column sizes are relatively small). In these situations, the formation of an undesirable soft-storey column-mechanism becomes likely, unless the beam over-strength is accurately determined from Eq. (2) using Ω_{mod} rather than Ω_{EC8}. It should also be noted that the satisfaction of Eq. (1) with a suitable value of k, rather than relying solely on Eq. (2) reduces the extent of this problem.

Another source of inaccuracy related to the use of beam over-strength in the capacity design of columns is that 'Ω' is based on the minimum value within all beams in a frame. In other words, it corresponds to the formation of the first plastic hinge rather than the overall frame capacity. Depending on the frame redistribution capabilities, columns may be subjected to higher actions than those based on the first plastic hinge. This redistribution can be accounted for by incorporating $\alpha_{\mathrm{u}}/\alpha_1$ into Eq. (2), such that:

$$M_{\mathrm{Ed,\,col}} = M_{\mathrm{Ed,\,G}} + 1.1\gamma_{\mathrm{ov}}\frac{\alpha_u}{\alpha_1}\Omega M_{\mathrm{Ed,\,E}} \tag{4}$$

Fig. 3 Accuracy of beam over-strength prediction as a function of gravity loading



Obtaining column design actions from relationships of the form proposed in Eq. (4), in conjunction with the suggested (Ω_{mod}) provides a more rational implementation of the intended capacity design objectives. Nevertheless, it is important to note that whilst codes aim for a 'weak-beam/strong-column' behaviour, some column hinging is often unavoidable. In the inelastic range, points of contra-flexure in members change and consequently the distribution of moments vary considerably from idealised conditions assumed in design. The benefit of meeting code requirements is to obtain relatively strong columns such that beam rather than column yielding dominates over several stories, hence achieving adequate overall frame performance.

3.2 Stability and drift criteria

Two deformation-related requirements, namely 'second-order effects' and 'inter-storey drifts', are stipulated in EC8. The former is associated with ultimate state whilst the latter is included as a damage-limitation (serviceability) condition.

Second-order ($P - \Delta$) effects are specified through an inter-storey drift sensitivity coefficient, or index, (θ) given as:

$$\theta = \frac{P_{\mathrm{tot}} d_{\mathrm{r}}}{V_{\mathrm{tot}} h} \tag{5}$$

where P_{tot} and V_{tot} are the total cumulative gravity load and seismic shear, respectively, at the storey under consideration; h is the storey height and d_{r} is the design inter-storey drift (product of elastic inter-storey drift from analysis and q, i.e. $d_{\mathrm{e}} \times q$). Instability is assumed beyond $\theta = 0.3$ and is hence considered as an upper limit. If $\theta \leq 0.1$, second-order effects could be ignored, whilst for $0.1 < \theta \leq 0.2$, $P - \Delta$ may be approximately accounted for in seismic action effects through the multiplier $1/(1 - \theta)$.

Close examination of the above relationships reveals a fundamental difference from that adopted in recent US provisions (ASCE/SEI 2005) which takes the following form:

$$\theta = \frac{P_{\mathrm{tot}} \Delta}{V_{\mathrm{tot}} h C_{\mathrm{d}}} \tag{6}$$

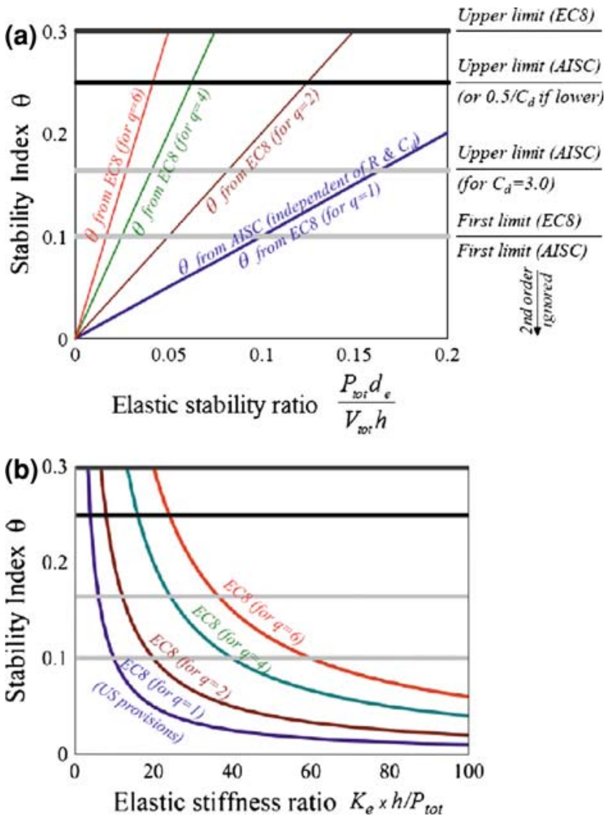


Fig. 4 Comparison of stability coefficients in European and US provisions. **a** Stability index versus elastic stability ratio; **b** Stability index versus elastic stiffness ratio

In the above relationship, Δ is essentially the same as d_r except that q_d is replaced by C_d (i.e. product of elastic inter-storey drift from analysis and C_d); this product is also divided by the importance factor I. Instability is assumed beyond $\theta = 0.25$ (or $0.5/C_d$ if smaller) as an upper limit. If $\theta \leq 0.1$, second-order effects could be ignored, whilst for $0.1 < \theta \leq 0.25$ (or $0.5/C_d$ if smaller) $P - \Delta$ may be approximately accounted for through the multiplier $1/(1 - \theta)$.

In effect, Eq. (6) above adopts an elastic stiffness approach to evaluate the stability coefficient since C_d is used in both the numerator and denominator. In other words, the value of θ only depends on the elastic inter-storey drift (d_e) irrespective of R or C_d. Accordingly, θ determined from EC8 is higher than that obtained from the US provisions by the behaviour factor (q). This comparison is illustrated in Fig. 4a which depicts the stability index according to both sets of provisions. Although the upper limit stipulated in US codes is lower than that in EC8, the resulting value of θ in the latter is considerably higher in most practical cases. As shown in Fig. 4b, to satisfy a specific value of θ, the elastic stiffness ($K_e = V_{tot}/d_e$) required by EC8 is likely to be significantly higher than that in frames designed to US provisions (particularly if the designer seeks to avoid second-order analysis by limiting θ to 0.1). As discussed later in the paper, this criterion can govern the design in many situations.

The above discussion implies that either EC8 is overly conservative in determining the stability coefficient, or that US provisions are inadequate. Assessment of other international codes also suggests that EC8 requirements are quite stringent. This is an area that requires further research, especially that previous studies (Gupta and Krawinkler 2000) point towards the irrationality of code approaches in general. The issue of second-order effects and frame susceptibility to dynamic instability cannot be captured realistically by a code-specified stability coefficient. It is more related to the presence, onset and extent of a possible negative slope within the overall base shear-lateral deformation response, which can lead to significant ratcheting effects under dynamic loading. The dominant plastic mechanism, stiffness of secondary lateral resisting elements, the inherent strain hardening as well as other idealisations and assumptions can all have a direct influence on this issue.

The second deformation-related requirement concerns the control of drift at serviceability. This is addressed in EC8 by limiting the inter-storey drift, 'd_r' in proportion to 'h' such that:

$$d_r \nu \leq \psi h \tag{7}$$

where ψ is suggested as 0.5, 0.75 and 1.0% for brittle, ductile or non-interfering non-structural components, respectively; ν is a reduction factor which accounts for the smaller more-frequent earthquakes associated with serviceability, recommended as 0.4–0.5 depending on the importance class. The above limits are broadly similar to the ranges incorporated in US provisions, although this depends on the importance category under consideration. However, the lower EC8 limit of 0.5% represents a consistently more stringent requirement in all cases.

The above deformation criteria are stipulated for all building types but, as expected, they are particularly important in steel moment frames due to their inherent flexibility. This has direct implications on seismic design as discussed below.

3.3 Lateral frame capacity

One of the most important characteristics influencing seismic response is the over-strength exhibited by the structure. There are several sources that can introduce over-strength, such as material effects caused by a higher yield stress compared to the characteristic value, or size effects due to the selection of members from standard lists, as in those used for steel sections. Additional factors include contribution of non-structural elements, or increase in member sizes due to other load cases or architectural considerations. Most notably, over-strength is often a direct consequence of inherent assumptions or simplifications within the design approach and procedures.

Direct application of the specific rules for moment frames in EC8, followed by inter-storey drift and second-order stability checks, often result in an overall lateral capacity which is notably different from that assumed in design. This can have significant consequences on seismic performance. To illustrate this, Fig. 5 shows qualitatively the key design parameters marked on a typical response obtained from push-over analysis. It depicts the relationship between the displacement at the top of the frame (% of overall height) and the base shear (normalised to V_1, corresponding to formation of the first plastic hinge). As described before, design usually entails reducing the base shear (V_e) obtained from the elastic response spectrum by 'q' to arrive at the design base shear (V_d)—or (F_b) in EC8. The actual resistance (V_y) can, however, be considerably higher than V_d. This additional strength has direct implications on seismic behaviour, particularly in terms of ductility demand on critical members and on forces imposed on other frame and foundation elements. This over-strength also implies the presence of two different behaviour factors: the first is that employed in design (V_e/V_d) whilst the second represents the actual force reduction (V_e/V_y), both being inter-related through the over-strength (V_y/V_d). Evidently, the maximum over-strength considered should not exceed the design behaviour factor employed.

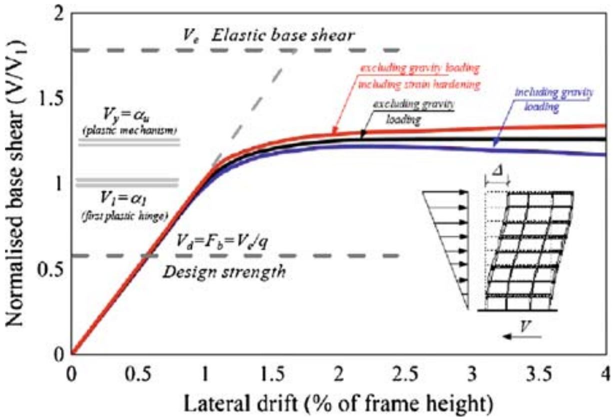


Fig. 5 Inelastic static response of steel moment frames

As discussed previously, over-strength in beam flexural capacity (including material and size effects) is accounted for through the use of $1.1\gamma_{\mathrm{ov}}\Omega$ in the capacity design of columns. EC8 also recognises the increase in strength due to redistribution through $\alpha_{\mathrm{u}}/\alpha_1$ (i.e. V_{y}/V_1 in Fig. 5), which represents the ratio of ultimate base shear to that corresponding to first plastic hinging. It depends on the frame configuration, and importantly on gravity loading as shown in Fig. 5 since it has a direct influence on the sequence of plastic hinging. The extent of redistribution reduces significantly for low levels of gravity load. For practical ranges, the value of 1.3 for $\alpha_{\mathrm{u}}/\alpha_1$ and the upper limit of 1.6 appear to capture this effect reasonably well.

Irrespective of redistribution levels, typical design to EC8 can result in significant over-strength (i.e. $V_{\mathrm{y}}/V_{\mathrm{d}}$ or V_1/V_{d}) depending on several factors including frame configuration, seismic action, behaviour factor, drift limits and gravity design. For typical moment frames, $V_{\mathrm{y}}/V_{\mathrm{d}}$ normally takes the form indicated in Fig. 6 as a function of the normalised elastic response acceleration (S_{e}/g) (Elghazouli 2007). Figure 6 is only indicative of possible over-strength ranges as actual numerical values may differ based on the assumptions made for various parameters.

Except for low S_{e}/g or low q, $V_{\mathrm{y}}/V_{\mathrm{d}}$ is normally governed by inter-storey drift limits, particularly when 0.5% is adopted. In this case (i.e. when member sizes are governed by drift), and due to the equal-displacement approach adopted in EC8, V_{y} is proportional to S_{e}/g, hence $V_{\mathrm{y}}/V_{\mathrm{d}}$ is directly related the value of q (noting that V_{d} is proportional to $S_{\mathrm{e}}/g \times 1/q$). This typically results in relatively constant over-strength, for a given 'q', irrespective of S_{e}/g. If drift limits are relaxed, $V_{\mathrm{y}}/V_{\mathrm{d}}$ becomes more dependent on seismic demand, and follows the trends indicated by the dashed curves in Fig. 6. For low S_{e}/g, depending on frame configuration and design assumptions, over-strength is more significantly influenced by 'θ' limits (or the beam size required for the gravity design situation). With close observation of Eq. (5), it can be deduced that when member sizes are governed by θ, V_{y} is proportional to q, hence $V_{\mathrm{y}}/V_{\mathrm{d}}$ is directly related the value of $q^2 \times g/S_{\mathrm{e}}$. In this case, over-strength increases considerably as S_{e}/g reduces. It is also worth noting that over-strength in excess of 'q' is unrealistic as forces higher than those associated with $q = 1$ would be implied.

The above discussion, in conjunction with Fig. 6, illustrates the considerable levels of lateral over-strength that can be present in frames designed to EC8, and examines the key

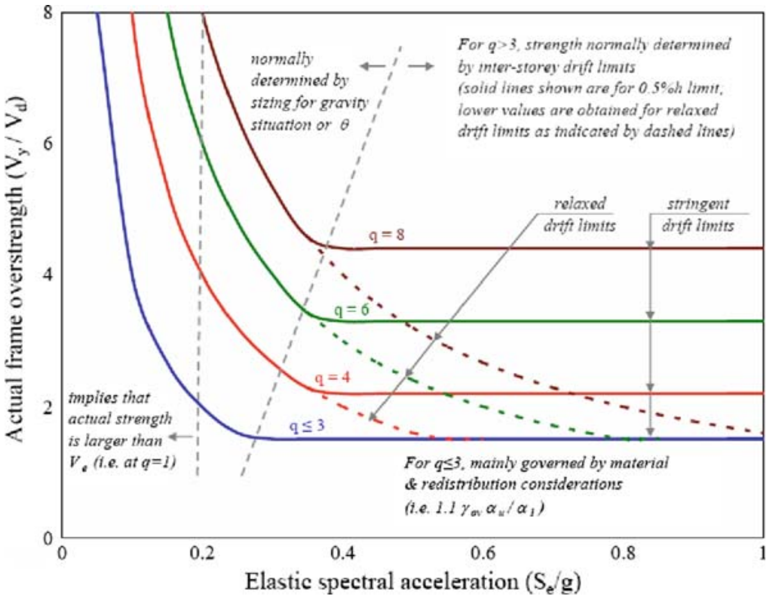


Fig. 6 Characteristics of lateral over-strength in moment frames designed to EC8

factors influencing it. It is not surprising therefore that a recent study (Sanchez-Ricart and Plumier 2008) in which nearly 14,000 ductile moment frames designed to EC8 where assessed, revealed substantial levels of over-strength, ranging between 1.4 and more than 16. Although this over-strength can be beneficial in compensating for the influence of other idealisations (such as those related to capacity design verifications), it is normally a source of unnecessary over-conservatism. Accordingly, it needs to be quantified and considered within the design process in order to achieve an optimum solution that achieves a balance between safety and economy.

Typically, the design process may involve selecting ‘q’ at or near the code limit. Member sizes are then normally modified to meet storey-drift limits. Figure 6 indicates that selecting a high ‘q’ can result in significant over-strength. A more rational procedure could be based on reducing ‘q’ after assessing drift considerations. When design is governed by deformation or gravity considerations, using a lower ‘q’ permits relaxation of local ductility requirements and reduces uncertainties related to capacity design of non-dissipative members and foundations. Only in situations which involve relatively high S_e values, and when relaxed drift limits are employed, does the use of a large ‘q’ appear justified. In any case, after finalising the design, it is desirable to evaluate the actual capacity. This can be carried out using push-over procedures (which are increasingly accessible) or through simplified plastic methods. Alternatively, the elastic analysis can be readily adopted to evaluate the base shear corresponding to the first plastic hinge (V_1 in Fig. 5), which can then be magnified by α_u/α_1 to obtain an estimate of lateral capacity.

3.4 Ductility of dissipative zones

Seismic codes include two related criteria associated with the ductility of dissipative zones in moment-resisting frames. The first is the stipulation of limits on the cross-section slenderness, as discussed before. The second is concerned with the expected rotation capacity of the plastic hinge region, including any deformation within the connection. In US provisions, a limit on the inter-storey drift angle of 0.04 radians is normally implied for SMF; this limit reduces to 0.02 radians for IMF. On the other hand, EC8 stipulates that the plastic rotation capacity of the plastic hinge region should not be less than 35 mrad for DCH and 25 mrad for DCM.

Assessment of the relationship between plastic deformation in dissipative zones and drift levels requires an evaluation of yield rotation. This depends on the frame configuration, yield strength and loading conditions. In general, examination of typical moment-frames indicates that the first plastic hinge formation corresponds to an inter-storey drift lower than 1%. As expected, this value reduces further with the increase in the extent of gravity loading considered in the seismic design situation. Accordingly, the deformation limits proposed for DCH frames in EC8 and for SMF in AISC appear to be of a broadly comparable level.

3.5 Beam-to-column connections

Assessment of the ductility of dissipative zones in moment frames is obviously directly related to the issue of connection performance. Although the topic is beyond the scope of this paper, it is worth noting that EC8 does not provide normative application rules for the design of connections to achieve the required rotation capacities. There is however considerable information available from research and in US guidance that followed the observation of significant connection damage in recent earthquakes (Bertero et al. 1994; SAC 1995, 1996; FEMA 2000). Design can be based either on prequalified connections proposed in the US (ANSI/AISC 2005) or on prototype tests. To this end, there is clearly a need for developing European guidance on appropriate prequalified connections, which is underpinned by experimental research using representative sections, materials and construction practices.

The possibility of using semi-rigid partial-strength connections in seismic design has also been examined by several researchers and appears to be a viable alternative particularly in areas of moderate seismicity (Elghazouli 1996). This enables the use of various forms of bolted connections that have economical and practical benefits. Accordingly, the most recent version of EC8 permits the use of partial-strength connections provided that they can be shown to have stable and ductile behaviour under cyclic loading. The code stipulates that dissipative connections should have a rotation capacity consistent with the global deformations, and that the members framing into the connections are demonstrated to be stable at the ultimate limit state.

Another important aspect of connection behaviour is related to the influence of the column panel zone. This has direct implications on the ductility of dissipative zones as well as on the overall performance of moment resisting frames. Recent research studies (Castro et al. 2005, 2008) involved the development of realistic modelling approaches for the panel zone within moment frames as well as an assessment of current design procedures employed in code provisions. One important issue is related to the treatment of the two yield points corresponding to the onset of plasticity in the column web and surrounding components, respectively, as illustrated in the typical moment-distortion relationship depicted in Fig. 7. Another key design consideration is concerned with balancing the extent of plasticity between the panel zone and the connected beams, an issue which can be significantly affected by the level of gravity applied on the beams. On the one hand, allowing a degree of yielding in the panel reduces the plastic hinge rotations in the beams yet, on the other hand, relatively weak panel zone designs can result in excessive distortional demands which can cause unreliable behaviour of other connection components particularly in the welds. It can be shown that

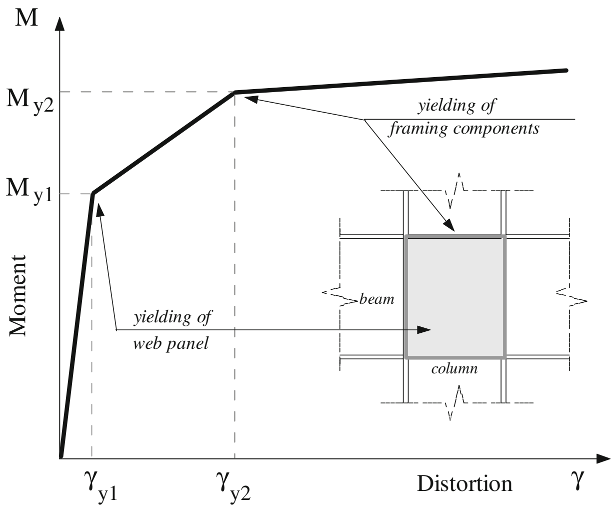


Fig. 7 Typical moment-distortion relationship for panel zones in moment frames

design undertaken according to the guidance proposed in FEMA350 (FEMA 2000) results in a more balanced performance in comparison with both AISC and EC8, with the latter relying on EC3 expressions which may lead to unrealistic evaluation of panel capacity. The rationale used in EC8 is based on the fact that the balance in the ratio of actual-to-nominal yield strength between beams and columns is uncertain. Hence, a pragmatic approach is adopted in the code whereby beams are identified as the preferred location for plasticity, but without employing over-strength to achieve capacity design of the panel zone. This relies on a design strength of the panel zone based on M_{y1}, knowing that the actual strength will be $M_{y2,}$ hence possibly achieving some yielding in the panel (which is limited by the code to 30% of the plastic rotation of the hinge). However, this approach needs to be interpreted cautiously if the panel capacity is determined based on current procedures in EC3. In general, there is clearly a need for further investigations in order to develop reliable procedures for the seismic assessment and design of panel zones.

4 Concentrically-braced frames

4.1 Member response

The response of concentrically braced frames, such as those shown in Fig. 1b–d, is typically dominated by the behaviour of its bracing members. This behaviour has been investigated previously by several researchers (e.g. Maison and Popov 1980; Popov and Black 1981; Ikeda and Mahin 1986; Goel and El-Tayem 1986) focusing mainly on the response under idealised cyclic loading conditions. A recent collaborative European project (Elghazouli 2003; Broderick et al. 2005; Goggins et al. 2006; Elghazouli et al. 2005; Broderick et al. 2008) also examined the performance of bracing members through analytical studies which were supported by monotonic and cyclic quasi-static axial tests as well as dynamic shake table tests. These investigations have provided an insight into the actual performance characteristics of bracing members and their influence on the overall structural behaviour. In this

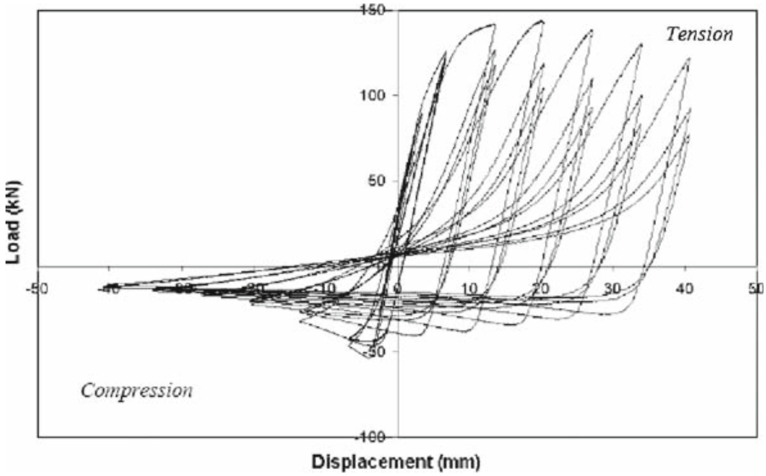


Fig. 8 Typical response of a bracing member under cyclic axial loading

section, a number of key aspects related to the response of bracing members are discussed and several EC8 requirements that merit further consideration are pointed out.

An example of the hysteretic axial load-deformation response of a bracing member is shown in Fig. 8 (Goggins et al. 2006). In compression, member buckling is followed by lateral deflection and the formation of a plastic hinge at mid-length which leads to a gradual reduction in capacity. On reversing the load, elastic recovery occurs followed by loading in tension until yielding takes place. Subsequent loading in compression results in buckling at loads lower than the initial strength due to the residual deflections, the increase in length as well as the Bauschinger effect. Moreover, due to the accumulated permanent elongation, tensile yielding occurs at axial deformations that increase with each cycle of loading.

The discrepancy between the actual and characteristic material strength coupled with strain hardening and strain rate effects have a direct influence on the actual tensile strength of bracing members, as observed in recent cyclic and shake table tests (Broderick et al. 2005; Goggins et al. 2006; Elghazouli et al. 2005; Broderick et al. 2008). These effects are largely covered by the material over-strength parameters incorporated in EC8 (i.e. $1.1\gamma_{ov}$). However, in the case of cold-formed members, the effect of cold-forming on yield strength should be accounted for if test data on section tensile resistance are not available. Moreover, where the evaluation of maximum tensile brace force is necessary in design, it seems irrational to apply partial safety factors (γ_m) for material strength, noting that these are not adopted in US provisions. This issue requires some clarification in EC8 as it can be a source of inconsistency in design.

As in the case of moment frames, the design of connections between bracing members and the beams and columns in a concentrically braced frames is only dealt with in a conceptual manner in EC8. The performance of bracing connections, such as those involving gusset plate components, has attracted significant research interest in recent years (e.g. Yoo et al. 2008; Lehman et al. 2008). In general, there appears to be a need for the development of supplementary European guidance, perhaps through separate manuals or complementary documents, on the design and detailing of recommended bracing connections for seismic resistance. This should be supported by experimental research using representative forms and materials.

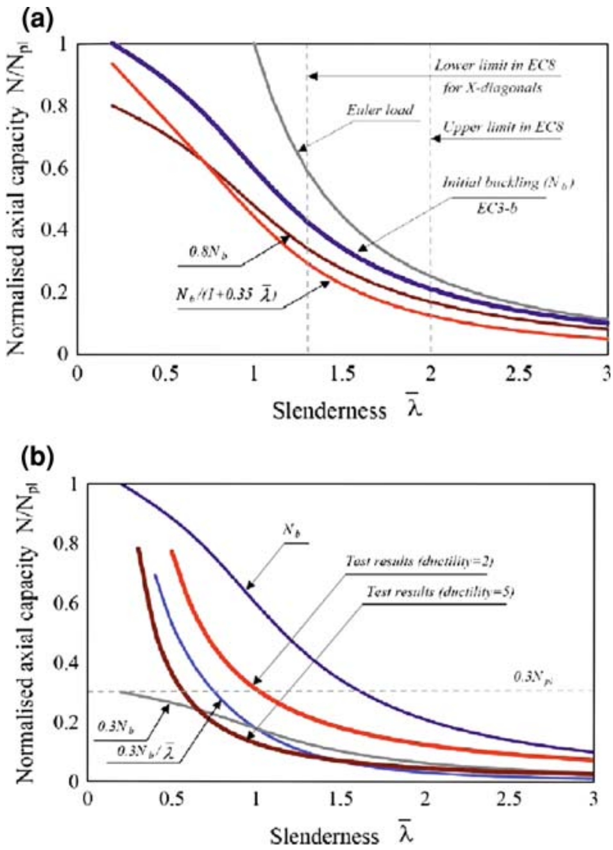


Fig. 9 Buckling and post-buckling resistance of bracing members. **a** Initial and reduced compressive strength; **b** Reserve compressive strength in the inelastic range

4.2 Buckling resistance

The buckling of a bracing member in compression is directly related to the member slenderness, represented in EC3 (Eurocode 3 2005) by the non-dimensional slenderness $\overline{\lambda}$. For non-slender cross-sections, $\overline{\lambda}$ is defined as $\sqrt{N_{\text{pl}}/N_{\text{cr}}}$, in which N_{pl} and N_{cr} are the plastic section capacity and theoretical elastic (Euler) buckling load, respectively. The actual buckling resistance (N_{b}) stipulated in design provisions reflects the influence of imperfections, residual stresses and unintentional eccentricity, and hence differs depending on the type of section and axis of buckling. Whilst the provisions vary from one code to the other, these differences are not significant. In this respect, 'Curve b' of EC3 provides a reasonable average which is representative of test results (Goggins et al. 2006), and is therefore selected herein to represent N_{b}.

Figure 9a depicts the variation of N_{b} with $\overline{\lambda}$, and also indicates the Euler buckling curve. Under cyclic loading, there is a characteristic reduction in buckling strength after one or two cycles of loading. This has been accounted for in earlier North American provisions (SEAOC 1990) by considering a reduced buckling strength $\left(N_{\text{b}}'\right)$ through a relationship of the form:

$$N'_b = \frac{N_b}{1 + 0.35\bar{\lambda}} \tag{8}$$

Alternatively, this may be accounted for through a factor of 80%, applied to N_b (AISC 2002). As indicated in Fig. 9a, adoption of either 0.8 N_b or N'_b from Eq. (8) does not result in significantly different reductions, except for relatively low slenderness values.

It is also important to evaluate the post-buckling resistance (N_{pb}) or reserve compressive resistance of the member under inelastic cyclic loading as it has direct implications on the forces developed in other frame members. To this end, based on the results of recent cyclic tests on bracing members of a wide range of slenderness (Broderick et al. 2005; Goggins et al. 2006), Fig. 9b depicts the reserve compressive resistance at ductility levels of two and five. The curves represent the regression lines based on the average of three cycles at each deformation level. Another typical relationship (Remennikov and Walpole 1988) for the reserve strength at $5\delta_y$ (i.e. $5 \times$ axial yield deformation) suggests the use of 0.3 $N_b/\bar{\lambda}$ (for $\bar{\lambda} > 0.3$), as shown in Fig. 9b. On the other hand, US provisions simply suggest the use of a value of around 0.3 N_b whereas specific sections in EC8 propose an even more simplified factor of 0.3 N_{pl}.

As shown in Fig. 9b, comparing various relationships with curves representing test results, it is clear that the adequacy of using $0.3N_{pl}$ or 0.3 N_b for predicting the reserve strength strongly depends on both the slenderness and ductility levels and hence can give misleading results. It seems rational therefore to relate the reserve strength (N_{pb}) to these two parameters. Close examination of Fig. 9b and the experimental curves, point towards a suggested relationship of the following form:

$$N_{pb} = \frac{0.6}{q\bar{\lambda}^{-1.5}} \quad (\ngtr N'_b) \tag{9}$$

The above relationship provides a close fit to the experimental results and captures the influence of the main parameters, namely slenderness and level of inelastic deformation. Based on the equal-displacement assumption employed in EC8, the behaviour factor q is directly representative of expected inelastic deformation. However, this may need to be modified if significant frame over-strength exists as it has a direct influence on the inelastic demand. The importance of Eq. (9) is dependent on the frame configuration and design check under consideration. For example, it is particularly relevant when checking the maximum forces imposed on the beams in frame with V and inverted-V bracing, as illustrated in Fig. 10. The curves in Fig. 10 are shown for ductility levels of two and five, with the former being more relevant in V-configurations since the q factor suggested in EC8 is in the range of 2.0–2.5 as noted in Table 1. Using Eq. (9), rather than 0.3 N_{pl} as suggested in EC8, appears more realistic for evaluating the out-of-balance forces imposed by the braces on the beam. In other frame configurations and design situations, in which an upper bound of the compressive resistance is necessary, then the use of N_b would be more appropriate. This is discussed further in subsequent parts of this paper.

4.3 Slenderness limits

Seismic codes normally impose an upper limit on $\bar{\lambda}$ in order to ensure satisfactory behaviour under cyclic loading. The limit recommended in seismic provisions generally varies between about 1.3 and 2.0. In earlier versions of EC8, $\bar{\lambda}$ has traditionally been limited to 1.5 but this was relaxed to 2.0 in the most recent version, a relaxation which is supported by observations from recent shake-table tests (Elghazouli et al. 2005). It should be noted that EC8 also

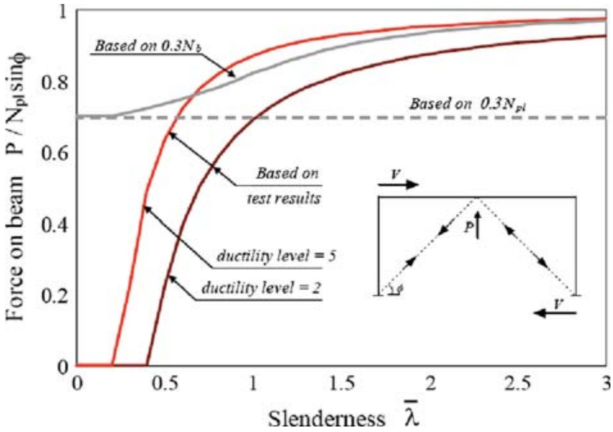


Fig. 10 Maximum force applied on beams in V and inverted V braced frames

imposes a lower limit on $\overline{\lambda}$ for specific frame configurations; this is an issue that is discussed in more detail in Sect. 4.5 below.

In US provisions, the upper limit on $\overline{\lambda}$ has also varied in design guides and over several versions of AISC. It has been assigned values in the range of 1.3–2.0 for typical material properties and depending on the type of frame. The most recent version of AISC (2005a) suggests limiting the slenderness (represented by KL/r) to $4\sqrt{E/F_y}$, where K is the effective length factor, L is the unsupported length, r is the radius of gyration, E is Young's modulus and F_y is the yield strength. This slenderness value is equivalent to $\overline{\lambda}$ of about 1.3 for typical material properties. Slenderness limits can be relaxed in some situations to $4\sqrt{E/F_y} \leq KL/r \leq 200$, which is broadly within the range of 1.3–2.0 for typical material properties.

The limits imposed by codes on $\overline{\lambda}$ have a considerable influence on the seismic design of concentrically-braced frames. In many cases, it may be the governing factor in the dimensioning of bracing members. Most significantly, depending on the design philosophy adopted in the specific code under consideration, $\overline{\lambda}$ of the braces has a direct effect on the performance and design of the overall structure, as discussed in Sect. 4.5.

4.4 Local failure criteria

Under the cyclic axial loading conditions applied on bracing members in seismic situations, failure is largely related to fracture of the cross-section following local buckling, provided that bracing connections are adequately designed and detailed. This was clearly illustrated in recent shake-table tests on tubular bracing members. High strains typically develop upon local buckling in the corner regions of the cross-section. Cracks eventually form in these regions, as shown in Fig. 11 (Elghazouli et al. 2005), and gradually propagate through the cross-section under repeated cyclic loading. The initiation of local buckling and fracture is influenced by the width-to-thickness ratio of the elements of the cross-section, as well as the applied loading history. There is also a dependence on brace slenderness, since for a given level of deformation, higher curvatures arise in plastic hinges that form in members with relatively low slenderness.

Seismic codes rely on the limits imposed on the width-to-thickness ratios of the cross-section in order to delay or prevent local buckling and hence reduce the susceptibility to low

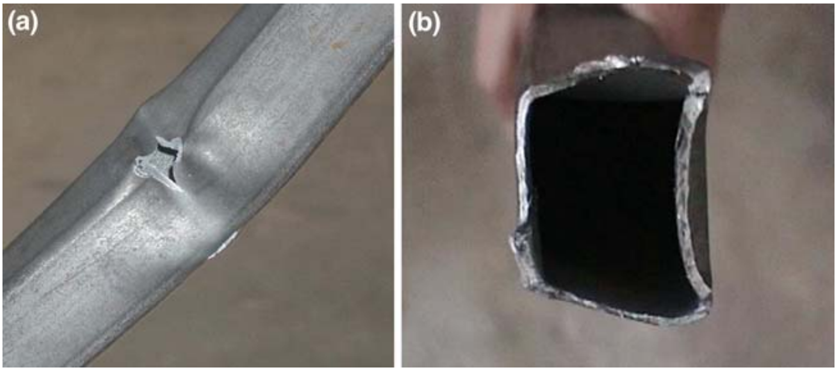


Fig. 11 Failure of bracing members in shake table tests. **a** Initiation of fracture at corner; **b** Full fracture of cross-section

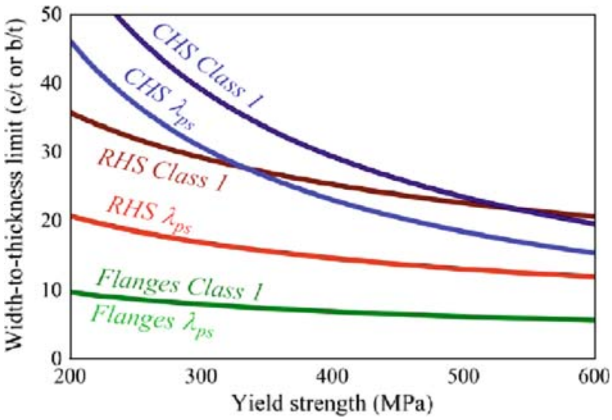


Fig. 12 Comparison of European Class 1 with seismically compact limits (λ_{ps}) in AISC

cycle fatigue and fracture. As noted previously, for dissipative elements in 'compression or bending', EC8 limits the width-to-thickness (b/t) of components depending on the ductility class and behaviour factor adopted. When significant ductility demand is expected, Class 1 cross-sections are required; this can be relaxed to other classes (Class 2 or 3) for lower ductility situations. Similarly, width-to-thickness limits are also imposed in AISC (2005a,b) where high ductility requirements necessitate the use of cross-sections satisfying 'seismically-compact' limits (λ_{ps}). Again, the limits can be relaxed for lower ductility conditions.

It is interesting to compare the width-to-thickness limits recommended in EC8 and AISC in order to ensure ductile behaviour, as shown in Fig. 12. The figure depicts the variation of the width-to-thickness limits in terms of b/t or c/t (with the geometric definition differing slightly for flange outstands) against the yield strength of the material. Both provisions account for the influence of yield strength, with European limits representing this through the parameter $\varepsilon = \sqrt{235/F_y}$, whilst US limits use $\sqrt{E/F_y}$. As illustrated in the figure, Class 1 and λ_{ps} limits for flange outstands in compression are virtually identical. However, there are significant differences for circular (CHS) and rectangular (RHS) hollow sections, which are commonly used for bracing members. For both CHS and RHS, the limits of λ_{ps} are significantly more stringent than Class 1, with the limit being nearly double in the case of RHS. Although the q factors for braced frames are generally lower than R factors (as noted before and shown in Table 1), the differences in cross-section limits in the two codes are significantly more severe. This suggests that tubular braces satisfying the requirements of EC8 are likely to be more vulnerable to local buckling and ensuing fracture in comparison with those designed to AISC.

4.5 Frame over-strength

A significant source of over-strength in concentrically braced frames arises from the simplification associated with the treatment of brace buckling in compression. To enable the use of linear elastic analysis tools, commonly employed in design practice, two different approaches are normally adopted in design methods. Whereas several codes, such as US provisions (AISC 2002, 2005a), base the design strength on the brace buckling capacity in compression (except in special cases), European provisions are largely based on the brace plastic capacity in tension (except for V and inverted-V configurations).

Using the compression-based approach, the design base shear (V_d) corresponds to the attainment of the buckling strength (N_b) in the compression braces, with the tension braces developing a similar value at this stage. Beyond this loading level, the force in the compression brace reduces whereas that in the tension braces increases until it reaches the tensile plastic capacity (N_{pl}). Assuming that the compressive force is not significantly reduced by the time the tension member yields (Elghazouli 2003), coupled with the influence of strain hardening in steel, and noting that α_u/α_1 is not significant in braced frames, the over-strength (V_y/V_d) at the critical storey can be determined as:

$$\frac{V_y}{V_d} = \frac{N_{pl} + N_b}{2N_b} \tag{10}$$

Using an appropriate buckling strength curve, as discussed before, the relationship between V_y/V_d and $\bar{\lambda}$ can be directly evaluated, as depicted in Fig. 13. It is clear from the figure that the lateral over-strength resulting from the compression-based approach increases with the increase of $\bar{\lambda}$. For slenderness values within the limits imposed by AISC, the over-strength seems consistent with the recommended Ω_o of 2.0 for this type of frame. However, the application of Ω_o in the design of all frame members, including the braces, would obviously contradict the philosophy of capacity design.

On the other hand, using the tension-based approach, the over-strength (V_y/V_d) can be represented by:

$$\frac{V_y}{V_d} = \frac{N_{pl} + N_b}{N_{pl}} \tag{11}$$

This relationship is also depicted in Fig. 13, which shows that the over-strength arising from the tension-based idealisation is insignificant for relatively slender braces but approaches a factor of two for relatively stocky braces. In contrast to AISC, EC8 does not employ system amplification factors. Based on Fig. 13 , this may not be necessary except when braces with relatively low slenderness are utilised. To satisfy capacity design in components other than the braces (i.e. to ensure that yielding of the diagonals in tension occurs before yielding or buckling of other components), the same concept used in moment frames and described by Eq. (2) is adopted in EC8, except that the main action is axial rather than moment, and Ω_i is obtained from $\Omega_i = N_{pl,Rd,i}/N_{Ed,i}$ of all the braces in the frame.

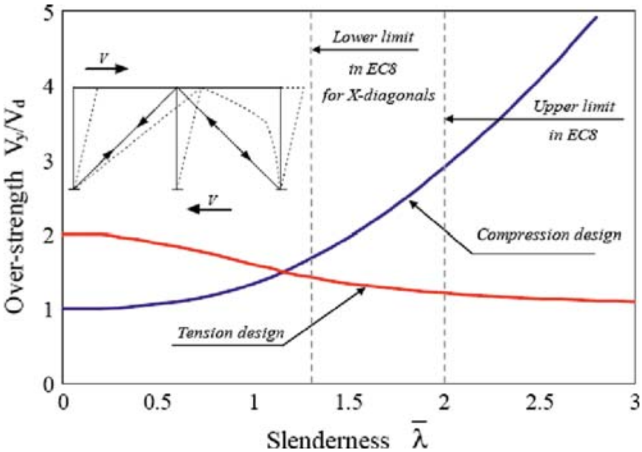


Fig. 13 Influence of brace slenderness on lateral over-strength

It is worth noting that whilst EC8 suggests using a tension-based design in most cases, this should not be interpreted as a means of non-conformance with cross-section width-to-thickness limits required for elements in 'compression and bending'. Another important issue related to tension-based design in EC8 is the lower limit of 1.3 imposed on $\bar{\lambda}$ for frames with X-bracing (such as that in Fig. 1c), in order to avoid overloading the columns prior to buckling of the braces. The maximum compression force F (normalised by N_{pl} sin ϕ) imposed on a column within an idealised storey of a frame with X-diagonals, due to actions developing in the braces, is depicted in the upper half (top curve) in Fig. 14. The actions in the braces would correspond to N_{pl} in tension and N_b in compression, as discussed before in relation to Fig. 13. This is based on the assumption that N_b is not significantly reduced by the time N_{pl} is reached, as illustrated in previous studies (Elghazouli 2003). Clearly, for $\bar{\lambda} > 1.3$ the force in the column can be more than 50% higher than that predicted using the tension-design adopted in EC8. This supports the lower limit of 1.3 on $\bar{\lambda}$ stipulated in EC8, especially that the code does not include any provision for assessing the influence of actions in the compression braces on the forces applied on the columns in frames with X-braces. However, it is important to note that the limit of $\bar{\lambda} > 1.3$ can often be difficult to satisfy in practical design situations, particularly when the braced frame is required to carry significant lateral seismic loads.

The forces imposed on columns in frames with decoupled braces (such as that in Fig. 1b) can differ significantly, as shown in the lower part of Fig. 14. In this case, the maximum force in the column (arising from actions in the braces) corresponds to the attainment of N_{pl} in tension and the minimum value in compression (i.e. N_{pb}, which depends on the ductility level). Evidently, in this situation, the tension-design approach would provide a conservative upper bound of the column. Accordingly, EC8 does not impose a lower limit on $\bar{\lambda}$ in such frames. However, the code stipulates that the design should take into account the implications of developing the buckling resistance in compression braces on the column forces. This suggests the use of N_b in compression, whereas based on the above discussion it would be more appropriate to use a value representative of N_{pb} in many situations.

It is clear from the discussions presented above that, depending on the type of braced frame (i.e. X-diagonals, decoupled diagonals, V or inverted-V, etc.) and the specific design situation, it may be necessary to estimate either the maximum or minimum force attained in

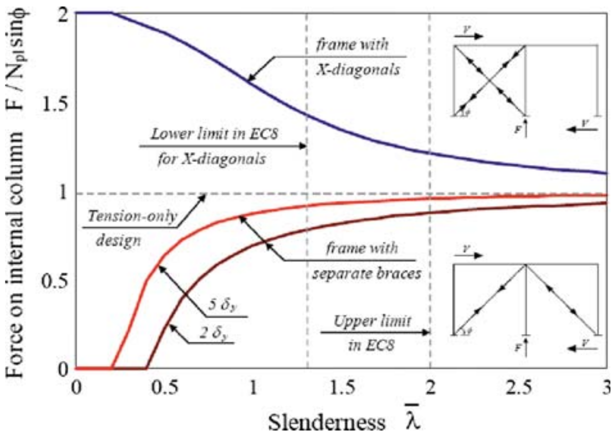


Fig. 14 Column forces in frames with X-diagonals or decoupled braces

compression braces. It was also noted above that imposing a lower value on $\overline{\lambda}$ (in order to limit the compression force in the brace) can cause difficulties in design practice. It seems rational therefore to avoid such limits but at the same time to ensure that forces applied on components other than the braces are based on equilibrium at the joints, with due account of the relevant actions in compression (either N_b or N_{pb}) depending on the design situation.

4.6 Inelastic demand

The over-strength in lateral capacity has a direct implication on the ductility demand imposed on dissipative zones within a frame. As expected, the ductility demand generally reduces with higher levels of over-strength, as illustrated in previous studies (Elghazouli 2003; Elghazouli et al. 2005). Another issue related to ductility demand in braced frames concerns the asymmetry between the tensile and buckling capacity of braces. This can encourage inelastic drifts to occur unevenly in one lateral direction, depending on the characteristics of the excitation. Consequently, codes normally include specific rules in order to limit the discrepancy between lateral brace capacities in opposite directions. EC8 deals with this issue through a simple rule which effectively limits the difference between the horizontal projections of the cross-sectional areas of the braces in both directions to less than 10%.

The tendency of concentrically-braced frames to form storey mechanisms is a particularly important behavioural aspect that warrants some discussion. Once yielding occurs in braces at a storey, the ductility demand is likely to concentrate at this level unless specific measures are considered to prevent the formation of a soft storey. This behaviour is characteristic of braced frames even when brace buckling is delayed or inhibited. With the objective of mitigating this effect by balancing the demand-to-capacity ratio over the height, EC8 limits the maximum difference in brace over-strength ($\Omega_i = N_{pl,Rd,i}/N_{Ed,i}$) over all the diagonals in a frame to within 25%. This limit is, in principle, a useful inclusion in EC8 that is not considered explicitly in other codes, and it can improve the relative behaviour under realistic seismic excitations (Elghazouli 2003, 2007). However, this requirement in isolation cannot eliminate the problem even when the 25% limit is considerably reduced. More importantly, it imposes additional design effort and practical difficulties in the selection of brace sizes.

Whilst relaxing or removing the 25% limit in EC8 could increase the potential for a storey mechanism, this can be offset by the continuity and stiffness of columns along the height.

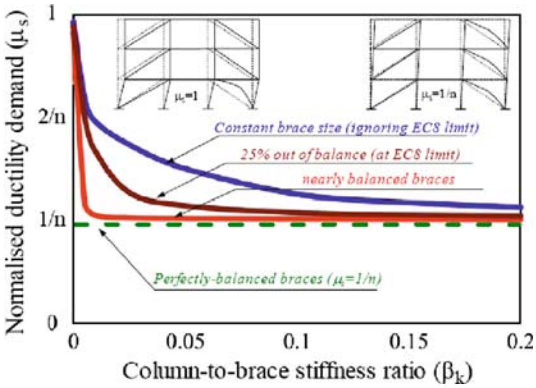


Fig. 15 Concentration of inelastic demand over height in concentrically braced frames

This has been examined through nonlinear dynamic simulations in recent studies (Elghazouli 2003). It is also illustrated in Fig. 15 by considering the inelastic static response of a frame of the form shown in Fig. 1b subjected to an idealised lateral load. Simple connections are considered in the beams, and columns are assumed continuous along the height but pinned at the base. Four variations in relative brace areas over the height are considered: (1) constant area in all braces (i.e. ignoring the EC8 rule); (2) variable brace areas which are 25% out-of-balance with the capacity demand (i.e. according to the limit in EC8); (3) nearly-balanced brace sizes with less than 1% out-of-balance; (4) variable brace areas over height matching exactly the capacity demand (i.e. perfectly-balanced braces). In all cases, brace sizes in the first storey are unchanged whilst those in upper levels are reduced as necessary.

The main measure examined is the relative bending stiffness of the columns $\left(\Sigma\ EI_c/h_c^3\right)$ in proportion to the lateral stiffness of the braces $(\Sigma EA_d \cos\phi/L_d)$, where A_d and L_d are the area and length of the diagonal braces, respectively, whilst I_c and h_c are the second moment of area and height of columns, respectively, and ϕ is the angle between the diagonal and the horizontal projection. If L_d, h_c and ϕ are constant, the stiffness ratio (β_k) reduces to:

$$\beta_k = \frac{L_d \sum I_c}{h_c^3 \cos\phi \sum A_d} \tag{12}$$

As shown in Fig. 15, β_k plays a significant role in determining the inelastic demand (μ_s) on a critical storey. This demand is represented as the ratio between the maximum inter-storey drift and the ultimate drift at the top of the frame. Clearly, values of μ_s approaching unity signify soft-storey behaviour, which would be expected if columns are either discontinuous or have a very low bending stiffness. On the other hand, an ideal demand distribution is achieved when μ_s approaches $1/n$ (where n is the number of storeys), which would be characteristic of frames with relatively rigid columns.

The curves presented in Fig. 15 demonstrate that ensuring column continuity (even with very low stiffness) is sufficient to attain favourable distribution for the case of nearly-balanced braces. On the other hand, if constant brace sizes are used, μ_s reduces with the increase in β_k, to values below $1.2/n$ for $\beta_k > 0.1$. If the 25% requirement of EC8 is met, β_k values needed to attain $\mu_s < 1.2/n$ reduce to under 0.05. Evidently, the stiffness ratio required to achieve an optimum ductility distribution over height increases as the design deviates from a balanced capacity-to-demand brace ratio. Therefore, adopting constant brace areas over the height (or at least over several storeys, in structures with a significant number of storeys) may be satisfactory if adequately stiff continuous columns are utilised thus reducing restrictions imposed on practical design.

5 Conclusions

The main principles and procedures entailed in the seismic design of steel frames according to the provisions of Eurocode 8 are examined in this paper. Key performance requirements and compliance criteria for both ultimate and serviceability levels are discussed. In general, it is shown that the European seismic code implements capacity design concepts for steel structures in a logical manner and through rational procedures. The provisions involve clear identification of dissipative zones, guidance on behaviour factors alongside associated ductility classes and cross-section requirements, and provision of capacity-design verifications for non-dissipative zones. The transparency of the underlying principles, coupled with the clarity of purpose for the parameters used within various procedures, results in a forward-looking code that can be readily adapted and modified based on new research findings and improved understanding of seismic behaviour. To this end, based on the assessments and discussions presented in this paper, areas that require additional consideration are highlighted and a number of modifications are suggested.

The study focuses on typical forms of moment-resisting and concentrically-braced frames. These represent only two main configurations within the range of systems addressed in Eurocode 8, noting that the code does not incorporate several other configurations such as steel-truss moment frames, steel-plate walls and buckling-restrained braces. It is anticipated that these will be considered in future revisions of the code.

For moment frames, it is shown that capacity-design application rules for columns ignore the important influence of gravity loads on the over-strength of beams. To account for this, Ω_{mod} from Eq. (3) is proposed as a replacement of the code-specified parameter Ω_{EC8}. Moreover, column design does not account for the over-strength due to redistribution beyond formation of the first plastic hinge. Accordingly, Eq. (4) is suggested as a substitute for Eq. (2) stipulated in the code by including the $\alpha_{\mathrm{u}}/\alpha_1$ parameter.

In comparison with North American and other international provisions, drift-related requirements in EC8 are significantly more stringent. This is particularly pronounced in case of the stability coefficient (θ), which is a criterion that warrants further detailed consideration. As a consequence of the stern drift and stability requirements and the relative sensitivity of steel moment frames to these effects, they can often govern the design, leading to considerable over-strength, especially if a high 'q' is assumed. This over-strength is also a function of spectral acceleration and gravity design. Whereas the presence of over-strength reduces the ductility demand in dissipative zones, it also affects forces imposed on frame and foundation elements. A rational application of capacity design necessitates a realistic assessment of lateral capacity (using push-over analysis or approximately through $F_{\mathrm{b}} \times \Omega_{\mathrm{mod}} \times \alpha_{\mathrm{u}}/\alpha_1$) after the satisfaction of all provisions, followed by a re-evaluation of global over-strength and the required 'q'. Although high 'q' factors are allowed for moment frames, in recognition of their ductility and energy dissipation capabilities, such a choice is often unnecessary and undesirable.

In terms of beam-to-column connections, there is clearly a need for a concerted effort to develop European guidance, in conjunction with the principles of EC8, on appropriate prequalified configurations using representative sections, materials and detailing practices. Whilst this applies mainly to moment frames, it is desirable to undertake a similar exercise for connections in braced frames. There is also a need for reviewing the design of column panel zones in moment frames, resulting from the combined application of the rules in EC3 and EC8. In particular, the definition of the yield point as well as the balance of plasticity between the panel and connected beams require further consideration.

A notable difference between European and US provisions concerns the cross-section classification. Comparison of the width-to-thickness limits in the Eurocodes and AISC reveals considerable differences in the case of rectangular and circular tubular members, which are commonly employed as bracing members. The limits of seismically-compact sections in AISC are significantly more stringent than those corresponding to Class 1 used in EC3 and EC8. Since the ductility capacity and susceptibility to fracture are directly related to the occurrence of local buckling, it seems necessary to conduct further assessment of the adequacy of Class 1 sections to satisfy the cyclic demands imposed under prevalent seismic conditions.

Apart from material and size effects, it is shown that over-strength in concentrically-braced frames is largely related to the idealisation of buckling in the compression braces. In several design situations, it is important to quantify this over-strength and assess the actual forces sustained by the braces in compression. Depending on the specific design situation and frame configuration, it may be necessary to estimate either the maximum or minimum forces attained in compression braces. Imposing a lower bound on $\overline{\lambda}$, suggested as 1.3 in EC8 for X-bracing, in order to limit the compression force in the brace can cause difficulties in design practice. It would be more practical to avoid placing such limits, yet ensure that forces applied on components other than the braces are based on equilibrium at the joints, with due account of the relevant actions in compression. These actions can be based on the initial buckling resistance (N_b or N'_b) or the post-buckling reserve capacity (N_{pb}) depending on the frame configuration and design situation. A realistic prediction of N_{pb}, such as that of the form suggested in Eq. (9) based on recent experimental results, should account for brace slenderness as well as expected level of ductility.

Another important consideration in braced frames is their vulnerability to the concentration of inelastic demand within critical storeys. To mitigate this effect, EC8 introduces a 25% limit on the maximum difference in brace over-strength (Ω_i) within the frame. Satisfying this rule may not eliminate the problem and can impose additional design effort. It is shown that the 25% limit can be significantly relaxed or even removed (over the full height or a number of successive storeys), if measures related to column continuity and stiffness are incorporated in design. This would reduce restrictions that can often be difficult to achieve in practical design situations.

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