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TOWARDS IMPROVED SEISMIC DESIGN PROCEDURES FOR STEEL STRUCTURES

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Abstract: *European seismic design procedures are currently undergoing a process of evolution and development. This process is guided by improved understanding of structural behaviour based on new research findings, coupled with the need to address issues identified from practical application in real engineering projects. Developments in design guidance however need to balance technical advancements with the desire to maintain a level of stability and simplicity in codified rules. To this end, this paper highlights some of the key changes proposed in the imminent revision of Eurocode 8 with respect to the design of steel structures, with emphasis on moment frames. Several proposed code modifications in terms of behaviour factors, ductility considerations, capacity design verifications, as well as stability-related requirements, are outlined and discussed. It is shown that a number of proposed code changes lead to fundamentally improved seismic performance as well as more rational and efficient design solutions. Nonetheless, further work is needed to address the imbalance in codes of practice between the focus given to capacity design and ductility supply compared to assessing the expected inelastic demands under realistic earthquake loading. To this end, this paper also summarises the main results and observations from recent research studies in which large sets of moment frames were utilised within a series of nonlinear dynamic assessments in order to examine the influence of salient seismic loading characteristics, such as frequency content and duration effects, on inelastic demands. Areas in which further developments are still required to improve the reliability of seismic design procedures are highlighted and discussed.*

Introduction

Following typical seismic design procedures, the design of steel structures in current European standards (CEN, 2004) is based on either non-dissipative or dissipative behaviour. The former is normally limited to areas of low seismicity or special structures (Elghazouli, 2017). Otherwise, economical design is typically sought by employing dissipative behaviour which, apart from for highly irregular structures, is usually performed by assigning a structural behaviour factor to reduce the code-specified forces resulting from idealised elastic response spectra. This is carried out in conjunction with capacity design procedures requiring the provision of sufficient ductility in dissipative zones and adequate over-strength in other regions.

As part of the current process of evolution and development of the structural Eurocodes, significant changes are proposed for the design of steel structures (CEN, 2019). These include additional systems, such as buckling-restrained braces and lightweight steel frame walls, amongst others. New guidance is also included on cyclic testing procedures, design of joints, and load-deformation relationships for use in nonlinear static analysis. Moreover, significant changes to existing design procedures are suggested, particularly in relation to behaviour factors, ductility classes, and drift-related requirements. A number of these proposed modifications are outlined herein, and their implications on seismic behaviour are discussed.

Although significant developments are gradually implemented in seismic codes to improve capacity design procedures and available ductility in dissipative zones, oversimplified approaches are still typically adopted for the prediction and distribution of drift demands. Importantly, due to the complexity and uncertainty in inelastic seismic response, codified approaches do not appropriately account for the inter-dependent relationships between drift demands and ground motion characteristics. This paper therefore also presents selected results from recent investigations which focus on assessing the influence of ground motion characteristics on inelastic seismic demands for moment frames designed to European codes.

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Ductility classes and behaviour factors

A number of fundamental changes are proposed in draft revisions of EC8 (CEN, 2019) compared to the current version (CEN, 2004), which are directly related to ductility classes and behaviour factors. These include: (i) modification of ductility classes from low, medium and high (i.e. DCL, DCM, DCH) to DC1, DC2 and DC3 - the main purpose is, on the one hand, to enable wider use of DC1 without imposing demanding ductility requirements and, on the other hand, to enable more practical use of DC2 and DC3 in which specific ductility and/or capacity design requirements are necessary; (ii) explicit representation of the behaviour factor (q) as the product of q_s (minimum over-strength, assumed as 1.5), q_R (redistribution, or α_u/α_1 representing ultimate to yield capacity) and q_D (reflecting actual demand); and (iii) specification of limits on seismic actions for design to DC1, DC2 and DC3, as opposed to only DCL in the current version. In addition, design seismic action is typically represented in terms of the short-period spectral acceleration (S_α) rather than the design ground acceleration (a_g).

Proposed changes to the limits on behaviour factors and seismic actions have been based on various assessments including recent research investigations, appraisal of previous European guidelines and comparison with current US provisions (ASCE, 2016; AISC, 2016). Table 1 outlines the behaviour factors for selected systems based on current and proposed provisions.

	CEN (2004)			CEN (2019)						
	DCL	DCM	DCH	DC1	DC2			DC3		
	q	q	q	q	q_D	q_R	q	q_R	q	q
Moment Frames	1.5	4.0	$5.0\alpha_u/\alpha_1$ (5.5-6.5)	1.5	1.3-1.8	3.3	2.0-3.5	3.3	1.1-1.3	5.5-6.5
Concentric Bracing										
X or separate	1.5	4.0	4.0	1.5	1.7	1.0	2.5	2.4	1.1	4.0
V-configurations	1.5	2.0	2.5	1.5	1.7	1.0	2.5	2.4	1.1	4.0
Eccentric Bracing	1.5	4.0	6.0	1.5	1.8	1.3	3.5	2.6	1.3	5.0
Buckling Restrained	--	--	--	--	--	--	--	2.8	1.2	5.0
Dual frames										
MRF/CBF	1.5	4.0	4.8	1.5	1.8	1.1	3.0	2.9	1.1	4.8
MRF/EBF	--	--		1.5	2.1	1.3	4.0	3.0	1.3	6.0
MRF/BRB	--	--		1.5	--	--	--	3.0	1.3	6.0
Lightweight frame-wall	--	--	--	1.5	1.0-1.7	1.0	1.5-2.5	1.3-2.0	1.0	2.0-4.0
Inverted pendulum	1.5	2.0	$2.0\alpha_u/\alpha_1$ (2.0-2.2)	1.5	1.3	1.0	2.0	--	--	--

Table 1 Ranges of behaviour factors in current and proposed provisions

A number of observations can be made with respect to the values in Table 1. Firstly, the proposed revisions provide guidance on additional systems which were absent, including buckling restrained braces and lightweight steel frame-wall. The 'q' values differ significantly from current guidance in some cases, such as for V-bracing. It should also be noted that whilst only DCL had a limit on a_g of typically 0.1g (i.e. S_α of about 0.25g), the revised guidance now recommends limits of S_α=0.5g for DC1 (apart from inverted pendulum cases for which the limit of 0.25g is retained). For DC2, higher limits are suggested (0.75g for dual frames, lightweight frame-wall; 0.65g for moment, concentric, eccentric frames; 0.5g for inverted pendulum). No limits are proposed for DC3. These limits were derived based on correspondence with those for seismic design categories in ASCE 7 (2016), and to widen the application of design to DC1 and DC2 in areas of moderate seismicity as well as for complex or special structures.

The main local ductility requirement for steel elements in compression or bending remains through the restriction of the width-to-thickness ratios (c/t or b/t) to avoid or delay local buckling and hence reduce the susceptibility to low cycle fatigue and fracture. The classification used in EC3 (CEN, 2005) is adopted in the current version but with restrictions related to the value of the q factor (DCM: Class 1, 2, 3 for $1.5 < q \leq 2.0$, or Class 1, 2 for $2.0 < q \leq 4$; DCH: Class 1 for $q \geq 4$). The same approach is retained in the proposed revisions, but with slight variations partly to reflect the addition of new systems, as follows: (DC2: Class 1, 2, 3 for $1.5 < q \leq 2.0$; Class 3 and 4 for portal frames, single-storey moment frames, and lightweight systems with $1.5 < q \leq 2.5$; or Class 1, 2 for $2.0 < q \leq 3.5$), (DCH: Class 1 for $q \geq 3.5$; or Class 3 and 4 with $1.5 < q \leq 4.0$ for lightweight systems).

As noted in previous studies (Elghazouli, 2010), comparison between the width-to-thickness limits in EC8 and AISC reveals notable differences for Class 1 compared to 'seismically-compact' limits (λ_{ps}) in AISC. Whilst the limits for flange outstands in compression are similar, there are significant differences for circular (CHS) and rectangular (RHS) hollow sections, which are commonly used for bracing and column 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. This is mitigated to some extent in the proposed revisions, for example, by ensuring that Class 1 should be used for diagonals in CBF systems for DC2 and DC3, in addition to a more stringent diameter-to-thickness limit of 19.4 $\varepsilon/(\gamma_{ov})^{0.5}$, where ε is $(235/f_y)^{0.5}$, in circular hollow sections, and a width-to-thickness ratio of 47.4 $\varepsilon^2/\gamma_{ov}$ in rectangular hollow sections.

Capacity design requirements and material considerations

A key distinction between the proposed ductility classes in the revised draft code (CEN, 2019) is on the ductility and capacity design requirements. In principle, DC1 implies behaviour which is largely elastic with no specific ductility requirements. For DC2, while specific ductility requirements are stipulated for dissipative zones, capacity design of non-dissipative members is applied through global over-strength factors (Ω_{ov}), unlike in current EC8 provisions and more akin to ASCE/AISC procedures. Accordingly, non-dissipative members in DC2 should be verified considering the most unfavourable combination of the axial force (N_{Ed}), bending moment (M_{Ed}) and shear force (V_{Ed}), as follows:

$$
\begin{aligned}
N_{Ed} &= N_{Ed,G} + \Omega_{ov}\, N_{ED,E} \\
M_{Ed} &= M_{Ed,G} + \Omega_{ov}\, M_{ED,E} \\
V_{Ed} &= V_{Ed,G} + \Omega_{ov}\, V_{ED,E}
\end{aligned}
\tag{1}
$$

where $N_{Ed,G}$, $M_{Ed,G}$ and $V_{Ed,G}$ are the actions due to gravity loads, while $N_{Ed,E}$, $M_{Ed,E}$ and $V_{Ed,E}$ are due to the lateral seismic loads, both within the seismic design combination. The proposed values for Ω_{ov} are fixed for each structural system, and are in the range of 1.5-2.0.

In the case of DC3, more detailed capacity design checks are required, using the minimum design over-strength (referred to as $\overline{\Omega}$, which replaces Ω in the current version), noting that such check is required for both DCM and DCH in the current version, but only for DC3 in the revised draft code. Non-dissipative members should hence be verified considering the most unfavourable combination of axial force (N_{Ed}), moment (M_{Ed}) and shear force (V_{Ed}), as follows:

$$
\begin{aligned}
N_{Ed} &= N_{Ed,G} + \gamma_{ov}\gamma_{sh}\, \overline{\Omega}\, N_{ED,E} \\
M_{Ed} &= M_{Ed,G} + \gamma_{ov}\gamma_{sh}\, \overline{\Omega}\, M_{ED,E} \\
V_{Ed} &= V_{Ed,G} + \gamma_{ov}\gamma_{sh}\, \overline{\Omega}\, V_{ED,E}
\end{aligned}
\tag{2}
$$

where γ_{ov} is the material over-strength factor and γ_{sh} is the over-strength factor due to strain hardening. The value of γ_{ov} (a default fixed value of 1.25 in the current EC8) depends on the steel grade (Landolfo, 2013), but not the section type as in US codes (AISC, 2016), and is proposed as 1.45 for S235, 1.35 for S275, 1.25 for S355 and 1.2 for S460. On the other hand, γ_{sh} replaces the factor of 1.1 employed in the current version, with a range of 1.1-1.8 (relatively low for dissipative axial members and high for dissipative shear elements). It should be noted that the combined effect of $\gamma_{ov}\gamma_{sh}$ can be significantly higher than in the current version, depending on the system and steel grade (i.e. up to 2.610 compared to 1.375).

Other inadequacies in capacity design procedures, pointed out in previous assessments (Elghazouli, 2010) have also been addressed in the proposed revisions. For example, it was shown that the design over-strength parameter (Ω=M_{pl}/M_{Ed}), as adopted in EC8 for moment frames, involves a major approximation as it does not account accurately for the influence of gravity loads. This issue 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 which the beam over-strength may be significantly underestimated. In the proposed draft, this has been corrected by specifying $\overline{\Omega}$ as M_{pl}-$M_{Ed,G}/M_{Ed,E}$. Other issues raised with respect to various structural systems such as for concentric and eccentric bracing, as summarised elsewhere (Landolfo, 2013), have also been addressed in the proposed provisions.

Lateral over-strength and drift-related requirements

The lateral over-strength exhibited by the structure can have a significant influence on seismic response. There are several sources that can introduce over-strength (Elghazouli, 2010). Most notably, over-strength is often a direct consequence of the application of drift-related requirements or inherent idealisations, particularly in the case of moment frames, as well as simplifications within the design approaches and procedures, such as in the case of concentrically braced frames.

Many of the drawbacks associated with significant and unintended over-strength have been addressed in the proposed code revisions. For example, it has been shown that, in comparison with US and other provisions, drift-related requirements in the current EC8 are significantly more stringent compared to other codes (Elghazouli, 2010). This is particularly pronounced in relation to the stability coefficient θ which, unlike in US provisions, is strongly dependent on the inelastic response and the behaviour factor. As a consequence of the strict drift and stability requirements and the relative sensitivity of framed structures to these effects, particularly in moment frames, they can often govern the design leading to considerable over-strength, especially if a large behaviour factor is assumed. This over-strength, represented as the ratio of the actual base shear V_y to the design value V_d, as shown in Figure 1, is also a function of the normalised elastic spectral acceleration (S_e/g) and the gravity design (Elghazouli, 2010).

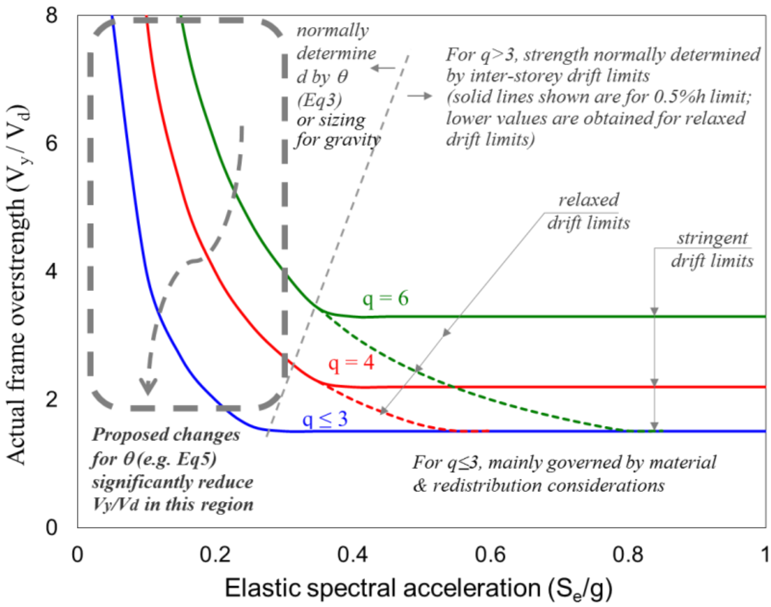

Figure 1 Typical expected levels of over-strength in moment frames

The presence of over-strength reduces the ductility demand in dissipative zones, and also affects the forces imposed on other frame and foundation elements. A rational application of capacity design necessitates a realistic assessment of lateral capacity 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, it should be noted that such a choice is often unnecessary and could lead to undesirable effects.

Apart from including inter-storey drift limits for various limit states in the proposed revisions, as opposed to damage limitation only in the current version, the issue of over-strength introduced through the stability coefficient θ is addressed by modifying the current representation of:

$$\theta = \frac{P_{tot} d_r}{V_{tot} h} \tag{3}$$

where P_{tot} is the total gravity load at a storey, d_r is the design inelastic inter-storey drift at the significant damage limit state, V_{tot} is the total seismic storey shear, and h is the storey height.

In order to address the resulting over-strength, the revision proposes a modified relationship, which accounts for the inherent over-strength and redistribution, as follows:

$$\theta = \frac{P_{tot} d_r}{q_S q_R V_{tot} h} \tag{4}$$

In the case of DC3 steel structures, θ can instead be determined as:

$$\theta = \frac{P_{tot} d_r}{\gamma_{ov} \bar{\Omega} q_R V_{tot} h} \tag{5}$$

With reference to Figure 1, by modifying θ to account for the actual over-strength that exists in the structure, a significant source of over-strength would be eliminated or significantly reduced, particularly for moderate levels of spectral design accelerations. This depends on a number of considerations including the gravity design situation, the steel grade and strain hardening factors, and the drift limits adopted. Overall, the revised draft provisions succeed to a large extent in reducing the levels of unintended over-strength in framed structures.

Global and local drift demands

Whilst drift related requirements are key to seismic design and have a direct influence on the resulting performance, oversimplified approaches are typically adopted for the prediction and distribution of drift demands in seismic codes. In order to examine the main parameters influencing the inelastic drift, a large set of steel moment frames were designed to the provisions of EC3 and EC8 in recent studies (Kumar et al, 2013; Elghazouli et al, 2014; Tsitos et al, 2018; Bravo-Haro et al, 2018). Seismic design was carried out using various combinations of peak ground acceleration, soil conditions, and drift limits. Over 50 frames with heights of 3, 5, 7 and 9 stories were considered. European steel profiles were used for the members. The frames were modelled within the finite element program OpenSees (McKenna, 2011). A large suite of over 70 far-field ground motion records was also considered. The Mean Period (T_m) proposed by Rathje et al (1998) was chosen as a frequency content measure. This is determined as the weighted mean of periods of the Fourier Amplitude Spectrum (FAS) over a pre-defined frequency range, where the weights are assigned based on the Fourier amplitudes.

Parametric studies were performed to assess the global and inter-storey drift demands. Incremental dynamic analyses (IDA) were carried out by scaling the records with respect to the fundamental period (T_1) to attain various levels of relative intensities represented by the effective behaviour factor (q'), which would be similar to the design behaviour factor (q) in the absence of significant structural over-strength (Elghazouli, 2010). It is also worth noting that for moment frames with D3 dissipative bending elements in S355 steel, the influence of proposed code changes (CEN, 2019) is typically insignificant since q' is used herein. The scaling factor (SF) required for an individual record to attain a given q' level is determined as:

$$S_F = q' \times \frac{V_1}{S_a(T_1) \times m \times \gamma} \tag{6}$$

where S_a (T_1) is the spectral acceleration of a given record at T_1; V_1 is the base shear corresponding to the formation of first yield, as obtained from static pushover analysis using a force profile based on the fundamental mode shape; m is the seismic mass of the structure; and γ represents the mass participation ratio corresponding to the first mode.

The ground motions were scaled in order to achieve four q' levels of: 3, 4, 5 and 6. For each analysis, the maximum roof displacement (Δ_{max}) and the maximum inter-storey drift (θ_{max}) were recorded. The results from each analysis were then processed to determine the global drift modification factor (δ_{mod}) as well as the maximum drift modification factor (θ_{mod}).

δ_{mod} is the ratio of maximum roof displacement (Δ_{max}) (recorded from IDA for a given q') to the product of q' and the roof yield displacement ($\Delta_{1,roof}$) (i.e. roof displacement at first yield obtained from pushover analysis using a force profile based on the fundamental mode shape), as follows:

$$\delta_{mod} = \frac{\Delta_{max}}{q' \times \Delta_{1,roof}} \tag{7}$$

On the other hand, θ_{mod} is the ratio of the maximum inter-storey drift (θ_{max}) (from IDA for a given q') to the product of q' and the maximum inter-storey drift at the formation of first yield ($\theta_{1,max}$) (from pushover analysis using a force profile based on fundamental mode shape), as follows:

$$\theta_{mod} = \frac{\theta_{max}}{q' \times \theta_{1,max}} \tag{8}$$

The IDA analysis revealed that the parameters that have the most significant influence on δ_{mod} and θ_{mod} are T_1/T_m and q', as depicted in Figures 2 and 3. Prediction relationships for δ_{mod} and θ_{mod} can be readily derived, as also indicated by the solid lines in the figures. These are able to capture the main trends, exhibiting a plateau that depends on q' for T_1/T_m ranging between unity and either about 2.8 (for δ_{mod}) or 1.7 (for θ_{mod}). The results also offered information that enables optimization of the relative stiffness of the different storeys, as a function of T_1/T_m in order to achieve a closely uniform distribution of inter-storey drifts over height (Elghazouli et al, 2014).

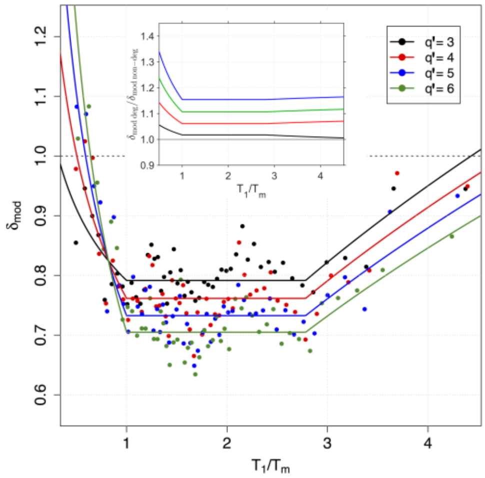

Figure 2 δ_{mod} vs T_1/T_m for various q' levels

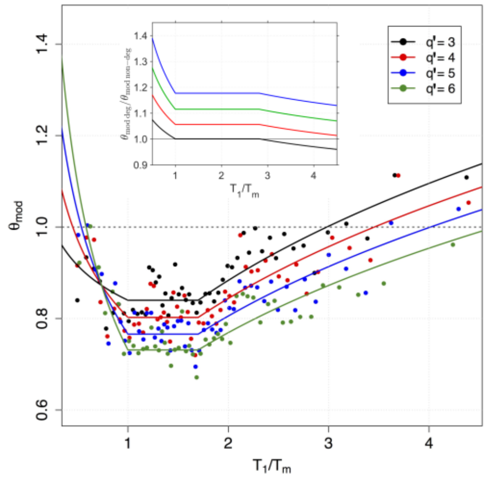

Figure 3 θ_{mod} vs T_1/T_m for various q' levels

For the estimation of drift demands, EC8 typically adopts the equal displacement rule, implying that both δ_{mod} and θ_{mod} are unity. On the other hand, US provisions propose seismic drift amplification factors (C_d). For ordinary moment frames (OMF), intermediate moment frames (IMF) and special moment frames (SMF), R (force reduction) factors of 3.5, 4.5 and 8.0, respectively, are proposed, and corresponding values of 3.0, 4.0 and 5.5 are suggested for C_d. Accordingly, δ_{mod} and θ_{mod} for OMF, IMF and SMF are 0.86, 0.89 and 0.69, respectively. When this is considered with respect to the results in Figures 2 and 3, it becomes evident that EC8 criteria are highly conservative except for short period ratios with large q' levels. On the other hand, US provisions are relatively close to predictions for intermediate period ratios, but under-predict demands for relatively low or relatively high T_1/T_m. More generally, the comparisons emphasise the oversimplified nature of inelastic drift demand criteria in design codes, which do not typically account for the influence of period ratios (nor behaviour factors in the case of EC8).

For design purposes, prediction relationships that account for T_1/T_m and q' can be proposed (Kumar et al, 2013; Bravo-Haro et al, 2018). The parameters adopted within such relationships can all be determined as part of the typical design process. The only exception is the need to determine T_m, although this was shown to be closely related to predefined spectral values (Elghazouli et al, 2014). Such procedures would lead to a significant enhancement in the reliability of design approaches. It is also important to note that Informative Annex B of the current EC8, which is primarily proposed for push-over analysis and assessment procedures, suggests an increase in target displacements for an equivalent SDOF system using a bi-linear response idealisation for T_1/T_c below unity. This approach goes some way towards addressing the increase in demand for short-period structures.

The above studies were extended using plastic hinge elements that employ hysteretic models that can describe deterioration phenomena (Ibarra et al, 2005; Lignos and Krawinkler, 2010). Several degradation calibration parameters, which are primarily related to local instability in steel sections and depend mainly on cross-section slenderness and loading conditions can be used. Inelastic demands were assessed considering both degrading and non-degrading models. Considering δ_{mod}, the effect of deterioration modelling proved to be more significant for T_1/T_m less than 1.5 and greater than 3.5, as shown in the inset sub-plot in Figure 2. However, in the medium T_1/T_m range, the effect of degradation led to 8%-16% higher response, proportional to the specified level of inelasticity q'. In terms of θ_{mod}, similar observations could be drawn, as depicted in the inset sub-plot in Figure 3. However, for mid-range T_1/T_m, consideration of deterioration led to a more significant amplification of θ_{mod} of 10%-30%, depending on the value of q'. Again, the influence of degradation increased along with q'. Based on these findings, relationships representing δ_{mod} and θ_{mod} can readily be modified to account for the increase in demand caused by cyclic degradation effects (Bravo-Haro et al, 2018).

Importantly, the IDA analysis revealed the increased susceptibility of the degrading systems to dynamic instability and collapse, compared to non-degrading cases as shown in Figure 4. The figure shows typical fragility curves for an example sub-set of frames for exceedance of code compliant limit states DL (first plastic hinge), SD (rotation of 0.025 rad in plastic hinge) and NC (rotation of 0.040 rad in plastic hinge) for both degrading and non-degrading cases (Bravo-Haro et al, 2018). The onset of deterioration phenomena also led to concentration of inter-storey drift demands in the lower levels, as illustrated in Figure 5. The figure shows median profiles of maximum inter-storey drifts for an example sub-set of frames for both degrading and non-degrading cases. Association of the response with code-compliant performance limit states showed that the degrading frames could reach the specified limits at seismic intensities up to 30% lower than their non-degrading counterparts. For optimum design cases, where over-strength would be minimal, the degrading structures might reach a ‘near-collapse’ state at ductility demand levels comparable or lower than the assumed design behaviour factor. Nevertheless, actual simulated collapse would occur at even higher demand levels.

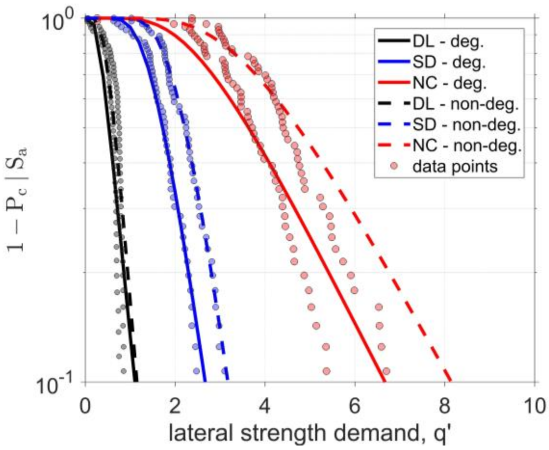

Figure 4 Fragility curves including degradation

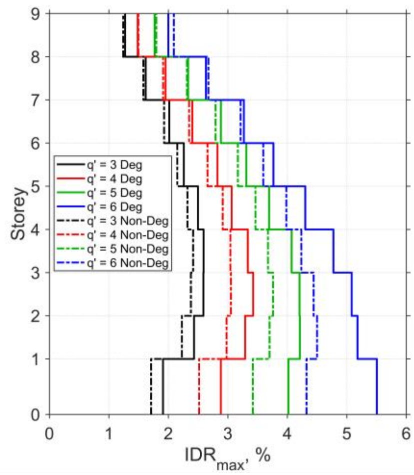

Figure 5 Distribution of median θ_{mod}

The IDA results showed that although modelling deterioration effects is typically considered as an unnecessary complexity for assessing design-level behaviour, its influence on inelastic drift demands can be significant. The response at local level, such as rotational demands in dissipative zones, could also be directly affected by cyclic degradation. Moreover, the onset of deterioration can have a direct impact on the global plastic mechanism and can often lead to concentration of seismic demands, resulting in considerable deviations from assumed uniform distributions along the height. Whilst the importance of degradation effects are usually recognised for collapse level evaluations, there seems to be a need to incorporate the influence of degradation effects in other design-level assessments.

Residual drift demands

The studies discussed above were also extended to examine the residual drift levels in the structures considered. The maximum residual drift demand, determined at any storey, was found to be typically influenced by the number of stories, the level of lateral strength demand, the degradation effects and the post-yield stiffness ratio (Bravo-Haro and Elghazouli, 2018a). Depending on whether the prediction of residual or peak drifts are carried out for design or assessment purposes, a ‘direct approach’ or an ‘indirect approach’ can be followed, and for which prediction relationships for global and inter-storey permanent drifts were proposed.

In the case of the ‘indirect approach’, the permanent drifts are determined as a function of the peak drifts, and the most relevant observation obtained from the IDA analysis is that, on average, residual drifts demands are typically about 30-40% of the maximum drift demand, for levels of demand (i.e. q') of 3-6 respectively, for all sets of frames. This is illustrated in Figures 6 and 7, depicting the mean maximum residual ratio (Max SRDR) which represents the residual-to-peak drift ratio, for both non-degrading and degrading cases, respectively.

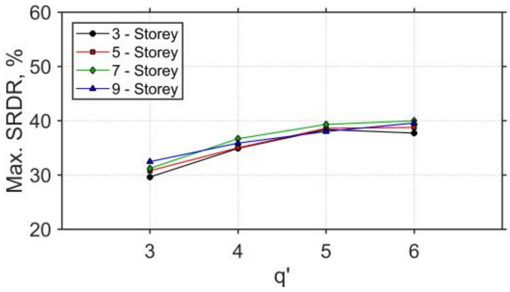

Figure 6 Max SRDR (non-degrading)

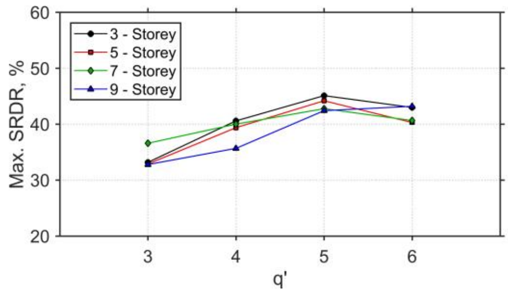

Figure 7 Max SRDR (degrading)

Overall, the IDA indicated that the structures experienced significant permanent residual drift, for all levels of demand, with an average amplitude between 0.7% and 2.5% for q' of 3 and 6, respectively. These values are both perceptible to occupants and would involve a high repair cost. If a threshold of 0.5% is assumed to represent a total loss, this would indicate that moment frames designed according to EC3 and EC8 have a high likelihood of incurring irreparable damage from an economic perspective. Probabilities of exceeding this limit were found to be on average between 66% and 91% for q' of 3 and 6, respectively, in the case of degrading structural systems. However, these findings need to be considered alongside the high record-to-record variability, and the frame over-strength which may exists when non-optimised design procedures based on the current EC8 are used, noting that proposed code provisions would result in significantly lower over-strength levels. Importantly, the IDA results point to the need for specific guidelines in seismic codes with respect to residual drift criteria. It also reinforces the importance of employing refined models which are capable of capturing degradation phenomena in order to obtain reliable predictions of both peak and residual drift demands.

Ground motion duration effects

In situations in which degradation effects are significant, ground motion duration can play a significant role in the resulting demand. The above studies were therefore extended to examine the influence of ground motion duration on seismic demands (Bravo-Haro and Elghazouli, 2018b). Whilst there are various approaches for quantifying ground motion duration (Bommer and Martinez-Perira, 1999), the significant duration (Trifunac and Brady, 1975) was used herein to characterise the ground motion records, as several studies indicated its suitability for assessing the performance of structural systems. Two paired sets of spectrally equivalent short and long records were selected (Bravo-Haro and Elghazouli, 2018b). A matching process was performed in order to minimise the mean squared error of the 5%-damped scaled linear response spectra between short and long records. Overall, over 70 pairs of scaled records, with identical mean response spectra, were prepared for this purpose.

As expected, in the presence of degradation, the inelastic drifts increase for longer significant duration of ground motion. Selected results illustrating the influence of ground motion duration on 4 steel multi-storey frames, subjected to IDA to develop collapse fragility curves, are shown in Figure 8. The 4 structural systems represent typical steel moment frames designed to European code procedures, comprising 4 different heights, namely 3, 5, 7 and 9 stories.

It is evident from Figure 8 that in all cases the probability of collapse is higher for the long duration set (blue lines) compared to the short duration set (red lines), for comparable spectral acceleration (S_a, 5%) values. The decrease in median collapse capacity due to the duration is shown individually in the plots. On average, the collapse capacity decreased by about 17% when the long duration records were employed.

In fact, for a broader range of fundamental periods, reduction of up to 40% in the collapse probabilities could be observed, highlighting the inadequacy of typical seismic performance based assessment procedures which are largely based on short duration records. The influence of duration was shown to be also significant for lower levels of performance, typically associated with design, particularly when high rates of cyclic degradation levels were used. Overall, the IDA results have emphasised the importance of considering the influence of ground motion duration in seismic assessment and design procedures, and the need to account for degradation effects in order to capture key response characteristics that are otherwise typically disregarded.

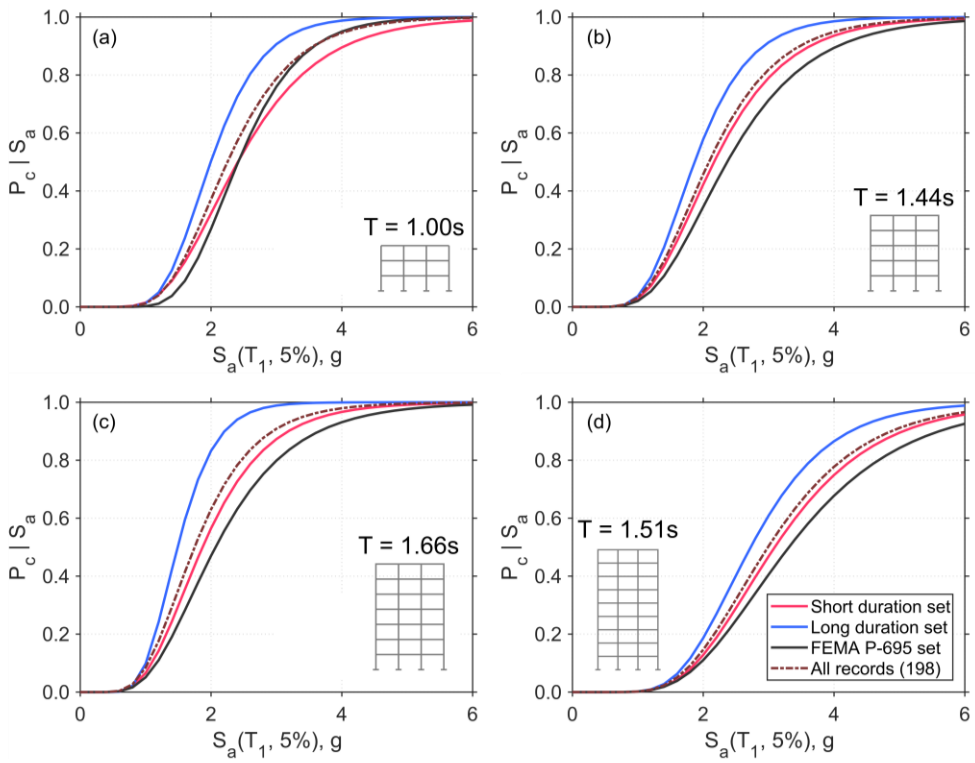

Figure 8 Collapse fragility curves for frames: a) 3-storey, b) 5-storey, c) 7-storey, d) 9-storey

Concluding remarks

This paper has highlighted several key changes proposed for the design of steel structures to EC8 as part of the current process of evolution of the structural Eurocodes. Particular focus was given to modifications to behaviour factors, ductility classes, and stability-related requirements. A number of the proposed changes lead to fundamentally improved performance as well as more rational and efficient design solutions, and to a higher degree of consistency with US provisions. Additional systems, such as buckling-restrained braces and lightweight steel frame walls, are incorporated. The revised provisions also include much-needed guidance on testing procedures, design of joints, and load-deformation relationships for use in pushover analysis.

Despite the significant developments in codified procedures, over-simplified approaches are still typically adopted for predicting demands, which rely solely on the design behaviour factor. The findings of a number of recent studies, aiming at improving current procedures for predicting inelastic seismic demands, were summarised. These capture the influence of frequency content of ground motion, represented through the ratio of mean period of ground motion to the fundamental period of the structure. When cyclic degradation effects are considered, the drift demands can increase by up to 30%. Although deterioration modelling is typically considered an unnecessary complexity for the design-level, and is usually recognised only for collapse level evaluations, its influence on drift demands can be significant. The onset of deterioration can have a direct impact on the global plastic mechanism and can often lead to concentration of seismic demands, resulting in considerable deviations from assumed uniform distributions.

In addition to peak demands, a prediction of residual drifts may also be required for assessment and design. It was found that, on average, residual drifts demands are typically about 30-40% of the maximum drift demand. For structures designed to EC8, significant residual drifts in the range of 0.7-2.5%, depending on the lateral strength demand, were obtained. These values are both perceptible to occupants and would involve a high repair cost. Importantly, the results point to the need for specific guidelines with respect to residual drift criteria. It also reinforces the importance of employing refined models which are capable of capturing degradation phenomena in order to obtain reliable predictions of both peak and residual drift demands.

In situations where degradation effects are significant, ground motion duration can also play a significant role in the resulting demand. Using paired sets of spectrally equivalent short and long records, it was found that the inelastic demand can significantly increase in the latter and, as a result, lead to a considerable reduction in the collapse capacity. The influence of duration was also found to be significant for lower levels of performance, typically associated with design, particularly when high rates of cyclic degradation levels were used. Overall, in addition to frequency content, the results emphasise the importance of considering the influence of duration in assessment and design procedures, and the need to account for degradation in order to capture key response characteristics that are otherwise typically disregarded.

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