Performance Based Seismic Design and Assessment of Irregular Steel Structures

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SUMMARY:
In recent years, Nonlinear Static Procedures (NSPs) became a powerful tool for seismic performance evaluation. Several seismic design/assessment Guidelines have recommended this type of procedure as design/evaluation technique. The aim of this work is to assess the performance of these procedures applied to a set of 1-storey steel structures with plan irregularities. To investigate the torsional phenomenon, two types of structures were considered, namely torsionally restrained and torsionally unrestrained.
The NSPs evaluated were the original N2 method recommended in Eurocode 8 (CEN, 2004), the Extended N2 method developed by Fajfar *et al.* (2005), the Capacity Spectrum Method (CSM) specified in ATC40 (ATC, 1996) with the improvements presented in FEMA 440 Report (ATC, 2005) and the Adaptive Capacity Spectrum Method (ACSM) proposed by Pinho *et al.* (2007). The accuracy of the different Nonlinear Static Procedures was examined through comparison of the results with those obtained from nonlinear dynamic time-history analysis.

Keywords: Pushover analysis, Capacity Spectrum Method, N2 method, irregular steel buildings.

1. INTRODUCTION

The structural analysis in earthquake engineering is a complex problem because the seismic action is dynamic and usually leads the structure into the nonlinear range. In principle, the nonlinear dynamic time-history analysis is the correct approach. However due to the random and level of uncertain that characterize the seismic action, it is not usual practice to perform this type of analysis for the design/assessment of common structures. For this reason, in recent years, a breakthrough of the nonlinear static procedures (NSPs) has been observed. However, most of the methods were developed to regular structures and its extension to irregular structures is not straightforward. The aim of this work is to investigate the performance of these procedures when applied to irregular structures, in particular plan-irregular steel structures. To evaluate the torsional phenomena that arise due to the plan irregularities, two types of structures were considered: torsionally unrestrained and restrained irregular structures.

The Nonlinear Static Procedures examined were the original N2 method recommended Eurocode 8 (CEN, 2004), the Extended N2 method developed by Fajfar *et al.* (2005), the Capacity Spectrum Method (CSM) from Report ATC40 (ATC, 1996) with the improvements presented in FEMA 440 Report (ATC, 2005) and the Adaptive Capacity Spectrum Method (ACSM) proposed by Pinho *et al.* (2007). The Capacity Spectrum Method was applied considering two types of bilinear curves: assuming zero post-elastic stiffness (α=0) and calculating the post elastic stiffness parameter ($\alpha \neq 0$). The accuracy of these procedures was assessed by comparing the results with the results obtained from nonlinear time-history (TH) analysis, which is considered as the analysis that provides the most adequate seismic response of structures.

2. STRUCTURES STUDIED

The study consisted in the assessment of the response of two types of 1-storey plan-irregular steel structures: torsionally restrained and torsionally unrestrained structures. The plan layouts are illustrated in Figures 2.1 and 2.2 and the four cases considered in the study are identified in Table 2.1.

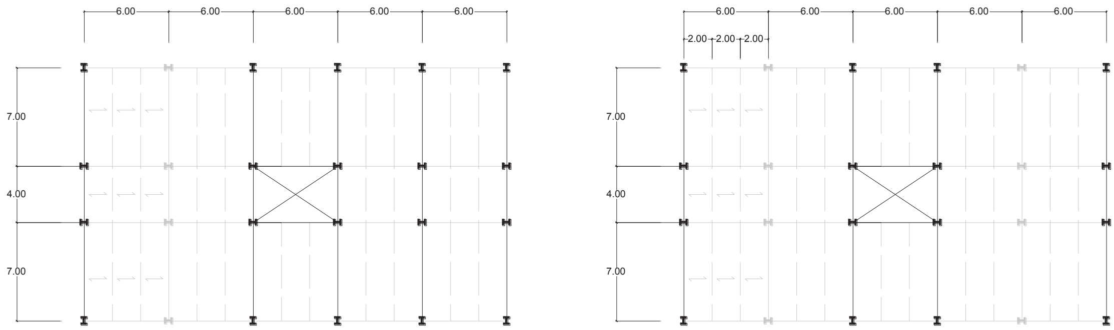

Figure 2.1 Torsionally unrestrained structure and corresponding reference structure

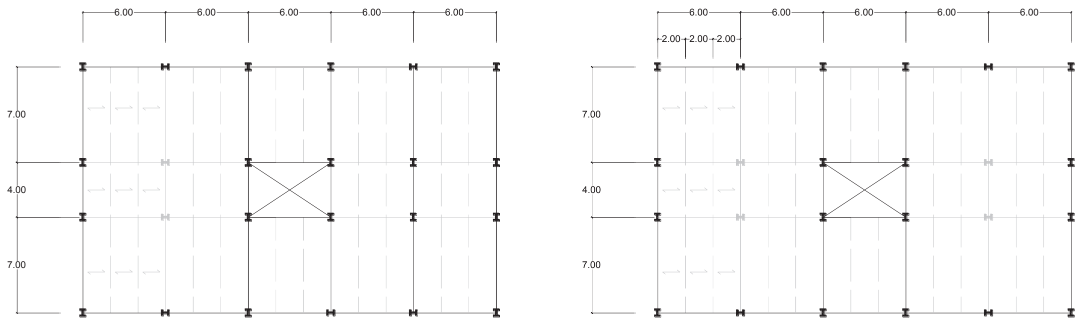

Figure 2.2 Torsionally restrained structure and corresponding reference structure

The structures were initially designed only for gravity loads according to Eurocode 3 (CEN, 2005). These loads comprised the self-weight of the structure with an allowance of 1 kN/m^2 for finishing and partitions and an imposed load of 2 kN/m^2. European IPE sections were used for the beams and HE sections were adopted for the columns. The steel grade considered was S275. The seismic design was performed according to Eurocode 8 (CEN, 2004) assuming a Type 1 response spectrum and soil type B for an intensity level of 0.3g. The behaviour factor was estimated as proposed by Villani *et al.* (2009) in the Improved Force-Based Design method. The periods of vibration, T, and behaviour factors, q, of the four structures considered in the study are listed in Table 2.1.

Table 2.1 Structures designation, dynamic properties and behaviour factors

Case	Designation	Direction x		Direction y	
		T(s)	q	T(s)	q
Structure torsionally unrestrained – plan irregular	TU-PI	0.60	3.5	0.54	2.0
Structure torsionally unrestrained – plan regular	TU-PR	0.66	3.0	0.53	2.5
Structure torsionally restrained – plan irregular	TR-PI	0.60	2.0	0.71	2.5
Structure torsionally restrained – plan regular	TR-PR	0.65	2.0	0.69	2.5

3. NUMERICAL MODELLING AND ANALYSIS PROCEDURES

The seismic assessment of the structures was carried out by means of different nonlinear static procedures and the results obtained were compared with results provide by nonlinear time-history analyses. Both the nonlinear static analyses (pushover analysis) and the nonlinear time-history analyses were conducted with the finite element analysis package OpenSEES (PEER, 2006).The material nonlinear behaviour was considered through a fibre modelling approach. Force-based elements were employed to represent beams and columns, adopting one element per member with seven integration points. Regarding the material model, a simplified bilinear stress-strain constitutive rule was assumed. The geometrical nonlinearities were also considered in the analyses. The conventional pushover analyses were performed assuming a lateral force load vector following the 1st mode of vibration of the structures, whereas the adaptive pushover analyses were carried out through the application of a lateral displacement load vector that is dependent on the actual deformed pattern calculated at each step of the analysis (Pinho *et al*., 2007). The loads were applied independently in the two horizontal directions and with positive/negative signal, resulting in four analyses. For each analysis, the target displacement was computed with the larger value in each direction being chosen.

The seismic input for the nonlinear time-history analysis consisted of seven records (Table 3.1) obtained from real earthquake events. The records selection was made using the software REXEL (Iervolino *et al.*, 2010), a tool that allows the search of sets of seismic records from the European Strong-motion Database based on a degree of compatibility with the Eurocode 8 (CEN, 2004) spectra.

Table 3.1 Ground Motion records considered

Earthquake Name	Station ID	Waveform ID	Scale Factor x dir. (0.3g)	Scale Factor y dir.(0.3g)
Montenegro 1979	ST63	000197	1.23	1.50
Umbria Marche 1997	ST228	000612	3.94	3.72
Montenegro (aftershock) 1997	ST77	000232	6.31	6.51
Racha (aftershock) 1991	ST200	000530	3.22	3.42
Dinar 1995	ST543	001720	9.88	9.21
Campano Lucano 1980	ST99	000293	3.65	3.62
Montenegro 1979	ST62	000196	0.79	1.18

In the nonlinear time-history analysis the damping was modelled using the Rayleigh damping formulation, considering the damping proportional to the tangent stiffness. The value of viscous damping considered was 2.5%.

4. RESULTS OF THE ANALYSES

The results obtained in the performed analyses are presented in this section. The monitored nodes are the centre of mass, CM, and the columns 5 and 37 in the case of the torsionally unrestrained structure and columns 7 and 12 for the case of torsionally restrained structure. The column nodes are located at the flexible and stiff edges of the structure, as shown in Figure 4.1.

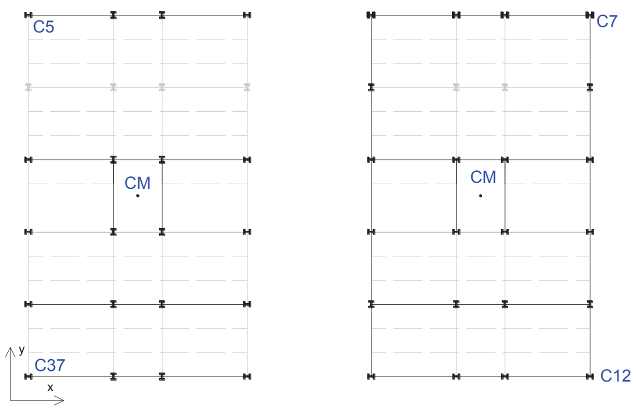

Figure 4.1 Monitored nodes for the irregular structures (TU-PI and TR-PI cases)

The seismic responses of the structures were estimated for two levels of intensity: the design intensity (0.3g) and twice the design intensity (0.6g). The evaluation of the NSPs was performed by means of storey drifts and parameters related with the torsional response of the structures.

4.1. Pushover curves

The pushover curves obtained for the irregular structures are plotted in Figures 4.2 to 4.5 along with the nonlinear time-history results (maximum top displacement at the CM versus the corresponding base shear).

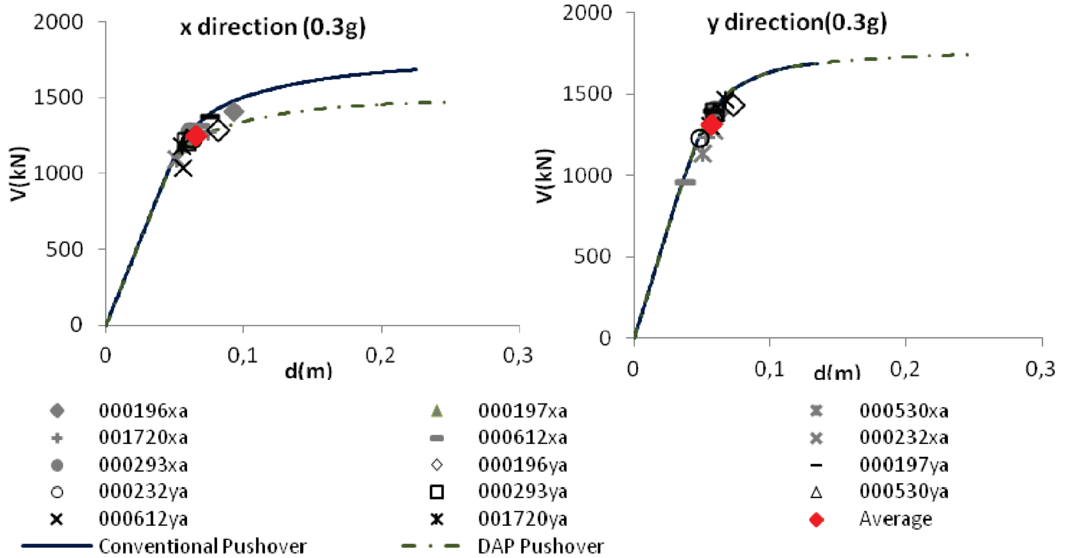

Figure 4.2 Pushover curves and time-history results for the intensity level of 0.3g (TU- PI structure)

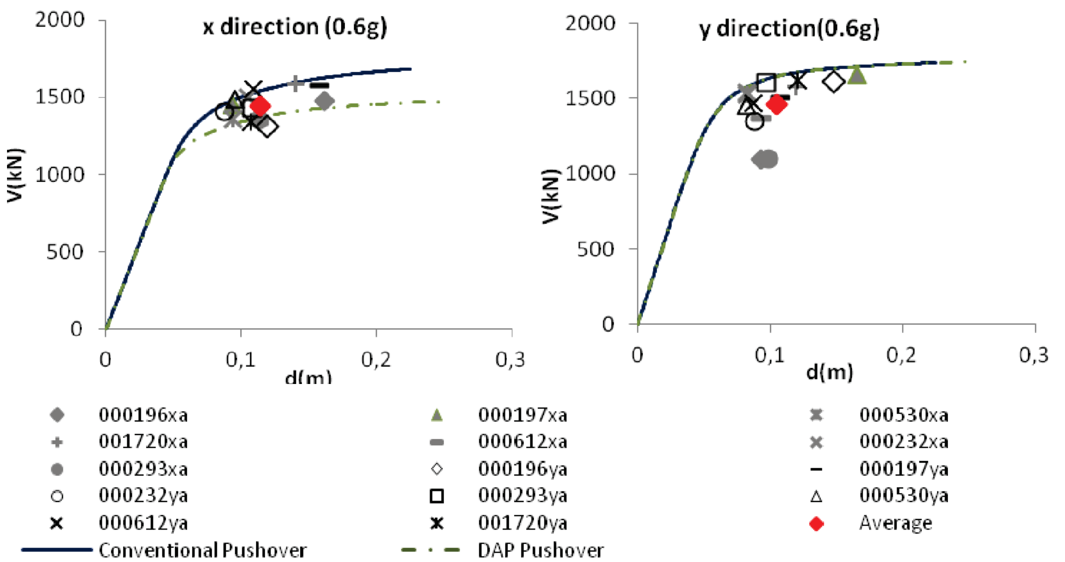

Figure 4.3 Pushover curves and time-history results for the intensity level of 0.6g (TU-PI structure)

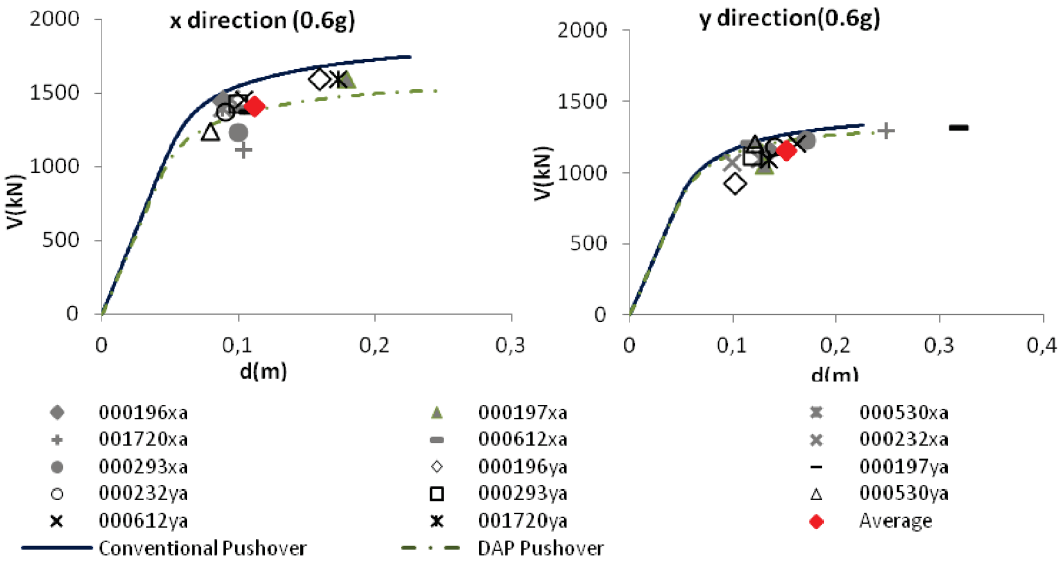

Figure 4.4 Pushover curves and time-history results for the intensity level of 0.3g (TR-PI structure).

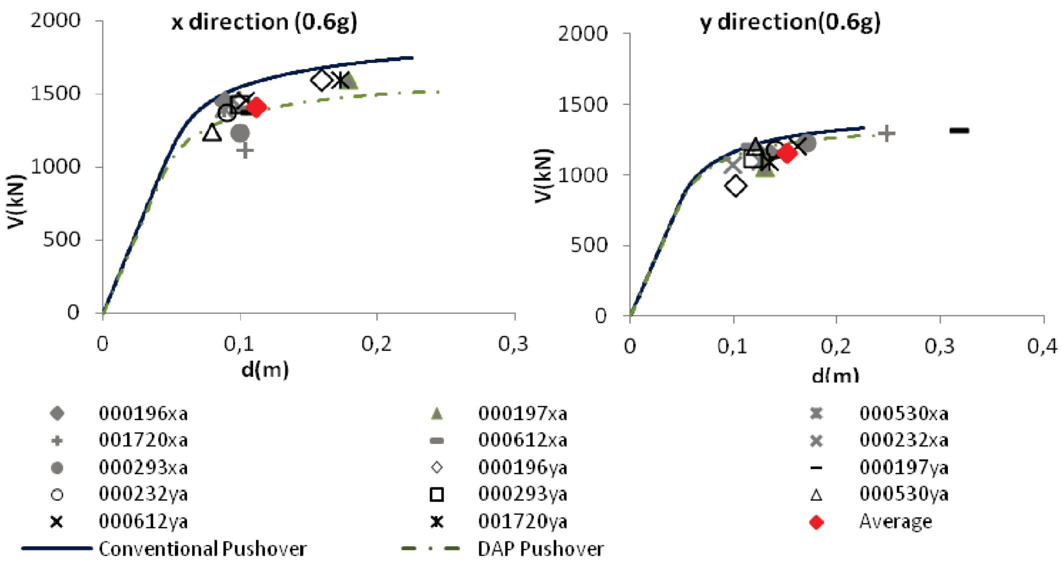

Figure 4.5 Pushover curves and time-history results for the intensity level of 0.6g (TR-PI structure).

The results obtained and depicted in Figures 4.2 to 4.5 emphasize the improvements of the DAP analysis in comparison with the conventional pushover. The reduction of the resistance in the direction of the eccentricities (direction x) is evident while the results of the two analyses are coincident for the direction without irregularities (direction y). Moreover, the DAP analysis seems to achieve closer results to the nonlinear time-history analysis for the structures studied.

4.2. Storey drifts

To assess the performance of the NSPs, the target displacements for the two seismic intensity levels were evaluated and the corresponding storey drifts, measured at the centre of mass, were compared. Although in this document only the results obtained for the irregular structures results are presented (Figures 4.6 and 4.7), it was observed that for the design intensity level (0.3g) the NSPs provide very similar estimates for all the structures and are conservative, i.e. always higher than the nonlinear time-history results.

In general, similar remarks can be made for the intensity level of 0.6g, with the exception of the CSM procedure that provides more conservative results than the N2 and ACSM methods. It is also observed that with the increase of the intensity level the dispersion of time-history results increases and the NSPs estimations are more conservative.

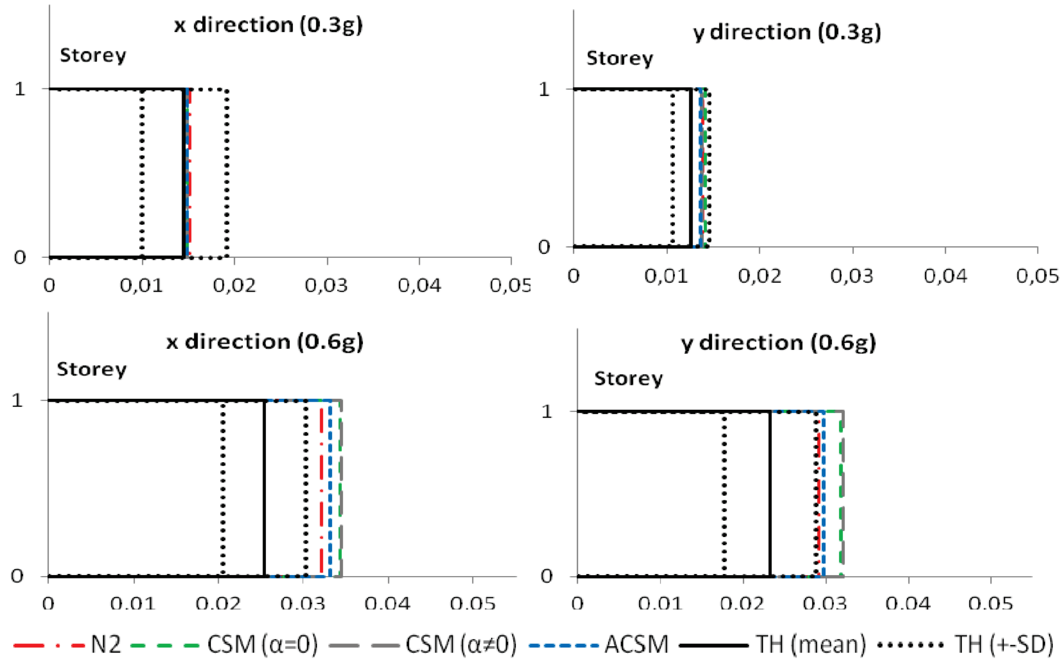

Figure 4.6 Storey drifts for the TU-PI structure in both direction of the analysis

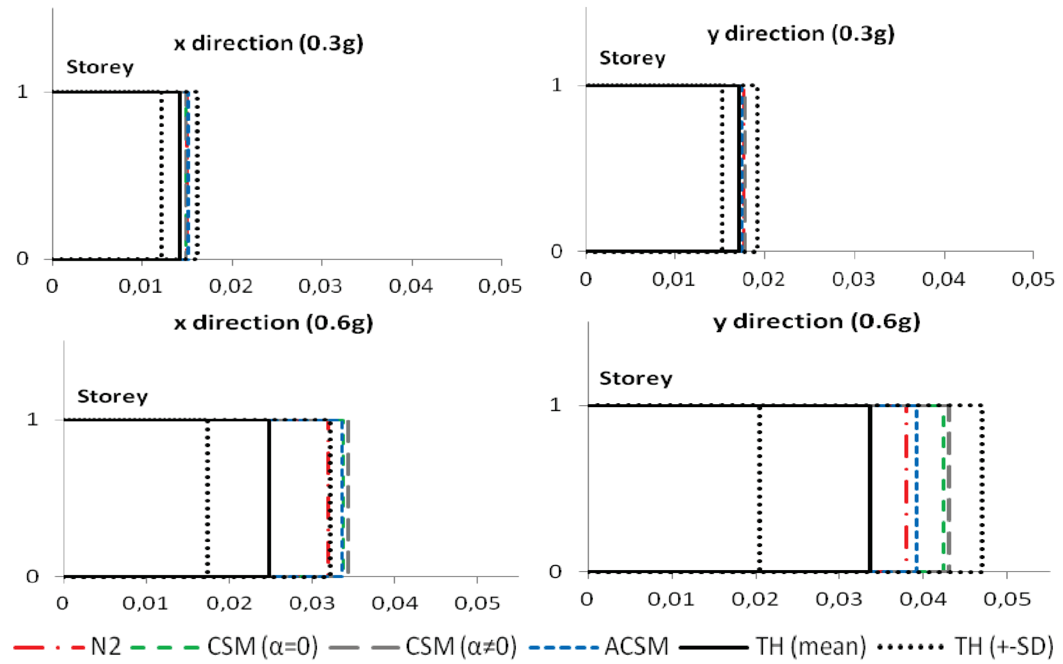

Figure 4.7 Storey drifts for the TR-PI structure in both direction of the analysis

4.3. Torsional responses

The torsional effects were estimated through the: a) normalization of the roof displacements with respect to the roof displacements of the corresponding reference structures (regular ones); b) normalization of the edges roof displacements (obtained at the instant of maximum displacement at the flexible edge) with respect to the corresponding centre of mass displacements. In the first case the results obtained for the plan irregular torsionally unrestrained and restrained structures are plotted in Figures 4.8 and 4.9, respectively.

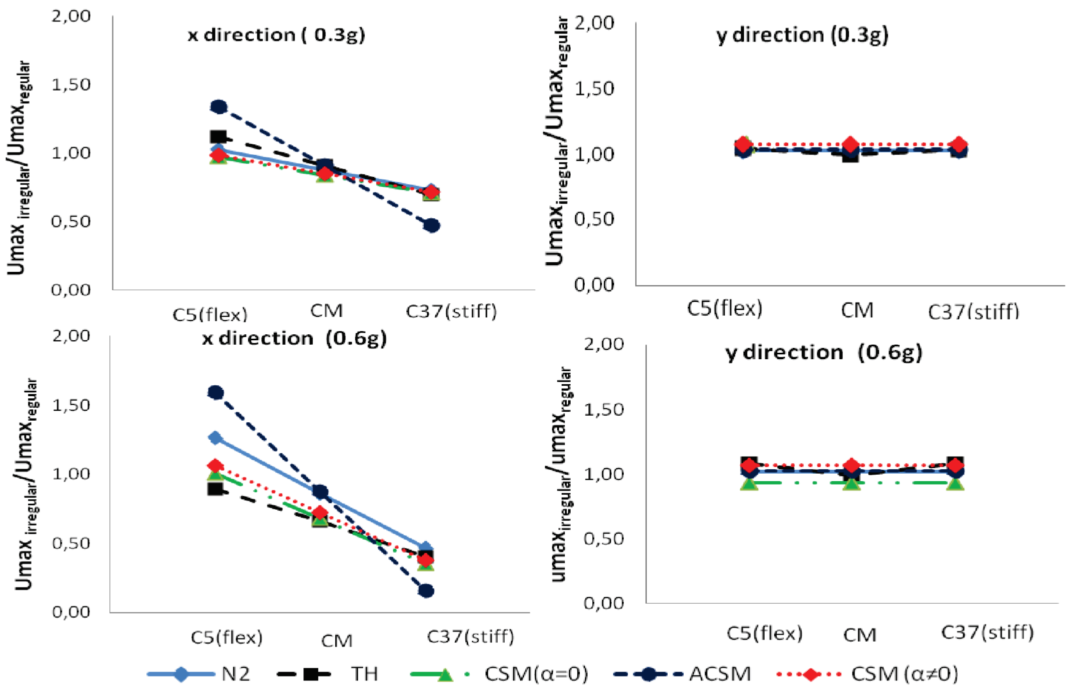

Figure 4.8 Normalized responses with respect to the reference structure (TU-PI structure)

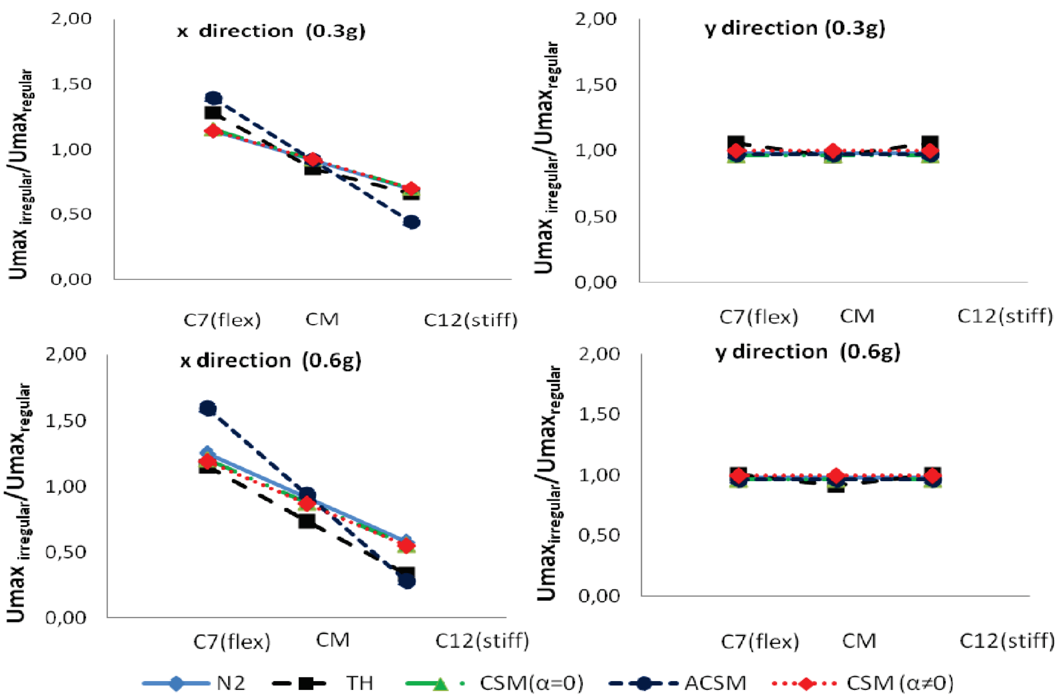

Figure 4.9 Normalized responses with respect to the reference structure (TR-PI structure)

As expected the results show that rotations occur when the load is applied in the direction of the eccentricities (direction x). It is also observed, especially in the direction of the eccentricities, that the displacements of the centre of mass of the regular structures are higher than the displacement of the irregular ones. This is due to the fact that the regular structures are more flexible in this direction. As indicated in Table 2.1, the periods of vibration in the x direction for the TU-PR and TU-PI structures are 0.66s and 0.60s, respectively, while for the TR-PR and TR-PI structures the periods are 0.65 and 0.60s, respectively. In the y direction the ratio between the displacement of the centre of mass of the irregular and regular structures is very close to unity because in this direction the structures have very similar dynamic characteristics.

Regarding the performance of the NSPs, although this normalization does not represent the deformed shape of the irregular structures, for the intensity level of 0.3g, it can be observed that N2 and CSM methods estimations, when compared with the time history results, for the flexible edge of the structures are lower while for the stiff edge are higher than the expected. The ACSM provides a relation of displacements at the flexible and stiff edges of the structures higher and lower than the time history results, respectively. For the intensity level of 0.6g, the N2 and CSM methods displacements relations at both edges of the structures are higher than the nonlinear time history relations while the ACSM displacements relation at flexible and stiff edges are lower and higher, respectively, than the expected results.

Figures 4.10 and 4.11 depict the normalized roof displacements, i.e. the edge roof displacements normalized to those of the centre of mass.

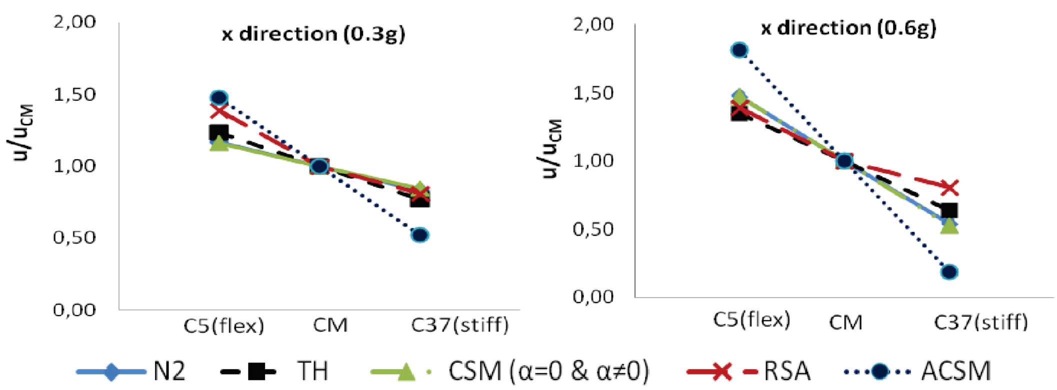

Figure 4.10 Normalized responses with respect to the CM (TU-PI structure)

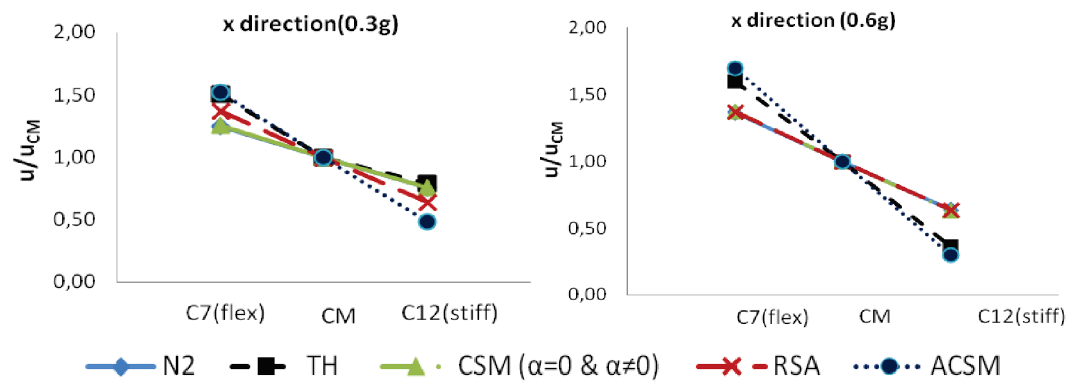

Figure 4.11 Normalized responses with respect to the CM (TR-PI structure)

The outcome of this normalization indicates, for the design intensity (0.3g), that the N2 and the CSM procedures underestimate the deformations at the flexible edge of both structures but overestimate the response at the stiff edge of the TU-PI structure and underestimate the response of the TR-PI structure. For the same level of intensity, the ACSM overestimates the results at the flexible edge and underestimates the results at the stiff edge of the structures. Regarding the intensity level of 0.6g, it can be concluded that the N2 and CSM methods overestimate the displacements at the flexible edge but underestimate the displacements at the stiff edge in the case of the TU-PI structure. In the case of the TR-PI structure, these methods underestimate the results at the flexible edge of (an observation that was not possible in Figure 4.9) and overestimate the results at the stiff edge of the structure. The ACSM method provides the same results as the obtained for the intensity level of 0.3g, however they seem more accurate in the case of the TR-PI structure.

4.4. N2 method versus Extended N2 method

The Extended N2 method proposed by Fajfar *et al.* (2005) was developed to capture the torsional effect in the seismic design/assessment of irregular structures. For this reason a comparative study was performed between this method and the original N2 method. The results obtained in the directions of interest are presented in Figures 4.12 and 4.13 for the torsionally unrestrained and restrained structures, respectively.

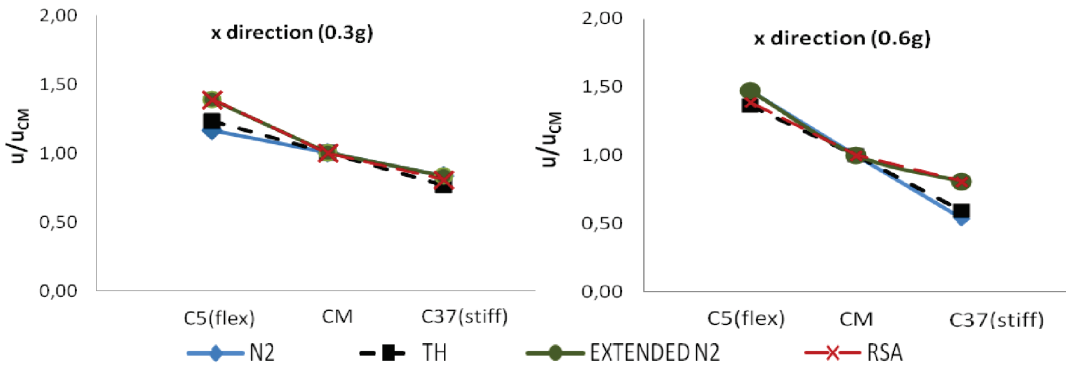

Figure 4.12 N2 versus Extended N2 (TU-PI structure)

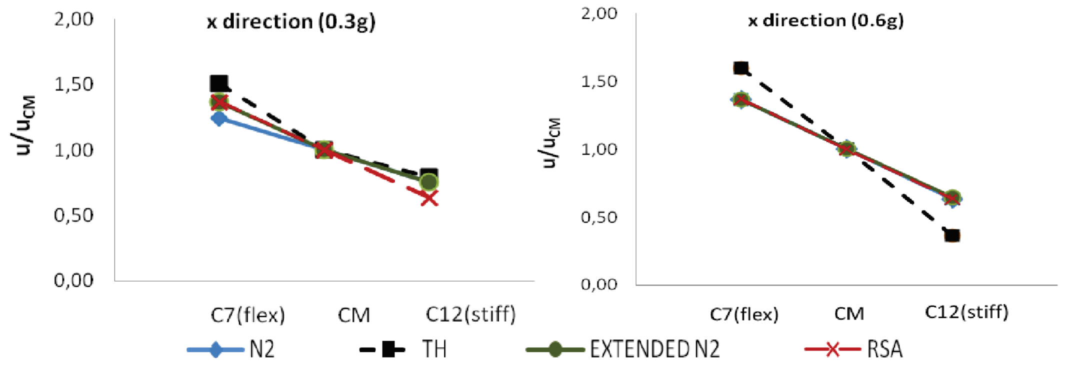

Figure 4.13 N2 versus Extended N2 (TR-PI structure)

In the case of the TU-PI structure, when the design intensity level is considered, it can be observed that the original N2 method underestimates the results at the flexible edge and overestimates the results at the stiff edge of the structure while the Extended N2 method overestimates the results for both edges of the structure. For the intensity level of 0.6g, both approaches overestimate the displacements at the flexible edge. At the stiff edge, the original N2 method underestimates the displacements but the Extended N2 overestimates the displacements of the structure. Regarding the TR-PI structure both variants of the method underestimate the results at both edges of the structure for the intensity level of 0.3g. If the intensity level of 0.6g is being analysed, the results are underestimated for the flexible edge and overestimated for the stiff edge of the structure, either for the original N2 and the Extended N2 methods. Although in some of cases the Extended N2 method still underestimates the results, especially for the case of the TR-PI structure, it should be noticed that the nonlinear time history results are mean values with significant dispersion (it was observed that the Extended N2 method results, whenever underestimated are within the standard deviation range).
Therefore, this comparative study between the original N2 and the extended N2 method allows concluding that is later predicts closer estimations to the results of nonlinear dynamic time-history.

5. CONCLUSIONS

In this study two plan-irregular steel structures were assessed in terms of seismic performance through the application of a group of nonlinear static procedures (N2 and Extended N2 methods, CSM with the features of FEMA440 and ACSM) with the purpose of testing these procedures effectiveness when compared with the nonlinear time-history analysis results. To quantify the torsional effects that arise due to the irregularities, normalizations were made with respect to the corresponding reference structures and to the centre of mass displacements. Regarding the translation responses it was observed that all the NSPs predictions are conservative. The analysis of the torsional parameters allowed concluding that the ACSM procedure overestimates the displacements at the flexible edge of the structure and underestimates the displacements at the stiff edge of the structures. The N2 and the CSM methods provide underestimated results regarding the flexible edge of the structures while for the stiff edge it depends of the structure configuration and the level of intensity considered.The comparative study between the two variants of the N2 method allowed concluding that the Extended N2 method predicts more accurate results than the original method. In general, for this particular group of structures, it seems reasonable to conclude that all NSPs lead to satisfactory results. However, due to the dispersion of the results, especially for the intensity of 0.6g, and the reduced number of structures studied, further parametric studies need to be performed in order to expand these conclusions.

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