Seismic design of steel structures to Eurocode 8

Ahmed Elghazouli
MSc, DIC, PhD, CEng, MIStructE
Dept. of Civil and Environmental Engineering, Imperial College London, UK

Received: 07/06
Modified: 01/07
Accepted: 03/07
Keywords: Seismic design, Steel, Frames, Eurocode 8, Buildings

Synopsis

This paper examines procedures employed in the seismic design of steel framed structures according to the provisions of Eurocode 8. General design considerations are described and capacity design verifications, for both moment and braced frames, assessed. The rationality and clarity of the underlying design principles utilised in EC 8 are highlighted. It is shown that several inherent idealisations and assumptions warrant careful treatment and consideration in the design process as these can lead to inadvertent departure from performance objectives. Possible misinterpretation of design procedures, coupled with unnecessary emphasis given to some code clauses, can also result in uneconomical or impractical solutions. Accordingly, a number of clarifications and modifications to code procedures are offered.

Introduction

The European code for seismic design has evolved over a number of years, changing status recently to a full European Standard. It consists of six parts covering respectively: buildings; bridges; assessment and retrofitting of buildings; tanks, silos and pipelines, foundations, geotechnical aspects and retaining walls; towers, masts and chimneys. Part 1 (General Rules, Seismic Actions and Rules for Buildings), which is of relevance to this paper, deals mainly with building structures and has the status of a British Standard (BS EN 1998-1:2004)[1]. From the ten sections in Part 1, this paper focuses on Section 6 (Specific Rules for Steel Buildings). In order to assess the overall design process, it is also necessary to refer to the general provisions given in Sections 2, 3 and 4 of Part 1.

The main design approaches for steel framed structures are examined in this paper, with emphasis on simple forms of moment-resisting and concentrically-braced frames. It is important to note that this study does not aim to provide a comprehensive description and evaluation of all code provisions or to cover all structural configurations. Instead, the purpose of this paper is to highlight several key design issues that are worthy of consideration in order to avoid impractical designs or unfavourable performance.

General design considerations

Structures may be designed to EC 8 based on non-dissipative or dissipative behaviour. The former implies largely elastic response and is normally limited to areas of low seismicity or to structures of special importance. Otherwise, economical dissipative design is usually sought whereby significant plastic deformations can be accommodated under extreme events. For most structures, dissipative design is performed by assigning a behaviour factor to reduce the code-specified lateral forces resulting from an idealised elastic response spectrum. This is carried out in conjunction with capacity design approaches, requiring the determination of a pre-defined plastic mechanism coupled with the provision of adequate ductility in plastic zones and appropriate strength in other regions. Compared to other codes, EC 8 adopts capacity design in an unambiguous manner[2] which is a useful feature since a direct relationship can be maintained between design procedures and overall capacity design concepts.

Two fundamental seismic design levels are considered in EC 8 namely 'no-collapse' and 'damage-limitation' which essentially refer to ultimate and serviceability states, respectively. No-collapse corresponds to seismic action based on a recommended probability of exceedance of 10% in 50 years, or a return period of 475 years, whilst damage-limitation relates to a recommended probability of 10% in 10 years, or a return period of 95 years. As expected, capacity design is more directly associated with large events, but several checks are included to ensure compliance with serviceability.

Reference elastic acceleration response spectra (S_e) are defined as a function of period of vibration (T) and design ground acceleration (a_g) on firm ground (Section 3.2.2.2 and Equations 3.2-3.5 of EC 8)[1]. The 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 vary with soil type and seismic source characteristics. As an example, Fig. 1 illustrates the shape of the normalised elastic spectrum assuming Spectrum Type 1 and Ground Type A.

For ultimate limit design, inelastic performance is incorporated through the behaviour factor (q) to obtain an acceleration design spectrum (S_d) (Section 3.2.2.5 and Equations 3.13-3.16 of EC 8)[1]. To avoid inelastic analysis, elastic spectral accelerations are divided by the behaviour factor (excepting some modifications for $T < T_B$ to account for inherent properties) to reduce the design forces in accordance with the structural configuration and expected ductility. The shape of the inelastic design spectrum for various 'q' is also illustrated in Fig 1. For structures satisfying several code-specified regularity criteria, a simplified equivalent static approach can then be adopted.

Rules for steel buildings

For dissipative design, rules related to behaviour factors (described in Sections 6.1-6.5 of EC 8) are summarised in Table 1. The limits for 'q' in moment frames are 4 and $5\alpha_u/\alpha_1$ for DCM (Ductility Class Medium) and DCH (Ductility Class High), respectively. The multiplier α_u/α_1 depends on the ultimate-to-first plasticity resistance ratio, related to the redundancy of the structure. This may be estimated from nonlinear static 'push-over' analysis, but should not exceed 1.6. In the absence of detailed evaluation, the values in Table 1 are recommended. For conventional concentrically-braced frames, 'q' is 4 for both DCM and DCH, but reduces to 2.0-2.5 for V-types. The reference values for q in Table 1 should be considered as an upper bound.

For regular structures in areas of low seismicity, a 'q' of 1.5 may be adopted without applying dissipative procedures, recognising the presence of inherent over-strength and ductility. In this case, the structure is classified as DCL (Ductility Class Low) for which global elastic analysis can be utilised, and the resistance of members and connections evaluated according to EC 3[3] without additional requirements. The application of q > 1.5 must be coupled with sufficient ductility within dissipative

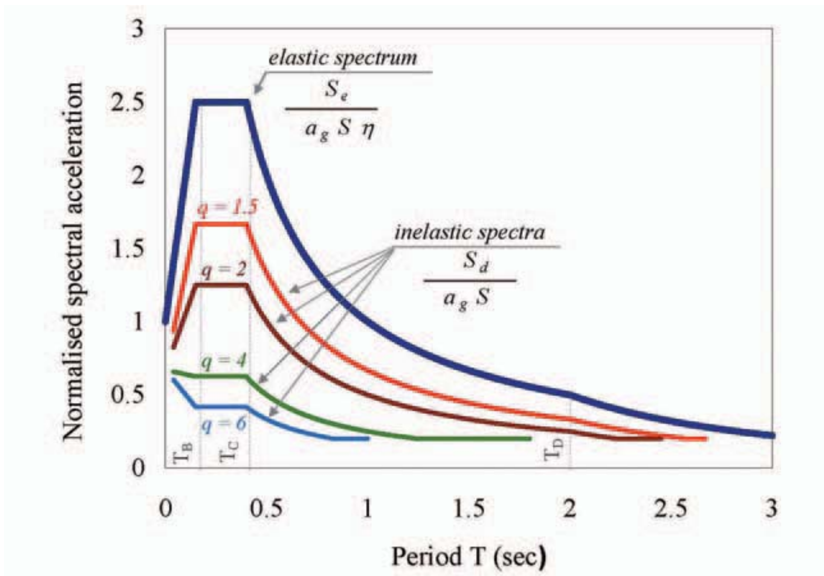
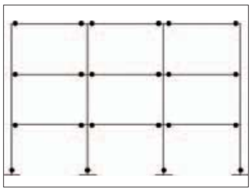
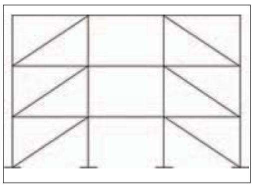


Fig 1. Normalised elastic and inelastic Type 1 spectra

Table 1: Upper limit of reference behaviour factor for regular moment-resisting and concentrically-braced frames according to EC 8

	Ductility class	Behaviour factor (q)	Remarks
Non-dissipative	DCL	1.5	Detailed to EC3 requirements only
Dissipative moment frames  dissipative zones mainly in beams	DCH DCM	$5\ \alpha_u/\alpha_1$ 4.0	**Recommended α_u/α_1:** single portal frames = 1.1; single-span multi-storey = 1.2; multi-span multi-storey = 1.3; $\alpha_u/\alpha_1 \leq 1.6$ **Cross-sections in dissipative zones**: DCM ($1.5 < q \leq 2.0$) = Class 1, 2 or 3; DCM ($2.0 < q \leq 4.0$) = Class 1 or 2; DCH ($q > 4.0$) = Class 1
Dissipative braced frames  dissipative zones mainly in tension diagonals	DCH DCM	4.0 4.0	**Cross-sections in dissipative zones**: DCM ($1.5 < q \leq 2.0$) = Class 1, 2 or 3; DCM ($2.0 < q \leq 4.0$) = Class 1 or 2; DCH ($q > 4.0$) = Class 1

zones. Similar to other codes, EC 8 recognises the direct relationship between local buckling and rotational ductility. As shown in Table 1, dissipative zones should satisfy cross-section classification depending on 'q'. The intended location of dissipative zones is also clearly identified. For moment frames, plastic hinges are sought at beam ends, but column hinges are allowed at the base and in the top storey. In the case of typical braced frames considered herein, dissipative zones are located mainly in the tension diagonals.

The adoption of 'q' enables the use of standard elastic analysis tools for the seismic design of regular structures, using a set of reduced forces. However, drifts obtained from elastic analysis need to be amplified to account for inelastic deformations. In EC 8, the same force-based behaviour factors (q) are proposed as displacement amplification factors (q_d), although these may differ in other seismic codes. A comparison between 'q' in EC 8 and force modification factors (R) in other provisions is presented elsewhere[2]. In principle, capacity design implies a specific lateral load resistance beyond which dissipative performance is ensured through appropriate ductility. In practice, inherent design assumptions and idealisations may result in a considerably different response.

Moment frames

It is assumed in this discussion that conditions for achieving relatively rigid and full-strength connections, with adequate seismic performance, are satisfied. This issue has received considerable attention following damage in recent earthquakes[4,5] and is beyond the scope of this paper. Moreover, the column panel zone is assumed here to have an insignificant influence on the behaviour and, as required by EC 8, non-dissipative response is ensured within this zone.

The seismic design scenario for a regular moment frame such as that shown in Table 1, typically involves elastic analysis incorporating lateral storey forces determined from the base shear (F_b). This in turn is a function of the spectral design acceleration $S_d(T)$ and the seismic mass (m) consisting of the unfactored dead load and a proportion of the imposed load. Based on results of elastic analysis, a set of code checks are required, largely to ensure that capacity design is satisfied. These rules are discussed below and are described mainly in Section 6.6 of EN 1998-1:2004[1].

Capacity design of members

For beams, the main requirement is to ensure that the full plastic moment resistance and rotation capacity are not impaired by co-existing compression and shear forces. To satisfy this for each critical section, the applied axial force (N_{Ed}) should not exceed 15% of the plastic axial capacity ($N_{pl,Rd}$) and the shear force (V_{Ed}) should be limited to 50% of the ultimate shear resistance ($V_{pl,Rd}$). Additionally, since the elastic analysis is based on forces obtained from an inelastic design spectrum (i.e. already reduced by 'q'), the applied moment (M_{Ed}) should not exceed the plastic design moment capacity ($M_{pl,Rd}$).

For columns, the main capacity design criterion concerns the desirable 'weak-beam/strong-column' behaviour. Related criteria have varied over draft versions of EC 8 on general requirements for column-to-beam capacity ratios and suggestions for specific application rules[2]. According to Section 6.6.3 of EN 1998-1:2004[1], the design bending moment ($M_{Ed,col}$) for columns can be obtained from:

$$M_{Ed,\,col} = M_{Ed,G} + 1.1\gamma_{ov}\Omega M_{Ed,E} \qquad ...(1)$$

$M_{Ed,G}$ and $M_{Ed,E}$ are the bending moments in the seismic design situation, due to the gravity loads and lateral earthquake forces, respectively, for the column under consideration. As illustrated schematically in Fig 2, $M_{Ed,G}$ results from gravity actions only (i.e. unfactored dead load and a proportion of imposed load – typically 30% but may be lower), whilst $M_{Ed,E}$ is due to lateral seismic loads obtained from the base shear (this clearly requires two cases of loading: gravity and lateral, which can be dealt with through load combinations in analysis or through a spread-sheet arrangement). The material overstrength factor (γ_{ov}) reflects the ratio of actual-to-design yield strength of steel, which can be assumed as 1.25 in the absence of measurements; γ_{ov} is further amplified by 1.1 to account for other material effects such as strain hardening and strain rate. Therefore, $1.1\gamma_{ov}$ typically amounts to 1.375.

The parameter 'Ω' is a beam over-strength factor determined as a minimum, within all dissipative beam zones, of:

$$\Omega_i = \frac{M_{pl,\,Rd,\,i}}{M_{Ed,\,i}} \qquad ...(2)$$

where $M_{Ed,i}$ is the design moment in beam 'i' in the seismic design situation and $M_{pl,Rd,i}$ is the corresponding plastic moment capacity.

The adequacy of Equations (1) and (2) in satisfying capacity design warrants some discussion. The purpose of this check is to ensure that plastic hinges form primarily in beams rather than columns, under any extreme situation. Critical column sections (except the base and top storey) should therefore be designed for actions corresponding to the development of plastic hinges in beams. Accordingly, actions obtained from elastic analysis should be magnified until the plastic moment is reached at the critical beam section. The extent of this magnification depends on the beam reserve strength (or overstrength). By close observation of Equation (2) and Fig 2, it is evident that EC 8 assumes that this magnification is applied to both $M_{Ed,G}$ and $M_{Ed,E}$. In reality, the gravity moments ($M_{Ed,G}$) remain constant and only the lateral seismic moments ($M_{Ed,E}$) are magnified with more severe events. Consequently, a more accurate definition for 'Ω' should take the form:

$$\Omega_{mod,\,i} = \frac{M_{pl,\,Rd,\,i} - M_{Ed,\,G,\,i}}{M_{Ed,\,E,\,i}} = \frac{M_{pl,\,Rd,\,i} - M_{Ed,\,G,\,i}}{M_{Ed,\,i} - M_{Ed,\,G,\,i}} \qquad ...(3)$$

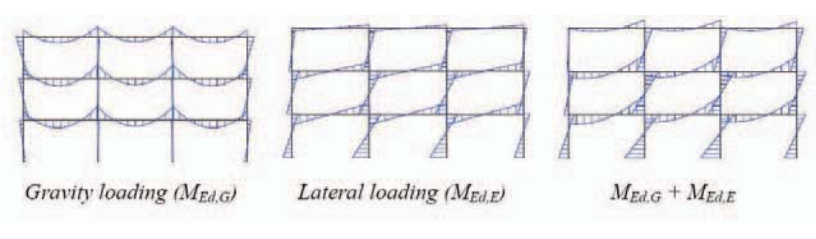


Fig 2. Moments due to gravity and lateral loading in the seismic design situation

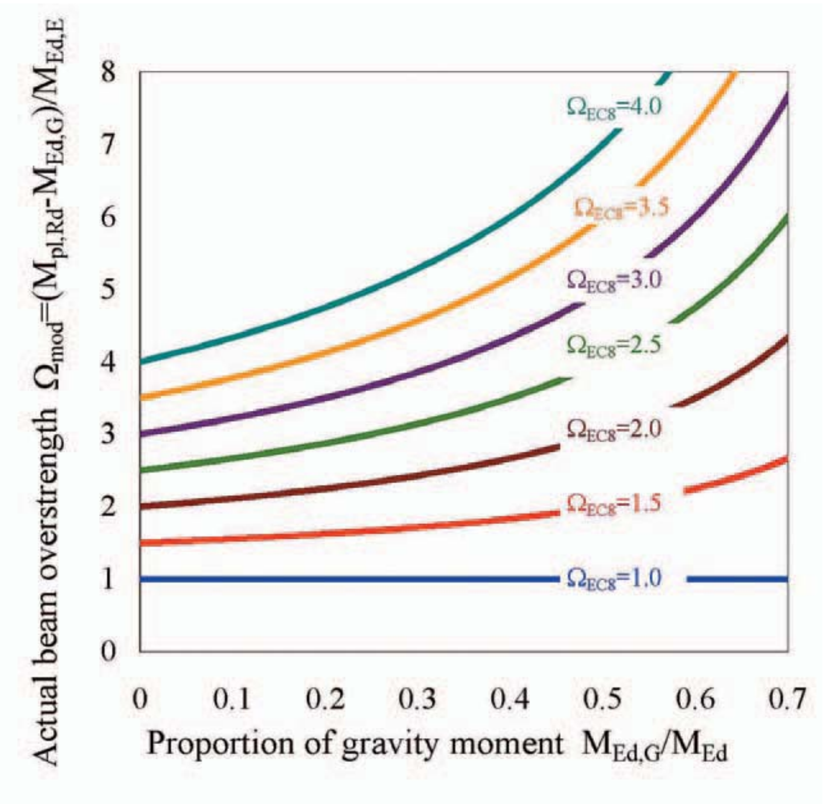


Fig 3. Influence of gravity loading on the accuracy of the beam over-strength parameter 'Ω' suggested in EC 8

Fig 3 compares Ω obtained from Equation (2) (Ω_{EC8}) with the modification suggested in Equation (3), for different levels of gravity moment ($M_{Ed,G}/M_{Ed}$). Equations (2) and (3) converge to the same value if $M_{Ed,G}$ is negligible and/or if $M_{pl,Rd}$ is the same as M_{Ed} (i.e. when gravity loading and/or beam reserve strength are insignificant). Otherwise, as in most practical cases, Equation (2) can significantly underestimate the beam over-strength, leading to violation of capacity design and significant column hinging under extreme events. 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 required for other loading situations are relatively small, hence would be vulnerable unless the beam over-strength is accurately estimated). It should also be noted that adopting Ω_{mod} instead of $\Omega_{EC\,8}$ does not involve additional design effort since it uses parameters already required for Equation (1).

In addition to the design moments from Equation (1), columns should be checked for co-existing axial and shear forces similarly obtained from:

$$N_{Ed,\,col} = N_{Ed,G} + 1.1\gamma_{ov}\Omega N_{Ed,E} \quad ...(4)$$

$$V_{Ed,\,col} = V_{Ed,G} + 1.1\gamma_{ov}\Omega V_{Ed,E} \quad ...(5)$$

where 'Ω' is as defined before (i.e. based on the beam flexural over-strength). From these actions and using EC 3 resistance requirements, columns are then checked for combined bending and axial effects. Also, the applied shear should not exceed 50% of the section plastic shear capacity.

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 α_u/α_1 into Equation (1), such that:

$$M_{Ed,\,col} = M_{Ed,\,G} + \gamma_{ov}\frac{\alpha_u}{\alpha_1}\Omega M_{Ed,\,E} \quad ...(6)$$

The same suggested modification can also be applied to Equations (4) and (5). Again, this adjustment does not require additional design effort since the code-recommended values for α_u/α_1 (in Table 1) can be used.

Obtaining column design actions from relationships of the form proposed in Equation (6), in conjunction with Ω from Equation (3), provides a more rational implementation of 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[2]. In the inelastic range, points of contraflexure 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.

Second-order and inter-storey considerations

Two deformation-related requirements, namely 'second-order effects' and 'inter-storey drifts', are stipulated in Sections (4.4.2.2) and (4.4.3.2) of EC 8[1]. The former is associated with ultimate state whilst the latter is included as a damage-limitation (serviceability) condition.

Second-order (P-Δ) effects are specified through an inter-storey drift sensitivity coefficient (θ) given as:

$$\theta = \frac{P_{tot}\, d_r}{V_{tot}\, h} \quad ...(7)$$

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. $d_e \times q$). Instability is assumed beyond θ = 0.3 and is hence considered as an upper limit. If θ ≤ 0.1, second-order effects could be ignored, whilst for 0.1 < θ ≤ 0.2 P-Δ may be approximately accounted for in seismic action effects through the multiplier 1/(1–θ).

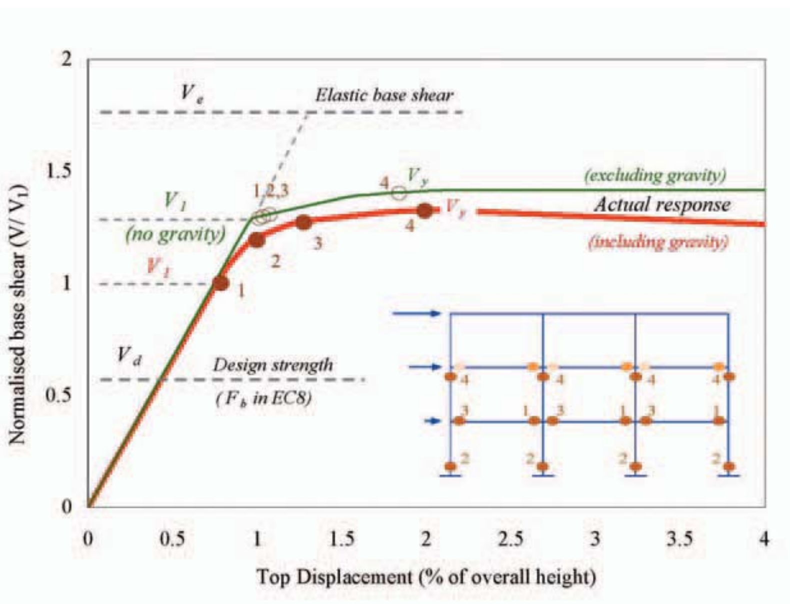
For serviceability, 'd_r' is limited in proportion to 'h' such that:

$$d_r\nu \le \psi h \quad ...(8)$$

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

Fig 4. Actual inelastic static response of moment frames



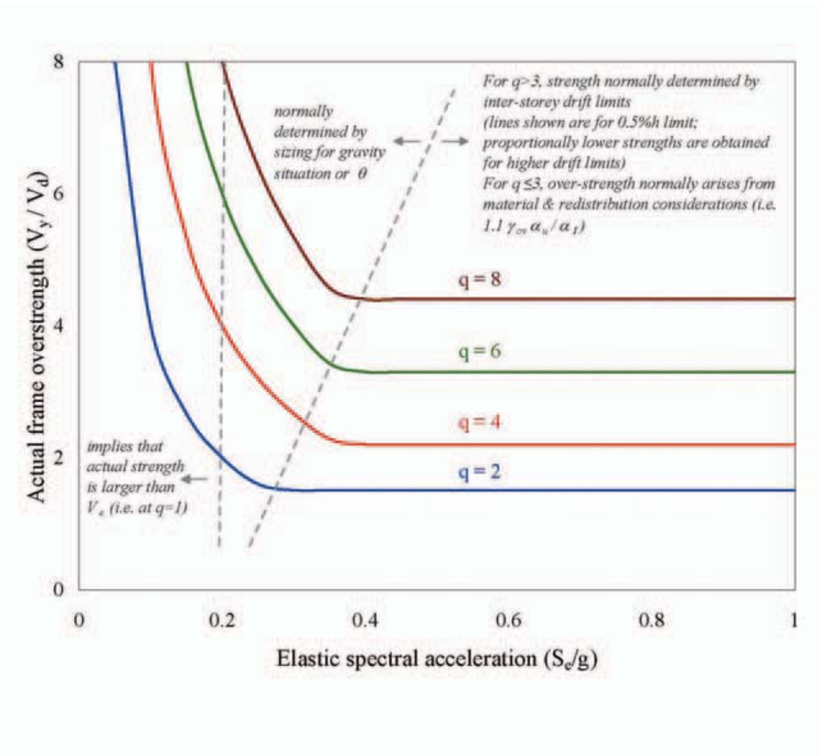


Fig 5. Characteristics of global over-strength in moment frames designed to EC 8

Realistic assessment of frame capacity

Direct application of the specific rules for moment frames, followed by general drift and second-order checks, often result in an overall lateral capacity which is significantly different from that assumed in design. This can have significant consequences on seismic performance. To illustrate this, Fig 4 qualitatively compares key design parameters with typical response obtained from push-over analysis[2]. 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 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 EC 8. 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.

Over-strength can be introduced from several sources ranging from direct material effects to indirect consequences of design idealisations[2]. As discussed previously, over-strength in beam flexural capacity (including material and size effects) is accounted for through the use of $1.1\gamma_{ov}\Omega$ in the capacity design of columns. EC 8 also recognises the increase in strength due to redistribution through α_u/α_1 (V_y/V_1 in Fig 4). This represents the ratio of ultimate base shear to that corresponding to first plastic hinging. Its value depends on frame configuration, and importantly on gravity loading as shown in Fig 4 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 α_u/α_1 recommended for this type of frame and the upper limit of 1.6 appear to capture this effect reasonably well.

Irrespective of redistribution levels, typical design to EC 8 can result in significant over-strength (V_y/V_d or V_1/V_d) depending on several factors including frame configuration, seismic action, behaviour factor, drift limits and gravity design. For a typical moment frame, V_y/V_d normally takes the form indicated in Fig 5 as a function of the normalised elastic response acceleration (S_e/g). Fig 5 is only indicative of possible over-strength ranges as numerical values differ based on assumptions made for various parameters.

Except for low S_e/g or low q, V_y/V_d is normally governed by inter-storey drift limits, particularly when 0.5% is adopted. This results in relatively constant over-strength, for a given 'q', irrespective of S_e/g, which is a consequence of the considerable reduction allowed in seismic forces coupled with stringent inter-storey drift limits. 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. In this case, over-strength increases considerably as S_e/g reduces. It is worth restating that over-strength in excess of the adopted 'q' is unrealistic as forces higher than those associated with q = 1 would be implied.

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. Fig 5 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. 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 from Table 1 to obtain an estimate of lateral capacity.

Braced frames

As noted before, this paper focuses on simple braced frame forms such as that indicated in Table 1. V or inverted-V arrangements (i.e. Chevron braces) have special features that require further attention. Moreover, K-types, in which diagonal members intersect columns at mid-height, are not recommended for dissipative design owing to the undesirable actions induced in columns.

The determination of the base shear (F_b) follows the same procedure discussed before for moment frames. The principles of capacity design are also applied, and most of the discussion made above for moment frames pertains, except that dissipative zones in this case are primarily located in the tension diagonals. Again, based on elastic analysis, a set of code checks are required, largely to ensure that capacity design is satisfied. These rules are described mainly in Section 6.7 of EN 1998-1:2004[1].

Capacity design requirements

For braced frames of the form shown in Table 1, the design should ensure that yielding of the diagonals in tension occurs before yielding or buckling of beams and columns. For the diagonal braces, since elastic analysis is based on forces obtained from an inelastic design spectrum (already reduced by 'q'), the applied axial force (N_{Ed}) should not exceed the plastic axial capacity ($N_{pl,Rd}$). Furthermore, to achieve satisfactory hysteretic behaviour and avoid shock loading under cyclic conditions[6], the non-dimensionless slenderness ($\bar{\lambda}$) should not exceed 2.0.

For beams and columns, to satisfy capacity design the design

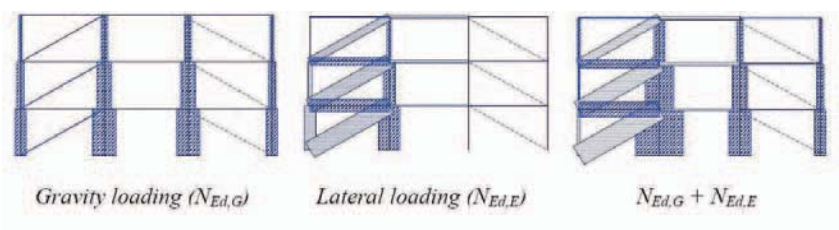


Fig 6. Axial forces due to gravity and lateral loading in the seismic design situation

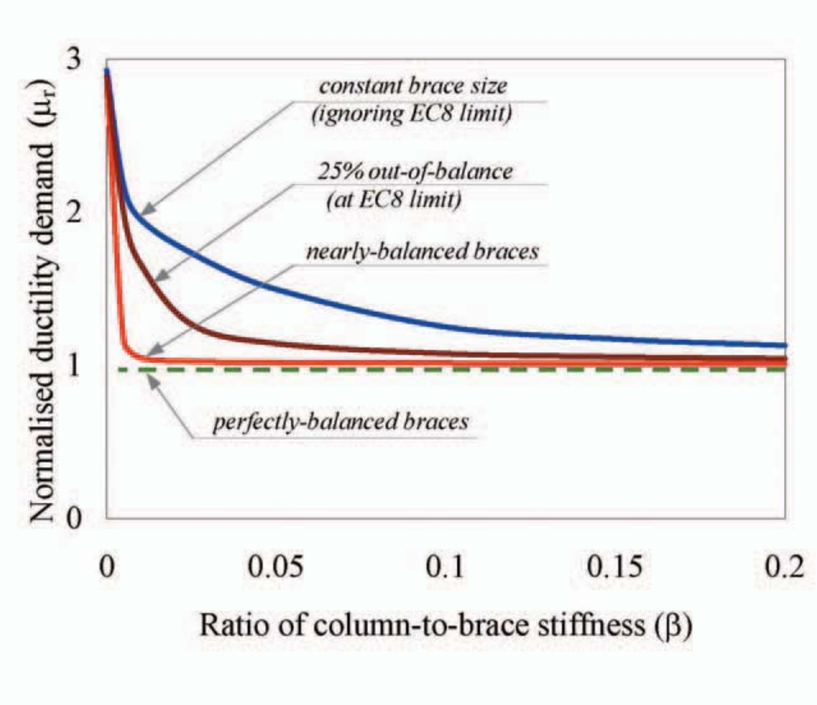


Fig 7. Influence of column stiffness on demand distribution in braced frames

axial load ($N_{Ed,m}$) should be determined from:

$$N_{Ed,m} = N_{Ed,G} + 1.1\gamma_{ov}\Omega N_{Ed,E} \quad ...(9)$$

where $N_{Ed,G}$ and $N_{Ed,E}$ are the axial forces due to gravity loads and lateral seismic forces, respectively, for the beam or column member under consideration. As illustrated in Fig 6, within the seismic design situation, $N_{Ed,G}$ results from gravity actions only whilst $N_{Ed,E}$ is due to lateral earthquake loads. For braced frames, 'Ω' is a brace over-strength determined as the minimum, over all the braces, of:

$$\Omega_i = \frac{N_{pl,Rd,i}}{N_{Ed,i}} \quad ...(10)$$

where $N_{Ed,i}$ and $N_{pl,Rd,i}$ are the design axial force and plastic capacity, respectively, for brace 'i'. Beams and columns should then be checked for buckling or yielding based on $N_{Ed,m}$ considering interaction effects from any co-existing moment (M_{Ed}) in the seismic condition.

Unlike for moment frames, Equations (9) and (10) would not normally require modification in order to satisfy capacity design principles since loading in the braces is typically determined by the lateral action. α_u/α_1 is also less significant in comparison with moment frames (hence assumed as unity in Table 1).

As for other structural types, braced frames should be checked for second-order and inter-storey drifts. Although these requirements may lead to modification of member sizes in some cases, their influence is less pronounced than for moment frames due to the relatively high lateral stiffness of braced forms.

Apart from the influence of Ω from Equation (10), resulting from the difference between the plastic brace capacity and the applied axial load, frame over-strength mainly arises from the treatment of brace buckling in compression[6]. For the frame type considered herein, EC 8 suggests basing the lateral capacity on the tension braces only. Hence, the over-strength (V_y/V_d) arising from this idealisation is insignificant for relatively slender braces, but approaches a factor of two for comparatively stocky diagonals.

Ductility demand

An important consideration is associated with the tendency of concentrically-braced frames to form soft-storeys. As opposed to 'weak-beam/strong-column' in moment frames, this behaviour is related to lateral demand-to-capacity ratios over the height in a braced frame. To deal with this, Clause 6.7.3.8 of EN-1998-1:2004[1] limits the maximum difference in brace over-strength (Ω_i) over all the frame diagonals to 25%. If achieved, this can improve relative behaviour under realistic earthquake excitations. However, this requirement in isolation cannot eliminate the problem, even when the 25% limit is considerably reduced[6]. Moreover, this rule imposes additional design effort and possible practical difficulties in the selection of brace sizes.

Although the 25% requirement is a useful inclusion which is not explicitly considered in other codes, a significant role is played by the column continuity and stiffness along the height. Fig 7 illustrates this by considering the inelastic static response of a three-storey braced-frame, of the form shown in Table 1, under an idealised lateral seismic load[2]. Simple connections are considered in 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: (i) constant area in all braces – i.e. ignoring the EC 8 rule; (ii) variable brace areas which are 25% out-of-balance with the capacity demand according to the limit proposed in EC 8; (iii) nearly-balanced brace sizes with less than 1% out-of-balance; (iv) 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 if/as necessary.

The relative bending stiffness of columns, in proportion to the lateral stiffness of tension braces at the lowest storey, is represented in Fig 7 by β given as[2]:

$$\beta = \frac{\sum \frac{I_c}{L_c^3}}{\sum \frac{A_d}{L_d}\cos\phi} = \frac{L_d \sum I_c}{L_c^3 \cos\phi \sum A_d} \quad ...(11)$$

where A_d and L_d are the area and length of the diagonal braces, respectively; I_c and L_c are the second moment of area and height of columns, respectively; ϕ is the angle between the diagonal and the horizontal projection. The simplified version of Equation (11) applies if L_d, L_c and ϕ are constant.

Fig 7 depicts the relationship between (β) and the normalised ductility demand (μ_r), defined herein as:

$$\mu_r = \frac{d_{r,max}\, n}{\Delta_{top}} \quad ...(12)$$

where $d_{r,max}$ is the maximum inter-storey drift within the frame, 'n' is the number of stories and Δ_{top} is the drift at the frame top. Clearly, values of μ_r approaching 3 for the three-storey frame considered 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_r approaches unity, characteristic of a relatively rigid column case.

As shown in the figure, relatively low column 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_r reduces with the increase in β, to values below 1.2 for $\beta > 0.1$. If the 25% requirement of EC 8 is met, β values needed to attain $\mu_r < 1.2$ 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 height may be satisfactory if adequately stiff continuous columns are utilised thus reducing restrictions imposed on practical designs.

Conclusion

The provisions of EN1998-1-2004[1] incorporate several desirable features including an explicit implementation of capacity design. Code procedures involve a clear identification of recommended dissipative zones, selection of behaviour factors alongside associated ductility classes and cross-section requirements, and capacity-design verifications for non-dissipative zones. However, several issues require careful interpretation and consideration in the design process.

For moment frames, 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

Equation (3) is proposed as a replacement of the code-specified $\Omega_{EC\,8}$ from Equation (2). Moreover, column design does not account for the over-strength due to redistribution beyond formation of the first plastic hinge. Accordingly, Equation (6) is suggested as a substitute for Equation (1) stipulated in the code by including the α_u/α_1 parameter.

Moment frames typically exhibit significant over-strength which affects forces imposed on frame and foundations elements, and ductility demand in dissipative zones. Drift limits can often govern the design, leading to considerable over-strength if a high 'q' is assumed. This over-strength is also a function of spectral acceleration, gravity design and stability limits. A rational application of capacity design necessitates a realistic assessment of lateral capacity (using push-over analysis or approximately through $F_b \times \Omega_{mod} \times \alpha_u/\alpha_1$) 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.

For concentrically-braced frames, apart from material and size effects, over-strength is largely related to the assumption that lateral resistance is based on tension braces only. Consequently, this over-strength is insignificant for slender braces and approaches two for relatively stocky braces. Another important consideration in braced frames is their vulnerability to demand concentration over height. To mitigate this effect, EC 8 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 relaxed or even removed, if measures related to column continuity and stiffness are incorporated in design.

REFERENCES

1. Eurocode 8. (2004). *Design of Structures for Earthquake Resistance, Part 1: General Rules, Seismic Actions and Rules for Buildings, BS-EN1998-1-2004*, European Committee for Standardization, CEN, Brussels
2. Elghazouli, A. Y. (2005). 'Assessment of capacity design approaches for steel framed structures', *J. Steel Structures*, **5**/5, 465-475
3. Eurocode 3. (2005). *Design of Steel Structures, Part 1.1: General Rules and Rules for Buildings, BS-EN1993-1-1-2005*, European Committee for Standardization, CEN, Brussels
4. SAC. (1995). *Survey and Assessment of Damage to Buildings Affected by the Northridge Earthquake of January 17, 1994*, SAC95-06, SAC Joint Venture, Sacramento, CA
5. FEMA. (2000). Federal Emergency Management Agency, 'Recommended seismic design provisions for new moment frame buildings', *FEMA 350,* Washington DC.
6. Elghazouli, A. Y. (2003) 'Seismic design procedures for concentrically braced frames', *Proc. Institution of Civil Engineers, Structures and Buildings,* 156, 381-394

IStructE CENTENARY 1908–2008

40-50 *years*

The President and Council congratulate the following Fellows and Members who have completed 40 and 50 years membership of the Institution as a Chartered Structural Engineer during January to March 2007. In writing to each member the President recorded this milestone of achievement and hoped that they will continue to enjoy many more years of association with the Institution.

40 years of membership from January to March 2007

Fellows

Adamides, Makis Costa
Gibbard, Denis Frederick Brynly
Medcalf, Roy
Piggot, John Ashton
Roe, Charles
Smart, Richard Arundell
Warren, Timothy Stafford
Wigfull, Robert John
Hewett, Peter
Larkin, Gerald Henry
Sarkar, Asit Kumar
Smith, Alan William
Smith, Gerald Charles
Whiffin, Martin Christopher Arthur

Members

Barnes, Peter David
Dollery, Derek Robert

50 years of membership from January to March 2007

Fellows

Alexander, George
Campbell, Peter Leonard
Ellis, William George
Graff, Samuel
Mornement, Peter Clews
Ng, Eng Hean
Shippen, Clive

Members

Burman, David Charles
Hunt, Herbert Arthur
Patel, Naginbhai Fakirbhai