



An overview on the damageability issue in the design of steel Moment Resisting Frames

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Abstract: This paper is initially associated with critical comparisons of Steel Moment Resisting Frames using European and American code methodologies, wherein capacity design philosophy is recommended for the design of steel frames and in which steel is used smartly and efficiently, thereby ensuring the assumed ductility of the system. Interested philosophical concepts have been laid down which are based on the observed issues during the design phase of moment resisting frames. After the application of capacity rules, the primary concern for most of the designers is the lateral stiffness of the frames. Therefore the vital influential parameters of these codes are the response modification factors/behaviour factors, overstrength factors and drift limitations. This research is aimed to disseminate the awareness about the discrepancy between damageability and ductility and thus to mitigate the complexities that exist as hidden inconsistencies during the design phase of MRFs. The here presented philosophical concepts and ideas could be handy and interesting for the Technicians involved in the design of Steel MRFs. Few suggested values are presented at the end of the paper in diminishing challenges while making deciding for ductility class and drift limit.

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1. Introduction

Assuring ductility by respecting stringent damageability rules for achieving safe and economic design of steel MRFs is a challenging task in today's seismic codes. Theoretical backgrounds are easy to provide but practical implementations and the desired objectives are cumbersome to achieve. Stringent limits, complicated rules, and a lot of perceptions many times lead to complexity of the system and make the task more laborious. Modern seismic codes are very advanced and often the designer apparently follows correctly the rules and the prescriptions but without achieving an optimum solution, as occasionally the rules are not compatible with each other. Consequently, when satisfying these rules oversizing of structural members take place, generally yielding un-economical solution.

Moment resisting frames are assumed to be highly ductile and therefore are believed to be more beneficial than other lateral load resisting systems in seismic conditions if designed properly; they are capable of extensive yielding having reasonable plastic deformations without significant loss of strength. In order to achieve the desirable mechanisms with a predicted ductility, modern

building codes prescribe precise capacity design rules. These rules are quite simple to understand and hence are easily applied by the Technicians. Unluckily, steel MRFs are often governed by deformability and damageability requirements, hence mixing strength and stiffness with a strong complication of the design process.

This paper aims at addressing background of European and American seismic codes for MRFs. It will be useful for the technicians in order to better pre-decide a relevant drift limit and behaviour factor which in the end could be fulfilled and remain compatible with the ductility class [1].

2. Seismic zonation of Pakistan

Steel moment resisting frames perform well during earthquakes if designed properly and efficiently. As an example, before the independence of Pakistan from the British in 1947, structures designed according to the recommendations provided by Engr. Kumar resisted the 1935 Quetta earthquake without significant damage. These structures were designed after the 1931 Mach earthquake and thus tested by the 1935 Quetta earthquake and evidenced

that even a modest design of steel structures saved lives in such earthquakes [2].

The trend of designing steel structures is time-of-need, and being motivated in Pakistan lying in a high seismic region; the October 2005 earthquake is listed high among many that occurred. Building Code of Pakistan (BCP) with the aid of “UBC 97” [3] is the reference code that is adopted in the Country and is recommended by Earthquake Reconstruction and Rehabilitation Authority. The seismic zones as shown in Figure 1 that are assumed by BCP are mentioned in Table 1 considering five seismic zones. Since it is believed and recommended by the design community that steel structures perform well during earthquake, therefore it is of high interest to the possibilities of applying more advanced design procedures in Pakistan. Seismic design procedures, accurate seismic zoning, and related earthquake design codes are the topics of high concern in Pakistan [4, 5].

Since Pakistan is a developing country, it is quite worth mentioning that the previous quakes have influenced the infrastructure massively in the corresponding areas resulting in mounting number of casualties along with notable damage to the building structures.

Many studies have been devoted to the comparisons of various seismic design codes, e.g. on the comparison of American and Japanese [6], European and Japanese [7] with overview on Japanese code [8] for the design of steel structures. Further, Elghazouli [9] extensively contributed in the assessment of European seismic design procedures and philosophies for several lateral load resisting systems, especially focusing moment resisting frames due to their paramount inelastic behaviour. Interesting studies have been presented by Paul et al. [10, 11] who provided state-of-the-art review of seismic design of steel moment resisting frames in U.S. and Indian codes.

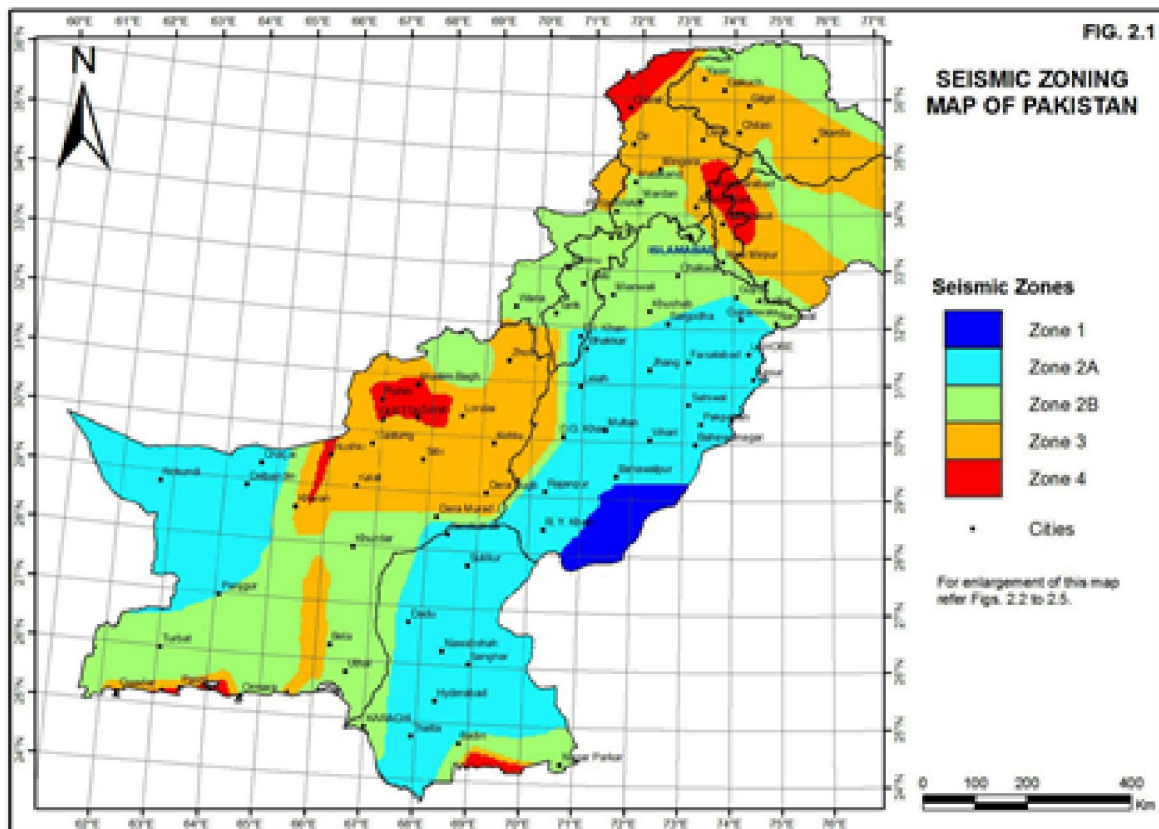


Figure 1: Seismic zoning map of Pakistan

Table 1: Seismic zonation of Pakistan

S. No	Seismic zone	Horizontal Peak Ground Acceleration	Hazard	Damage	Damage cost
1	1	0.05 to 0.08g	Low	Negligible	Low
2	2A	0.08 to 0.16g	Moderate	Minor	Medium
3	2B	0.16 to 0.24g		Moderate	
4	3	0.24 to 0.32g	Large	Severe	High
5	4	>0.32g	Severe	Collapse	Huge

NOTE: Where “g” is the acceleration due to gravity. The acceleration values are for Medium hard rock (S_B) site condition with shear wave velocity (v_s) of 760 m/sec.

3. Performance based approach in seismic codes

For a strong earthquake, the choice and the use of a performance objective for a building structure in terms of safety and economy is the most vital and sophisticated task. In the last decades, the most advanced seismic codes and provisions have been based on the performance based design (PBD) approach. These days, it is common for the vendor to decide the performance objective of the structure keeping in view the budget of the owner, the life time of the structure (considered earthquake) and most importantly the life safety of the occupants. This has led structural engineers to develop structural design techniques that are capable of delivering a predictable performance during a specified earthquake. The three integral steps of PBD are the estimation of hazard, the evaluating vulnerability, and the computation of consequences, shown schematically in Figure 2 [12]. Increasing the hazard level (PGA), thus the damage level increases and thereby causing simultaneous increase in damage and vice versa.

In EC8 [13] two basic seismic design levels namely ‘no-collapse’ and ‘damage-limitation’ are

presented. For the ‘no collapse’ design level, seismic actions are based on a recommended probability of exceedance of 10% in 50 years (return period of 475 years) which corresponds to a rare earthquake. On the contrary the later relates to a recommended probability of 10% in 10 years (return period of 95 years). AISC [14, 15] considers the FEMA recommendations [16-19], where performance levels are defined as: i) Immediate Occupancy (IO), where only minor structural damage may occur and corresponds to a recommended probability of exceedance of 50% in 50 years; ii) Life safety (LS), where probable structural damage is tolerable with no collapse and with minimal falling hazards, with a recommended probability of exceedance of 10% in 50 years; iii) Collapse prevention (CP), which is a post-earthquake state where complete or near complete collapse of the structure takes place, corresponding to a recommended probability of exceedance of 2% in 50 years. Moreover, an additional limit state is referred to as iv) Operational limit state wherein members do not yield at all.

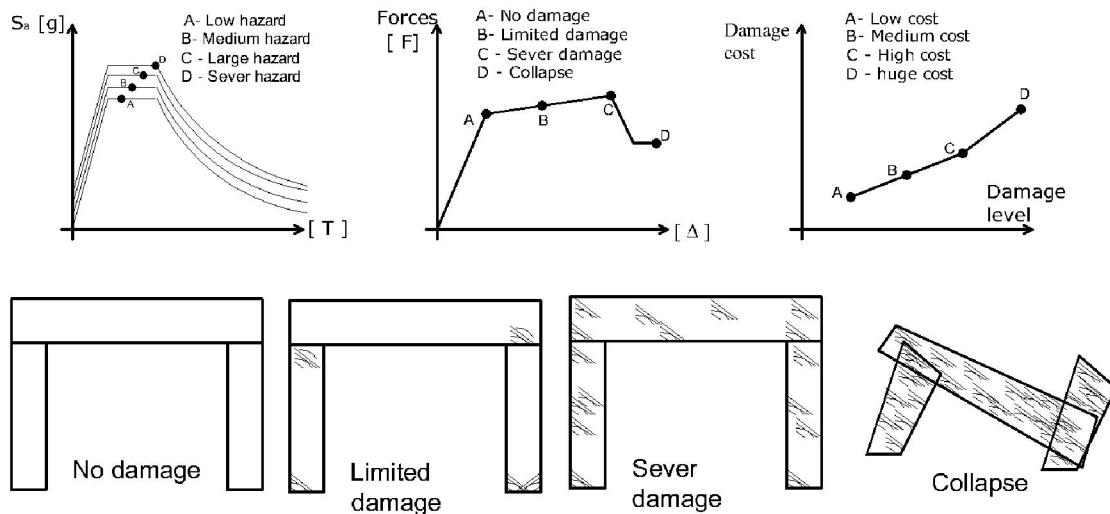


Figure 2: Steps of performance based design

4. European and American design Approaches

Comparison of the capacity design rules in Eurocodes ([20]-[21]) versus AISC-ASCE [14]-[22], the noticeable prescriptions for designing MRF provided by the relevant codes are put forth briefly in the synoptic comparative scheme given in Table 2 [23].

According to Table 2, *DCL* is Ductility Class Low, *DCM* is Ductility Class Medium and *DCH* is Ductility Class High; *SMF* is Special Moment resisting Frames, *IMF* is Intermediate Moment resisting Frames and *OMF* is Ordinary Moment resisting Frames. These abbreviations are henceforth used to describe the various parameters that govern

the design of MRF in the current paper. In EC8 as mentioned in Table 2, the multiplier α_{nl}/α_1 with behaviour factor (q) shape the redundancy factor. In evaluating Strong Column Weak Beam (*SCWB*) criteria as mentioned in Table 1 of EC8, ΣM_{Rc} and ΣM_{Rb} are the sum of the design values of moments of resistance framing the joint of the columns and beams, respectively. However, in *SCWB* criteria of AISC, ΣM_{pc}^* is the sum of moments in the column above and below the joint at the intersection, and ΣM_{pb}^* is the sum of moments in the beams at the intersection of the beam and column centrelines as defined by AISC.

Table 2. Seismic related factors and checks

<i>Description</i>	<i>Eurocodes (EC3/EC8)</i>	<i>AISC/ASCE</i>	<i>Remarks</i>
Energy dissipation philosophy	Prescribed by means of DCL, DCM and DCH	Given by OMF, IMF and SMF	<i>IMF and OMF are restricted to limited heights in high seismic categories</i>
Seismic load reduction factor	A behaviour factor (q) equal to 4 for DCM and $5\alpha_w/\alpha_l$ for DCH is provided	A response modification factor (R) equal to 4.5 for IMF and 8 for SMF is given	<i>An almost same criterion is considered</i>
Cross section limitations	For $q > 4$ only class 1 sections are allowed, for $2 < q \leq 4$ class 1 and class 2 and for $1.5 < q \leq 2$ class 1, 2 and 3 are allowed	Limits λ_p to λ_{ps} , i.e. to use seismically compact section and is obtained by modified slenderness ratio	<i>Class 1 and seismically compact sections are unaffected by local buckling</i>
Rotation capacity (local ductility concept)	Plastic hinge rotation is limited to 35 mrad for structures of DCH and 25 mrad for structures of DCM	SMF and IMF are designed to accommodate plastic hinge rotation of 30 mrad and 10 mrad , respectively with inter-storey drifts in the range of 0.04 and 0.02 radians, respectively	<i>For high seismicity it is recommended by both codes to apply ductility concept</i>
Overstrength factor	$\Omega = \frac{M_{pl,Rd,i}}{M_{Ed,i}}$	Ω_o equal to 3 for MRFs	Ω_o in EC8 is $(1.1\gamma_{ov} \Omega)$
Strength checks for dissipative elements (Beam checks)	$\frac{M_{E,d}}{M_{pl,Rd}} \leq 1.0, \frac{N_{E,d}}{N_{pl,Rd}} \leq 0.15, \frac{V_{E,d}}{V_{pl,Rd}} \leq 0.5$	No additional checks are required except strength checks using AISC/LRFD	<i>Additional checks to be carry out for the seismic conditions</i>
Non dissipative elements (e.g. Columns checks in MRFs)	$N_{Ed} = N_{Ed,G} + 1.1\gamma_{ov}\Omega N_{Ed,E}$ $M_{Ed} = M_{Ed,G} + 1.1\gamma_{ov}\Omega M_{Ed,E}$ $V_{Ed} = V_{Ed,G} + 1.1\gamma_{ov}\Omega V_{Ed,E}$	Verification of strength with loads computed from special load combinations having Ω_o	<i>Stability checks are normally employed for these conditions</i>
Strong column weak beam (SCWB) philosophy	$\sum M_{Rc} \geq 1.3 \sum M_{Rb}$	$\frac{\sum M_{pc}^*}{\sum M_{bc}^*} \geq 1.0$	<i>EC8 accounts 1.3, while AISC considers a factor $1.1R_y$ to increase the nominal beam strength</i>
Panel Zone philosophy	Strong-PZ with weak beam is recommended	Both weak/intermediate or strong PZ with weak beam are allowed	<i>Intermediate PZ is preferred in order to have high dissipative capacity</i>
Panel Zone (PZ) (Stability check)	$\frac{h_w}{t} \leq \frac{72\varepsilon}{\eta}$ with $\varepsilon = \sqrt{\frac{235}{f_y}}$ where f_y is in Mpa, and η is a factor with 1.2 as recommended value.	$t \geq \frac{d_z + w_z}{90}$ where d_z , w_z and t are length, width and thickness of PZ respectively	<i>EC8 refers to EC3 for stability check of PZ. [24]</i>

Table 3. Deformability related parameters and checks

Description	Eurocodes (EC3/EC8)	AISC/ASCE	Remarks
Second order effects	A simplified procedure is allowed by amplifying computed seismic forces and displacements by a factor $1/(1-\theta)$, where $\theta = \frac{P_{tot} \times d_r}{V_{tot} \times h}$ but $0.1 < \theta \leq 0.2$. In any case, θ may not exceed 0.3	$\theta = \frac{P_x \times \Delta}{V_x \times h_{sx} \times C_d}$ and $\theta_{max} = \frac{0.5}{\beta C_d}$ if $\theta > 0.1$, use θ_{max} , where β is the ratio of shear demand to shear capacity (conservatively it can be taken as 1.0)	The factor θ is used to classify the structures into sway and non-sway frames
Drift philosophy (Reduction)	Spectrum is reduced by 2.0 and 2.5 for importance classes I & II, and III & IV, respectively	Reduction factor is $(C_d/R)(5.5/8=1.45)$ for SMF and $(4.5/4=1.125)$ for IMF	Overall EC8 check for drift is more stringent
Drift criteria for MRFs (Limit)	$0.005h$, $0.0075h$ and $0.01h$, where h is the storey height	$0.02h$, where h is the storey height	

For checking second order criteria, described in Table 3 in the European formulation, P_{tot} is the total vertical load acting on the level under consideration; d_r is the design storey drift resulting from V_{tot} , where V_{tot} is the total seismic storey shear force, h is the inter-storey height. In AISC-ASCE, the C_d factor is brought in which is referred to as deflection amplification factor, while Δ is the storey drift resulting from V_x , V_x is seismic shear acting between levels x and $x-1$ and h_{sx} is the story height below level x , P_x is the total gravity load at and the above storey in the seismic design scenario.

5. Criticisms on code provisions

The definition of overstrength factor (Ω) differs in the two codes. As far as American codes are concerned, overstrength is a fixed value (3.0) for steel MRFs, inspite of a complete different philosophy by Eurocode 8. As the definition of overstrength factor is directly related to the strength of beams, Ω is defined by EC8 as the ratio of “plastic moment of beams” to “the internal moment in the beams arises due to the seismic condition” as given by Eq (1).

$$\Omega = \frac{M_{pl,rd,i}}{M_{Ed,i}} \quad (1)$$

$M_{Ed,i}$ is the design value of the bending moment in the i^{th} beam in the seismic design situation and $M_{pl,rd,i}$ is the corresponding plastic moment.

Furthermore when dealing with flexible frames, beams are normally more influential than columns while satisfying the damageability criteria. Therefore, the recommended overstrength drastically increases when the stringent limit (e.g. $0.005h$) is used. If beams are not influenced by the drift limits, the overstrength factor is normally less than 3.0. In cases when the overstrength factor is attained from strength governing design, and if it is less than the one recommended by AISC/ASCE (3.0), the design of frames using Eurocode 8 could be convenient as it might give an economical solution, comparatively. The problem may confront when the overstrength factor exceed 3.0, where column size may increase drastically and could probably lead to overdesign [25], which often occurs when the design is dictated by drift limitations.

In American code Ω for SMF and IMF is 3.0; the R factor for SMF is 8, whereas it is 4.5 for IMF. The ratio of these two factors represents the reserve overstrength; these are given by Eq (2) and Eq (3) for SMF and IMF, respectively, and are shown in Figure 3:

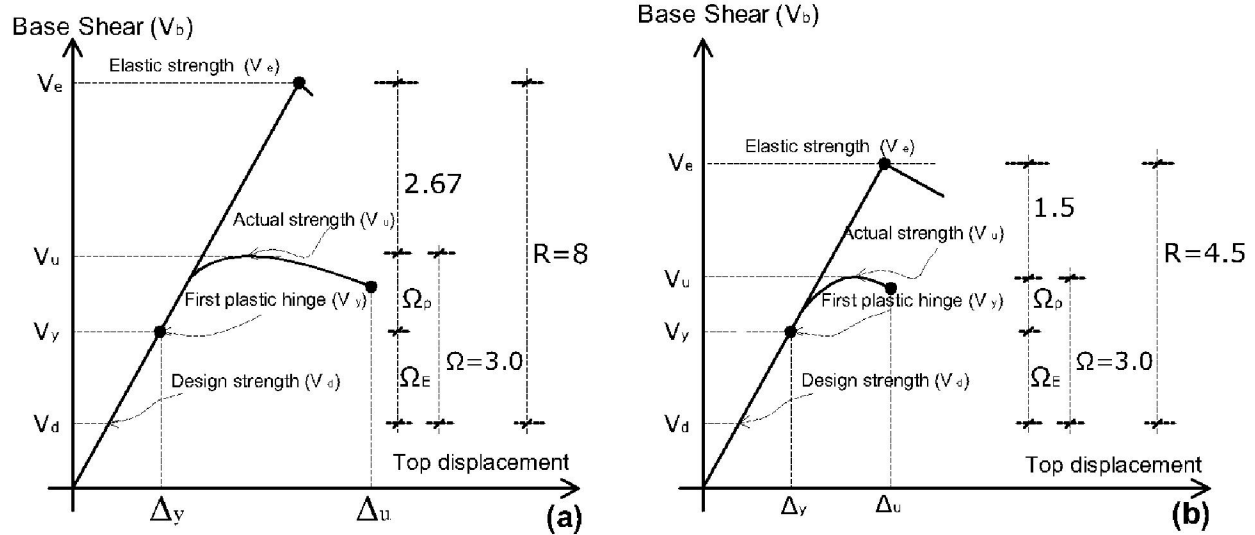


Figure 3: Reserve overstrength according to AISC/ASCE for (a) SMF and (b) IMF

$$\Omega_{r-SMF} = \frac{R}{\Omega} = \frac{8}{3} = 2.67 \Rightarrow \frac{1}{\Omega_{r-SMF}} = 0.37 \quad (2)$$

$$\Omega_{r-IMF} = \frac{R}{\Omega} = \frac{4.5}{3} = 1.5 \Rightarrow \frac{1}{\Omega_{r-IMF}} = 0.67 \quad (3)$$

This reserve overstrength reveals that in SMF the beams are designed for seismic condition where 37% of the seismic forces are used (Eq 2). Similarly, in case of IMF (Eq 3), design of beam is carried out for seismic condition where 67% of the seismic forces are utilised (beams in IMF are designed for higher seismic forces). This could lead frames perform well especially in case of SMF only, if and only strength dictates. Contrarily, when calculating the interstorey drift, the elastic spectrum is allowed to be reduced by Eq (4) and Eq (5) for SMF and IMF, respectively.

$$\frac{1}{V_{SMF}} = \frac{R}{C_d} = \frac{8}{5.5} = 1.45 \Rightarrow V_{SMF} = 0.67 \quad (4)$$

$$\frac{1}{V_{IMF}} = \frac{R}{C_d} = \frac{4.5}{4} = 1.125 \Rightarrow V_{IMF} = 0.89 \quad (5)$$

To reduce elastic spectrum in order to verify the drift limitations is different for both SMF and IMF (see Eq 4 and Eq 5), it is higher for SMF. Using an out of the box approach, it is worth noting that the capacity design of American code is quite understandable and may not completely mix with the deformability criterion. Instead Eurocode 8 allows for a different behaviour factor (DCH, DCM) to check the ultimate limit state but the deformability criteria always remains the same. Also the overstrength calculations are not straight and simple as iterations are usually required when the drift criterion governs the design due to the re-evaluation of overstrength. When the deformability checks affect the design, the capacity design rules are re-checked and sometimes the designer chooses a behaviour factor for a given lateral load resisting system which in the end cannot be utilised as the effectively demanded ductility is much lower than the one adopted in the design.

Eurocode 8 suggests the use of three different drift limits, namely (i) relax drift limit (L1=0.01h) when there are no non-structural elements that follow the deformations of the structural system. (ii) Intermediate drift limit (L2=0.0075h) when non-structural elements are ductile, (iii) stringent drift

limit ($L_3=0.005h$) when brittle non-structural elements are attached to the structure (notably, ordinary masonry infills).

ASCE drift limits are related to the building collapse hazard thereby employing the following drift limits: (i) 0.02h (Occupancy I and II, all buildings except occupancy III and IV), (ii) 0.015h (Occupancy III) related to important facilities (for examples, public assembly etc.), (iii) 0.01h (Occupancy IV related to essential facilities, for example, hospitals).

The two codes propose different drift limits which seems un-matching. In previous studies [26, 27] it has been observed that the Eurocode 8 drift limitations are quite stringent than the ASCE drift limitations. For instance, the extreme relax drift limit of EC8 is 0.01h whereas the extreme drift limits in ASCE for category I and II is 0.02h. Although in EC8, the spectrum is reduced by 2.0 (importance classes I & II) and 2.5 (importance classes III & IV), whereas it is reduced by 1.45 (Eq 4) and 1.125 (Eq 5) for SMF and IMF, respectively, which illustrate that the drift limit of EC8 are comparatively stringent for such made assumptions.

6. Damageability and ductility

Going along with the contradictions in the two codes on global overstrength, reserve overstrength, response modification/behaviour factor and damageability criteria (reduction of elastic spectrum and drift limits in term of interstorey height), gigantic parametric analyses were carried out. These consist of seismic design of perimeter steel moment resisting frames of 9, 7, and 5 storeys with several bay widths (9.15m, 7.63m, 6.54m, and 5.08m) that are designed according to Eurocode 8 using 4 different q factors (6.5, 4, 3, and 2). All these considerations constitute 144 cases; these are properly designed according to Eurocode 8, delicately analysed and then elaborated critically. A group of 36 cases are designed using Ductility Class High (DCH) having behaviour factor of 6.5 and another 36 cases with Ductility Class Medium (DCM) using behaviour factor of 4.0. Similarly another 36 frames are designed for DCL with behaviour factor of 2.0 according to EC3 where capacity design rules are eliminated, and in addition 36 cases have been examined using a conventional q of 3.0, in order to understand the optimum design of

MRF also compatible with the drift limitation. From parametric analysis it was observed that drift limit (0.0075h and 0.005h) are not compatible with the high ductility class. Similarly, drift limit 0.005h is not compatible with the medium ductility class, whereas all the drift limits are compatible if a conventional ductility class (DCC) defined by behaviour factor of 3. Also the drift limits can be satisfied using elastic analysis. In spite of the results obtained, code does not limit the use of ductility class to be used with a drift limit, whereas interestingly in New Zealand according to Pathon [28], when the drift limits dictate the design of the frames the designer has to use limited ductility and the same has to be adopted for Eurocode 8. It is indeed questionable that how to limit the ductility and adopt a suggested drift limit when an assumed ductility class is employed or which type of non-structural element is useable for a given ductility class. In the present version of EC8, no such rules exist in order to cater this challenge. Further, it is still a hard task to opt for conventional ductility class in order to satisfy drift limit L_3 (0.005h) because when $q=3.0$ is adopted in the design, the resulting elastic overstrength is exceedingly high which is uneconomical and an inconvenient design solution. It can be summarized that when drift limit L_3 is required all time better option is the use of elastic analysis.

Figure 4 (5 and 6 bays) and Figure 5 (7 and 9 bays) shows the over stiffness of all the previously mentioned 144 analysed cases. When the frame satisfies drift limitation, this ratio should be unity or else a value more than one could lead to state that the design is governed by strength, known as over stiffness of the frame. The frames with over stiffness as shown when close to 1.0 are optimally design with stiffness (serviceability limit state), whereas the frames giving rise to greater over stiffness are designed with strength and stiffness. When the frame design is governed by the drift limitation, q cannot be optimally used because in that case the use of high ductility remains futile. This leads to uneconomical design situation thereby paying extra due to the assumed ductility (detailing) together with paying for deformability (huge profiles).

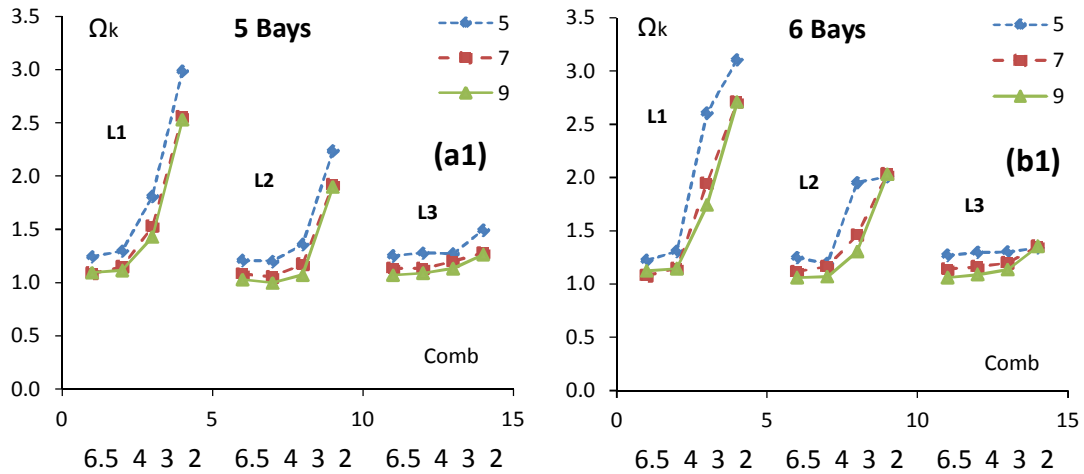


Figure 4: Graphs showing Ω_k for 5 bays (a1) and for 6 bays (b1)

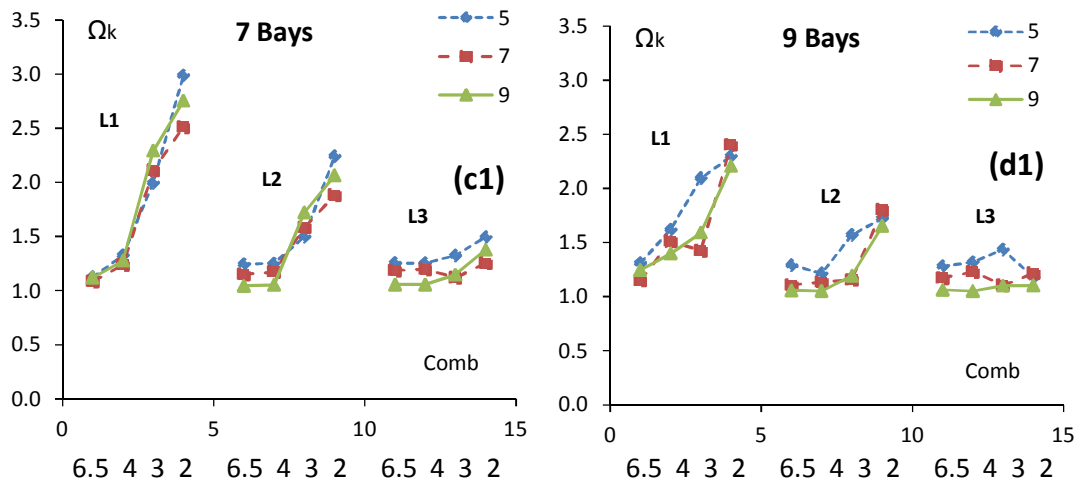


Figure 5: Graphs showing Ω_k for 7 bays (c1) and for 9 bays (d1)

Table 4: Drift limit versus assumed ductility class

S.No	Ductility class	Compatible drift limits		Remarks	Suggestions
		Description	Drift limit		
1	DCH (q=6.5)	Relax drift limit	L1(0.01h)	Convenient	Optimum q
2		Intermediate drift limit	L2(0.0075h)	NOT convenient	Un-economical
3		Stringent drift limit	L3(0.005h)	NOT convenient	Un-economical
4	DCM (q=4.0)	Relax drift limit	L1(0.01h)	Convenient	Optimum q
5		Intermediate drift limit	L2(0.0075h)	Convenient	Optimum q
6		Stringent drift limit	L3(0.005h)	NOT convenient	Un-economical
7	DCC (q=3.0)	Relax drift limit	L1(0.01h)	Convenient	Optimum q
8		Intermediate drift limit	L2(0.0075h)	Convenient	Optimum q
9		Stringent drift limit	L3(0.005h)	Less convenient	Less economical
10	DCL (q=2.0)	Relax drift limit	L1(0.01h)	convenient	Un-economical
11		Intermediate drift limit	L2(0.0075h)	convenient	Un-economical
12		Stringent drift limit	L3(0.005h)	convenient	Un-economical

In view of the above remarks, some suggestions are made which have been shown in Table 4 in obtaining the required effectively demanded ductility. Therefore it is advisable to use reduced ductility when using stringent drift limit and vice versa.

7. Conclusions

Summarized the above mentioned in a nutshell it could be stated that the capacity design rules of American code appears to be quite clear and simple, having strength and deformability criteria not completely mixing. On the contrary, as Eurocode 8 allows for a different behaviour factor for DCH and DCM, but having the same deformability criterion, therefore strength and stiffness checks are mutually immensely influencing. It is also observed that the overstrength calculations are not straight forward as iterations are normally required especially when the drift criteria governs the design due to the re-evaluation of overstrength. On the other hand, consistency of the American approach as compared to that of the EC8 for the design of steel MRFs is reflected in a heavier structure due to huge overstrength factor.

In addition to the parametric analysis; it is eventually suggested to preliminarily choose a suitable cum compatible drift limit before the ductility of the frame is been chosen. Another way could be the use of the cladding system when a ductility class is assumed, for example, when high ductility is assumed for the framing system (primary member), this is in fact erroneous to opt for a brittle cladding system. Similarly, if again a brittle system of cladding is assumed in the design it is not handy to go for a primary element that is highly ductile as they can be designed with elastic analysis for a frequently occurring earthquake. For completeness, in the end it could be quoted that the use of high ductility with brittle material at façade is a choice that leads to pay double by the owner; this means that the choice of structural elements and non-structural elements are directly related to each other in one way or the other.

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