Assessment of rigid steel frames: ductility versus damageability

Muhammad Tayyab Naqash[1*]

[1] *Department of Civil Engineering, Islamic University in Madinah, Saudi Arabia*
[*] *Corresponding author. E-mail: engr.tayyabnaqash@gmail.com*

Received: Apr. 01, 2021; Accepted: Apr. 25, 2021

Modern seismic code relies on the capacity design approach to assure ductility. When the capacity design dictates, the frames are dimensioned for strength. Inadvertently, stiffness often presents itself as a dominating parameter in MRFs (Moment Resisting Frames), especially with strict drift limits. In this research, rigid steel moment-resisting frames have been designed as per Eurocodes provisions. Initially, the results obtained from modal analysis and nonlinear analyses are concisely elaborated in such a way to highlight the complexities of Eurocode 8 design procedures. Various parameters are assessed to evaluate their influences on high and medium ductility classes. A simplified predictive method is proposed through tables, graphs, and flowcharts. Several combinations are suggested where for an assumed ductility class, a specific drift limit is defined. The elastic overstrength, redundancy factor, and reserve overstrength factors are indicated with the confidence to allow an un-iterated design approach for steel moment-resisting frames designed with a pre-determined strategy for failure mechanisms by improving the design approach of Eurocode 8.

Keywords: Damageability, Ductility, Drift limits, Eurocodes, Moment resisting frames, Seismic design, Capacity design

http://dx.doi.org/10.6180/jase.202112_24(6).0012

1. Introduction

Till forth, by parametric analysis, the problem of damageability, flexibility, and ductility of steel frames are dealt with [1–3] to investigate Eurocode 8 [4] design procedure. The modern seismic code philosophy is based on the capacity design approach to assure ductility. As shown in Fig. 1 [5], so-called chain strength vindicates the concept of capacity design adopted by modern seismic codes. The chain's overall capacity is the strength of its weakest link; one ductile link may be used to achieve ductility for the entire chain. The ductile link's nominal yield strength is subject to strain hardening and imperfections in the material.

The other links are believed to be brittle; however, their failure can be prevented if their strength is higher than the actual strength of the weak ductile link at predicted ductility.

It is evidence that when the capacity design dictates, the frames are sized for strength. Inadvertently, stiffness often presents itself as a dominating parameter in MRFs,

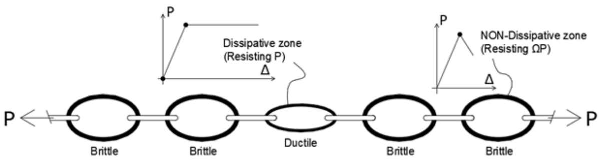


Fig. 1. Principle of capacity design.

especially with strict drift limits. When these are employed with high behaviour factors, an inconsistent capacity design might arise. Therefore, while designing seismic resistance frames, the rigidity, strength, and ductility should be taken into consideration in such a way so that they interrelate. Additionally, in the capacity design, large global overstrength, to size the non-dissipative, affects the amount of the so-called reserve overstrength or ductility [2] as well.

In American provisions [5], the Special Moment resisting Frame (SMF) exhibits high reserve overstrength compared to Intermediate Moment resisting Frame (IMF), in-

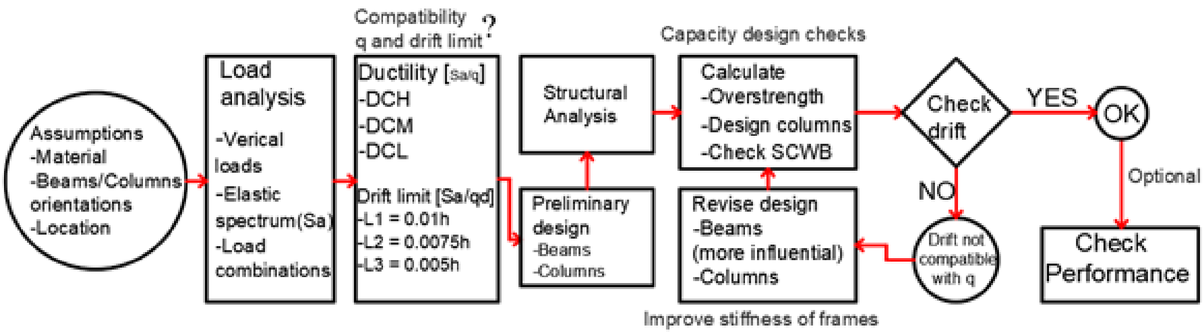


Fig. 2. Eurocode 8 design procedure.

vestigated by Elnashai [6]. Contrarily, in Eurocode 8, this overstrength is usually calculated and might be the same for different ductility classes. The rationale behind this is the iterated procedure related to the modification in beams' dimensions due to the proposed inter-story drift limits [7].

Research on nonlinear analysis, behaviour factors, and performance of frames has been the subject of many researchers in the past as they are highly deformable. For example, Gupta and Krawinkler [8] studied the seismic demands for steel MRFs' performance evaluation. Yun et al. [9] provided procedures for seismic performance evaluation of steel MRFs, with a simple method for estimating the confidence level for satisfying the performance level for a given seismic hazard. Furthermore, Foutch and Yun [10] focused on modelling steel moment-resisting frames for earthquake loads. Besides, Lee and Foutch studied the seismic performance of steel frames with brittle connections. Early literature on nonlinear lateral load analysis, such as FEMA-356 [11] includes Freeman's work [12]. Fajfar and Fischinger [13], Fajfar et al. [14, 15] developed the so-called N2 method for checking the demand and capacity of a frame with pushover analysis. With the existence of FEMA-356, statics and dynamic non-linear techniques have been the subject of several studies. In this context, the study of Krawinkler and Seneviratna [16], Tso and Moghadam [17], Mwafy and Elnashai [18] are worthy of being referenced here. Mazzolani and Piluso [19] studied the inelastic design of steel frames and proposed an approach for steel frames failing in a global collapse mode. Vahid et al. evaluated global and local damage indices of story failure mechanism in Mohsenian and Mortezaei [20]. They concluded that the proposed maximum values for drifts in different standards and codes are not reliable. An interesting study has been conducted by Aidcer et al. [21] on the strain limits, drift, and ductility demands for frames. It concludes that several time drift criteria govern the design with ductility far smaller than that implied by the same code's reduction factor. Nonlinear dynamic analysis has been the subject of many researchers, for example, Sivandi et al., in [22, 23] proposes equations of modal damping ratios of hybrid buildings for the dynamic analysis of frame structures.

Lastly, as during an earthquake or in extreme winds, the potential failure of infill panels such as glass breakage will not give any warning, therefore can lead safety hazards to the public indoors or beside the building. If the drift limitations are not satisfied, the anchored façade structure might fail. It could provoke cladding material damage due to the interstorey drifts induced by lateral loads [24]. Therefore, if brittle materials are attached, they can lead to failure because of interstorey drifts [2, 25, 26].

2. Eurocode 8 synopsis

Necessary design requirements as per Eurocode 8 are illustrated in Table 1.

The behaviour factor (q) is the ratio of "elastic strength" to the "design strength," or simply the ratio of the elastically induced forces to the prescribed design forces at the ultimate state. It is used to reduce the ultimate seismic event, and the amount of reduction depends on ductility. The behaviour factor q and displacement amplification factor q_d for MRF is illustrated in Table 2.

The multiplier α_u/α_1 with the behaviour factor (q) stands for the redundancy factor. This ratio represents "the structure capable lateral force" to "the lateral force when the first element of the structure reaches its resistant capacity." This ratio can be obtained using pushover analysis and should not exceed 1.6.

Since the formation of local buckling in a cross-section reduces energy dissipation capability to a great extent, avoiding local buckling EC8 restricts the use of a class of cross-section for a given ductility class shown in Table 3. Class 1 sections are unaffected by local buckling and can

Table 1. Synthetic scheme for Eurocode 8.

Description	Eurocode 8 criteria (EC8 and EC3)
Seismic Weight	$\sum G_{k,j} + \sum \psi_{E,i} \times Q_{k,i}$
Seismic combination for beams (dissipative members)	$\sum G_{k,j} + E + \sum \psi_{2,i} \times Q_{k,i}$
Seismic spectra	Two primary spectra are considered, i.e., for high seismicity with surface-wave M_s >5.5 and moderate seismicity M_s <5.5
Cross-section limitations	Class 1 sections for q >4 only, Class 1 and Class 2 for 2 <q $\leq$ 4 and Class 1, 2 and 3 for 1.5 <q $\leq$ 2 are allowed
Rotation capacity (local ductility concept)	In DCH: Plastic hinge rotation is restricted to 35 mrad and In DCM, it is 25 mrad
Beam strength checks (dissipative members)	$\frac{N_{E,d}}{N_{pl,Rd}} \leq 0.15,\ \frac{M_{E,d}}{M_{pl,Rd}} \leq 1.0,\ \frac{V_{E,d}}{V_{pl,Rd}} \leq 0.5$
Columns strength checks (Non-dissipative members)	$N_{E,d} = N_{Ed,G} + 1.1\gamma_{ov}\Omega N_{Ed,E}$ $M_{E,d} = M_{Ed,G} + 1.1\gamma_{ov}\Omega M_{Ed,E}$ $V_{E,d} = V_{Ed,G} + 1.1\gamma_{ov}\Omega V_{Ed,E}$
Strong column weak beam (SCWB) philosophy	$\sum M_{Rc} \geq 1.3 \sum M_{Rb}$

Table 2. Behaviour factor for frames.

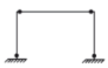
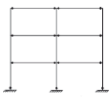
Lateral load resisting system (MRF)	Behaviour factor (q)/Displacement amplification factor (qd)		
	High	Medium	Low
 α_u/α_1=1.1  α_u/α_1=1.2 (1 bay) α_u/α_1=1.3 (multi-bay)	$5\alpha_u/\alpha_1$	4	(1.5 to 2), for low seismicity regions, elastic analysis

Table 3. Restriction of Cross-section for behaviour factor.

Ductility class	Behaviour factor q	Cross-sectional class
Medium	1.2 <q $\leq$ 2	class 1, 2 or 3
	2 <q $\leq$ 4	class 1 or 2
High	q >4	class 1

develop and maintain their fully plastic resistances until a collapse mechanism forms. Due to local buckling, Class 2 cross-sections may develop plastic moment resistance but have limited rotation capacity. In the remaining 2 cross-sections, classes 3 and 4, local buckling occurs before attaining yield stress in one or more parts of the cross-section.

EC8 prescribes the following limitations for drift criteria, as shown in Table 4.

Where d_r denotes is the design interstorey drift, v is the reduction factor, and h is the story height.

Table 4. EC8 interstorey drift limitations.

Description	Limit
Structures having brittle non-structural elements	dr.$v \leq$ 0.0050h
Structures having ductile non-structural elements	dr.$v \leq$ 0.0075h
Structures having no non-structural elements OR non-structural elements fixed in a way to not interfere with structural deformations	dr.$v \leq$ 0.010h

3. Study case

In this study, 144 different frames (refer to Tables 5 and 6) are designed according to Eurocodes [4, 27] to highlight the issues and complexity in the capacity design approach. These frames are analysed by nonlinear analysis, following the FEMA 356 acceptance criteria. The typical floor plan for 6 bays is shown in Fig. 3(a), whereas frame elevation for 9 storeys is shown in Fig. 3(b). The secondary beam's orientation is provided for the external bays only; nevertheless, the secondary beams in the internal bays are oriented similarly. The beams are oriented in such as to have a symmetrical effect on the overall frames. The modal analysis outcomes are presented in this section; these include discussions on alpha critical, fundamental period, codified overstrength, and frames' weight. For design purposes, the frames have been considered on the building's perimeter (perimeter configuration). The beams to column connections are considered rigid. Modal response spectrum analysis using SAP 2000 [28] has been performed on two-dimensional planner frames. Besides, buckling analysis and pushover analysis are also carried out to check the second-order effects and evaluate frames' performance [2, 29–31].

4. Material properties

In this section, the material properties used in modeling the frame obtained from the design are mentioned. The material properties for the adopted steel grade (EN 10025-2 S275) are shown in Table 7. Since there are 144 frames and all the frames have varying cross-sections, therefore in such paper, it is difficult to provide the cross-section profiles. Nevertheless, details are presented in [29].

5. Modal analysis

Modal analysis is performed, a fundamental tool for assessing the dynamic behaviour of a structure. It provides valuable information on the behaviour of structure to dynamic forces in the elastic range. In general, this analysis makes it possible to analyse the following representative quantities:

- The periods of vibration
- The participating masses corresponding to different modes of vibration.
- The participation coefficients of the individual modes.

From the modal analysis, it is observed that with the variation of drift limits from 0.01 h (the relaxed one) to 0.0075 h (intermediate) and then to 0.005 h (the most stringent one), the following equalities hold:

- The design base shear increases.
- The elastic base shear increases.
- The fundamental period decreases with the increase in the frame stiffness.
- The overstrength increases for "DCH" in the cases of drift limit (0.005h) its value is around 4.5, therefore, giving rise to very low reserve overstrength.
- The alpha-critical increases.
- The weight of the frames increases
- The base shear obtained from the load combinations of capacity design increases.

The frames are also designed using a conventional behavior factor that equals 3.0, where capacity design rules are used. The obtained overstrength factor is lower in this case than in the other two ductility classes with a small reserve overstrength.

Finally, in the low ductility class, frames to be proportioned elastically with adequate stiffness and strength to resist seismic forces as per Eurocode 3. Therefore, the capacity design rules are avoided in such a case, the overstrength factor is considered as a unity, and the material overstrength and strain hardening effects are ignored.

The period is related to the considered behavior factor and obviously to the drift limit. Eurocode 8 (mainly for all other codes) relationship for estimating the period is given by a function of overall height (H). The height is a simple known parameter before detailed design, explaining the ratio between the building's stiffness and mass. Using EC8, the period for 5, 7, and 9 story frames is 0.8s, 1.03s, and 1.25s; these are not provided in the figure since the Fig. 4 and 5 are grouped for bays. Instead, the period obtained from the dynamic analysis is related to the frames' mass and stiffness. Therefore, as the drift limit changes, the stiffness changes, and, accordingly, the period varies. The periods obtained from codified formulations and modal analysis are normalized; their values are provided for 5 and 6 bays in Fig. 4 and 7 and 9 bays in Fig. 5. These figures reveal that the specified code-based fundamental periods agree with the period of the frames obtained from modal analysis when the strictest drift limit (0.005 h) is considered. In contrast, these are higher in drift limits (0.0075 h) and (0.01 h), which shows that the code-based period is related to the considered drift limit in the design.

6. Non-linear static analysis

The importance and need to consider the plastic field response require numerical software tools to perform the structure's non-linearity. The numerical tools currently available can do this using two different approaches: modelling using plastic hinges with "concentrated" plasticity) or modelling with "diffuse" plasticity. In this work, only concentrated plasticity modelling is considered. The nonlinearity of the structure, therefore, remains concentrated in a few elements. This technique's advantage is that it allows working mainly with easily manageable elastic elements, leaving the concentration of the material's non-linearity to a few structure points. The hinges can be assigned to a frame element in any position along the member's free span following FEMA 356 standards [11]. The hinges are allocated to the ends of beams and columns base in the case under consideration, using a hard-hardening plastic rigid

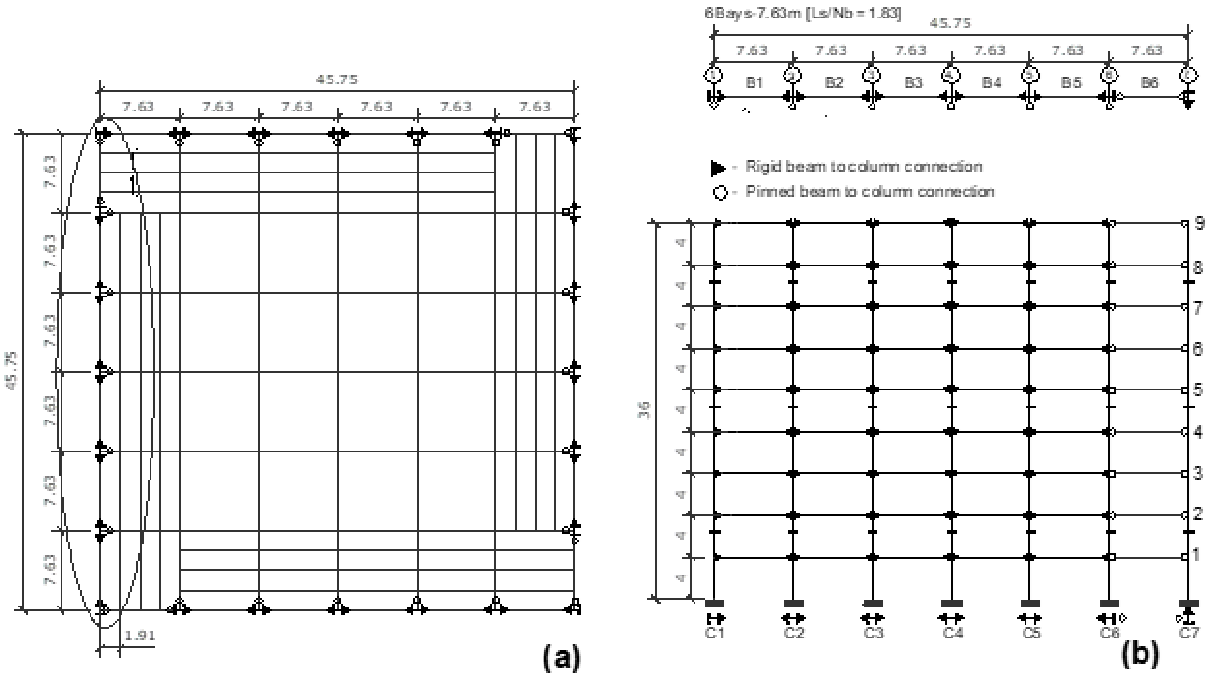


Fig. 3. 6 Bays perimeter MRFs (a) Typical floor plan and (b) elevation.

Table 5. Considered parameters for 9, 7, and 5 story frames.

No. of stories (height)	No. of bays (width)	Behaviour factor (q)	Drift limits (Δ/h)
5 (H=20 m)	5 (9.15 m)	DCH (q=6.5)	L1 (0.01h)
7 (H=28 m)	6 (7.63 m)	DCM (q=4)	L2 (0.0075h)
9 (H=36 m)	7 (6.54 m)	DCC (q=3)	L3 (0.005h)
	9(5.08 m)	DCL (q=2)	

bond starting from a hardening elastic-plastic constitutive bond.

As shown in Fig. 6, the linear behaviour is described from point “A” and “B” of effective yield. The gradient from “B” to “C” is a small percentage of the elastic slope (0 to 10%), necessary to represent the hardening phenomenon. Point “C” has an abscissa value equal to the deformation at which a significant degradation of resistance begins and an ordinate that represents the resistance. After point “D”, there is a local rupture. The section still carries some load, but it is defined as loss of resistance between C and D. The behavioural law of a hinge of this type can be traced automatically by SAP2000. Therefore, M flexural hinges were automatically assigned to all beams and P-M2-M3 hinges to all columns. Naturally, an utterly similar procedure is followed for the columns.

The calculated overstrength (Ω_{calc}) of the frame is the ratio of "apparent strength arises due to the seismic condition" to the "design member strength." It is strongly related to the adopted drift limit and, consequently, to the beam dimensions. The overstrength is calculated using EC8 provisions for the designed frames and is given as the ratio of "plastic moment of beams" to "the internal moment in the beams due to the seismic condition" given by Eq. **??**.

$$\Omega = \frac{M_{pl,rd,i}}{M_{Ed,i}} \tag{1}$$

EC8 recommends a minimum value of "Ω"; therefore, the structural behavior corresponds to the "first plastic hinge" in beam elements, and hence columns are subjected to higher forces due to the redistribution of forces. In contrast, maximum "Ω" provides foreseen malfunctions of beams than columns but oversize the members’ dimensions, leading to an uneconomical design solution. The calculated overstrength decreases with the q factor decrease, as shown in Fig. 7 (a1) for 5 and Fig. 7 (b1) 6 bays, Fig. 8 (c1) 7 bays, and Fig. 8 (d1) for 9 bays. The worst cases are of DCH (especially with drift limits "L2" and "L3"), where higher values are obtained, reflecting that drift criterion

Table 6. Considered cases for 9, 7, and 5 story frames.

Class	No	Storys (H)	Bays (B)	q (Ductility)	Δ/h	Combination	L-q-Combination
	1,2,3,4				0.01	Comb-1	L1-6.5-Comb-1
1	5,6,7,8	5 (H=20 m)			0.0075	Comb-2	L2-6.5-Comb-2
	9,10,11,12				0.005	Comb-3	L3-6.5-Comb-3
	13,14,15,16		5 (9.15 m)		0.01	Comb-1	L1-6.5-Comb-1
2	17,18,19,20	7 (H=28 m)	6 (7.63 m)	High (6.5)	0.0075	Comb-2	L2-6.5-Comb-2
	21,22,23,24		7 (6.54 m)		0.005	Comb-3	L3-6.5-Comb-3
	25,26,27,28		9 (5.08 m)		0.01	Comb-1	L1-6.5-Comb-1
3	29,30,31,32	9 (H=36 m)			0.0075	Comb-2	L2-6.5-Comb-2
	33,34,35,36				0.005	Comb-3	L3-6.5-Comb-3
	37,38,39,40				0.01	Comb-4	L1-4-Comb-4
4	41,42,43,44	5 (H=20 m)			0.0075	Comb-5	L2-4-Comb-5
	45,46,47,48				0.005	Comb-6	L3-4-Comb-6
	49,50,51,52		5 (9.15 m)		0.01	Comb-4	L1-4-Comb-4
5	53,54,55,56	7 (H=28 m)	6 (7.63 m)	Medium (4.0)	0.0075	Comb-5	L2-4-Comb-5
	57,58,59,60		7 (6.54 m)		0.005	Comb-6	L3-4-Comb-6
	61,62,63,64		9 (5.08 m)		0.01	Comb-4	L1-4-Comb-4
6	65,66,67,68	9 (H=36 m)			0.0075	Comb-5	L2-4-Comb-5
	69,70,71,72				0.005	Comb-6	L3-4-Comb-6
	73,74,75,76				0.01	Comb-7	L1-3-Comb-7
7	77,78,79,80	5 (H=20 m)			0.0075	Comb-8	L2-3-Comb-8
	81,82,83,84,85				0.005	Comb-6	L3-3-Comb-9
	85,86,87,88		5 (9.15 m)		0.01	Comb-7	L1-3-Comb-7
8	89,90,91,92	7 (H=28 m)	6 (7.63 m)	Conventional (3.0)	0.0075	Comb-8	L2-3-Comb-8
	93,94,95,96		7 (6.54 m)		0.005	Comb-9	L3-3-Comb-9
	97,98,99,100		9 (5.08 m)		0.01	Comb-7	L1-3-Comb-7
9	101,102,103,104	9 (H=36 m)			0.0075	Comb-8	L2-3-Comb-8
	105,106,107,108				0.005	Comb-9	L3-3-Comb-9
	109,110,111,112				0.01	Comb-10	L1-2-Comb-10
10	113,114,115,116	5 (H=20 m)			0.0075	Comb-11	L2-2-Comb-11
	117,118,119,120				0.005	Comb-12	L3-2-Comb-12
	121,122,123,124		5 (9.15 m)		0.01	Comb-10	L1-2-Comb-10
11	125,126,127,128	7 (H=28 m)	6 (7.63 m)	Low (2.0)	0.0075	Comb-11	L2-2-Comb-11
	129,130,131,132		7 (6.54 m)		0.005	Comb-12	L3-2-Comb-12
	133,134,135,136		9 (5.08 m)		0.01	Comb-10	L1-2-Comb-10
12	137,138,139,140	9 (H=36 m)			0.0075	Comb-11	L2-2-Comb-11
	141,142,143,144				0.005	Comb-12	L3-2-Comb-12

governs the design. Since beams are increased drastically to satisfy the drift criteria; therefore, the overstrength increases, so the columns' corresponding dimensions. Fig. 7 and Fig. 8 show that the drift limit "L3" indicates that the calculated overstrength is around 5.0, which is practically inconvenient. The AISC limit of overstrength to be taken as 3.0 usually is also shown by the dotted line in Fig. 7 and Fig. 8. Interestingly, Elghazouli, in [32], states that the overstrength factor in lateral capacity is related to the ductility demand of the dissipative zones. Therefore, for high overstrength (when drift is governing), the optimum use of behavior factor is possible only if the frames are designed for a lower value of q factor.

7. Over-stiffness

When the frames satisfy drift limitations, the evaluated over-stiffness (Ω_k) equals 1.0; therefore, Ω_k greater than 1.0 means strength is dictating, and the frame has considerable over-stiffness. Furthermore, frames with an over-stiffness factor around 1.0 mean that it optimally designed to satisfy stiffness (damageability criterion). Hence, when drift limitation dominates the design, the q-factor might not be optimally used, heading to an uneconomic design situation (see Fig. 9). Patton in [33] states that when damageability regulates the design, the members need only to be designed and detailed for limited ductility.

As it is evident from the present study, on the one hand,

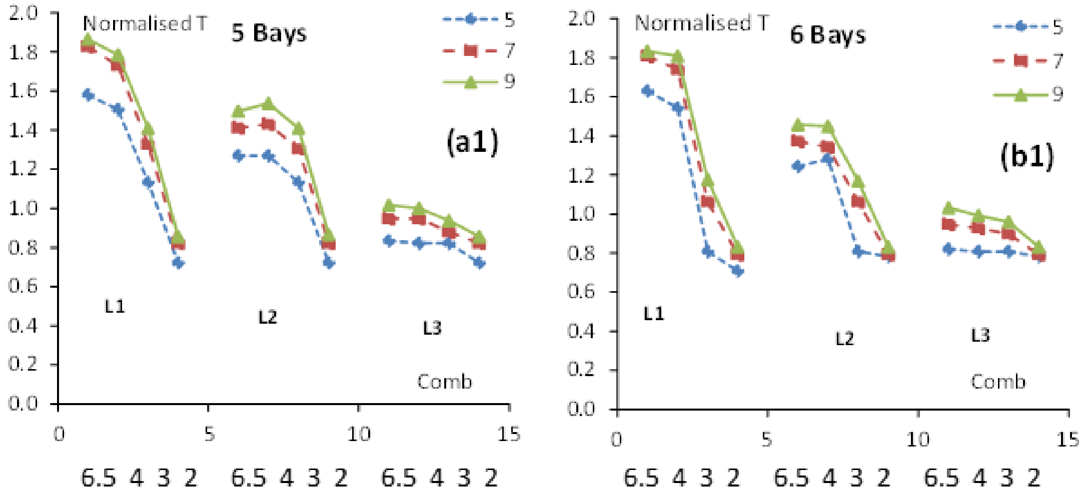


Fig. 4. Modal period normalized to code period for 5 bays (a1) and 6 bays (b1).

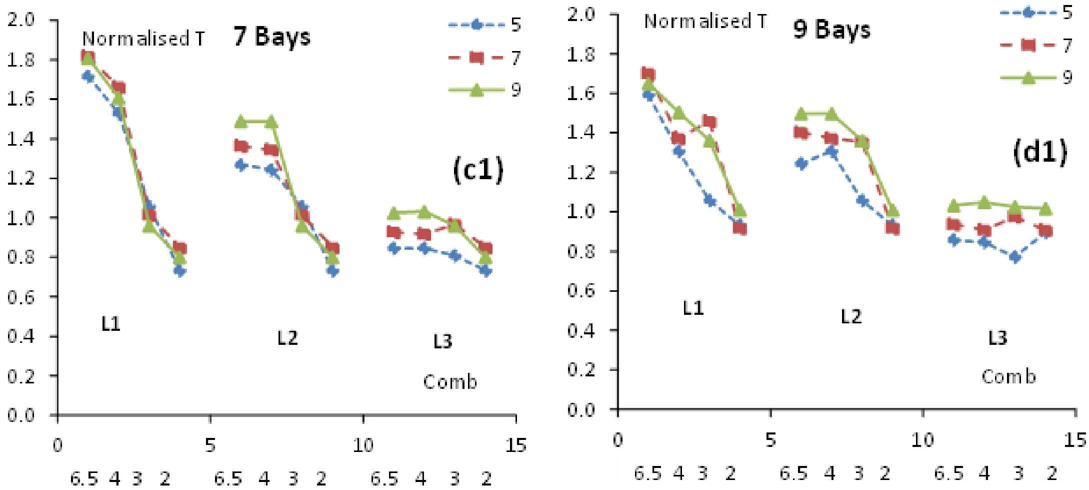


Fig. 5. Modal period normalized to code period for 7 bays (c1) and 9 bays (d1).

the limitations of Eurocode 8 are stringent. On the other hand, no such restriction exists to let the designer know about the importance of optimally using adopted ductility and the frame's damageability.

8. Proposed method

This section proposes a method for optimum steel MRFs' design based on pre-defined parameters. Compatible combinations between damageability and ductility are presented as well. Since the design moment must be greater than the resistance moment, as an ideal scenario, the elastic overstrength (Ω_E) might be unity. As Ω_E varies according to the drift limit, it increases when the drift limit changes from 1% of story height to 0.5% of story height. Consequently, for cases where drift limits dictate the design, this overstrength must be fixed; otherwise, the columns may be massive, leading to uneconomical design situations. Suppose beams are sized considerably more than the minimum required. The structure might yield a high lateral force than the effectively assumed analysis due to the drift limitations. Therefore, the column forces may be greater than predicted by the analysis as factored by $\Omega_{G,E}$ (global overstrength). A factor of 1.375 is used to allow for strain hardening in the beam plastic hinges and provide additional reserve strength.

Redundancy occurs when multiple elements yield or fail before a complete collapse mechanism establish. Structures that retain less redundancy must be more resistant to damage; therefore, seismic design forces are amplified. Consequently, it is assumed that structures having high ductility exhibits high redundancy and vice versa. In the following, the elastic overstrength values are provided (see

Table 7. Sectional and material properties for the profiles used.

Label	H (mm)	B (mm)	t_w (mm)	t_f (mm)
HE1000M	1008	302	21	40
H10x5	1056	314	36	64
H10x8	1056	400	40	80
H11x5	1156	314	36	64
H11x8	1156	400	40	80
H12x8	1200	420	40	80
H12x10	1250	450	45	100
H12x15	1250	500	50	150

NOTE: Adopted steel grade: EN 10025-2 S275 having unit density (ρ) = 76.9 kN/m^3, poissons ratio (v) = 0.3, modulus of elasticity (E in MPa)=2.10E+05, yeild stress (f_y in MPa) = 275, ultimate stress (f_u in MPa) = 275, expected yield stress (f_{ye} in MPa) = 302.5 and expected ultimate stress(f_u in MPa) = 473.

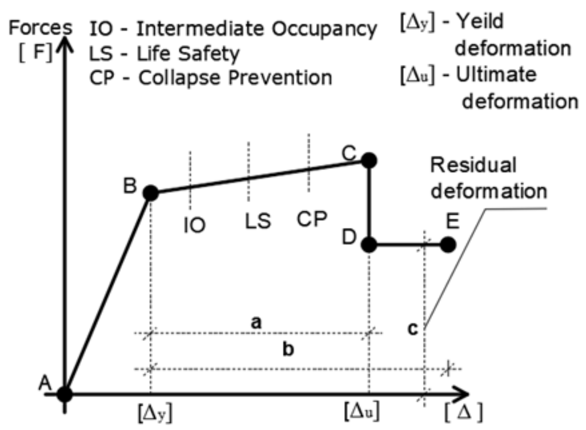


Fig. 6. Plastic hinge illustration by FEMA 356.

Table 8) that helps fix several factors for an assumed ductility class, such as redundancy and reserve overstrength. The elastic overstrength obtained from the pushover analysis is averaged for each drift limit to account for the bay's width (long, intermediate, and short span) and the number of stories. The suggested elastic overstrength values range between 1.5 and 3.0. In Table 8, q factors are related to compatible drift limits, and overstrength factors are also provided. These are shown in Fig. 10a, Fig. 10b and Fig. 10c are related to DCC (L1 = 0.01 h), DCC (L2 = 0.0075 h), and DCC (L3 = 0.005 h), respectively. Similarly, Fig. 11a for DCH using the 0.01h drift limit, in Fig. 11b for DCM with a 0.01 h drift limit, and in Fig. 11c for DCM with a 0.0075 h drift limit.

Elnashai and Mwafy in [18] state that ductile structures show higher reserve strength and are related to using a higher force reduction factor, which reduces the design forces, thus magnifying gravity loads. The suggested technique allow pre-assuming of damageability for an assumed ductility class; therefore no iterations are required to fulfill the capacity design for a global failure mechanism.

9. Concluding remarks

The paper dealt with assessing frames composed of 5, 7, and 9 stories, having a story height of 4.0m. Different bays, i.e., 5, 6, 7, and 9 of 5.08 m, 6.54 m, 7.63 m, and 9.15 m, respectively, have been considered. Four different behavior factors have been assumed with three drift limits of Eurocode 8, concluding 144 cases. Each case has been designed and analysed carefully. Important outcomes, reflecting the ductility class, the definition of overstrength, and the drift limitations of Eurocode 8 have been examined. The following conclusions are summarized:

- Optimization rules for frame design have been provided with suggestions for a realistic design approach.
- Recommendations for the optimum design of steel moment-resisting frames have been proposedwith several outcomes such as when a ductility class for the primary members is defined, an acceptable drift limit is required, so that the two limit states do not indulge each other. Therefore, it necessitates to preliminary choose a proper drift limit before choosing the frame's ductility.it is foreseen that the overstrength of the frames in Eurocode 8 is defined well and is convenient when the seismic situation governs the design for strength; contrarily,
- when the design of the structure is governed by drift, Eurocode 8 definition of overstrength does not hold any stability and vary drastically. Therefore, in the presented paper, some suggestions are provided to use the overstrength factor.
- This technique allow to assume damageability for a ductility class; therefore no iterations are required to fulfill the capacity design for a global failure mechanism.
- The proposed methodology will help the designers to use Eurocode 8 more consistently for the seismic design of frames and let a coherent design approach without unnecessary iteration.
- In a future study the applicability of the proposed methodology will be verified by conducting a similar parametric study.

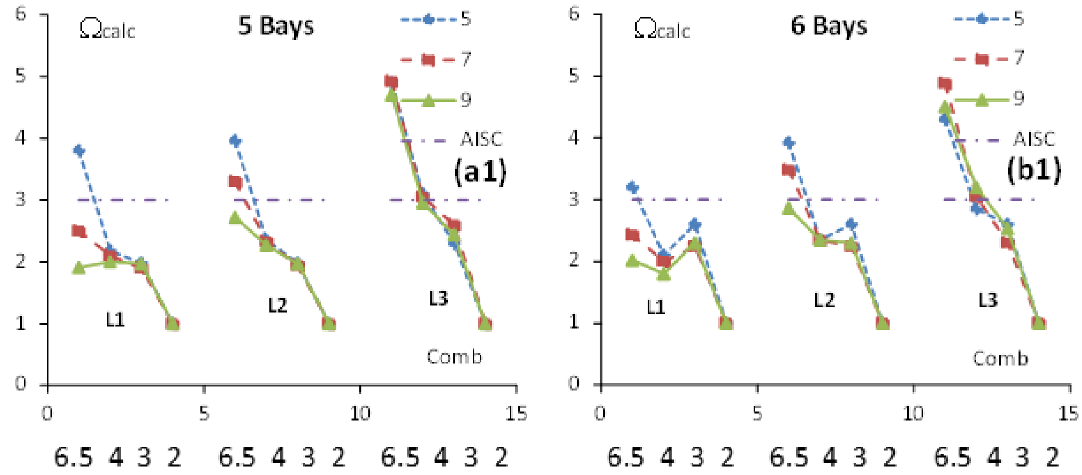


Fig. 7. Codified overstrength for 5 bays (a1) and 6 bays (b1).

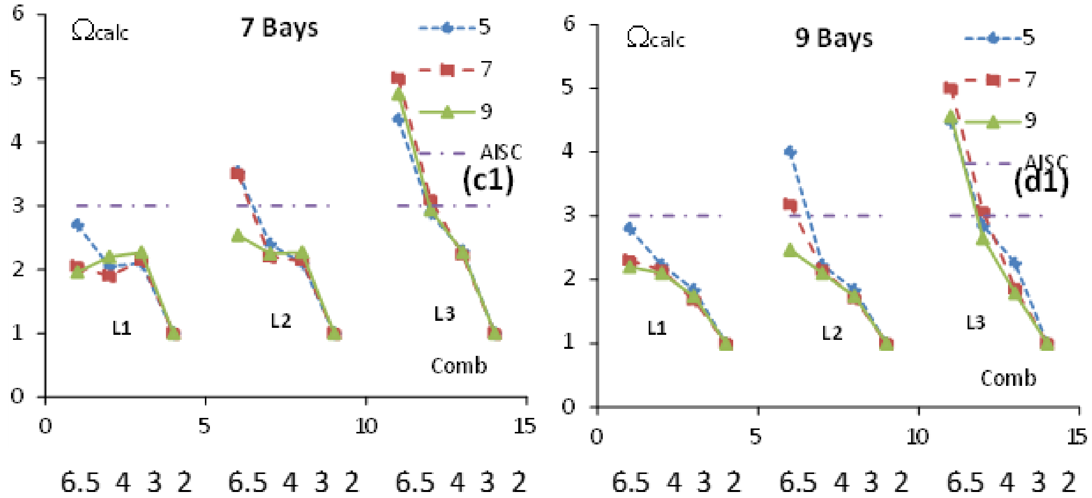


Fig. 8. Codified overstrength for 7 bays (c1) and 9 bays (d1).

10. Notations

MRFs = Moment Resisting Frames

$M_{Ed,i}$ = design value of the bending moment in the ith beam in a seismic design situation

$M_{pl,Rd,i}$ = plastic moment in the ith beam in a seismic design situation,

FEMA = Federal Emergency Management Agency

DCH = Ductility Class High (q = 6.5)

DCM = Ductility Class Medium (q = 4)

DCC = Ductility Class Conventional (q = 3)

DCL = Ductility Class Low (q = 2)

SMF = Special Moment Resisting Frame

EC0 = Eurocode 0

EC1 = Eurocode 1

EC8 = Eurocode 8

EC3 = Eurocode 3

q = Behavior Factor

Drift limit 1 = L1 (0.01h, equivalent to the AISC)

Drift limit 2 = L2 (0.0075h) and

Drift limit 3 = L3 (0.005h, structures with brittle cladding materials)

h = Interstorey height

Ω_E = elastic overstrength

Ω_{calc} = calculated overstrength

$\Omega_{G,E}$ = global overstrength

$G_{k,j}$ = denotes the dead load,

$Q_{k,i}$ = denotes the live or variable load.

$\psi_{2,i}$ = considers the reduced likelihood of variable loads, ranging from 0 to 0.8.

Vb = Base shear

Vu = Actual strength

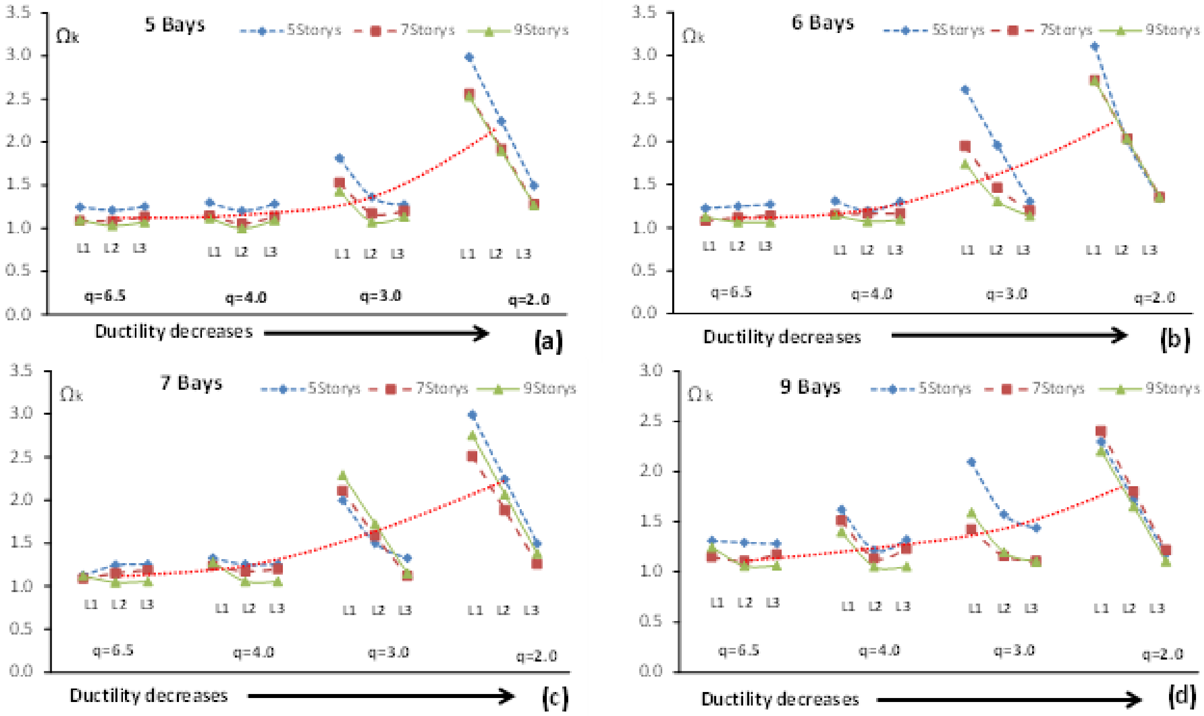


Fig. 9. Over-stiffness for 5 bays (a), 6 bays (b), 7 bays (c) and 9 bays (d).

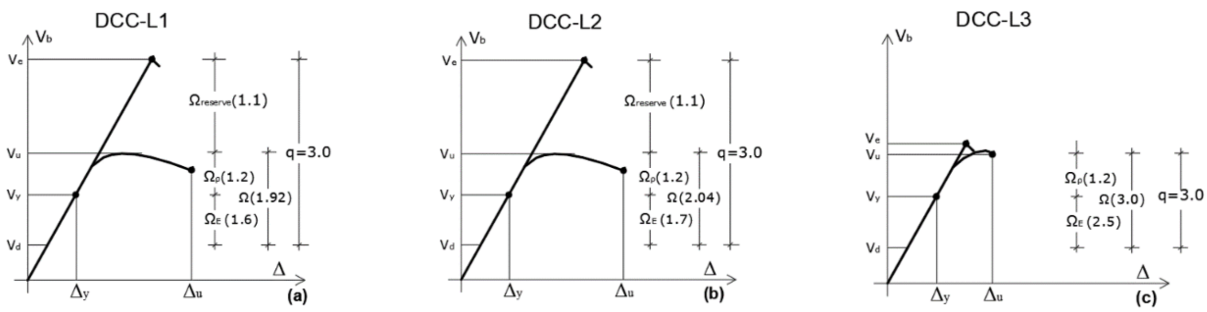


Fig. 10. Proposed global and reserve overstrength for; DCC-L1 (a), DCC-L2 (b), and DCC-L3 (c).

Vd = Design strength
Vy = Yeild base shear
Ve = Elastic strength

References

[1] M. T. Naqash, G. De Matteis, and A. De Luca. "Effects of Capacity Design Rules on Seismic Performance of Steel Moment Resisting Frames". In: *15th World Conference on Earthquake Engineering*. Lisbon, 2012, 1–10.

[2] M. T. Naqash, (2012) *"Optimum design of steel moment resisting frames"*:

[3] M. T. Naqash, A. Formisano, and G. De Matteis. "Aluminium framing members in facades". In: *Key Engineering Materials*. ***710***. 2016, 327–332. DOI: 10.4028/www.scientific.net/KEM.710.327.

[4] European Commitee for Standardization, (2004) *"Eurocode 8: Design of structures for earthquake resistance - Part 1 : General rules, seismic actions and rules for buildings"* **European Committee for Standardization *1***(English): 231.

[5] *Minimum design loads and associated criteria for buildings and other structures*. 2017, 1–889. DOI: 10.1061/9780784414248.

[6] E. I. Katsanos, A. G. Sextos, and A. S. Elnashai, (2014) *"Prediction of inelastic response periods of buildings based*

Table 8. Drift limit, behavior factor, proposed global and reserve overstrength.

S. No	Ductility class	q	Drift limit	Ω_E	Ω_p	$\Omega_{E,p}$	$\Omega_{G,E}$	$\Omega_{Reserve}$
1	DCH	6.5	L1(0.01h)	1.6	1.3	2.08	2.9	2.3
2		6.5	L2(0.0075h)	-	-	-	-	-
3		6.5	L3(0.005h)	-	-	-	-	-
4	DCM	4	L1(0.01h)	1.6	1.3	2.08	2.9	2.3
5		4	L2(0.0075h)	1.7	1.3	2.21	3	1.3
6		4	L3(0.005h)	-	-	-	-	-
7	DCC	3	L1(0.01h)	1.6	1.2	1.92	2.6	1.1
8		3	L2(0.0075h)	1.7	1.2	2.04	2.8	1.1
9		3	L3(0.005h)	2.5	1.2	3	4.1	0.7
10	DCL	2	L1(0.01h)	1	1.1	1.1	1.5	1.3
11		2	L2(0.0075h)	1	1.1	1.1	1.5	1.3
12		2	L3(0.005h)	1	1.1	1.1	1.5	1.3

$\Omega_{G,E}$ = ($\Omega_{E,p}$)×1.1×γ_{ov} with γ_{ov} as 1.25 and represents global overstrength

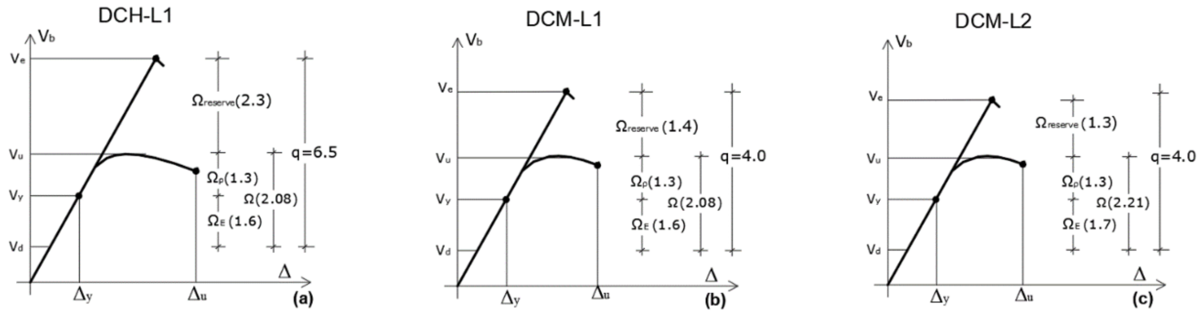


Fig. 11. Proposed global and reserve overstrength for; DCH-L1 (a), DCM-L1 (b), and DCM-L2 (c).

on intensity measures and analytical model parameters" **Engineering Structures** ***71***: 161–177. DOI: 10.1016/j.engstruct.2014.04.007.

[7] M. T. Naqash. *Study on the fundamental period of vibration of steel moment resisting frames*. Tech. rep. 01. 2014, 2319–5347.

[8] A. Gupta and H. Krawinkler, (2000) *"Estimation of seismic drift demands for frame structures"* **Earthquake Engineering and Structural Dynamics** ***29***(9): 1287–1305. DOI: 10.1002/1096-9845(200009)29:9<1287::AID-EQE971>3.0.CO;2-B.

[9] S.-Y. Yun, R. O. Hamburger, C. A. Cornell, and D. A. Foutch, (2002) *"Seismic Performance Evaluation for Steel Moment Frames"* **Journal of Structural Engineering** ***128***(4): 534–545. DOI: 10.1061/(asce)0733-9445(2002)128:4(534).

[10] D. A. Foutch and S. Y. Yun, (2002) *"Modeling of steel moment frames for seismic loads"* **Journal of Constructional Steel Research** ***58***(5-8): 529–564. DOI: 10.1016/S0143-974X(01)00078-5.

[11] FEMA 356, (2000) *"FEMA 356 Prestandard"* **US Federal Emergency Management Agency, 2000.** (November): 2–15.

[12] S. Freeman. *Prediction of response of concrete buildings to severe earthquake motion, Publication SP-55, American Concrete Institute, Detroit, MI, 589-605*. Tech. rep. 1978.

[13] P. Fajfar and M. Fischinger, (1988) *"N2- A Method For Non-linear Seismic Analysis of Regular Buildings"* **Ninth World Conference on Earthquake Engineering**: 111–116.

[14] P. Fajfar, (2000) *"A Nonlinear Analysis Method for Performance-Based Seismic Design"* **Earthquake Spectra** ***16***(3): 573–592. DOI: 10.1193/1.1586128.

[15] P. Fajfar, V. Kilar, D. Marusic, I. Perus, and G. Magliulo. "The extension of the N2 method to asymmetric buildings". In: *Proceedings of the 4th European workshop on the seismic behaviour of irregular and complex structures*. 41. 2005, 291–308.

[16] H. Krawinkler and G. D. Seneviratna, (1998) *"Pros and cons of a pushover analysis of seismic performance*

evaluation" **Engineering Structures** ***20***(4-6): 452–464. DOI: 10.1016/S0141-0296(97)00092-8.

[17] W. K. Tso and A. S. Moghadam, (1998) *"Pushover procedure for seismic analysis of buildings"* **Progress in Structural Engineering and Materials** ***1***(3): 337–344. DOI: 10.1002/pse.2260010317.

[18] A. M. Mwafy and A. S. Elnashai, (2001) *"Static pushover versus dynamic collapse analysis of RC buildings"* **Engineering Structures** ***23***(5): 407–424. DOI: 10.1016/S0141-0296(00)00068-7.

[19] F. M. Mazzolani and V. Piluso, (1997) *"Plastic design of seismic resistant steel frames"* **Earthquake Engineering and Structural Dynamics** ***26***(2): 167–191. DOI: 10.1002/(SICI)1096-9845(199702)26:2<167::AID-EQE630>3.0.CO;2-2.

[20] V. Mohsenian and A. Mortezaei, (2019) *"New proposed drift limit states for box-type structural systems considering local and global damage indices"* **Advances in Structural Engineering** ***22***(15): 3352–3366. DOI: 10.1177/1369433219863299.

[21] A. L. Vidot-Vega and M. J. Kowalsky, (2013) *"Drift, strain limits and ductility demands for RC moment frames designed with displacement-based and force-based design methods"* **Engineering Structures** ***51***: 128–140. DOI: 10.1016/j.engstruct.2013.01.004.

[22] A. Sivandi-Pour, M. Gerami, and D. Khodayarnezhad, (2014) *"Equivalent modal damping ratios for non-classically damped hybrid steel concrete buildings with transitional storey"* **Structural Engineering and Mechanics** ***50***(3): 383–401. DOI: 10.12989/sem.2014.50.3.383.

[23] M. Gerami, Y. Sharbati, and A. Sivandi-Pour, (2013) *"Nonlinear seismic vulnerability evaluation of irregular steel buildings with cumulative damage indices"* **International Journal of Advanced Structural Engineering** ***5***(1): DOI: 10.1186/2008-6695-5-9.

[24] C. Aiello, N. Caterino, G. Maddaloni, A. Bonati, A. Franco, and A. Occhiuzzi, (2018) *"Experimental and numerical investigation of cyclic response of a glass curtain wall for seismic performance assessment"* **Construction and Building Materials** ***187***: 596–609. DOI: 10.1016/j.conbuildmat.2018.07.237.

[25] M. T. Naqash, (2019) *"Design and Fabrication of Aluminum Cladding and Curtain Wall of a Sports Club"* **Open Journal of Civil Engineering** ***09***(01): 1–17. DOI: 10.4236/ojce.2019.91001.

[26] M. Umar, S. A. A. Shah, K. Shahzada, T. Naqash, and W. Ali, (2020) *"Assessment of seismic capacity for reinforced concrete frames with perforated unreinforced brick masonry infill wall"* **Civil Engineering Journal (Iran)** ***6***(12): 2397–2415. DOI: 10.28991/cej-2020-03091625.

[27] EN 1993-1-1/AC. *Eurocode 3: Design of steel structures - Part 1-1: General rules and rules for buildings (corrigendum)*. 2009.

[28] Computers and Structures INC. *Structural Software for Analysis and Design | SAP2000*. 2018.

[29] M. T. Naqash, G. D. Matteis, and A. D. Luca, (2012) *"Seismic Design of Steel Moment Resisting Frames-European versus American Practice"* **NED University Journal of Research, Thematic issue on earthquake** (November 2011):

[30] S. A. A. Shah, M. A. A. Gul, T. Naqash, Z. Khan, and M. Rizwan, (2021) *"Effects of fiber reinforcements on the strength of shotcrete"* **Civil Engineering and Architecture** ***9***(1): 176–183. DOI: 10.13189/cea.2021.090115.

[31] M. T. Naqash, Q. U. Farooq, and O. Harireche, (2019) *"Seismic Evaluation of Steel Moment Resisting Frames (MRFs)—Supported by Loose Granular Soil"* **Open Journal of Earthquake Research** ***08***(02): 37–51. DOI: 10.4236/ojer.2019.82003.

[32] A. Y. Elghazouli, (2010) *"Assessment of European seismic design procedures for steel framed structures"* **Bulletin of Earthquake Engineering** ***8***(1): 65–89. DOI: 10.1007/s10518-009-9125-6.

[33] R. N. Patton, (1985) *"Analysis and design methods"* **Bulletin of the New Zealand National Society for Earthquake Engineering**: