Steel and Composite Structures, *Vol. 22, No. 5 (2016) 1115-1140*
DOI: http://dx.doi.org/10.12989/scs.2016.22.5.1115

An extension of an improved forced based design procedure for 3D steel structures

R. Peres [1b], J.M. Castro [2a] and R. Bento [*1]

[1] *Instituto Superior Técnico, Universidade de Lisboa, Lisboa, Portugal*
[2] *Faculdade de Engenharia da Universidade do Porto, Porto, Portugal*

(*Received June 15, 2016, Revised November 01, 2016, Accepted November 11, 2016*)

Abstract. This paper proposes an extension of the Improved Forced Based Design procedure to 3D steel structures. The Improved Forced Based Design (IFBD) procedure consists of a more rational sequence of the design checks proposed in Eurocode 8 and involves a more realistic selection of the behaviour factor instead of selecting an empirical value based on the ductility class and lateral resisting system adopted. The design procedure was tested on a group of four 3D steel structures, composed by moment-resisting frames with three storeys height and the same plan configuration in all storeys. The plan configuration was defined in order to target lateral restrained or unrestrained systems as well as plan regular or irregular structures. The same group of structures was also designed according to the force-based process prescribed in Eurocode 8. The member sizes obtained through the two approaches were compared and the seismic performance was assessed through nonlinear static and time-history analyses. The limit states referred to structural and non-structural damage, considering the two levels design approach, which are the serviceability and the ultimate limit states, were examined. The results obtained reveal that the IFBD leads to more economical structures that still comply with the performance requirements prescribed in Eurocode 8.

Keywords: 3D steel structures; forced-based design; behaviour factor; moment resisting frames; Eurocode 8

1. Introduction

Current seismic design codes are still mainly focused in strength control and collapse prevention. However, in recent years, significant efforts have been made to develop new design procedures that focus on damage control. Concerning the design of steel structures, these recent displacement-based methods are still under development and hence their incorporation in seismic codes is not expected to occur in a near future. Consequently, improvements and clarifications of the traditional force-based procedures are still needed in order to avoid uneconomical and unpractical solutions. The Improved Force-Based Design (IFBD) Procedure proposed by Villani *et al*. (2009) came out as the result of these needs. This procedure consists of a more rational sequence of the design steps prescribed in Eurocode 8 (CEN 2004) involving a more realistic selection of the behaviour factor to be adopted in the design process. Its development and application was initially made to planar moment-resisting frames, aiming to overcome some

*Corresponding author, Associate Professor, E-mail: rita.bento@tecnico.ulisboa.pt
[a] Assistant Professor, E-mail: miguel.castro@fe.up.pt
[b] Ph.D. Student, E-mail: rita.peres@tecnico.ulisboa.pt

http://www.techno-press.org/?journal=scs&subpage=6 ISSN: 1229-9367 (Print), 1598-6233 (Online)

limitations associated with the current version of Eurocode 8, particularly in what concerns to the treatment of P-Delta effects at the design stage. In the present version of the European seismic code, the inter-storey drift sensitivity coefficient, θ, is directly dependent on the value of the behaviour factor. The IFBD represents therefore an approach that targets a rational selection of the behaviour factor, consistent with the actual ductility demands that are expected to be imposed to the structure. In this way, the designer can avoid adopting an excessive value for the behaviour factor which could then lead to difficulties in keeping the inter-storey drift sensitivity coefficient within the limits prescribed by the code. The proposers of the IFBD method demonstrated that it leads to more optimised and economical structures, which consistently perform according to the performance objectives considered at the design stage.

The aim of the present work is to extend the application of the IFBD procedure to 3D steel structures composed by moment-resisting frames. The effectiveness of the procedure is demonstrated by comparing the design solutions obtained using the IFBD with those obtained with a direct application of Eurocode 8 (EC8) using a recommended value for the behaviour factor. The seismic performance of the structures is assessed through nonlinear static and time-history analyses. An economic comparison of the design solutions is also made.

2. State of the art

The response modification factors were initially proposed in ATC3-06 (1978) and then developed in ATC-19 (1995) and ATC-34 (1995) as a product of overstrength, ductile and redundancy factors. Afterwards, a large number of studies were performed over the years to assess this parameter such as: Kappos (1999) that focused on the evaluation of behaviour factors with due consideration to both their ductility and overstrength; Lee *et al*. (1999) determined the ductility factor considering different hysteretic models; Maheri and Akbari (2003) investigated the behaviour factors of steel–braced reinforced concrete framed dual systems; Karavasilis *et al*. (2007) proposed simplified expressions to estimate the behaviour factor of plane steel moment resisting frames; Costa *et al*. (2010) proposed a probabilistic methodology for the calibration of behaviour factors and, more recently, Ferraioli *et al*. (2014) evaluated the overstrength, redundancy and ductility response modifications factors of steel moment-resisting frames.

The capacity of the structures to deform in the nonlinear range up to a certain level without significant loss of strength has been extensively investigated holding different theories with different ranges of applicability. On the other hand, the acceptance of this concept and inherent studies (largely supported with experimental tests) were also the first indicator of the inadequacy of a force control design procedure. The first proposals that focused on displacement control were made by Paulay and Priestley (1992) and Priestley and Calvi (1991). The design approaches regarding the different structural systems, such as frames, cantilever and coupled walls, dual systems, bridges, timber and masonry structures and seismic isolated structures are presented in Priestley *et al*. (2007). The Direct Displacement Based Seismic Design (DDBD) method proposed by Priestley *et al*. (2007) is likely to serve as the basis for future seismic design codes. However, regarding the design of steel structures, these types of methods are still under development. Recently, Tzimas *et al*. (2013) proposed a hybrid forced/displacement seismic design method for steel building frames and Sullivan (2013) provided details about the application of the DDBD procedure to eccentrically braced steel frames. More recently, Roldán *et al*. (2016) proposed a method for the DDBD of steel moment-resisting frames, with specific consideration of beam-to-column joint characteristics. Notwithstanding all the above, further improvements of forced-based design procedures are still possible and necessary.

3. Description of the seismic design procedures

In this section, an overview of Eurocode 8 (CEN 2004) provisions regarding the design rules of general structures and steel structures is made. A comparison between the EC8 design procedure and the original Improved Forced Based Design (IFBD) procedure is presented and an extension of the IFBD to 3D steel structures is proposed.

3.1 An overview of the Eurocode 8 provisions

The main objectives of Eurocode 8, which are defined in Chapter 2 of the European code, are the protection of human life and the structures damage limitation insuring the operational state of important structures for civil protection. To ensure these objectives, the structures designed according the EC8 (Sections 2.1 and 2.2) should fulfil the no-collapse and damage limitation requirements. The no-collapse requirement ensures that structures are designed to resist the seismic action without local or global collapse. The return period considered depends of the importance of the structure, being higher for structures with higher importance. For typical buildings, the return period is 475 years which corresponds to an intensity with a 10% probability of being exceeded in a period of 50 years. The damage limitation requirement aims at ensuring that structures are designed to withstand a more frequent earthquake event without observing the occurrence of damage and use limitations that are disproportionately high in comparison to the replacement cost. A return period of 95 years is proposed in EC8, which corresponds to an intensity with a 10% of probability of exceedance in 10 years. Two specific limit states result from the performance requirements described above, namely the Ultimate and Serviceability (or Damage Limitation) limit states. The Ultimate Limit State (ULS) conditions, defined in Section 4.4.2 (Chapter 4) of EC8, are considered satisfied if the structural system has sufficient resistance and energy dissipation capacity. Additionally, it should be safeguard that both the foundation material and members are able to resist the action effects resulting from the response of the superstructure without occurrence of permanent deformations. In practical terms, the ULS is considered fulfilled when both resistance and ductility conditions are met. The resistance conditions are satisfied if the design value of the effect of the seismic action is less than the corresponding design resistance, computed in accordance with the specific rules of the structures material. The second order effects should be incorporated in the analysis to satisfy the resistance condition, if necessary. According to Section 4.4.2.2 of EC8, the inter-storey drift sensitivity coefficient value, θ, should be computed at each storey. If this value is lower than 0.1 then second-order effects can be neglected, otherwise, if it is between 0.1 and 0.2, they should be incorporated in the analysis through the application of an amplification factor given by $1/(1 - \theta)$. Although not explicitly stated, for values of inter-storey drift sensitivity coefficients greater than 0.2, a second order analysis must be performed. Instability is assumed to occur for values greater than 0.3.

The expression proposed in Eurocode 8 for the inter-storey drift sensitivity coefficient is given by the following expression

$$\theta = \frac{P_{tot} \cdot d_r}{V_{tot} \cdot h} \tag{1}$$

where P_{tot} and V_{tot} are the total cumulative gravity loading and the seismic shear, respectively, at the storey under consideration, h that is the storey height and d_r that is the design inter-storey drift.

The ductility conditions are set to ensure that both the structural elements and the structure as a whole have adequate ductility. A hierarchy of resistance of the various structural components has to be established in order to ensure an intended configuration of plastic hinges and to avoid a soft storey mechanism. When the selected structural system is a moment-resisting frame (MRF), the ductility conditions should ensure that plastic hinges form in the beams instead of the columns. The required hinge formation can be achieved by application of the following local capacity design criterion (Section 4.4.2.3 of EC8)

$$\sum M_{Rc} \geq 1.3 \sum M_{Rb} \quad (2)$$

where $\sum M_{Rc}$ is the sum of the design values of the moments of resistance of the columns framing the joint and $\sum M_{Rb}$ is the sum of the design values of the moments of resistance of the beams framing the joint.

The Serviceability Limit State (SLS is considered to be satisfied if the inter-storey drifts d_r values are limited in accordance to the values defined by EC8 (Section 4.4.3.2), function of the non-structural building elements:

- Buildings with non-structural elements of brittle materials attached to the structure

$$d_r \cdot v \leq 0.005 \cdot h \quad (3)$$

- Buildings with ductile non-structural elements

$$d_r \cdot v \leq 0.0075 \cdot h \quad (4)$$

- Buildings having non-structural elements fixed in a way so as not to interfere with structural deformations, or without non-structural elements

$$d_r \cdot v \leq 0.01 \cdot h \quad (5)$$

Where h is the storey height and υ is the reduction factor that takes into account the lower return period of the seismic action associated with the damage limitation requirement. The reduction factor values can be found in the country's National Annex, depending on the seismic hazard conditions of the country seismic zones and on the protection of property objective. However, the EC8 (Section 4.4.3.2) recommends reduction factors of 0.4 for importance classes of III and IV and 0.5 for importance classes I and II; this is a simplification that can lead to both high and low inelastic seismic actions for serviceability checks, depending on the site seismic hazard.

The inter-storey drift, d_r, is evaluated as the difference of the lateral displacements, d_s, at the top, $d_{s,top}$, and at the bottom, $d_{s,bot}$, of the storey under consideration

$$d_r = d_{s,top} - d_{s,bot} \quad (6)$$

Regarding the design of steel structures, presented in Chapter 6 of EC8, two design concepts which are associated with the energy dissipation level of the structure: the low dissipative structural behaviour and the dissipative structural behaviour. The main implication of the use of these design concepts is related to the range of force reduction factor (i.e., the behaviour factor q) allowed to be adopted in the analysis. In fact, the structure's capacity to dissipating energy through nonlinear behaviour is actually considered by means of the adoption of behaviour factor, q and displacements amplifications factors. The behaviour factor accounts for the decrease of base shear that the structure will be subjected to if its response deviates from full elastic to elastic-plastic behaviour while the displacement amplification factor accounts for the increase of displacements in a structure subjected to reduced seismic forces in the light of its plastic deformations.

The reference values of behaviour factors, q, established by EC8 (in Table 6.1) for the low dissipative structural behaviour are between 1.5 and 2.0. This principle is associated to a low ductility class (DCL) thus the use of Class 1, 2 or 3 cross-sections is allowed (in EC8, Table 6.3). The dissipative structural behaviour is based on the possibility of controlling inelastic behaviour by means of specific location of dissipative zones whose plastic deformation would ensure a ductile collapse mechanism. The upper limits of reference values of behaviour factors, q, for regular systems in elevation, depend of the type of system and ductility structural class (in EC8, Tables 6.1 and 6.3), which may be medium or high (DCM or DCH, respectively). For non-regular systems in elevation these values have to be reduced by 20%.

3.2 An Improved Forced Based Design (*IFBD*) *procedure*

The seismic design process currently prescribed by EC8 consists of a sequence of design steps that, if not careful analysed, can lead to uneconomical structures whose performance does not reflect the initial design performance intentions. Villani *et al*. (2009) proposed an Improved Force-Based Design (IFBD) procedure for planar structures which consists of a modified sequence of the

Table 1 Comparison of design steps sequence: IFBD versus FBD-Eurocode 8

Improved Forced-Based Design procedure (IFBD)	**Eurocode 8 design procedure (EC8)**
1. Selection of the lateral resisting system and static design for gravity (and wind) loads	1. Selection of the lateral resisting system and static design for gravity (and wind) loads
2. Determination of the seismic elastic forces based on the structure's fundamental period	2. Selection of the behaviour factor, q (from Table 6.2 of Eurocode 8)
3. SLS - inter-storey drift checks and eventual increase of the structural stiffness	3. Determination of the seismic design forces followed by elastic structural analysis
4. Selection of the behaviour factor, q	4. P-Δ checks at ULS and possible amplification of the seismic design base shear and member sizes
5. Determination of the seismic design forces followed by elastic structural analysis	5. ULS strength checks for the final set of seismic forces
6. P-Δ checks at ULS and possible amplification of the seismic design base shear and member sizes	6. Determination of the seismic elastic forces based on the structure's fundamental period
7. ULS strength checks for the final set of seismic forces;	7. SLS - inter-storey drift checks and eventual increase of the structural stiffness

EC8 design steps and, more importantly, of a more accurate evaluation of the behaviour factor, overcoming the gap between the initial performance expectations and the performance actually observed. A comparison of the IFBD and EC8 design steps sequences is presented in Table 1.

The design checks for the two limit states performed in steps 5 and 7 of the EC8 procedure are the critical ones. However, if the lateral resisting system selected is a moment-resisting frame, it is likely that the checks related with *P*-Delta (*P*-Δ) effects may become the govern design criterion. This sometimes may erroneously lead the designer to change his initial option in terms of member sizes or, in some cases, in terms of lateral resisting system.

A close examination of Eqs. (1) and (3), (4) and (5), which represent the ultimate and serviceability limit states checks in terms of inter-storey drift sensitivity coefficient and inter-storey drift limits verifications, respectively, allows concluding that while the choice of the behaviour factor is of fundamental importance for *P*-Δ checks (the design drifts are related with the q-factor through the design base shear applied to the structure), serviceability limit state checks are not affected by this choice as long as the equal displacement rule applies.

Moreover, the direct dependency between the inter-storey drift sensitivity coefficient, θ, and the behaviour factor can be easily established by rewriting the Eq. (1) in the form

$$\theta = \frac{P_{tot} \cdot \frac{d_{el}}{q} \cdot q_d}{\frac{V_{tot,el}}{q} \cdot h} \tag{7}$$

where the $V_{tot,el}$ is the elastic total seismic storey shear, d_{el} the elastic inter-storey drift of the storey and q_d the displacement amplification (relating the maximum inelastic, $d_{\max}$, to elastic, d_{el}, deformations) – Fig. 1. In Eq. (7) the numerator becomes independent of the q factor if the equal displacement rule is valid ($q = q_d$) and the denominator is greatly reduced by it, leading to high sensitivity coefficient values, proportional to the behaviour factor.

For this reason, Villani *et al.* (2009) proposed an expression to estimate the behaviour factor instead of selecting an empirical value from Table 6.2 of EC8. The q-factor evaluation is based in the assumption that the design force (V_d) is equal to the first yield base shear (V_{1y})

$$q = \frac{V_{el}}{V_d} = \frac{V_{el}}{V_y} \cdot \frac{V_y}{V_d} = q_u \cdot \Omega = \frac{V_{el}}{V_{1y}} \tag{8}$$

where V_{el} is the elastic seismic force obtained from the elastic response spectrum; V_d is lateral force considered in the design process; V_y is lateral capacity of the structure; V_{1y} is lateral force reached at the formation of the first plastic hinge in the structure; q_u and Ω are, respectively, the ductility reduction factor and system overstrength factor – Fig. 1.

The ductility reduction factor, q_u, summarizes the energy that the structure is able to dissipate through hysteretic behaviour, provided that deformations beyond yield can be accommodated by the structural members. The system overstrength factor, Ω, accounts for all sources of overstrength, such as the internal force redistribution, strain hardening, differences between the assumed and expected material strength, etc., represents the higher strength provided to the structure with respect to the strength that is needed and it is given by the ratio V_y / V_d (Fig. 1).

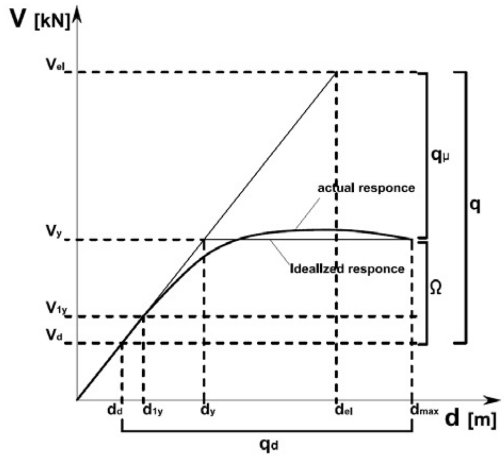


Fig. 1 Ductility and overstrength components of the behaviour factor (Villani *et al*. 2009)

3.3 An extension of IFBD procedure to 3D steel structures

In the following section a detailed description of the IFBD procedure is given and a proposal of its application to 3D steel structures is presented.

Step 1- Selection of the lateral resisting system and static design for gravity loads

First it has to be defined the structure plan configuration which consists in the selection of the type and number of lateral resisting systems that will be assigned to each horizontal direction. After the static design of these systems for the gravity loads, a modal analysis to assess the dynamic characteristics of the structure should to be made.

Step 2 - Determination of the seismic elastic forces

The seismic elastic forces are obtained through a linear dynamic response spectrum analysis. However, if the structure response is not significantly affected by the contribution of torsion and higher modes of vibration, the equivalent lateral force method of analysis can still be applied. In this case, the elastic base shear force in each plan

$$V_{el} = \lambda \cdot m \cdot S_e(T) \tag{9}$$

where λ is the correction factor, m is the mass of the system, T is the fundamental period of vibration of the system and S_e (T) is the ordinate of the elastic acceleration response spectrum. The correction factor λ is 0.85 if $T < 2 \times T_c$ (with T_c being the upper limit of the period of the constant spectral acceleration branch) and the structure has more than two storeys. Otherwise λ is equal to 1.

Step 3 - Serviceability Limit States (SLS): inter-storey drift checks and possible need to increase the lateral stiffness of the structure.

The Serviceability Limit State is considered to be satisfied if the inter-storeys drift values, computed according to Eq. (6), are within the limits defined by Eqs. (3), (4) and (5).

These verifications must be performed in both directions and in the case of irregular structures the inter-storey drifts values should take into account the torsional effects and be evaluated as the difference of the average of the lateral displacements in each direction.

Step 4 - Selection of the behaviour factor, q, to adopt in the design

The behaviour factor should be computed in both directions for each seismic lateral resisting system, s, as follows

$$q_s = \frac{V_{el\_s}}{V_{1y\_s}} \tag{10}$$

where V_{el_s} is the is the elastic seismic base shear and V_{1y_s} is the base shear due to the gravity loads combined to the seismic combination which leads to the formation of the first plastic hinge of the lateral resisting system under evaluation, s.

In practical terms, the calculation of the base shear at the formation of the first plastic hinge, V_{1y}, can be obtained through the sum a set of normalized equivalent lateral forces $\overline{F_i}$, combined with the gravity loads, and multiplied by the load factor (α) that leads to the formation of the first plastic hinge in a structural element – Eq. (11). This load factor is the minimum value from all the loads factors obtained for each structural member

$$V_{1y\_s} = \sum_i \overline{F_i} * \min(\alpha) \tag{11}$$

The normalized equivalent lateral forces $\overline{F_i}$ result from the application of a unit seismic base shear distributed along the height (Fig. 2), through a load pattern preconized by EC8.

The calculation of the load factor depends of the selected type of lateral resisting system. For moment-resisting frames (MRF), where the dissipative members are beams or columns, it is given by

$$\alpha = \frac{M_{pl,Rd\_n} - M(G + \psi_2 \cdot Q_k)}{M(E_{Ek})} \tag{12}$$

Where M_{pl,Rd_n} is the plastic moment of the n^{th} structural element computed according to Eurocode 3 (CEN 2005), M (E_{Ek}) is the moment due to the application of a base shear equal to

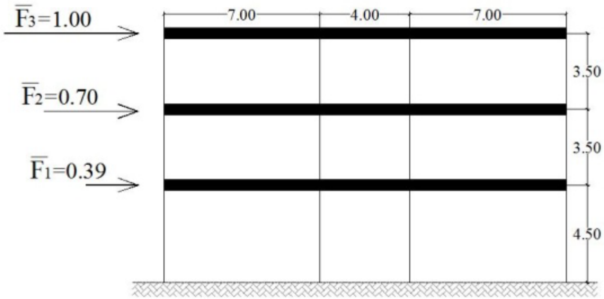


Fig. 2 Set of normalized lateral forces to compute V_{1y} in a MRF with 3 storeys height

unity, $M\,(G + \psi_2 \cdot Q_k)$ is the moment due to the gravity loads and α is the load factor that leads to the occurrence of the plastic hinge in the element.

For concentric braced frames (CBF), where the dissipative elements are the braces, the load factor is given by

$$\alpha = \frac{N_{pl,Rd\_n} - N(G + \psi_2 \cdot Q_k)}{N(E_{Ek})} \tag{13}$$

Where N_{pl,Rd_n} is the plastic axial capacity of the nth brace calculated according to Eurocode 3, $N\,(E_{Ek})$ is the axial load due to the application of a base shear equal to unity, $N\,(G + \psi_2 \cdot Q_k)$ is the axial load due to gravity loads and α is the load factor that leads to axial yielding of the brace.

After computing the behaviour factors of all lateral resisting systems in the direction under consideration, the value adopted has to be equal or higher than the maximum value obtained, q_s - Eq. (14) - otherwise the strength requirements will not be fulfilled

$$q \geq \max q_s \tag{14}$$

In fact, if the behaviour factor is less than the maximum obtained for a specific lateral resisting system, it means that, for that system, the resistance of the element, where the formation of the first plastic hinge is expected to occur, is lower than the design force. Moreover, it is important to ensure that the behaviour factor does not exceed the maximum value recommended in the code.

Step 5 - Determination of the design forces

The seismic design forces are obtained through the performance of linear dynamic response spectrum analysis or an equivalent lateral force method using, for each direction, the values of the behaviour factors obtained previously.

If an equivalent lateral force method analysis is used, the seismic design base shears are given by the Eq. (15).

$$V_d = \lambda \cdot m \cdot S_d(T) \tag{15}$$

where $S_d\,(T)$ is the ordinate of the design response spectrum which is a function of the behaviour factor.

Step 6 - P-Δ checks and possible amplification of the seismic design base shear

The second order effects should be evaluated through the quantification of the inter-storey drift sensitivity coefficient using Eq. (1). The inter-storey drift sensitivity coefficients, θ, have to be evaluated for both plan directions, using the corresponding behaviour factor of the structure obtained for each direction. In the case of irregular structures, the design inter-storey drift should be evaluated as the difference of the average of the lateral displacements. If $\theta < 0.1$ no amplification is needed and the final set of seismic forces corresponds to that obtained in Step 5, otherwise a linear dynamic response spectrum analysis must be performed again considering the amplification factor $1 / (1 - \theta)$ to compute a new set of seismic design forces.

Step 7 - Ultimate Limit State (ULS) checks for the final set of seismic forces

The design and detail of the structural elements have to fulfil all the resistance and ductility conditions established in EC8 for the final set of seismic forces computed in Step 5 or Step 6.

However, regarding the capacity design conditions of the non-dissipative elements, the seismic combination prescribed in EC8 (Eq. 16) should consider the updated expression of Ω, proposed by Elghazouli (2009), which accounts for the gravity loading effects (Eq. (17)).

The load combination that defines the design value of a generic internal action, R_{Ed}, is given by

$$R_{Ed} = R_{Ed,G} + 1.1 \cdot \gamma_{OV} \cdot \Omega \cdot R_{Ed,E} \tag{16}$$

where $R_{Ed,G}$ and $R_{Ed,E}$ are the effect of gravity and earthquake loads, respectively; the γ_{OV} is the ratio between the expected yield stress and nominal yield value for a given steel grade, which can be taken as 1.25, and Ω the overstrength factor defined according the following equation

$$\Omega = \min\left(\frac{R_{pl,Rd} - R_{Ed,G}}{R_{Ed,E}}\right) \tag{17}$$

were $R_{Ed,G}$ and $R_{Ed,E}$ are, respectively, the effect of gravity and earthquake loads and $R_{pl,Rd}$ is the corresponding plastic resistance.

Note that, if the behaviour factor is well estimated, the value of the overstrength factor should be close to unity.

4. Case studies

4.1 Geometry

The case studies analysed in this work consist of two groups of steel structures; the first group was designed using the IFBD procedure while the second group was designed according to the EC8 provisions by adopting a fixed value for the behaviour factor. The structural systems selected to resist the lateral seismic forces are moment-resisting frames (MRF). The reason for this choice is related with the design governing criteria that, for this structural system type located in low to moderate seismicity regions, are typically the checks related with the control of P-Δ effects, which in turn, according to EC8, depend directly of the value adopted for the behaviour factor.

The structures are three storeys height with a plan configuration defined in order to target both torsionally restrained and unrestrained systems, as well as plan regular and irregular structures. The storey height is 4.5 m in the first storey and 3.5 m in the upper stories. The plan dimensions are 30×18 m^2 with a central opening of 6×4 m^2 for core stairs and elevator. Table 2 summarises all the case studies considered.

Table 2 Case studies

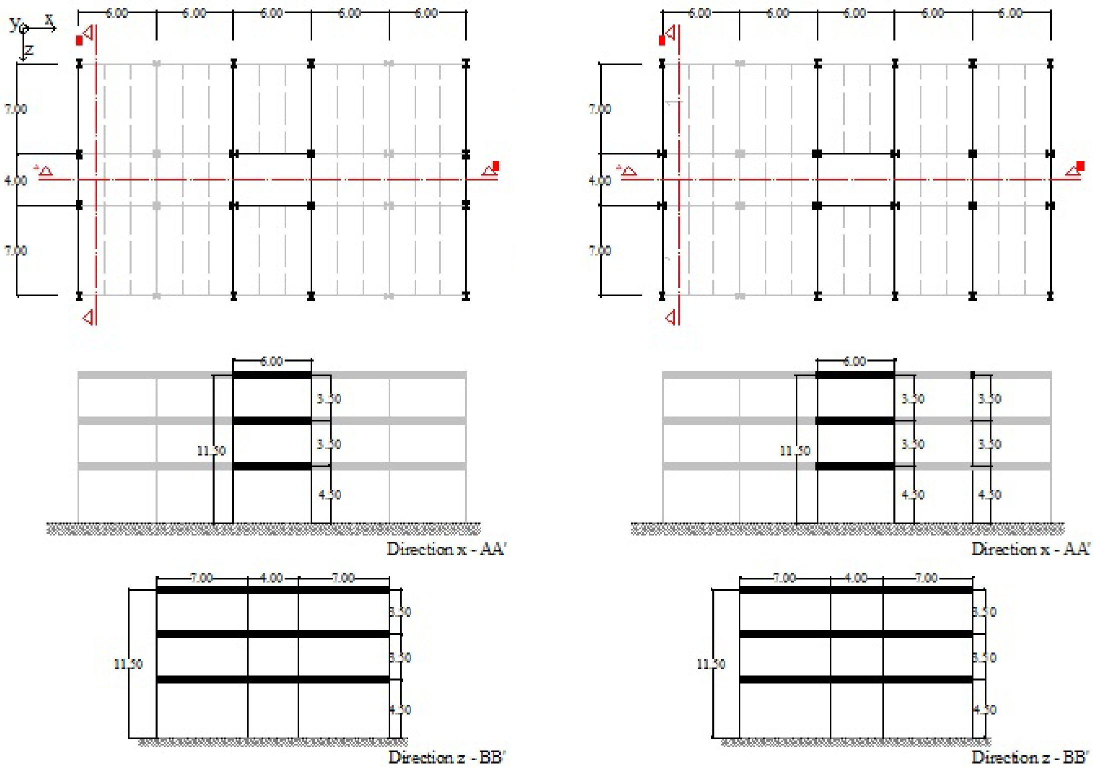
Structures	1st Group designation	2nd Group designation
Structure torsionally unrestrained – plan regular	Case 1- IFBD	Case 1- EC8
Structure torsionally unrestrained – plan irregular	Case 2- IFBD	Case 2- EC8
Structure torsionally restrained – plan regular	Case 3- IFBD	Case 3- EC8
Structure torsionally restrained – plan irregular	Case 4- IFBD	Case 4- EC8

The plan configuration of the torsionally restrained and unrestrained structures depends on the location of the lateral resisting systems in the outside or inside perimeter of the structures, respectively. The regularity in plan is obtained through the location of the lateral resisting systems in such a way that the structure's centres of mass and stiffness are coincident. The irregularity is achieved by placing an additional moment-resisting frame that in one of the horizontal plan direction in order to shift the structure's centre of stiffness so that it does not coincide with the centre of mass in that direction. The structures final plan configurations and elevation views are illustrated in Fig. 3.

4.2 Structural design

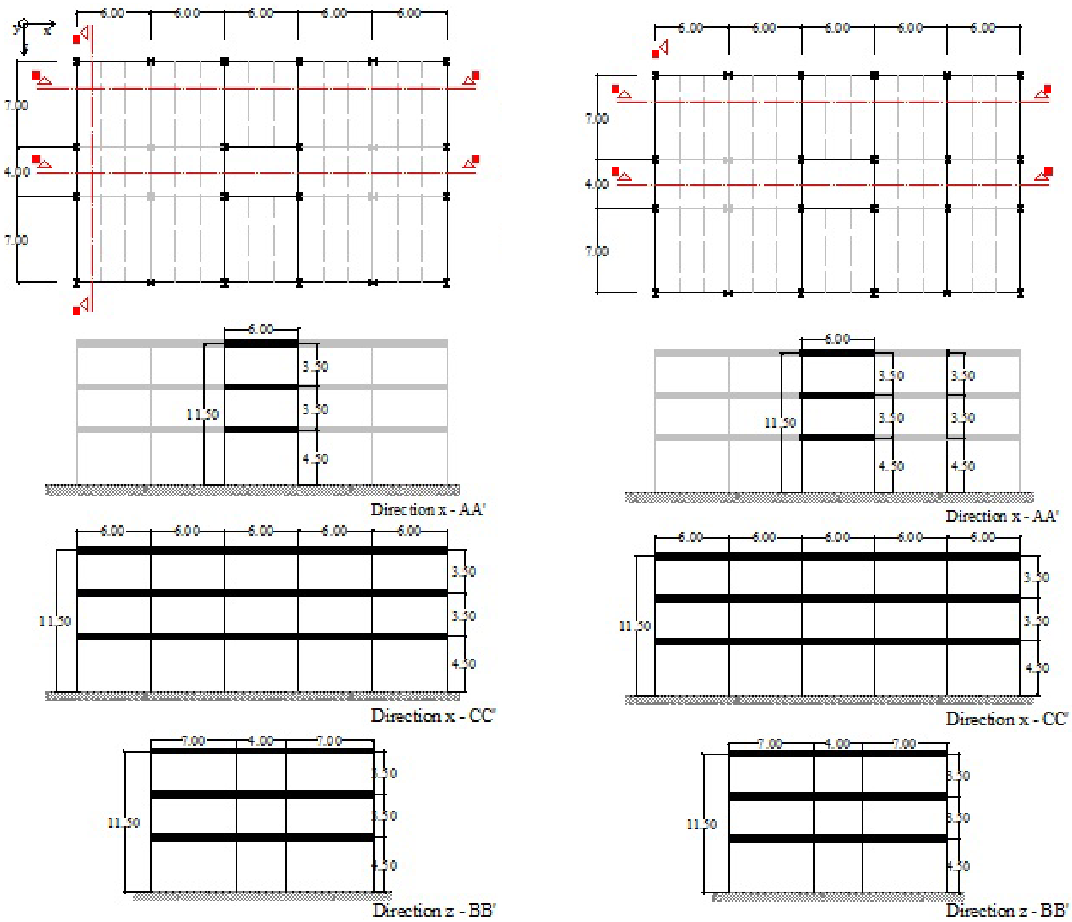
The structures were initially designed only for gravity loads according to Eurocode 3 (CEN, 2005). These loads comprised the self-weight of the structure with an allowance of 1 kN/m^2 for finishings and partitions and an imposed load of 2 kN/m^2. European IPE sections were used for the beams and HEB sections were adopted for the columns. The steel grade considered was S275.

The seismic action was evaluated according to the EC8 provisions. It was assumed a Type 1 response spectrum and soil type B for an intensity level of 0.3g. The design response spectrum, S_d (T), was obtained from the elastic response spectrum by dividing it by the behaviour factor which was computed as proposed by the IFBD procedure for the first group of structures and selected



(a) Regular structure torsionally unrestrained (b) Irregular structure torsionally unrestrained

Fig. 3 Structures plan configuration and correspondent elevation views (dimensions in [m])



(c) Regular structure torsionally restrained (d) Irregular structure torsionally restrained

Fig. 3 Continued

according to the recommendations of EC8 for the second group of structures. The selection of the behaviour factor was made considering a medium ductility class (DCM), which corresponds to a value equal to 4.

The irregular structures were designed ignoring the serviceability limits checks; otherwise the inherent eccentricities would be lost and the structures would become classified as regular in plan. For the same reason the accidental eccentricities were not considered in the design process.

A summary of the main parameters considered in both design procedures, namely the periods of vibration, T, the behaviour factors, q, the inter-storey drift sensitivity coefficients, θ, the serviceability interstorey drifts (SLS) and the overstrength factors, Ω, is provided in Tables 3, 4, 5 and 6 for Cases 1, 2, 3 and 4, respectively. The comparison of these parameters allows concluding that the structures designed with the IFBD procedure are always more flexible than the structures designed with the EC8 procedure with a fixed value for the behaviour factor. This is due to the fact that the behaviour factors considered in the application of the IFBD procedure were generally lower and, consequently, the inter-storey drift sensitivity coefficients are also lower. The reduced

Table 3 Design summary using IBFD and EC8 procedures - Case 1

		IFBD					EC8				
CASE 1	Storey	T(s)	q	$\theta_{\_IFBD}$	$SLS_{\_IFBD}$	Ω	T(s)	q	$\theta_{\_EC8}$	$SLS_{\_EC8}$	Ω
	3			0.02	0.031				0.02	0.030	
Direction x	2	1.08	3.00	0.03	0.030	1.51	0.97	4.00	0.03	0.027	2.13
	1			0.03	0.023				0.04	0.020	
	3			0.03	0.027				0.04	0.003	
Direction z	2	1.18	2.70	0.08	0.040	1.38	0.92	4.00	0.08	0.033	2.64
	1			0.09	0.045				0.07	0.029	

Table 4 Design summary using IBFD and EC8 procedures - Case 2

		IFBD					EC8				
CASE 1	Storey	T(s)	q	$\theta_{\_IFBD}$	$SLS_{\_IFBD}$	Ω	T(s)	q	$\theta_{\_EC8}$	$SLS_{\_EC8}$	Ω
	3			0.02	0.030				0.04	0.030	
Direction x	2	1.03	2.90	0.03	0.029	1.53	0.92	4.00	0.06	0.028	2.23
	1			0.03	0.021				0.06	0.020	
	3			0.03	0.027				0.04	0.030	
Direction z	2	1.07	2.70	0.08	0.040	1.38	0.84	4.00	0.08	0.033	2.65
	1			0.09	0.045				0.07	0.029	

Table 5 Design summary using IBFD and EC8 procedures - Case 3

		IFBD					EC8				
CASE 1	Storey	T(s)	q	$\theta_{\_IFBD}$	$SLS_{\_IFBD}$	Ω	T(s)	q	$\theta_{\_EC8}$	$SLS_{\_EC8}$	Ω
	3			0.02	0.022				0.03	0.020	
Direction x	2	0.99	2.70	0.05	0.028	1.41	0.88	4.00	0.06	0.025	2.58
	1			0.05	0.029				0.07	0.026	
	3			0.03	0.023				0.04	0.025	
Direction z	2	1.05	2.50	0.06	0.032	1.38	0.81	4.00	0.08	0.027	2.24
	1			0.07	0.034				0.08	0.021	

Table 6 Design summary using IBFD and EC8 procedures - Case 4

		IFBD					EC8				
CASE 1	Storey	T(s)	q	$\theta_{\_IFBD}$	$SLS_{\_IFBD}$	Ω	T(s)	q	$\theta_{\_EC8}$	$SLS_{\_EC8}$	Ω
	3			0.02	0.021				0.03	0.019	
Direction x	2	0.99	2.30	0.05	0.027	1.38	0.87	4.00	0.07	0.024	2.70
	1			0.05	0.027				0.07	0.024	
	3			0.02	0.023				0.03	0.024	
Direction z	2	0.95	2.60	0.06	0.031	1.38	0.73	4.00	0.05	0.026	2.32
	1			0.07	0.034				0.05	0.021	

values of the overstrength observed for the structures designed with the IFBD procedure results from the fact that the design lateral force is equal to the lateral force reached at the formation of the first plastic hinge in the structure. The higher stiffness associated with the structures fully designed to EC8 is reflected in the lower values of the serviceability inter-storey drifts.

The final member sizes obtained using IBFD and EC8 procedures are presented in Table 7 (Cases 1 and 2) and in Table 8 (Cases 3 and 4).

Table 7 Final member sizes– Case 1 & Case 2

CASE 1	CASE 2	IFBD	EC8
		Sections (storeys 3, 2 & 1)	Sections (storeys 3, 2 & 1)
BEAMS (Direction BB')	BEAMS (Direction BB')	IPE300/IPE330 /IPE330	IPE 300/IPE360 /IPE360
INTERNALCOLUMNS (direction BB')	INTERNALCOLUMNS (direction BB')	HEB550	HEB500/HEB650 /HEB650
EXTERNAL COLUMNS (Direction BB'-frames 1 &4)	EXTERNALCOLUMS (Direction BB'-frames 1 &5)	HEB300	HEB400/HEB500 /HEB500
EXTERNAL COLUMNS (Direction BB'-frames 2&3)	EXTERNAL COLUMNS (Direction BB'-frames 2,3 &4)	HEB 340	HEB 400/HEB500 /HEB550
BEAMS (Direction AA')	BEAMS (Direction AA')	IPE 330/IPE500 /IPE500	IPE330/IPE500 /IPE500

Table 8 Final member sizes– Case 3 & Case 4

CASE 3	CASE 4	IFBD	EC8
		Sections (storeys 3, 2 & 1)	Sections (storeys 3, 2 & 1)
BEAMS (Direction BB')	BEAMS (Direction BB')	IPE300/IPE330 /IPE330	IPE 300/IPE330 / IPE330
INTERNALCOLUMNS (direction BB' -frames 1 &4)	INTERNALCOLUMNS (direction BB'-frames 1, 4 &5)	HEB340 HEB400	HEB400/HEB650 /HEB650
INTERNAL COLUMNS (Direction BB'-frames 2 & 3)	INTERNAL COLUMS (Direction BB'-frames 2 &3)	HEB400	HEB650
EXTERNAL COLUMNS (Direction BB'- frames 2 & 3)	EXTERNAL COLUMNS (Direction BB'- frames 2 & 3)	HEB300	HEB400
BEAMS (Direction AA'-frames 2 & 3)	BEAMS (Direction AA'-frames 2 & 3)	IPE 330/IPE360 /IPE360	IPE 330/IPE500 /IPE500
BEAMS (Direction AA'-frames 1&4)	BEAMS (Direction AA'-frames 1&4)	IPE300/IPE360/IPE360 IPE300/IPE330/IPE330	IPE300/IPE360 /IPE360
EXTERNAL COLUMNS (Direction AA'-frames 1&4)	EXTERNAL COLUMNS (Direction AA'-frames 1 &4)	HEB400	HEB400

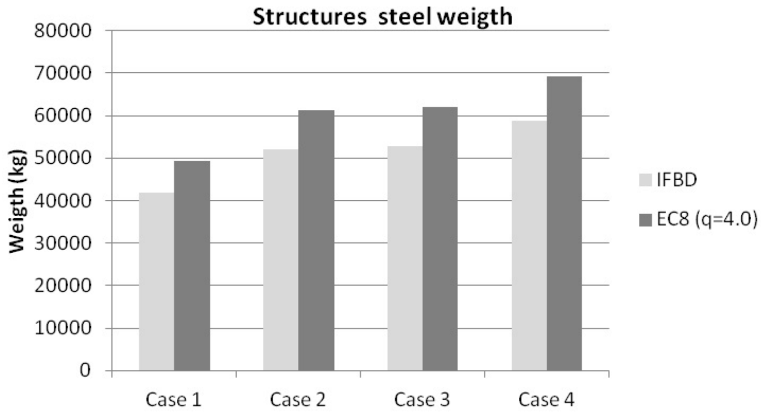


Fig. 4 Structures steel weight

Comparing the final set of design solutions obtained by both procedures, it can be observed that the EC8 procedure always leads to an increase of the section member sizes. The structures steel weights evaluation (Fig. 4) shows that the IFBD procedure leads to lighter and, consequently, more economical design solutions. The steel weight reduction that results from IFBD procedure application is, in general, about 15%.

5. Numerical modelling, seismic action and analysis procedures

The seismic assessment of the structures was carried out by means of nonlinear static (pushover) and time-history analyses. The structures were modelled in the finite element analysis package OpenSEES (PEER 2006). The material nonlinear behaviour was considered through a fibre modelling approach. Force-based elements were employed to represent beams and columns, adopting one element per member with ten integration points. Regarding the material model, a simplified bilinear stress-strain constitutive rule was assumed, considering a value of 1% for the strain hardening. Geometrical nonlinearities were also considered in the analyses. The strength and stiffness deterioration of the beams was not modelled because the degradation process resulting from the occurrence of local buckling is not expected to occur for the levels of plastic deformations experienced by the structural elements (Tenchini *et al*. 2014).

The seismic input for the nonlinear time-history analysis consisted of fifteen records (Table 9) obtained from real earthquake events. The selection of the records was conducted with the SelEQ tool (Araújo *et al*. 2016). The record set consists of a combination of 15 records, considering the two horizontal components, obtained from the PEER ground motion database and compatible with the EC8 spectrum (Type 1; Soil type B, PGA = 0.3 g).

The viscous damping was modelled using the Rayleigh damping formulation, considering proportionality to the tangent stiffness. The damping coefficient considered was 2.5%.

The performance evaluation was assessed through time-history analyses conducted with the 15 records mentioned above. The two horizontal record components were considered acting simultaneously first on X and Z directions and after on Z and X directions resulting in a final set of 30 analyses.

Table 9 Ground Motion records considered

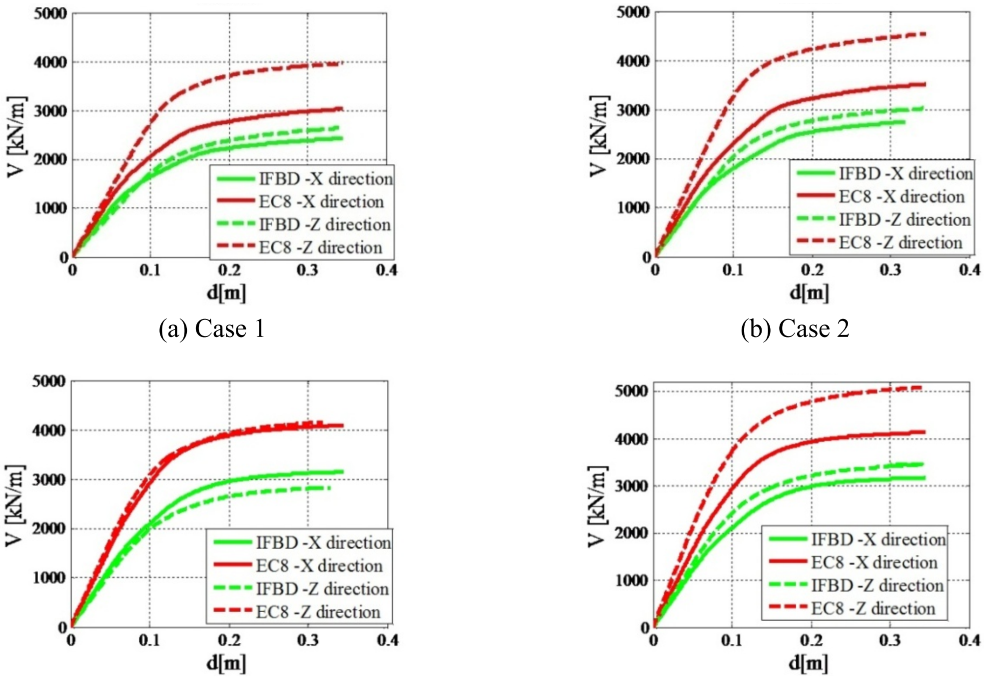
Earthquake name	Earthquake ID	Station name	Scaling factor
Tabas, Iran	0046	Dayhook	1.28
Imperial Valley-06	0050	Superstition Mtn Camera	6.17
Victoria, mexico	0064	SAHOP Casa Flores	5.79
Irpinia, Italy-01	0068	Torre Del Greco	7.86
Coalinga-01	0076	Parkfield - Stone Corral 3E	4.33
N. Palm Springs	0101	Anza Fire Station	7.00
Chalfant Valley-02	0103	Tinemaha Res. Free Field	9.00
Whittier Narrows-01	0113	La Habra - Briarcliff	8.00
Loma Prieta	0118	Woodside	4.00
Northridge-01	0127	LA - N Westmoreland	2.83
Kocaeli, Turkey	0136	Mecidiyekoy	8.58
Chi-Chi, Taiwain	0137	HWA038	7.69
Chi-Chi, Taiwain-03	0172	TCU053	10.0
Chi-Chi, Taiwain-05	0174	CHY087	5.63
Chi-Chi, Taiwain-06	0175	TCU068	6.00

6. Discussion of the results

The results of the analyses conducted on the two groups of structures are now presented in both plan horizontal directions: X direction (direction AA' shown in Figure 3) and Z direction (direction BB' shown in Figure 3). The capacity curves are presented first for each analysed structure. After that the results obtained with nonlinear dynamic time-history analyses are presented and discussed.

6.1 Pushover analysis – Capacity curves

The capacity curves obtained for all structures in the *X* and *Z* directions are presented in Fig. 5. The analysis of the curves allows concluding that the structures designed using the IFBD procedure have similar resistance in both directions. Conversely, the structures designed following the EC8 procedure exhibit a significant increase in resistance on the *Z* direction, with the exception of the Case 3 structure. This is due to the fact that this structure has four frames distributed regularly along each plan direction and the increase in the beams and columns sections that results from the application of the EC8 design procedure was made uniformly in both directions. For the structures of Case 1 and Case 2, the increase of resistance in the *Z* direction is more significant than in the X direction because only the columns sections were increased and the contribution of these elements to the lateral resistance in the *Z* direction is higher. Regarding the Case 4 structure, the increase of resistance in the *Z* direction is mainly due to the increase in the internal columns sections of the additional frame (frame 4) introduced to turn the structure irregular in plan in that direction. Finally, it is worth noting that it is clear from all the capacity curves that the structures design according to the EC8 procedure are stiffer than the ones obtained by means of the IFBD procedure.



(a) Case 1 (b) Case 2

(c) Case 3 (d) Case 4

Fig. 5 Capacity curves for the *X* and *Z* directions

In order to understand the inelastic behaviour of the structures designed according to the different approaches (IFBD procedure and EC8 provisions), the plastic hinge patterns developed were assessed for a global drift of 3% for all structures. This drift value is often considered in seismic performance assessment studies (Elghazouli *et al*. 2008). Figs. 6 and 7 illustrate the plastic hinge patterns obtained for Cases 1 and 3 considering the effects of both directions pushover analysis, in *X* and *Z* directions. In the figures, the red circles represent the occurrence of plastic hinges at the structural element ends when the IFBD or the EC8 procedures are applied whereas the blue circles represent the additional plastic hinges that take place when using only the EC8 procedure.

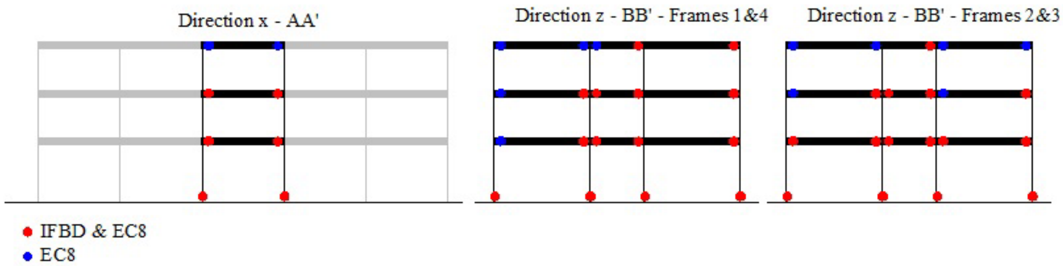


Fig. 6 Plastic hinge patterns obtained for Case 1 (IFBD and EC8) for a global drift of 3%

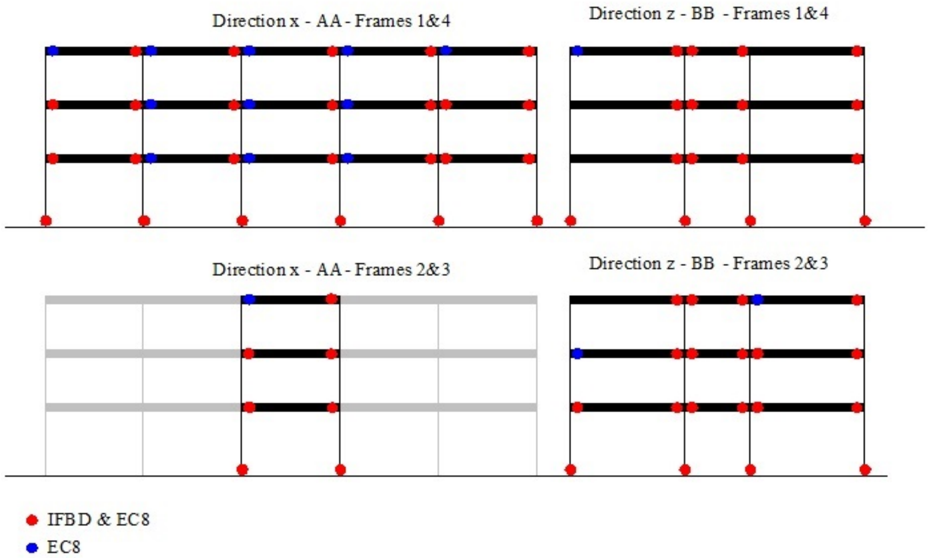


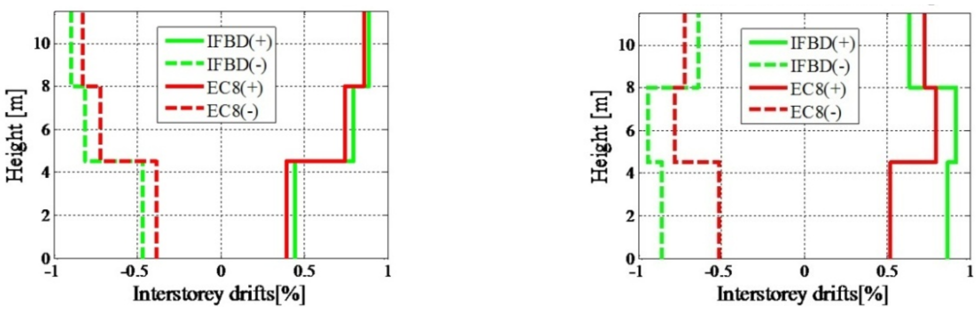
Fig. 7 Plastic hinge patterns obtained for Case 3 (IFBD and EC8) for a global drift of 3%

Despite the increase of resistance observed in the pushover curves when the structures are designed with the EC8 procedure, it is observed that the structures inelastic behavior for the level of lateral deformation considered is similar for both methods. As expected, all structures experience plastic deformations at the base of the columns and at the beams ends, according to the capacity design procedure adopted in both methods. Regarding Cases 2 and 4, although these are irregular structures, the results obtained for the same level of lateral deformation are identical to those obtained for Cases 1 and 3, being that the reason why the plastic distributions obtained for Cases 2 and 4 are not presented. The plastic hinge patterns for Cases 2 and 4 involve plastic deformations in the column sections located at the base of the structures, while the beams develop plastic hinges at all sections ends. The difference in the behaviour of the irregular structures in comparison with the regular ones is in the number of beams that develop plastic hinging at both ends. In the case of the irregular structures, and for both procedures, all the beams develop plastic hinges at both ends.

6.2 Nonlinear time-history analysis

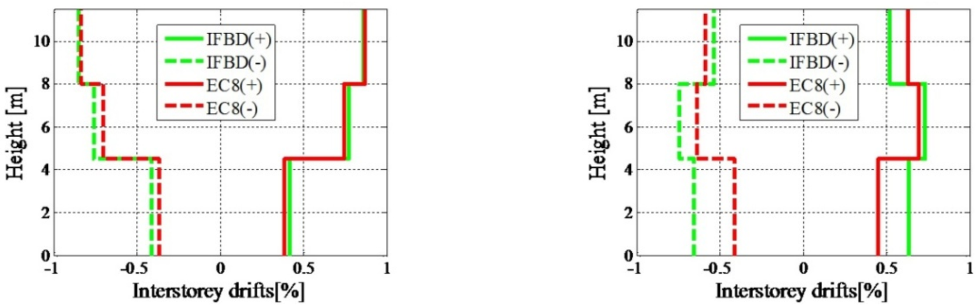
The global performance of the structures was assessed through the examination of the maximum inter-storey drifts at the centre of mass (CM) in the two horizontal directions (*X* and *Z*). Regarding the irregular structures, the maximum torsional displacements at the roof level and the inter-storey drifts at the edge frames were also inspected in the direction of the irregularity (*Z* direction). The maximum torsional displacements are obtained by the average of the edge frame displacements with respect to the displacement of the centre of mass in each direction.

The levels of seismic intensity considered were those corresponding to the serviceability (SLS) and ultimate limit states (ULS), i.e., PGA equal to 0.15 g and 0.30 g, respectively. The analyses for the serviceability level were performed applying a factor of 0.5 to the records, consistently with the EC8 approach.



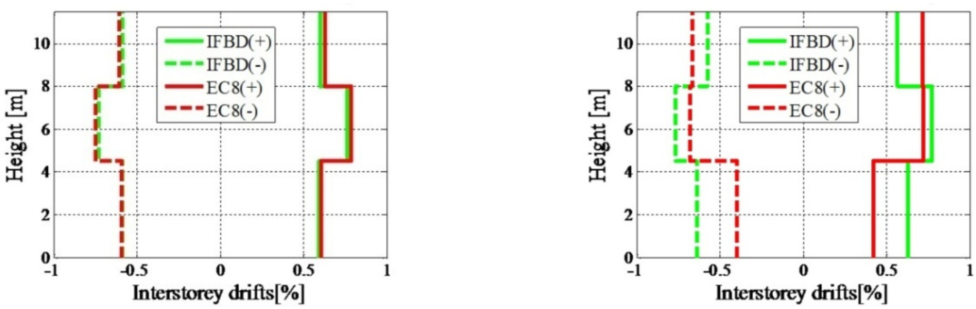
(a) *X* direction (b) *Z* direction

Fig. 8 Inter-storey drifts distribution at CM for the SLS intensity – Case 1



(a) *X* direction (b) *Z* direction

Fig. 9 Inter-storey drifts distribution at CM for the SLS intensity – Case 2



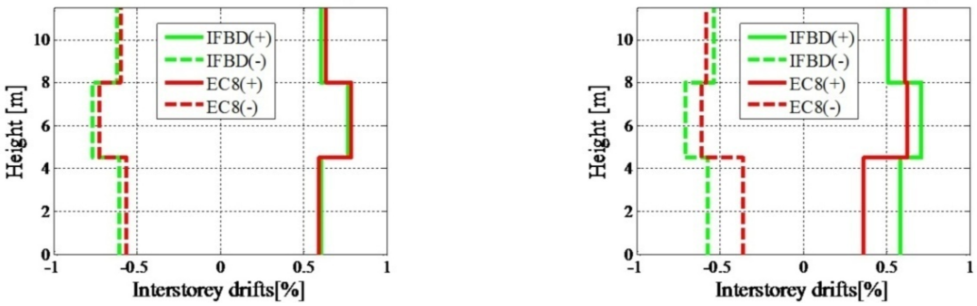
(a) *X* direction (b) *Z* direction

Fig. 10 Inter-storey drifts distribution at CM for the SLS intensity – Case 3

6.2.1 Serviceability Limit State (SLS)

Figs. 8 to 11 show the distributions of maximum inter-storeys drifts obtained at the centre of mass, in the *X* and Z directions, for the intensity level corresponding to the Serviceability Limit State.

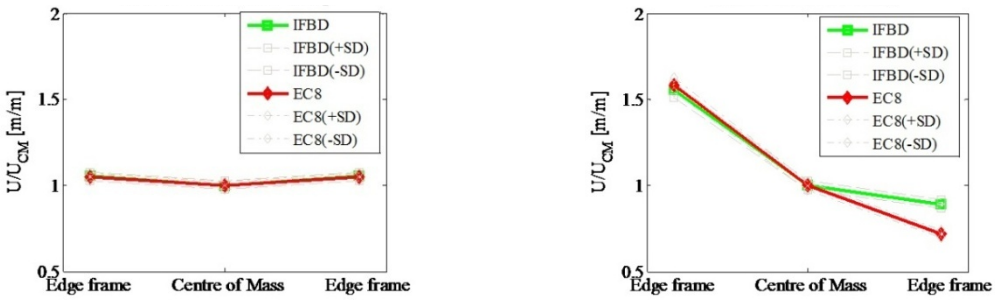
The analysis of the results allows concluding that the inter-storey drifts obtained in the X direction are very similar for all the cases considered, indicating therefore that there is no significant difference between the structures designed with the EC8 and the IFBD procedures.



(a) *X* direction (b) *Z* direction

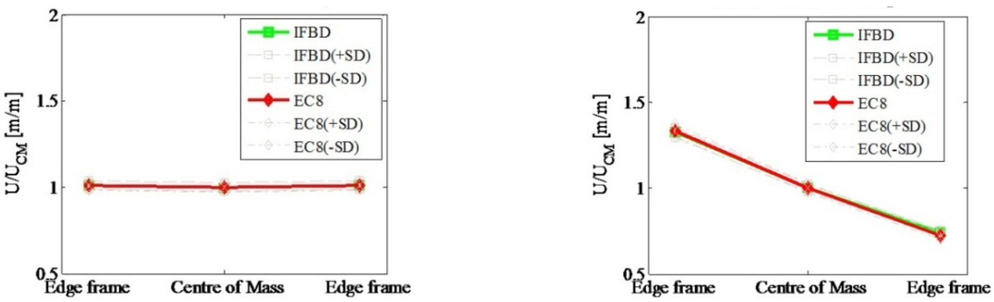
Fig. 11 Inter-storey drifts distribution at CM for the SLS intensity – Case 4

The increase in the columns and beams sections sizes that was required in the EC8 designs in order to keep the inter-storey sensitivity coefficients within the code limits did not introduce any improvement in terms of the seismic performance in the X direction. However, in the Z direction, the columns and sections sizes enhance the seismic performance by reducing the inter-storey drifts at the first two storey levels. Nevertheless, the inter-storey drifts obtained in the two horizontal directions using the IFBD procedure are considerably below the limit of 1% considered at the design stage.



(a) *X* direction (b) *Z* direction

Fig. 12 Torsional displacements for the SLS intensity – Case 2



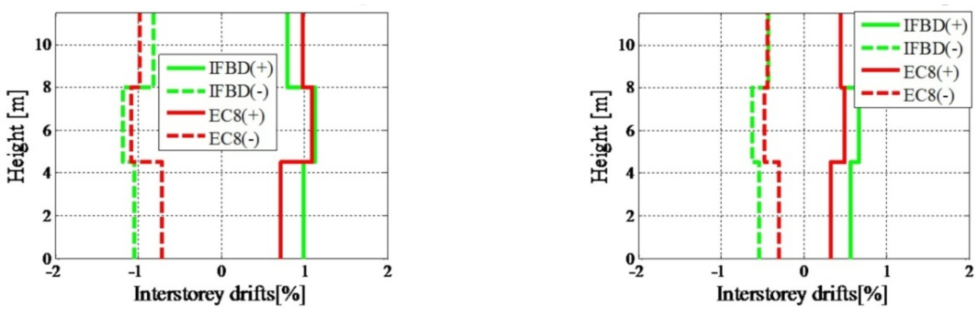
(a) *X* direction (b) *Z* direction

Fig. 13 Torsional displacements for the SLS intensity – Case 4

As expected, torsional deformations were observed for Case 2 (Fig. 12) and Case 4 (Fig. 13). The structures designed according to both IFBD and EC8 procedures exhibited similar performance. The examination of the results obtained for Case 2 and Case 4 indicates that the unrestrained irregular structure (Case 2) is subjected to higher torsional deformations than the restrained irregular structure (Case 4). This is due to the presence of the additional perimeter frames in the *X* direction in the Case 4 structure, which clearly contribute to reduce the lateral displacements of the edge frames aligned in the *Z* direction.

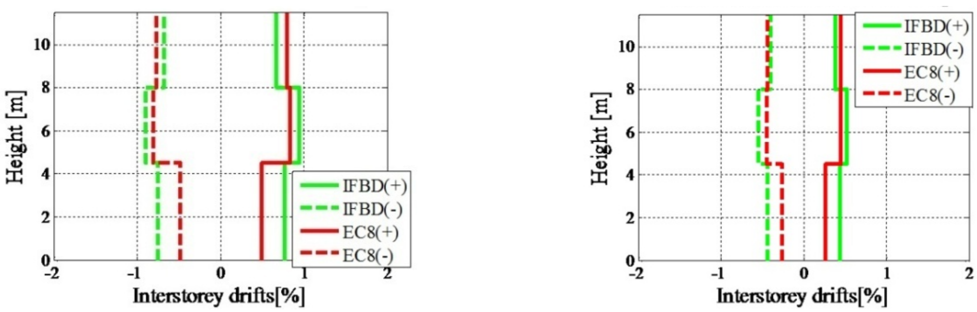
The maximum inter-storeys drifts in the *Z* direction at (a) the left; and (b) right edge frames of Case 2 and Case 4 structures, are plotted in Figs. 14 and 15, respectively.

The results depicted in Figs. 14 and 15 show that the increase in the columns and beams sizes at the first two storeys, which results from the application of the EC8 design procedure with a fixed value of the behaviour factor, leads to a reduction of inter-storey drifts in the same two storeys, consistently with the inter-storeys drifts distribution obtained at the centre of mass of these structures in the *Z* direction. However, for the Case 2 structures, the left edge frame experiences inter-storey drifts higher than the 1%, especially at the second storey level, and independently of the design procedure applied. This is due to the fact that the serviceability limits were not satisfied at these frames during the IFBD design process (see Table 4), in order to keep the structures irregular, otherwise the inherent eccentricities could be lost and the structures would turn out to be regular in plan (the inter-storey displacement observed at the second storey level is 0.40 m when



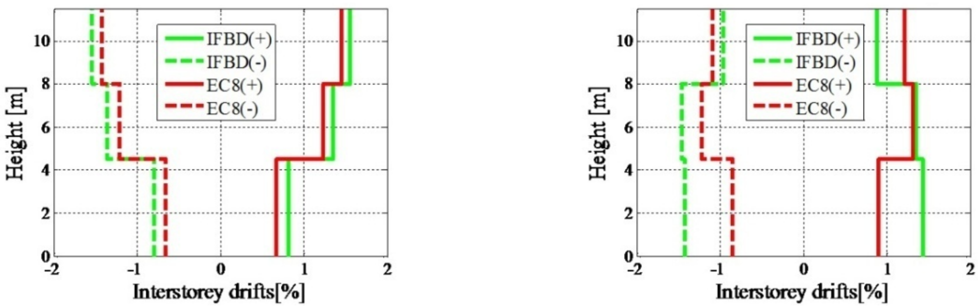
(a) Left edge frame (b) Right edge frame

Fig. 14 Inter-storey drifts distributions in *Z* direction for the SLS intensity – Case 2



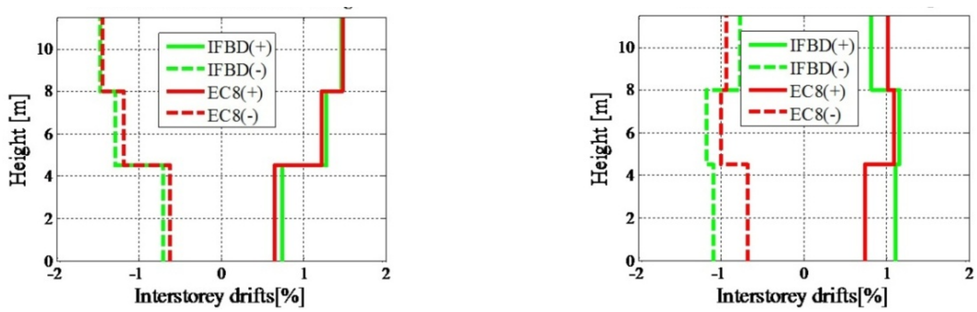
(a) Left edge frame (b) Right edge frame

Fig. 15 Inter-storey drifts distributions in *Z* direction for the SLS intensity – Case 4 the limit allowed is 0.35 m). Regarding the EC8 approach, the inter-storey drifts limits were fulfilled at the design stage (the inter-storey displacement observed at the second storey level is 0.33 m). However, the seismic performance assessment shows that these limits are also being exceeded at the second storey level. Thus, these results also indicate that the application of the IFBD procedure provides more realistic information about the structure seismic performance during the design process.



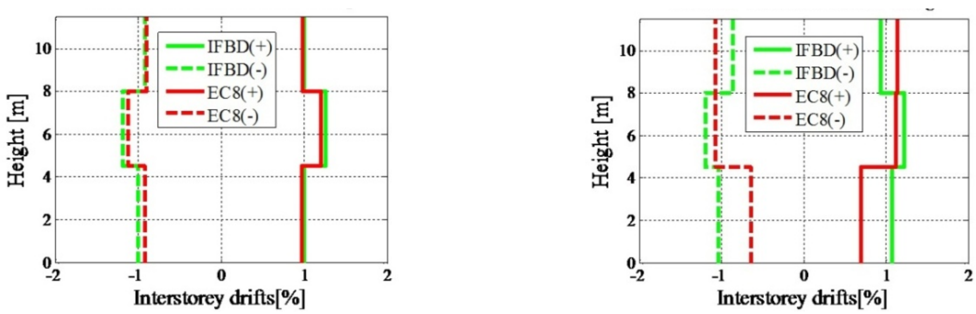
(a) *X* direction (b) *Z* direction

Fig. 16 Inter-storey drifts distribution at CM for the ULS intensity – Case 1



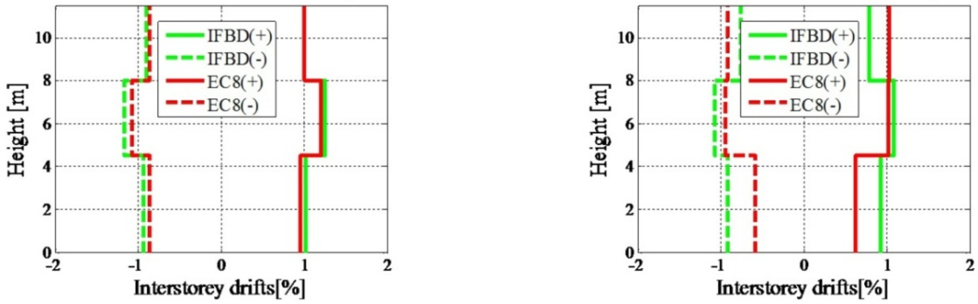
(a) *X* direction (b) *Z* direction

Fig. 17 Inter-storey drifts distribution at CM for the ULS intensity – Case 2



(a) *X* direction (b) *Z* direction

Fig. 18 Inter-storey drifts distribution at CM for the ULS intensity – Case 3



(a) *X* direction (b) *Z* direction

Fig. 19 Inter-storey drifts distribution at CM for the ULS intensity – Case 4

6.2.2 Ultimate Limit State (ULS)

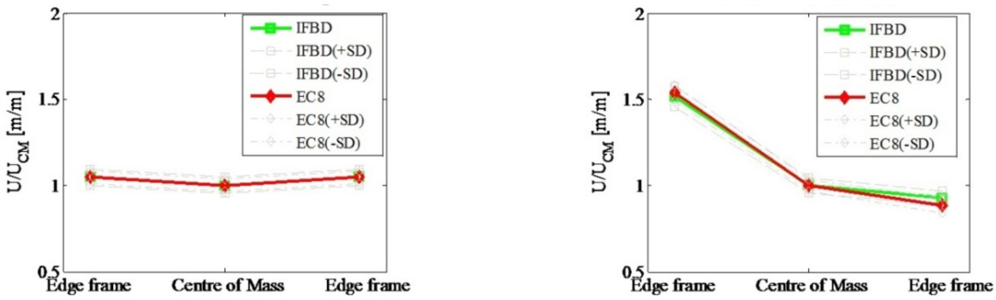
Figs. 16 to 19 show he maximum inter-storeys drifts distributions obtained at the centre of mass (CM), in the *X* and *Z* directions, for the seismic intensity level corresponding to the Ultimate Limit State.

It is observed that, in both directions, and for all cases, the maximum inter-storey drifts at the centre of mass, are well below 2%, a limit value that is often considered in seismic design and assessment codes for limit states associated with significant damage. As mentioned before, the response obtained in the *X* direction is similar using both the IFBD and the EC8 procedures. On the other hand, in the *Z* direction, differences are observed at the first two storey levels, being the EC8 procedure more conservative than the IFBD procedure. It is also observed, in both directions, that inter-storeys drift limits are more uniform when the IFBD procedure is applied.

Regarding torsional deformations, the results obtained for the Case 2 and Case 4 structures are shown in Figs. 20 and 21, respectively.

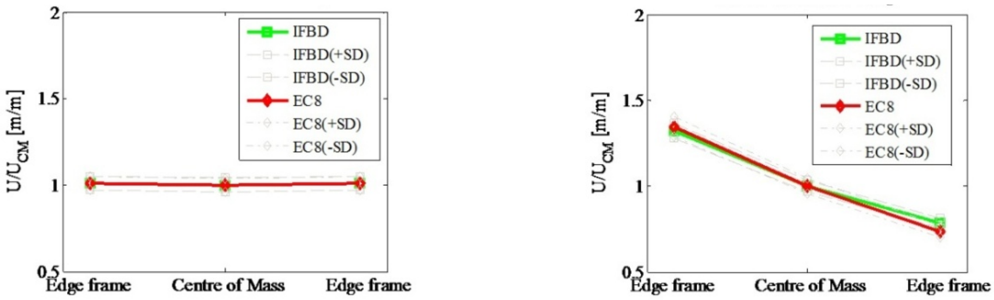
The results are similar to those obtained for the seismic intensity level corresponding to the serviceability limit state. The torsional deformations obtained for the design intensity level for the structures designed with the IFBD and EC8 procedures are practically coincident.

The inter-storey drifts distribution obtained in the *Z* direction at (a) the left; and (b) right edges frames of Case 2 and Case 4 structures, for the design seismic intensity level, are illustrated in Figs. 22 and 23, respectively.



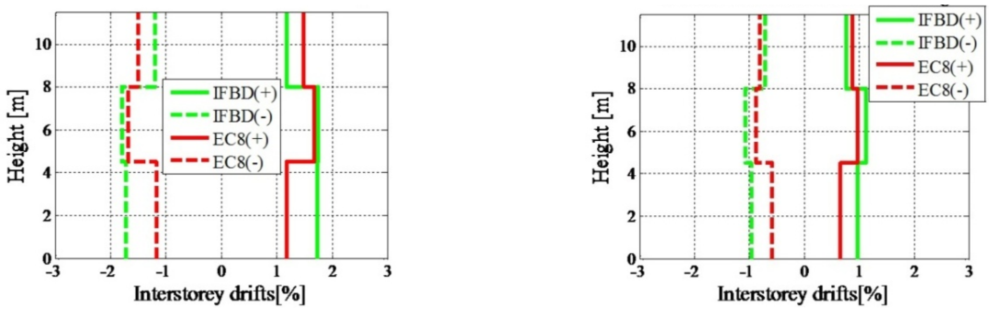
(a) *X* direction (b) *Z* direction

Fig. 20 Torsional displacements for the ULS intensity – Case 2



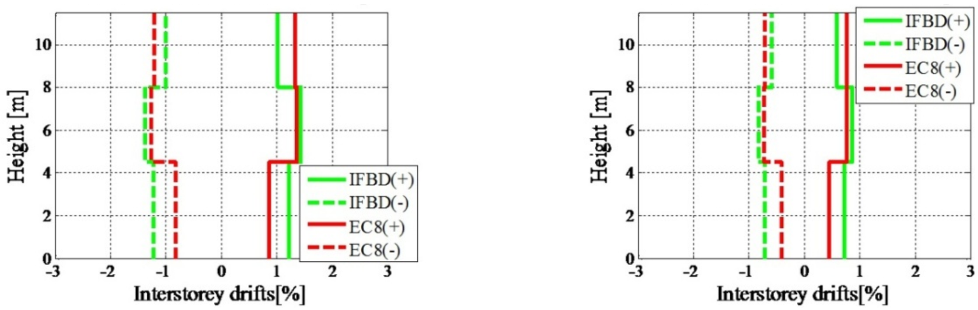
(a) *X* direction (b) *Z* direction

Fig. 21 Torsional displacements for the ULS intensity – Case 4



(a) Left edge frame (b) Right edge frame

Fig. 22 Inter-storey drifts distributions in Z direction for the ULS intensity – Case 2



(a) Left edge frame (b) Right edge frame

Fig. 23 Inter-storey drifts distributions in Z direction for the ULS intensity – Case 4

As previously observed, the Case 2 structure is subjected to higher torsional deformations than the Case 4 structure. The inter-storey drifts distribution clearly show that the Case 2 structure performs well and within the limit of 2%, even in left edge frame, where the highest lateral displacement is expected. For both cases the inter-storey drift distributions are more uniform when the structures are designed using the IFBD procedure. In summary, both structures perform satisfactory, independently of the chosen design procedure.

7. Conclusions

In this paper, a proposal for an extension of the Improved Forced Based Design (IFDB) procedure to 3D structures was made. It consists of a modified version of the current forced based design process implement in Eurocode 8, involving a more rational sequence of the design checks and a more realistic evaluation of the behaviour factor. The proposed procedure was applied to a group of four steel structures, with three storeys height and the same plan configuration in all storeys. The plan configuration was defined in order to obtain lateral restrained and unrestrained and plan regular and irregular structures. The same group of structures was also designed according to Eurocode 8 using a fixed value for the behaviour factor. The member sizes obtained through the two approaches were compared and the seismic performance was assessed through nonlinear static and time-history analyses, for the two limit states considered at the design stage.

The results reveal that the IFBD design solutions are more economical and that the seismic performance was satisfactory for the two limits states considered. Additionally, the structures designed using the IFBD procedure exhibited similar resistance in both horizontal directions. It was also observed that the inter-storey drift distributions are more uniform when the IFBD procedure is applied.

The IFBD design process was governed by the serviceability inter-storey drift checks and the seismic response was consistent with these assumptions. On the other hand, the structures designed according to EC8 with a fixed value for the behaviour factor, were governed by the strict requirements related with the control of P-Delta effects. These structures performed well for the seismic intensity corresponding to the ultimate limit state but developed inter-storey drifts higher than the serviceability limits assumed at that design stage.

Moreover, the evaluation of the behaviour factor based on the actual properties of the structure instead of its selection according the structural system and the ductility class and provides the designer a more realistic idea of the capacity of the structure and hence of the structure expected behaviour.

However, given the limited number of studied cases, further analyses on steel buildings should be carried out in order to extract definite conclusions regarding the validation of the improved seismic design procedure adopted in this research.

References

Araújo, M., Macedo, L., Marques, M. and Castro, J.M. (2016), "Code-based record selection methods for seismic performance assessment of buildings", *Earthq. Eng. Struct. Dyn.*, **45**(1), 129-148.

ASCE 7/SEI (2010), Minimum Design Loads for Buildings and other Structures; American Society of Civil Engineers/ Structural Engineer Institute, Reston, VA, USA.

ATC3-06 (1978), Tentative provisions for the development of seismic regulations for buildings; Applied Technology Council, Redwood City, CA, USA.

ATC-19 (1995), Structural response modification factors; Applied Technology Council, Redwood City, CA, USA, pp. 5-32.

ATC-34 (1995), A critival review of current approaches to earthquake-resistance design; Applied Technology Council, Redwood City, CA, USA.

CEN (2004), EN1998-1-3, Eurocode 8: Design of structures for earthquake resistance- Part 1: general rules, seismic actions and rules for buildings; European Committee for Standardization, Brussels, Belgium.

CEN (2005), EN1998-1-1, Eurocode 3: Design of steel structures- Part 1: general rules, seismic actions and rules for buildings; European Committee for Standardization, Brussels, Belgium.

Costa, A., Romão, X. and Sousa Oliveira, C. (2010), "A methodology for the probabilistic assessment of behaviour factors", *Bull. Earthq. Eng.*, **8**, 47-64.
Elghazouli, A.Y. (2007), "Seismic design of steel Structures to Eurocode 8", *The Structural Engineer*, **85**(12), 26-31.
Elghazouli, Y. (2009), "Assessment of European seismic design procedures for steel framed structures", *Bull. Earthq. Eng.*, **8**, 65-89.
Elghazouli, A.Y., Castro, J.M. and Izzuddin, B.A. (2008), "Seismic performance of composite moment-resisting frames", *Engineering Structures*, **30**(7), 1802-1819.
FEMA (2009), Recommended seismic provisions for new buildings and other structures (FEMA P 750); Federal Emergency Management Agency, Washington, D.C., USA.
Ferraioli, M, Lavino, A. and Mandara, A. (2014), "Behaviour factor of code-designed steel moment–resisting frames", *Int. J. Steel Struct.*, **14**(2), 243-254.
Karavasilis, T.H., Bazeos, N. and Beskos, D. (2007), "Behaviour factor for performance-based seismic design of plane steel moment resisting frames", *J. Earthq. Eng.*, **11**(4), 531-559.
Kappos, J. (1999), "Evaluation of behaviour factors on the basis of ductility and overstrength studies", *Eng. Struct.*, **21**(9), 823-835.
Lee, L.H., Han, S.W. and Oh, Y.H. (1999), "Determination of ductility factor considering different hysteretic models", *Earthq. Eng. Struct. Dyn.*, **28**(9), 957-977.
Maheri, M.R. and Akbari, R. (2003), "Seismic behaviour factor, R, for steel X-braced and knee-braced RC buildings", *Eng. Struct.*, **25**(12), 1505-1513.
Paulay, T. and Priestley, M.J.N. (1992), *Seismic Design of concrete and Masonry Structures*, Jonh Wiley and Sons, London, UK.
Priestley, M.J.N. and Calvi, G.M. (1991), "Towards a capacity design assessment for reinforced concrete frames", *Earthq. Spectra*, **7**(3), 413-437.
Priestley, M.J.N., Calvi, G.M. and Kowalsky, M.J. (2007), *Displacement Based Seismic Design of Structures*, Istituto Universitario di Studi Superiori di Pavia, Pavia, Italy.
PEER (2006), OpenSEES: Open System for Earthquake Engineering Simulation. Pacific Earthquake Engineering Research Center, University of California, Berkeley, CA, USA.
Sullivan, T.J. (2013), "Direct displacement based seismic design of steel eccentrically braced frames structures", *Bull. Earthq. Eng.*, **11**(6), 2197-2231.
Roldán, R., Sullivan, T.J. and Della Corte, J. (2016), "Displacement-based design of steel moment resisting frames with partially-restrained beam-to-column joints", *Bull. Earthq. Eng.*, **14**(4), 1017-1046.
Tenchini, A., D'Aniello, M., Rebelo, C., Landolfo, R., da Silva, L.S. and Lima, L. (2014), "Seismic performance of dual-steel moment resisting frames", *J. Construct. Steel Res.*, **101**, 437-454.
Tzimas, A.S., Karavasilis, T.H., Bazeos, N. and Beskos, D. (2013), "A hybrid force/displacement seismic design method for steel building frames", *Eng. Struct.*, **56**, 1452-1463.
Villani, A., Castro, J.M. and Elghazouli, A.Y. (2009), "Improved seismic design procedure for steel moment frames", *Proceedings of the 6th International Conference Stessa-2009, Behaviour of Steel Structures in Seismic Areas*, Philadelphia, PA, USA, August, pp. 673-678.

CC