Direct Displacement-Based Seismic Design of Steel Moment Frames

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SUMMARY:
A new generation of performance based seismic design procedures is emerging during the two last decades. These methodologies aim to provide the designers with better control of deformation and, consequently, of the structural damage. The Direct Displacement-Based Design (DDBD) is one of the most promising displacement-based design methods. In this paper a comparison between different seismic design procedures for steel moment-resisting frames is presented. A set of steel moment-resisting structures are designed according to Eurocode 8 (EC8) and to the direct displacement-based design. The performance assessment of the structural systems is performed through nonlinear static and time-history analyses. The results obtained allow concluding that the structures designed according to DDBD do not necessarily exhibit better structural performance in comparison with those designed according to EC8.

Keywords: Steel; Seismic design; Direct Displacement-Based; Force-based design; EC8

1. INTRODUCTION

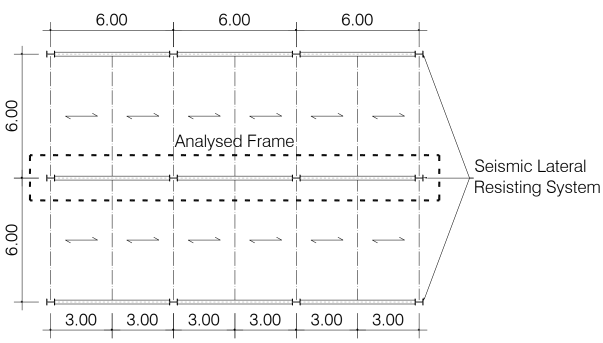
Steel moment-resisting frames are well known for their ductile and stable hysteretic behaviour. Nonetheless, past experience demonstrated that these structural systems are still prone to unreliable seismic performance. Current seismic design codes focus largely on strength control with collapse prevention in mind. Therefore, limited attention is paid to deformation and damage control. However, a new generation of design procedures has been developed during the last two decades aiming to provide designers with better control of seismic performance. The remarkable advantage of these promising methods is the ability to define and control the performance objective for each structure. These emerging methods are the so-called displacement-based design methods and, as mentioned before, they allow a more reliable control of the inelastic deformations and hence of the amount of damage that the structure will develop. Two of the many displacement based design methods are the Direct Displacement-Based Design (DDBD) proposed by Priestley *et al.*, (2007) and the yield point spectra (YPS) proposed by Ascheim and Black (2000). Both methods are based on the transformation of the multi-degree into a single degree of freedom system and have as the starting point of the design the definition of the target displacement which is often related with the performance level required for the structure. The YPS method is based on the initial stiffness of the structure and resorts to the inelastic response spectra to obtain the required base shear. On the other hand, the DDBD is based on the substitute structure approach (Shibata and Sozen, 1976) and uses the secant stiffness associated with the elastic response spectra but introducing the equivalent viscous damping concept. A review of the most prominent displacement-based design procedures applied to reinforced concrete structures can be found in Sullivan (2002). These methods have been developed and widely applied to reinforced concrete structures and applications to steel moment-resisting frames and concentric braced frames are still limited (Maley *et al.*, 2010). The objective of this paper is to present an application of the DDBD to steel moment-resisting systems and to identify current limitations associated with that design procedure in terms of its applicability to the case of steel structures. The same systems are also designed according to the force-based procedure defined in Eurocode 8. The seismic performance of the structures is carried out through nonlinear static and time-history analyses.

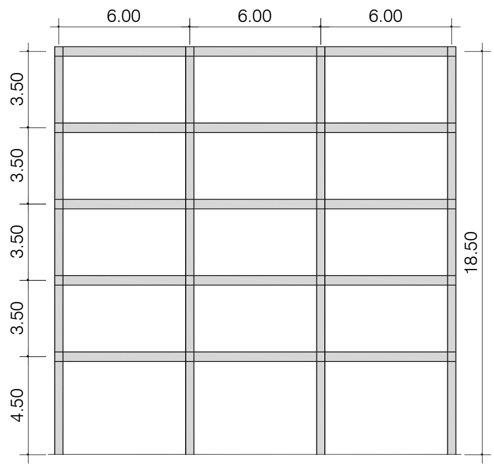
2. DESCRIPTION OF THE STUDY

The structures considered in this study consist of a group of four regular steel moment-resisting frames with different heights, namely 5, 8, 12 and 18 storeys. Due to space limitations, only the results obtained for the 5-storey frame will be presented in this paper.

2.1. Building Configuration

The structural configuration in plan and elevation of the studied building is shown in Fig. 2.1. As shown in the figure, the building consists of three moment-resisting frames spaced at 6 m and connected by 6 m secondary beams that support a concrete slab. Resistance to seismic loads is provided by the three moment-resisting frames in the longitudinal direction and by a bracing system in the transverse direction. The moment-resisting frame (MRF) to be studied is identified in Fig. 2.1(a).





(a) **(b)**

Figure 2.1. (a) Plan view and (b) elevation of the 5-storey structures

2.2. Frame Design

The frame was firstly designed for gravity loads in accordance with the provisions of Eurocode 3 (2005) for sectional resistance, stability checks and deflection limits. European HE sections were adopted for the columns and IPE sections for the beams. The steel grade considered was S275.

The seismic design of the frames was performed according to the EC8 and DDBD procedures. The frames were designed to resist seismic action Type 1 prescribed in Eurocode 8 considering a value of PGA of 0.3g, soil type of class B. The inter-storey drift ratio (IDR) for the damage limitation check was limited to 1%.

Concerning the EC8 designs, three cases were considered corresponding to three different values of the behaviour factor. According to Table 6.2 of EC8 the maximum values of the behaviour factors that can be adopted for multi-storey multi-bay steel moment frames are 4.0 for medium ductility class systems and 6.5 for high ductility class systems. In this work these two values were considered in addition to a third design case in which a more rational selection of the behaviour factor was considered, following the Improved Force-Based Design (IFBD) procedure proposed by Villani *et al.*, (2009). A summary of the four design cases considered is provided in Table 2.1.

Table 2.1. Studied cases

Design Case	Design Procedure	
Case 1	EC8	q=4.0
Case 2	EC8	q=6.5
Case 3	EC8	q (IFBD)
Case 4	DDBD	

2.2.1. Eurocode 8 designs

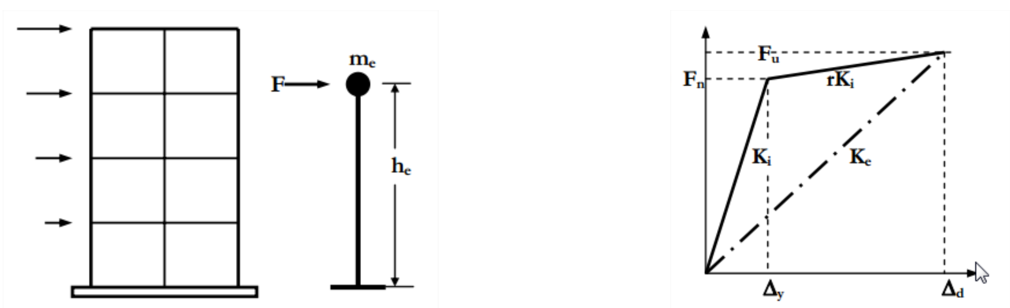
Two limit states need to be verified according to EC8, namely collapse prevention and damage limitation limit states. Although it is not specifically defined in EC8, it is considered by the authors that the first step of the design process should be that of checking the damage limitation level as it often governs the design, particularly in the case of flexible structures located in moderate to high seismicity regions. Moreover, the verification of this limit state is independent of the value of the behaviour factor as the code refers to the application of the equal displacement rule for the calculation of the structural deformations when the fundamental period is above the corner period of the acceleration response spectrum. If the maximum inter-storey drifts exceed the code limit then the member sizes must be increased. Regarding the collapse prevention limit state, the design process consists of checking the dissipative elements followed by capacity design of the non-dissipative members. The capacity design was conducted according to the EC8 criteria with the modifications proposed by Elghazouli (2009). Additionally, the potential influence of second-order (or P-Δ) effects should be checked through the calculation of the inter-storey sensitivity coefficient, or stability factor, θ. In all the analysis the θ coefficient was limited to 0.2, meaning that lateral load amplification had to be performed during the design process. The cross-sections adopted for beams and columns are listed in Table 2.2.

Table 2.2. Beam and column sections for *q*=6.5, *q*=4.0 and IFBD.

	q=6.5			*q*=4.0			IFBD (*q*=2.1)		
Floor	Beams	Int. Col.	Ext. Col.	Beams	Int. Col.	Ext. Col.	Beams	Int. Col.	Ext. Col.
1	IPE400	HE450B	HE400B	IPE360	HE400B	HE340B	IPE360	HE340B	HE280B
2	IPE400	HE450B	HE400B	IPE360	HE400B	HE340B	IPE360	HE300B	HE260B
3	IPE400	HE360B	HE340B	IPE330	HE340B	HE280B	IPE330	HE300B	HE260B
4	IPE330	HE360B	HE340B	IPE300	HE340B	HE280B	IPE330	HE240B	HE220B
5	IPE300	HE360B	HE340B	IPE300	HE340B	HE280B	IPE300	HE240B	HE220B

2.2.2. Displacement-based designs

Conversely to force-based procedures, the displacement-based design (DBD) methods focus on the control of the lateral displacements of the structure. Since the damage is easier to correlate with displacements than with forces, these methods aim to design a structure that respects a pre-defined maximum displacement. From all the available DBD methodologies the direct displacement-based design (DDBD) procedure (Priestley *et al.*, 2007) is probably the one gaining more acceptance from the scientific community. The main steps of the DDBD are illustrated in the Fig. 2.2.



(a) SDOF substitute structure

(b) Effective stiffness at the design displacement

(c) Equivalent damping vs. ductility (d) Design displacement spectra

Figure 2.2. Direct displacement-based design (Priestley et al., 2007)

By assuming a target displacement, based on pre-defined performance criteria, and a lateral displacement shape, the DDBD replaces the multi-degree of freedom (MDOF) system by an equivalent single-degree of freedom system (SDOF) (Fig. 2.2 (a)), for which it is possible to define the effective mass (m_e), the effective height (h_e) and the target ultimate displacement (Δ_d). The next step of the method consists of obtaining the equivalent viscous damping of the SDOF system. The effective viscous damping is defined as function of the displacement ductility demand ($\mu=\Delta_d/\Delta_y$), obtained using empirical relationships for the yield displacement, and structural system (Fig. 2.2 (c)). The equivalent viscous damping is then used to reduce the elastic displacement spectrum. Finally, by entering the spectrum with the target displacement one can find the effective period of the substitute structure (T_e) and hence the effective or secant stiffness:

$$K_{eff} = 4\pi^2 \frac{m_e}{T_e^2} \tag{2.1}$$

where m_e and T_e are the effective mass and the effective period, respectively. The base shear can be obtained using Eqn. 2.2 as follows:

$$V_b = K_{eff} \cdot \Delta_d \tag{2.2}$$

The base shear (V_b) is then distributed over the height of the structure as a set of lateral forces proportional to the assumed lateral displacement shape and to the floor masses. The required member strengths are finally obtained using an equilibrium approach (Priestley *et al.*, 2007).

In the application of the DDBD to the structure of Case 4, the lateral displacement shape considered was based on the expression proposed in the DDBD model code (Calvi *et al.*, 2009). The yield drift of the structure, used in the calculation of the yield displacement, was calculated according to the equation proposed by (Garcia *et al.*, 2009), as follows:

$$\theta_y = \frac{\phi_{yb} L_b + 0.9\phi_{yc} L_c}{6} \tag{2.3}$$

The calculation of the design displacement (Δ_d) was based on the limitation of the maximum inter-storey drift to 2%, a value typically considered when checking collapse prevention. The evaluation of the equivalent viscous damping was performed according to the ductility-equivalent viscous damping relationship proposed by (Priestley *et al.*, 2007):

$$\xi_{eq} = 0.05 + 0.577 \cdot \left(\frac{\mu - 1}{\pi\mu} \right) \tag{2.4}$$

Table 2.3 provides a summary of the parameters involved in the design process of Case 4.

Table 2.3. Parameters considered in the DDBD of Case 4 structure

Parameter	Value
Design displacement (Δ_d)	0.222 m
Effective mass (m_e)	292.9 t
Effective height (H_e)	12.96 m
Yield displacement	0.19 m
Displ. ductility (μ)	1.16
Equiv. viscous damping (ξ_{eq})	7.5 %

The cross-sections adopted for beams and columns are listed in Table 2.4.

Table 2.4. Beam and column sections obtained with the DDBD procedure.

DDBD			
Floor	Beams	Int. Col.	Ext. Col.
1	IPE 300	HE 260 B	HE 220 B
2	IPE 300	HE 260 B	HE 220 B
3	IPE 300	HE 220 B	HE 200 B
4	IPE 300	HE 220 B	HE 200 B
5	IPE 300	HE 220 B	HE 180 B

As discussed in the following section, the solution obtained with the DDBD procedure is substantially different from those obtained with EC8.

2.2.3. Discussion of the structural solutions

The obtained solutions for the different design methodologies are now compared in terms of the frame steel weight. The main difficulties identified during the design process are also highlighted. The steel weight of each solution is presented in Fig. 2.3. It is clear from the figure that the lighter solution is that designed according to the DDBD. Among the EC8 designs, the most optimized solution is that in which the behaviour factor was selected according to the IFBD procedure. Notwithstanding these observations, it should be noted that the DDBD solution is effectively the solution that resulted from the gravity loading design. However, it is worth noting that, according to a modal response spectrum analysis the structure complies with the inter-storey drift limit of 1% adopted for the damage limitation limit state. Moreover, the structure also complies with the capacity design criteria prescribed in EC8, namely the local ductility condition which is evaluated at every joint.

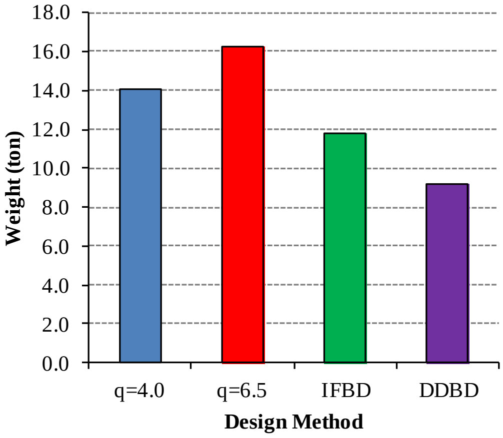


Figure 2.3. Frame steel weight

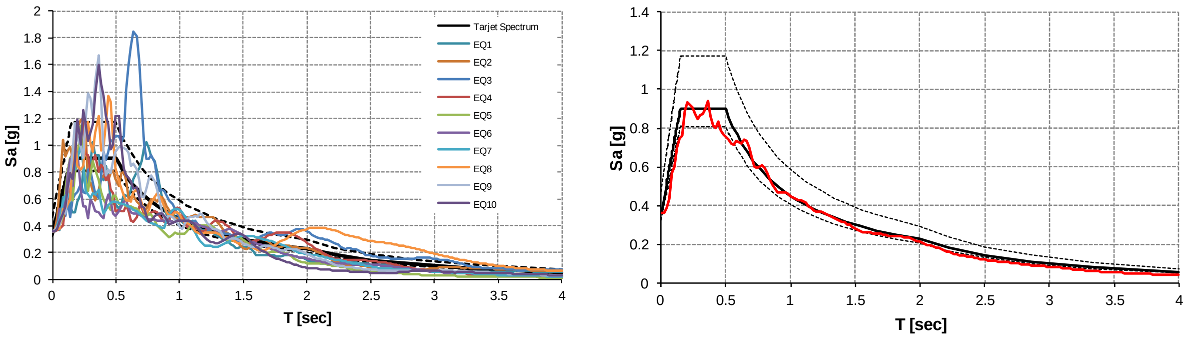
It is worth pointing out that among the EC8 designs, the lighter solution was obtained with the lowest behaviour factor (q=2.1). This observation may seem contradictory. However, the reason for the stiffer solutions obtained with higher behaviour factors is justified with the stringent storey stiffness requirements to control the level of P-Δ effects (θ<0.2) (Villani *et al.*, 2009).

Regarding the DDBD solution, several questions have been raised related with the application of the procedure to the studied frames. Some of the issues have been recently reported by Kappos *et al.* (2010; 2012). The problem of applying the DDBD to structures located in regions of low and moderate seismicity is one of the key issues. In all the frames designed in the context of this research, the target displacement of the substitute structure was higher than the maximum displacement obtained from the 5% EC8 elastic response spectrum. Another issue is related with the value of the corner period of the displacement spectrum. The current value of EC8 is low in comparison with the values found in other seismic codes and that contributes in a significant manner to the previous issue discussed above. Some proposals have been made by several researchers (e.g. Bommer and Elnashai, 1999, Faccioli *et al.*, 2004) that suggest that the corner period (T_D) of the design displacement spectrum should not be a fixed value and should be function of other parameters such as the earthquake magnitude.

Another issue found with the application with the DDBD is related with the fact that the yield displacement of the 5-storey frame calculated with Eqn. 2.3 was almost equal to the maximum value of the 5% damped displacement response spectrum (Δ_y=0.19 and $\Delta(T_D)$=0.22). That situation indicated that the frame response should be largely elastic when subjected to the design earthquake. However, the value of 1.48% of yield drift obtained with Eqn. 2.3 is expected to be associated with some larger degree of plasticity within the frame.

3. NUMERICAL MODELLING AND ANALYSIS PROCEDURES

The assessment of the structures was carried out through nonlinear static and dynamic analysis conducted with the finite element analysis program OpenSees (PEER, 2006). The material nonlinear behaviour was considered through a fibre modelling approach. The beams and columns members were modelled using forced-based fibre elements with 10 integration points. The panel zones were represented with a beam-column joint element that is available in the program. The material model used for the steel was a bi-linear stress-strain curve with a strain hardening of 1.0%. For the panel zones, the Krawinkler (1978) tri-linear moment-distortion relation was adopted. A static analysis considering the gravity loads was firstly conducted for each frame and the resulting stress and strain state was considered as the initial state for the seismic analysis. The pushover analyses were performed by assuming a linear load pattern proportional to the floor mass and height. Nonlinear time-history analyses have been performed subjecting the frames to a set of ten ground motion records obtained from the European Strong Motion database. The records have been selected based on the geophysical parameters M and R and imposing compatibility with the EC8 type 1 elastic response spectrum using the computer aided record selection REXEL (Iervolino *et al.*, 2010). A maximum deviation of 10% of the average spectrum from the EC8 spectrum was imposed in the period range of interest (0.6s to 2.0s). Fig. 3.1(a) shows the elastic response spectra for the ten records along with the EC8 response spectrum. Fig. 3.1(b) depicts the average and code elastic response spectra. Tangent stiffness proportional damping has been considered assuming a viscous damping of 2% for the first mode.



(a) (b)

Figure 3.1. Mathematical models for nonlinear response history analysis

4. DISCUSSION OF RESULTS

4.1. Nonlinear static response

The pushover curves obtained for the four frames are plotted in Fig. 4.1. As expected, the frames designed according to EC8 with higher behaviour factors exhibit significant lateral strength due to the member size increase required to comply with the requirements imposed by EC8 to limit P-Δ effects. The pushover curves obtained for these design cases show an important discrepancy between the design base shear and the actual lateral strength. A summary of the elastic base shear and design base shear for the frames designed according to EC8 is presented in Table 4.1.

Table 4.1. Elastic and design base shear corresponding to the EC8 designs.

Design Case	Behaviour factor (*q*)	V_{el} [kN]	V_d [kN]
Case 1	4.0	780.5	195.1
Case 2	6.5	921.8	141.8
Case 3	IFBD (*q*=2.1)	698.8	329.6

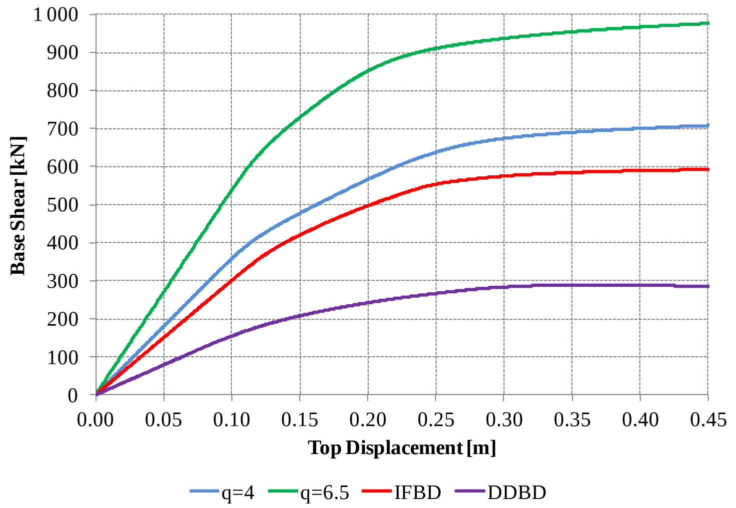
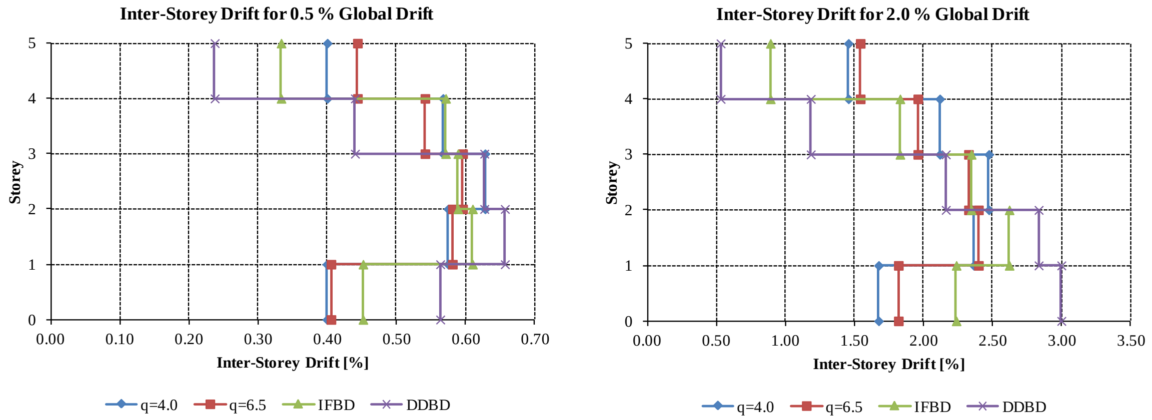


Figure 4.1. Mathematical models for nonlinear response history analysis

It is worth noting that the frame designed according to EC8 in which the IFBD procedure was adopted for the selection of the behaviour factor shows a good agreement between the design base shear (about 330 kN) and the lateral strength at the formation of the first plastic hinge.

Finally, Fig. 4.1 clearly shows that the frame designed according to the DDBD procedure exhibits substantially lower lateral strength in comparison with the other frames. Although in the design process it was anticipated that the seismic response of the frame would be largely elastic, it is possible to conclude from the pushover curve that the frame will observe inelastic response with an expected global ductility demand of about 2.0 when subjected to the design earthquake. This conclusion can be easily inferred after a simple estimate of the elastic base shear which is around 505 kN. This observation clearly points out to some limitations associated with the current proposals of DDBD for steel moment frames.

The lateral deformation of the frames is now examined based on the analysis of the inter-storey drift distributions. Fig 4.2 shows the inter-storey drift profiles for two different levels of global drift, namely 0.5% and 2%.



(a) (b)

Figure 4.2. Inter-storey drift distributions for a global drift of (a) 0.5% and (b) 2.0%

The inspection of the figure reveals that the four structures develop different inter-storey drift profiles, particularly for higher levels of global lateral deformation. Another interesting observation is that the critical storey of the DDBD solution shifts from the 2nd storey at low levels of deformation to the 1st storey when the structure is subjected to larger deformation demands. The EC8 solutions develop similar inter-storey drift profiles but the structure designed with the lower behaviour factor exhibits more uniform inter-storey drifts over the height of the system.

4.2. Nonlinear Time-History Analysis (NLTH)

The performance of the frames is now examined based on the interpretation of the results provided by the time-history analyses. Fig. 4.3 shows the mean values of the maximum inter-storey drifts recorded in the four frames subjected to the 10 seismic records which have been scaled to match the EC8 elastic response spectrum adopted in the design process.

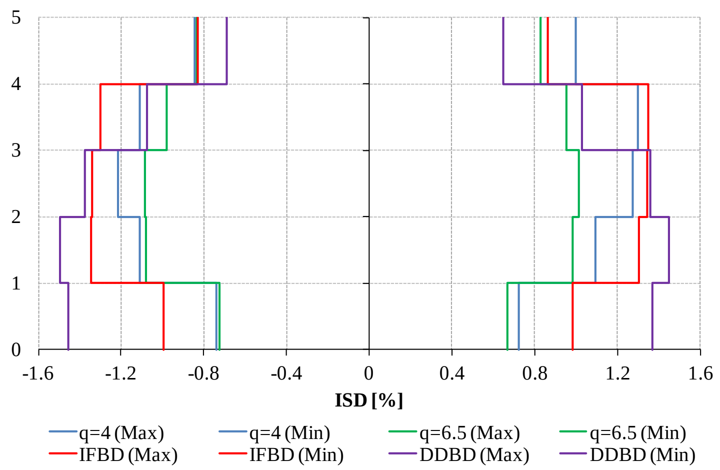
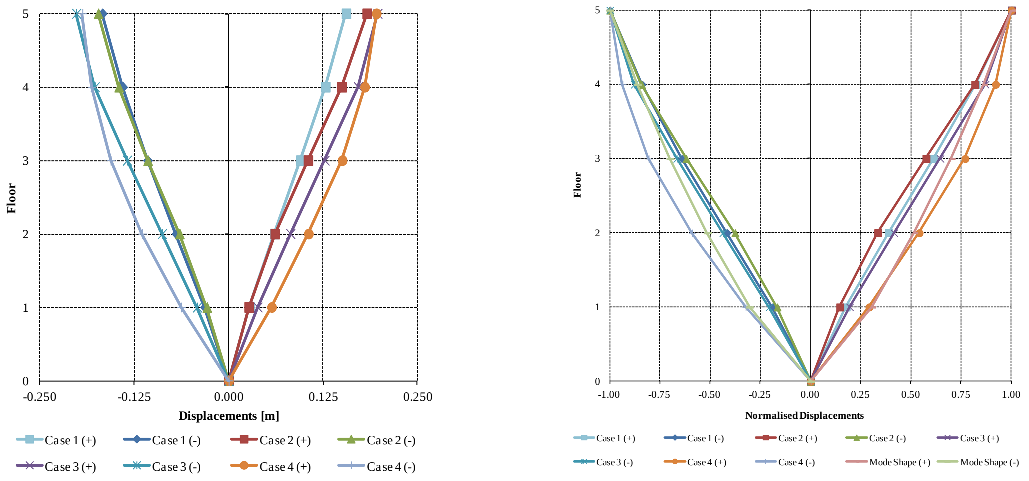


Figure 4.3. Mean values of the maximum inter-storey drifts obtained from the time-history analyses

Although EC8 does not define limits for the inter-storey drifts at the design earthquake intensity level, the figure allows concluding that the inter-storey drifts recorded in all frames do not exceed 1.6%, a value that is clearly below the typical limits of 2.0% to 2.5% prescribed in other seismic codes with the aim of ensuring collapse prevention. The frames designed according to EC8 with values of 4.0 and 6.5 for the behaviour factor clearly develop lower inter-storey drift levels, confirming once again the high lateral stiffness associated with these structural solutions.

The mean lateral displacement profiles at peak inter-storey drift and the normalised lateral displacement profiles are depicted in Fig. 4.4 (a) and (b), respectively. The expression of the lateral mode shape proposed in the DDBD model code is also plotted in Fig. 4.4 (b) for comparison purposes.



(a) (b)

Figure 4.4. (a) Mean lateral displacement profiles at peak inter-storey drift; (b) Normalised lateral displacement profiles and comparison with DDBD model code expression for mode shape.

The figure allows concluding that the displacement shapes of the frames designed according to EC8 are very similar although, as expected, Case 3 frame exhibits larger displacements. The lateral displacement shape of the frame designed according to the DDBD (Case 4) is very different from the remaining frames, particularly at the lower floor levels. It is interesting to note in Fig. 4.4(b), that at the bottom stories the normalised displacement shape of the structure is close to the normalised assumed mode shape considered at the design stage. However, for the intermediate stories of the structure, there is a deviation between the actual displacement shape and the assumed mode shape.

5. CONCLUSIONS

In this paper a parametric study was conducted with the aim of investigating the performance of steel moment frames designed according to Eurocode 8 and the direct displacement-based design. Concerning the structures designed according to EC8, it has been found that the frames designed with higher behaviour factor were governed by stiffness requirements due to the need to limit the presence of P-Δ effects. This situation has been already reported in previous studies (Villani *et al.*, 2009 and Peres and Castro, 2010) which showed that the more optimised solutions obtained with the current prescriptions of EC8 are achieved with the adoption of lower behaviour factors.

The application of the DDBD allowed identifying some limitations associated with the current version of the method, namely *i)* the difficulties of applying the method to structures located in regions of low and moderate seismicity, *ii)* the value of the corner period of the displacement spectrum which in some cases requires an iterative process if full optimization is envisaged by the designer and the iii) high values of the yield drift that are obtained with existing expressions that in some cases may wrongly indicate elastic seismic response to the design earthquake. These issues clearly point out to the need for the development of new research on this field in order to turn the DDBD procedure into an effective design tool for steel structures.

AKCNOWLEDGEMENT
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