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SEISMIC DESIGN OF STEEL-FRAMED STRUCTURES TO EUROCODE 8

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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 distribution of inelastic demand in braced frames, are areas that merit careful consideration within the design process.

1. INTRODUCTION

The European code for seismic design (Eurocode 8, 2004) 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. 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.

Two fundamental seismic design levels are considered in EC8 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 EC8). 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. 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 EC8). 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. For structures satisfying several code-specified regularity criteria, a simplified equivalent static approach can then be adopted.

2. BEHAVIOUR FACTORS

For dissipative design, rules related to behaviour factors (described in Sections 6.1-6.5 of EC8) 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, α_u/α_1 may be assumed as 1.1, 1.2 and 1.3 for single portal, single-span multi-storey and multi-span multi-storey frames, respectively. 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.

Table 1 Behaviour factors for steel frames

Type	*Ductility Class*	*q*	q_d
Non-dissipative	DCL (detailed to EC3)	1.5-2.0	1.5-2.0
Moment frames	DCM DCH	4.0 $5\alpha_u/\alpha_1$	4.0 $5\alpha_u/\alpha_1$
Concentrically-braced frames (diagonal bracing)	DCM DCH	4.0 4.0	4.0 4.0
Concentrically-braced frames (V-bracing)	DCM DCH	2.0 2.5	2.0 2.5

For regular structures in areas of low seismicity, a 'q' of 1.5-2 may be adopted without applying dissipative procedures, recognizing 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 EC3 (Eurocode 3, 2005) without additional requirements. The application of q>1.5-2 must be coupled with sufficient ductility within dissipative zones. Similar to other codes, EC8 recognizes the direct relationship between local buckling and rotational ductility. Dissipative zones should satisfy cross-section classification depending on 'q' (Class 1, 2 or 3 for DCM and $1.5<q\leq 2.0$; Class 1 or 2 for DCM and $2.0<q\leq 4.0$, and Class 1 for DCH and $q>4.0$). 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 assumed mainly in the tension diagonals (but in both braces in the case of V-types). 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 EC8, the same force-based behaviour factors (q) are proposed as displacement amplification factors (q_d), although these differ in other seismic codes. A comparison between 'q' in EC8 and force modification factors (R) in US provisions is presented elsewhere (Elghazouli, 2005), indicating notable differences in behaviour factors as well as cross-section slenderness limits. 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.

3. MOMENT FRAMES

The seismic design scenario for a regular moment frame 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. It should be noted that discussions presented in this paper assume 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 (e.g. Bertero *et al*, 1994; SAC, 19995; FEMA, 2000), and led to the specification of prequalified connections (AISC, 2005), but is beyond the scope of this paper. Moreover, the column panel zone is assumed here to have an insignificant influence on the behaviour, although this is an important issue that is treated inconsistently in design codes as discussed in detail elsewhere (Castro *et al*, 2008).

3.1. Capacity Design of Members

Apart from a number of checks to ensure that the full plastic moment resistance and rotation capacity of plastic hinges in beams are not impaired by co-existing compression and shear forces, the main capacity design requirements are related to the desirable 'weak-beam/strong-column' behaviour. Related criteria have varied over draft versions of EC8 on general requirements for column-to-beam capacity ratios and suggestions for specific application rules (Elghazouli, 2005). According to Section 6.6.3 of EC8, the design bending moment ($M_{Ed,col}$) for columns can be obtained from:

$$M_{Ed,col} = M_{Ed,G} + 1.1\gamma_{ov}\Omega M_{Ed,E} \tag{3.1}$$

$M_{Ed,G}$ 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 (Elghazouli, 2007). The material over-strength 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 of $\Omega_i = M_{pl,Rd,i}/M_{Ed,i}$ of all beams in which dissipative zones are located, where $M_{Ed,i}$ is the design moment in beam 'i' and $M_{pl,Rd,i}$ is the corresponding plastic moment capacity.

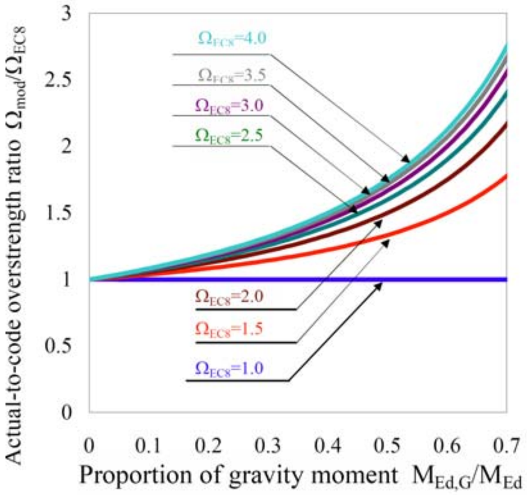

Figure 1 Accuracy of beam over-strength

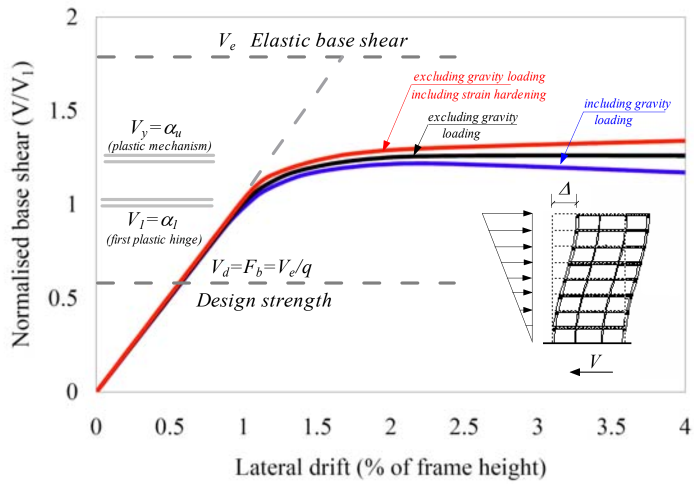

Figure 2 Inelastic static response

The purpose of the check in Equation (3.1) is to ensure that plastic hinges form primarily in beams rather than columns, under any extreme situation. Critical column sections (except at 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. EC8 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. 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):

$$\Omega_{\text{mod},i} = \frac{M_{pl,Rd,i} - M_{Ed,G,i}}{M_{Ed,E,i}} \tag{3.2}$$

As shown in Figure 1, the actual beam over-strength (Ω_{mod}) may be up to 2 or 3 times that implied by EC8 (Ω_{EC8}). 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 Equation (3.2) using Ω_{mod} rather than Ω_{EC8}. It should also be noted that the satisfaction of a simple column-to-beam over-strength ratio (as stipulated in Section 4 of EC8 and in other seismic codes) 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 α_u/α_1 into Equation (3.1) such that:

$$M_{Ed,col} = M_{Ed,G} + 1.1\gamma_{ov}\frac{\alpha_u}{\alpha_1}\Omega\ M_{Ed,E} \tag{3.3}$$

Obtaining column design actions from relationships of the form proposed in Equation (3.3), 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 Sections (4.4.2.2) and (4.4.3.2) of EC8 (2004). 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:

$$\theta = \frac{P_{tot} d_r}{\mathrm{V}_{\mathrm{tot}} h} \tag{3.4}$$

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 $\theta = 0.3$ 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-Δ may be approximately accounted for in seismic action effects through the multiplier $1/(1-\theta)$.

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

$$d_r\ \nu \leq \psi\ h \tag{3.5}$$

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. It is worth noting that EC8 requirements for θ are quite stringent in comparison with other codes; the same applies to inter-storey serviceability drift, particularly if the lower limit of 0.5% is adopted in design.

3.3 Lateral 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, Figure 2 qualitatively compares key design parameters with typical response obtained from push-over analysis (Elghazouli, 2007). 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 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.

Over-strength can be introduced from several sources ranging from direct material effects to indirect consequences of design idealisations. 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. EC8 also recognises the increase in strength due to redistribution through α_u/α_1 (V_y/V_1 in Figure 2). Its value depends on frame configuration, and importantly on gravity loading as shown in Figure 2 since it has a direct influence on the sequence of plastic hinging. For practical ranges, the value of 1.3 for α_u/α_1 recommended for multi-span/multi-storey moment frames and the upper limit of 1.6 appear to capture this effect reasonably well.

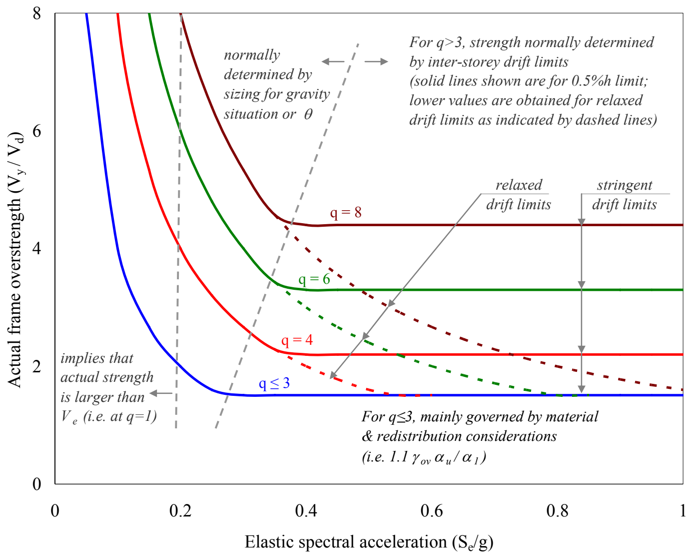

Figure 3 Lateral overstrength in moment frames

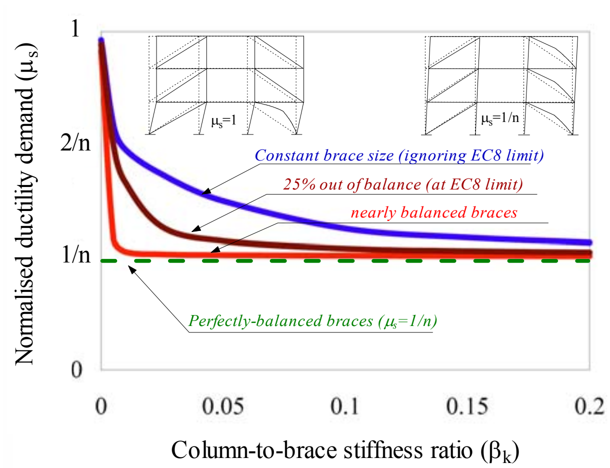

Figure 4 Demand distribution in braced frames

Irrespective of redistribution levels, typical design to EC8 can result in significant over-strength depending on several factors including frame configuration, seismic action, behaviour factor, drift limits and gravity design. These conclusions are in agreement with the results of a recent study involving nonlinear dynamic analysis of a large number of frames designed to EC8 (Sanchez-Ricart and Plumier, 2008). For a typical frame, V_y/V_d normally takes the form indicated in Figure 3 as a function of the normalised elastic response acceleration (S_e/g). Figure 3 is only indicative of possible over-strength ranges as numerical values differ based on various assumptions. 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, due to the considerable reduction allowed in seismic forces coupled with stringent inter-storey drift limits. If drift limits are relaxed, V_y/V_d becomes more dependent on seismic demand, and follows the trends indicated by the dashed lines in Figure 3. 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 noting 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. Figure 3 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 Figure 3), which can then be magnified by α_u/α_1 to obtain an estimate of lateral capacity.

4. BRACED FRAMES

This discussion focuses on simple braced frames in which the diagonals meet the beams at the joints. 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 Part 1.1 of EC8 (2004).

4.1. Capacity Design Requirements

For typical braced frames, 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 (Elghazouli, 2003), the non-dimensionless slenderness ($\bar{\lambda}$) should not exceed 2.0.

For beams and columns, to satisfy capacity design the design axial load ($N_{Ed,m}$) should be determined from:

$$N_{Ed,m} = N_{Ed,G} + 1.1\gamma_{ov}\Omega N_{Ed,E} \tag{4.1}$$

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. 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 $\Omega_i = N_{pl,Rd,i}/N_{Ed,}i$, 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, Ω would not 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).

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, 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 (Elghazouli, 2003). For the frame type considered herein, EC8 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.

4.2. Ductility 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; Broderick *et al*, 2008). The tendency of concentrically-braced frames to form storey mechanisms is a particularly important aspect. 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. 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. This has been examined through nonlinear dynamic simulations in recent studies (Elghazouli, 2003). It is also illustrated in Figure 4 by considering the inelastic static response of a 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: (i) constant area in all braces (i.e. ignoring the EC8 rule); (ii) variable brace areas which are 25% out-of-balance with the capacity demand (i.e. according to the limit in EC8); (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 as necessary.

The main measure examined is the relative bending stiffness of the columns ($\Sigma EI_c/h_c^3$) 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:

$$\beta_k = \frac{L_d \sum I_c}{h_c^3 \cos\phi \sum A_d} \tag{4.2}$$

As shown in Figure 4, 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 Figure 4 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 β_k>0.1. If the 25% requirement of EC8 is met, β_k values needed to attain μ_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. CONCLUSION

The provisions of Eurocode 8 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. For moment frames, capacity-design application rules for columns do not account for the influence of gravity loads on the over-strength of beams. To incorporate this, Ω_{mod} is proposed as a replacement of the code-specified Ω_{EC8} Moreover, column design does not consider the over-strength due to redistribution beyond formation of the first plastic hinge, which can be readily taken into account 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. In the case of 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, 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 relaxed or even removed, if measures related to column continuity and stiffness are incorporated in design.

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