COMPARISON OF THREE SEISMIC DESIGN METHODS FOR STEEL MR FRAMES

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1. INTRODUCTION

The traditional procedure for seismic design of building structures has been generally termed the force-based design (FBD) method. The implementation of FBD in seismic codes, e.g. EC8, 2004 does not clearly define and employ the terms performance objective, performance level or limit state and expected level of seismic motion or seismic intensity. In contrary, the FBD method implements performance objectives in a very approximate manner, actually through the use of one performance level, that is the ultimate limit state (ULS) and one seismic intensity for 475-year period ground motions. The FBD method uses the importance factor for different seismic intensities and the reduction factor ν for different limit states. The FBD method usually overestimates the inelastic displacements.

The design procedure would be more rational if the performance of the structure was quantified through a target value of deformation treated as an input variable in that design procedure. This target value of deformation can be assigned to different performance objectives and is the starting point for the development of the direct displacement-based seismic design (DDBD) method (Loeding et al., 1998 [1], Priestley et al, 2007[2]). This design method is being advocated as a promising method for the next generation of seismic codes and has already been adopted by seismic design provisions in the United States (SEAOC, 1999 [3]) as one of the proposed displacement-based seismic design methods for the performance-based seismic design.

The third seismic design method presented and compared in this work was originally proposed in a sketchy manner by Bazeos and Beskos, 2003 [4] and further developed in

detail by Karavasilis, 2007 [5]. This method is called the hybrid force/displacement-based seismic design (HFD) method as it combines the best elements from both the force-based and the displacement-based methods in order to produce an effective design scheme. The starting point in the HFD method is the maximum allowable roof displacement of the MDOF structure computed through a new simple expression proposed by Karavasilis et al, 2007 [6], which takes into account structure properties and seismic excitation characteristics. Then, a new relation is used for the calculation of the behaviour factor (Karavasilis et al, 2007 [6]), which is used in a similar way as in the FBD method.
The objective of this article is to critically compare three seismic design methods, the two well established force-based and displacement-based methods, and the new hybrid method.

2. FORCE BASED DESIGN (FBD)

The FBD method relies on the ability of the behaviour factor q to estimate both strength and displacement demands. According to EC8 seismic code, the following steps shall be followed for the design of a building in terms of the FBD method:

2.1 Definition of the performance level of the building

EC8 seismic code identifies two levels of performance: (1) no-(local)-collapse requirement which is associated with the ultimate limit state (ULS) of the structure subjected to destructive earthquakes; (2) damage limitation requirement which is associated with the serviceability limit state (SLS) of the structure for frequent seismic action.

2.2 Design at the ultimate limit state (ULS)

Current design practice implies that the structure is first designed at the ULS and subsequently checked at SLS. Thus, firstly the design seismic action is determined by adopting an appropriate elastic response spectrum with damping corresponding to the construction site and then the behaviour factor q is computed. According to EC8, for high ductility class of buildings, the maximum allowable behaviour factor q is given by the equation

$$q = 5\frac{a_u}{a_1} \tag{1}$$

where a_u is the value by which the horizontal seismic design action is multiplied in order to form plastic hinges in a number of sections sufficient for the development of overall structural instability and a_1 is the value by which the horizontal seismic design action is multiplied in order to first reach the plastic resistance in any member in the structure.

2.3 Computation of inelastic displacements and interstorey drifts

The inelastic displacement $u_{inel,i}$ and maximum interstorey drift ratio $IDR_{max,i}$ in storey i of a multi-degree of freedom (MDOF) structure are computed by the equations

$$u_{\mathrm{inel,i}} = q \cdot u_{\mathrm{d,i}} \text{ and } IDR_{\max,i} = \frac{q \cdot (u_{\mathrm{d,i}} - u_{\mathrm{d,i-1}})}{h_i} \tag{2}$$

respectively, where $u_{\mathrm{d,i}}$ is the elastic displacement of storey i calculated in step 2 and h_i is the storey height.

3. DIRECT DISPLACEMENT BASED DESIGN (DDBD)

The direct displacement-based design method (DDBD) is probably the most well known displacement-based design method and is adopted by modern design codes. The following steps shall be followed for the design of a building in terms of the DDBD method:

3.1 Definition of the performance level of the building

Performance-based seismic engineering adopts different design levels to meet different ‘performance levels’ at different seismic ‘hazard levels’ (Fardis, 2002 [7]). In the DDBD method the hazard levels are identified by the appropriate elastic displacement response spectrum and the performance levels are determined by the maximum IDR values defined according to a modern design code, e.g., SEAOC 1999 [3].

3.2 Computation of the target displacement of the SDOF

The target displacement $u_{eff,t}$ of the equivalent substitute single-degree of freedom (SDOF) system to the MDOF given structure is computed by the equation

$$u_{\mathrm{eff,t}} = \frac{\sum_{i=1}^{n} m_{\mathrm{i}} u_{\mathrm{i}}^{2}}{\sum_{i=1}^{n} m_{\mathrm{i}} u_{\mathrm{i}}} \tag{3}$$

where n is the number of stories and m_i the mass and u_i the maximum displacement of storey i, respectively. The maximum displacement profile is an important issue in the DDBD, because it correlates the damage index (IDR_{max}), as defined for a specific performance level, with the maximum displacement of the stories. For the computation of the maximum displacement profile, one can use the profiles suggested by Loeding et al (1998) [1] for regular reinforced concrete frames in the elastic range.

3.3 Computation of the yield displacement of the SDOF

The yield displacement $u_{eff,y}$ of the equivalent substitute SDOF system is computed by Eq. (3) for damage index (IDR_y) corresponding to the yielding of the frame.

3.4 Computation of the equivalent ductility and equivalent damping of the SDOF

The ductility μ_{eff} of the equivalent substitute SDOF system is calculated through the expression

$$\mu_{\mathrm{eff}} = \frac{u_{\mathrm{eff,t}}}{u_{\mathrm{eff,y}}} \tag{4}$$

where the $u_{eff,t}$ and $u_{eff,y}$ are the maximum and yield displacement, respectively, of the SDOF substitute system.

The effective damping associated with a specific level of ductility can be calculated by using the expression proposed by Borzi et al., 2001, [8]

$$\xi_{eff} = a \cdot \left(1 - \frac{1}{\mu_{eff}}\right) + \xi_v \tag{5}$$

where α is a coefficient depending upon the ductility and ξ_v stands for the viscous part of the damping, usually taken to be 5%.

3.5 Computation of the equivalent period of the SDOF

With a target displacement $u_{eff,t}$ and an effective damping ξ_{eff} calculated in the previous steps, the effective structural period T_{eff} can be directly obtained from the elastic displacement response spectrum (*Fig. 1*)

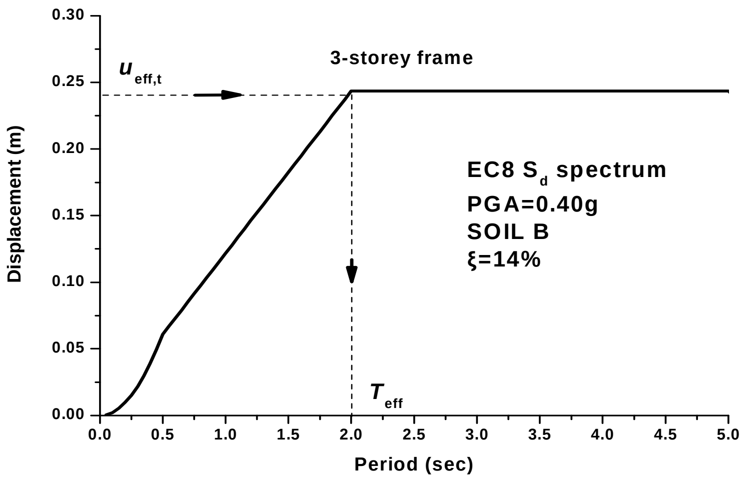

Fig. 1. Elastic displacement response spectrum.

3.6 Computation of the equivalent stiffness and base shear of the SDOF

The effective stiffness K_{eff} is calculated by using the simple relation of structural dynamics

$$K_{\text{eff}} = \frac{4 \cdot \pi^2}{T_{\text{eff}}^2} \cdot M_{\text{eff}} \tag{6}$$

where M_{eff} is the mass of the *SDOF* structure given by

$$M_{eff} = \sum_{i=1}^{n} m_i \frac{u_i}{u_{eff,t}} \tag{7}$$

Finally, the required shear strength V_{eff} of the SDOF system can be computed by the equation

$$V_{\text{eff}} = K_{\text{eff}} \cdot u_{\text{eff,t}} \tag{8}$$

3.7 Design of the MDOF structure

The strength of the structure refers to the maximum inelastic base shear which the structure should have in order to achieve the requirements of the performance level under consideration (step 1 of the DDBD method). This base shear V_{eff} should be divided by the overstrength factor Ω in order to produce the design one, V_d, i.e.,

$$V_d = \frac{V_{eff}}{\Omega} \tag{9}$$

The overstrength factor Ω depends on the number and the sequence of development of plastic hinges in the structure and on the geometric and mechanical characteristics of the structure. One may rely on the recommendation of EC8 and take Ω=1.3.

4. HYBRID FORCE/DISPLACEMENT BASED DESIGN (HFD)

The current version of the newly proposed hybrid force/displacement (HFD) seismic design method is valid for plane steel frames (moment resisting, x-braced, with setbacks or with mass irregularities) (Karavasilis, 2007 [5]). The following steps shall be followed for the design of a plane steel moment resisting frame in terms of the HFD method:

4.1 Definition of the performance level of the building

In the HFD method the seismic hazard levels are identified with the appropriate elastic acceleration response spectrum, as it is defined in EC8 and the performance levels are determined by the maximum IDR values.

4.2 Computation of the maximum allowable roof displacement of the building

The maximum allowable roof displacement $u_{r,max}$ of the building can be computed on the basis of the maximum IDR value, as defined in the previous step, with the aid of the expression

$$u_{r,max,IDR} = \beta \cdot IDR_{max} \cdot H \tag{10}$$

where H is the building height from its base and β is a coefficient depending on building properties and seismic excitation characteristics, which can be calculated through the equation (Karavasilis 2007 [5])

$$\beta = 1.0 - 0.193 \cdot (n_s - 1.0)^{0.54} \cdot \rho^{0.144} \cdot \alpha^{-0.19} \tag{11}$$

In the above, n_s is the number of stories, ρ is the stiffness ratio of the frame calculated for the storey closest to the mid-height of the frame via the expression

$$\rho = \frac{\sum (I/l)_b}{\sum (I/l)_c} \tag{12}$$

with *I* and *l* being the second moment of inertia and length of the steel member (column *c* or beam *b*), respectively and *α* is the ratio defined as

$$\alpha = \frac{M_{RC,1,av}}{M_{RB,av}} \tag{13}$$

where $M_{RC,1,av}$ is the average of the plastic moments of resistance of the columns of the first storey and $M_{RB,av}$ is the average of the plastic moments of resistance of the beams of all the stories of the frame.

4.3 Calculation of the required strength and design of structure

First the maximum allowable roof displacement ductility factor μ_δ is computed by the expression

$$\mu_\delta = \frac{u_{r,max,IDR}}{u_{r,max,y}} \tag{14}$$

where $u_{r,max,IDR}$ and $u_{r,max,y}$ are defined in the previous step. Then, the behaviour factor q can be calculated by the equations (Karavasilis 2007 [5])

$$q = 1 + 1.39 \cdot (\mu_\delta - 1) \quad \text{for} \quad \mu_\delta \leq 5.8 \tag{15a}$$

$$q = 1 + 8.84 \cdot (\mu_\delta^{0.32} - 1) \quad \text{for} \quad \mu_\delta > 5.8 \tag{15b}$$

The behaviour factor q is used in a similar way as in FBD.

5 COMPARISON OF THE METHODS THROUGH A DESIGN EXAMPLE

Consider a S275 plane steel moment resisting frame with three storeys and two bays. All bay widths are assumed equal to 7 m and all storey heights equal to 3,5m. The gravity load on beams is equal to 30kN/m and the viscous damping ratio ξ is equal to 3%. The expected ground motion is represented by the design elastic spectrum of the EC8 seismic code with peak ground acceleration equal to 0.4g and a soil class B. The frame is designed according to EC3, 1992 with the aid of the commercial analysis and design software package SAP2000. HEB profiles are used for the columns and IPE profiles for the beams. This frame was seismically designed by the three methods described previously and comparisons were made with respect to their accuracy, efficiency and degree of conservatism.

5.1 Force based seismic design (FBD)

The value of q is selected equal to 6.5 according to equation (1) since the ratio α_u/α_1 for moment resisting frames with more than one bay is equal to 1.3 (EC8). Elastic modal analysis and design leads to the optimum sections HEB240 for columns and IPE330 for beams. The maximum roof displacement $u_{d,r}$= 3.4 cm, while the maximum interstorey drift ratio occurs at the second floor and is equal to 0.46%. Thus, their maximum inelastic counterparts are $u_{inel,r}$=6.5*3.4 = 22.1 cm and IDR_{max}= 6.5*0.0046 = 2.9%, respectively. Finally, the base shear is V_b=130kN.

5.2 Displacement based seismic design (DDBD)

For comparison purposes with the FBD method described previously, the value of IDR_{max} computed in the last step of the FBD method is chosen here as the appropriate damage level, i.e. IDR_{max}=2.9%. By using equation (3) and the displacement profile proposed by Loeding et al (1998) [1], the target displacement $u_{eff,t}$ of the SDOF system is computed equal to 24.0 cm. Similarly, using equation (3) with interstorey drift ratio at yielding of the equivalent elasto-plastic SDOF system, i.e., IDR_y=1.0%, the yield displacement $u_{eff,y}$ of the SDOF system is calculated equal to 8.1 cm. Then, the ductility μ_{eff} and the effective damping ξ_{eff} are evaluated from equations (4) and (5) and they are equal to 2.96 and 14.3%, respectively. By entering the displacement response spectrum (*Fig. 1*) with a damping value equal to 14.3% and a displacement equal to 24.0 cm, the effective period T_{eff} is found equal to 2.00 sec. The effective mass is calculated through equation (7) and consequently the effective stiffness is found through equation (6) to be equal to 1071 kN/m. The required strength in terms of the base shear is found through equation (8) to be equal to 257 kN. Thus, the design base shear V_d is computed according to equation (9) with a value of Ω equal to 1.3 and found to be equal to 197.7 kN. This design base shear is distributed linearly along the height of the frame and an elastic analysis and design is performed, which leads to the optimum sections HEB280 for columns and IPE360 for beams. The maximum roof displacement is estimated from the displacement profile proposed by Loeding et al, 1998 [1] to be equal to $u_{d,r}$= 30.0 cm.

5.3 Hybrid force/displacement seismic design (HFD)

For comparison purposes with the previous methods, a value of IDR_{max} equal to 2.9% is chosen. The maximum allowable roof displacement $u_{r,max}$ of the building is then computed by using Eqs (10) and (11), with H=10.5 m, n_s=3, ρ=0.5 and α=1.3 and found to be equal to 23 cm. The maximum roof displacement $u_{r,max,y}$ can be calculated from Eq. (10) for IDR_{max} equal to IDR_y. Also $u_{r,max,y}$= IDR_y*H, since the displacement profiles are linear in the elastic response of the frame and IDR_y is equal to 0.4% for S275 (Karavasilis et al, 2007a [6]). Thus one obtains $u_{r,max,y}$=4.20 cm. The maximum allowable roof displacement ductility factor μ_δ as computed by Eq. (14), is obtained equal to 5.48, while the behaviour factor q as evaluated by Eq. (15a) is found to be equal to 7.20. The elastic response spectrum of EC8 is divided by q and an elastic modal analysis is performed which yields the optimum sections HEB240 for columns and IPE330 for beams. These sections are the same with those of the FBD method. The fundamental natural period of the frame is found to be equal to 1.20 sec and the base shear is found to be equal to 108kN. Finally, the values of ρ, α and $u_{r,max,y}$ are computed, compared to those initially assumed and found to be very close to them.

6. CONCLUSION

Table 1 summarizes the results of the three seismic design methods. In order to compare the three seismic design methods, nonlinear time history analyses of the designed frames are performed using the well-known program DRAIN-2DX. Eight semi-artificial accelerograms compatible with the EC8 spectrum were generated via a deterministic approach. The results, shown in *Table 1*, reveal that the proposed method along with the FBD method yield more economical sections than the DDBD method, but the proposed method seems to predict more accurately the maximum roof displacement, $u_{r,max}$, compared to the two other methods.

	FBD	DDBD	HFD	Nonlinear	Nonlinear
Sections	HEB240-IPE330	HEB280-IPE360	HEB240-IPE330	HEB240-IPE330	HEB280-IPE360
IDR_{max} (-)	2.9%	2.9%	2.9%	3.0%	2.4%
$u_{r,max}$(cm)	22.1	30.0	23.0	24.0	19.0
V_b (kN)	130	197	108	-	-

Table 1. Comparison results

7. REFERENCES

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ΣΥΓΚΡΙΣΗ ΤΡΙΩΝ ΜΕΘΟΔΩΝ ΑΝΤΙΣΕΙΣΜΙΚΟΥ ΣΧΕΔΙΑΣΜΟΥ ΜΕΤΑΛΛΙΚΩΝ ΠΛΑΙΣΙΩΝ

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ΠΕΡΙΛΗΨΗ

Τρεις μέθοδοι αντισεισμικού σχεδιασμού παρουσιάζονται και συγκρίνονται μεταξύ τους. Η πρώτη μέθοδος είναι η γνωστή και ευρέως διαδεδομένη μέθοδος των δυνάμεων (ή φασματική μέθοδος, όπως αναφέρεται στον ΕΑΚ), η δεύτερη είναι η μέθοδος των μετακινήσεων, μια νέα μέθοδος αντισεισμικού σχεδιασμού κατασκευών, η οποία πρόσφατα έχει υιοθετηθεί από ορισμένους αντισεισμικούς κανονισμούς. Η τρίτη μέθοδος είναι μια υβριδική μέθοδος που συνδυάζει και βελτιώνει στοιχεία και από τις δύο προηγούμενες μεθόδους. Σύμφωνα με την υβριδική μέθοδο η μετακίνηση σχεδιασμού υπολογίζεται από το επίπεδο επιτελεστικότητας που επιλέγεται για τον αντισεισμικό σχεδιασμό της κατασκευής. Ο υπολογισμός αυτός επιτυγχάνεται με την βοήθεια προτεινομένων σχέσεων, οι οποίες λαμβάνουν υπόψη γεωμετρικά χαρακτηριστικά της κατασκευής και χαρακτηριστικά του σεισμού σχεδιασμού. Επίσης δίνονται νέες απλές σχέσεις για τον υπολογισμό του συντελεστή συμπεριφοράς q συναρτήσει της απαιτούμενης πλαστιμότητας της κατασκευής. Τέλος, οι τρεις μέθοδοι εφαρμόζονται σε επίπεδο μεταλλικό πλαίσιο και τα αποτελέσματα συγκρίνονται με μη γραμμικές δυναμικές αναλύσεις οκτώ διαφορετικών σεισμικών διεγέρσεων.