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Nonlinear Static Procedures on the Seismic Assessment of Existing Plan Asymmetric Buildings

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ABSTRACT

The use of Nonlinear Static Procedures (NSPs) for the seismic assessment of plan regular buildings and bridges is nowadays widespread. Their good performance in such cases is widely supported by an extensive number of papers. However, the applicability of NSPs on plan-irregular 3D buildings has so far been the object of a limited number of scientific studies. This limitation leads to a minor use of these methods to assess actual existing structures, the majority of which do tend to be irregular in plan. In this paper, the Capacity Spectrum Method (CSM) with the features presented in the FEMA440 guideline, the Extended N2 method for plan irregular buildings, the Modal Pushover Analysis (MPA) and the Adaptive Capacity Spectrum Method (ACSM) are applied and compared on real existing plan asymmetric buildings. The NSPs are tested in a wide range of intensities in order to understand their performance in different stages of structural inelasticity being the results compared with the most exact incremental nonlinear dynamic analyses. The structures analysed in this paper are two existing RC buildings in Turkey with five and eight storeys. They are both irregular in plan, presenting important torsional features in the structural response.

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1. INTRODUCTION

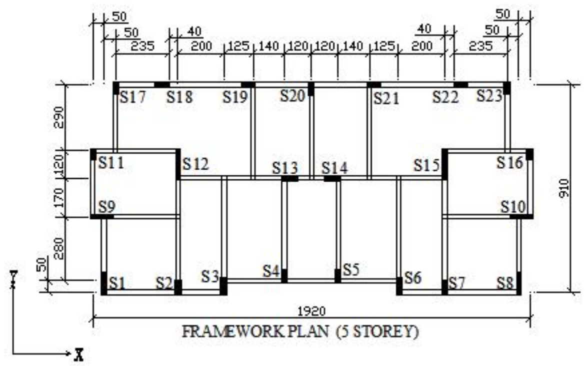
The extension of the use of Nonlinear Static Procedures (NSP) to the case of plan-irregular structures has so far been the object of a limited number of studies (Chopra and Goel 2004; Fajfar et al. 2005), which effectively ends up by limiting significantly the employment of NSPs to assess actual existing structures, the majority of which tend to be irregular in plan. In addition, such few studies have typically concentrated on the application and verification of a single NSP approach, thus not providing useful elements of comparison between the different methodologies available.

Therefore, in this work four commonly employed nonlinear static procedures (CSM, N2, MPA and ACSM) are applied on the assessment of two real Turkish RC five and eight storey buildings. The results are compared with nonlinear dynamic analyses in order to evaluate the accuracy of the different NSPs.

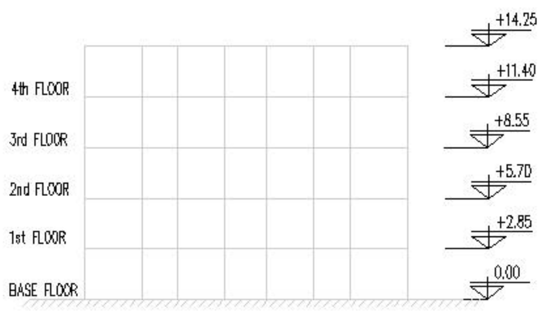
2. CASE STUDIES

The first 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 asymmetric along the X axis, Fig. 1a), and all the floors have the same height, Fig. 1b). There are beams framing into beams leading to possible weak connections in the structure. There are also walls and elongated columns, as presented in Fig. 1a).



(a)



(b)

Fig. 1. (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).

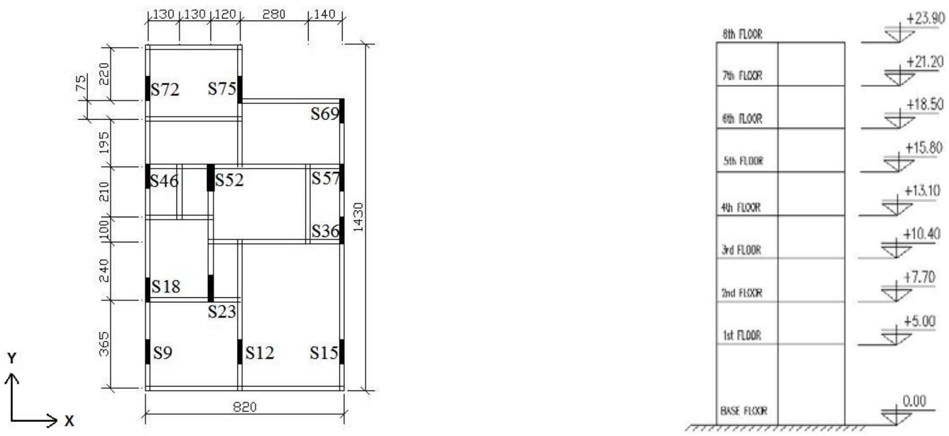
The mass of each story is considered to be 263ton, except in the last storey where the mass is 150ton.

The second case study is a real Turkish reinforced concrete eight storey building. It is a plan irregular structure – it is asymmetric along the X and Y axis, Fig. 2a). The first storey height amounts to 5.00m and the other floors have the same 2.70m height, Fig. 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 Fig. 2a), 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.

The mass in the first storey of the eight storey building is 73ton, in the upper storeys is 65ton and in the last storey the mass is 56ton.



(a) (b)

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

Both 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.

3. MODELLING ISSUES

The structural analysis software used in this study was SeismoStruct (SeismoSoft 2006), a fibre-based structural analysis program. It is 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 program is able to run eigenvalue analysis, nonlinear static (conventional and adaptive) analysis and nonlinear dynamic analysis.

The 3D buildings were represented with space frame models 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.

Hysteretic damping was already implicitly included in the nonlinear fibre model formulation of the inelastic frame elements. It was used a 5% tangent stiffness-proportional damping in order to take into account for possible non-hysteretic sources of damping.

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 incorporated 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 16.7 MPa was considered.

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). Average yield strength of 371 MPa was assumed.

The rigid diaphragm effect was modelled using the Nodal Constraints Rigid Diaphragm with Penalty Functions option. The penalty function exponent used was 10^7.

4. SEISMIC ASSESSMENT

4.1 Seismic Action

In this study, three bi-directional semi-artificial ground motion records were considered. The three real records taken from the PEER's database website (PEER 2009) are presented in Table 1.

Table 1. Records used in this study.

Earthquake Name	YEAR	ClstD (km)	Earthquake Magnitude	Site Classification Campbell's geocode	Mechanism Based on Rake Angle
Tabas, Iran	1978	13.94	7.35	Firm Rock	Reverse
Whittier Narrows-01	1987	40.61	5.99	Very Firm Soil	Reverse - Oblique
Northridge-01	1994	37.19	6.69	Firm Rock	Reverse

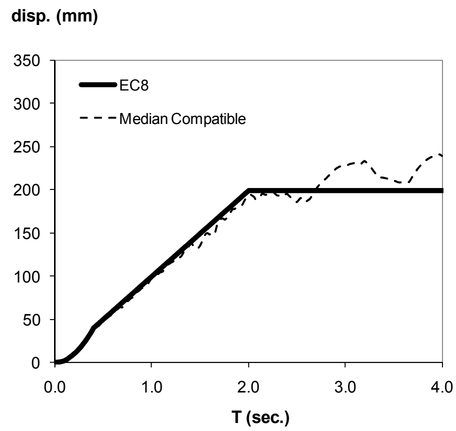


Fig. 3. Displacement response spectra, 0.4g.

The records were fitted to the Eurocode 8 (CEN 2004) 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 for several intensity levels of peak ground accelerations.

For the NSPs the response spectra used are the median of the response spectra defined, compatible with the considered accelerograms (Fig. 3).

4.2 Considered Nonlinear Static Procedures

The NSPs herein scrutinised may be split into two main groups.

The first set of NSPs comprises the pioneering Capacity Spectrum Method (CSM), introduced by Freeman and collaborators (Freeman et al. 1975; Freeman 1998) and implemented in ATC-40 guidelines (ATC 1996), and the equally innovative N2 method suggested by Fajfar and co-workers (Fajfar and Fishinger 1988, Fajfar 2000) and later included in Eurocode 8 (CEN 2004). These first proposals are characterised by their simplicity and usually consider a first mode and/or uniform load distributions in the computation of the pushover/capacity curve. Each one of these two approaches was considered in two modalities: N2/Extended N2 and CSM-ATC40/CSM-FEMA440. The Extended N2 method (Fajfar et al. 2005) consists of an extension to the 3D space of the original N2 method, whilst the CSM-FEMA440 variant features the improved target displacement calculation rules given in the FEMA-440 report (ATC 2005).

The second group features the more recent proposals of Chopra and Goel (Chopra and Goel 2002 and 2004) on a Modal Pushover Analysis (MPA) and of Casarotti and Pinho (2007) by means of Adaptive Capacity Spectrum Method (ACSM). All of them present improvements with respect to their predecessors, such as the inclusion of higher modes contribution, the consideration of progressive damage, and alternative definitions of reference node; the latter can result very opportune in 3D analysis.

4.3 Structural Analyses Carried Out

Two types of pushover analyses were carried out: the so-called conventional force pushover and the Displacement-based Adaptive (DAP) pushover algorithm (Antoniou and Pinho 2004). For the former, two load patterns - mass-proportional and modal - were applied in the structure. In DAP, the displacements were applied on all mass nodes of the structure and spectral scaling was considered to weigh the contribution of the different modes. In both cases, the force/displacement loads were applied independently in the two horizontal positive/negative directions. For each of the resulting eight loading cases, the target displacement was evaluated with the larger value in each direction being chosen.

For the nonlinear dynamic analysis, the aforementioned three bidirectional semi-artificial ground motion records were employed. Each record was applied twice in the structure changing the direction of the components, resulting in 6 runs of incremental dynamic analyses (0.2, 0.4, 0.6 and 0.8g for the five storey building, and 0.1, 0.2 and 0.4g for the eight storey building).

The results in terms of top displacements, displacement patterns, interstorey drifts, chord rotations and top rotations in the two directions were calculated and compared for all seismic intensity levels, and for all nonlinear static and dynamic analysis.

4.4 Preliminary Optimization

A comparison between the extended N2 method, proposed by Fajfar (Fajfar et al. 2005) to overcome the specificities of the plan-irregular buildings, and its former version was made in terms of storey drifts, normalized top displacements, lateral displacements pattern and chord rotations (Bhatt and Bento 2010b)). The same comparison was made between the CSM with the features proposed in the ATC40 and in FEMA440 (Bhatt and Bento 2010a)).

The preliminary comparison between the N2 and the Extended N2 methods showed that for the buildings herein evaluated the Extended N2 procedure leads to better results than its original version. In the other hand, the CSM-FEMA440 proved to be a much improved version with respect to its CSM-ATC40 predecessor.

The Extended N2 method and the CSM-FEMA440 were chosen to be used in the subsequent plots.

5. PARAMETRIC STUDY RESULTS

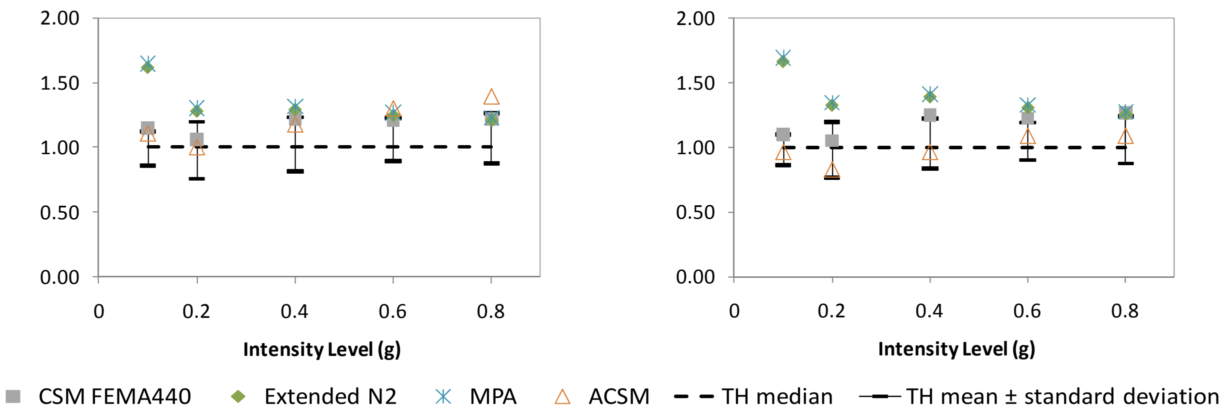
In this section, the five and eight storey buildings are assessed comparing the NSPs under evaluation with the *timehistory* results.

5.1 Five Storey Building

The roof displacements determined from different NSPs were normalized by the corresponding median responses of nonlinear dynamic analysis, 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. As it was mentioned before, the NSPs must never lead to underestimated results, therefore these ratios should always be higher than one. Ideally one would desire such ratios to tend to unity, which means the NSPs would perfectly match the *timehistory* median results.

$$Top\ Displacement\ ratio = \frac{NSP's\ top\ displacement}{Time\ history\ median\ top\ displacement} \qquad (1)$$

The top displacement ratios in the five storey building's centre of mass in both directions are presented in Fig. 4. Note that the top displacements in the centre of mass correspond to the target displacements. In each figure and for each level of intensity, a line representing the dispersion of the *timehistory* results ([mean – standard deviation ; mean + standard deviation]) is also plotted.



(a) (b)

Fig. 4. Top displacement ratios, centre of mass (a) X direction; (b) Y direction.

In the elastic or almost elastic range (0.1g and 0.2g) the CSM-FEMA440 leads to estimations very close to the *timehistory* in both directions. For these levels of intensity the ACSM leads to very good results in the X direction and it under predict the estimations in the Y direction. The extended N2 and the MPA overestimate the top displacements in this elastic stage.

In the inelastic regime (0.4g to 0.8g) in the X direction all the methods lead to approximately same predictions and always conservative. The exception occurs for 0.8g where the ACSM leads to higher results than the other methods.

In the Y direction for 0.4g, the ACSM leads to slightly underestimated results, while the other methods overestimate the response. For 0.6g and 0.8g the ACSM perfectly matches the *timehistory* while the other methods lead to approximately same results and always conservative.

From the plots, one can confirm that the dispersion of the *timehistory* results is not very high, leading to the conclusion that the number and type of records chosen for this study proved to be enough to get reliable results in the five storey building. One can also observe that, to what top displacement ratios is concerned, the ACSM and the CSM-FEMA440 lead in general to results close to the *timehistory* median and always within the range [mean – standard deviation ; mean + standard deviation]. This fact proves their good performance on estimating such measure. The extended N2 and the MPA are generally close to the upper bound of this range, mean + standard deviation.

The comparison of the different NSPs and the nonlinear dynamic results in terms of lateral displacement profiles are plotted in Fig. 5.

In the inelastic regime, all NSPs tend to reproduce conservatively the response of the building in the X direction.

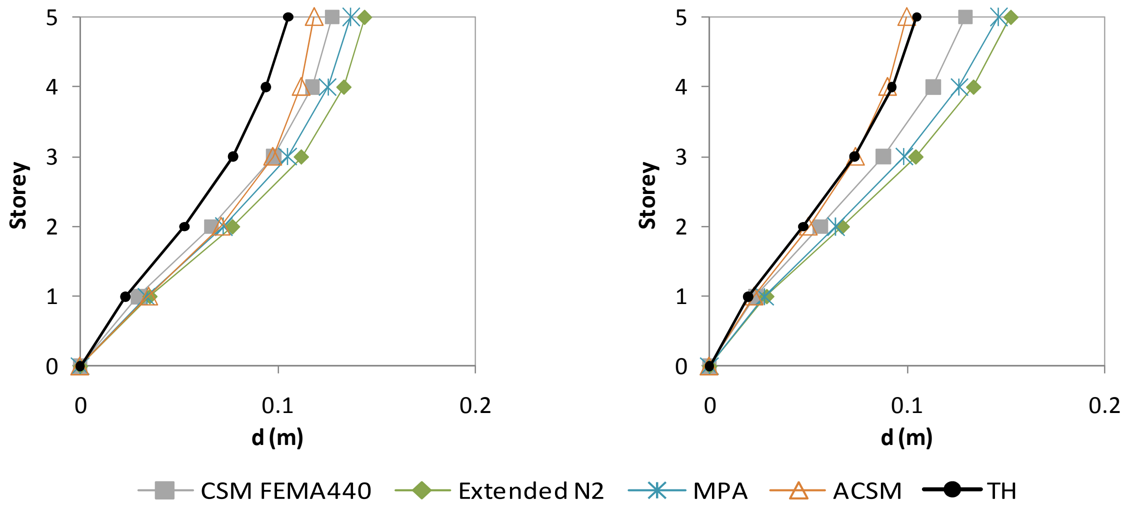
For medium levels of inelasticity (0.4g) in the Y direction one can observe that:

- The ACSM matches the *timehistory* analysis. However this method sometimes leads to non conservative results in the upper floors;
- The other three methods lead to conservative results, but the CSM-FEMA440 is the one closer to the nonlinear dynamic results.

For higher levels of inelasticity (0.6g and 0.8g) in the Y direction:

- The results computed with the ACSM get close to the *timehistory* response;

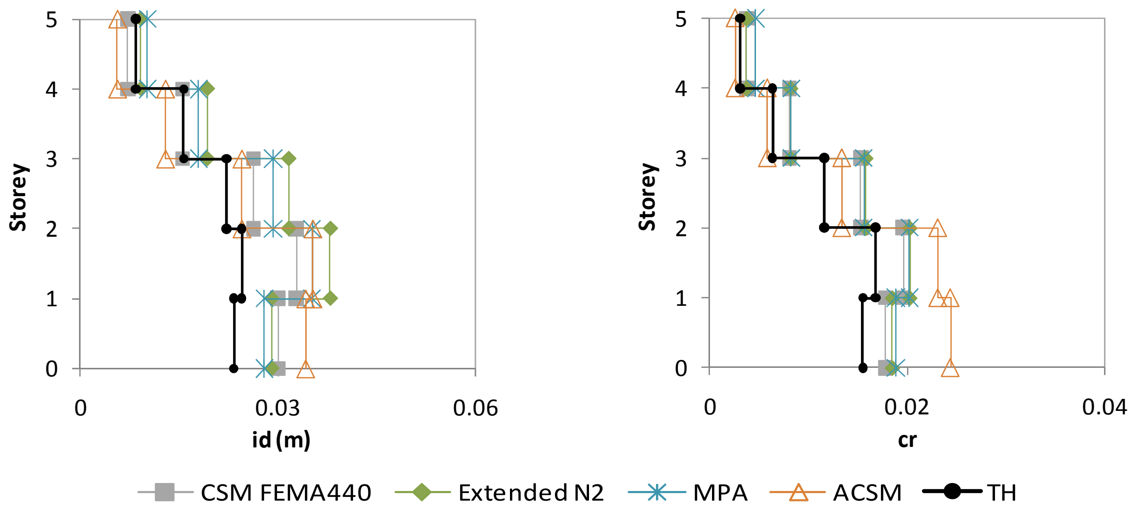
- The CSM-FEMA440, the extended N2 and the MPA lead to similar and conservative results.



(a) (b)

Fig. 5. Lateral displacement profile (m), column S1, 0.4g (a) X direction; (b) Y direction.

The interstorey drifts and chord rotations profiles are presented in Fig. 6.



(a) (b)

Fig. 6. X direction (a) Interstorey drifts profile (m), column S23, 0.4g; (b) Chord rotations profile, column S13, 0.6g.

In the X direction in the inelastic range:

- The extended N2, the MPA and the CSM-FEMA440 lead to slightly conservative results but generally close to the *timehistory*;

- The ACSM leads to conservative results in the down floors and to slightly non-conservative estimations in the upper floors;
- For high levels of inelasticity (0.6g and 0.8g) the ACSM cannot reproduce the interstorey drift nor the chord rotation patterns, mainly in the first two floors;
- The other three methods reproduce in a very good fashion the response patterns through all inelastic stages in all the floors.

In the Y direction for medium levels of inelasticity:

- The CSM-FEMA440 leads to slightly conservative results;
- The ACSM leads to slightly underestimated responses;
- The other two methods lead to conservative results;
- All methods are able to correctly reproduce the response patterns in all the floors.

In the Y direction for high levels of inelasticity:

- The CSM-FEMA440, the extended N2 and the MPA lead to slightly conservative results;
- The ACSM lead to slightly non-conservative estimations on the upper floors and to conservative results on the down floors;
- All methods are able to correctly reproduce the response patterns in all the floors, except the ACSM that is not able to reproduce the pattern on the first storey.

The pattern of roof displacements in plan normalized by the same at centre of mass as shown in Fig. 7, gives an idea how torsional rotation changes the displacement demands at the edges.

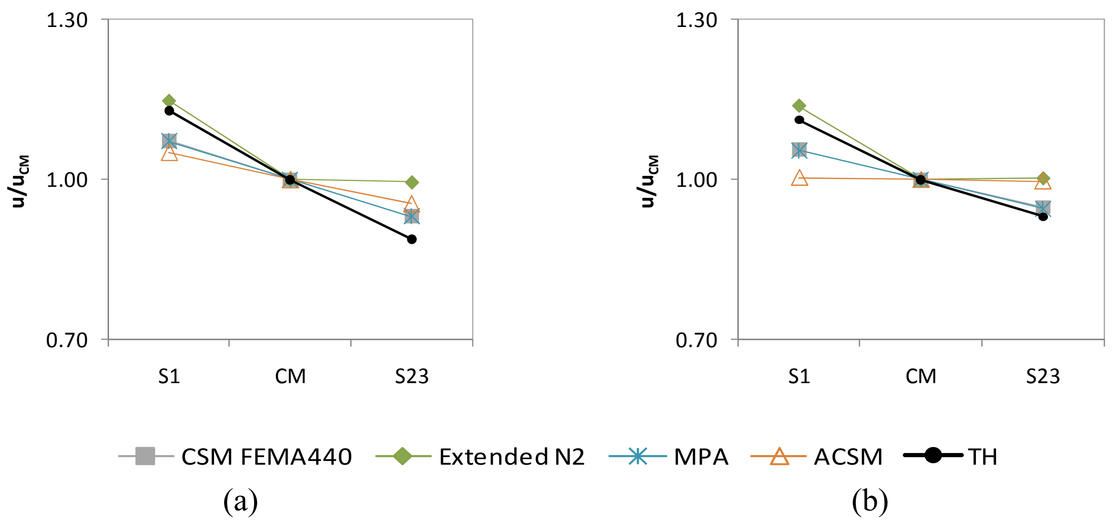


Fig. 7. Normalized top displacements X direction (a) 0.2g; (b) 0.6g.

In terms of normalized top displacements, the extended N2 was the method that better reproduced the torsional motion of the building in the X direction (the asymmetric direction of the structure). In fact, the method perfectly captures the torsional amplification on the flexible edge of the building, column S1, for 0.2g and 0.6g. For 0.4g and 0.8g it slightly overestimates the response. The extended N2 method led to conservative results on the stiff side of the building, column S23, through all the seismic intensities, because it does not consider any de-amplification effect due to torsion.

The CSM-FEMA440, the MPA and the ACSM always predicted the torsional motion of the building in a linear way from one side of the building to the other, usually underestimating the torsional amplification on the flexible side and overestimating the response on the stiff edge.

5.2 Eight Storey Building

The results obtained for the eight storey building are herein presented.

The ratios of top displacements and the *timehistory* dispersion for all the seismic intensities studied are plotted in Fig. 8.

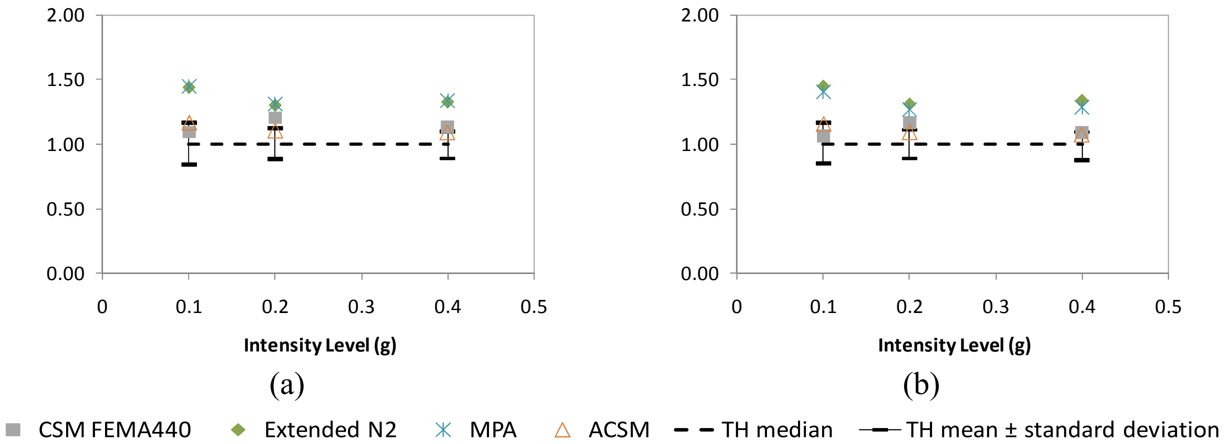


Fig. 8. Top displacements ratios, X direction a) centre of mass; b) column S9.

In terms of top displacements, one can observe that the ACSM practically matches the *timehistory* results through all the seismic intensities. The CSM-FEMA440 almost matches the *timehistory* for 0.1g and 0.4g, but it slightly overestimates the results for a seismic intensity of 0.2g. The extended N2 and the MPA generally lead to conservative results.

From the plots it is evident once again that the distribution of the *timehistory* results has a relatively small dispersion. This corroborates the idea that the number and type of records used in this study seem to be enough to get reliable results in the eight storey building. One can also observe that the ACSM and the CSM-FEMA440 lead to top displacement ratios very close to the upper bound of the *timehistory* distribution range – mean + standard deviation – while the other two methods lead to values slightly above this range.

The results in terms of lateral displacement and chord rotations profiles are plotted in Fig. 9 and in terms of interstorey drifts in Fig. 10.

From the plots it is clear that the building presents a soft storey mechanism on the first floor along the X direction, collapsing due to this local mechanism for 0.4g. In fact, the interstorey drifts and chord rotations are much higher on the first storey than in the upper floors. This trend is observed through all the seismic intensities tested. The ACSM and the CSM-FEMA440 are the methods that better reproduce this phenomenon. The extended N2 and the MPA slightly overestimate this mechanism. In fact, the ACSM is able to predict correctly the soft storey mechanism because it uses the DAP where the properties of the damaged structures are updated and fed into the model in each analysis step. The soft storey mechanism on the first floor can be explained by the considerable difference between the heights of the first and the second floors, inducing a considerable difference on the stiffness between these two storeys. In fact, the first storey height amounts to 5m and the upper floors to 2.70m, therefore the first floor is more flexible than the upper ones, leading to a local mechanism. These characteristics lead the building to behave inelastically only in the X direction, keeping the response elastic in the Y direction.

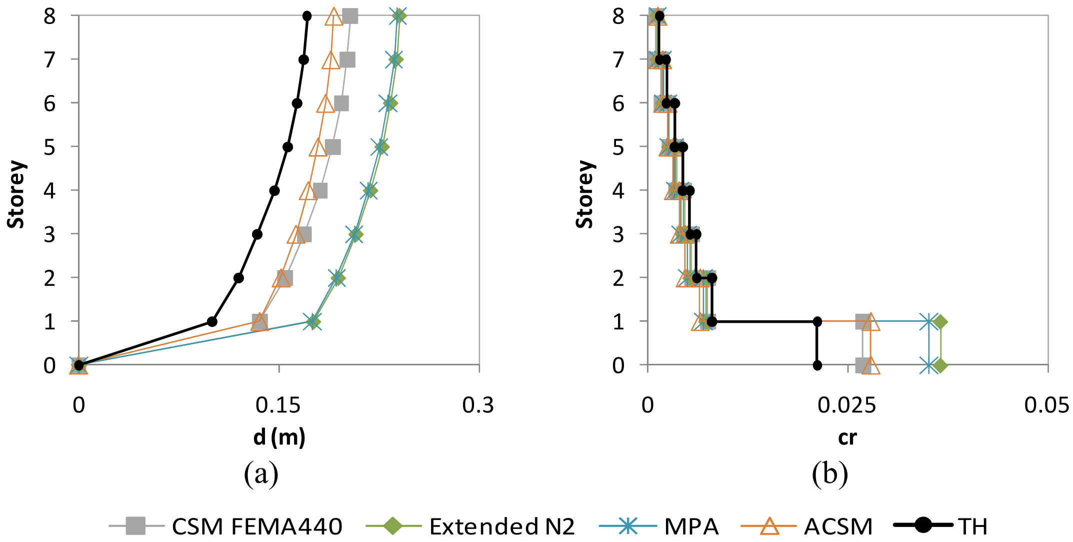


Fig. 9. X direction a) Lateral displacement profile (m), column S69, 0.4g; b) Chord rotations profile, column S9, 0.4g.

In the inelastic range, all the methods seem to overestimate the lateral displacement profiles. Although, the ACSM and the CSM-FEMA440 are the methods that lead to results closer to the nonlinear dynamic analysis.

In terms of interstorey drifts and chord rotations, one can confirm that the ACSM and the CSM-FEMA440 are the methods that better reproduce the soft storey mechanism on the first floor. The other two procedures lead to conservative estimations. For the upper storeys, all the NSPs lead to the same results, very close to the *timehistory*.

It is interesting to note that all pushover methods could reproduce in a very good way the specific characteristics of the building's structural response through all the seismic intensities tested, namely:

- The unbalanced stiffness distribution between the two directions;
- The Y direction is much more stiff than the X direction;
- The response of the building remains elastic in the Y direction through all the intensity levels analysed;
- The collapse of the building due to a soft storey mechanism in the first floor along the X direction.

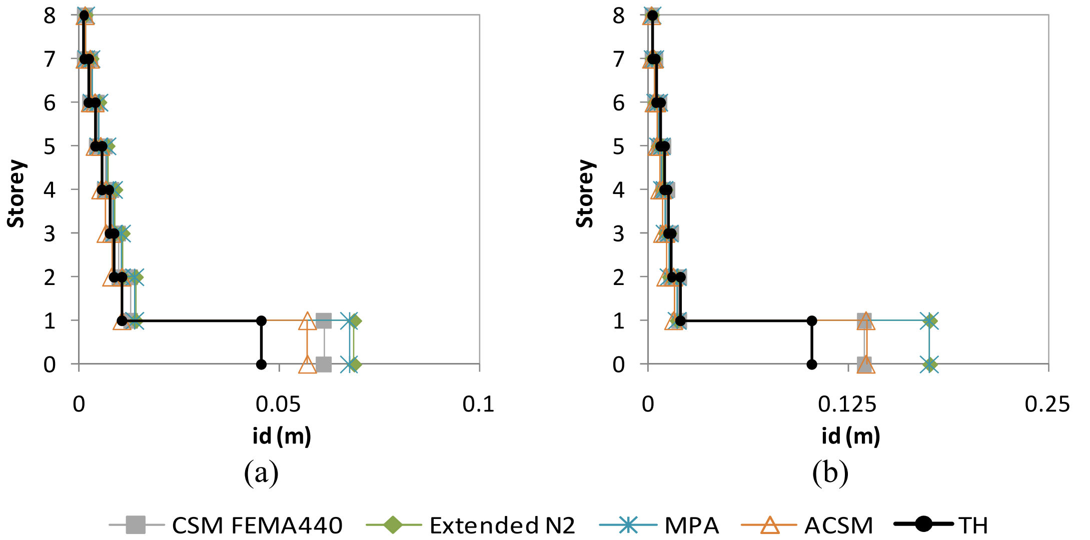


Fig. 10. Interstorey drifts profiles X direction a) Column S72, 0.2g; b) Column S52, 0.4g.

The normalized top displacements in both X and Y directions are plotted in Fig. 11.

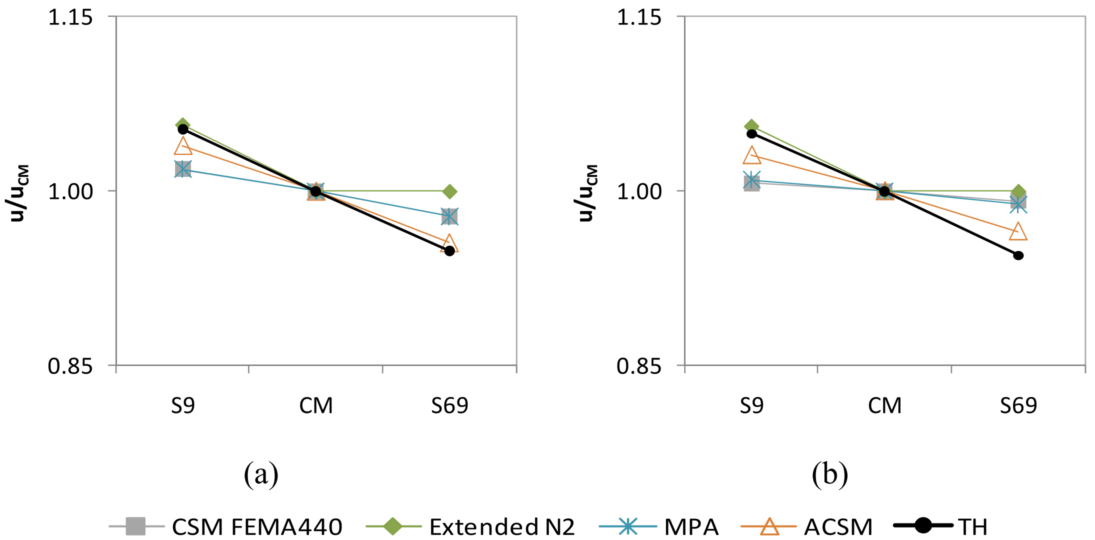


Fig. 11. Normalized top displacements, X direction a) 0.2g; b) 0.4g.

From the plots it is clear that the extended N2 method could perfectly capture the torsional amplification on column S9 in the X direction. On the opposite edge, the method overestimates the seismic response because it does not consider any de-amplification due to torsion.

The CSM-FEMA440, the ACSM and the MPA lead to similar results in the X direction. They estimate linearly the response from one side of the building to the other, underestimating the torsional amplification on column S9 and overestimating the results on column S69.

CONCLUSION

In the current endeavour, the effectiveness with which four commonly employed Nonlinear Static Procedures (CSM, N2, MPA, ACSM) are able to reproduce the actual dynamic response of two real Turkish RC five and eight storey buildings was assessed.

The results obtained in this study, in terms of top displacements, lateral displacement profiles, interstorey drifts and chord rotations, showed that the CSM-FEMA440 and the ACSM were the methods that better reproduced the nonlinear dynamic median response profiles, although the ACSM was usually closer from the *timehistory*.

The good performance of the CSM can be explained due to its accurate procedure to calculate the target displacement proposed in FEMA440, which includes: a new and efficient algorithm to compute the effective period and the effective damping; an accurate demand spectrum reduction factor coupled with the new concept of modified acceleration-displacement response spectrum (MADRS).

The ACSM apparently managed to follow slightly better the change of response characteristics with the increase of seismic intensity, most likely because of the fact that such method uses an adaptive displacement pushover (DAP) and an equivalent SDOF structural displacement built on the current deformed pattern (which can turn very useful when dealing with 3D plan asymmetric buildings).

The extended N2 method and the MPA usually led to conservative results in terms of lateral displacement profiles, top displacements, interstorey drifts and chord rotations. All the methods seemed to lead to approximate and conservative estimations for high levels of inelasticity.

In terms of normalized top displacements, the extended N2 method was the only method capable of reproducing the torsional motion of the buildings through increasing seismic intensities. The reason for this trend lies on the fact that such method uses correction factors based on an elastic response spectrum analysis, without considering any de-amplification of displacements due to torsion. Therefore the method is able to capture the torsional amplification on the flexible edge of the buildings, and it generally leads to conservative results on the stiff side. The other NSPs generally reproduced in a linear way the torsional motion from one side of the building to the other. For each intensity level in each direction, these methods were only able to capture the torsional behavior of just one side of the building, under predicting the other.

The eight storey building herein studied has a bad stiffness distribution between the two orthogonal directions, collapsing due to a soft storey mechanism on the first floor along the X direction. All NSPs were able to reproduce this local mechanism and the specific features on the seismic response of the building through all the intensity levels tested.

Additional work considering other real buildings must be carried out in order to reach definitive conclusions.

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REFERENCES

Antoniou, S., Pinho, R. (2004). "Development and verification of a displacement-based adaptive pushover procedure", *Journal of Earthquake Engineering* Vol. **8**(5), 643-661.

Applied Technology Council (ATC) (1996). "Seismic Evaluation and Retrofit of Concrete Buildings", Vol. 1 and 2, Report No. ATC-40, Redwood City, CA.

Applied Technology Council (ATC) (2005). "Improvement of Nonlinear Static Seismic Analysis Procedures", FEMA 440 Report, Redwood City, CA.

Bento, R., Bhatt, C., Pinho, R. (2010). "Using Nonlinear Static Procedures for Seismic Assessment of the 3D Irregular SPEAR Building", *Earthquakes and Structures*, Vol. **1**(2), 177-195.

Bhatt, C., Bento R. (2010a)). "Extension of the CSM-FEMA440 to plan-asymmetric real building structures", *Earthquake Engineering and Structural Dynamics*, Published online in Wiley Online Library (wileyonlinelibrary.com) doi: 10.1002/eqe.1087.

Bhatt, C., Bento, R. (2010b)). "Assessing the Seismic Response of Existing RC Buildings Using the Extended N2 Method", *Bulletin of Earthquake Engineering*, DOI: 10.1007/s10518-011-9252-8.

Casarotti C., Pinho R. (2007). "An Adaptive Capacity Spectrum Method for assessment of bridges subjected to earthquake action", *Bulletin of Earthquake Engineering*, Vol. **5**(3), 377-390.

Chopra A.K., Goel R.K., (2002). "A modal pushover analysis procedure for estimating seismic demands for buildings", *Earthquake Engineering and Structural Dynamics*, Vol. **31**, 561-582.

Chopra, A.K., Goel, R.K. (2004). "A modal pushover analysis procedure to estimate seismic demands for unsymmetric-plan buildings", *Earthquake Engineering and Structural Dynamics*, Vol. **33**, 903-927.

Comité Européen de Normalisation (CEN) (2004). "Eurocode 8: Design of structures for earthquake resistance. Part 1: general rules, seismic actions and rules for buildings", EN 1998-1:2004, Brussels, Belgium.

Fajfar, P. (2000). "A nonlinear analysis method for performance-based seismic design", *Earthquake Spectra*, Vol. **16**(3), 573-592.

Fajfar, P., Fischinger, M. (1988). "N2 – A method for non-linear seismic analysis of regular buildings", *Proceedings of the Ninth World Conference in Earthquake Engineering*, Tokyo-Kyoto, Japan, Vol. **5**, 111-116.

Fajfar, P., Marusic, D., Perus I. (2005). "Torsional effects in the pushover-based seismic analysis of buildings", *Journal of Earthquake Engineering*, Vol. **9**(6), 831-854.

Filippou, F.C., Popov, E.P., Bertero, V.V. (1983). "Modelling of R/C joints under cyclic excitations", *Journal of Structural Engineering*, Vol. **109**(11), 2666-2684.

Freeman, S.A. (1998). "Development and use of capacity spectrum method", *Proceedings of the Sixth U.S. National Conf. Earthquake Engineering*, Seattle, Oakland, USA.

Freeman, S.A., Nicoletti, J.P., Tyrell, J.V. (1975). "Evaluation of existing buildings for seismic risk – A case study of Puget Sound Naval Shipyard, Bremerton, Washington", *Proceedings of U.S. National Conference on Earthquake Engineering*, Berkley, USA., pp. 113-122.

Hancock, J., Watson-Lamprey, J., Abrahamson, N.A., Bommer, J.J., Markatis, A., McCoy, E., Mendis, R. (2006). "An improved method of matching response spectra of recorded

earthquake ground motion using wavelets", *Journal of Earthquake Engineering*, Vol. **10**(S1), 67–89.
Mander, J.B., Priestley, M.J.N., Park, R. (1988). "Theoretical stress-strain model for confined concrete", *ASCE Journal of Structural Engineering*, Vol. **114**(8), 1804-1826.
Martinez-Rueda, J.E.; Elnashai, A. S. (1997). "Confined concrete model under cyclic load", *Materials and Structures*, Vol. **30**(197), 139-147.
Menegotto, M., Pinto, P.E. (1973). "Method of analysis for cyclically loaded RC plane frames including changes in geometry and non-elastic behaviour of elements under combined normal force and bending", *Symposium on the Resistance and Ultimate Deformability of Structures anted on by well defined loads*, International Association for Bridge and Structural Engineering, Zurich, Switzerland; 15-22.
PEER (2009). Strong Ground Motion Database, http://peer.berkeley.edu/nga/
SeismoSoft (2006). SeismoStruct - A computer program for static and dynamic nonlinear analysis of framed structures, available online from http://www.seismosoft.com.
Vuran, E., Bal, Ý. E., Crowley, H. and Pinho, R. (2008). "Determination of equivalent SDOF characteristics of 3D dual structures", *Proceedings of the 14th World Conference on Earthquake Engineering*, Beijing, China, paper no: S15-031.