

Effect of Interstorey Drift Limits on High Ductility in Seismic Design of Steel Moment Resisting Frames

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Abstract

The current research activity deals with the seismic design of perimeter steel moment resisting frames of 9, 7 and 5 storeys with several span lengths (9.15m, 7.63m, 6.54m and 5.08m) using Eurocode 8. In total 24 cases are designed and analysed using Ductility Class High (DCH) having behaviour factor equals 6.5. In order to shed light on the drift limitations of Eurocode 8, the designed frames are then checked by means of iteration to investigate the optimal behaviour factor. The evaluated behaviour factor is then compared with the code provided behaviour factor and with the evaluated ductility factor of frames, obtained through the use of static nonlinear analysis. Hence the influence of drift criteria on the capacity design rules of Eurocode 8 is investigated. The frame performances are measured in terms of over strength and redundancy factors, strength demand to capacity and drift demand to capacity ratios allowing to the point highlighted conclusions.

Key Words: Moment Resisting Frames, High Ductility Class, Drift Limits, Eurocode

1. Introduction

Moment resisting frames are designed to resist seismic forces on the hypothesis that they are capable of extensive yielding without significant loss of strength. In order to check performance of structures, various types of analyses can be performed. In this regard the actual structural response can be attained with the use of time history analyses, which are generally are practical for design purpose. Since they are found cumbersome with numerous outcomes, nevertheless such practices are useful for analyses and research purposes. Contrarily, the code specified method “force based design” which is based on static linear analysis is simplified approach but it is quite conservative. In this context the use of non-linear static, the so-called pushover analyses, are quite widespread among the designers to check the performance of the structure at the end of the design and therefore are widely adopted by the technical community [1-3].

One of the prime tasks in designing structures in seismic zones is to assure ductility such that unreliable failures may not occur. For achieving global ductility and avoiding soft story mechanisms “weak energy dissipation” of a structural system, dissipative

and non-dissipative zones are generally defined by the codes; while non-dissipative zones should remain in the elastic field, the dissipative ones should experience large inelastic deformation. To control such a global structural behaviour, codes give the so-called criterion of capacity design, firstly initiated in 1980's in New Zealand. In particular, Paulay and Priestley [4] proposed “Strong Column and Weak Beam” concept in the design of moment resisting frames thereby suggesting of providing reduce stiffness of beams compared to columns. In capacity design then on-dissipative members are designed for comparatively higher seismic forces than the dissipative members. Further, dissipative members are kept at such locations to oblige them to fail before the brittle members and subsequently protect non-ductile elements by overstressing. The selection and therefore, the design of dissipative zones are of prime importance for assurance a suitable collapse mechanism. However, the code procedures are quite conventional where limit states (Ultimate and Serviceability) need to be fulfilled, thus mixing each other and causing unpredictable mechanisms [5]. The present research work, which is a continuation of [3, 6] aims to better recognise the influence of such limit states on the capacity design and therefore on the failure mechanisms of steel moment resisting frames.

2. The Case Study

2.1 Building Description

In order to check the applicability of the proposed high ductility class of Eurocode 8 [7], 9, 7 and 5-storeys steel moment resisting frames are designed using different span configuration (9.15m, 7.63m, 6.54m and 5.08m). The typical floor plans of the perimeter frames for different bays are shown in **Figure1a**, and elevation of 5-bays frame in **Figure1b**. The inter-storey height is 4.0m, thereby giving rise to an overall height of 36.0m, 28.0m and 20.0m for 9, 7 and 5 storeys frames, respectively. For design purposes, the building is considered to be composed of moment resisting frames as lateral load resisting system along the perimeter (perimeter configuration), therefore a torsional amplification factor of 1.6 as proposed by EC8 is considered. It is because; the analysis is performed by using two planner models and therefore the torsional affect is determined by doubling the accidental eccentricity.

Table 1 Geometrical Parameters for the Analysed cases for 9, 7 and 5 Storey Frames

Label	Description of frame	\square Limit	Length of
5B-	5 Bays	0.01h	9.15
6B-	6 Bays		7.63
7B-	7 Bays		6.55
9B-	9 Bays		5.08
5B-	5 Bays	0.0075h	9.15
6B-	6 Bays		7.63
7B-	7 Bays		6.55
9B-	9 Bays		5.08

2.2 Design Criteria

Vertical loads acting on frames are evaluated according to EC0 [8] and EC1 [9], providing as a result of, a total gravity loading (structural and non-structural) equal to 4.6 kN/m² for roof and 7.8 kN/m² for typical floor; these includes imposed load of 0.4 kN/m² and 3.0 kN/m² for non-accessible roof and typical floor, respectively. The secondary beams are assumed to be simply supported with a bay width of 2.29m, oriented in such a way to have an optimized structural grid. All the secondary beams are designed using S-275 grade steel; these are HEB-220 for roof and HEB-280 for typical floor. The flooring system is composed of COMFLOR-46 [10], using A252 mesh and is comprised of 145mm thick concrete slab with 0.9mm steel sheeting. The masses according to EC8 for perimeter frames at typical floor level are 60061 kg-sec²/m while 50068 kg-sec²/m for roof.

Based on the provisions of EC3 and EC8, the primary beams are designed in order to satisfy both the ultimate and serviceability limit states using steel grade S-275. In particular primary beams are initially designed for gravity loads and then checked with reference to the seismic loading condition. The reference frames are designed according to EC8 with DCH ($q=6.5$), assuming type C soil stratigraphic profile (dense sand or gravel or stiff soil), important class II ($\gamma_r=1.0$), type 1 elastic response spectrum and 0.25g peak ground acceleration (see **Figure 2**).

2.3 Analysis and Design of Fames

Firstly, a linear modal dynamic analysis is developed using SAP 2000 [11] for the purpose of seismic design of the frames; then pushover analysis

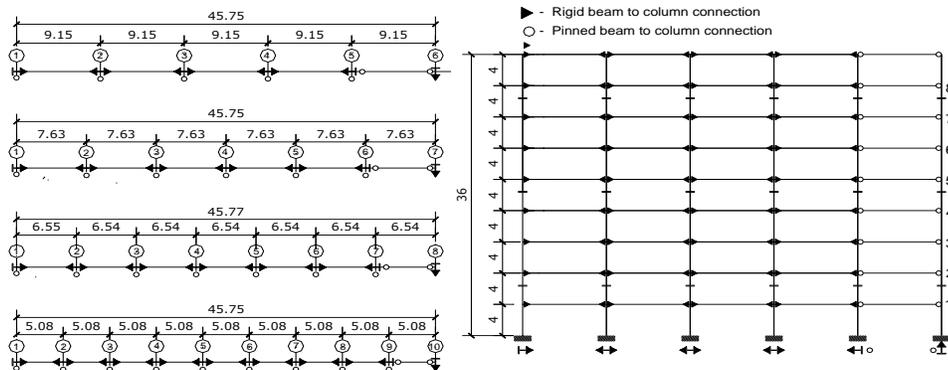


Fig. 1 (left) Typical Floor plan Of Perimeter MRFs and (right) Perimeter Frame Elevation

is performed in order to check the performance of the frames. The fundamental period of vibration from the codified formulation is found 1.3sec for 9-storeys, 1.03sec for 7-storeys and 0.8 sec for 5-storeys, which is lower than the period obtained from the modal response spectrum analysis (see **Table 2**).

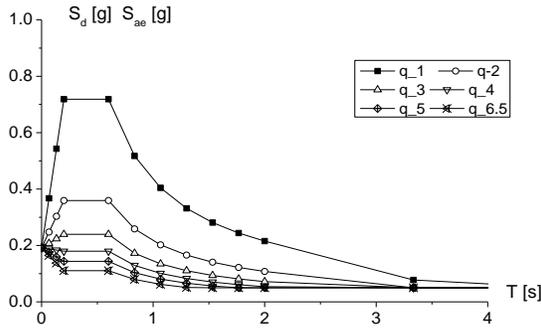


Fig. 2 Eurocode 8 Design Spectra for Various q Factors

This is due to the fact that simplified formulae given by seismic codes tend to underestimate the fundamental period of vibration, as they are based on empirical evaluation, therefore globally accounting for the stiffening effects of non-structural elements too, e.g. partition walls and in-fills etc. The connections of the examined frames are assumed fully rigid, therefore the detail discussions and their influences are assumed beyond the scope of the current research. All the framing members are

designed using EN 10025-2 S275 grade structural steel with the following properties: Adopted steel grade: EN 10025-2 S275 having unit density (ρ) = 76.9 kN/m³, Poisson's ratio (ν) = 0.3, modulus of elasticity (E in MPa) = 2.10E+05, yield 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.

3. Non-Linear Analysis

3.1 General

Static pushover analysis has been carried out using FEMA-356[12] recommendations for evaluating the lateral load resisting performance of the frames. For this reason triangular distribution (unit load at roof level) of static incremental loads has been applied and the displacement at the roof level has been controlled. For the ultimate rotation capacity of an element, acceptance criteria is defined, this is represented as IO (Immediate Occupancy), LS (Life Safety) and CP (Collapse Prevention). FEMA 356 acceptance criteria for non-linear procedure are adopted here. Mechanical non-linearity of the members has been assumed to be concentrated in plastic hinges at the ends (lumped plasticity) of the elements. Furthermore, as steel moment resisting frames own relatively long period therefore the “equal displacement rule” is employed to evaluate the so-called common parameters like “over-strength factor”, “ductility factor”, “elastic over-strength” and “redundancy factor”.

Table 2. Fundamental Period and Design Base Shear

Label	9 storeys			7 storeys			5 storeys		
	T [sec]	$V_{d-static}$ [kN]	V_d [kN]	T [sec]	$V_{d-static}$ [kN]	V_d [kN]	T [sec]	$V_{d-static}$ [kN]	V_d [kN]
5B-L1	2.26	2654	3499	1.84	2576	2719	1.32	2448	2022.0
6B-L1	2.29		3459	1.83		2773	1.28		2286
7B-L1	2.29		3248	1.86		2713	1.23		2323
9B-L1	2.00		3260	1.69		2666	1.30		2250
5B-L2	1.86		3639	1.50		2708	1.05		2302
6B-L2	1.84		3742	1.46		2882	1.05		2509
7B-L2	1.90		3628	1.41		3686	1.02		3819
9B-L2	1.84		3462	1.46		3150	1.00		3243

3.2 Pushover Curves

The obtained structural capacity curves are plotted in Figure 3(b, e), Figure 4 (b, e) and Figure 5(b, e) for 9, 7 and 5 storey frames using DCH in terms of total base shear (V_b) versus top

displacement (D_t) of the frames. Additionally, in Figure 3(a, d), Figure 4(a, d) and Figure 5(a, d), V_b is normalised with respect to V_y (the lateral load producing the first plastic hinge) giving rise to redundancy factors (Ω_p) for 9, 7 and 5 storey frames, respectively.

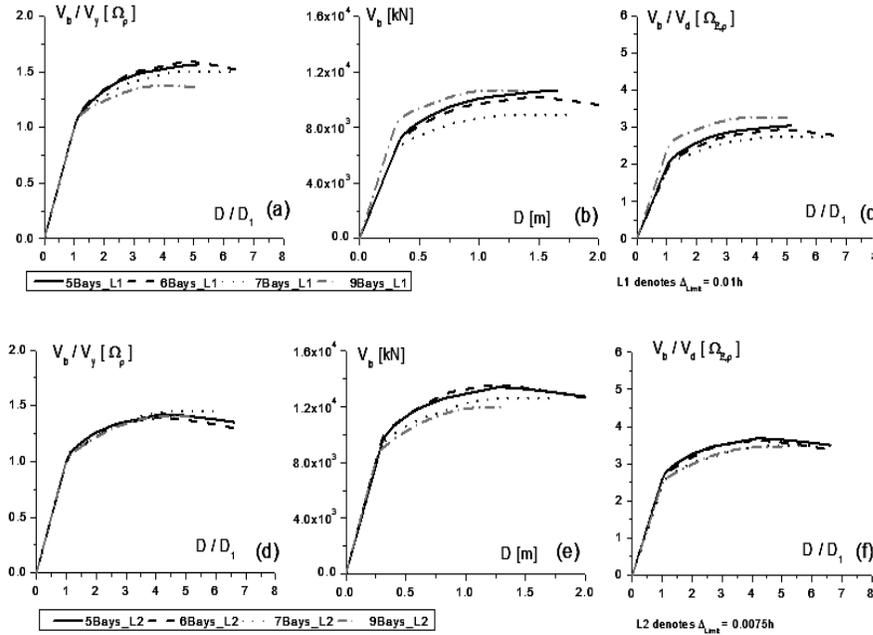


Fig.3 Pushover Curves and Normalised Pushover Curves for 9 Storeys Frames: (a, d) Redundancy Factor, (b, e) Pushover Curve and (c, f) Global over-strength

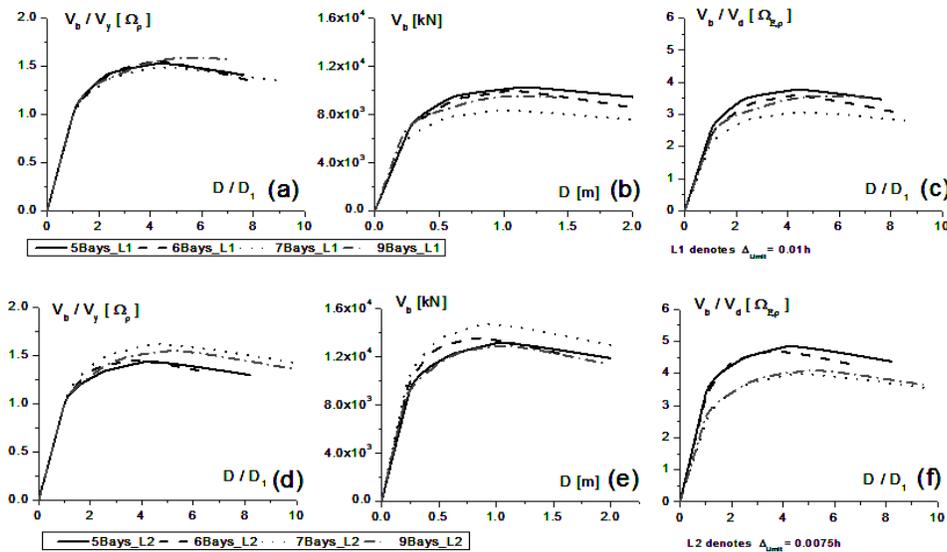


Fig.4 Pushover Curves and Normalized Pushover Curves for 7 Storeys Frames: (a, d) Redundancy factor, (b, e) Pushover Curve and (c, f) Global over-strength

Finally, in these graphs the total base shear (V_b) is normalised with respect to V_d (the design base shear), therefore giving rise to global over strength factors (Ω_{EP}) as shown by Figure 3(c, f), Figure 4(c, f) and Figure 5(c, f).

In the normalised graphs the top displacement (D_t) is normalised with δ_1 (the displacement corresponding to the first plastic hinge) therefore showing the corresponding ductility of the frames. In all these graphs the top row shows graphs for the frames when drift limit $L1$ ($0.01h$) is employed in the design with high ductility, whereas the bottom row shows graphs when drift limit $L2$ ($0.0075h$) is employed in the design. It is evident that as the number of storey increases:

- The global over-strength decreases, for example see Figure 3c (9 storeys) and Figure 4c (7 storeys) in which global over-strength is high for 7 storey frame,
- The base shear increases, for example see Figure 3b (9 storeys) and Figure 4b (7 storeys) where slightly high base shear can be observed in the case of 9 storeys.
- The redundancy factors remain approximately in the same range, for example see Figure 3a (9 storeys) and Figure 4a (7 storeys).

The effect of the drift limits $L1$ ($0.01h$) and $L2$ ($0.0075h$) can also be observed, for instance, the redundancy factors are in the same range approximately as expected, the base shear increases when drift limit $L2$ is employed for a corresponding frame (see **Figure 3b** for $L1$ and **Figure 3e** for $L2$) and the global over-strength also increases [13, 14].

3.3 Stiffness and Over-stiffness of the Analysed Frames

In this section, stiffness and over-stiffness of the designed frames are reported. It is normal that as the earthquake forces pushes the structure, the redistribution of the seismic forces take place due to the formation of plastic hinges. This redistribution causes the reduction of stiffness of the structures thereby the ductility of the structure increases. The reduction in stiffness due the increase in fundamental period that accompanies ductile behaviour tends to increase the amount of displacement the structure will experience as it is pushed by earthquake forces. The over-stiffness (Ω_k) is given by eq (1):

$$\Omega_k = \frac{\left(\frac{V_b}{\Delta}\right)}{\left(\frac{V_{elastic}}{\Delta_{Limit}}\right)} \quad (1)$$

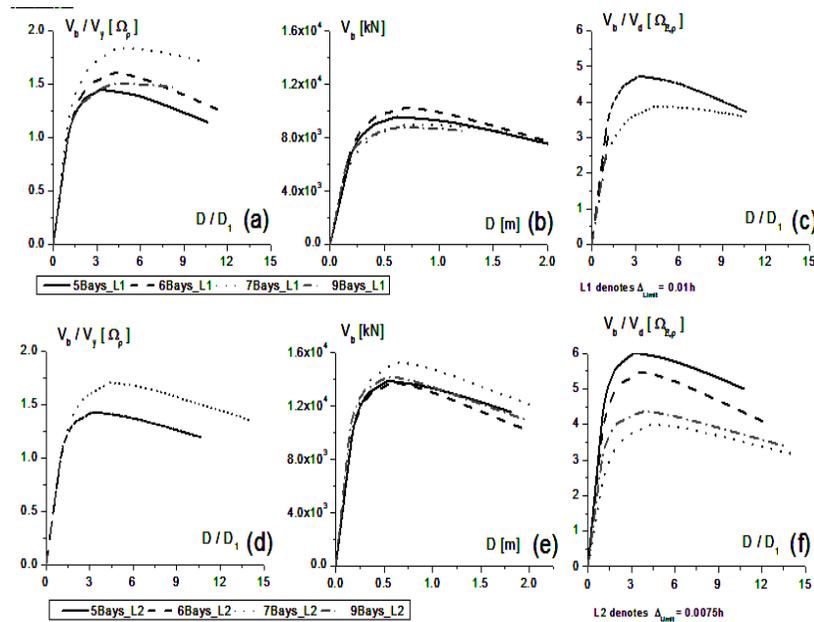


Fig. 5 Pushover Curves and Normalized Pushover Curves for 5 Storeys Frames: (a, d) Redundancy Factor, (b, e) Pushover Curve and (c, f) Global over-strength

Where V_b is the base shear obtained from pushover analyses, Δ is the corresponding displacement in the push-over, $V_{elastic}$ is the base shear obtained from modal analyses using the elastic spectrum reduced by a factor equals 2.0 (that allow for the lower return period of the seismic event

related to the damageability limit state) and Δ_{Limit} is the Inter-storey drift limit.

The stiffness and the over-stiffness of the designed frames are shown in Table 3 and are illustrated by Figure 6, Figure 7 and Figure 8 for 9, 7 and 5 storey frames, respectively.

Table 3 Stiffness and Overstiffness of 9, 7 and 5 Storeys Frames

Label	9 storeys			7 storeys			5 storeys		
	$K_{elastic}$ [kN/m]	$K_{obtained}$ [kN/m]	Ω_k	$K_{elastic}$ [kN/m]	$K_{obtained}$ [kN/m]	Ω_k	$K_{elastic}$ [kN/m]	$K_{obtained}$ [kN/m]	Ω_k
5B-L1	24828.6	27295.4	1.10	23096.4	25565.5	1.11	21410.7	25274.5	1.18
6B-L1	25590.2	28813.7	1.13	24383.5	27385.4	1.12	23526.3	29403.2	1.25
7B-L1	28178.4	31642.8	1.12	26622.4	29285.2	1.10	25033.2	29023.9	1.16
9B-L1	35167.4	45626.2	1.30	32958.1	38410.7	1.17	29809.3	38662.9	1.30
5B-L2	29271.4	30097.7	1.03	27378.6	28264.5	1.03	26589.3	31186.0	1.17
6B-L2	31289.5	33080.1	1.06	29672.9	32881.6	1.11	28150.4	33449.1	1.19
7B-L2	33215.8	34358.5	1.03	33576.8	39202.2	1.17	31634.9	39629.2	1.25
9B-L2	38437.2	41290.9	1.07	36595.3	40454.8	1.11	37674.4	49478.7	1.31

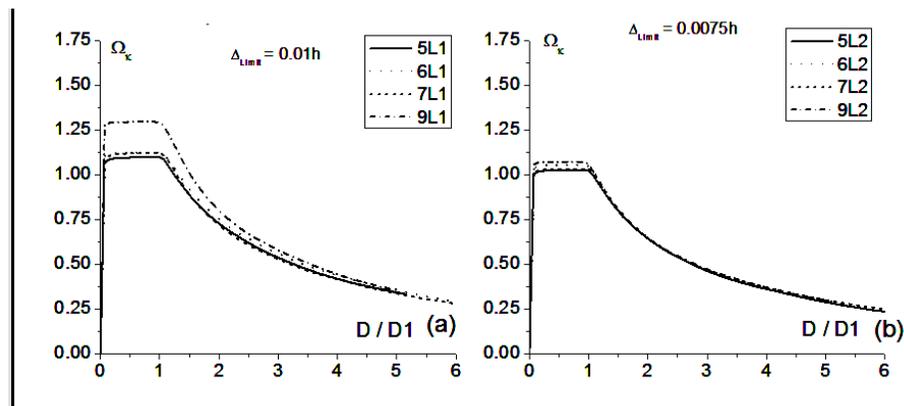


Fig.6. Over-stiffness Factors for 9 Storeys frames: (a) $\Delta_{Limit} = 0.01h$ and (b) $\Delta_{Limit} = 0.0075h$

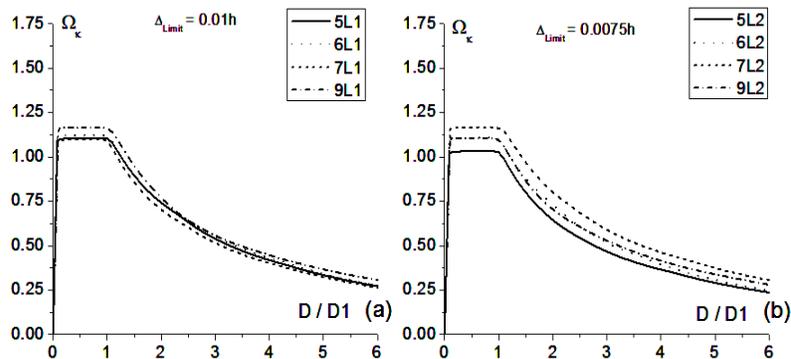


Fig. 7. Over-stiffness Factors for 7 Storeys Frames: (a) $\Delta_{Limit} = 0.01h$ and (b) $\Delta_{Limit} = 0.0075h$

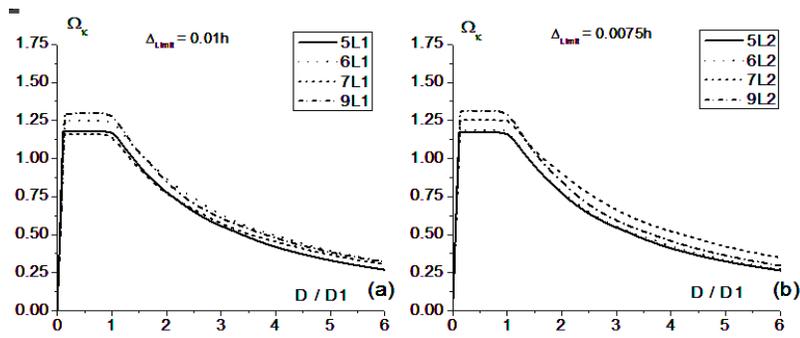


Fig. 8 Over-stiffness Factors for 5 Storeys Frames: (a) $\Delta_{Limit} = 0.01h$ and (b) $\Delta_{Limit} = 0.0075h$

The over-stiffness for a given frame when its around 1.0 shows that the design of the frame is dictated by damageability (drift criteria), instead when it is greater than 1.0 means that strength controls the design. It is clear in the case of 9 story frame when sized for drift limit L2 (0.0075h), the drift governed the design of the frames (see Figure 6b) and hence the assumed ductility (q equals 6.5) is not utilised completely. In order, to see the influences of drift limits in the forthcoming sections, the ductility factors are evaluated in this section. Further the optimum behaviour factor is evaluated for each frame by iterations.

3.4 Ductility, Redundancy and Overstrength Factor

In Figure 9, the calculated overstrength from the codified formulations (Ω_{calc}) is shown for all the 24 designed frames. From these graphs, an increasing trend can be observed as the No. of storeys of frame decreases from 9 to 5. Furthermore, over-stiffness of the frames (Ω_k) are also reported which were mentioned in the previous section. In addition, elastic overstrength (Ω_E), global overstrength (Ω_{EP}) demonstrate an increasing trend whereas redundancy factor (Ω_r) for each frame is always constant.

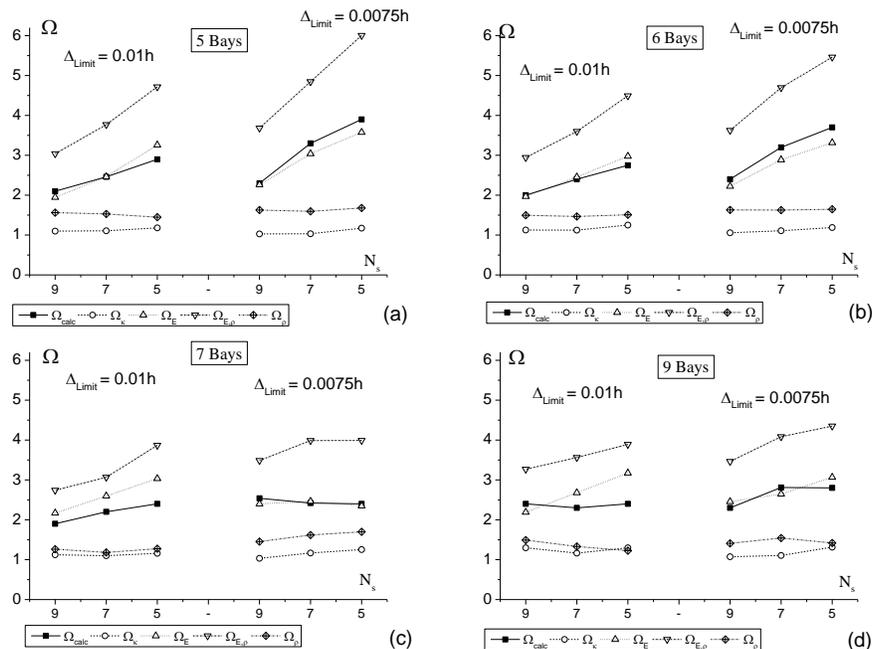


Fig. 9 Over-strength Factors for the Analysed Frames: (a) 5 Bays (b) 6 Bays (c) 7 Bays and (d) 9 Bays

From all these parameters the effect of drift limit can be easily observed as all such parameters except over stiffness of frames increases when the drift limit changes from $L1$ ($0.01h$) to $L2$ ($0.0075h$).

3.5 Optimum Versus Code Prescribed Behaviour Factors

In Figure 10, Figure 11 and Figure 12 the behaviour factors are plotted but it is observed that the obtained behaviour factor from pushover analysis are high from the code specified factor for short span frames in the cases of both drift limits ($L1$ and $L2$). It has to be mentioned here that the ultimate base share is defined as the maximum obtained from the pushover analysis. The optimum q factors are obtained by iterative procedure from response spectrum analysis.

These are strictly related to the span of the frames as well as to the drift limitations. From the design and analysis of 9 storeys frames, Eq (2) can be used to illustrate the case.

$$q_{code} > q_{optimum} \text{ and } q_{optimum} < \frac{\Delta_u}{\Delta_y} \quad (2)$$

Therefore, leads to declare that these frames will be suitable if designed with medium ductility ($q = 4.0$) rather than 6.5. Similarly for 7 and 5 storey frames the relation as shown by Eq (3) holds, representing that the ductility of the frames increases as the number of storeys decreases.

$$q_{code} < q_{calculated} \text{ and } q_{optimum} < \frac{\Delta_u}{\Delta_y} \quad (3)$$

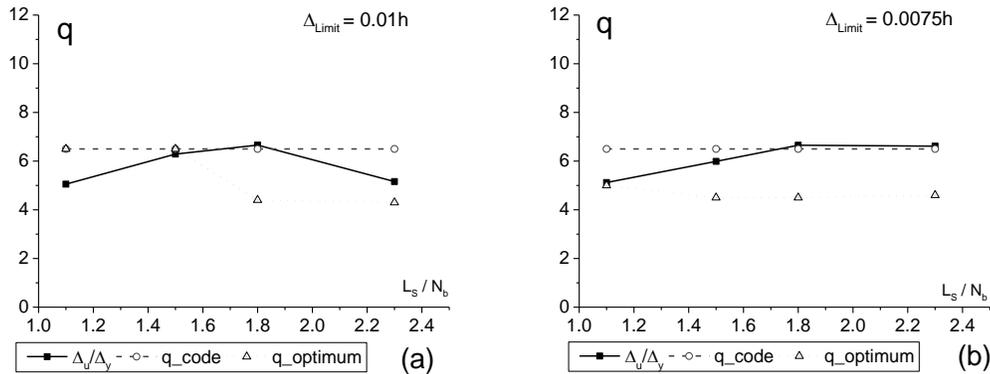


Fig. 10 Ductility and behaviour Factors for 9 Storeys Frames: (a) $\Delta_{Limit} = 0.01h$ and (b) $\Delta_{Limit} = 0.0075h$

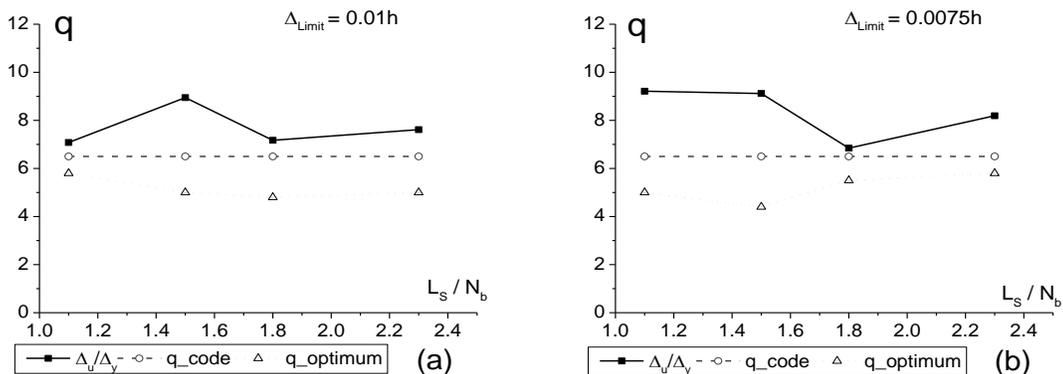


Fig. 11 Ductility and behaviour Factors for 7 Storeys Frames: (a) $\Delta_{Limit} = 0.01h$ and (b) $\Delta_{Limit} = 0.0075h$

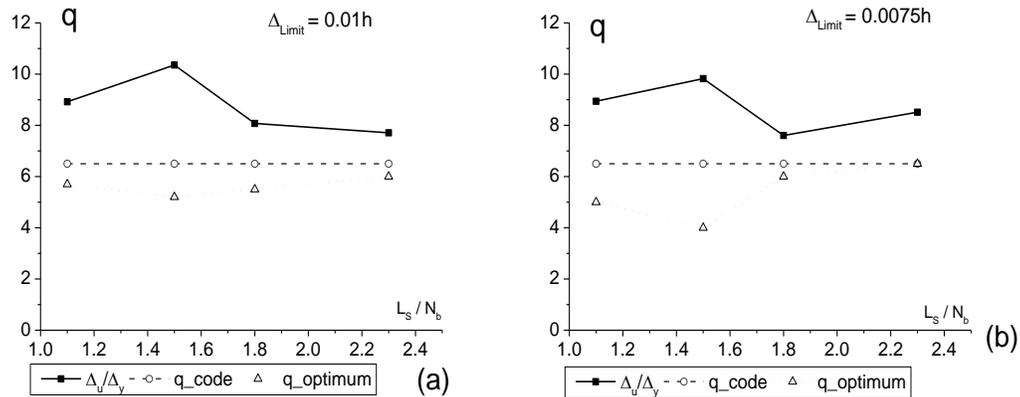


Fig. 12 Ductility and behaviour Factors for 5 Storeys Frames: (a) $\Delta_{Limit} = 0.01h$ and (b) $\Delta_{Limit} = 0.0075h$

In the above designed frames the calculated behaviour factor from pushover analysis are strictly related to the period of the structures, as the period of the frame increases the ductility and thus the corresponding behaviour factor decreases. For instance for the analysed cases, the fundamental period (T) for 9-story frames are in the range of 1.7sec to 2.4sec and the ductility factors range from 5.0 to 7.0, similarly periods for 7 and 5 story frames are (1.4 to 2.0 sec) and (0.9 to 1.4 sec), respectively and the ductility factors ranges from (7.0 to 9.0) and (7.0 to 10.0), respectively.

At the end it is found that if these frames are designed with medium ductility (q equals 4.0) it might result in optimum use of behaviour factor and therefore shall lead to more economical solution. Furthermore, in these cases if a frame is designed with high ductility (q equals 6.5) the capacity design rules could be relaxed by redefining the over-strength factor or at least could limit the elastic over-strength.

4. Conclusions

From the presented paper it is concluded that as the number of storey increases:

The global overstrength decreases, the base shear increases and the redundancy factor remains approximately in the same range.

Furthermore, the main outcomes of the case study may be synthesised as follows:

The behaviour factor specified by the code for high ductility is not completely utilised due to the high Interstorey drift limits given by the code, this needs either to design such frames with Medium Ductility or relaxing the capacity design rule.

When drift limitation is stringent will lead to govern the design, thereby the behaviour factor especially for high ductility cannot be optimally used and therefore leads to uneconomical design situation as the targeted ductility cannot be achieved at the end of the design.

It is therefore required to propose the design of frames in a more sophisticated way as it gives high performance and completely avoiding or relaxing the capacity design rules.

The code specified ductility class (high) is not compatible with the code proposed drift limits; instead it is strictly important to limit the ductility when the design is govern by drift.

In view of the above, it is aimed to follow and proceed with the current research activity for presenting optimised rules to allow the technicians to design steel Moment Resisting Frames more easily, efficiently and economically.

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