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A. A. Vasilopoulos[a], G.S. Kamaris[b,*]

[a]Decentralized Administration Peloponnisos-Western Greece & Ionion, Directorate of Technical Inspection, Athinon 105, GR-26110 Rio-Patras, Greece
[b]Department of Civil Engineering, Liverpool John Moores University, Liverpool, L3 3AF, UK

Abstract: A rational and efficient seismic design method for regular space steel frames using inelastic (pushover) analysis (PA) is presented. This method employs an advanced static finite element analysis that takes into account geometrical and material non-linearities and member and frame imperfections. Resistances (strengths) are computed according to Eurocodes 3. The PA is employed with multimodal lateral loads along the height of the building combining the first few modes. The design starts with assumed member sections, continues with deformation and damage check at three performance levels with the aid of PAs and ends with the adjustment of member sizes. Thus, it can sufficiently capture the limit states of displacements, strength, stability and damage of the structure and its individual members so that separate member capacity checks through the interaction equations of Eurocode 3 or the use of the behavior factor q suggested in Eurocode 8 are not required. One numerical example dealing with the seismic design of a one bay in both horizontal directions and three storey steel moment resisting frame is presented to illustrate the method and demonstrate its advantages.

Keywords: Seismic design; Space steel frames; Finite element method; Inelastic static (pushover) analysis; Eurocode 8; Eurocode 3

1. Introduction

The advantages of using advanced methods of analysis in the seismic design of plane and space steel moment resisting frames (MRFs) have been demonstrated by Vasilopoulos and Beskos [1,2]. It was shown in those works that seismic design of steel MRFs can be rationally and efficiently accomplished with the aid of an advanced dynamic finite element method of analysis taking into account material and geometrical nonlinearities. In this work, dynamic nonlinear analysis is replaced by static nonlinear (pushover) analysis (PA) in an effort to simplify the seismic design process. According to the PA, the structure is subjected to gravity loads and to monotonically increasing static lateral loads with a triangular or uniform distribution along its height. This method has emerged as a very simple and economic alternative of a complete dynamic inelastic analysis, which requires a group of suitable seismic input records, more complicated software and higher computational cost [3]. This is the case, provided that the accuracy of PA in comparison with the exact dynamic analysis is acceptable and its results are

*Corresponding author.
E-mail: G.Kamaris@ljmu.ac.uk

reliable. There are various factors affecting the accuracy of PA, such as the effect of higher modes or torsional effects in a three dimensional (3-D) context.

Higher mode effects can be taken into account at an increased computational cost by the modal PA [4], which requires a series of PAs with fixed load distributions analogous to the first few mode shapes of vibration. In order to account for the continuous mode changes, due to inelastic deformations, adaptive pushover procedures have been developed in which the load pattern changes during the deformation [5]. However, these procedures are computationally very costly and do not provide significant advantages over the conventional (non-adaptive) PA [6].

In this work, the PA is employed for the performance-based seismic design of regular space steel MRFs. Thus, dynamic non-linear analysis used for seismic design of regular space steel frames in [2], is replaced here by PA in an effort to simplify the whole design process at the expense of losing a degree of accuracy. The PA used here employs a fixed improved multimodal loading distribution [7] believing that simplicity with reasonable accuracy is better than complication with higher accuracy as in modal PA and especially adaptive techniques.

Treatment of PA mainly as a performance evaluation procedure has been reported by many authors as it has been already discussed. However, complete seismic design methodologies using PA as a performance evaluation tool have been reported only by Gioncu and Mazzolani [8] and Hasan et al [9] in connection with plane steel frames. Thus, the present work comes to complement the last two works on plane steel frames by extending them to the case of space steel frames. This extension, however, is by no means straightforward even for the case of regular space frames due to the fact that lateral loads now act along two orthogonal directions simultaneously with two different magnitude combinations.

Summarizing, the present work develops a seismic design procedure for regular space steel MRFs, which is accomplished with the aid of advanced methods of pushover analysis in conjunction with three limit states of performance, which are related to specific strength values, displacements, rotations, damage indices (DIs) and plastic hinge patterns. In other words, an effort is made to develop a design procedure that captures, in a practical and simple way all aspects of structural behavior by appropriately modifying the computer program DRAIN-3DX [10].

Thus, the main limitations of EC3 [11] and EC8 [12], namely the neglect of the interaction of strength and stability between the members and the whole structure, the use of a two stage design procedure consisting of an elastic first order global analysis to determine internal member design forces and a strength check of every member with the aid of the inelasticity based interaction equations, and finally the employment of the approximate behavior factor *q*, are eliminated with the use of the proposed method of seismic design.

2. Seismic Analysis and Design Procedure

A seismic design procedure for regular space steel structures based on PA and developed in a format appropriate for incorporating EC3[11] and EC8[12] design codes is proposed herein. The basic features of the proposed method are the following:

i. Selection of the appropriate lateral force distribution in order for the higher modal effects of the building and the diaphragm torsional moments along the building height to be included.

ii. Non-linear geometrical effects, member and frame imperfections, member strength and residual stresses through a modified version of the software DRAIN-3DX [10].

iii. The establishment of performance design criteria related to the plastic hinge pattern, the total, storey and member seismic DIs, the serviceability checks (drifts and/or rotations) and the total system strength (base shear)defined for three performance levels.

In the following, the most important aspects of the proposed seismic design method for steel structures are briefly described:

The improved multimodal distribution of loading proposed in [7] is adopted herein because in comparison with the inverted triangular or the uniform load distribution, it results in more accurate response magnitudes. In this load distribution [7], lateral forces F_d along the height of the building can be expressed as

$$F_d = \sqrt{(M^*_{1,d}S_{1,d})^2 + (M^*_{2,d}S_{2,d})^2 + (M^*_{3,d}S_{3,d})^2 + ... + (M^*_{n,d}S_{n,d})^2} \quad (1)$$

where the weight M_i^* is the effective mass for the i_{th} mode given by

$$M_{i,d}^* = \left(\sum_{j=1}^{n} m_j^* \phi_{ji,d} \right)^2 \Big/ \sum_{j=1}^{n} m_j^* \phi_{ji,d}^2 \tag{2}$$

Where m_j^* is the modal mass and $S_{i,d}$ is

$$S_{i,d} = [\bar{M}]\{\phi_{i,d}\}, \qquad i=1,2,3....n \tag{3}$$

with $[\bar{M}]$ being the generalized mass, $\{\phi_i\}$ the i_h mode vector, n the number of modes considered and the d index defining the x and z directions of a diaphragm level. After the calculation of the lateral force distributions with respect to the two horizontal directions according to equation (1), the diaphragm torsional moments along the building height are calculated as $M_T = F_d*0.05L_i$, where L_i is the storey width perpendicular to the exposed direction of the lateral force distribution.

A simple and efficient approach for representing inelasticity in steel frames is the refined elastic-plastic hinge method [1,13]. In this approach, the lateral torsional buckling effect is taken into account in an approximate manner in conformity with EC3 [11] and the aid of [1,2]. Gradual yielding effects due to residual stresses are taken into account by a tangent modulus expression depending on the axial force of the member [1,2,13].

Geometric nonlinearities in usual frame analysis include P-δ and P-Δ effects as well as imperfection effects. DRAIN-3DX [10] accounts for the P-δ and P-Δ effects. Imperfections are taken into account by the simple, yet effective, approach described in Chen and Kim [13], which reduces the tangent modulus by a reduction factor of 0.85.

The quantification of damage is accomplished by the use of the DI, which provides a comprehensive design measure in conjunction with the performance levels of the structure. The DI can be determined at member level, I_{dm}, storey level, I_{ds}, or global structural level, I_{dg} [1,2] and they are different for different performance levels. All the necessary modifications, concerning the damage estimation of the abovementioned DIs, have been implemented in [10] for PA.

Three performance levels are considered here: the immediate occupancy (IO), the life safety (LS) and the collapse prevention (CP) corresponding to the frequent occurred earthquake (FOE), the design basis earthquake (DBE) and the maximum considered earthquake (MCE), respectively.

3. Seismic Design Procedure via Advanced Analysis

On the basis of the preceding discussion, one can establish a seismic design procedure for regular space steel MRFs including the following steps:

Step1: Types of loads and design load combinations. The loads are gravity loads (dead and live) and static lateral seismic loads. The design load combinations are described in EC3 [11].

Step2: Preliminary member sizing. This depends on the experience of the designer or some simplified analysis. For example, beam sections are usually selected by assuming that beams are simply supported and subjected to gravity loads only and column sections on the basis of the overall drift requirements rather than the tedious strength checks of individual columns. The capacity demand design concept (strong columns-weak beams) should be applied in the preliminary member sizing.

Step3: Modeling of elements and geometric nonlinearities. Columns and beams are modeled by two finite elements for acceptable accuracy. The geometrical and material non-linearities as well as residual stresses are taken into account directly by the modified software DRAIN-3DX [10].

Step 4: PAs at the LS level and member adjustment. A sequence of three dimensional PAs of the structure under gravity loads are stepwise performed by monotonically increasing the intensity of the lateral loads. In the LS performance level, the allowable limits for the response magnitudes are shown in Table 1.

Table 1. Allowable response limits for the LS performance level

1.	Damage indices 10%<D<20%
2.	Storey drifts<1.50%h (h=storey height)
3.	Plastic rotations θ_{pl} at the ends of every member < $6\theta_y$ [10], where θ_y is the rotation at first yielding
4.	Plastic hinges only in beams (capacity design)

During the inelastic static analyses all the structure performance objectives are checked at every load step in order to find out that step or point of the lateral load-roof displacement pushover curve at which all of them approach from below the LS allowable limits. This point signifies the beginning of the post elastic region and helps to estimate the inelastic base shear V_{in}^{LS}. During this monotonical increase of the lateral loads, if some response quantities attain values well below their limit values, members without plastic hinges are replaced by lighter ones. However, if some response quantities attain values higher than their limit values, members with plastic hinges are replaced by heavier ones. By this approach at every load step the hinge formation pattern is checked in order to assure that no hinge appears in columns (except at their ground bases) and thus, satisfy capacity design requirements and a global collapse mechanism.

Step 5: PA at the CP level. An additional check of the optimum structure for the LS level is also performed at the CP level for which the allowable response limit values are shown in Table 2. Thus, PAs are performed until the allowable limits for the CP level are approached from below as close as possible. Possible member adjustments are done as in the previous step.

Table 2. Allowable response limits for the CP performance level

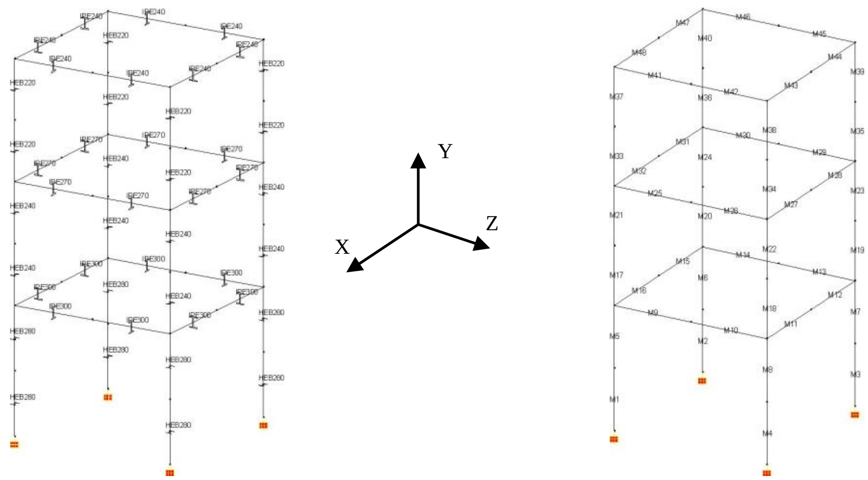
1.	Damage indices 20%<D<50%
2.	Storey drifts <2.50%h (h=storey height)
3.	Plastic rotations θ_{pl} at the ends of every member > $6\theta_y$ [10], where θ_y is the rotation at first yielding
4.	Plastic hinge formation on beams and columns

Step 6: PA at the IO level. The optimum structure for the LS and CP levels is finally checked for the IO level. Thus, this structure is analyzed so that its response is elastic (no plastic hinges, no damage) and its deformation has an IDR value very close from below to the limit value 0.5%.

4. Application of the proposed method

A regular three-dimensional steel MRF that has been studied in [2] is considered here. The geometry, a typical member section selection and the finite element numbering of the structure are shown in Fig.1. The beam and column sections are IPE and HEA, respectively, with a steel grade of S275. All the structural members of the specific steel frame have been modeled with the aid of the DRAIN-3DX program [10] using two improved fiber elements E15 per member (Fig.1).

According to the PA, the frame is laterally loaded along the two horizontal directions x and z by loads $\{F_x\}$ and $\{F_z\}$ with height-wise distribution given by Eq. (1) and monotonically increasing intensity. The response is obtained according to EC8 [12] as a result of the two possible load combinations $\{F_x\}+0.3\{F_z\}$ and $\{F_z\}+0.3\{F_x\}$, which for reasons of simplicity are symbolized in the following as x+0.3z and z+0.3x, respectively.



HEB280/240/220(columns)
IPE300/270/240(beam)

Structural members numbering

Figure 1. Geometry, member sections and finite element numbering of the three storey space steel frame

Table 3 presents five alternative member section selections of the space frame, which come from the proposed iterative design procedure and represent five different structures (Frame A,B,…,E) from the design point of view. Slashes (/) separate member sections of different storeys. When there is only one slash (/), the first number indicates sections of the first two storeys, while the second one sections of the last (upper) storey.

Table 3. Five different member section selections for the three storey space steel frame

Space frame	Member section selection
A	HEB280/240/220(columns) - IPE300/270/240(beams)
B	HEB260/240/220(columns) - IPE270/240(beams)
C	HEB240/220/200(columns) – IPE270/240(beams)
D	HEB220/200(columns) – IPE270/240(beams)
E	HEB220/200(columns) – IPE240/220(beams)

Starting with the x+0.3z lateral load combination for space frame A, it is found that the storey drifts reach the LS allowable values (4.80 cm) in the x direction (Fig. 2) with maximum values at the second and third storey equal to 4.79 cm and 3.92 cm, respectively. The storey drifts with respect to the z direction (Fig. 2) have very small values in comparison with the LS allowable limits. The total damage equals to 14.62% < 20.00% (Fig. 3) and plastic hinges have been formed only at beams (satisfaction of capacity design criterion) with respect to the x direction (Fig. 4). Table 4 shows that member DIs have values not exceeding the allowable limit of 20.00%. The x direction base shear value from PA equals 419.94 kN. Similarly, the z direction base shear value from PA equals 122.00 kN. From the above, it is evident, that the main design objectives for the LS are satisfied, and its members (especially those without plastic hinges) may be replaced by lighter ones. Similarly, for z+0.3x lateral load combination for space frame A, the main design objectives for the LS are satisfied (Figs. 5-7). Thus, for both space frame sides and load combinations, the main design objectives for the LS are satisfied.

This procedure was repeated in turn for frames B, C, D and E and it was found that all the LS allowable limits are satisfied by the corresponding response measures indicating acceptable designs with lighter member sections as going from frame B to frame E of Table 3. Some results for these frames are shown in Figs 2-7. Considering the results of all the frames examined, it is concluded that the optimum design is frame E.

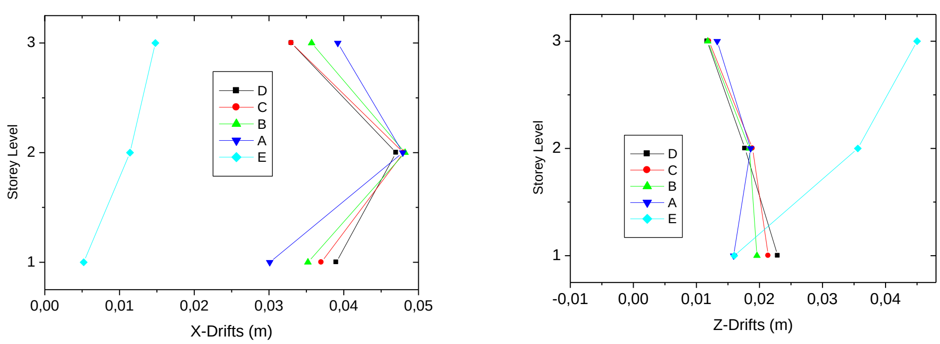


Figure 2. Storey drifts along x and z from pushover analyses (x+0.3z /LS)

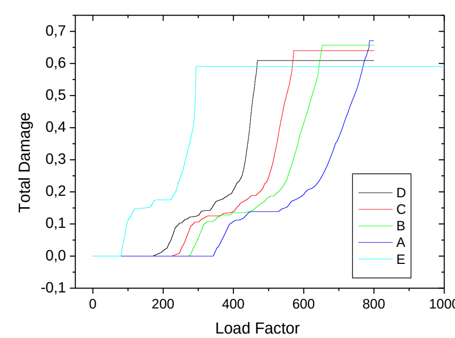
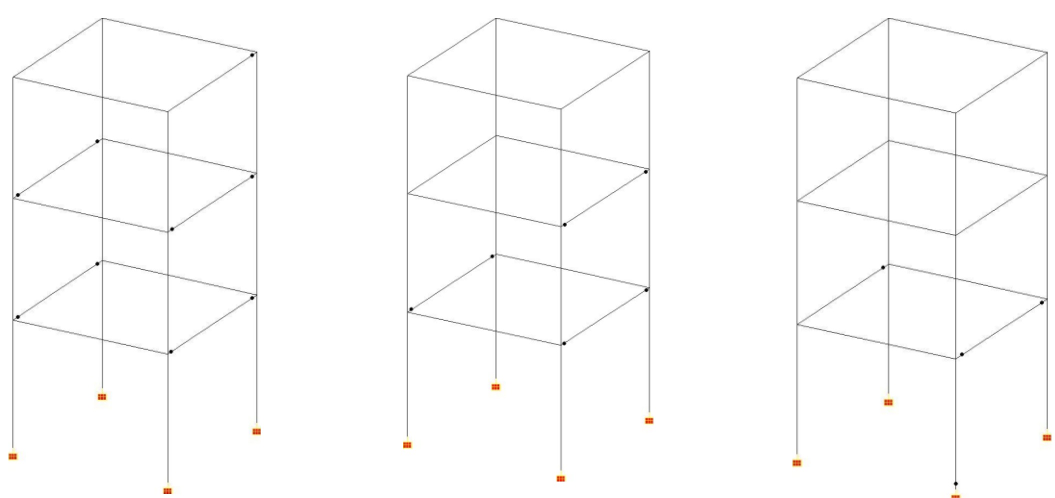
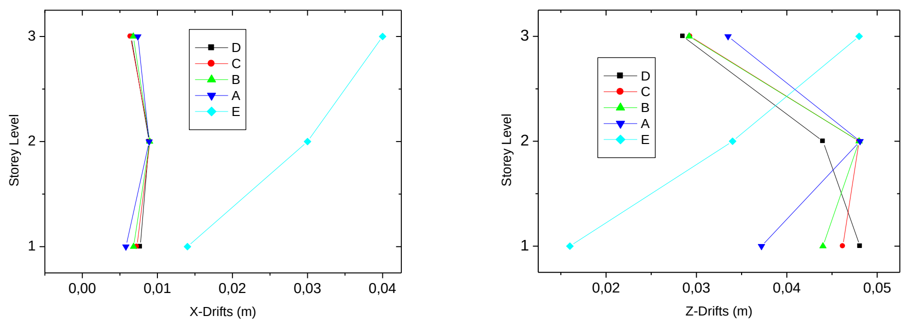


Figure 3. Total DIs (x+0.3z/LS)



Frame A Frame D Frame E

Figure 4. Plastic hinge formation from pushover analyses (x+0.3z/LS)



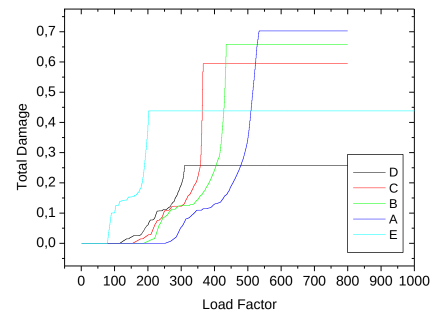
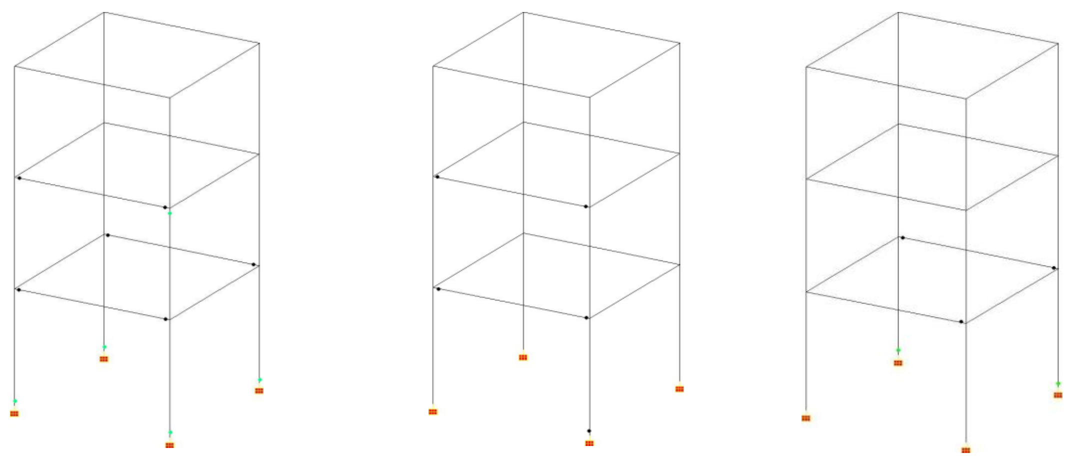


Figure 5. Storey drifts from pushover analyses (z+0.3x /LS)

Figure 6. Total DIs (z+0.3x/LS)



Frame A Frame D Frame E

Figure 7. Plastic hinge formation from pushover analyses (z+0.3x/LS)

Table 4. Member DIs (x+0.3z/LS)

Space frame A			
M.	**Damage Start**	**i-end Damage**	**j-end Damage**
11	353	16.39%	
12	343		16.63%
15	474	11.94%	
16	465		12.00%
27	362	16.37%	
28	356		16.41%
31	472	11.96%	
32	467		12.08%
44	517		2.41%

Space frame E			
M.	**Damage Start**	**i-end Damage**	**j-end Damage**
2	99	0.70%	
11	86	11.94%	
12	80		14.07%
15	97	1.01%	

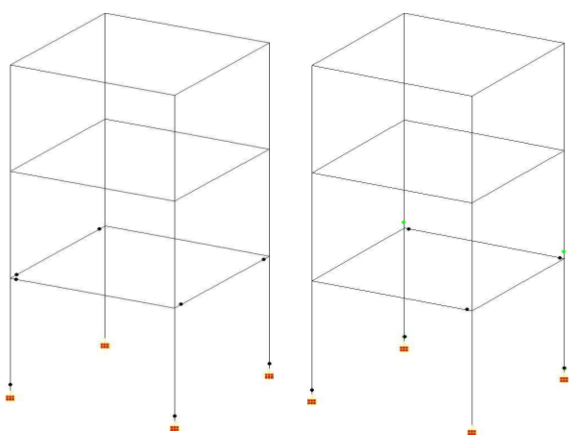
Table 5. Member DIs (z+0.3x/LS)

Space frame A			
M.	**Damage Start**	**i-end Damage**	**j-end Damage**
1	311	6.11%	
2	388	6.08%	
3	285	3.08%	
4	372	2.07%	
9	299	15.78%	
10	279		16.29%
13	368	11.83%	
14	352		12.63%
25	347	11.94%	
26	331		13.67%

Space frame E			
M.	**Damage Start**	**i-end Damage**	**j-end Damage**
3	94	1.01%	
4	85	2.11%	
10	81		1.91%
13	90	11.89%	
14	78		15.24%

Frame E is also checked for the CP performance level. More specifically, it was found that the storey drifts limits are satisfied under the x+0.3z and the z+0.3x load combinations. The total DIs under the seismic combinations x+0.3z and z+0.3x are equal to 29.57% and 15.86%, respectively. Concerning the member DIs, Tables 6 and 7 show that there are some members that exceed the value of 20.00%. Finally, Fig 8 presents the plastic hinge formation pattern under the seismic combinations x+0.3z and z+0.3x. It is observed that plastic hinges are formed mostly in the beams of the frames and the bases of the lowest columns. Hinges are also formed in the upper ends of a very few columns. In conclusion, damage is rather extensive but collapse of the structure is prevented.

For comparison purposes, the commercial design software SAP 2000 [14] is used for the seismic design of the frame, based on EC3/EC8 method. It was found that the optimum member selection is the one corresponding to Frame D of Table 3. Thus, member sizes determined by the proposed method are close to those determined by the EC3/EC8 method, but slightly lighter. The same frame has also been designed with the aid of dynamic nonlinear analyses in [2] and the results coincide with the present ones.



(x+0.3z) (z+0.3x)

Figure 8. Plastic hinge formation of frame E for CP

Table 6. Member DIs (x+0.3z/CP)

Space frame E			
M	**Damage Start**	**i-end Damage**	**j-end Damage**
1	168	12.83%	
2	110	19.68%	
3	172	20.51%	
9	209	16.01%	
11	86	29.39%	
12	80		27.78%
15	199	16.14%	
16	195		16.23%
17	180	3.57%	

Table 7. Member DIs (z +0.3x/CP)

Space frame E			
M	**Damage Start**	**i-end Damage**	**j-end Damage**
1	131	2.87%	
3	94	4.61%	
4	85	4.58%	
10	143		2.79%
13	90	17.05%	
14	78		20.80%

5. Conclusions

A seismic design method for regular space steel frames based on PA has been proposed. The advantages of this design method are the following:

1. It captures the limit state strength and stability of the structural system and its individual members directly. It also captures the inelastic redistribution of internal forces throughout a structural system, and allows an economic use of material.
2. Compared to the EC3/EC8 method, the employment of the PA provides more information on structural behavior.
3. The method is capable of providing the values of all the important performance indices for each frame section selection. Thus, a performance-based design is achieved.

4. Member sizes determined by the proposed method are close to those determined by the EC3/EC8 and as a result, the proposed method provides at least a more rational and efficient alternative to the EC3/EC8 design method.

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