**Extension of the hybrid force/displacement (HFD) seismic design method to 3D steel moment-resisting frame buildings**

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ABSTRACT

The hybrid force/displacement (HFD) seismic design method for planar steel frames developed by the authors is extended to 3D steel buildings using moment-resisting frames. HFD combines the advantages of both the displacement-based and the force-based seismic design methods and reduces or eliminates their disadvantages. An extensive response databank is developed through nonlinear dynamic analyses on 38 steel space frames designed according to Eurocodes 3 and 8 and subjected to 42 pairs of earthquake ground motions. This response databank is then utilized for the development of empirical formulae providing the behavior factor as a function of the geometrical and dynamic characteristics of the building, including its accidental eccentricity, as well as the target maximum interstorey drift ratio and local ductility. Thus, the proposed seismic design method, eventhough works as a force-based design one, controls structural and non-structural damage through the use of a behavior factor~~,~~ which is a function of seismic deformation demands. Numerical examples are presented to illustrate the proposed method and demonstrate its merits over the force-based seismic design method of Eurocode 8.

**Keywords**: Steel space frames, Hybrid force/displacement design, Moment resisting frames, Regular frames, Behavior factor controlling deformation

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1. **Introduction**

The performance-based seismic design (PBSD) philosophy has been successfully implemented in seismic design and assessment methods for new or existing buildings [1-3]. Vision 2000 Report [4] and subsequent documents [5-7] include some of the early procedures for PBSD; known as first generation PBSD. These documents outline the basic goal of this design philosophy as the achievement of a desired structural performance for various levels of seismic action. This is accomplished by defining the seismic action levels and the corresponding desired damage levels. Every pair of seismic action and damage level constitutes a performance level, while the whole set of performance levels comprises the performance objective or design goal. The damage levels refer to structural and non-structural elements, while usual performance metrics are seismic forces, peak floor accelerations and deformations with limit values corresponding to specific performance levels, such as Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP). A true PBSD approach, now known as second-generation PBSD, allows the satisfaction of specific performance targets stated in terms of annual frequency and a maximum allowable value of response, such as monetary losses, downtime, and casualties, which are expressed in terms of the direct interest of various stakeholders [8, 9].

Current seismic design codes for building structures, such as EC8 [10], UBC [11] or BSJ [12], use the force-based design (FBD) method, which employs the seismic force as the basic design parameter. EC8 [10] adopts two limit states, the ultimate limit state (ULS) and the damage limit state (DLS). The design of the structure is first performed with respect to the ULS by using the design seismic action (475 years return period). The later is represented by the inelastic design spectrum, which is derived from the corresponding elastic one by dividing its ordinates with the behavior (strength reduction in USA) factor *q*. Use of the inelastic spectrum in conjunction with a response spectrum analysis leads to the lateral design forces and the member dimensioning in a trial and error fashion. After this ULS strength-based design, one checks the adequacy of the structural stiffness to limit the deformation under specific target values that satisfy serviceability requirements for the frequent earthquake (95 years return period). If this is not the case, the structure is re-designed for increased stiffness. Stiffness cannot be arbitrary increased, and therefore, a new force-based design is typically needed for a lower value of the *q* factor leading to increased strength, and indirectly, to the desired increased stiffness for a given yield strength of material.

With a view to better control the seismic damage in a structure, displacement-based design (DBD) methods have been also developed [1, 13, 14]. Out of the various DBD methods, the direct displacement-based design (DDBD) method [13] is the most widely used one. A description and comparison of various DBD methods has been reported in Sullivan et al. [15]. The DDBD method is based on the equivalent linearization [13] along with a substitution of the multi degree of freedom (MDOF) structure with a single degree of freedom (SDOF) structure. The method starts with the target interstorey drift ratio (*IDR*) and in conjunction with a displacement design spectrum and the substitute SDOF structure, determines the seismic base shear force required for the structure to experience the desired deformation. The method controls both structural and non-structural damage by imposing limits only on *IDR*.

Recently, the hybrid force/displacement (HFD) design method for planar steel building frames has been proposed by the present authors [16]. This is a preliminary PBSD method (i.e. should be seen as an alternative to the FBD and DDBD), and therefore, the actual risk (i.e. mean annual frequency) of exceeding a performance level is not necessary guaranteed [16]. The method combines the advantages of both the FBD and DDBD in a hybrid force/displacement design scheme. The method has been evolved from previous works of the authors [17-23] on planar steel frames ~~of~~ with different mechanical and geometrical characteristics. A comparison of the FBD, DDBD and HFD methods for planar steel moment resisting frames (MRF) carried out by Bazeos [24] yielded favorable results for the HFD. The HFD method has been mainly studied in association with far-fault ordinary ground motions, while an effort was made in [25] to extend it to the case of near-fault pulse-like ground motions. The main advantage of the HFD seismic design method over the FBD and DDBD methods is the ability to direct control both structural and non-structural damage with fewer design iterations without the use of a substitute SDOF structure [24]. The HFD starts by using both *IDR* and local ductility target values and transforms them to a target roof displacement in order to obtain a behavior factor *q*. HFD then determines seismic design forces by utilizing an acceleration design spectrum along with response spectrum analysis; similarly to the FBD method. Therefore, the engineer works with familiar to him concepts and tools and avoids the employment of a highly damped displacement design spectrum, which is used by the DDBD.

The SEAOC [1] recommends limit values for the *IDR* to describe damage of both structural and non-structural elements. Drifts are good and direct measures for non-structural damage, while for structural members, damage indices which take into account the variability of yield deformation such as local ductility indices proposed by ASCE 41-13 [26] are required. The proposed HFD method controls both non-structural and structural damage by adopting predefined values of the *IDR*max and local ductility *μ*θ, respectively.

In this work, the HFD is extended to regular in height and plan space steel MRFs under far-fault ordinary ground motions. Torsional effects are taken into consideration by means of accidental eccentricity. The study involves the design of 38 space frames with three values of eccentricity (0%, 5% and 10%) and non-linear dynamic analysis for 42 pairs of horizontal ground motions and four different performance levels. The peak seismic response results are used to derive empirical expressions used within the framework of the HFD. The importance of employing non-linear dynamic analysis for multistorey space frames exhibiting torsional motion has been highlighted in the recent works of Vasilopoulos and Beskos [27] and Anagnostopoulos and co-workers [28-30]. The use of the existing empirical expressions of the HFD method derived for planar steel MRF [20] to estimate the peak inelastic response of space frames is also investigated.

1. **Geometry and seismic design of frames considered**

In this work, 38 steel space MRFs regular along their height and with a rectangular plan view are considered. The storey height is 3.00 m while the bay span is either 5.00 m or 7.50 m and may be the same or different in the two horizontal plan directions. The space frames consist of three groups A, B and C, as shown in Figs 1a-1c. Groups A and B include 5 frames of 3, 6, 9, 12 and 15 stories, while group C 4 frames of 3, 6, 9 and 12 stories. The bay dimensions along the two x and y directions of every group are shown in Figs 1a-1c. Fig. 1d shows in 3D a typical 6 stories and 4 bays (in both directions) space frame with square plan view. In frame groups A, B and C, accidental eccentricities of 0% and 5% (along both directions x and y separately) are considered. In addition, for group A, an accidental eccentricity of 10% as well as different steel grades in beams are considered. It is noted that 0%, 5% and 10% are typical values of accidental eccentricity proposed by codes, e.g. EC8 [10].

The frames were designed in accordance with Eurocodes 3 and 8 [10, 31] with the aid of the commercial software SAP 2000 [32]. In addition,  effects and capacity design considerations were taken into account based on EC8 [10] provisions. Both interior and exterior frames are MRFs. The steel was assumed to be of grade S235 for beams and S355 for columns.

The dead and live design loads were assumed to be G=6.5 kN/m2 and Q=2.0 kN/m2 resulting in a G + 0.3Q combination equal to 7.1 kN/m2. The above value of G does not include the structural self-weight, which is added during the analysis. The seismic action is represented by the Type 1 elastic design spectrum of EC8 [10] for soil class B and peak ground acceleration (PGA) equal to 0.24g, where g=9.81m/s2 is the acceleration of gravity. The behavior factor *q* was assumed equal to 6.5 for both x and y building directions. The design actions were assumed to be G + 0.3Q ± Ex ± 0.3Ey, G + 0.3Q ± Ey ± 0.3Ex and 1.35G + 1.5Q, where Ex and Ey denote the seismic actions along the x and y directions, respectively.

The structural steel sections were selected to be IPE for the beams and square hollow sections (SHS) for the columns. In all cases, sections were selected to be of class 1. Rigid full strength beam-to-column joints were assumed. Composite action in the beams is not considered. SHS columns are part of the lateral resisting system in both x and y directions~~,~~ and thus are subjected to bidirectional bending and axial load due to the gravity and seismic design situation. The final sections of the frames are shown in Table 1. In all design cases, the interstorey drift sensitivity coefficient *θ* of EC8 [10] governs the design. The designed frames of this work correspond to frames with low values of the stiffness parameter *ρ* andhigh values of the strength parameters *α*, as they defined in [20]. It is observed that the sections of the frames of groups A and B were found to be the same. It should be noticed that in Table 1, F stands for frame and C for column, while subscripts x and y denote the respective directions and e and i stand for the words exterior and interior, respectively. Furthermore, Table 2 provides the three first natural periods of the frames with accidental eccentricity e = 0%.

1. **Nonlinear modeling and seismic motions considered**

Nonlinear dynamic analyses were carried out by considering large displacements. The Ruaumoko 3D software (Carr [33]) was used for that purpose. The hysteretic behavior of beams and columns is represented by bilinear elastoplastic hinges at their ends. Strain hardening in the moment-rotation relation was assumed equal to 3% [34]. A plastic hinge is formed in beams when the internal bending moment becomes equal to the plastic moment. The use of a bilinear elastoplastic model is considered acceptable for dynamic nonlinear analysis that involves design level seismic events. However, at higher seismic intensities, the use of more refined models [35] able to capture stiffness and strength deterioration effects is recommended. Considering plastic hinge formation in columns, the effect of the axial force on the plastic moment strength is taken into account through the interaction formulae [31]

 (1)

where N, My and Mz are the axial force and the bending moments in the cross-section of the column, Npl,Rd is the axial plastic resistance, and Mpl,y,Rd and Mpl,z,Rd are the plastic moments of resistance.

Modeling of the frames is based on the center-line representation of their members, which ignores panel zone effects. A diaphragmatic action at the level of every floor is considered due to the presence of the composite slab. The mass and the mass moment of inertia of each floor is assumed to be concentrated at the center of mass of that floor. An inclusion of the accidental eccentricity is made by moving the centre of mass of each floor from its nominal location, in each direction, as defined in EC8 [10]. Damping is assumed to be of the Rayleigh type in conjunction with the tangent stiffness matrix [33]. The mass and stiffness coefficients of the resulting secant damping matrix are obtained by assuming 3% of critical damping in the first and *n*th mode, where *n* is the number of the stories of the frame. The abovementioned damping modelling assumptions are based on the recommendations of Carr [33, 36] to avoid potential unrealistic damping forces that result in underestimation of peak displacement demands and overestimation of peak strength demands.

The space frames are subjected to 42 pairs of far-fault earthquake ground motions taken from PEER [37] and listed in [38]. Fig. 2 depicts the elastic spectra for the two components of the ground motions. The selection of ground motions was based on the comparison between the spectral ordinates of each ground motion against the spectral ordinates of the design basis earthquake at the fundamental period of each frame. In that way the scaling factor of each ground motion can be controlled in order not to take excessive values at higher performance levels (see discussion in Section 4).

1. **Parametric analyses and creation of response databank**

For the creation of the seismic response databank, 12432 dynamic nonlinear analyses were conducted in the framework of incremental dynamic analysis (IDA) [39]. According to this approach, a structure is repeatedly subjected to a single ground motion by scaling the amplitude of the later. In that way, peak structural response quantity versus seismic intensity curve is constructed. Thus, for every pair of a frame and ground motion, one can determine the multiplication factors of the ground motions that drive a frame to the four levels of performance proposed by SEAOC [1] for MRFs, i.e. a) Occurrence of the first plastic hinge; (2) *IDR*max equal to 1.8%; (3) *IDR*max equal to 3.2 and (4) *IDR*max equal to 4%. The 42 pairs of the seismic motions in [38] are used twice for every frame by alternating the x and y directions; doubling in that way the number of results in the databank.

The determination of the appropriate scaling factor (SF) for every performance level is done by the bisection method. This procedure, for one of the space frames, a specific performance level, expressed in terms of *IDR*max, and a pair of ground motions involves the following steps:

1. Select a pair of ground motion.
2. Consider a lower bound of SF, e.g. SF1 = 0.1, for a pair of ground motion in order for the response of the frame to be in the linear range and an upper bound of SF, e.g. SF2 = 8, in order for the response of the frame to be in the inelastic range.
3. Multiply the ground motions with SFm = (SF1 + SF2)/2 and run dynamic non-linear analysis.
4. Check if the frame reaches the specific performance level at x or y direction of the plan view (Fig. 1). If the response is higher than the target, then SF1new = SFm and SF2new = SF2, else SF1new = SF1 and SF2new= SFm.
5. Repeat step 3 and 4 until a converge can be achieved, |SFm,new – SFm,old|≤0.01.

Although scaling of earthquake motions has been criticized, especially when the SF exceeds a certain large value, e.g. SF= 10 or 12 [40, 41], its use becomes practically necessary as it is difficult to find natural seismic records that can drive the structure to high performance levels. In this work, the highest value of SF is taken to be 8. That means in cases SF has to be higher than 8 in order to drive the structure to the desired performance level, the corresponding response results are not taken into account in the response databank. This exclusion, reduced the actual number of analysis used for the creation of the databank from 12432 to 9585. Once this factor is known, the appropriate ground motion that will drive the frame to the desired performance level (defined by its *IDR*max) becomes available and through a dynamic nonlinear analysis the maximum response of the frame can be determined. This response consists of the following quantities: the maximum roof displacement, *u*r,max, *IDR*max, the maximum local rotational ductility of the frame members, *μ*θ, the maximum roof displacement ductility, *μ*r, the behavior factor *q* and the fundamental periods of vibration. For a steel beam in flexure, the maximum rotational ductility, *μ*θ, is equal to 1+*θ*p /*θ*y, where *θ*p and *θ*y are the plastic rotation and the yield chord rotation at the ends of the member, respectively. The maximum roof displacement ductility, *μ*r, is defined as the ratio of the maximum roof displacement corresponding to a specific performance level over the maximum roof displacement at the appearance of the first plastic hinge. The behavior factor *q* is defined as the ratio of the SF driving the frame to a specific performance level (*IDR*max = 1.8%, 3.2% and 4%) over the SF corresponding to the appearance of the first plastic hinge. The above definitions of *μ*r and *q* have been used in previous studies (Karavasilis et al. [20, 42]). The definition of the proposed *q* factor does not comply with the traditional *q* factor used in FBD as it conveys information on deformation demands explicitly defined via the selection of the allowable *IDR* and *μ*θ values for the selected seismic design actions. In the literature there are different methodologies to determine the *q*-factor, such as FEMA P-695 [43] guidelines where a probabilistic basis methodology is presented for the derivation of *q*-factor in new building structures.

1. **Seismic response results**

This section presents a detailed assessment of the seismic structural results, which helps to identify the importance of the various parameters and establish the functional form of the empirical design equations used by the HFD.

Consider first the response results for group frames A, B and C with accidental eccentricity e=5%. Similar results are obtained for the case of zero accidental eccentricity. Due to space limitations, results are provided only for 6 and 12 storey frames. Results for 3, 9 and 15 storey frames can be found in Tzimas [38]. Figs 3 and 4 show the normalized median of the maximum values of lateral storey displacements and *IDR* along the height of the frames for the four performance levels considered. One can observe that for frames of group A with square plan view, the maximum response is the same along the x or y direction, while this is not the case with frames of groups B and C with rectangular plan view, where the maximum responses along the x and y directions are different and both are recorded in those figures. An inspection of Figs 3 and 4 and the remaining ones in [38], reveals that the response of the frames of the three groups is similar. In particular, the response of frames of groups A and B is very similar, indicating that the small difference in the number of bays along the x and y directions does not play an important role. In addition, *IDR* profiles of frames of groups A and B are almost the same, while displacements of frames of group A are between those of directions x and y of frames of group B and almost the same with those of frames of group B for zero accidental eccentricity. Frames of group C exhibit displacement and *IDR* profiles different than those of frames of groups A and B, especially for the case of *IDR*. The observed differences in *IDR* and displacement profiles are due to differences in stiffness distribution along the height of the frames as a result of design. Furthermore, these differences in displacement profiles increase for increasing values of the angular deformations of the frames and increasing values of their inelastic deformations. Finally, one can observe from the *IDR* profiles that for the buildings considered here the higher stories are those that determine the level of performance of the frames. The distribution of *IDR* along the height can become more uniform by increasing the beam sizes at the upper floors, but in this case the structures will become heavier.

Figs 5 and 6 show the normalized median of the maximum values of lateral storey displacements and *IDR* along the height of the frames of group A for the four performance levels considered here and three values of accidental eccentricity (e=0%, 5%, 10%). Due to space limitation only the case of 6 and 12 storey frames are shown in those figures. Results for 3, 9 and 15 storey frames can be found in Tzimas [38]. One can observe from Figs 5 and 6 and those in [38] that displacement and *IDR* profiles have the same shape with small differences with respect to the amplitudes. Increasing values of accidental eccentricity lead to a reduction of displacement demands for higher levels of performance and an increase of those demands during the appearance of first yielding. However, it should be noted that frames with accidental eccentricities require smaller seismic intensity levels to reach certain values of *IDR*max compared to frames with zero accidental eccentricity.

Fig. 7 shows the heightwise variation of local ductilities of beams around the perimeter of 6 and 12 storey frames of group A for three values of accidental eccentricity (e=0%, 5%, 10%). For frames with nonzero accidental eccentricity, the local ductilities are the maximum and minimum ones resulting in the flexible and stiff beams of the perimeter parallel to the direction defining the performance level due to the torsional deformation. As a final note concerning the space frames with square plan view, one can mention that the highest SF was 5.48 (for 15-storey frames with *IDR*=4% and e=0%) and that this factor increases with the number of stories and the *IDR*. Moreover, for the same number of stories and *IDR*, the highest SF value corresponds to zero eccentricity.

1. **Design equations for the HFD method**

The HFD method, as described in detail in Tzimas et al. [16] for the case of steel planar frames, aims to determine the maximum (target) design roof displacement *u*r,max(d) as the minimum of the maximum (target) roof displacements *u*r,max(*IDR*) and *u*r,max(*μ*) corresponding to non-structural and structural deformation, i.e.

 (2)

On the basis of this *u*r,max(d), the HFD method determines the design roof ductility *μ*r,d from the equation

 (3)

where *u*r,y is the maximum roof displacement at first yielding. An initial estimate of *u*r,y can be obtained by performing a strength-based design for the frequent earthquake. Using this ductility, one can finally compute the behavior factor *q* from an empirical equation of the functional form

 (4)

The above behavior factor, in view of Eqs (2)-(4), is a function of the non-structural and structural target deformations of the frame, and thus, its use in a response spectrum analysis ensures that deformation and hence damage can be controlled.

In this section, on the basis of the created response databank described in the previous section, one can develop by regression analysis specific empirical expressions for *u*r,max(*IDR*), *u*r,max(*μ*) and *q* in terms of basic geometrical attributes of the frames and the desired target deformations. For that purpose, non-linear regression analysis was employed in MATLAB [44].

Thus, it was found that the *u*r,max(*IDR*) can be expressed in the form

 (5)

where *IDR* stands for the desired target *IDR*, *H* denotes the height of the frame and the constants *b*1 and *b*2 are given in Table 3 in terms of the number of stories of the frame and the value of *IDR*. One can observe that, on the basis of the response results of the previous section, Eq. (5) does not depend on the number of bays and the value of accidental eccentricity. Fig. 8 provides graphically a comparison of the ratio *u*r,max,app/*u*r,max,exact as obtained here (for space frames) and in Karavasilis et al. [20] (for planar frames) by considering the *IDR* to be known. The word “app” stands for approximate and is associated with the value obtained by using the proposed empirical equations, like Eq. (5) in this case, while “exact” is associated with the value of the databank obtained by dynamic nonlinear analysis. In the results of Fig. 8a use was made of the databank including space frames with 0%, 5% and 10% accidental eccentricities, while in those of Fig. 8b, space frames with only 0% accidental eccentricity in conformity with the planar frames of [20]. Fig. 8 also provides the mean, median and standard deviation (Std) values of its two (a and b) parts. On the basis of Fig. 8, one can observe that Eq. (5) gives very good results, while Eq. (5.4) of Karavasilis et al. [20] gives results with larger standard deviation. Furthermore, Eq. (5) is much simpler to use than Eq. (5.4) of [20] since it does not involve the use of stiffness and strength parameters *ρ* and *α*. The reason is that the *ρ* values of the frames of this work correspond to the lower values used in [20], whereas the *a* values correspond to the higher values used in [20]. In general, the lower the *ρ* value is, the higher the value *a* becomes [20]. However, as was shown in [20] the *ρ* and *α* parameters affect the structural behaviour and thus further investigation is needed using frames with a wider range of *ρ* and *α* values, to include these parameters to the proposed expressions.

The *u*r,max(*μ*) can be determined, as in the case of planar frames [16], by the relation

 (6)

where *μ*r,θ is the maximum rotational ductility of the top storey, which can be expressed in terms of the maximum local rotational ductility *μ*θ. Indeed, with the use of the created response databank, one can find the relation

 , for *μ*θ≤ 4.68

(7)

 , for *μ*θ> 4.68

which is valid for all the space frames considered here, since the results of the created databank did not show any significant effect of accidental eccentricity and the plan view form on the *μ*r,θ versus *μ*θ relation. Fig. 9a provides graphically the distribution of the ratio *μ*r,θ,app/*μ*r,θ,exactassuming that *μ*θ is known and using Eq. (7) for the computation of *μ*r,θ,app. Fig. 9b provides the distribution of the same ratio as in Fig. 9a but with the *μ*r,θ,app obtained by Eq. (5.11) of Karavasilis et al. [20] for planar frames. It should be noticed that the use of (5.11) of [20] for planar frames was compared with space frames with rectangular plan view and zero accidental eccentricity. On the basis of Fig. 9, one can conclude that Eq. (7) gives very good results and certainly better than Eq. (5.11) of [20].

The behavior factor *q* is defined as

*q* = SFt / SFy  (8)

where SF stands for the scaling factor of the ground motion, t for target, and y for first yielding. On the basis of the results of the created response databank, one can observe that there is no influence of the number of stories and the number of bays but there is a small influence of the accidental eccentricity on the relation between q and the roof ductility *μ*r as it is shown in Fig. 10 for the case of space frames of the group A. Fig. 10 is based on the median values of the behavior factor *q* versus the roof ductility *μ*r for the four performance levels considered here. A similar trend holds true also for the space frames of groups B and C. In addition, no effect of the period of vibration and frequency content of the ground motion on the relationship between *q* and *μ*rwas identified. Thus, the expression providing *q* in terms of *μ*r and the accidental eccentricity e was found to be of the form

*q*=1+1.35∙(*μ*r – 1) e ≠ 0

(9)

*q*=1+1.30∙(*μ*r– 1) e = 0

In the above equations, as *μ*r one should use the design ductility *μ*r,d of Eq. (3). Eq. (9) fulfills the condition *q*=1 for *μ*r=1. Similar to previous studies on inelastic seismic response of planar steel MRFs (e.g. Karavasilis et al. [20]), Eq. (9) shows that the equal-displacement rule provides an overestimation of the maximum floor displacements.

Fig. 11 shows graphically Eq. (9) for space frames of groups A, B and C and e=5% and 0% together with the databank results, the relations of Karavasilis et al. [20] for planar frames and those based on the equal displacement rule (EC8 [10]). The present results for e=0% are very close to those corresponding only to the first branch of Karavasilis et al. [20]. Fig. 11 shows considerable variability in the *q* value at higher roof ductility levels which can be related to record-to-record variability. At this point one could state that since values of *q* greater than 8 are not practically realistic, any comparison of the present results with those of the second branch of [20] is practically meaningless. Fig. 12 provides the distribution of the values of the ratio *μ*r,app/*μ*r,exact for *q* assumed to be known for the cases of using Eq. (3) and EC8 [10] for determining *μ*r,app and databank results for determining *μ*r,exact for space frames with rectangular plan view. The same figure also provides median, mean and standard deviation values for the two abovementioned cases. It is observed that the present results are more accurate than the EC8 [10] ones which over-predict ductility demands. Indeed, from the databank results and Fig. 12 one can see that ductility values of the top floor and the intermediate floors are smaller than the behavior factor *q* provided in EC8 [10].

All the previously presented design equations have been derived on the assumption that the steel grade was S355 for the columns and S235 for the beams. To verify the effectiveness of the proposed equations, for the beams of space frames of group A with an accidental eccentricity 5%, S275 steel grade was also considered for the beams. It should be noted that although the beam cross-sections did not change, the space frames still satisfy the capacity design rules of EC8 [10]. Fig. 13 shows the curves of the median values of the behavior factor *q* versus the roof ductility *μ*r for the two steel grades in beams and for the four performance levels considered here. This figure indicates that changing the steel grade of the beams from S235 to S275 does not affect the results, which highlights the effectiveness of the proposed equations.

1. **Basic steps of the HFD seismic design method**

This section briefly describes the basic steps of the HFD seismic design method as applied to steel space MRF with rectangular plan view. These basic steps follow those of Tzimas et al. [16] for the case of the corresponding planar frames and utilize the design equations developed in the previous section. Thus, the HFD seismic design method for steel space MRFs with rectangular plan view consists of the following steps:

1. Definition of the basic frame attributes. This includes the number of stories, *n*s, number of bays, *n*b, storey heights, *h*, and bay widths, *b*, and limits on the depth of beams and columns due to architectural requirements.
2. Definition of the performance level. For example, IO under the frequently occurred earthquake (FOE; return period of 95 years), LS under the design basis earthquake (DBE; return period of 475 years) or CP under the maximum considered earthquake (MCE; return period of 2500 years). The earthquake intensity level is represented by the appropriate elastic response spectrum.
3. Definition of input parameters (performance metrics). Definition of limit values for the maximum interstorey drift ratio (*IDR*max) and maximum rotational ductility, *μθ*, for beams and columns. These limit values are selected on the basis of the performance level defined in Step (2) and can be obtained, e.g., from Table 4 taken from ASCE 41-13 [26].
4. Estimation of the input variables. The only input variables here are the yield roof displacement *u*r,y and the fundamental period, *T*, of the frame. Initial estimates of these variables may be obtained by designing the frame only for strength requirements under the FOE by assuming elastic behavior. The capacity design rules and the gravity load combination should be also taken into account in order to achieve an improved initial estimation of these input variables. An initial estimate of the period *T* can also be obtained from the simple empirical formula *T* = 0.116×*H*0.8, where *H* is the total height of the frame in m [45].
5. Transformation of performance metrics to the target roof displacement. Transformation of the *IDR*max to target maximum roof displacement, *u*r,max(*IDR*), is done by using the relation

 (10)

where *H* is the total height of the frame (in m) and *b*1 and *b*2 coefficients given by Table 3 in terms of the number of stories, *n*s, and the level of *IDR*max. Transformation of local rotational ductility, *μ*θ, to target roof displacement, *ur,max(μ)*, is done by using the relation

 (11)

where *μ*r,θ is the maximum rotational roof ductility given in terms of the maximum local ductility *μ*θ as

 for *μ*θ ≤ 4.68

 for *μ*θ > 4.68 (12)

Thus, the maximum design roof displacement *u*r,max(d) is obtained as

 (13)

1. Calculation of the behavior (or strength reduction) factor. Computation of the roof displacement ductility, *μ*r,d, as

 (14)

and then computation of the behavior factor, *q*, from the relation

*q*=1+1.35∙(*μ*r,d – 1) for e≠0

*q*=1+1.30∙(*μ*r,d– 1) for e=0 (15)

where e is the accidental eccentricity.

1. Design of the frame. Divide the ordinates of the elastic design spectrum by the *q* factor and design the frame on the basis of an elastic response spectrum analysis in conjunction with the capacity and ductile design rules of seismic codes (EC8 [10]). Use of the strength reduction (or behavior) factor q makes this design a strength-based one. Stiffness requirements are automatically satisfied through the dependence of *q* on deformation (step (6)).
2. Iterative design procedure. Iteration with respect to the input variable *u*r,y. The required number of iterations for convergence depends on the initial estimate of the input variable *u*r,y. A good initial estimate of *u*r,y can be easily obtained by designing the frame only for strength under the FOE by assuming elastic behavior.

As it has been noticed in Tzimas et al. [16], the iterative seismic design of a structure by using response spectrum analysis in conjunction with target damage levels, as it is the case here, may not be feasible. Target damage levels are here the target values of *IDR*max and *μ*θ, or equivalently the target maximum design roof displacement, *u*r,max(d). The reason for this lack of convergence is that every design spectrum has a maximum response demand, which may lead to a lower *u*r,max value than the desirable one. In such a case, one has to modify the target response value of *u*r,max(d) in order to achieve convergence of the iterative design procedure.

This modification can be accomplished with the aid of the corresponding single degree of freedom (SDOF) system to the multi degree of freedom (MDOF) system under design. This SDOF system is defined to have as its period the fundamental period of the MDOF system. For this SDOF system its maximum displacement is calculated with the aid of the displacement design spectrum obtained from the corresponding acceleration one of EC8 [10]. Then this maximum displacement is compared against the desirable maximum roof displacement *u*r,max(d) of the MDOF. If the displacement of the SDOF system is smaller than the *u*r,max(d), as shown in Fig. 14, one has to assume *u*r,max(d) equal to that of the SDOF and from the design equations of the HFD method determine the new *IDR*max and *μ*θ.

A modification factor to relate the spectral displacement of an equivalent SDOF system to the roof displacement of the building MDOF system is required. ASCE 41-13 [26] on the basis of the coefficient method for calculating the target displacement of a building, proposes an appropriate value of this factor equal to 1.0, 1.2, 1.3, 1.4 and 1.5 for buildings with 1, 2, 3, 5 and ≥10 number of floors, respectively, to multiply the resulting spectral displacement. Thus, one can use these multiplier factors to estimate more realistically the response demand of an MDOF system by using an equivalent SDOF system.

1. **Seismic design examples**

Two design examples are presented in order to illustrate the proposed HFD seismic design method and demonstrate its advantages over the FBD seismic design method of EC8 [10].

Two space frames were designed with the aid of the commercial computer program SAP 2000 [32] and use of the EC8 [10] and EC3 [31] provisions. The grade of steel was assumed S235 and S355 for beams and columns, respectively. The column sections were square hollow ones (SHS), while those of beams IPE. The dead and live design loads were assumed to be G=6.5 kN/m2 and Q=2kN/m2, respectively.The G + 0.3Q load combination results in 7.1 kN/m2. The above loads do not include structural self-weight, which is taken into account separately.

For the space frames considered herein, it was assumed that IO under the FOE, LS under the DBE and CP under the MCE are the appropriate performance levels for seismic design. The FOE, DBE and MCE are expressed through the Type 1 elastic design spectra of EC8 [10] (Fig. 15) for soil class B by assuming two different seismic zones which have peak ground accelerations under DBE (PGADBE) equal to 0.36g and 0.40g. The peak ground accelerations under the FOE and the MCE are equal to 0.3 x PGADBE and 1.5 x PGADBE, respectively. Table 4 provides limit values for performance metrics according to ASCE 41-13 [26].

*8.1 Example 1: Six storey steel space MRF*

Consider a space MRF with six stories of height 3.0m each and a square plan view with four bays of 6.0m each in both directions. It is assumed that this space frame has an accidental eccentricity of 5%. The peak ground acceleration for the DBE, PGADBE, is equal to 0.36g.

An initial elastic design for the FOE, yields the dimensions of Table 5. Table 5 also provides the first three natural periods of the designed MRF, with the fundamental period to be translational and equal to 1.23 sec. The roof displacement under the FOE is *u*r,y = 0.091 m, while *IDR*y= 0.69%, which fulfils the IO demands of Table 4. The above values serve as initial estimates for the input variables of the HFD. In addition, an assessment of the designed structure for the LS and the CP performance levels is made. The *q* factor of the space MRF under the DBE is easily obtained as (PGADBE/PGAFOE) = 3.33. This value is used in order to estimate the response of the space MRF under the DBE, i.e., by employing Eq. (15) *μ*r,d = 1+(3.33-1)/1.35 = 2.73, by employing Eq. (14) *u*r,max = 2.73×0.091 = 0.248 m, by employing Eq. (12) *μ*θ = 1+(2.73-1)/0.81 = 3.14 and by employing Eq. (10) *IDR*max = (0.248/(0.93×3\*6))1/1.11 = 2.25%. The *q* factor of the space MRF under the MCE is easily obtained as (PGAMCE/PGADBE)×*q*DBE = 5. This value is used in order to estimate the response of the space MRF under the MCE, i.e., by employing Eq. (15) *μ*r,d = 1+(5-1)/1.35 = 3.96, by employing Eq. (14) *u*r,max = 3.96×0.091 = 0.360 m, by employing Eq. (12) *μθ* = 1+(3.96-1)/0.81 = 4.65 and by employing Eq. (10) *IDR*max = (0.360/(0.93×3\*6))1/1.11 = 3.15 %. According to the above results, using the limit values of Table 4, the space MRF which satisfies the IO performance level satisfies also the LS and CP performance levels.

The target values of the *IDR*max and *μ*θ for the LS performance level are equal to 2.5% and 9, respectively (Table 4). By employing Eq. (10), the target roof displacement *u*r,max(*IDR*) = 6×3×0.93×0.0251.11 = 0.279 m and therefore, the target roof displacement ductility *μ*r,*IDR* becomes equal to 0.279/0.091 = 3.07. Employing Eq. (12), the target roof rotational ductility *μ*r,θ is calculated as 2.58+0.38×(9-1) = 5.62. Thus, the design roof ductility *μ*r,d is equal to the min (*μ*r,*IDR*, *μ*r,θ) = min(3.07, 5.62) = 3.07 and therefore, drift controls the LS performance level design. According to EC8 [10] displacement design spectrum (Fig. 15), for an SDOF system which has the same period as the space MRF, the maximum displacement would be equal to 0.165×1.42 = 0.234 m for the case of the DBE, where the above multiplier 1.42 is proposed by ASCE 41-13 [26] for six storey buildings. This means that there is a need to revise the target values of the *IDR*max and *μ*θ, because the demand displacement of 0.234 m as obtained by the displacement design spectrum is smaller than the *u*r,max(*IDR*) = 0.279 m. By employing Eqs. (14, 12 and 10) and by adopting as target displacement the 0.234 m, the demand values of *IDR*max and *μ*θ of this structure cannot be much higher than 2.13% and 2.94, respectively. These values fulfil the demands of Table 4. In addition, the response spectrum analysis/design under the FOE spectrum fulfils the demand of Table 4. The use of *IDR*max = 2.13% as design input will lead to heavier structure compared to the designed structure under the FOE spectrum (estimated *IDR*max = 2.25% under the DBE). Therefore, the IO performance level controls the design of the space MRF and there is no need for a new design in the case of DBE.

The target values of the *IDR*max and *μ*θ for the CP performance level are equal to 5% and 11, respectively (Table 4). By employing Eq. (10), the target roof displacement *u*r,max(*IDR*) = 6×3×1.51×0.051.25 = 0.643 m and therefore, the target roof displacement ductility *μ*r,*IDR* becomes equal to 0.643/0.091 = 7.07. Employing Eq. (12), the target roof rotational ductility *μ*r,θ is calculated as 2.58+0.38×(11-1) = 6.38. Thus, the design roof ductility *μ*r,d is equal to the min (*μ*r,*IDR*, *μ*r,θ) = min(7.07, 6.38) = 6.38 and therefore, local ductility controls the CP performance level design and *u*r,max(d) = 6.38×0.091 = 0.581 m. According to EC8 [10] displacement design spectrum (Fig. 15), for an SDOF system which has the same period as the space MRF, the maximum displacement would be equal to 0.248×1.42 = 0.352 m for the case of the MCE, where the above multiplier 1.42 is proposed by ASCE 41-13 [26] for six storey buildings. This means that there is a need to revise the target values of the *IDR*max and *μ*θ, because the demand displacement of 0.352 m as obtained by the displacement design spectrum is smaller than the *u*r,max(d) = 0.581 m. By employing Eqs. (14, 12 and 10) and by adopting as target displacement the 0.352 m, the demand values of *IDR*max and *μ*θ of this structure cannot be much higher than 3.08% and 4.54, respectively. These values fulfil the demands of Table 4. In addition, the response spectrum analysis/design under the FOE spectrum fulfils the demand of Table 4. The use of *IDR*max = 3.08% as design input will lead to a heavier structure compared to the designed structure under the FOE spectrum (estimated *IDR*max = 3.15% under the MCE). Therefore, the IO performance level controls the design of the space MRF and there is no need for a new design in the case of MCE.

The required iterations for the design of the six storey space frame based on the HFD method were few. The initial estimation of the roof yield displacement (step 4 in Section 7) can be considered as a key point in the design under the DBE and MCE. In addition, this example shows how to change the initial target response (*IDR*max and *μ*θ) based on the maximum response demand of the design spectrum to achieve convergence of the iterative design procedure, as described in Section 7.

Using SAP 2000 in conjunction with EC8 [10] and EC3 [31] the space MRF of this example was also designed by the FBD method. The behavior factor *q* = 6.5 was chosen in accordance with EC8 [10]. Table 5 provides the dimensions of the space frame and its three first natural periods. According to FBD, the LS performance level controls the design. The space frame remains elastic under the IO earthquake and experiences *u*r,y = 0.088 m, *IDR*max = 0.66% and *μ*θ = 1. Under the DBE, the space frame will experience *u*r,max = 0.088×PGADBE/PGAFOE = 0.088/0.3 = 0.293 m and *IDR*max = 0.66%×PGADBE/PGAFOE = 0.66%/0.3 = 2.20%. Under the MCE, the space frame will experience *u*r,max = 0.293×PGAMCE/PGADBE = 0.293×1.5 = 0.440 m and *IDR*max = 2.20%×PGAMCE/PGADBE = 2.20%×1.5 = 3.30%.

*8.2 Example 2: Nine storey steel space MRF*

Consider a space MRF with nine stories of height 3.0m each and a rectangular plan view with four bays of 6.0m and 8.0 m in each direction, respectively. It is assumed that this space frame has an accidental eccentricity of 5%. The peak ground acceleration for the DBE, PGADBE, is equal to 0.40g.

An initial elastic design for the FOE, yields the dimensions of Table 6. Table 6 also provides the first three natural periods of the designed MRF, with the fundamental period to be translational and equal to 1.65 sec. The roof displacement under the FOE is *u*r,y = 0.130 m, while *IDR*y= 0.69%, which fulfils the IO demands of Table 4. The above values serve as initial estimates for the input variables of the HFD. In addition, an assessment of the designed structure for the LS and the CP performance levels is made. The *q* factor of the space MRF under the DBE is easily obtained as (PGADBE/PGAFOE) = 3.33. This value is used in order to estimate the response of the space MRF under the DBE, i.e., by employing Eq. (15) *μ*r,d = 1+(3.33-1)/1.35 = 2.73, by employing Eq. (14) *u*r,max = 2.73×0.130 = 0.355 m, by employing Eq. (12) *μ*θ = 1+(2.73-1)/0.81 = 3.14 and by employing Eq. (10) *IDR*max = (0.355/(2.07×3\*9))1/1.37 = 2.49%. The *q* factor of the space MRF under the MCE is easily obtained as (PGAMCE/PGADBE)×*q*DBE = 5. This value is used in order to estimate the response of the space MRF under the MCE, i.e., by employing Eq. (15) *μ*r,d = 1+(5-1)/1.35 = 3.96, by employing Eq. (14) *u*r,max = 3.96×0.130 = 0.515 m, by employing Eq. (12) *μ*θ = 1+(3.96-1)/0.81 = 4.65 and by employing Eq. (10) *IDR*max = (0.515/(2.38×3\*9))1/1.41 = 3.26 %. According to the above results, using the limit values of Table 4, the space MRF which satisfies the IO performance level satisfies also the LS and CP performance levels.

The target values of the *IDR*max and *μ*θ for the LS performance level are equal to 2.5% and 9, respectively (Table 4). By employing Eq. (10), the target roof displacement *u*r,max(*IDR*) = 9×3×2.07×0.0251.37 = 0.356 m and therefore, the target roof displacement ductility *μ*r,*IDR* becomes equal to 0.356/0.130 = 2.74. Employing Eq. (12), the target roof rotational ductility *μ*r,θ is calculated as 2.58+0.38×(9-1) = 5.62. Thus, the design roof ductility *μ*r,d is equal to the min (*μ*r,*IDR*, *μ*r,θ) = min(2.74, 5.62) = 2.74 and therefore, drift controls the LS performance level design. According to EC8 [10] displacement design spectrum, for an SDOF system which has the same period as the space MRF, the maximum displacement would be equal to 0.246×1.48 = 0.364 m for the case of the DBE, where the above multiplier 1.48 is proposed by ASCE 41-13 [26] for nine storey buildings. This means that there is no need to revise the target values of the *IDR*max and *μ*θ, because the demand displacement of 0.364 m as obtained by the displacement design spectrum is higher than the *u*r,max(*IDR*) = 0.356 m. The required behavior factor *q* is calculated equal to 3.35 based on Eq. (15). By using this factor, the DBE design spectrum is reduced and the space frame under the DBE is designed with spectrum analysis. This *q* factor does not change the sections of the designed space MRF for the IO (Table 6). This design fulfils the demands of Table 4. Therefore, both the IO and the LS performance levels leads to the same design.

The target values of the *IDR*max and *μ*θ for the CP performance level are equal to 5% and 11, respectively (Table 4). By employing Eq. (10), the target roof displacement *u*r,max(*IDR*) = 9×3×2.38×0.051.41 = 0.941 m and therefore, the target roof displacement ductility *μ*r,*IDR* becomes equal to 0.941/0.130 = 7.24. Employing Eq. (12), the target roof rotational ductility *μ*r,θ is calculated as 2.58+0.38×(11-1) = 6.38. Thus, the design roof ductility *μ*r,d is equal to the min (*μ*r,*IDR* , *μ*r,θ) = min(7.24, 6.38) = 6.38 and therefore, local ductility controls the CP performance level design and *u*r,max(d) = 6.38×0.130 = 0.829 m. According to EC8 [10] displacement design, for an SDOF system which has the same period as the space MRF, the maximum displacement would be equal to 0.369×1.48 = 0.546 m for the case of the MCE, where the above multiplier 1.48 is proposed by ASCE 41-13 [26] for nine storey buildings. This means that there is a need to revise the target values of the *IDR*max and *μ*θ, because the demand displacement of 0.546 m as obtained by the displacement design spectrum is smaller than the *u*r,max(d) = 0.829 m. By employing Eqs. (14, 12 and 10) and by adopting as target displacement the 0.546 m, the demand values of *IDR*max and *μ*θ of this structure cannot be much higher than 3.40% and 5.26, respectively. These values fulfil the demands of Table 4. In addition, the response spectrum analysis/design under the FOE spectrum fulfils the demand of Table 4. The use of *IDR*max = 3.40% as design input will lead to more flexible structure compared to the designed structure under the FOE spectrum (estimated *IDR*max = 3.26% under the MCE), thus the demands of Table 4 for the IO will be violated. Therefore, the IO performance level controls the design of the space MRF and there is no need for a new design in the case of MCE.

The required iterations for the design of the nine storey space frame based on the HFD method were few. The initial estimation of the roof yield displacement (step 4 in Section 7) can be considered as a key point in the design under the DBE and MCE. In addition, this example there was no need to change the initial target response (*IDR*max and *μ*θ) based on the maximum response demand of the design spectrum.

Using SAP 2000 in conjunction with EC8 [10] and EC3 [31] the space MRF of this example was also designed by the FBD method. The behavior factor *q* = 6.5 was chosen in accordance with EC 8 [10]. Table 6 provides the dimensions of the space frame and its three first natural periods. According to FBD, the LS performance level controls the design. The space frame remains elastic under the IO earthquake and experiences *u*r,y = 0.120 m, *IDRmax* = 0.69% and *μ*θ = 1. Under the DBE, the space frame will experience *ur,max* = 0.120×PGADBE/PGAFOE = 0.120/0.3 = 0.400 m and *IDR*max = 0.69%×PGADBE/PGAFOE = 0.69%/0.3 = 2.30%. Under the MCE, the space frame will experience *u*r,max = 0.400×PGAMCE/PGADBE = 0.400×1.5 = 0.600 m and *IDR*max = 2.30%×PGAMCE/PGADBE = 2.30%×1.5 = 3.45%.

*8.3 Evaluation of the designs through nonlinear dynamic analyses*

In order to conduct a more detailed comparison of the two design methods (HFD and FBD), the two designed structures by the above methods were seismically analysed through dynamic nonlinear analyses performed by the Ruaumoko program [33]. Five pairs of artificial accelerograms compatible with the elastic design spectrum of EC 8 [10] with soil class B and PGA equal to 0.36g and 0.40g, obtained by using a specialized software (Karabalis et al. [46]), were used for the analyses (Fig. 16).

Table 7 provides the response results of the dynamic nonlinear analyses together with those obtained by the two design methods. One can observe that the HFD method is capable of controlling better the damage compared to the FBD method. The reason is that the former method uses a *q* factor which depends on deformation (and hence damage), while the latter method uses a constant value for *q*. Furthermore, the FBD overpredicts the *IDR*max and the *u*r,max in contrast to the HFD, which provides a value for them very close to the ones from dynamic nonlinear analysis. For the six storey space MRF the HFD method predicts well the maximum *μ*θ, whereas it underestimates it for the nine storey space frame. Concerning now the total steel weight of the two designed six storey space MRFs, the two methods lead to weights very close to each other, i.e., 1721 kN and 1690 kN, for the HFD and FBD designed structures, respectively. As far as the total steel weight of the two designed nine storey space MRFs, the HFD method leads to a lighter structure, with total steel weight 3588 kN compared to 3960 kN of the FBD designed structure.

1. **Conclusions**

On the basis of the results presented in this paper, the following conclusions are drawn:

1. For a given value of interstorey drift ratio (*IDR*), the maximum local ductility *μ*θ is the same for all the frames. Moreover, frames with accidental eccentricities require certain values of *IDR* at smaller seismic intensities compared to frames with zero accidental eccentricity.
2. The relations between maximum roof displacement *u*r - *IDR* and roof displacement ductility *μ*r - *μ*θ do not depend on the value of accidental eccentricity. Nonzero accidental eccentricity slightly affects the behavior factor *q* – *μ*r relation.
3. Based on the created database, new empirical expressions for the hybrid force/displacement (HFD) seismic design method are proposed, where torsional effects are taken into consideration by means of accidental eccentricity.
4. The proposed equations applied to frames with low values of the stiffness parameter *ρ* andhigh values of the strength parameters *α*, as they defined in Karavasilis et al. [20].
5. The advantages of the HFD method over the force-based design (FBD) method include its ability to identify the performance level which controls the design and to provide more accurate predictions of response quantities like the *IDR*, *μ*θ and *u*r. Resulting total weights of the designed frames by HFD and FBD methods were found to be very close.
6. Empirical expressions of the HFD method derived for planar steel frames can be also used for space frames with small eccentricities to estimate the *u*r and correlate the *q* factor with the *μ*r. However, for medium to high eccentricities and relative low values of *ρ*, only the present relations for space frames are recommended. In general, expressions to estimate the maximum local ductility have high dispersion.
7. The use of the proposed equations of the HFD method is restricted to frames which are compatible with the modeling assumptions (i.e. bilinear model with rigid connections, disregard of panel zone effects). The accuracy and the extension of the proposed equations can be further improved by including panel zone and deterioration effects.
8. The inclusion of peak floor acceleration as an additional performance metric of the HFD method can further control non-structural damage.

**Acknowledgements**

The first author is grateful for the support provided to him through the “K. Karatheodoris” research program of the University of Patras, Greece. Thanks are also due to Mrs M. Dimitriadi for her technical assistance.

**References**

[1] Structural Engineers Association of California, SEAOC. Recommended Lateral Force Requirements and Commentary. 7th ed., California, Sacramento, CA; 1999.

[2] Bozorgnia Y, Bertero, VV. Earthquake Engineering: From Engineering Seismology to Performance-Based Engineering. CRC Press, Boca Raton, FL, USA; 2004.

[3] Federal Emergency Management Agency, FEMA. Next-generation performance-based seismic design guidelines, Program plan for new and existing buildings. Report FEMA-445, Washington DC; 2006.

[4] Structural Engineers Association of California, SEAOC. Vision 2000 – A framework for performance based earthquake engineering. Vol. 1, January 1995.

[5] Applied Technology Council, ATC. Seismic Evaluation and Retrofit of Concrete Buildings. Report No. ATC-40, Redwood City, CA; 1996.

[6] Federal Emergency Management Agency, FEMA. NEHRP guidelines for the seismic rehabilitation of buildings. Report FEMA 273. Washington DC; 1997.

[7] Federal Emergency Management Agency, FEMA. Prestandard and commentary for the seismic rehabilitation of buildings. Report No*.* FEMA-356, Washington, DC; 2000.

[8] Krawinkler H, Zareian F, Medina RA, Ibarra LS. Decision support for conceptual performance-based design. Earthquake Engineering and Structural Dynamics 2005; 35(1): 115–133.

[9] Vamvatsicos D, Kazantzi AK, Aschheim MA. Performance-based seismic design: avant-garde and code-compatible approaches. ASCE-ASME Journal of Risk and Uncertainty in Engineering Systems, Part A: Civil Engineering 2016; 2(2): C4015008.

[10] Eurocode 8, EC8. Design of Structures for Earthquake Resistance, Part 1: General Rules, Seismic Actions and Rules for Buildings, European Standard EN 1998-1, Stage 51 Draft. European Committee for Standardization (CEN), Brussels; 2004.

[11] Uniform Building Code, UBC. International Conference of Building Officials. Whittier, CA, USA; 1997.

[12] BCJ. Structural Provisions for Building Structures, Building Center of Japan. Tokyo, Japan; 1997.

[13] Priestley MJN, Calvi GM, Kowalsky MJ. Direct Displacement-Based Design. IUSS Press, Pavia, Italy; 2007.

[14] Chopra AK, Goel RK. Direct displacement-based design: use of inelastic vs elastic design spectra. Earthquake Spectra 2001; 17 (1): 47-63.

[15] Sullivan TJ, Calvi GM, Priestley MJN, Kowalsky MJ. The limitations and performances of different displacement based design methods. Journal of Earthquake Engineering 2003; 7(1): 201-241.

[16] Tzimas AS, Karavasilis TL, Bazeos N, Beskos DE. A hybrid force/displacement seismic design method for steel building frames. Engineering Structures 2013; 56: 1452-1463.

[17] Karavasilis TL, Bazeos, N, Beskos, DE. A hybrid force/displacement seismic design method for plane steel frames. Proceedings of STESSA Conference,Yokohama, Japan, 14-17 August, 2006, Taylor & Francis, pp. 39-44.

[18] Karavasilis TL, Bazeos, N, Beskos, DE. A hybrid force/displacement seismic design method for plane steel frames.In: Proceedings of 1st European Conference on Earthquake Engineering and Seismology (1st ECEES), Geneva, Switzerland, 3-8 September, 2006, Paper No 1013.

[19] Karavasilis TL, Bazeos, N, Beskos, DE. Estimation of seismic drift and ductility demands in plane regular X-braced steel frames. Earthquake Engineering and Structural Dynamics 2007; 36(15): 2273-2289.

[20] Karavasilis TL, Bazeos, N, Beskos, DE. Drift and ductility estimates in regular steel MRF subjected to ordinary ground motions: A design-oriented approach. Earthquake Spectra 2008; 24(2): 431-451.

[21] Karavasilis TL, Bazeos, N, Beskos, DE. Seismic response of plane steel MRF with setbacks: estimation of inelastic deformation demands. Journal of Constructional Steel Research 2008; 64(6): 644-654.

[22] Karavasilis TL, Bazeos, N, Beskos, DE. Estimation of seismic inelastic deformation demands in plane steel MRF with vertical mass irregularities. Engineering Structures 2009; 30(11): 3265-3275.

[23] Stamatopoulos H, Bazeos N. Seismic inelastic response and ductility estimation of steel planar chevron-braced frames. In: Boudouvis AG, Stavroulakis GE, editors. Proceedings of 7th GRACM Conference on Computational Mechanics, Athens, Greece, 30 June-2 July; 2011.

[24] Bazeos N. Comparison of three seismic design methods for plane steel frames. Soil Dynamics and Earthquake Engineering 2009; 29(3): 553-562.

[25] Karavasilis TL, Makris N, Bazeos N, Beskos DE. Dimensional response analysis of multi-storey regular steel MRF subjected to pulse-like earthquake ground motions. Journal of Structural Engineering (ASCE) 2010; 136(8): 921-932.

[26] ASCE/SEI Standard 41-13. Seismic evaluation of retrofit of existing buildings. American Society of Civil Engineers, Reston, Virginia, USA; 2014.

[27] Vasilopoulos AA, DE Beskos. Seismic design of space steel frames using advanced methods of analysis. Soil Dynamics and Earthquake Engineering, 2009; 29(1): 194-218.

[28] Kyrkos MT, Anagnostopoulos, SA. An assessment of code designed asymmetric steel buildings under strong earthquake excitations. Earthquakes and Structures, 2011; 2(2): 109-126.

[29] Kyrkos MT, Anagnostopoulos, SA. Improved earthquake resistant design on torsionally stiff asymmetric steel buildings. Earthquakes and Structures, 2011; 2(2): 127-147.

[30] Anagnostopoulos SA, Kyrkos MT, Stathopoulos KG. Earthquake induced torsion in buildings: Critical review and state of the art. Earthquakes and Structures 2015; 8(2): 305-377.

[31] Eurocode 3, EC3. Design of Steel Structures, Part 1.1: General Rules for Buildings, European Prestandard ENV 1993-1-1. European Committee for Standardization (CEN), Brussels; 2005.

[32] SAP2000. Static and Dynamic Finite Element Analysis of Structures. Version 11.0.4, Computers and Structures inc., Berkeley, California; 2007.

[33] Carr AJ. Ruaumoko-3D - A Program for Inelastic Dynamic Analysis. Technical Report, Department of Civil Engineering, University of Canterbury, Christchurch, New Zealand; 2005.

[34] Gupta A, Krawinkler H. Seismic Demands for Performance Evaluation of Steel Moment Resisting Frame Structures. Report No 132, John A Blume Earthquake Engineering Center, Department of Civil Engineering, Stanford University, Stanford, CA, USA; 1999.

[35] Lignos DG, Krawinkler HK. Deterioration modeling of steel components in support of collapse prediction of steel moment frames under earthquake loading. Journal of Structural Engineering 2011; 137(11): 1291–1302.

[36] Carr AJ. Damping models for time-history structural analyses. Proceedings Asia Pacific Vibration Conference ’97, Gwangju, Korea, November 1997, pp. 42-48.

[37] PEER, Pacific Earthquake Engineering Research Centre, Strong Ground Motion Database, 2009, <http://peer.berkeley.edu/>.

[38] Tzimas AS. A New Hybrid Force/Displacement Method for Seismic Design of Space Steel Structures. Ph.D. Thesis, Department of Civil Engineering, University of Patras, Patras, Greece 2013 (in Greek).

[39] Vamvatsikos D, Cornell CA. Incremental dynamic analysis. Earthquake Engineering and Structural Dynamics 2002; 31: 491-514.

[40] Hancock J. The influence of duration and the selection and scaling of accelerograms in engineering design and assessment. Ph.D. Thesis, Department of Civil and Environmental Engineering, Imperial College, University of London, 2006.

[41] De Luca F, Iervolino I, Cosenza E. Unscaled, scaled, adjusted and artificial spectral matching accelerograms: displacement-and energy-based assessment. Proceedings of 13th Congress of the Italian National Association of Earthquake Engineering (ANIDIS), Bologna, Italy, June 28-July 2, 2009, 10 pages.

[42] Karavasilis TL, Bazeos N, Beskos DE. Behavior factor for performance-based seismic design of plane steel moment resisting frames. Journal of Earthquake Engineering 2007; 11: 531–559.

[43] Federal Emergency Management Agency, FEMA. Quantification of building seismic performance factors. ATC-63 Project, Report FEMA-P695*.* Applied Technology Council. California. USA; 2008.

[44] MATLAB. The Language of Technical Computing. Version 2009a, The Mathworks Inc., Natick, MA, USA; 2009.

[45] Goel RK, Chopra, AK. Period formulas for moment-resisting frame buildings. Journal of Structural Engineering (ASCE) 1997; 123(11): 1454-1461.

[46] Karabalis DL, Cokkinides GJ, Rizzos DC. Seismic Record Processing Program. Version 1.03. Report of the College of Engineering, University of South Carolina, Columbia, SC, USA; 1993.