Using nonlinear static procedures for seismic assessment of the 3D irregular SPEAR building

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Abstract. This paper presents an appraisal of four nonlinear static procedures (CSM, N2, MPA and ACSM) employed in seismic assessment of plan-irregular buildings. It uses a three storey reinforced concrete plan-irregular frame building exemplifying typical older constructions of the Mediterranean region in the early 1970s that was tested in full-scale under bi-directional pseudo-dynamic loading condition at JRC, Ispra. The adequacy and efficiency of the simplified analytical modelling assumptions adopted were verified. In addition, the appropriate variants of code-prescribed NSPs (CSM and N2) to be considered for subsequent evaluation were established. Subsequent parametric studies revealed that all such NSPs predicted reasonably well both global and local responses, having the benchmark values been determined through nonlinear dynamic analyses using a suit of seven ground motions applied with four different orientations. The ACSM, however, predicted responses that matched slightly better the median dynamic results.

Keywords: Seismic assessment; 3D irregular SPEAR building; nonlinear static procedures; numerical simulation.

1. Introduction

The extension of Nonlinear Static Procedures (NSPs) to the case of plan-irregular structures has so far been the object of only restricted scrutiny (Chopra and Goel 2004, Fajfar *et al.* 2005, Pinho, Bento and Bhatt 2008, Bento and Pinho 2008), which effectively ends up by limiting significantly the employment of NSPs to assess actual existing buildings, the majority of which do tend to be irregular in plan. In addition, such few studies were typically concentrated on the application and verification of a single nonlinear static procedure only, rather providing a comparative evaluation among different available methodologies describing its relative accuracy and limitations.

In this work, therefore, four commonly employed nonlinear static procedures (CSM, N2, MPA, ACSM) are applied in the assessment of well-known SPEAR building – an irregular 3D structure tested pseudo-dynamically in full-scale under bi-directional seismic loading – with an objective to provide the necessary comparative evaluation of relative accuracy of different NSPs and their possible limitations of use, if any. The use of analytical model verified using experimental results further substantiates the adequacy of the evaluated procedures in predicting structural responses

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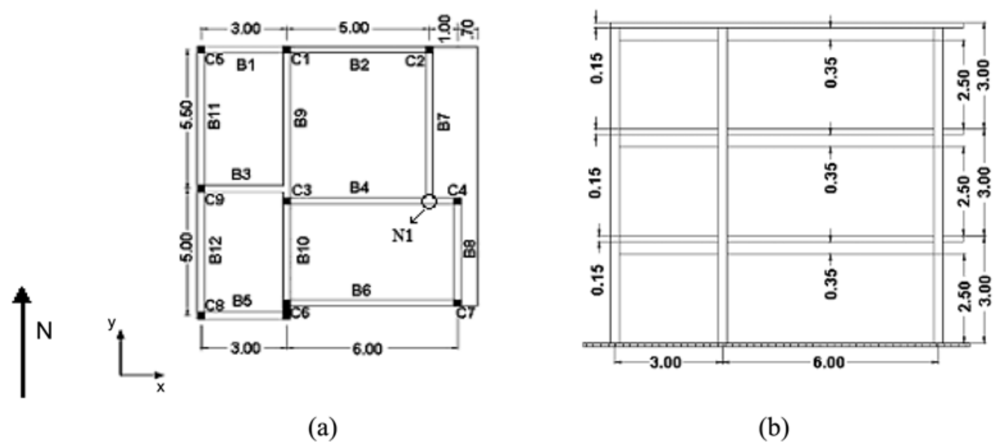


Fig. 1 Building configuration: (a) in plan, (b) at the south west facade (units meters)

under severe earthquakes. Note that this work somehow continues and complements the previous study by Fajfar and collaborators (Fajfar *et al.* 2005).

The analysed building represents a typical older three-storey reinforced concrete frame building constructed in the Mediterranean region following the construction practice and materials used in Greece in the early 1970s. The structure was designed only to gravity loads, with no provisions for earthquake resistance, as per the concrete design code implemented in Greece between 1954 and 1995. It was experimentally and numerically investigated in the SPEAR project (an European project within 6th framework). The details of which can be found elsewhere (Fardis and Negro 2006, Fardis 2002). A schematic plan and elevation of the building is presented in Fig. 1.

2. SPEAR building numerical modelling

2.1 General modelling strategy

The SPEAR building was modelled by an assemblage of inter-connected frame elements using centreline dimensions and incorporating distributed material inelasticity through displacement based formulation along with geometric nonlinearity utilizing corotational formulation. The storey heights amounted to 2.75 m for first floor and 3.00 m for upper stories. Each element was discretized into four sub-elements with two integration points each. Fiberized cross-sections – representing sectional details such as cover and core concrete and longitudinal reinforcements – were then defined at respective integration points, whereby every fiber was assigned to an appropriate material constitutive relationship, as described below. The sectional responses were obtained by integrating the material responses across a section using mid-point rule, whilst element-level responses were determined through Gauss-Lengendre integration scheme using section responses at integration points within the element. Further discussions can be found in (Calabrese *et al.* 2010).

In order to keep the analytical model simple, the effect of beam-column joints, slippage and pull-out of smooth reinforcing bars etc. were not included in the model. Exclusion of these effects, as shown later, did not affect much on the accuracy of the results as verified from experimental data, but reduced significantly the analysis time which allowed to perform large parametric study as required in the present work. Further details about numerical modelling can be found elsewhere (Meireles *et al.* 2006). A freely downloadable fibre element based finite element program SeismoStruct (SeismoSoft 2006) was employed to perform all the aforementioned nonlinear static procedures as well as nonlinear dynamic analyses.

2.2 Materials

The constitutive relationship proposed by Mander *et al.* (1998) along with the cyclic rules introduced by Martinez-Rueda and Elnashai (1997) was deployed to model the behaviour of unconfined concrete. In absence of sufficient transverse reinforcement, the confinement effects were not considered for core concrete. The mean compressive strength of unconfined concrete was taken as 25 MPa.

The constitutive model used for the steel was proposed by Menegotto and Pinto (1973) including the modifications due to isotropic hardening proposed by Filippou *et al.* (1983). An average yield strength of 360 MPa and ultimate strength of 450 MPa were assumed for reinforcements.

2.3 Mass and loading

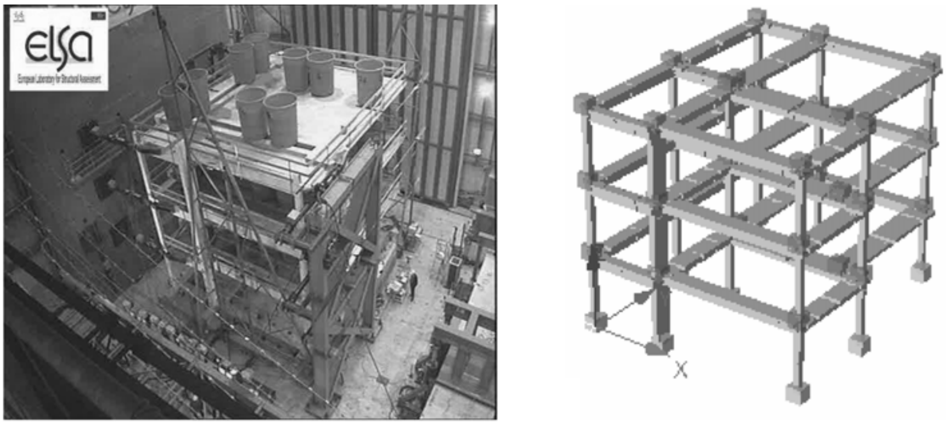
A lumped mass modelling strategy was adopted, in which masses were lumped at the nodal points according to its tributary area. Total translational masses amounted to 67.3 tonnes each for first two floors and 62.8 tonnes for the roof. The eccentric disposition of centre of rigidity (CR) with respect to centre of mass (CM) of each floor, by distances of 1.3 m and 1.0 m along the $x-$ and $y-$ axis respectively, effectively rendered the structure as irregular, according to the criteria set forth by Eurocode 8 (CEN 2004). The sustained gravity loads were automatically computed by the software, using the defined masses.

2.4 Diaphragm Modelling

The floor slabs of the building possessed very high in-plane stiffness compared to the out-of-plane (flexural) one and thus can safely be modelled as 'rigid diaphragm'. In the present work, such diaphragms were modelled by imposing kinematic constraints on the lateral displacements of all nodes at each floor so that they (nodal displacements) can be expressed by three rigid body motions of the respective floors, namely two horizontal translations and one rotation about the normal to the floor-plane. This reduces significantly the number of dynamic degrees of freedom and hence increases the efficiency for large parametric studies. The effects of flexural stiffness of slab were considered by assigning appropriate flange widths to the beams. Further details about relative accuracy of other slab modelling approaches can be found elsewhere (Pinho, Bhatt, Antoniou and Bento 2008).

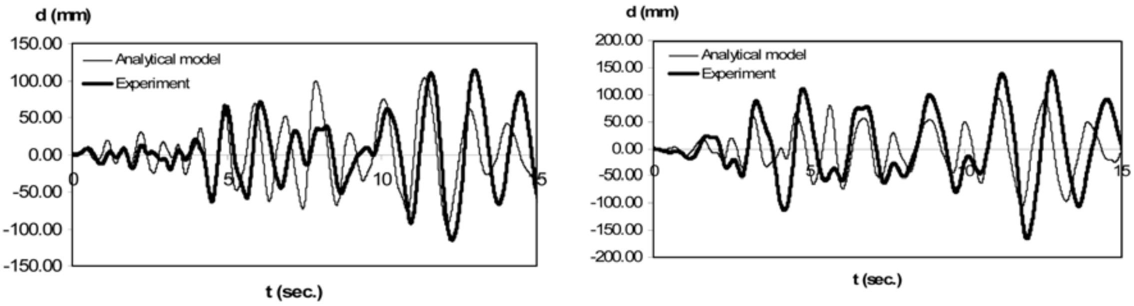
3. Comparison with experimental results

The SPEAR building was pseudo-dynamically tested (Fig. 2(a)) with a bi-directional loading based on a ground motion recorded at Hercegnovi station during the 1979 Montenegro earthquake and scaled to match with the EC8 type I spectrum for soil type C. This bi-directional record was



(a) (b)

Fig. 2 SPEAR building: (a) Experiment specimen, (b) Analytical model



(a) (b)

Fig. 3 (a) Top displacement in node C7, x-direction, (b) Top displacement in node N1, y-direction

applied to the structure in three runs of linearly increasing intensity of peak ground acceleration (pga), such as 0.02 g, 0.15 g and 0.20 g. The very same input motion was used to authenticate the adequacy of the current analytical model, so that the same model can be used for further parametric study with confidence.

Fig. 3 shows a comparison between experimental and analytical results for displacement histories at two orthogonal directions. Despite being a simplified analytical model, it reproduced the experimental results with appreciable accuracy. The observed discrepancies are due to the lack of various modelling aspects, like beam-column joints, slippage and pull-out of smooth reinforcing bars etc., incorporation of which into the current model would increase considerably the computation time and thus prevents from performing a large parametric study as required in this work. The current analytical model provides perhaps the best trade-off between accuracy and efficiency; and therefore used for the further parametric study.

The software used in this study takes into account both material inelasticity and geometric nonlinearities, although the authors tend to feel that the P-delta effects are likely not to play a critical role in the response of structures such as the one considered in this study.

4. Seismic assessment of SPEAR building - parametric study

4.1 Seismic input

A suit of seven bi-directional accelerograms, summarized in Table 1, scaled to match with the EC8 Type I design spectrum for soil type C, used in the original SPEAR project, was used for the nonlinear dynamic analyses. These ground motions were further scaled for different intensity levels of peak ground accelerations, namely 0.05 g, 0.10 g, 0.20 g and 0.30 g to assess whether NSP-predictions are sensitive to the intensity levels. The performance points for NSPs were computed using the median of the 7 ground motions response spectra in each direction. Fig. 4 depicts the correspondent median spectra, compatible with the EC8 design spectrum.

4.2 Considered Nonlinear Static Procedures (NSPs)

The nonlinear static procedures can be categorized into two major groups. The first group comprises the pioneering simplistic procedures, namely the Capacity Spectrum Method (CSM),

Table 1 Ground motion records considered (Fardis and Negro 2006)

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

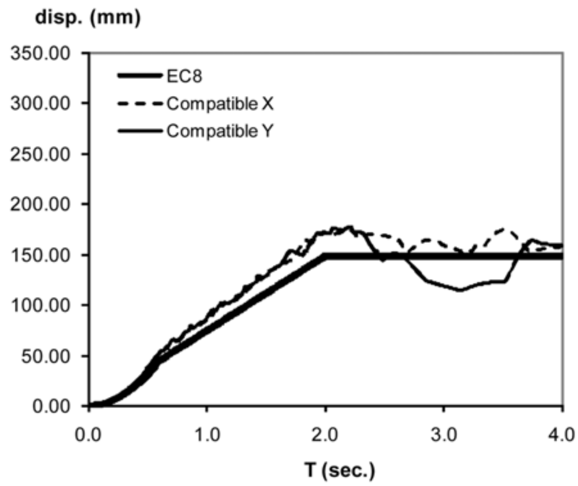


Fig. 4 Median displacement spectra compatible with the accelerograms used for 0.2 g

introduced by Freeman and collaborators (1975 and 1998) and implemented in ATC-40 guidelines (1996), and equally innovative the N2 method suggested by Fajfar and co-workers (1988 and 2000) and included later in Eurocode 8 (CEN 2005). These procedures are similar in deriving pushover/capacity curve – usually employ an invariant single load vector preferably proportional to the fundamental vibration mode shape – and differ in determining performance point – the N2 method uses constant ductility inelastic spectrum whilst the CSM utilizes overdamped elastic spectrum to account for the hysteresis energy dissipation. Both methods were updated recently and thus two variants of each method were evaluated through a preliminary investigation. The first variants are the original versions represented here as N2 and CSM-ATC40. The second variants feature updated versions, namely the N2 torsion and CSM-FEMA440. The N2 torsion method (Fajfar *et al.* 2005) incorporates the torsional effects due to plan-asymmetry through a correction factor which the original N2 procedure lacks. The CSM-FEMA440 variant, on the other hand, features an enhanced spectral scaling approach (ATC 2005) compared to its original edition.

The second group comprises the improvement-intended procedures, whereby capacity curve(s) is/are computed either by performing multiple pushover analyses using invariant/adaptive modal force vectors, such as the Modal Pushover Analysis (Chopra and Goel 2002, 2004) and the Adaptive Modal Combination Procedure (Kalkan and Kunnath 2006), or by single pushover analysis using adaptive displacement vector encompassing the effects of higher modes of vibration and progressive damage at each pushover step, such as the Adaptive Capacity Spectrum Method (Casarotti and Pinho 2007, Casarotti *et al.* 2007). Since the adaptive modal combination procedure has developed for planar frames only, it is not considered for the present parametric study.

4.3 The analyses

Two different classes of nonlinear analyses were performed on the analytical model of the SPEAR building. First set represents a series of nonlinear static analyses using two different algorithms, namely conventional pushover analysis and displacement based adaptive pushover analysis (Antoniou and Pinho 2004). The conventional pushover analyses were performed for different load patterns as required by the different NSPs. For example, the CSM requires fundamental mode shape proportional load vector, the N2 method calls for three different load vectors, namely proportional to fundamental mode shape, proportional to mass distribution up the height and envelope of the two, and the MPA warrants load vectors proportional to the vibration mode shapes that contribute significantly to the structural responses. The ACSM, on the other hand, employs displacement based adaptive pushover analysis whereby a single displacement vector is imposed on the structure at all the locations where lumped masses are defined. Each pushover analysis, be it conventional or adaptive, was performed independently for two orthogonal translational directions with positive and negative signs of load vector, which resulted four analyses for each pushover load case. The performance points were computed for each pushover curve and the maximum value in each direction was chosen as being the required performance point for the NSP under consideration. The results were then combined in the two directions using the SRSS combination.

The nonlinear dynamic analyses, similar to the static analysis, were performed for four different orientations of aforementioned suit of seven semi-artificial ground motions, namely X+Y+, X+Y-, X-Y-, X-Y+, for a set of varying intensity of 0.05 g, 0.10 g, 0.20 g and 0.30 g. The median response among all the analyses (7 ground motions x 4 orientations = 28 analyses) for each intensity level were considered as the 'true' response of the building.

5. Seismic assessment of SPEAR building – parametric study results

This section presents some representative parametric study results at locations shown schematically in Fig. 5, where SE represents the stiff edge, FE the flexible edge and CM the centre of mass. A central node corresponding to the central column, is marked as node C.

5.1 Preliminary evaluation

As mentioned earlier, two variants of N2 and CSM were appraised with an objective to identify the best possible option for the further parametric study. The evaluation was based on the prediction of interstorey drift and storey displacement profiles in addition to the roof displacement pattern in plan normalized by the corresponding centre of mass displacement. In the following some representative plots are presented.

5.1.1 N2 versus N2 torsion

Fig. 6 shows comparative interstorey drift plots for two orthogonal directions at the flexible edge (FE) with two different intensity levels. Although both variants of N2 underestimated the 'true' responses, it is the torsional variant of N2 that predicted closer estimation. Moreover, both variants

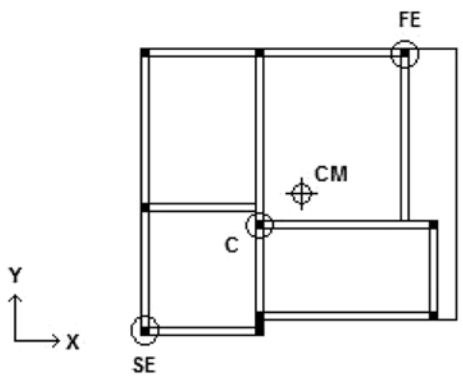


Fig. 5 Monitored nodes

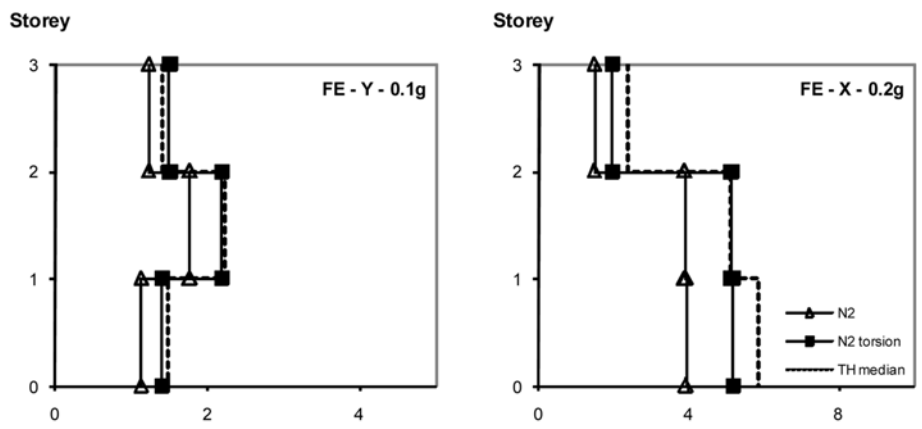


Fig. 6 Interstorey drifts (cm)

captured the profile shape, except first floor in the X-direction, very well, the magnitudes were different though.

Fig. 7 compares NSP-prediction of roof displacements at edges along X-direction normalized by the corresponding displacement at centre of mass with the 'true' dynamic responses. This gives an insight how well a NSP predicts the torsional rotation. Similar to the pervious observation, the torsional variant gave conservative prediction at either side, flexible and stiff edges, whilst original N2 underestimated significantly the flexible edge displacement.

Fig. 8 presents storey displacement profiles at flexible edge for two different intensity levels. The prediction of torsional variant of N2 once again matched very well with the 'true' dynamic profiles as compared to the original version. Consequently, the torsional variant of N2 procedure is used for further parametric study.

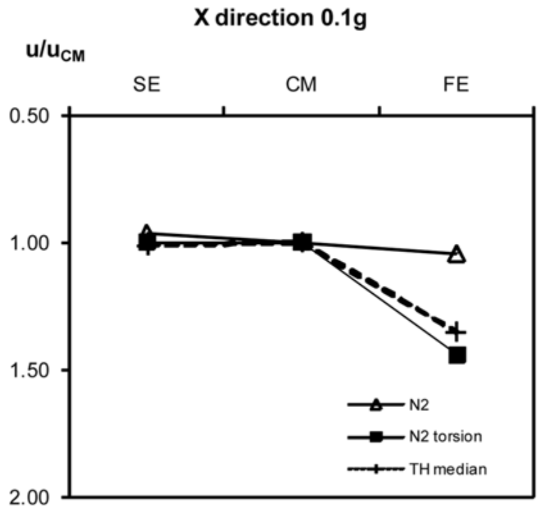


Fig. 7 Normalized top displacements

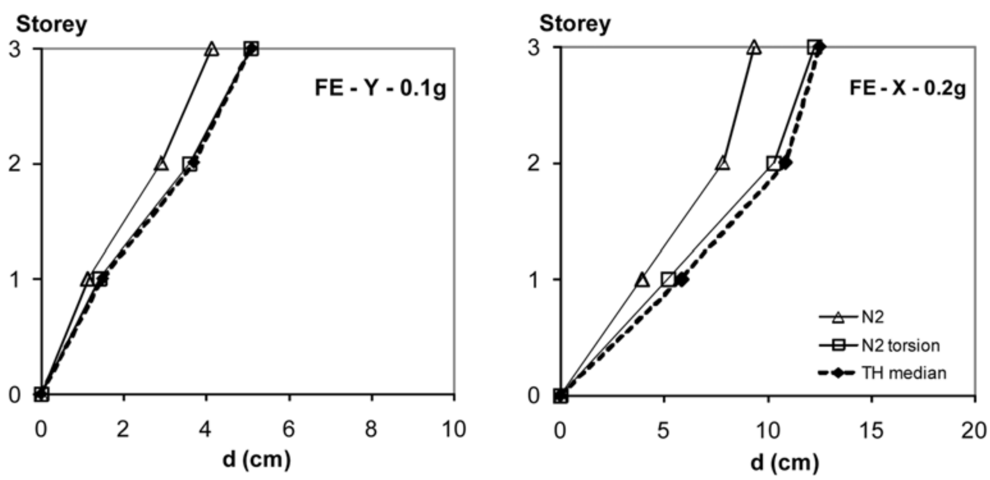


Fig. 8 Lateral displacements pattern

5.1.2 CSM-ATC40 vs. CSM-FEMA440

Fig. 9 demonstrates comparative plots of interstorey drift profiles at flexible edge and centre column. The improved version of CSM matched conspicuously well with the 'true' dynamic response both in shape and magnitude, whilst the original version of CSM underestimated significantly the interstorey drift profile.

Fig. 10 compares NSP-prediction of normalized roof displacements with the 'true' dynamic responses. There is no significant difference in prediction of both the variants. Both variants underestimated stiffer edge displacement and overestimated the flexible edge one.

As shown in Fig. 11, the improved version of CSM matched closely with the dynamic prediction of storey displacement profile at the centre of mass location. The original version of CSM, on the other hand, underestimated considerably the storey displacement profile. Consequently, the improved version of CSM, CSM-FEMA440, is used for subsequent parametric study.

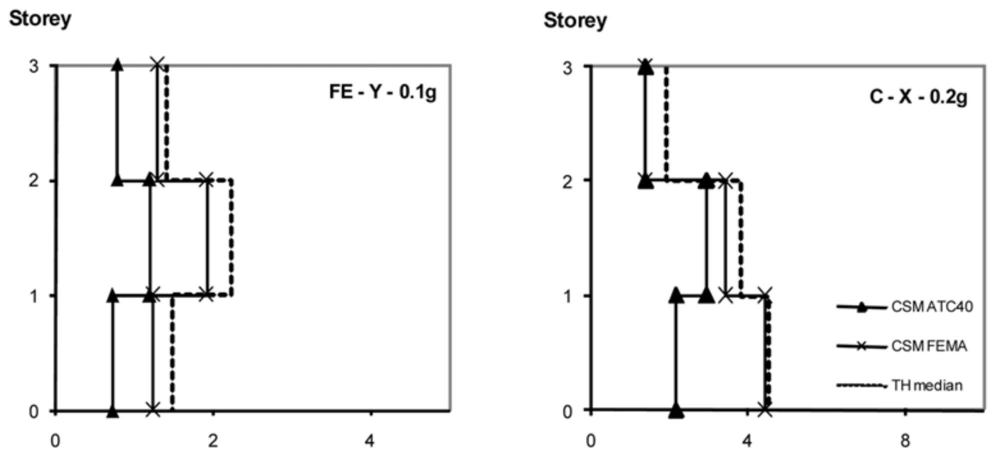


Fig. 9 Interstorey drifts (cm)

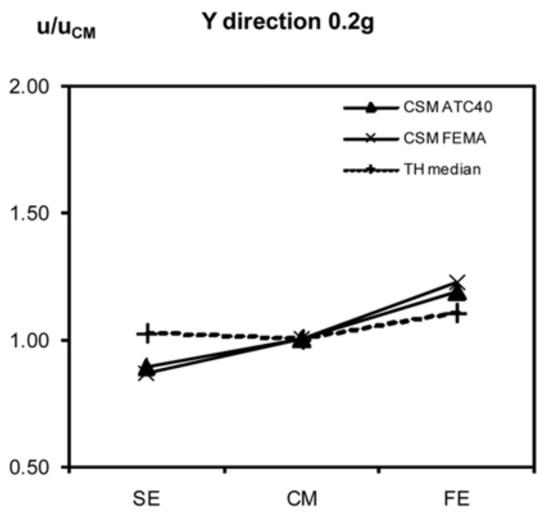


Fig. 10 Normalized top displacements

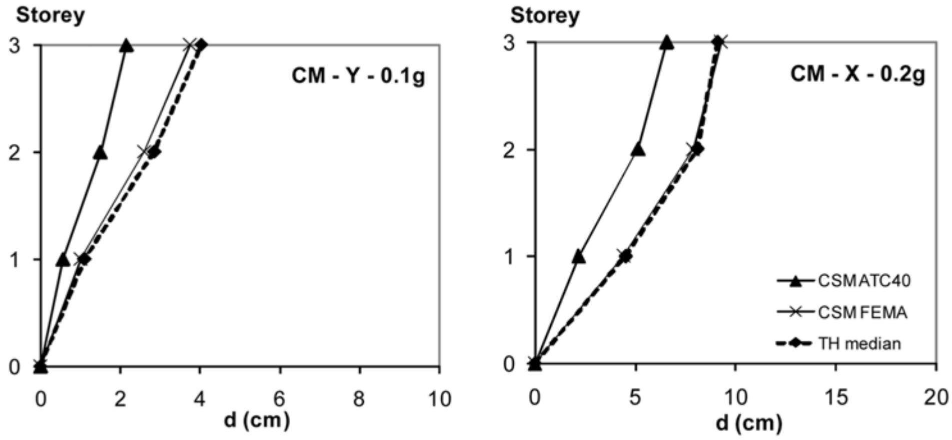


Fig. 11 Lateral displacements pattern

5.2 Evaluation based on global responses

Presented herein are the representative plots that provide an idea about relative accuracy of the NSPs under consideration. The NSPs were evaluated by comparing its predictions of global responses with the 'true' dynamic responses obtained from median over a suit of ground motions. Five responses were considered for this purpose, namely roof displacement, base shear, torsional rotation at roof level, storey displacement pattern and element shear force profile. Further results can be found in Pinho *et al.*(2008).

5.2.1 Roof displacements and base shears

The roof displacements and base shears determined from different NSPs were normalized by the corresponding median responses of dynamic analysis using a suit of seven ground motions, as shown in Eq. (1), which give an estimate of bias – how good or bad is the NSP under scrutiny for predicting that particular response – as the target reference value in ideal condition should simply be unity. An NSP is said to be biased towards underestimating the response if normalized response is less than one and overestimating the same if the ratio exceeds one. This provides a point of comparison among different NSPs.

$$\text{Normalized response} = \frac{\text{NSP-prediction}}{\text{Time history median}} \xrightarrow[\textit{ideally}]{} 1.0 \qquad (1)$$

Figs. 12 and 13 illustrate how prediction of normalized roof displacements from four aforementioned NSPs varies with increasing intensity levels. All the NSPs gave reasonably good predictions at centre of mass location, whereas biasness increased, albeit inconsistently throughout the intensity range, for normalized ratios at the flexible and stiffer edges. The CSM and MPA procedures underestimated considerably the 'true' responses at the edges for all the intensity levels except at 0.30g where these procedures overestimated the response at flexible edge. The N2 procedure underestimated the responses for most of the intensity levels and locations, even at the centre of mass location. The ACSM, on the other hand, predicted normalized roof displacement conspicuously well for all intensity levels except at 0.30 g where, it underestimated the same.

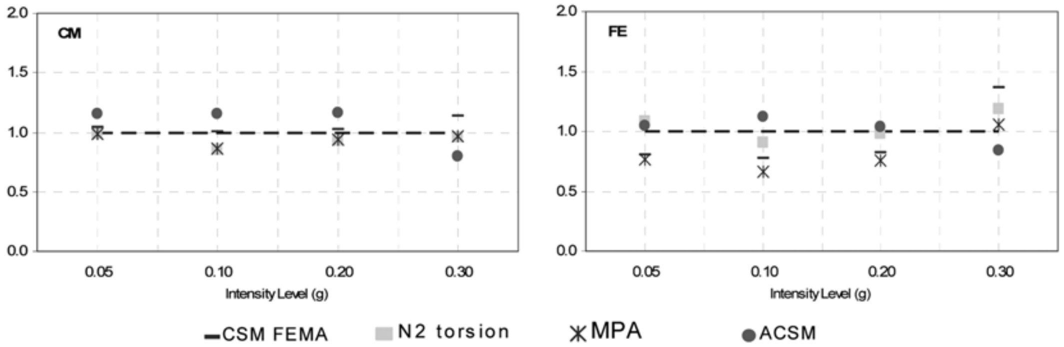


Fig. 12 Normalized roof displacements in the X direction

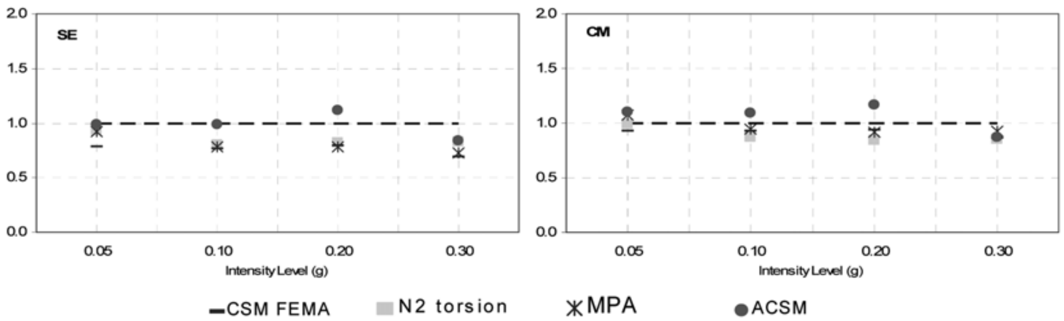
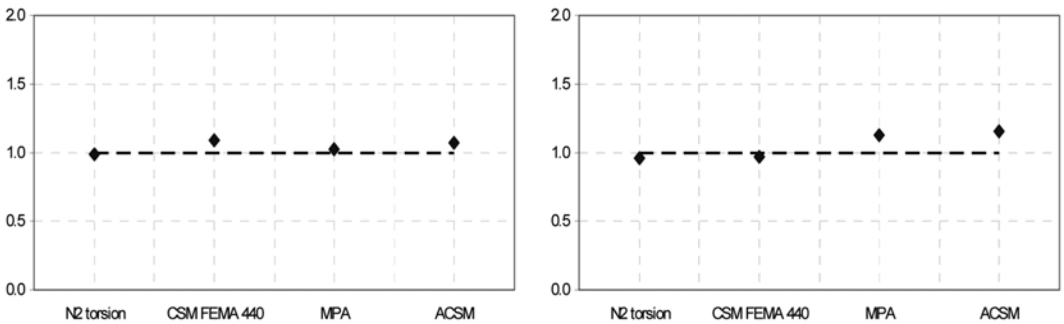


Fig. 13 Normalized roof displacements in the Y direction



(a) X direction, intensity 0.1g (b) Y direction, intensity 0.2g

Fig. 14 Normalized base shear

Fig. 14 demonstrates that all the NSPs predicted normalized base shear ratio appreciably well with a slight consistent overestimation by the ACSM.

5.2.2 Torsional rotation

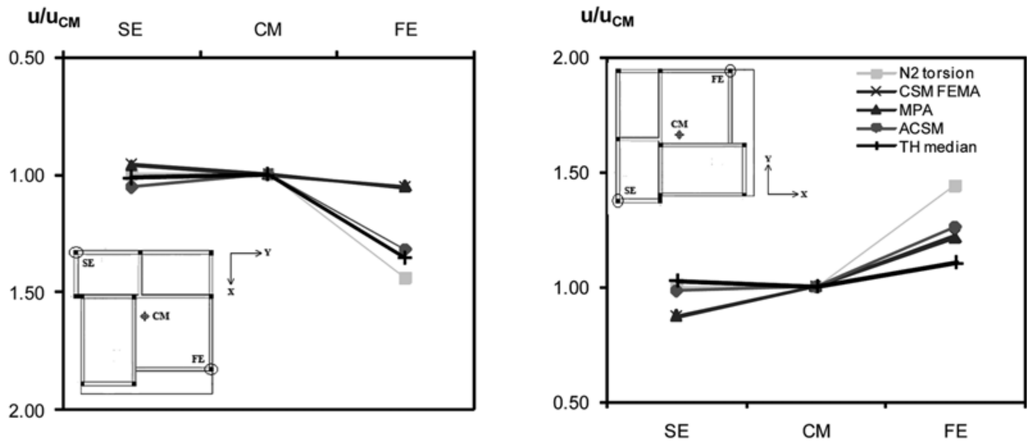
The pattern of roof displacements in plan normalized by the same at centre of mass, as shown in Fig. 15, gives an idea how torsional rotation changes the displacement demands at the edges. Comparing the NSP-predictions with the ‘true’ dynamic responses, it is observed that the ACSM and torsional variant of N2 captures reasonably well the dynamic response whilst the MPA and CSM underestimated the same considerably at some locations, for example flexible edge along X-direction, stiff edge along Y-direction.

5.2.3 Storey displacements pattern

Fig. 16 presents some representative plots of storey displacement profiles determined from different NSPs comparing with median profiles estimated from dynamic analyses. Although all the NSP predicted underestimated responses, only N2 and ACSM provided a closer match.

5.2.4 Element shear profile

Fig. 17 compares the element shear force profile at two critical edges for two different intensity levels. The ACSM, provided a closer match with the dynamic median response.



(a) X direction, intensity 0.1g (b) Y direction, intensity 0.2g

Fig. 15 Roof displacements normalized by CM displacement

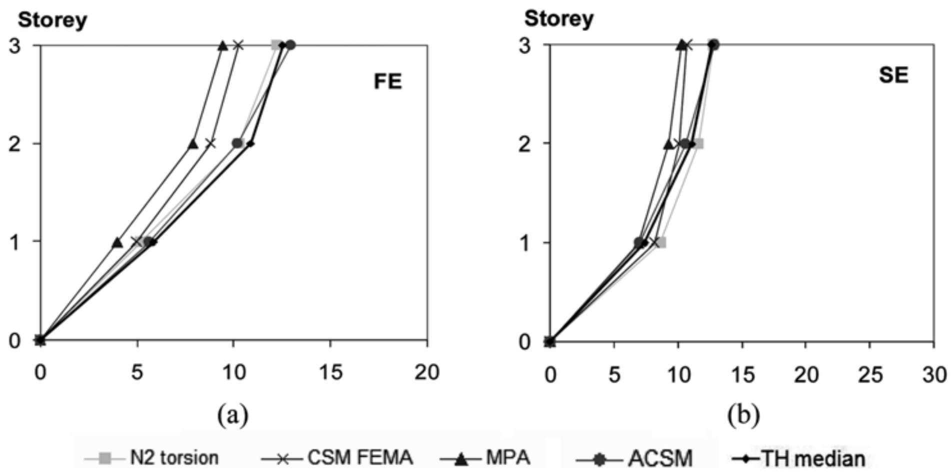


Fig. 16 Storey displacement profiles (cm): (a) 0.20g, X, FE, (b) 0.30g, X, SE

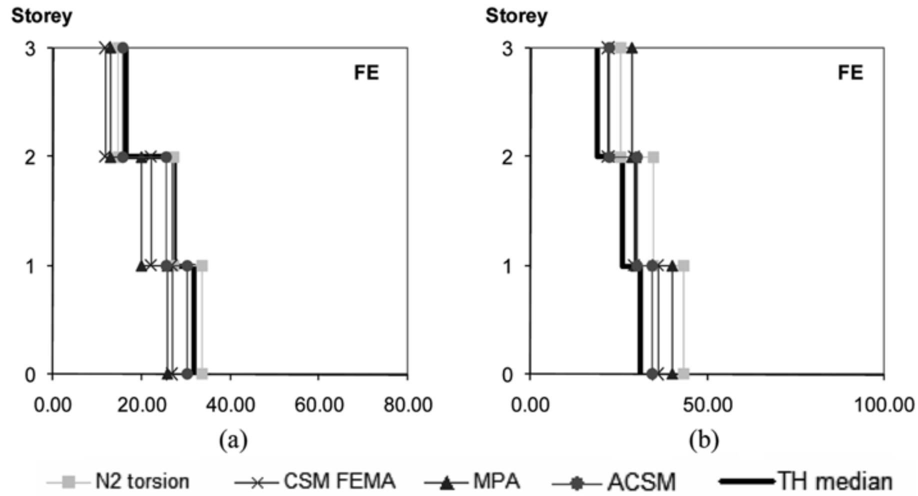


Fig. 17 Shear (kN) profiles: (a) 0.1g, X, FE, (b) 0.2g, Y, FE

5.3 Evaluation based on local responses

Finally, the NSPs were appraised following the code prescriptions (e.g., Eurocode 8) based on the local responses, such as interstorey drift, chord rotation etc. Accordingly, the number of elements that exceeded their capacities set forth in the Eurocode 8 corresponding to desire performance limit state, such as Damage Limitation (DL), are compared in a tabular form. Some representative plots showing the deviations of NSP-predictions from the code-prescribed limits are presented as well for the sake of completeness of this study. Note that local capacities corresponding to the damage limit state as recommended in Eurocode 8 are same as that in ATC-40 report.

5.3.1 Storey drifts

Tables 2 and 3 summarize the number of elements that exceeded their storey drift capacities at

Table 2 Number of elements exceeding EC8's DL: storey drifts, X direction

	N2 torsion	CSM-FEMA440	MPA	ACSM	Time history median
0.05g	0	0	0	0	0
0.10g	0	0	0	1	1
0.20g	4	3	3	6	6
0.30g	6	6	6	6	6

Table 3 Number of elements exceeding EC8's DL: storey drifts, Y direction

	N2 torsion	CSM-FEMA440	MPA	ACSM	Time history median
0.05g	0	0	0	0	0
0.10g	0	0	0	0	0
0.20g	2	3	3	6	6
0.30g	6	6	6	6	6

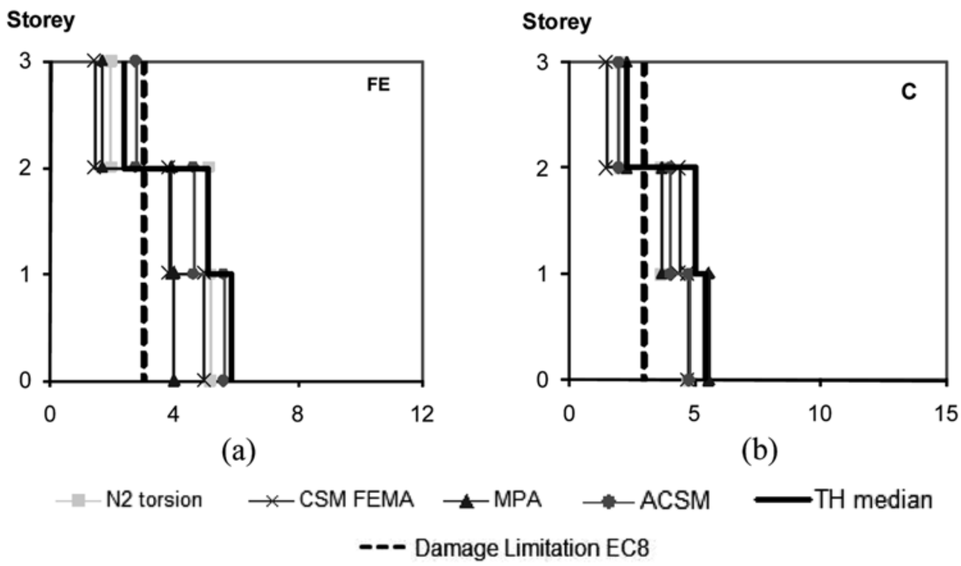


Fig. 18 Storey drifts (cm) in columns: (a) 0.2g, X, FE ; (b) 0.3g, Y, C

DL limit state for different levels of seismic intensity determined from different analysis procedures. Similar to the previous comparisons, median of dynamic analyses provides the benchmark numbers. At high intensity level of 0.30 g, all the NSPs predicted very well the number of elements exceeding DL capacities, whereas at moderate intensity levels, 0.10 g and 0.20 g, it is only ACSM that matched well with the prediction of dynamic analysis. Fig. 18 shows how well ACSM followed the prediction of median dynamic response. Note that all the procedures predicted correctly the number of elements exceeding DL capacities; it is only ACSM that provided results similar to the median dynamic response.

5.3.2 Chord Rotations

Tables 4 and 5 present the number of elements that exceeded their chord rotation capacities at DL limit state as stipulated in Eurocode 8 for increasing seismic intensity levels determined from different aforementioned analysis procedures. Similar to the previous observations, all the NSPs emulated dynamic prediction only at high intensity level of 0.30 g, whilst at moderate intensity levels, 0.10 g and 0.20 g, only ACSM followed the same. Fig. 19 substantiates this observation further.

Table 4 Number of elements exceeding EC8's DL: chord rotations, X direction

	N2 torsion	CSM-FEMA440	MPA	ACSM	Time history median
0.05g	0	0	0	0	0
0.10g	1	0	0	1	1
0.20g	5	3	4	5	6
0.30g	6	6	6	5	6

Table 5 Number of elements exceeding EC8's DL: chord rotations, Y direction

	N2 torsion	CSM-FEMA440	MPA	ACSM	Time history median
0.05g	0	0	0	0	0
0.10g	0	0	0	0	0
0.20g	3	3	3	6	6
0.30g	6	6	6	6	7

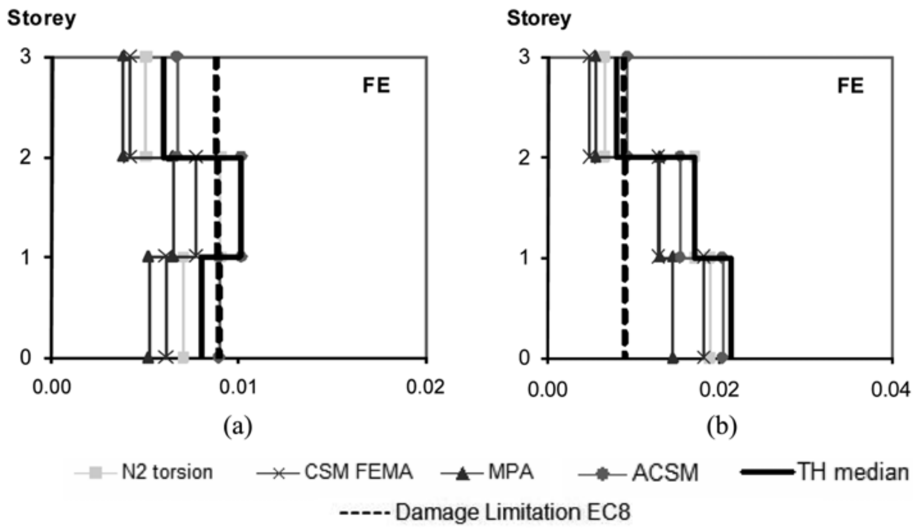


Fig. 19 Chord rotations: (a) 0.1g, X, FE, (b) 0.2g, X, FE

5.4 An insight view of ACSM

A further probe into the performance of ACSM in emulating dynamic responses reveals that results of ACSM appeared satisfactorily close to the median values and fitted well within the appreciable dispersion range defined by median and plus-minus one standard deviation of dynamic results. Figs. 20, 21 and 22 exemplify such observation through the comparative plots of interstorey drift, normalized roof displacements and storey displacement profiles.

6. Conclusions

This study started with the evaluation of a simplified fiber element based modelling approach for a three-storey plan-irregular reinforced concrete frame building representing typical old constructions in the Mediterranean area. A model of the same structure was tested in full-scale under pseudo-dynamic conditions at JRC, Ispra. Comparing with the experimental results, the analytical model proved to be an optimum trade-off between accuracy and efficiency, which is an absolutely necessary condition to perform large parametric studies, as those required for this work.

Subsequently, a preliminary evaluation was performed to identify the right variations of two

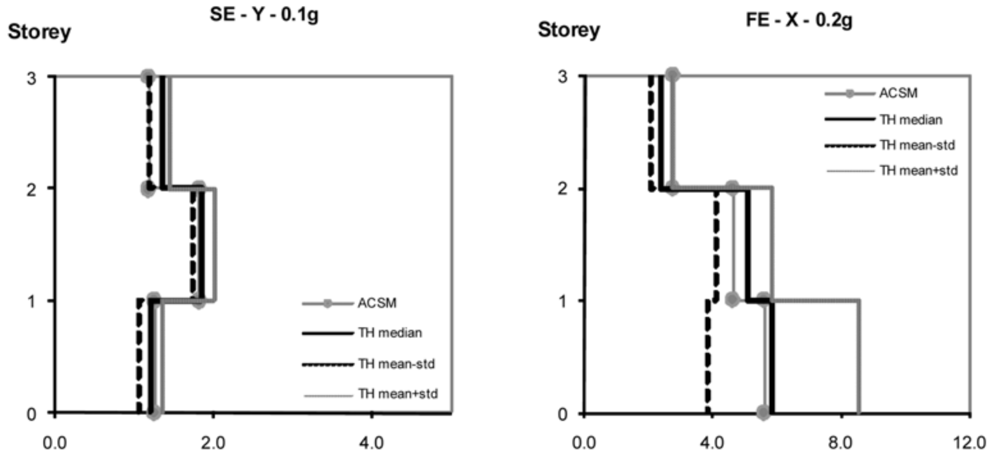


Fig. 20 Storey drifts (cm)

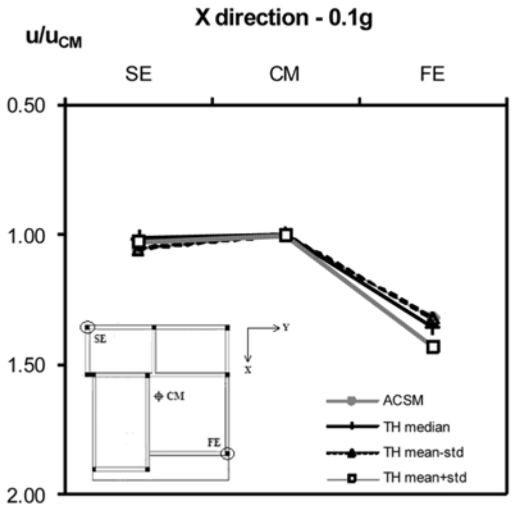


Fig. 21 Normalized roof displacements

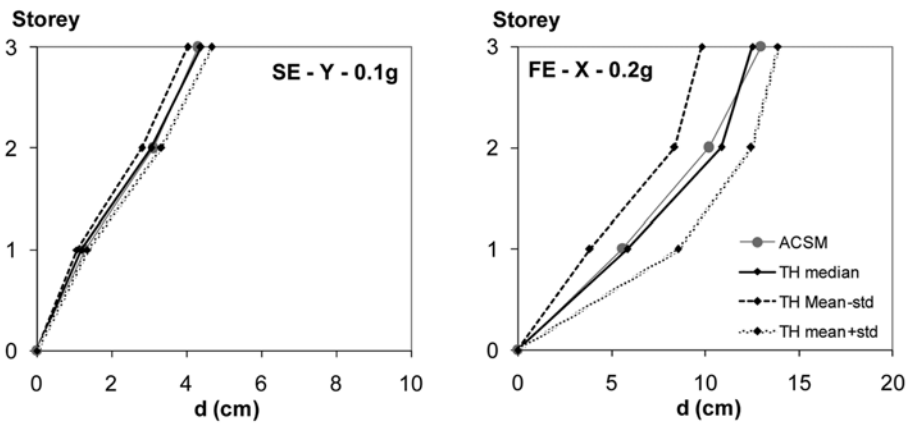


Fig. 22 Storey displacement patterns

codified NSPs, namely, the CSM and the N2. Results have shown that recent improvements of these two methods – namely, the FEMA440 variant of CSM and the torsional variant of N2 –, have substantially enhanced its accuracy and, hence, used for the further probing.

Finally, four aforementioned NSPs – FEMA440 variant of CSM, torsional variant of N2, MPA and ACSM – were appraised based on their ability to predict both global and local responses by comparing with the 'true' responses. The latter, were determined from the median values of dynamic analyses using a suit of seven ground motions applied in four different orientations. Although all the NSPs predicted reasonably well, it is the ACSM that seems to be better equipped to cope with the peculiarities resulting from plan-irregularities, hence closely following the median dynamic responses. Further probing on the ACSM has revealed that the dispersion of its results from the median values were also within the acceptable range; i.e., the median of the dynamic responses, plus-minus one standard deviation of the dynamic responses.

In terms of computational demand the ACSM procedure applied to this building takes about 25% more time to run than the conventional non-adaptive NSPs. The authors tend to feel that the accuracy gain provided by this adaptive method somewhat justifies the modest increase of computing time.

The better performance of the ACSM procedure, results from the use of a displacement based adaptive pushover analysis; the latter defines an equally adaptive capacity curve that not only encompasses the effects of higher vibration modes, but also those of a progressive damage, all without any prior requirement of reference node. Similar accuracy was observed in previous verification studies of ACSM using plane frames (Pinho *et al.* 2008) and bridges (Pinho *et al.* 2008). This study, therefore, encourages further parametric studies considering different plan-irregularities and torsional properties to develop definitive recommendations.

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