A HYBRID FORCE/DISPLACEMENT SEISMIC DESIGN METHOD FOR STEEL BUILDING FRAMES

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ABSTRACT :

This paper proposes a performance-based seismic design methodology for steel building frames which combines the advantages of the well-known force-based and displacement-based seismic design methods in a hybrid force/displacement design scheme. The proposed method controls structural performance by first transforming user-specified values of the interstorey drift ratio (non-structural damage) and local ductility (structural damage) to a target roof displacement and then, calculating the appropriate strength reduction factor for limiting ductility demands associated with the target roof displacement. The main characteristics of the proposed method are: (1) treats both drift and ductility demands as input variables for the initiation of the design process; (2) does not use a substitute single degree of freedom system; (3) makes use of current seismic code approaches as much as possible (e.g., conventional elastic response spectrum analysis and design); (4) includes the influence of the number of stories; (5) recognises the influence of the type of the lateral load resisting system (moment resisting frame or concentrically braced frame); (6) recognises the influence of geometrical (setbacks) or mass irregularities. A realistic design example serves to demonstrate the advantages of the proposed method over the currently used force-based design procedure.

KEYWORDS: Performance-based seismic design, steel frames, moment resisting frames, concentrically braced frames, setbacks, mass irregularities

1. INTRODUCTION

The current procedure for seismic design of new building structures is termed force-based design (FBD) since it uses seismic forces as the main design parameters. This design method [1] demands the design of the building against structural failures which might endanger human life on the basis of recommended constant values of the behavior (or strength reduction) factor, q, and checks deformations beyond which service requirements are no longer met after the detailing of the structure. The tentative guidelines for PBSD according to SEAOC [2] present two alternative forms of displacement-based design of new structures, namely, the direct displacement-based design (DDBD) method and the equal-displacement-based design (EDB) one. Contrary to FBD, those displacement-based procedures employ the maximum interstorey drift ratio (*IDR*) for describing performance levels and also treat user-defined values of the *IDR* as input variables for the initiation of the design process. However, these methods are limited in that they are applicable only to regular frames, adopt an equivalent linear (DDBD) or nonlinear (EBD) SDOF representation of the building, do not recognise basic differences in the response due to different lateral load resisting systems and try to control both structural and non-structural damage by imposing limits only on drift demands.

This paper proposes a performance-based seismic design methodology for steel building frames which combines the advantages of the well-known force-based and displacement-based seismic design methods in a hybrid force/displacement (HFD) design scheme. The method has been evolved from previous preliminary works of the authors [3,4] and its present (latest) version is firmly supported by extensive parametric studies of the authors [5-8] on the inelastic seismic response of planar steel frames. The main characteristics of the proposed method are: (1) treats both drift and ductility demands as input variables for the initiation of the design process; (2) does not use a substitute single degree of freedom system; (3) makes use of current seismic code approaches as much as possible (e.g., conventional elastic response spectrum analysis and design); (4) includes the influence of the number of stories; (5) recognizes the influence of the type of the lateral load resisting system (moment resisting frame or concentrically braced frame); (6) recognizes the influence of geometrical (setbacks) or mass irregularities. A recent comparison of the HFD, FBD and DDBD procedures for plane regular steel MRF yielded favorable results for HFD [9], while recent research work explores the extension of the method to the case of pulse-like (near-fault or soft-soil sites) earthquake ground motions [10].

2. STEPS OF THE PROPOSED DESIGN PROCEDURE

The proposed hybrid force/displacement (HFD) seismic design procedure can be summarized in the following steps:

(1) *Definition of the basic building attributes*. With reference to the types of frames depicted in Fig. 1, definition of the number of stories, n_s, number of bays, n_b, bay widths and storey heights, presence of setbacks (geometrical irregularity), different use of a specific floor compared to the adjacent ones (mass irregularity) and possible limits on the depth of beams and columns due to possible architectural requirements.

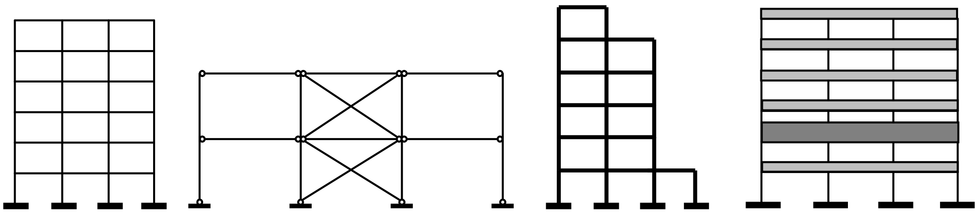

Figure 1 Types of planar steel building frames considered in the HFD seismic design method

(2) *Definition of the design performance level*. For example, immediate occupancy (IO) under the frequently occurred earthquake (FOE), life safety (LS) under the design basis earthquake (DBE) or collapse prevention (CP) under the maximum considered earthquake (MCE).

(3) *Definition of the input parameters (performance and irregularity metrics).*

i) Performance metrics. Definition of the acceptable values of the maximum *IDR* and maximum local ductility (rotation ductility, μ_θ, for beams/columns and cyclic elongation ductility, μ_{cb}, for braces) along the height of the frame.

ii) Irregularity metrics. For MRF with setbacks, quantification of the geometrical irregularity through the indices Φ_s and Φ_b which, with reference to Fig. 2, are given by the formulae [7]

$$\Phi_s = \frac{1}{n_s - 1} \cdot \sum_{i=1}^{i=n_s-1} \frac{L_i}{L_{i+1}} \qquad \Phi_b = \frac{1}{n_b - 1} \cdot \sum_{i=1}^{i=n_b-1} \frac{H_i}{H_{i+1}} \tag{2.1}$$

For MRF with mass irregularity, definition of the location (bottom, mid-height or top of the building) of the location (bottom, mid-height or top of the building) of the mass discontinuity [8].

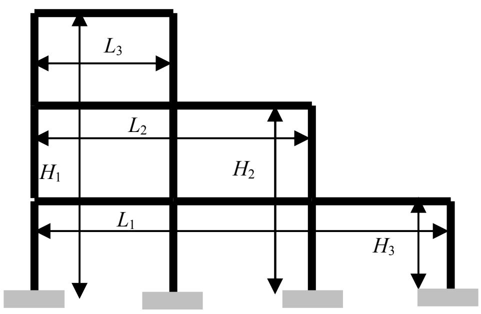

Figure 2 Geometry of frame with setbacks for definition of geometrical irregularity indices

(4) *Estimation of the input variables (yield roof displacement and mechanical characteristics).*

i) Estimation of the yield roof displacement, u_{ry}. This displacement corresponds to the formation of the first plastic hinge (for MRF) or to the initiation of buckling (for CBF).

ii) Estimation of mechanical characteristics. For MRF, estimation of the column-to-beam strength ratio, a, and beam-to-column stiffness ratio, ρ [6]

$$a = \frac{M_{\mathrm{RC,1,av}}}{M_{\mathrm{RB,av}}} \qquad \rho = \frac{\sum (I/l)_b}{\sum (I/l)_c} \tag{2.2}$$

where $M_{\mathrm{RC,1,av}}$ is the average of the plastic moments of resistance of the columns of the first storey, $M_{\mathrm{RB,av}}$ is the average of the plastic moments of resistance of the beams of all the stories of the frame, and I and l are the second moment of inertia and length of the steel member (column c or beam b). For CBF, estimation of the fundamental period of vibration, T, brace slenderness, λ, and ratio, a (contribution of the columns over that of the diagonals to the storey stiffness) [5]

$$\lambda = \frac{l}{\pi \cdot r} \cdot \sqrt{\frac{f_y}{E}} \qquad \mathrm{a} = \frac{n_{\mathrm{c}} \cdot I_{\mathrm{c}} \cdot L_{\mathrm{d}}}{n_{\mathrm{d}} \cdot A_{\mathrm{d}} \cdot h^3 \cdot \cos^2 \theta} \tag{2.3}$$

where l is the buckling length, r is the radius of gyration of the cross section, f_y is the yield strength of the material, E is the Young's modulus, n_c and n_d are the number of columns and diagonals belonging to the storey, respectively, A_d and L_d are the cross-section area and the length of the diagonals, respectively, I_c is the second moment of inertia of the columns, h is the storey height and θ the angle between diagonals and beams.

Good initial estimates of the aforementioned input variables may be obtained by designing the frame only for strength requirements under the FOE earthquake by assuming elastic behaviour, i.e., with q=1. The capacity design rules and the gravity load combination should be also considered in order to improve the initial estimation of the input variables of the proposed method.

The characteristics λ, a and ρ vary along the height of a steel frame and therefore, their nominal values are taken equal to those of the storey closest to mid-height of the building.

(5) *Transformation of local performance metrics to target roof displacement.* Transformation of the $IDR_{\max}$ to target maximum roof displacement, u_{rmax}, by employing the relation

$$u_{\mathrm{rmax}(IDR)} = \beta \cdot IDR_{\max} \cdot H \tag{2.4}$$

where H is the building height from its base and β is a coefficient depending on building properties and calculated through

$$\beta = 1 - 0.19 \cdot (n_s - 1.0)^{0.54} \cdot \rho^{0.14} \cdot \alpha^{-0.19} \tag{2.5}$$

for regular MRF [6];

$$\beta = 1 - 0.12 \cdot (n_{\mathrm{s}} - 1.0)^{0.31} \cdot \lambda^{-0.11} \cdot \alpha^{-0.21} \cdot \left(\frac{T}{T_{\mathrm{c}}}\right)^{0.14} \tag{2.6}$$

for regular CBF [5], where T_c is the period corresponding to the transition from the constant acceleration to the constant velocity regime of the design spectrum;

$$\beta = 1 - 0.13 \cdot (n_{\mathrm{s}} - 1.0)^{0.52} \cdot \Phi_{\mathrm{s}}^{0.38} \cdot \Phi_{\mathrm{b}}^{0.14} \tag{2.7}$$

for irregular MRF with setbacks [7];

$$\beta = 1 - 0.18 \cdot (n_{\mathrm{s}} - 1)^{0.4} \cdot a^{-0.13} \tag{2.8.a}$$

$$\beta = 1 - 0.17 \cdot (n_{\mathrm{s}} - 1)^{0.64} \cdot a^{-0.47} \tag{2.8.b}$$

$$\beta = 1 - 0.12 \cdot (n_{\mathrm{s}} - 1)^{0.66} \cdot a^{-0.28} \tag{2.8.c}$$

for irregular MRF with mass discontinuities located near to bottom, midheight and top of the building, respectively [8].

Transformation of local ductility to target roof displacement by employing the relation

$$u_{\mathrm{rmax}(\mu)} = \mu \cdot u_{\mathrm{ry}} \tag{2.9}$$

where the roof displacement ductility, μ, is associated with local ductility through

$$\mu = 1 + 1.35 \cdot (\mu_\theta - 1)^{0.86} \cdot \alpha^{0.43} \cdot n_s^{-0.31} \quad (2.10)$$

for regular MRF [6];

$$\mu = 1 + 1.51 \cdot \left(\mu_{cb} - 1\right)^{0.73} \cdot n_s^{-0.18} \cdot \lambda^{1.9} \cdot \left(\frac{T}{T_c}\right)^{0.28} \quad (2.11)$$

for regular CBF [5];

$$\mu = 1 + 0.44 \cdot (\mu_\theta - 1)^{1.26} \cdot \alpha^{0.26} \quad (2.12)$$

for irregular MRF with setbacks [7];

$$\mu = 1 + 0.50 \cdot (\mu_\theta - 1)^{1.12} \cdot \alpha^{0.36} \quad (2.13.a)$$

$$\mu = 1 + 0.78 \cdot (\mu_\theta - 1) \quad (2.13.b)$$

$$\mu = 1 + 0.50 \cdot (\mu_\theta - 1)^{1.18} \cdot \alpha^{0.39} \quad (2.13.c)$$

for irregular MRF with mass discontinuities located near to the bottom, midheight and top of the building, respectively [8].

The design target roof displacement, $u_{rmax(d)}$, is defined as the minimum value of $u_{rmax(IDR)}$ and $u_{rmax(\mu)}$.

(6) *Calculation of the behavior (or strength reduction) factor.* Calculation of the design value of the roof displacement ductility

$$\mu_d = \frac{u_{r\max(d)}}{u_{ry}} \quad (2.14)$$

and then, calculation of the required strength reduction factor

$$q = 1 + 1.39 \cdot (\mu_d - 1) \qquad \text{for } \mu_d \leq 5.8 \quad (2.15.a)$$

$$q = 1 + 8.84 \cdot (\mu_d^{0.32} - 1) \qquad \text{for } \mu_d > 5.8 \quad (2.15.b)$$

for regular MRF [6];

$$q = 1 + 0.86 \cdot (\mu_d - 1)^{0.62} \cdot n_s^{0.34} \cdot \lambda^{0.70} \cdot \alpha^{-0.10} \cdot \left(\frac{T}{T_c}\right)^{0.24} \quad (2.16)$$

for regular CBF [5];

$$q = 1 + 1.92 \cdot (\mu_d - 1)^{0.85} \cdot \Phi_s^{-0.17} \quad (2.17)$$

for irregular MRF with setbacks [7];

$$q = 1 + 2.26 \cdot \left(\mu_d - 1\right)^{0.69} \quad (2.18.a)$$

$$q = 1 + 2.42 \cdot \left(\mu_d - 1\right)^{0.68} \quad (2.18.b)$$

$$q = 1 + 2.45 \cdot \left(\mu_d - 1\right)^{0.60} \quad (2.18.c)$$

for irregular MRF with mass discontinuities located near to the bottom, midheight and top of the building, respectively [8].

(7) *Design of the structure.* Divide the ordinates of the elastic design spectrum with the q factor and design the building on the basis of an elastic response spectrum analysis by taking into account the capacity and ductile design rules of seismic codes [1]. The design is strength-based, i.e., the designer does not need to impose limits on the required stiffness (or period of vibration) of the frame. The required stiffness (or period for given mass) of the frame is controlled by the assumed value of the yield displacement (see step 4), while the required strength is imposed by the value of the strength reduction factor.

(8) *Iteration.* Iterate with respect to the input variables u_{ry}, ρ, α, T, λ, a. The sufficient number of iterations for achieving convergence depends on the initial estimations of the input variables (see step 4 of the method). Good initial estimates of the aforementioned input variables can be easily obtained by designing the frame only for

strength requirements under the FOE earthquake by assuming elastic behaviour, i.e., with q=1 (see design example in the next section of the paper).

3. APPLICATION OF THE PROPOSED HFD DESIGN METHOD

3.1. Description of building and design assumptions

The proposed HFD and the already used in practice FBD method are applied to the seismic design of the 5-storey office S275 steel building shown in Fig. 3. The building has storey heights equal to 3 m and bay widths equal to 6 m. Lateral load moment resisting frames are located only at the perimeter of the building, while gravity load resisting frames are arranged in the interior of the building. Only one perimeter lateral load resisting frame was considered in analysis, while a "dummy" column was used for simulating the gravity load columns and P-delta effects. Braced and irregular MRF frames are not examined herein since their treatment is similar with that of regular MRF, while simple design examples for braced and irregular MRF can be found in [5,7,8].

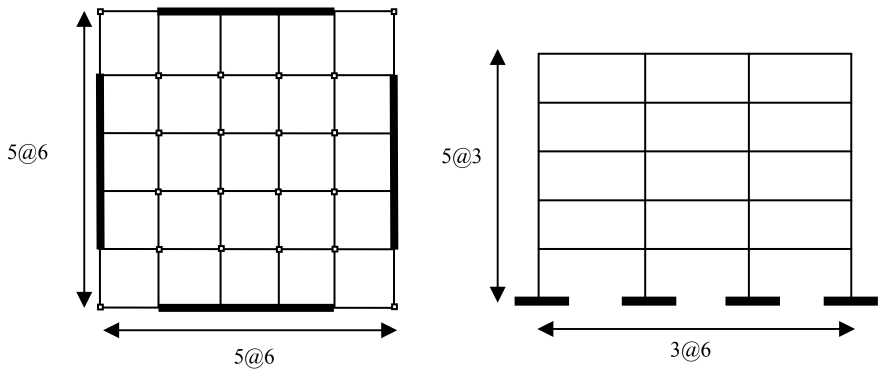

Figure 3 Five-storey steel office building structure

3.2. Definition of seismic performance levels

For the office building considered herein, it is assumed that immediate occupancy (IO) under the frequently occurred earthquake (FOE), life safety (LS) under the design basis earthquake (DBE) and collapse prevention (CP) under the maximum considered earthquake (MCE) are the appropriate performance levels for seismic design. The FOE, DBE and MCE earthquakes are expressed through the elastic design spectrum of EC8 for soil class B and damping ratio equal to 3%, while assumed seismological data provide the corresponding values of the peak ground acceleration (PGA) as PGA_{DBE}=0.35g, PGA_{FOE}=0.3 x PGA_{DBE}=0.1g and PGA_{MCE}=1.5 x PGA_{DBE}=0.53g (Fig. 4). The design criteria (target drift and ductility values) for special moment resisting frames (SMRF) are adopted from FEMA-273 [11] and provided in Table 3.1.

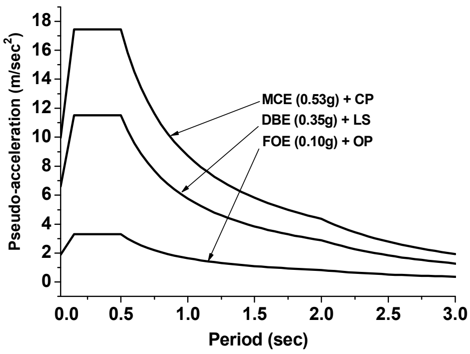

Figure 4 EC8 elastic design spectra

3.3. Moment resisting frames (MRF)

Table 3.1 Target values of performance metrics for SMRF according to FEMA-273 (1997)

Immediate Occupancy		Life Safety		Collapse Prevention	
Interstorey Drift	Local Ductility	Interstorey Drift	Local Ductility	Interstorey Drift	Local Ductility
0.7% transient negligible permanent	2.00	2.5% transient 1.0% permanent	7.00	5% transient or permanent	9.00

An initial elastic design for the FOE earthquake yields "HEB400-IPE400, HEB400-IPE500, HEB360-IPE400, HEB360-IPE400, HEB340-IPE300" sections for the five stories of the frame, respectively. According to Eqn. 2.2, the values of the parameters ρ and a of the frame are equal to 0.2 and 2.4, respectively, while roof displacement under the FOE response spectrum equals 0.1 m. The above values serve as initial estimates for the input variables of the HFD. The target values of the $IDR_{\max}$ and μ_θ for the LS performance level are equal to 2.5% and 7.0, respectively [11]. The value of the ratio β is calculated on the basis of Eqn. 2.5 and found to be equal to 0.73. By employing Eqn. 2.4, the target roof displacement $u_{\mathrm{rmax}(IDR)}$ is calculated equal to 0.73 x 2.5% x (5 x 3) = 0.27 m, while by employing Eqn. 2.10 the displacement ductility μ is found to be equal to 1 + 1.35 x $(7\text{-}1)^{0.86}$ x $2.4^{0.43}$ x $5^{-0.31}$=6.6 and therefore, the target roof displacement $u_{\mathrm{rmax}(\mu)}$ becomes equal to 0.1 x 6.6=0.66 m. The design roof displacement is equal to min (0.27, 0.66) = 0.27 and therefore, drift controls the LS performance level design. The design value of the roof displacement ductility is 0.27/0.1=2.7. The required strength reduction factor is calculated on the basis of Eqn. 2.15 and found to be equal to 3.4. The DBE response spectrum is reduced by this factor and the design yields "HEB450-IPE400, HEB450-IPE450, HEB400-IPE450, HEB400-IPE400, HEB360-IPE360" sections for the five stories of the frame, respectively. The new values of the input variables of HFD are u_{ry}=0.095, ρ=0.22 and a=3. A second execution of the hand calculations of HFD provides a value of q equal to 3.5, which does not change the sections obtained with response spectrum analysis/design and therefore, the design with respect to the LS performance level is finalized. Apparently, this frame remains elastic under the IO response spectrum since it has larger sections than the frame which serves to provide initial estimates of the input variables. Elastic analysis under the IO response spectrum provides a value of the IDR equal to 0.65% and roof displacement equal to 0.09 m and therefore, the frame which satisfies the LS performance level satisfies also the IO performance level. The same initial values (u_{ry}=0.1, ρ=0.2 and a=2.4) are used for designing the frame with respect to the CP damage state under the MCE earthquake. The first execution of the hand calculations of HFD prove again that drift controls design with a corresponding value of the q factor equal to 7.2 which is used to divide the MCE spectrum. A response spectrum analysis/design under the reduced MCE spectrum provides a lighter frame than the one obtained by designing with respect to the LS performance level. Therefore, the LS performance level controls the design of the frame. Strictly speaking, the designer may want to obtain the expected values of the u_{r}, IDR and μ_θ under all earthquake intensities. Under the FOE earthquake, the frame remains elastic and thus μ_θ=1.0, while the u_{rmax} and $IDR_{\max}$ were calculated equal to 0.65% and 0.09 m, respectively. Under the DBE earthquake, the $IDR_{\max}$ and u_{rmax} were calculated equal to 2.5% and 0.27 m, respectively, while by using the values q=3.5 and a=3 in Eqn. 2.15 and Eqn. 2.10, the μ_θ is calculated equal to 2.41. The q factor of the frame under the MCE earthquake is easily obtained as $(PGA_{\mathrm{MCE}}/PGA_{\mathrm{LS}})*q_{\mathrm{LS}}$=1.5*3.5=5.25. This value is used in order to estimate the response of the frame under the MCE earthquake, i.e., u_{rmax}=0.41 m, $IDR_{\max}$=3.65% and μ_θ=3.67.

According to EC8 [1], the FBD of the frame starts by performing strength-based (no drift control) design under the DBE earthquake on the basis of a constant value of the q factor which equals to 6.5 (= 5 x 1.3, where 1.3 is the recommended overstrength factor for MRF) which yields "HEB240-IPE300, HEB240-IPE300, HEB220-IPE270, HEB220-IPE270, HEB200-IPE240" sections for the five stories of the frame, respectively. Then, EC8 demands drift control for the IO earthquake. The latter frame behaves inelastically under the IO earthquake, while the equal-displacement rule provides a value of the $IDR_{\max}$ equal to 1.23% which is larger than the 0.7% drift limit recommended by FEMA [11]. Thus, an iterative design process is needed in order to find larger sections of the frame that satisfy this drift limit. Finally, the sections which are obtained are "HEB400-IPE400, HEB400-IPE500, HEB360-IPE400, HEB360-IPE400, HEB340-IPE300". This frame remains elastic under the IO earthquake and experiences u_{rmax}=0.10 m, $IDR_{\max}$=0.7% and μ_θ=1.0. According to FBD, the IO performance level controls the design. Under the DBE earthquake, the frame will experience u_{rmax}=3.5 x 0.1=0.35 m and IDR_{max}=3.5 x 0.7=2.45%, while under the MCE earthquake the frame will experience u_{rmax}=5.3 x 0.10=0.53 m and IDR_{max}=5.3 x 0.7=3.7. These drifts are calculated in order to check (see next section of the paper) the validity of the equal-displacement rule and they are not used to check the performance of the frame under the FOE and DBE earthquake, i.e., they should not be compared with the target drift values of Table 3.2. Even if the target drifts of Table 3.2 were smaller (for instance, *IDR*=1.5% under the DBE earthquake), the design product of FBD would be the same since it satisfies drifts under the FOE earthquake and strength under the DBE earthquake. Strictly speaking, FBD may be used for drift control for any performance level by employing the iterative process which was employed for the FOE earthquake.

Data of the MRF frames designed according to the HFD and FBD procedures are presented in Table 3.2. The MRF obtained by using FBD is slightly lighter than the one obtained by using HFD.

Table 3.2 Data pertinent to the designed frames

	HFD	FBD (EC8)
Sections	HEB450-IPE400 HEB450-IPE450 HEB400-IPE450 HEB400-IPE400 HEB360-IPE360	HEB400-IPE400 HEB400-IPE500 HEB360-IPE400 HEB360-IPE400 HEB340-IPE300
T (sec)	1.70	1.75
Steel weight (KN)	147	144
Controlling performance level	LS	IO

3.4. Evaluation of the design through nonlinear dynamic analyses

Ten semi-artificial accelerograms, compatible with the frequency content of the EC8 [5] design spectrum, were generated via a deterministic approach [30]. The ordinates of these ground motions were scaled in order to match the three design levels of seismic intensity considered here and then, nonlinear time-history analyses were executed by using the program ANSR-PC [31]. The mean values of the maximum response quantities from time-history analyses (TH) are compared with the estimations (EST) of the HFD and FBD design methods in Table 3.3. The results reveal the consistency of HFD to accurately estimate inelastic deformation demands and the tendency of FBD to overestimate the maximum roof displacement and to underestimate the maximum interstorey drift ratio along the height of the frame.

Table 3.3 Time history analyses results and comparison with design estimations

	HFD						FBD (EC8)					
	FOE		DBE		MCE		FOE		DBE		MCE	
	TH	EST	TH	EST	TH	EST	TH	EST	TH	EST	TH	EST
IDR_{max} (%)	0.63	0.65	2.4	2.5	3.75	3.65	0.67	0.70	3.00	2.45	4.20	3.7
u_{rmax} (m)	0.11	0.09	0.26	0.27	0.43	0.41	0.12	0.10	0.31	0.35	0.46	0.53
μ_θ	1.0	1.0	2.7	2.4	3.6	3.7	1.0	1.0	2.8	-	3.8	-

5. CONCLUSIONS

A performance-based seismic design methodology for steel building frames which combines the advantages of the well-known force-based and displacement-based seismic design methods in a hybrid force/displacement (HFD) design scheme has been proposed. The method has been applied to the case of a realistic 5-storey office building structure and compared with the force-based design (FBD) procedure of the EC8 seismic code. The advantages of the HFD method over the FBD method were illustrated by exploring (a) the ability of the methods to identify the performance level which truly controls the design and (b) by comparing the inelastic deformation estimates of both methods with the results of the rigorous nonlinear dynamic analysis. More specifically, the HFD method identified the life safety performance level as the critical one, while the FBD design was controlled by the immediate occupancy performance level. The results of the nonlinear time history analyses for three earthquake intensities (FOE, DBE and MCE) revealed the consistency of the HFD to accurately estimate inelastic deformation demands and the tendency of the FBD to overestimate the maximum roof displacement and to underestimate the maximum interstorey drift ratio along the height of the frame.

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