

CRITICAL OVERVIEW ON THE SEISMIC DESIGN OF STEEL MOMENT RESISTING FRAMES

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ABSTRACT

The paper deals with a critical analysis on the design of Steel Moment Resisting Frames using European and American codes methodologies. Both codes follow capacity design philosophy where steel need to be used smartly and efficiently, in order to assure adequate ductility to the structural system. Interesting concepts are given that are based on some critical issues which were observed during the design phase of several study cases dealing with design of multi-storey frames. The effect of the main influential parameters of modern seismic codes, namely the response modification factors/behaviour factors, overstrength factors and drift limitations are investigated. In the whole, the aim of this study is an attempt to mitigate the confusion and the complexities that exist as hidden inconsistencies in the design of MRFs while using the present codes.

Keywords: Seismic codes, overstrength, ductility, damageability, behaviour factor, steel moment resisting frames

1. INTRODUCTION

In regions of high seismic risk, devastating earthquakes rarely occurs at intervals of hundreds of years and affecting typical building sites. Due to the rare return period of earthquakes, it is always un-economical and impractical to design structures for resisting severe earthquakes without damage. Nevertheless, building codes rely on the philosophy to protect life safety by avoiding earthquake-induced collapse in severe events by assuring ductility of the structures and consequently permitting extensive structural and non-structural damage.

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 difficult 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 leading to un-economical solution.

Steel is an elite material and the technique of capacity design approach in seismic codes on the one hand is very interesting and easy to understand but on the other hand needs to be applied smartly so that the adopted parameters like behaviour factor are optimally used. A very important aspect of seismic design with steel structures is the use of the material itself, which needs to be adopted in a proper way, otherwise improper failures can happen that are unacceptable for a corresponding seismic zone. Increase of steel amount in the dissipative zone (e.g. in beams in the case of MRFs) can leads to unpredictable and unreliable failure mechanisms, while if the non-dissipative zones (e.g. columns) are increased by some amount of steel, it might result in an improved failure mechanism with acceptable ductility, provided that other members are well conceived.

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 in understanding and hence are easily applied by the technicians. Unlucky, 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 is aimed at addressing background of European and American seismic codes for MRFs. This will allow the awareness on the inconsistency between damageability and ductility

and therefore is an attempt to mitigate the confusions and the complexities that exist as hidden inconsistencies during the design phase of 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 (Naqash, 2012).

2. SEISMIC ZONATION OF PAKISTAN

Steel moment resisting frames perform well during earthquakes if designed properly. As an example, before the independence of Pakistan from the British in 1947, structures designed according to the recommendation provided by Eng. Kumar resisted the 1935 Quetta earthquake without significant damage (see Figure 1). 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 (Kumar, 1933).

The trend of design of steel structures is highly motivating in Pakistan as it is a high seismic region; the October 2005 earthquake is a clear example. Building Code of Pakistan (BCP) with the aid of "UBC 97" (Whittier, 1997) 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 2 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. The major earthquakes that hit this Country are mentioned below (Zafar, 2009) (BCP, 2007).

- In 1935 at 05 30 - Quetta, Pakistan - M 7.5 Fatalities 30,000
- In 1945 at 11 27 - Makran Coast, Pakistan - M 8.0 Fatalities 4,000
- In 1974 at 12 28 - Northern Pakistan - M 6.2 Fatalities 5,300
- In 2005 at 10 08 – Kashmir Pakistan - M 7.6 Fatalities 86,000
- In 2008 at 10 28 - Pakistan - M 6.4 Fatalities 166
- In 2011 at 01 18 - Pakistan - M 7.2 Fatalities 3

Since Pakistan is a developing Country, the aforementioned quakes strongly influenced the infrastructure in the corresponding areas and caused huge number of casualties and high damage to the building structures.

3. STEEL MOMENT RESISTING FRAMES: PROS AND CONS

Steel moment resisting frames consist of framing assembly that are composed of steel beams and columns. Floor and roof framing consists of cast-in-place concrete slabs or metal deck with

concrete fill supported by steel beams, open web joists, or steel trusses. Lateral forces are resisted by steel moment resisting frames that develop their stiffness through rigid or semi-rigid beam-to-column connections. In the following some advantages and disadvantages are addressed on the use of steel moment resisting frames.

Structural steel framing easily and economically accommodates large clear spaces with office layouts, therefore gives more flexibility to the owner. The steel frames consist of smaller columns than concrete framing, which mean more useable space and less visual obstructions. Clear spans, cantilevers, sloping surfaces, curved framings, lobbies, floor openings, special aesthetic features and unusual loadings are all compatible with steel. If additional loads are added to the structure in the future, steel can be easily reinforced to handle the weight of the desired extensions. Likewise, new floor openings for elevators, stairwells, and architectural or mechanical requirements can be easily accommodated. Steel framing can be rapidly purchased, fabricated and erected. The ease of design and construction of steel material readily lends itself to fast-track construction. Steel lighter weight allows for a smaller less expensive foundation. Steel can be erected more quickly and the building can be occupied sooner, which reduces interest costs on construction financing. The ductility of steel makes it the premiere structural material for economically resisting earthquakes. An owner needs to consider both life safety and how much of the structure is recoverable after a seismic event, steel framing maximizes both of these factors. Most steel sold today has been recycled, and nearly all steel taken from demolition projects is 100% recycled.

One of the disadvantages of steel moment resisting framing is the large lateral deformability due to bending moment, especially for buildings with more than 4 stories. Ensuring high performance of frames by assuring ductility is also a difficult task as in the case of providing ductility to the frames, the detailing and the construction need to be of very high standard with skilled labours. A conventional design could lead to unrealistic and too heavy structures, due to overstrength factors suggested by modern codes. Due to weak design unreliable failure mechanisms like, soft-story could happen. In order to assure full strength rigid connections, the designer can face difficulties during the design phase in addition rigid connections require high skilled workmanship too. Above all lateral stiffness of steel frame is a globally recognised concern, which influences the damage of non-structural elements requiring the application of more sophisticated design rules.

Many studies has been devoted to the comparisons of various seismic design codes, e.g. on the comparison of American and Japanese (Nakashima, Roeder, & Maruoka, 2000), European and Japanese (Marino, Nakashima, Mosalam, 2005) with overview on Japanese code (Nakashima, Chusilp, 2003) for the design of steel structures. Further, Elghazouli in (Elghazouli, 2010) extensively contributed in the assessment of European seismic design procedures and

philosophies for several lateral load resisting systems, especially concerning moment resisting frames due to their paramount inelastic behaviour. Interesting studies have been presented by (Paul et al, 2000) (Paul et al, 2000) who provided state-of-the-art review of seismic design of steel moment resisting frames in U.S. and Indian codes.

4. CONVENTIONAL PARAMETERS IN SEISMIC DESIGN

Since steel frames are generally highly flexible, therefore they are characterized by relatively high fundamental period. Further, as it is revealed in (Miranda & Bertero, 1994), (Fajfar, 2000) for medium and long period, the “equal displacement rule” implies as shown in Figure 3b. Therefore the main factors affecting the structural behaviour under seismic excitation can be obtained from pushover analysis using the “equal displacement rules”, as shown in Figure 3a and explained in the following (SEAOC, 2008).

Ductility reduction factor ($q_{\mu,p}$): It is the ratio of “the elastic base shear (V_e)” to “the base shear obtained at the arrival of first plastic hinge (V_y)” as shown in Eq (1), it can also be termed as expected behaviour factor.

$$q_{\mu,p} = \frac{V_e}{V_y} \quad (1)$$

Redundancy factor (Ω_p): It is the overstrength given by the redistribution of the plastic hinges and termed as redundancy factor; it is the ratio of “the ultimate base shear (V_u) obtained from pushover analysis” to “the base shear obtained at the arrival of first plastic hinge (V_y)” defined by Eq (2). Redundancy exists when multiple elements must yield or fail before a complete collapse mechanism forms. Structures possessing low inherent redundancy are required to be stronger and more resistant to damage and therefore seismic design forces are amplified. Therefore, normally it is assumed that structures having larger global ductility exhibits high redundancy and vice versa.

$$\Omega_p = \frac{V_u}{V_y} \quad (2)$$

Elastic overstrength factor (Ω_E): It is the allowable stress reduction factor and is given by “the ratio of “the base shear corresponding to the arrival of first plastic hinge (V_y)” to “the design base shear calculated from the prescribed code (V_d)” given by Eq (3). As an ideal scenario in the design the elastic overstrength (Ω_E) might be unity. Since the structural capacity must not be less than the design forces, Ω_E is always at least 1.

$$\Omega_E = \frac{V_y}{V_d} \quad (3)$$

Global overstrength factor ($\Omega_{E,\rho}$): It is given by the ratio of “the ultimate base shear (V_u)” to “the design base shear calculated from the prescribed code (V_d)”; it corresponds to the product of redundancy factor and elastic overstrength, evaluated from eq (4).

$$\Omega_{E,\rho} = \Omega_E \times \Omega_\rho = \left(\frac{V_y}{V_d} \right) \times \left(\frac{V_u}{V_y} \right) = \frac{V_u}{V_d} \quad (4)$$

Reserve ductility (q_μ): It is the ratio of “the elastic base shear (V_e)” to “the ultimate base shear (V_u)” as given in Eq (5) as well from Eq (7) and Eq (8).

$$q_\mu = \left(\frac{V_e}{V_u} \right) \quad (5)$$

Behaviour factor (q_d): It is given by the ratio of “the elastic base shear (V_e)” to “the design base shear calculated from the prescribed code (V_d)”, it is simply the product of elastic overstrength, redundancy factor and reserve ductility defined by Eq (6).

$$q_d = \Omega_E \times \Omega_\rho \times q_\mu = \left(\frac{V_y}{V_d} \right) \times \left(\frac{V_u}{V_y} \right) \times \left(\frac{V_e}{V_u} \right) = \frac{V_e}{V_d} \text{ or } q_d = \left(\frac{\Delta_u}{\Delta_y} \right) \quad (6)$$

$$q_\mu = \frac{V_e}{V_u} = \frac{q_d}{\Omega_{E,\rho}} \quad (7)$$

$$q_{\mu,\rho} = \frac{V_e}{V_y} = q_\mu \times \Omega_\rho \quad (8)$$

5. EUROPEAN AND AMERICAN DESIGN APPROACHES

5.1 General

In this section, the main concepts used by the two codes for the design purpose are summarized with special regard to seismic design and more specifically the capacity design approach is illustrated. Apart from these comparisons it is interesting to note that in Europe mostly spatial frames are commonly designed with short span, instead in America perimeter frames with long spans are generally preferred.

5.2 Limit state design approach

Limit state design approach requires the structure to satisfy two main conditions, i.e. the Ultimate Limit State (ULS) for strength checks which is based on partial factor of safety concept and the Serviceability Limit State (SLS) for the comfort of the occupants and protection of non-structural elements.

Eurocode 3 “Design of Steel Structures” (Eurocode3, 2005) is composed of a set of design guidelines for wide range of steel structures. The rules of EC3 are based on limit state design

philosophy, such that the effects of the induced actions (E_d) should be less than the design resistance (R_d) of the structure. The design philosophy is the same as that of the AISC/LRFD (ANSI/AISC, 2010). The design rules in EC3 are based on three modes of failure: i). Mode 0: excessive deformations by yielding before failure; ii). Mode 1: stability failure induced by imperfections and yielding; and iii). Mode 2: fracture failure after yielding. Brittle/fragile failures should be avoided always. The recommended values of safety factors for the corresponding limit states are $\gamma_{M0}=1.0$ for yielding, $\gamma_{M1}=1.0$ for buckling and $\gamma_{M2}=1.25$ for fracture.

AISC/LRFD approach for structural steel buildings is based also on the limit state philosophy and termed as Load and Resistance Factor Design (LRFD) (ANSI/AISC, 2010). This approach is based on the limit states of strength and serviceability combined with a first-order probability analysis for the determination of load and resistance factors. In AISC/LRFD the members are designed such that no applicable limit state is exceeded when the members will be subjected to appropriate load combinations. In AISC/LRFD, the nominal strength of the member is multiplied by a resistance factor (ϕ), which takes in to account the uncertainties of the material and geometric properties. The values of resistance factors correspond to 0.75 and 0.90 for fracture and yielding/instability limit states, respectively.

5.3 Seismic forces

Seismic codes generally follow force based design approach in which forces are reduced by a single reduction factor to arrive at the design force level; the reduction of the induced forces are guaranteed by the provision of the desired ductility. Codes give the so called design spectrum for defining the ground acceleration, which is normally reduced by response modification factor (R)/behaviour factor (q). The force reduction factor is represented by the ratio of the “elastic strength demand” to “the design strength demand” or simply it is the ratio of “the elastically induced forces” to “the prescribed design forces at the ultimate state” under the specified ground motion during an earthquake. The amount of reduction of spectrum depends on the overstrength, reserve overstrength and ductility of the structure. These terms are related to the capacity design approach given by the codes and are addressed in the following.

5.4 Capacity design rules

Eurocode 8 (Eurocode8, 2005) recommends the use of behaviour factor q as response modification factor for MRF as illustrated in Table 2. Three ductility classes are allowed for the seismic design of MRFs i.e. Ductility Class High (DCH), Ductility Class Medium (DCM), and Ductility Class Low (DCL), these are given in Table 2. A flowchart for the prescribed design procedure is shown in Figure 4. From the flowchart it is evident that the design is revised normally due to the re-evaluation of overstrength factor (Ω) in Eurocode 8 as it is related to the plastic resistance of beams.

The energy dissipation capacity for different frames in AISC/ASCE (ANSI/AISC, 2010) is taken into account by the use of response modification factor (R) which depends on the type of frame used for the lateral load resisting system and accounts for (i) ductility, (ii) overstrength, (iii) redundancy, (iv) damping, and (v) past behaviour etc. Since the reduced spectrum has to be used for the verification of interstorey drifts, therefore specific seismic amplification factors C_d is multiplied with the estimated loads for strength. The values of C_d are generally lower than the R values as shown in Table 3.

(AISC/ASCE, 2010) prescribes the use of SMF (Special Moment resisting Frames), IMF (Intermediate Moment resisting Frames), and OMF (Ordinary Moment resisting Frames), these are comparable to DCH, DCM and DCL, respectively in Eurocode 8. From the flow chart it is evident that design of MRFs using AISC specification is not iterated as Ω is a pre-defined fixed value.

The design requirements in the 2010 AISC seismic provisions (AISC/ASCE, 2010) generally follow the format of the AISC/LRFD specifications where the design strengths (resistance factor multiplied by the nominal resistance) of members or components should equal or exceed the required strengths. For force-controlled components proportioned by following capacity design principles, the required strengths are generally based on capacities of deformation controlled components, which are adjusted to account for material overstrength, strain hardening and other factors that increase the strength of the members beyond their nominal values. A flowchart for the design procedure using AISC/ASCE is shown in Figure 5. The main difference in the flowchart using Eurocodes and American practices is the overpassing of the iteration for overstrength in the case of AISC/ASCE.

Further, AISC limits height of IMF when designing in Seismic Design Categories (SDCs). For example, there is no height restriction for IMF in SDCs “B” and “C”, whereas in SDC “D” the height is limited to 10.7m and it is restricted to not be used in SDCs “E” and “F”. There is no height limitation for the use of OMF in SDCs “B” and “C” whereas it is restricted to be used in SDCs “D”, “E”, and “F”.

More generally, in both codes global ductility is achieved by the implementation of Strong Column Weak Beam “SCWB” philosophy. In this context, Eurocode 8 suggests the condition shown in Eq (9) must be fulfilled at all seismic beam-to-column joints:

$$\sum M_{Rc} \geq 1.3 \sum M_{Rb} \quad (9)$$

$\sum M_{Rc}$ and M_{Rb} are the sum of the design values of the moments of resistance framing the joint of the columns and beams respectively. The factor 1.3 takes into account the strain hardening and the material overstrength which is obtained generally by the multiplication of 1.1 with γ_{ov} and as a

general rule is considered as 1.3. Generally this check is often satisfied at the joint when capacity design is employed in the design.

In AISC seismic provisions, Eq (10) needs to be satisfied for SMF at the beam-to-column connections.

$$\frac{\sum M_{pc}^*}{\sum M_{pb}^*} > 1.0 \quad (10)$$

$\sum M_{pc}^*$ is the sum of the moments in the column above and below the joint at the intersection of the beam and column centrelines; it is calculated from Eq (11).

$$\sum M_{pc}^* = \sum Z_c \left(F_{yc} - \frac{P_{uc}}{A_g} \right) \quad (11)$$

$\sum M_{pb}^*$ is the sum of the moments in the beams at the intersection of the beam and column centrelines.

$$\sum M_{pb}^* = \sum (1.1R_y F_{yb} Z_{RBC} + M_{uv}) \quad (12)$$

Z_c and Z_{RBS} are the plastic section modulus of column and minimum plastic modulus at reduced beam section respectively, F_{yb} and F_{yc} are the specified minimum yield stress of beam and column respectively, R_y is the ratio of expected yield stress to specified minimum yield stress, P_{uc} is the required compressive strength using LRFD load combinations, M_{uv} is the additional moment due to shear amplification. It is evident from Eq (12), where plastic moment of beam is amplified by a factor $1.1R_y$ which resembles Eq (9) as suggested by EC8. The value of R_y ranges from 1.1 to 1.5.

6. CODE CRITICISMS

The definition of overstrength factor (Ω) follows different approach in the two codes. As far as American codes are of concern, overstrength is a fixed value (3.0) for steel MRFs, instead a complete different philosophy is provided 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 (13).

$$\Omega = \frac{M_{pl,rd,i}}{M_{Ed,i}} \quad (13)$$

$M_{Ed,i}$ 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, in the case of 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 obtained 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 compared to AISC/ASCE. The problem may arise when the overstrength factor is higher than 3.0, where columns may increase drastically and could lead to overdesign (Matteis, 2005), 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 is evaluated which represents the reserve overstrength; these are given by Eq (14) and Eq (15) for SMF and IMF, respectively, and are shown in Figure 6:

$$\Omega_{r-SMF} = \frac{R}{\Omega} = \frac{8}{3} = 2.67 \Rightarrow \frac{1}{\Omega_{r-SMF}} = 0.37 \quad (14)$$

$$\Omega_{r-IMF} = \frac{R}{\Omega} = \frac{4.5}{3} = 1.5 \Rightarrow \frac{1}{\Omega_{r-IMF}} = 0.67 \quad (15)$$

In seismic conditions, this reserve overstrength shows that in SMF the beams are designed for seismic condition where 37% of the seismic forces are used (Eq 14). Similarly, in the case of IMF (Eq 15), beams are designed for seismic condition where 67% of the seismic forces are utilised (beams in IMF are design for higher seismic forces). This could lead to a better performance of the frame especially in the case of SMF only, if strength is dictating. Contrarily, in order to evaluate the interstorey drift, the elastic spectrum is allowed to be reduced by Eq (16) and Eq (17) for SMF and IMF, respectively.

$$\frac{1}{v_{SMF}} = \frac{R}{C_d} = \frac{8}{5.5} = 1.45 \Rightarrow v_{SMF} = 0.67 \quad (16)$$

$$\frac{1}{v_{IMF}} = \frac{R}{C_d} = \frac{4.5}{4} = 1.125 \Rightarrow v_{IMF} = 0.89 \quad (17)$$

The reduction of the elastic spectrum for verifying the drift limitations is different for SMF and IMF (see Eq 16 and Eq 17), it is higher for SMF. In the whole, 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) while checking the ultimate limit state but the deformability criteria is always the same. Also the overstrength calculations are not straight forward as iterations are normally required especially when the drift criteria govern 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 ($L_1=0.01h$) when there are no non-structural elements that follow the deformations of the structural system. (ii) intermediate drift limit ($L_2=0.0075h$) 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 hazard that is represented by the building collapse and the following drift limits are employed: (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 prescribe different drift limits which seems contradictory. In previous studies (Naqash, Matteis, & Luca, 2012) and (Naqash, Matteis, & Luca, 2012) it has been observed that the Eurocode 8 drift limitations are quite stringent compared to 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 16) and 1.125 (Eq 17) for SMF and IMF, respectively, which demonstrate that the drift limit of EC8 are stringent even for such assumptions.

7. CONCLUDING REMARKS

The seismic provisions given by European and American codes for moment resisting steel frames have been analysed in the paper by a case study developed for short span MRFs (6.6m) having regular building plan (Naqash, Matteis, & Luca, 2012). This simple case study allows many interesting conclusions to be drawn. In particular it appeared that the load combinations of EC8 are lighter than the analogous ones in the American code. Also, the Ω factor in EC8 is generally smaller compared to AISC/ASCE, therefore giving rise to lighter seismic combinations for non-dissipative elements. This aspect on the one hand resulted in a more reasonable design because of smaller cross sections of structural elements but on the other hand revealed that the damage limit state played a more important role, which generally governed the design of the structural elements, provoking the increase of beam profiles. This is a reason of larger complexity for the structural designer as strong column weak beam "SCWB" criterion generally needs to be rechecked in order to accomplish with the capacity design criteria, compelling the designer to increase the columns profiles as well. Contrarily, with AISC/ASCE the load combinations for seismic analysis appear to be more conservative, with the fixed Ω factor and the relaxed drift criterion compared to EC8. On the other hand ASCE used scaling factors for design forces and drift calculations (when the base shear force using modal analysis is less than 85 % of the calculated base shear force using the equivalent static load procedure given by the code), which

further produces the increase of the member dimension. Almost similar results were found in (Naqash, Matteis, & Luca, 2012) by the examination of the codes together with the developed case study on long span MRF (9.15m). In fact, the ductility class of EC8 for 6 storeys MRF had insignificant influence on the cross section dimension especially for perimeter frames.

In the whole, it can be concluded that the capacity design rules of American code appears 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 more mutually influencing. Also, it should be observed that the overstrength calculations are not straight forward as iterations are normally required especially when the drift criteria govern the design due to the re-evaluation of overstrength.

On the other hand, the more consistency of the American approach for the design of steel MRFs is reflected in a heavier structure compared to EC8.

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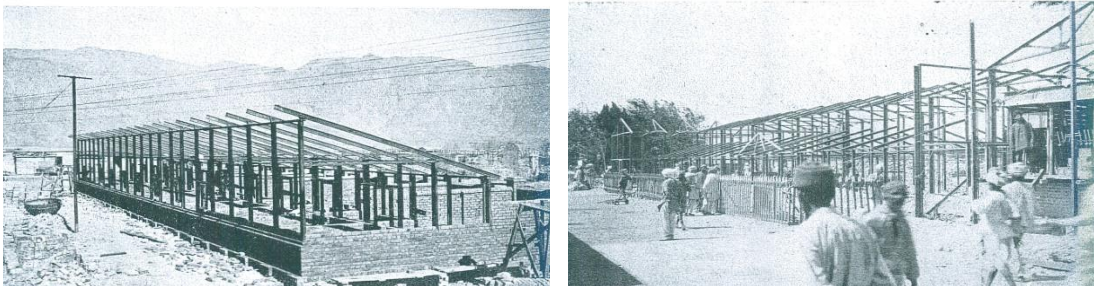


Fig 1. Undamaged steel structures tested by 1935 Quetta earthquake

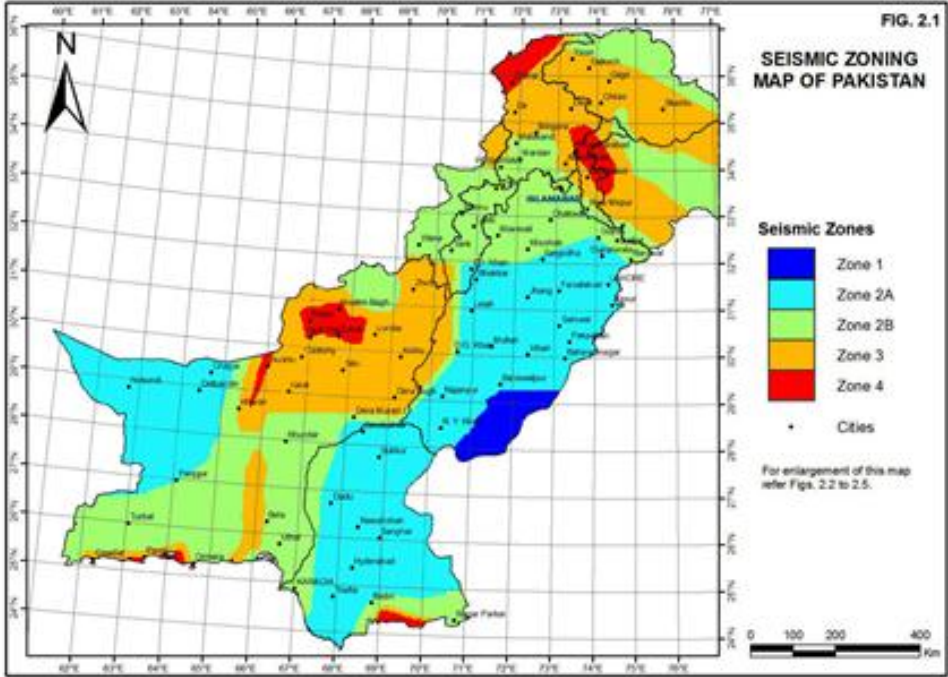


Fig 2. Seismic zoning map of Pakistan

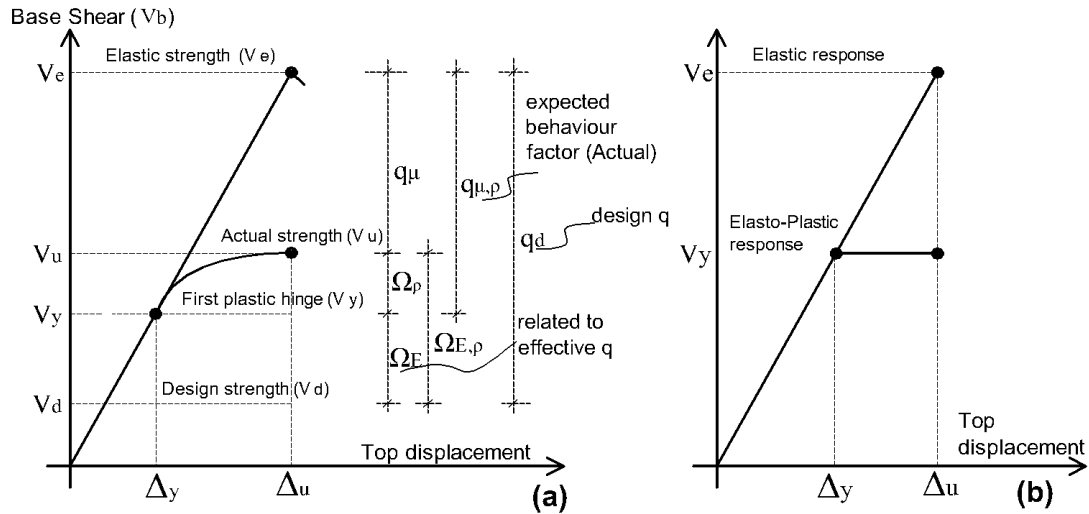


Fig 3. Common factors used in seismic codes (a), and equal displacement approximation (b)

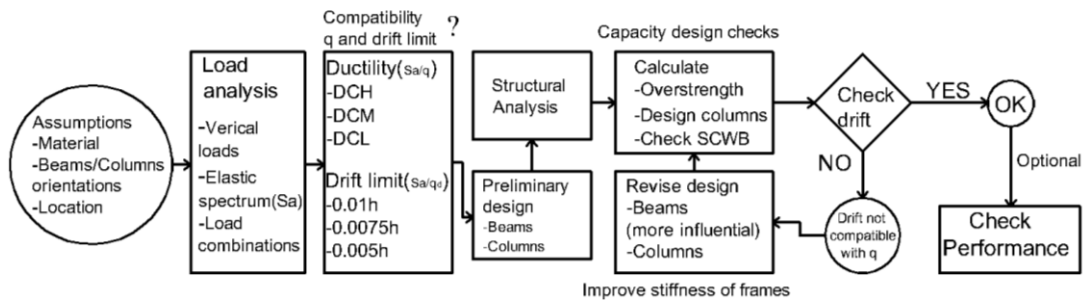


Fig 4. Capacity design flowchart for steel MRFs using Eurocodes process

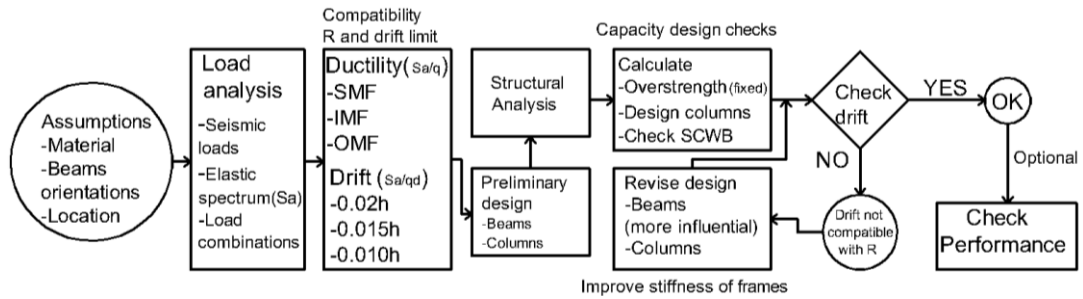


Fig 5. Capacity design flowchart for steel MRFs using AISC/ASCE process

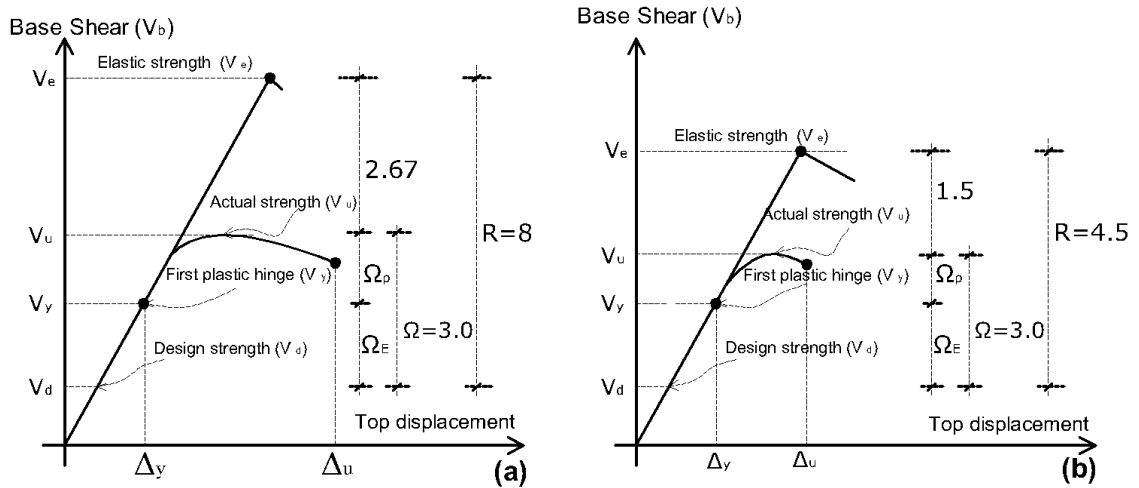


Fig 6. Reserve overstrength according to AISC/ASCE for (a) SMF and (b)


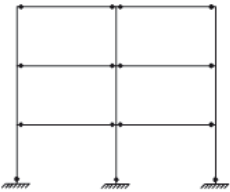
IMF

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	Sever	High
5	4	>0.32g	Sever	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.

Table 2: Behaviour factor for steel MRF in EC8

Moment resisting frames		Behaviour factor (q)/displacement amplification factor (q_d)		
Single bay single storey	Multiple bay multiple storeys	DCH	DCM	DCL
		$5 \alpha_0 / \alpha_1$	4	Recommended for low seismic prone areas (1.5 to 2) with elastic analysis as per EC3 provisions

In Moment resisting frames the dissipative zones are located at the beam's end and at the column's bases.

Table 3: Design factor for steel MRF in AISC/ASCE

Seismic load resisting system	Inelastic deformation	Ω	R	C_d
Special Moment Frames (SMF)	Significant	3	8	5.5
Intermediate Moment Frames (IMF)	Limited	3	4.5	4
Ordinary Moment Frames (OMF)	Minimal	3	3.5	3
Special Truss Moment Frames (STMF)	Significant	3	7	5.5

OMF is permitted instead of IMF in Seismic Design Categories A, B and C.