An Extension of the CSM-FEMA440 to Plan Irregular Buildings

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SUMMARY:

The limitation of the commonly used Nonlinear Static Procedures (NSPs), including the ones recommended by the seismic codes, is their inability to capture the torsional behaviour of plan-irregular buildings. In this paper an extension of the CSM-FEMA440 is proposed to overcome the torsional problem in plan irregular buildings. This proposal was based on the results presented by Fajfar and collaborators and advocates that torsional amplification can be computed through the combination of a linear elastic analysis and a pushover analysis. The outcomes outlined by this team were taken to extend the N2 method to the case of buildings with this kind of irregularities. The case studies analysed in this work are three existing RC buildings with three, five and eight storeys, and irregular in plan. The torsional demands estimated by the proposed NSP are duly compared with the ones obtained with the most precise nonlinear dynamic analysis for several levels of seismic intensity. It was concluded that in the majority of buildings an upper bound of the torsional amplifications can be determined by a linear dynamic (response spectrum) analysis also in the inelastic range.

Keywords: Torsion, plan irregular buildings, nonlinear seismic response, pushover analysis, linear response spectrum analysis

1. INTRODUCTION

The use of NSPs on the seismic assessment or design of structures has increased in the last few years, backed by a large number of extensive verification studies that have demonstrated its relatively good accuracy in estimating the seismic response of regular structures (planar frames and bridges).
However, the extension of such use to the case of 3D irregular structures is not yet consolidated, therefore limiting the application of this simplified methods to assess actual existing structures, the majority of which do tend to be non-regular (Fajfar et al. 2005a) and b), Chopra and Goel 2004, D'Ambrisi et al. 2009, Erduran and Ryan 2010).
The major limitation of the existing NSPs, including the ones recommended by the seismic codes, is their inability to capture the torsional behaviour of plan irregular buildings. Generally they cannot capture the torsional effects distorting the real structural response.
Extensive parametric studies have been performed by Fajfar and his co-workers (Fajfar et al. 2005a) and b) in order to investigate the parameters that influence the inelastic torsional response of building structures. Based on the results obtained the following conclusions were taken, which are important for the development of simplified analysis methods and code guidelines:

- A conservative estimation of the amplification of displacements due to torsion in the inelastic range can be determined by a dynamic elastic analysis;
- Any reduction of displacements on the stiff side compared to the counterpart symmetric building, obtained from elastic analysis, will decrease or even disappear in the inelastic range.

These conclusions were used by Fajfar and his team (Fajfar et al. 2005a) and b) to develop an extension of the N2 method to plan irregular building structures.
In this paper, an Extended version of the CSM-FEMA440 for plan irregular buildings is presented based on the conclusions previously drawn by Fajfar and his team for the Extension of the N2 method.

This paper is a summary of the one already published (Bhatt and Bento 2011). The results obtained in three existing plan irregular buildings, using the extended CSM-FEMA440, are compared with the nonlinear dynamic median results for several levels of seismic intensity. Final conclusions are outlined in the end.

2. THE EXTENDED CSM-FEMA440

The extended version of the CSM-FEMA440 presented herein takes into account the contribution of a pushover analysis, with the calculation of the target displacement based on the FEMA440 recommendations, and the contribution of a linear response spectrum analysis in order to capture the amplification due to torsion. The reduction of demand due to torsion is neglected. The entire procedure can be summarized in the following steps:

1) Perform pushover analyses with positive and negative sign for each X and Y direction of a 3D numerical model. Compute the target displacement – displacement demand at the centre of mass (CM) at roof level – for each direction as the larger value of the + and – sign pushover. For this calculation use the CSM-FEMA440 recommendations;

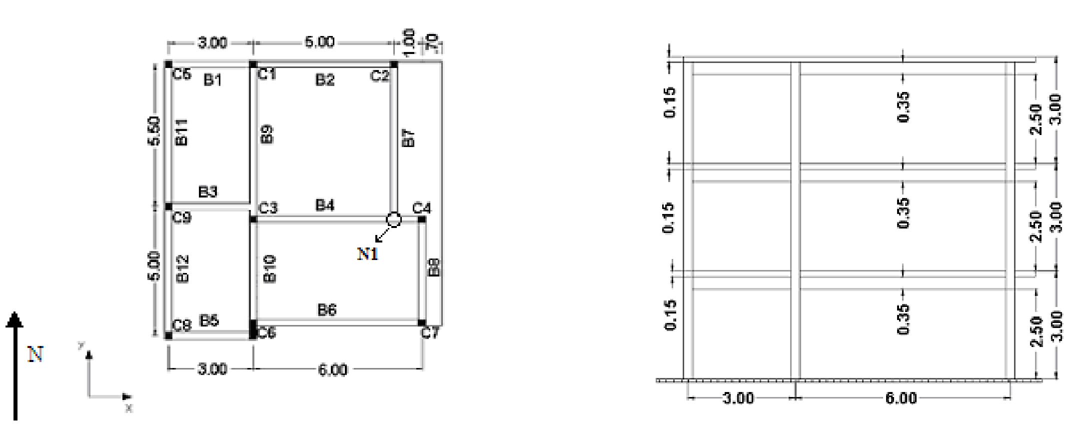
2) Perform a linear modal response spectrum analysis in two X and Y direction combining the results according to the SRSS rule;

3) Determine the torsional correction factors. This factor is computed doing the ratio between the normalized roof displacements obtained by the elastic response spectrum analysis and by the pushover analysis. The normalized roof displacement is obtained by normalizing the displacement value at a specific location with respect to those of the centre of mass (CM). If the normalized roof displacement obtained from the elastic response spectrum analysis is smaller than 1.0, one should consider 1.0 to avoid any favourable torsional effect (reduction of displacements) given by the elastic analysis;

4) Multiply the quantity under study for a certain location by the correction factor calculated for that location.

3. CASE STUDIES

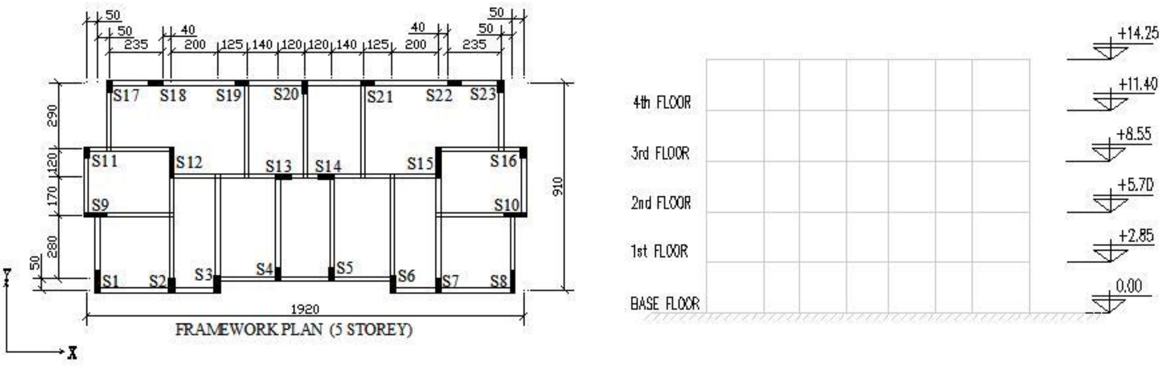
Three real plan-irregular RC buildings were analysed in this endeavour. The first case study is the three storey SPEAR building. It represents typical existing three-storey buildings in the Mediterranean region following Greece's concrete design code in force between 1954 and 1995. This structure was designed only for gravity loads based on the construction practice applied in the early 1970s that included the use of smooth rebars. A prototype was tested in full scale at Ispra within the European framework project SPEAR. Further details on the structure and its pseudo-dynamic testing can be found in (Fardis 2002, Fardis and Negro 2006). The SPEAR building is plan-irregular in both X and Y directions but it is regular in elevation (Figure 1). Total translational masses in the three storey building amounted to 67.3ton each for the first two floors and 62.8ton for the roof.



a) b)

Figure 1. Three storey building configuration: a) in plan; b) at the south west facade (units in meters)

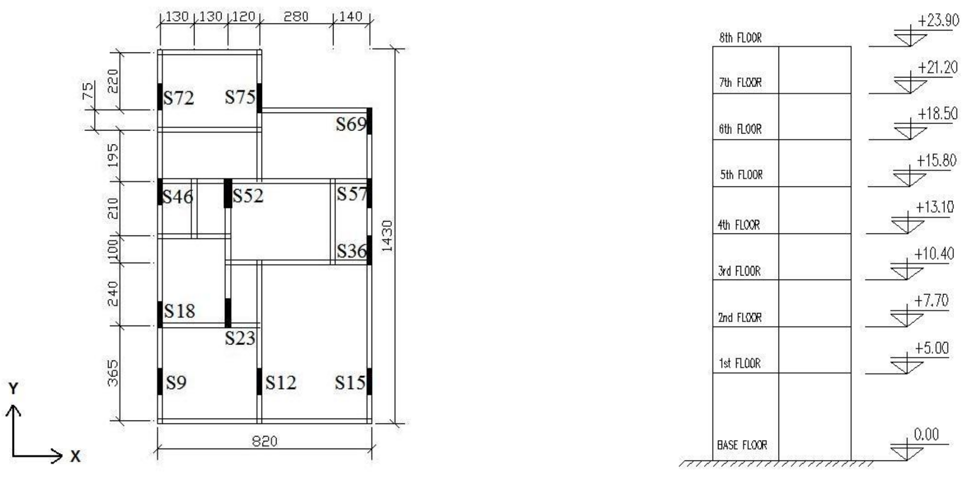
The second building selected for this work is a real Turkish reinforced concrete five storey building. It experienced the 1999 Golcuk earthquake without any damage.
The building is irregular along the X axis, Figure 2a), and all the floors have the same height, Figure 2b). There are beams framing into beams leading to possible weak connections in the structure. There are also walls and elongated columns, as presented in Figure 2a).



(a) (b)

Figure 2. (a) Plan View (cm), (b) Lateral View (m)

The columns sections keep the same geometrical and reinforcement features along the height of the building. The beam sections are mainly $0.20x0.50m^2$ except the two located in the centre of the building that are $0.20x0.60m^2$. The stirrups have 20cm spacing both for beams and columns. The slabs are 0.10m and 0.12m thick. For more details on the building's characteristics see (Vuran et al. 2008).
The mass of each storey is considered to be 263ton, except in the last storey where the mass is 150ton.
The third case study is an existing Turkish reinforced concrete eight storey building. It is a plan irregular structure – it is irregular along the X and Y axis, Figure 3a). The first storey height amounts to 5.00m and the other floors have the same 2.70m height, Figure 3b). There are beams framing into beams leading to possible weak connections in the structure. There are also walls and elongated columns, as presented in Figure 3a), with the higher dimension always along the Y direction. For this reason, the structure will be more stiff and resistant along the Y direction.
The columns sections and reinforcement keep the same geometrical features along the height of the building, except the column S52 that varies from $1.1x0.3m^2$ (on the first floor) to $0.8x0.3m^2$ (on the last floor). The height of this section is reduced in 0.1m at every two storeys.
The beam sections are mainly $0.20x0.50m^2$ except the two located in the centre of the building along the X direction that are $0.30x0.50m^2$ and $0.25x0.50m^2$ respectively. The slabs are 0.12m thick.



(a) (b)

Figure 3. (a) Plan View (cm), (b) Lateral View (m)

The mass in the first floor of the eight storey building is 73ton, in the upper storeys is 65ton and in the roof the mass is 56ton.
Both Turkish buildings were designed according to the 1975 Seismic Code of Turkey.
In this work, it is assumed that the structures are properly designed for shear, and therefore the collapse of the buildings is not due to brittle failures.

4. MODELING FEATURES

The analysis software adopted in this work was SeismoStruct (Seismosoft 2006), a fibre element based finite element program, capable of predicting the large displacement behaviour of space frames under static or dynamic loading, taking into account the inelastic behaviour of the materials as well as the geometric nonlinearities of the elements.
The 3D models representing the buildings under analysis were built using space frames assuming the centrelines dimensions. The inelastic behaviour of the structural elements was modelled using a fibre element model, with each fibre being characterised by the material relationships described below.
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 it was used a tangent stiffness-proportional damping. For the SPEAR building it was used a value of 2%, according to the experimental results at ISPRA, and for the Turkish buildings it was considered a 5% value.
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 and 16.7MPa for the Turkish buildings.
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 for the SPEAR building and 371MPa for the Turkish buildings.
The Nodal Constraints with Penalty Functions option was taken to model the rigid diaphragm effect in the Turkish buildings. The penalty function exponent used was 10^7. To model this characteristic of the slab in the SPEAR building it was used the Rigid Diaphragm with Lagrange multipliers modelling strategy. This option resulted from the calibration of the analytical model with the experimental results (Pinho et al. 2008).
The comparisons between the analytical results and the experimental tests for the SPEAR building can be found in Bento et al. (2010).

5. SEISMIC ASSESSMENT

In this section the parametric study is described in terms of seismic action definition and structural analyses performed.

5.1. Seismic Action

Seven bi-directional semi-artificial ground motion records from the SPEAR project fitted to the EC8 (CEN 2004) elastic design spectrum (Type 1 soil C) were used in the three storey building case.
For the five and eight storey buildings, combinations of three bi-directional semi-artificial ground motion records were applied. The three considered ground motions are real records from the PEER's database website (PEER 2009). They were fitted to the Eurocode 8 elastic design spectrum (with the Turkish code features – Type 1 soil A) using the software RSPMatch2005 (Hancock et al. 2006).
The ground motions were scaled and applied for a wide range of peak ground intensities in order to

assess the performance of the NSPs throughout different levels of structural inelasticity. The accelerograms were scaled for peak ground accelerations of 0.05, 0.1, 0.2 and 0.3g for the three storey building, to 0.1, 0.2, 0.4, 0.6 and 0.8g for the five storey building and to 0.1, 0.2 and 0.4g for the eight storey building. The median displacement response spectra of each set of ground motions were used to compute the nonlinear static procedures response.

5.2. Structural analyses

In pushover analyses, lateral forces were applied to the structure in the form of modal load pattern. The loads were applied independently in the two horizontal positive/negative directions, resulting in four analyses. For each one, the target displacement was computed with the larger value in each direction being chosen. The final results were combined in the two directions using the SRSS combination.
For the nonlinear dynamic analysis of the three storey SPEAR building the aforementioned seven bidirectional semi-artificial ground motion records were applied in 4 different configurations: X+Y+, X+Y-, X-Y-, X-Y+.
For the nonlinear dynamic analysis of the Turkish buildings, the abovementioned three bidirectional semi-artificial ground motion records were used. Each record was applied twice in the structure changing the direction of the components, resulting in six models, each one with five intensity levels for the five storey building and three intensity levels for the eight storey building.

6. ANALYSES RESULTS

As previously referred, the proposed procedure is applied to the three, five and eight storey buildings under analysis and compared with the original CSM-FEMA440 and with the median *time-history* response.
When dealing with plan-irregular buildings, the normalized roof displacements is a measure that gives a good illustration of the torsional behaviour of the structure. This measure is obtained by normalizing the edge displacement values with respect to those of the centre of mass. Several plots are presented in the following showing the performance of the analysed procedure in estimating the torsional motion of the evaluated buildings, Figure 4 and 5. In each of the subsequent plots, RSA represents the results of the elastic response spectrum analysis and TH the median results of the *time-history* analysis.

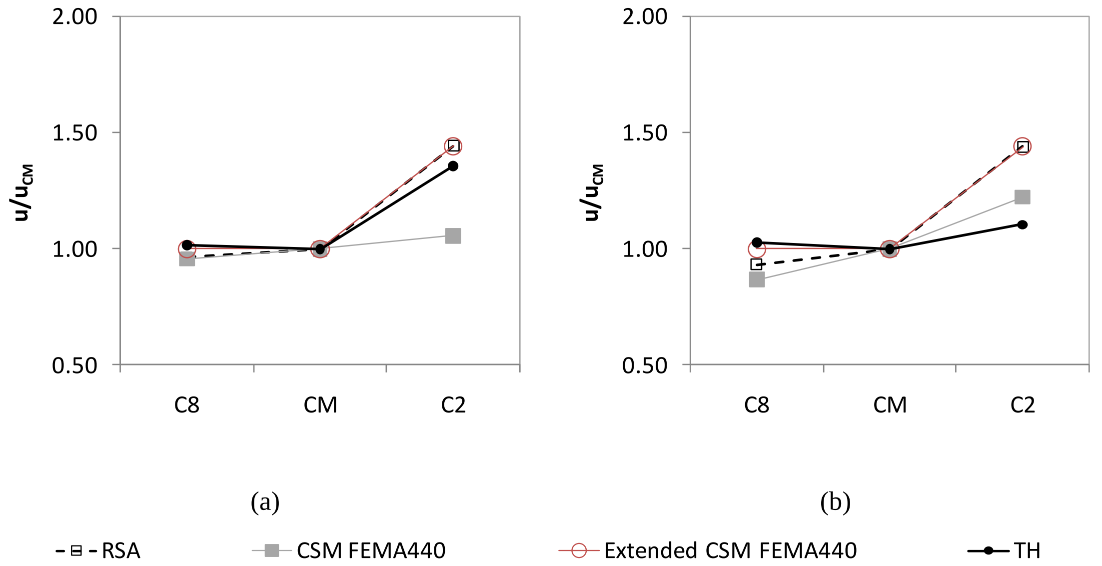


Figure 4. Normalized roof displacements, three storey building a) X direction, 0.1g; b) Y direction, 0.2g

Figure 6a) and 7 illustrate the lateral displacement, the interstorey drifts and the chord rotation profiles on the flexible edge, column C2, of the three storey building, in both X and Y directions through different levels of inelasticity.

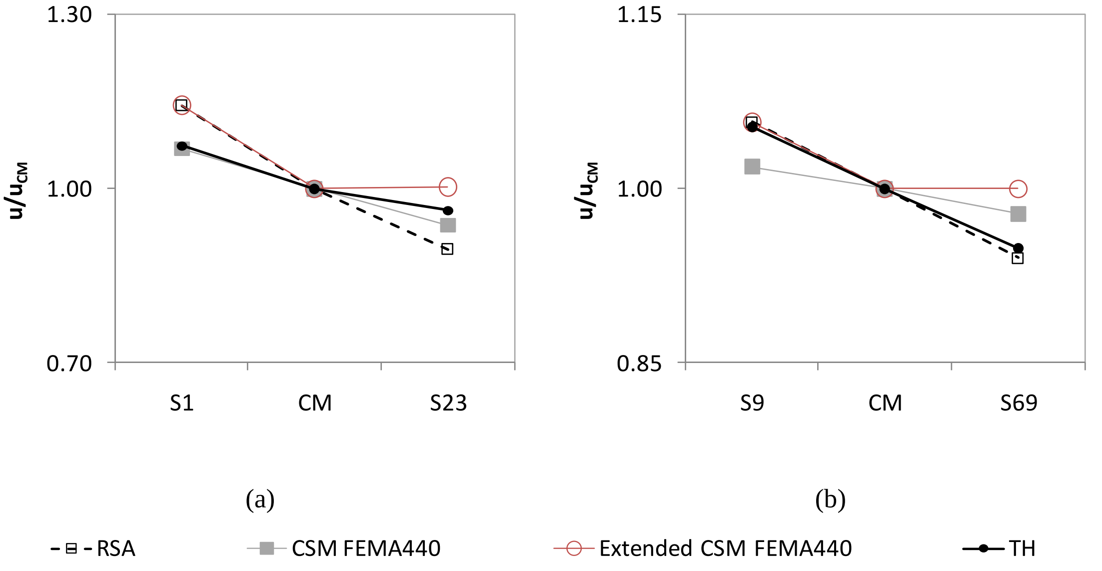


Figure 5. Normalized roof displacements a) five storey building, X direction, 0.4g; b) eight storey building, X direction, 0.2g

In order to get a quick overview of how the different NSPs perform, ratios of the values obtained with the latter for top displacements and the corresponding median estimates coming from the nonlinear dynamic analysis (Eq. 6.1) are computed. Ideally one would desire such ratios to tend to unity. Figure 6b) shows the top displacement ratios on column C2 of the three storey building.

$$\textit{Top Displacement ratio} = \frac{\textit{NSP's top displacement}}{\textit{Time history median top displacement}} \tag{6.1}$$

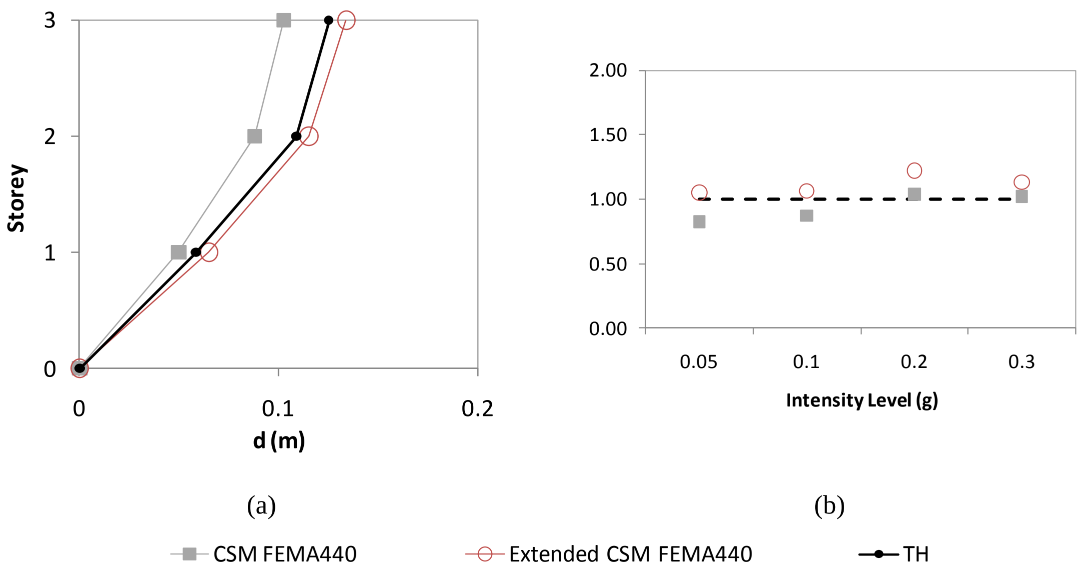


Figure 6. Three storey building, column C2: a) Lateral displacement profiles, X direction, 0.2g; b) Top displacement ratios, Y direction

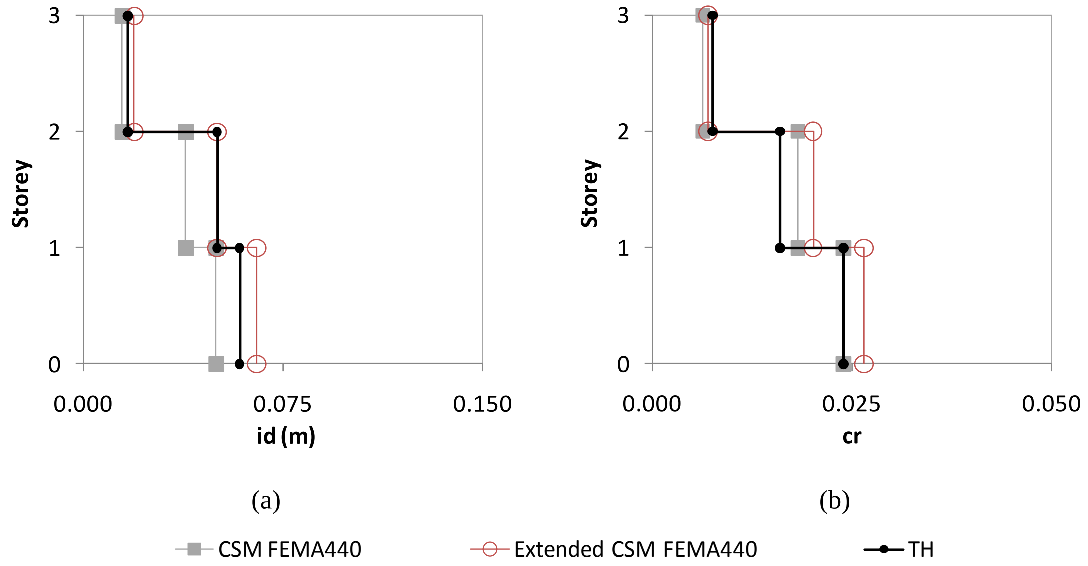


Figure 7. Three storey building, column C2: a) Interstorey drifts, X direction, 0.2g; b) Chord rotations, Y direction, 0.3g

From the results presented in this study one can conclude that the proposed Extended CSM-FEMA440 captures in a more accurate fashion the torsional motion of the analysed buildings, for all seismic intensities, than the original CSM-FEMA440 procedure.
From the plots presented above it is observed that the Extended CSM-FEMA440 adequately reproduces the torsional amplification in all buildings through all the seismic intensities in both directions. This happens because the method uses a correction factor based on a RSA which also leads to very good estimations of the torsional amplifications, as shown in the plots. The original CSM-FEMA440 generally underestimates the torsional amplification in the buildings.
The plots also show that both RSA and the original CSM-FEMA440 consider the torsional de-amplification. In some cases these methods led to underestimated results. On the other hand the Extended CSM-FEMA440 does not consider any positive effect due to torsion, as explained before in the description of the procedure, leading in some cases to very accurate results and in others to conservative responses. One can say that this is a safe criterion for designing. In fact it must not be forgotten that these simplified procedures are developed to be applied in design offices where the results should rather be conservative than almost close to *time-history* but slightly underestimated.
The results obtained herein seem quite optimistic regarding the implementation of this extended procedure in future codes, namely in the ATC guidelines. Nevertheless, more studies in different buildings should be developed in order to consolidate this nonlinear static approach.

7. CONCLUSIONS

In this paper, an Extended version of the CSM-FEMA440 was proposed in order to overcome the torsional problem of plan-irregular buildings. This extension was conceived based on the results presented by Fajfar and his team and used to extend the N2 method for plan irregular buildings.
The extended method consists on the application of a correction factor to the pushover results determined by the CSM-FEMA440 recommendations. These correction factors are computed based on a linear RSA and on a pushover analysis.
This procedure was assessed on three existing plan irregular buildings by comparing the results with nonlinear dynamic analyses for different levels of seismic intensity.
As far as the torsional effect is concerned it was possible to achieve more realistic and conservative results using the proposed extension than the original CSM-FEMA440 version for all the studied buildings.

The proposed extension should be further tested and, if the results came in line with the ones obtained for the three buildings studied herein, it should be incorporated in future codes, namely the ATC guideline, as a methodology capable of estimating the torsional amplification in plan-irregular buildings.

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