Nonlinear Static Procedures proposed in American and European Seismic Codes and its Extensions

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ABSTRACT
The Nonlinear Static Procedures (NSPs), using simplified nonlinear techniques are generally adequate for the seismic assessment and design of short and regular buildings; otherwise nonlinear dynamic procedures are the most appropriated method to be used. To overcome the limitations of NSPs for irregular buildings, some improvements have been proposed all over the years. In this study, the Capacity Spectrum Method (CSM), with the features used in the American codes/guidelines ATC40 and FEMA440, the original N2 method, adopted in the current version of European code (Eurocode 8), as well as the extensions of the Coefficient method (FEMA 273) presented in ASCE41-06 and of the N2 method are applied to a plan-asymmetric structure. Different seismic response quantities are compared with the ones obtained by means of nonlinear dynamic analyses, in order to assess the accuracy of different NSPs, in particular the precision of the extensions of the original NSPs presented in the European and American Codes.

Keywords: Nonlinear Static Procedures, torsion, plan-asymmetric buildings, Seismic Codes

1. INTRODUCTION

Nowadays, NSPs are considered a very practical tool to evaluate the seismic behaviour of regular structures. A large number of works have been proving this, fundamentally comparing results obtained through NSPs with Nonlinear dynamic analyses (Fajfar and Fischinger, 1988; Freeman, 1998). So Seismic Codes claim that nonlinear procedures should be used for analysis of buildings where linear procedures are not allowable according to the criteria defined in each code. In 1975, Freeman and collaborators presented for the first time the innovative capacity spectrum method—the so-called CSM (Freeman, et al., 1975). Since then, this method has gained considerable popularity among pushover users, and the ATC40 guidelines (ATC, 1996) included it as the recommended NSP to be used. Later, the FEMA440 report (ATC, 2005) came out with an updated version of the method increasing the precision of its results. On the other hand Eurocode8 (CEN 2004) proposed the N2 method, developed by Fajfar and his team (Fajfar and Fischinger 1988), the NSP to be used when performing pushover analyses. The problem of the commonly used NSPs, as the ones referred, is their inability to deal with irregular or tall structures, in particular plan-asymmetric buildings. Generally they cannot capture the torsional effects distorting the real structural response. Some efforts have been made to take this generation of NSPs to a higher performance level (Chopra and Goel, 2004; Fajfar et al. 2005; Bento et al. 2010; Stefano and Pintucchi 2010; Erduran and Ryan 2010; Bhatt and Bento, 2012). Thus, in the last American code ASCE 41-06 (ASCE 2007), the Displacement Coefficient method described in FEMA 273 Guidelines (FEMA, 1997) and improved in FEMA 356 (ASCE, 2000), FEMA440 (ATC, 2005), together with some recommendations to take into account the torsion effects, are proposed. In the European code a new version of the N2 method should be added in the future, since Fajfar and his team developed the Extended N2 method (Fajfar et al. 2005). This extension is able to capture the torsional behavior of plan-asymmetric buildings. In this paper, American and European codes procedures are evaluated and compared: the CSM (Freeman et al. 1975, Freeman 1998) with the features proposed by ATC40 and FEMA440, the method proposed in ASCE41-06 and both N2 and Extended N2 method. The case study is the well known SPEAR building. Comparison of the results obtained with nonlinear dynamic analyses (NDA), through the use of semi-artificial ground motions, enables the evaluation of the accuracy of the different NSPs.

2. NONLINEAR STATIC PROCEDURES (NSP)

The CSM included in ATC40 and FEMA440, the coefficient method present in ASCE41-06 and the N2 method proposed in Eurocode 8, rely on a pushover analysis using invariant load patterns (the load pattern does not change during the analysis, only the force intensity) to estimate the capacity of the structure and then the deformation demands under seismic loads. The forces used in the pushover analysis are proportional to the relevant mode of vibration of the structure under analysis (in most of the structures) or with a uniform distribution. In CSM (FEMA440 or ATC40 guidelines), the demand spectrum is reduced using a reduction factor. The N2 method represents the seismic demand by an inelastic spectrum. The coefficient method, completely described in ASCE41-06, provides a direct numerical process for calculating the displacement demand. The Extended N2 method correspond to the extension to the original N2 method (Fajfar & Fischinger, 1988), presented in Eurocode 8 (CEN, 2004), which was proposed by Fajfar et al (Fajfar, et al., 2005) in order to overcome the torsional problem in these structures. This extension intends to handle the torsional problem in plan-asymmetric buildings by adjusting the pushover results, computed with the original N2 method (Fajfar and Fischinger, 1988), by means of correction factors based in linear dynamic response spectrum procedures.

3. CASE STUDY – SPEAR BUILDING

The SPEAR building represents typical existing 3-storey buildings in the Mediterranean region following Greece's concrete design code between 1954 and 1995. This structure was designed only for gravity loads based on the construction practice applied in the early 1970s that included the use of smooth rebars. It was tested in full-scale under pseudo-dynamic conditions, and subjected to bi-direction seismic loading, at JRC Ispra within the European SPEAR project framework. Plan and elevation views are shown in Fig. 3.1, while further details on the structure and its pseudo-dynamic testing can be found in (Fardis, 2002) and (Fardis and Negro, 2006).

The building is plan-asymmetric in both *x* and *y* directions, but it is regular in elevation (Fig.3.1). The storey heights amounted to 2.75 m for the first floor and 3.00 m for the upper storeys. Eight of the nine existing columns have a square cross-section of $250\times250mm^2$. The column C6 has a rectangular section of $250\times750mm^2$, with the higher dimension oriented along the Y direction implying a 'weak direction' along the X-axis. The column C6 and the presence of a balcony on the east side of the structure are the major causes for the in-plan irregularity, shifting the centre of mass (CM) away from the centre of stiffness (CR), which is close to the central column (C3), thus causing the eccentricity to be larger in the Y direction. The translational masses, obtained from the seismic combination, are 67.3 tonnes each for the first two floors and 62.8 tonnes for the roof.

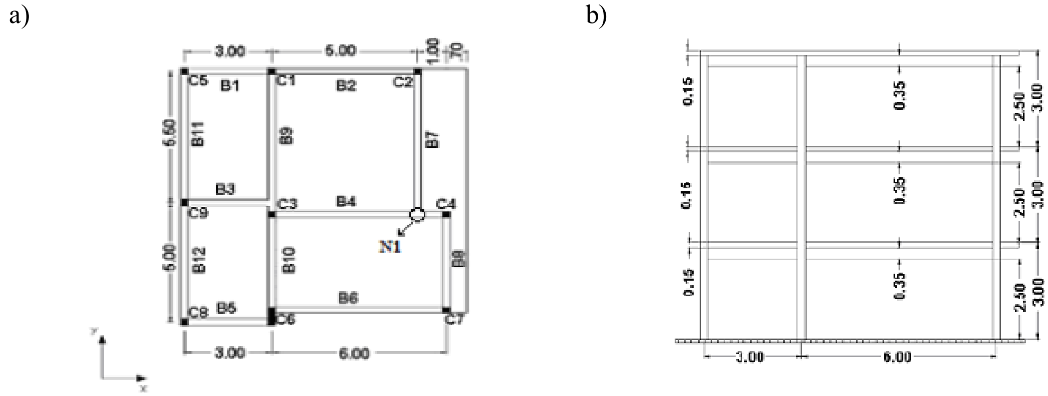

Figure 3.1 – SPEAR building: a) Plan View; b) Lateral View

4. MODELLING ISSUES

The analysis software adopted in this work was SeismoStruct v6.0 (SeismoSoft, 2006), a downloadable fibre element-based finite element program. The 3D model representing the building under analysis was built using space frames assuming the centreline dimensions. Sections were defined with 300 fibres. Each fibre was characterized by the respective material relationship. The column–beam end connections were not modelled with rigid offsets; however, elongated columns were modelled as wall elements due to their larger dimension. Hysteretic damping was already implicitly included in the nonlinear fibre model formulation of the inelastic frame elements. In order to take into account for possible non-hysteretic sources of damping, a tangent stiffness proportional damping was used, following the suggestions of Priestley and Grant (2005). Two values of damping coefficient were used: 0%, and 2%. The last one was used, according to the experimental results at ISPRA. The concrete was represented by a uniaxial model that follows the constitutive relationship proposed by Mander *et al*. (1988) and the cyclic rules proposed by Martinez-Rueda and Elnashai (1997). The confinement effects provided by the lateral transverse reinforcement are taken into account through the rules proposed by Mander *et al*. (1988) whereby constant confining pressure is assumed throughout the entire stress–strain range. A compressive strength of 25MPa was considered for the SPEAR building. The constitutive model used for the steel was the one proposed by Menegotto and Pinto (1973) coupled with the isotropic hardening rules proposed by Filippou *et al*. (1983). The average yield strength of 360MPa was assumed. The Nodal Constraints with Penalty Functions option were taken to model the rigid diaphragm effect. The penalty function exponent used was 10^7. Since the slab was not explicitly modelled, the effects of flexural stiffness of the slab were considered by assigning appropriate flange widths to the beams. The weight of the slab was applied lumped in the node.

5. SEISMIC FEATURES

Seven bi-directional semi-artificial ground motion records from the SPEAR project were considered (Table 5.1). These records had been fitted to the Eurocode 8 elastic design spectrum (Type 1, soil C) for a Peak Ground Acceleration (PGA) of 0.2g. For the other intensities analyzed (0.05g, 0.1g and 0.3g) the records were scaled with the respective scaling factor. About Nonlinear Dynamic Analyses, due to be impossible to know the position of the building relatively to the components of the records, all records were assigned to the building in two different ways: x component of the record according to the x component of the building and y component of the record assigned to the y component of the building; and the opposite, i.e. the x component of the record assigned to the y direction of the building and the y component of the record assigned to the x component of the building. The final seismic response is determined by the mean of the 14 results obtained.

Table5.1 – Ground motion records considered for SPEAR building (Fardis, 2002)

Earthquake Name	Station Name
Imperial Valley 1979	Bonds Corner
Loma Prieta 1989	Capitola
Kalamata 1986	Kalamata – Prefecture
Montenegro 1979	Herceg Novi
Friuli 1976	Tolmezzo
Montenegro 1979	Ulcinj2
Imperial valley 1940	El Centro Array #9

6. NUMERICAL RESULTS AND DISCUSSION

In this chapter, the seismic response of SPEAR obtained by the aforementioned procedures is presented in terms of top displacement ratios, lateral displacement profiles, interstorey drifts, normalized top displacements and Shear Forces for different levels of seismic intensities.
The discussion will be focused mainly in the results obtained in the inelastic range of the structure, because the NSPs were developed to evaluate the structural response at this stage. The modal properties of the building are presented in Table 6.1, showing the periods and the effective modal mass percentages in both X and Y directions (U_x and U_y).

Table 6.1 – Periods and effective mass percentages

Mode	Period (sec)	[Ux]	[Uy]
1	0.62	60.5%	7.8%
2	0.53	23.5%	43.0%
3	0.44	3.2%	31.6%

The SPEAR building has a fundamental mode of 0.62sec characterized by translation along the X direction, a second mode of 0.53sec with torsional motion and a third mode of 0.44sec with translation along the Y direction. Both translational modes are coupled with torsion, and the structure is torsional flexible in the Y direction.

Displacement ratios between the target displacements obtained with the analyzed NSPs and the corresponding mean estimates coming from the nonlinear dynamic analysis are computed (equation 1). The NSPs must never lead to underestimated results, therefore these ratios should always be higher than 1. For this case, one would desire such ratios to be between the line which corresponds to the unity and the second line which shows the ratio between the results obtained by nonlinear dynamic analyses, considering 0% of viscous damping and 2% (the one which correspond to the unity); which means the NSPs would match to the time-history mean results. Figure 6.1 shows the top displacement ratios at the centre of mass, for X and Y direction.

$$Top\ \ Displacement\ ratio = \frac{NSP's\ top\ displacement}{Time\ History\ mean\ top\ displacement} \tag{1}$$

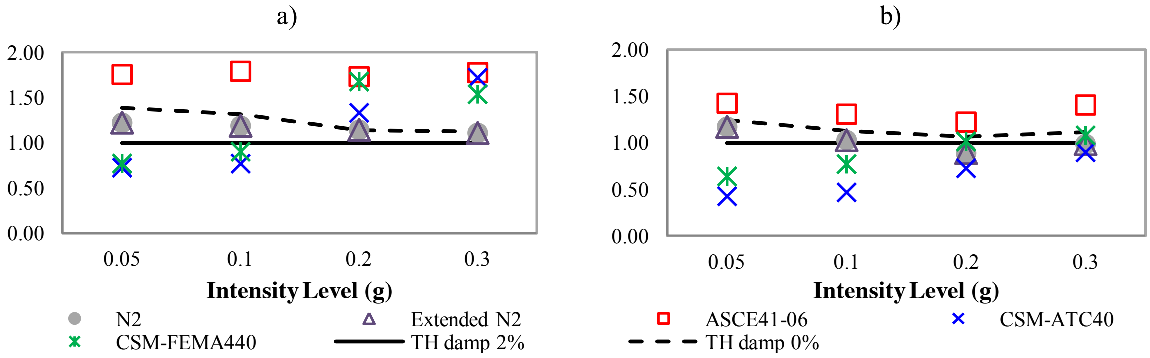

Figure 6.1 – Top Displacement Ratios: a) Centre of Mass, X direction; b) Centre of mass, Y direction

From the plots, one can see that the target displacements obtained with CSM-ATC40 are generally underestimated. An exception is observed in X direction in the inelastic range. The same conclusions are taken through CSM-FEMA440, for low seismic intensities. The original and extended N2 method, presenting the same values, lead to accurate results of target displacements, while ASCE41-06 NSP lead always to overestimated results. In the latter, this fact is mainly due to the torsion corrective factor considered in this American code, to be applied together with the coefficient method.

Figures 6.2 and 6.3 depict the lateral displacements profiles and the interstorey drifts, respectively, for different sides of the building (C2 flexible side, C3 centre of the building and C8 stiff side), different directions and different seismic intensities.

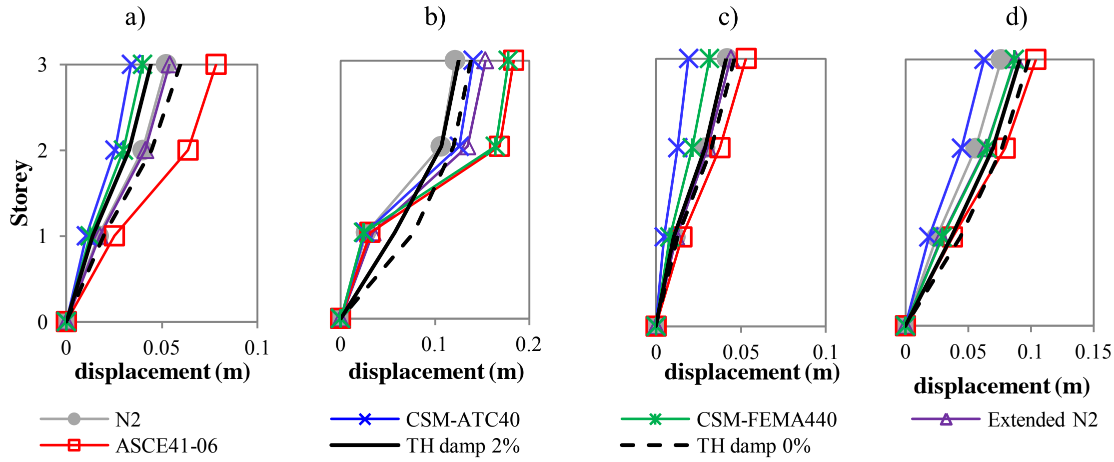

Figure 6.2 - Lateral Displacements Profiles: a) X direction, Column C3, 0.1g; b) X direction, Column C2, 0.2g; c) Y direction, Column C8, 0.1g; d) Y direction, Column C3, 0.2g

The maximum displacements in the building occur along X direction, as expected, as it is the flexible direction of the building. The extended N2 method generally leads to the most accurate results when compared to nonlinear dynamic analyses. Despite the deformed shapes of the columns are not the same obtained by means of nonlinear dynamic analyses, mainly for the flexible side of the building (herein represented by column C2), the results of applying the extended N2 method are very close to the ones of NDA, in terms of top displacements in both directions for all intensities. The original N2 method shows some underestimated displacements in the edge columns by the fact that torsion is not considered in this version. As expected, in terms of displacements, CSM-ATC40 leads to inferior results when compared with CSM-FEMA440, being in general non-conservative for lower intensities. These differences can be due to different estimations of effective period and damping computed by these two methods. On the other hand the maximum displacements obtained through ASCE41-06 NSP are conservative, due the torsion corrective factor calculated at each floor and applied to all displacements of the nodes.

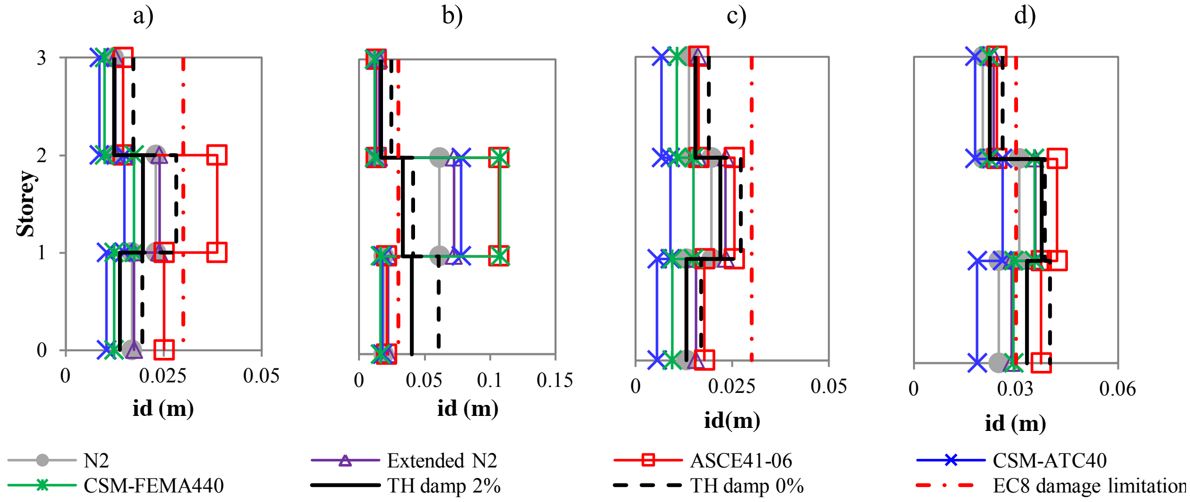

Figure 6.3 – Interstorey drifts in: a) direction X, Column C3, 0.1g; b) X direction, Column C8, 0.2g; c) Y direction, Column C8, 0.1g; d) Y direction, Column C3, 0.2g

As shown in Fig.6.3 b), the maximum interstorey drifts obtained in X direction, for higher intensities of seismic action, i.e. when the building behaves in inelastic regime, do not match with NDA results, neither in terms of values nor deformed shape. One can see that the maximum interstorey drift obtained through NDA is in the 1st floor and, on the other hand, in all NSPs the maximum interstorey drift occurs in the 2nd floor. Nevertheless, for Y direction for all seismic intensities and X direction in linear regime, extended N2 and ASCE41-06 lead to results close to nonlinear dynamic analyses; generally CSM-ATC40 leads to unconservative results as well as CSM-FEMA440 when the structure is in the inelastic range and the original N2 method, as observed in the lateral displacements plot as well, leads to an unconservative estimate of the interstorey drifts. The damage limitation requirement according to Eurocode 8 is herein represented in terms of interstorey drift limitation, and it is verified that for higher intensities this criterion is not verified, in both directions by all NSPs and even for NDA.

In order to study the torsional behaviour of the building, the trend of normalized top displacements is analysed (Fig. 6.4). This measure is obtained by normalizing the edge displacement values with respect to those of the centre of mass. The torsional response in the NDA is taken from the step of the analysis correspondent to the maximum top displacement (in absolute value) in the centre of mass.

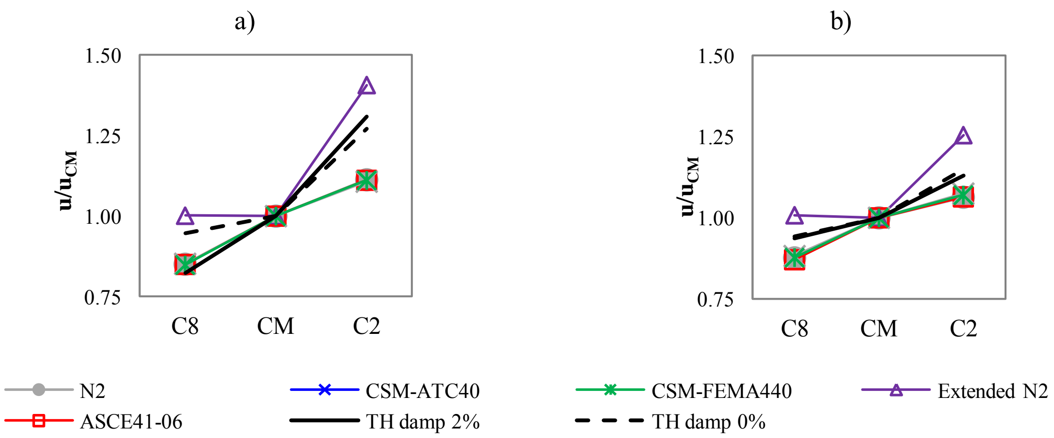

Figure 6.4 – Normalized Top Displacements: a) X direction, 0.2g; b) Y direction, 0.2g

Only the extended N2 method is able to capture the torsional amplification in both sides of the building. The original N2 method, CSM-ATC40 and CSM-FEMA440 estimate linearly the response from one side of the building to the other, underestimating the torsional amplification on the both sides of the building. Regarding the ASCE41-06 NSP, although the results in terms of displacements are conservative, the torsional amplification is not captured. It happens due the aforementioned fact of the method taking in account the torsional effects multiplying the displacements by the same factor in all points at each floor.

As far as Shear Forces concerned, it was considered that the building analyzed was properly designed to shear, and therefore it does not have brittle failures. In order to confirm this assumption, some studied columns were analyzed in terms of shear behavior (demand *vs* capacity) for the seismic intensities considered. This shear capacity was calculated based on the two codes, ATC40 and Eurocode 8 but the latter generally leads to smaller values of capacity (thus this values are the ones depicted in Fig. 6.5). For SPEAR building analyzed, the internal shear forces results through NSPs and NDA do not reach the shear capacity of the columns for any intensity of seismic action. The results obtained for column C6, the only column which section is not square with principal direction of inertia along Y, are also represented in the following plots, Fig. 6.5.
The CSM-ATC40 and CSM-FEMA440 procedures lead to underestimated shear forces for low and high intensities. On the other hand, original N2 method, extended N2 method and ASCE41-06 NSP lead to similar results for low intensities comparing with NDA. However in inelastic regime, such methods also lead to non-conservative results in both directions.

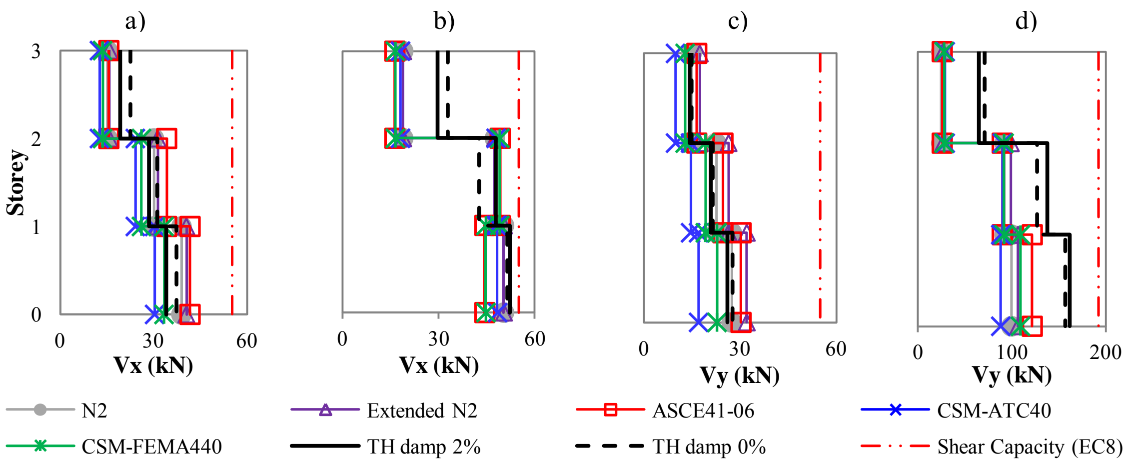

Figure 6.5 – Shear Forces: a) X direction, Column C3, 0.1g; b) X direction, column C6, 0.2g; c) Y direction, column C2, 0.1g; d) Y direction, Column C6, 0.2g

7. CONCLUSIONS

In this paper NSPs proposed in American and European seismic codes/guidelines are herein applied to an asymmetric plan building. From the American documents, CSM-ATC40, CSM-FEMA440 and the NSP proposed in the most recent code ASCE41-06 are applied as well as the N2 method proposed in Eurocode 8. Moreover, the improved version of N2 (extended N2), to take into account the effects of the torsion of the building, is also studied. Due the uncertain of the proper non-hysteretic damping to use, two different values were adopted for the building, so the reference results obtained through Nonlinear Dynamic analyses are not presented as right single values for each type of response: the mean values obtained for 0% and 2% damping coefficient are presented and compared. However, it is noticed, and as expected, that this factor pales while the intensity of ground motion increases, decreasing the range of comparable values. In terms of results obtained by NSPs for this case study, extended N2 shows the most accurate results in terms of displacements, forces and torsional response as well. The ASCE41-06 always show conservative results in terms of displacements, however the torsional behaviour is not still captured. In order to obtain the target displacement, all NSP tested, except CSM-ATC40 which show non-conservative results, lead to accurate results when compared with nonlinear dynamic results. Since the building analysed shows a considerable torsion in the translational modes, none of the NSPs are able to get the real deformed shape in an accurate way, when the structure exhibit inelastic behaviour.

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