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Assessing the seismic response of existing RC buildings using the extended N2 method

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Abstract Extensive studies have confirmed the good performance of the N2 method, recommended by Eurocode8, when performing pushover analyses in regular structures. However, this procedure shows lack of accuracy in predicting the torsional motion of plan-asymmetric buildings. In order to overcome this problem, Peter Fajfar and his team have proposed an extension of the method based on a combination of a pushover analysis and of an elastic response spectrum analysis. Since definitive answers about this topic have not yet been reached, this paper intends to proceed the study applying the extended N2 method to real existing RC buildings. Three real plan-asymmetric buildings with three, five and eight storeys were assessed. The results obtained with the extended N2 method were compared with the ones evaluated by means of the original N2 and with the nonlinear dynamic analysis through the use of semi-artificial ground motions. The analyses were performed for different seismic intensities in order to evaluate the torsional response of the building through different stages of structural inelasticity. The results obtained show that the extended N2 method generally reproduces in a very good fashion the real torsional behavior of the analyzed buildings. The conclusions herein outlined, added to the ones already published by the aforementioned authors, seem to confirm that the extended N2 method can be introduced in the next version of Eurocode8 as a nonlinear static procedure capable of accurately predicting the torsional response of plan-asymmetric buildings.

Keywords Seismic assessment · 3D pushover analysis · Extended N2 method · Real plan-asymmetric buildings · Torsion

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1 Introduction

Nowadays the nonlinear static procedures (NSPs) constitute a very practical tool to perform nonlinear structural analyses since they require less computational effort in terms of time-consuming than the nonlinear dynamic analyses. This feature is crucial mainly in design offices where the time constraints are restrictive. These procedures have been subjected to extensive studies, showing their good performance in regular buildings, planar frames and bridges (Fajfar and Fischinger 1988; Freeman 1998; Freeman et al. 1975).

Eurocode8 (CEN 2004) introduced the N2 method developed by Fajfar and his team (Fajfar and Fischinger 1988) as the recommended nonlinear static procedure to be used when performing pushover analyses. The problem of the commonly used NSPs, including the N2 method, is their inability to deal with plan-asymmetric buildings. Generally they cannot capture the torsional effects distorting the real structural response. This kind of buildings constitutes the common case in real life so it is urgent to implement updating formulations in these procedures turning the structural response as realistic as possible. Some efforts have been made to take this generation of NSPs to a higher performance level (Chopra and Goel 2004; Fajfar et al. 2005; Bento et al. 2010; Stefano and Pintucchi 2010; Erduran and Ryan 2010). The Eurocode8-1 (*4.3.3.4.2.7 Procedure for the estimation of the torsional effects*) includes some guidelines for the determination of torsional effects based on elastic analysis (e.g. an early version of the extended N2 method), but they are only restricted to torsionally flexible buildings.

Fajfar and his team have developed the Extended N2 method (Fajfar et al. 2005) which is able to capture the torsional behavior of plan-asymmetric buildings. This procedure is based on the application of correction factors to the pushover results obtained with the N2 method. The correction factors depend on a dynamic elastic analysis and on a pushover analysis.

In this paper the seismic behavior of three real existing plan-asymmetric buildings is assessed. The original N2 method and its extension are applied to these structures within a wide range of seismic intensities and compared with nonlinear dynamic analysis through the use of semi-artificial ground motions. The results were split in two main groups: the ones corresponding to elements located in the centre of the buildings and the ones located on the edges. The first group allows the evaluation of the original N2 in estimating the target displacements. Since the torsional effect is not very pronounced in this part of the buildings, both original and extended N2 lead to the same results. The second group shows the performance of both methods in reproducing the torsional motion induced by the earthquake in the structures.

The results presented herein corroborate the idea that this extended procedure exhibits potential to be included in the next version of the Eurocode8 as a nonlinear static method able to correctly deal with the torsional problems in plan-asymmetric buildings. Although, additional studies should be developed in order to reach definitive conclusions. Buildings with different typologies should be tested.

2 The extended N2 method

The existing NSPs usually lead to good estimations of the seismic response of regular structures. There are several studies proving their good performance in planar frames and bridges. When the building is plan-asymmetric, the torsional motion turns the structural response more complex. In such cases, the building has to be analysed using a 3D model instead of a planar frame model. As it was mentioned before, the extension of the NSPs to the 3D case has so far been object of a small number of studies. One of the most important is the one developed by Fajfar and his team (Fajfar et al. 2005), creating the so-called Extended N2 method. This procedure intends to handle the torsional problem in plan-asymmetric buildings by adjusting the pushover results, computed with the original N2 method proposed in the Eurocode8, by means of correction factors. These torsional correction factors are computed calculating the ratio between the normalized roof displacements obtained by an 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. 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 reduction of displacements due to torsion, given by the elastic analysis. In fact, the studies developed by these authors proved 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. In this extended procedure the reduction of demand due to torsion is neglected.

3 Case studies

Three real plan-asymmetric RC buildings were analyzed in this endeavour. The first case study is the three storey SPEAR building. It is a reinforced concrete structure that represents typical Mediterranean construction. A prototype was tested in full scale at Ispra within the European framework project SPEAR. It was designed only for gravity loads. Further details on the structure and its pseudo-dynamic testing can be found in Fardis (2002) and Fardis and Negro (2006). The SPEAR building is plan-asymmetric in both X and Y directions but is regular in elevation (Fig. 1). Eight of the nine existing columns have a square cross-section of $0.25 \times 0.25\,\text{m}^2$. The column C6 has a rectangular section of $0.25 \times 0.75\,\text{m}^2$, with the higher dimension along the Y direction creating a "weak direction" along the X-axis. The column C6 and the presence of a balcony on the east side of the structure are the major causes for the in-plan irregularity.

The second case study is a five storey building. It is a real Turkish reinforced concrete structure which experienced the 1999 Golcuk earthquake without any damage. The building is

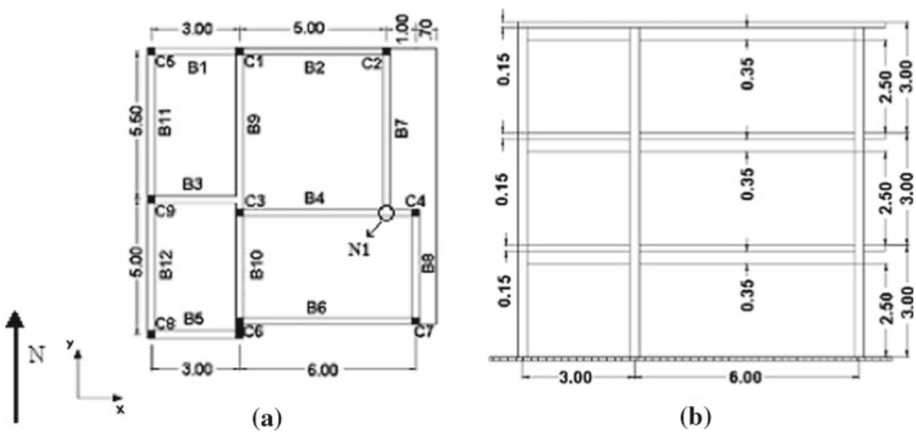


Fig. 1 Building configuration: **a** in plan; **b** at the south west facade (units in meters)

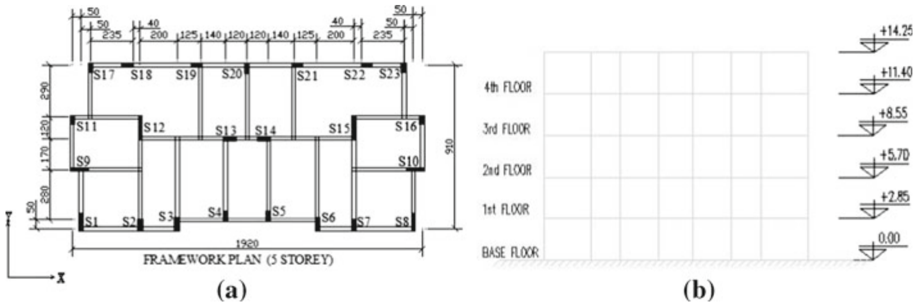


Fig. 2 **a** Plan view (cm), **b** lateral view (m)

asymmetric along the X axis, Fig. 2a, and all the floors keep the same 2.85 m height, Fig. 2b. There are potential weak connections due to the existence of beams framing into beams. There are also walls and elongated columns, as presented in Fig. 2a. The columns sections keep the same geometrical and reinforcement features along the height of the building. The beams sections are mainly $0.20 \times 0.50\,\mathrm{m}^2$ except the two located in the centre of the building that are $0.20 \times 0.60\,\mathrm{m}^2$. The stirrups have 20 cm spacing both for beams and columns. The slabs are 0.10 and 0.12 m thick. For more details on the building's characteristics see Vuran et al. (2008).

The last case study is a real existing 8 storey Turkish reinforced concrete building. It is a plan-asymmetric structure in both X and Y axis, see Fig. 3a. The first storey is 5.00 m height and the upper floors are 2.70 m height, Fig. 3b. The walls and elongated columns have the higher dimension always along the Y direction, for this reason the structure will be more stiff and resistant along this direction. There are beams framing into beams leading to possible weak connections in the structure. The columns sections and reinforcement keep the same geometrical features along the height of the building, except the column S52 that varies from $1.10 \times 0.30\,\mathrm{m}^2$ (on the first floor) to $0.80 \times 0.30\,\mathrm{m}^2$ (on the last floor). The height of this section is reduced in 0.10 m at every two storeys. Columns S9, S12, S15, S18, S46, S69 and S72 are $0.20 \times 1.00\,\mathrm{m}^2$ and columns S23, S36, S57 and S75 are $0.20 \times 1.10\,\mathrm{m}^2$. The beams sections are mainly $0.20 \times 0.50\,\mathrm{m}^2$ except the two located in the centre of the building along the X direction that are $0.30 \times 0.50\,\mathrm{m}^2$ and $0.25 \times 0.50\,\mathrm{m}^2$ respectively. The slabs are 0.12 m thick. For more details on the structural features see Bhatt et al. (2010).

4 Parametric study

In this section the parametric study is described, moreover the seismic action definition and the numerical model used in the performed structural analyses.

4.1 Seismic action

Seven bi-directional semi-artificial ground motion records from the SPEAR project (Table 1) fitted to the EC8 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 (Table 2) taken from the PEER's database website (PEER 2009). They were fitted to the

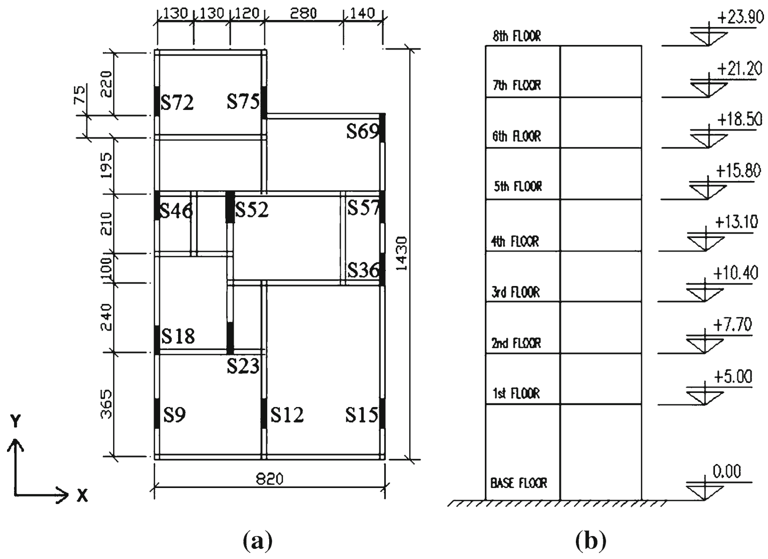


Fig. 3 **a** Plan view (cm), **b** lateral view (m)

Table 1 Ground motion records considered

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

Table 2 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

Eurocode8 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 extended N2 method throughout different levels of structural inelasticity. The accelerograms were scaled for peak ground accelerations of 0.05, 0.1, 0.2 and 0.3 g for the three storey building, for 0.1, 0.2, 0.4, 0.6 and 0.8 g for the five

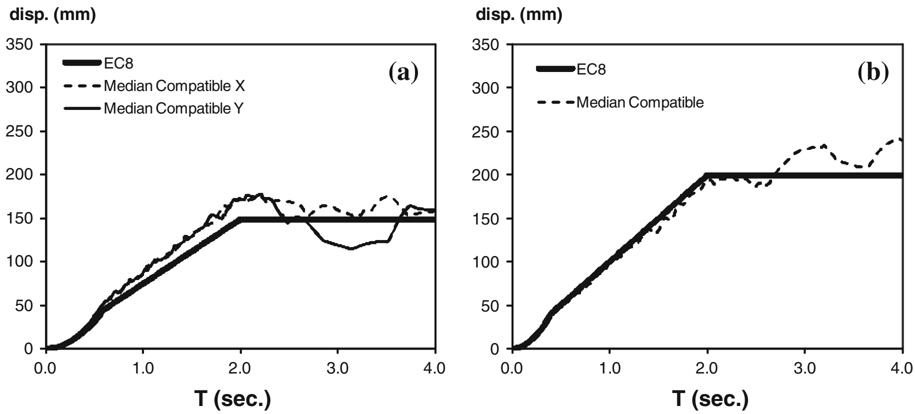


Fig. 4 Displacement response spectra **a** three storey, **b** five and eight storey buildings

storey building and for 0.1, 0.2 and 0.4 g for the eight storey building. The median response spectra of each set of ground motions were used to compute the nonlinear static procedures response. In the three storey building, the median spectrum in the X direction was obtained from the set of response spectra compatible with the X components of the ground motions used. In the Y direction the procedure was the same but now considering the Y components of the accelerograms. For the five and eight storey buildings, and since each pair of accelerograms was applied twice in the structure changing the direction of the components, the median spectrum in the X direction was obtained from the set of response spectra compatible with all X and Y components of the accelerograms. Therefore the median spectrum in the Y direction is the same of the one used in the X direction. They are represented in Fig. 4a, b as defined for the three storey building and for the Turkish buildings, respectively. The reference response spectra defined according to EC8 are also depicted in these figures.

4.2 Numerical modeling

The 3D models used to simulate the buildings, Fig. 5, were developed using SeismoStruct (SeismoSoft 2006), a fibre element based finite element program. Fibre elements were used to model the member's inelasticity and the hysteretic damping was automatically considered in their formulation. A tangent stiffness-proportional damping was considered to take into account for possible non-hysteretic sources of damping. This approach was used following Priestley and Grant (2005). 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 rigid diaphragm effect on the five and eight storey buildings was modelled using the Nodal Constraints with a Penalty Function exponent of 10^7. In the three storey building this effect was considered through the use of a Rigid Diaphragm with Lagrange multipliers modelling strategy.

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

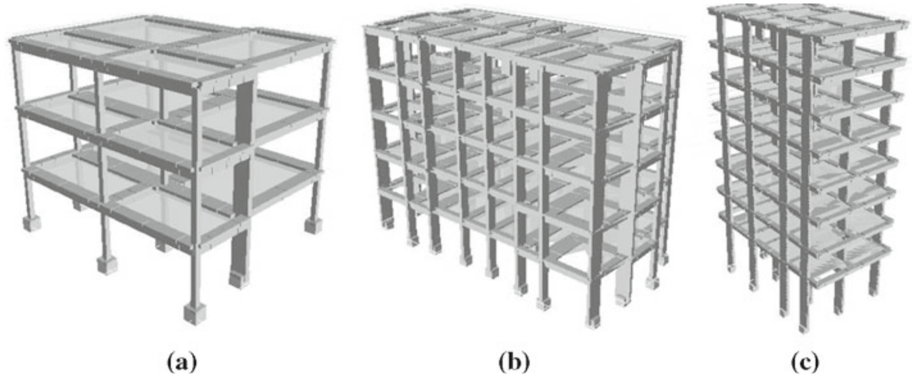


Fig. 5 3D models of the analyzed buildings **a** three storey, **b** five storey, **c** eight storey

strength of 25 MPa was considered for the SPEAR building and 16.7 MPa 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 360 MPa was assumed for the SPEAR building and 371 MPa for the Turkish buildings.

The modelling options used in the three storey building were calibrated with the experimental results. The comparisons between the analytical results and the experimental tests for the SPEAR building can be found in Bento et al. (2010).

Masonry infill panels have not been modelled for the buildings examined herein.

4.3 Structural analyses

Lateral forces were applied in the pushover analysis in the form of modal proportional and mass proportional load patterns. The loads were applied independently in the two X and Y directions in positive and negative senses, resulting in four analyses for each building. The target displacement was computed for each building choosing the larger value in each direction, being the results combined in the two directions using the SRSS combination. For the nonlinear dynamic analysis of the three storey SPEAR building the set of seven bidirectional semi-artificial ground motion records were employed in 4 different configurations: X+Y+, X+Y−, X−Y−, X−Y+. For the nonlinear dynamic analysis of the Turkish buildings, the set of three bidirectional semi-artificial ground motion records were applied with different combinations. Each record was applied twice in the structure changing the direction of the components, resulting in 6 models, each one with five intensity levels for the five storey building and three intensity levels for the eight storey building. The results of the pushover analyses were compared with the *timehistory* results for the different ground motion intensities analyzed.

5 Parametric study—results and discussion

In this section, the results of the aforementioned parametric study are analysed and discussed. The main objective is to assess the seismic behavior of the buildings, evaluating the accuracy of the extended N2 method. The results will be divided in two main sections: the ones

Table 3 Periods (in seconds) and effective modal mass percentages

Mode	Three storey			Five storey			Eight storey		
	Period	[Ux] (%)	[Uy] (%)	Period	[Ux] (%)	[Uy] (%)	Period	[Ux] (%)	[Uy] (%)
1	0.617	60.45	7.83	0.617	76.72	0.00	1.445	91.22	0.00
2	0.527	23.49	42.99	0.593	0.00	77.94	0.636	0.41	1.81
3	0.441	3.15	31.59	0.509	5.02	0.00	0.482	0.21	79.21
4	0.217	7.40	0.77	0.194	10.21	0.00	0.446	4.69	1.87
5	0.180	2.83	3.91	0.173	0.00	12.29	0.241	0.77	0.00
6	0.150	1.64	0.00	0.153	0.40	0.00	0.198	0.00	0.20

corresponding to the elements located in the centre of the buildings, where the torsional effects are not so important as the ones on the extremities; and the ones corresponding to the elements located on the edges of the buildings, where the influence of the seismic torsional motion is very important in their structural response. From the first set one expects to better evaluate the accuracy of the N2 method in computing the target displacements and the response in the central columns for different intensity levels. Since the torsional effect is not very evident for such elements, the extended N2 method uses a correction factor equal to one, see the theoretical background in Chap. 2. Therefore both original and extended procedures lead to the same results. From the second set, one can compare the performance of the original N2 and its extended version in estimating the torsional response in both stiff and flexible sides of the buildings through increasing seismic intensities.

5.1 Dynamic properties

The modal properties of the three analyzed buildings are herein presented. Table 3 shows the periods and the effective modal mass percentages (Ux and Uy) of the three analysed buildings in the X and Y directions. In the Z direction, the effective modal mass percentage is zero for the presented modes.

The three storey building has a fundamental mode of vibration of 0.617 s characterized by translation along the X direction, a second mode of 0.527 s with torsional motion and a third mode of 0.441 s with translation along the Y direction. In this building, translational modes in X and Y directions are coupled with torsional motions. The five storey building presents a first mode of vibration of 0.617 s with translation along the X direction, a second mode of 0.593 s with translation along the Y direction and a third mode of 0.509 s characterised by torsional motion. Finally, the eight storey building has a first mode of vibration of 1.445 s with translation along the X direction, a second mode of 0.636 s with torsional motion and a third mode of 0.482 s with translation along the Y direction. The buildings under analysis present important torsional features due to their irregularities in-plan.

5.2 Assessing the seismic response in the centre of the buildings

The assessment results of the elements located in the centre of the buildings analysed are herein presented. The torsional influence in these elements is not considerable; therefore the results of the original N2 and the extended N2 method are pretty much the same, as it was previously explained. The maximum roof displacement in the centre of mass of each building,

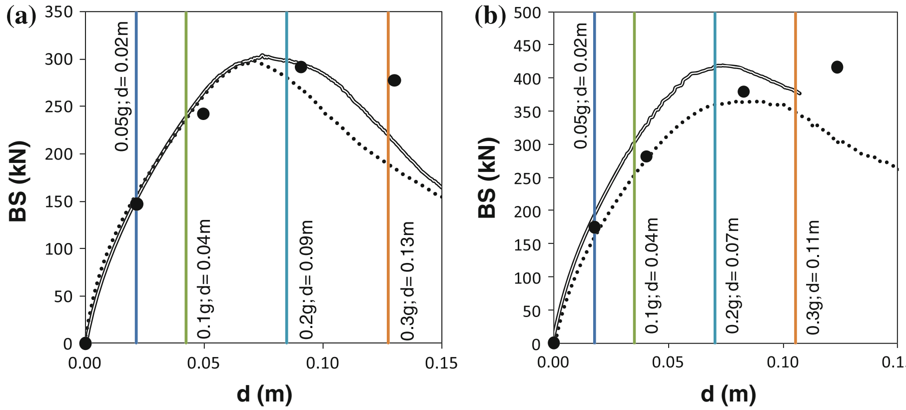


Fig. 6 Three storey, max roof displacement computed with the N2 method **a** X direction, **b** Y direction

computed with the N2 method for different seismic intensities, is plotted with the pushover curves and the median *timehistory* results. Thus, one can better understand how far in the inelastic range the buildings are deformed. Each plot from Figs. 6, 7 and 8, corresponding to the pushover curves and to the nonlinear dynamic analysis, is identified as follows:

•••••Modal X+ ═Modal X- ● TH •••••Modal Y+ ═Modal Y-

The plots identified as Modal X+ correspond to the pushover curves obtained in the X direction applying a modal proportional load pattern along the positive sense of the X direction. On the other hand the Modal X− corresponds to the pushover curves obtained with the same load pattern but now applied along the negative sense of the X direction. The same explanation applies to the Y direction. The TH represents the median *timehistory* results for the different seismic intensities analysed in each case. Obviously, the maximum roof displacement in the centre of mass will be the same either it is computed with the N2 method or with the extended N2 method, because the correction factor in the centre of mass used in the extended N2 procedure is equal to one.

From Fig. 6, one can observe that the three storey building remains elastic in both X and Y directions for seismic intensities of 0.05 and 0.1 g. The building goes through the inelastic range for 0.2 and 0.3 g. For this last intensity the building is in a very high stage of inelasticity for which it actually collapses. From the pushover curves it is clear that the building has slightly more strength along the Y direction. This happens mainly because the elongated column C6, see Fig. 1, has its larger dimension along the Y direction.

The five storey building behaves elastically for seismic intensities of 0.1 and 0.2 g, as shown in Fig. 7. The structure goes through the inelastic stage for 0.4, 0.6 and 0.8 g. For this last intensity the building collapses.

In the three storey building for a seismic intensity of 0.3 g and in the five storey building for a seismic intensity of 0.8 g, the plastic behavior takes place in a number of sections enough to create a collapse mechanism. Therefore, the buildings collapse for these levels of intensity.

The eight storey building presents an unbalanced stiffness and strength distribution between the two X and Y direction. From Fig. 8 one can see that the building has more strength and stiffness along the Y direction. In fact, Fig. 8b shows that the building remains elastic in this direction through all the seismic intensities analysed. On the other hand, the

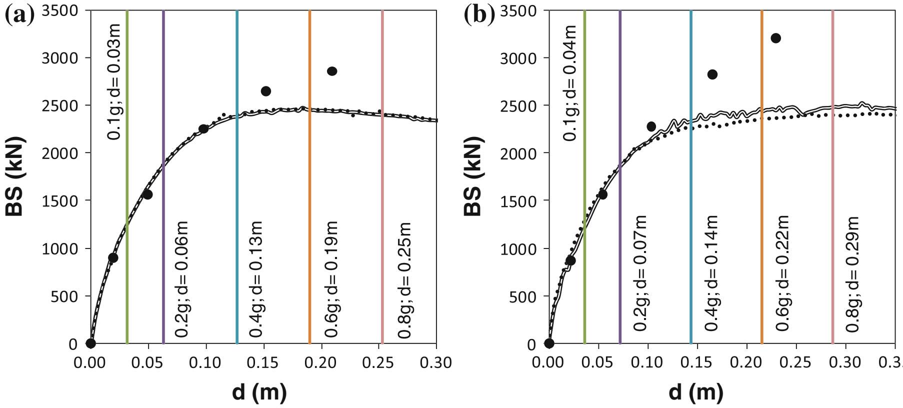


Fig. 7 Five storey, max roof displacement computed with the N2 method **a** X direction, **b** Y direction

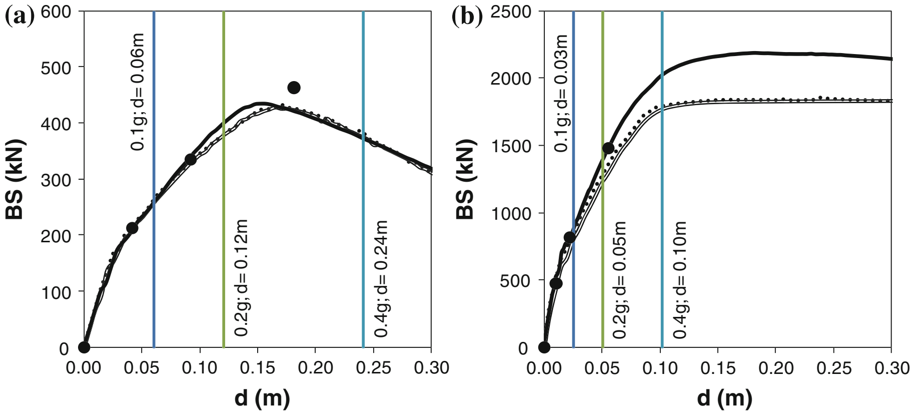


Fig. 8 Eight storey, max roof displacement computed with the N2 method **a** X direction, **b** Y direction

structure remains elastic in the X direction only for 0.1 g and goes through the inelastic stage for 0.2 and 0.4 g. For this last intensity, the building collapses due to a soft storey mechanism in the first floor along the X direction.

From the presented pushover curves one can observe that the N2 method generally leads to conservative maximum roof displacements when compared with the nonlinear dynamic analysis. The exception is the three storey building where the results have a good match with the *timehistory*. It is also clear from the pushover curves and from the *timehistory* median results that the buildings under analysis have poor ductility. The pushover curves match quite well the nonlinear dynamic analysis results in all the buildings analysed.

Figures 9 and 10 illustrate the lateral displacement pattern, the interstorey drifts and the chord rotations profiles for columns located in the centre of the buildings near the centre of mass. Once again, both N2 and extended N2 lead to the same results. Each method is represented in the subsequent plots, Figs. 9 and 10, where TH represents the *timehistory* results:

From the plots, one can generally conclude that the N2 method reproduces in a very good manner the seismic behavior of the columns in the centre of the studied buildings through

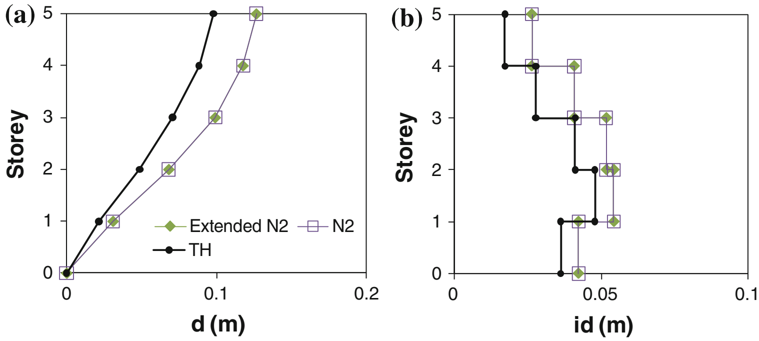


Fig. 9 Five storey building, **a** Lateral displacement pattern S13, X 0.4 g, **b** Interstorey drifts S14, Y 0.6 g

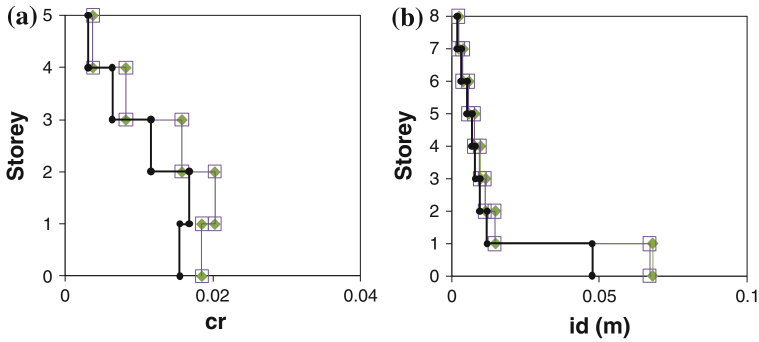


Fig. 10 **a** Five storey building, chord rotation S13, X 0.6 g, **b** Eight storey building, interstorey drifts S23, X 0.2 g

all the seismic intensities tested. Although, the method slightly overestimates the response in such elements. It can capture the soft storey mechanism in the first floor of the eight storey building overestimating its interstorey drift, as shown in Fig. 10b.

This overestimation of the response by the N2 method can be justified for the following reasons:

1. The N2 method calculates the single degree of freedom (SDOF) period independently of the seismic action intensity, thus also independently of the current structural stage. It uses the point of maximum acceleration of the capacity curve of the SDOF in order to bilinearise it and calculate the period. For SDOF periods higher than the spectral Tc the procedure advocates that the response of the SDOF is elastic. In the case studies analysed, mainly in the Turkish buildings, the periods of the SDOFs are always higher than the respective Tc. For intensity levels in which the response spectrum intersects the SDOF curve before the point of maximum acceleration, the period computed with the N2 method will be higher than the real one. Therefore, the displacements calculated elastically will be conservative. Note that, for instance, the CSM with the features proposed in FEMA440 computes the effective period of the SDOF based on the seismic intensity and on the current structural stage. For such, the method bilinearises the SDOF capacity curve in the point of intersection with the response spectrum, computing the ductility and then the effective period.

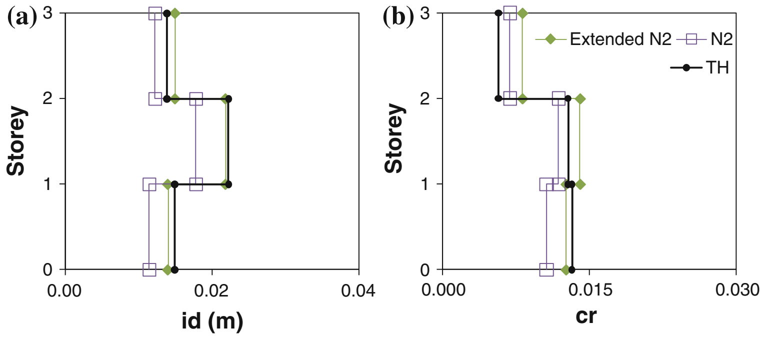


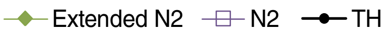
Fig. 11 Three storey building **a** interstorey drifts C2, Y 0.1 g, **b** chord rotation C2, Y 0.2 g

2. The use of the equal displacement rule assumed by the method for SDOF periods higher than Tc, may lead to conservative results (Fajfar 2000). In fact, the experience in nonlinear analysis shows that the peak responses are usually smoothed in the inelastic stage.

One can conclude that the N2 method gains on simplicity, leading to conservative but not exaggerated results. The use of this simplified procedure in design offices is worthwhile since there is an important and considerable saving on computation time when compared with *timehistory* analysis.

5.3 Assessing the seismic torsional response on the edges of the buildings

In this section the extended N2 method results are compared with the original N2 method proposed in Eurocode8 and with the nonlinear dynamic *timehistory* median results for the columns located in the extremities of the buildings. Therefore, one can evaluate the performance of each method in estimating the torsional behavior of the case studies. This comparison is done for all the buildings under analysis and for all seismic ground motion intensities. Next, are presented the interstorey drifts and chord rotations for the three storey building in the flexible edge, column C2, for different seismic intensities. The plots in Fig. 11, are represented as follows (the same legend as Figs. 9, 10):



From Fig. 11, one can see that the N2 method generally underestimates the interstorey drift and the chord rotation profiles through all the intensity levels analysed, in the flexible side of the building. This happens because the method cannot capture the seismic torsional effect that amplificates the response on this extremity of the building. On the other hand, the extended N2 method can accurately capture this behavior due to the use of a correction factor based on an elastic response spectrum analysis.

When dealing with plan-asymmetric buildings the normalized top displacements is the measure one should analyse in order to understand the torsional behavior of the structure (Fajfar et al. 2005). This measure is obtained by normalizing the edge displacement values with respect to those of the centre of mass. Several plots are presented showing the performance of the analyzed procedures in estimating the torsional motion of the analyzed buildings. Figures 12, 13, 14, 15, 16, 17 and 18 illustrate the torsional response of the three analyzed buildings by comparing the results obtained with both original and extended N2, with the *timehistory* and with the response spectrum analysis. In the response spectrum

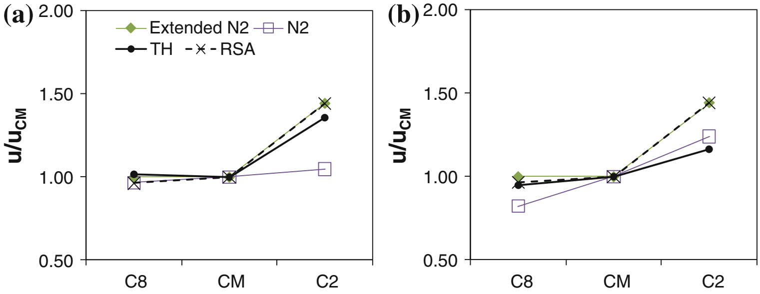


Fig. 12 Three storey X direction **a** 0.1 g, **b** 0.3 g

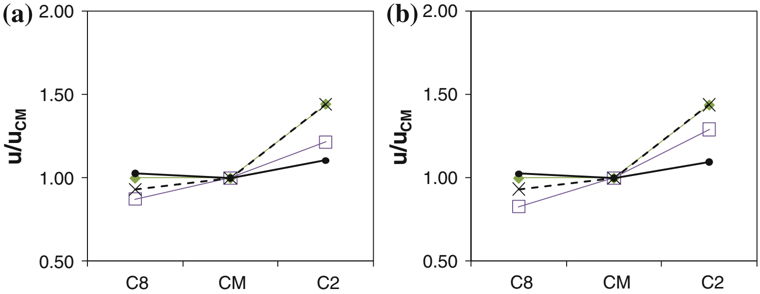


Fig. 13 Three storey Y direction **a** 0.2 g, **b** 0.3 g

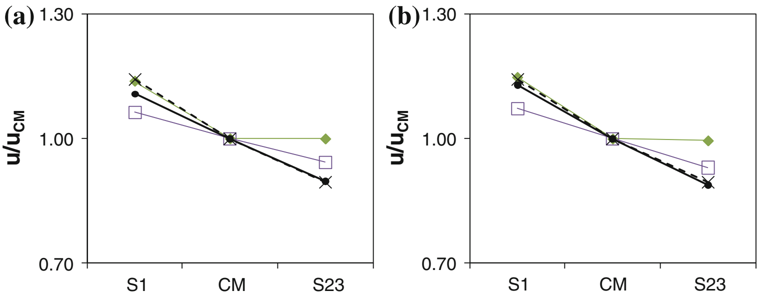


Fig. 14 Five storey X direction **a** 0.1 g, **b** 0.2 g

analysis, the elastic response spectra were applied in the structure in both directions at the same time. The results were combined in the two directions using the SRSS combination, and the modes of vibration were combined using the CQC combination. In Figs. 12, 13, 14, 15, 16, 17 and 18, TH represents the *timehistory* and RSA the elastic response spectrum analysis.

From Fig. 12a one can see that, for the three storey building, for 0.1 g in the X direction, the extended N2 method captures very well the torsional response in both sides of the building. It perfectly reproduces the amplification of displacements on the flexible side of the building, column C2, while the original N2 method provides non-conservative results for this column. Both methods reproduce in a very good fashion the response on the stiff edge, column C8. The RSA is able to capture the torsional response in both sides of the structure.

Figure 12b shows that the extended N2 method reproduces in a very accurate way the response of column C8 for 0.3 g, in the X direction. In fact, this extended procedure does

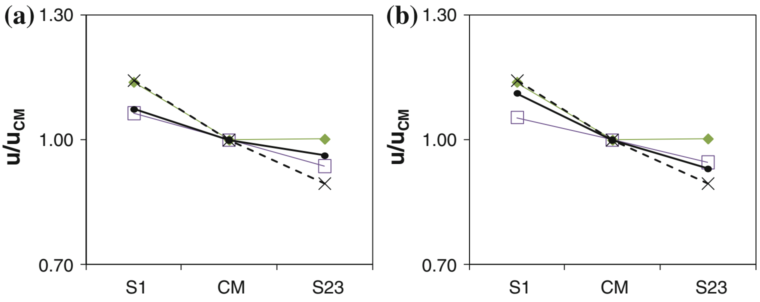


Fig. 15 Five storey X direction **a** 0.4 g, **b** 0.6 g

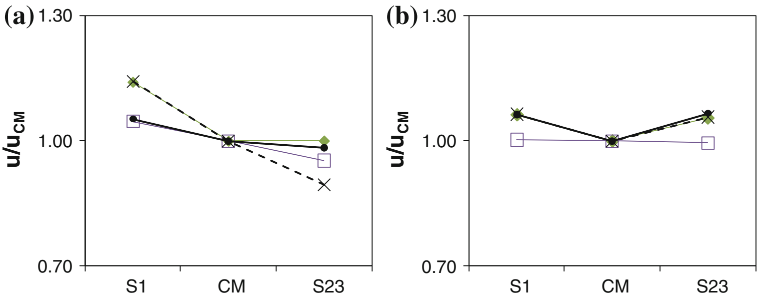


Fig. 16 Five storey **a** X direction 0.8 g, **b** Y direction 0.8 g

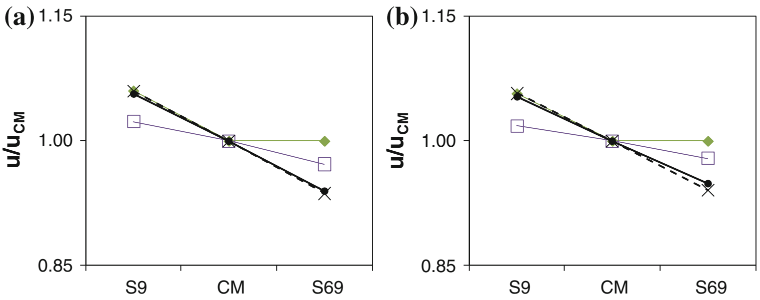


Fig. 17 Eight storey X direction **a** 0.1g, **b** 0.2 g

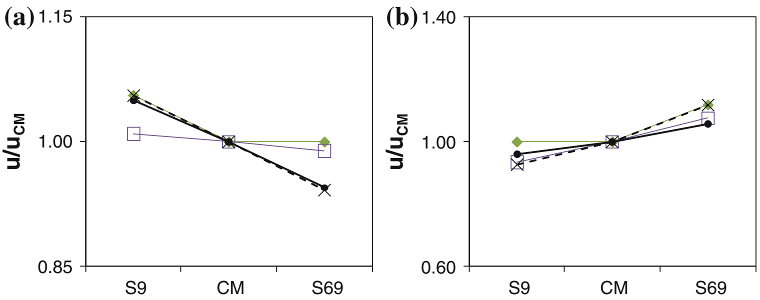


Fig. 18 Eight storey **a** X direction 0.4 g, **b** Y direction 0.4 g

not consider any de-amplification of displacements on the stiff side. On the contrary, the original N2 considers the de-amplification in this node, leading to underestimated results. The extended method overestimates the response on the flexible side for this intensity, while the original N2 reproduces in a very good way the *timehistory* results. The RSA leads to the same results as the ones obtained with the extended N2 method.

From Fig. 13, for 0.2 and 0.3 g in the Y direction, one can see that the extended N2 method reproduces in a very good fashion the results in column C8 and overestimates the response of column C2. The original N2 method and the RSA underestimate the results on the stiff edge and overestimate the displacements on the flexible one.

In the three storey building one can conclude that the N2 method always gives a linear estimation of the structural torsional motion from one side of the building to the other, for all intensity levels analyzed. It usually considered the de-amplification on the stiff edge, leading to underestimated results. On the other hand the extended N2 method did not consider any de-amplification of displacements due to torsion.

Figures 14, 15 and 16 illustrate the torsional response of the five storey building in both X and Y directions, for the different intensity levels analyzed.

In the five storey building, in the X direction for all intensity levels (except for 0.8 g), Figs. 14 and 15, one can observe that the extended N2 method accurately predicts the response of column S1, the flexible edge of the building, and overestimates the results in column S23, the stiff edge.

This procedure does not consider the de-amplification of displacements due to torsion in column S23, the stiff side of the building. For 0.8 g, Fig. 16a, the extended N2 method overestimates the response in column S1 and reproduces in a very accurate fashion the response of column S23.

In the X direction, for 0.1 and 0.2 g, Fig. 14, the N2 method underestimates the results in column S1 and overestimates the response in column S23. This procedure considers the de-amplification of displacements on the stiff edge caused by torsion.

From Figs. 15 and 16a, one can see that the original N2 method estimates correctly the response of the column S1 for 0.4 and 0.8 g, and of the column S23 for 0.6 g. It underestimates the displacements of column S23 for 0.4 and 0.8 g, and of column S1 for 0.6 g. The results obtained by the RSA in the X direction match the extended N2 on the flexible side of the building for all seismic intensities. This procedure leads to the same results as the *timehistory* on the stiff side of the building for 0.1 and 0.2 g. For 0.4, 0.6 and 0.8 g the RSA underestimates the results computed with the nonlinear dynamic analysis on this side of the building. Therefore, the theoretical approach of the extended N2 method, by considering the RSA ratio equal to one when it is in fact smaller than one, allows the method to give conservative results.

In the Y direction, the extended N2 method reproduces quite well the torsional behavior of the building. The original N2 method generally underestimates the real response, except in column S1 for 0.1 and 0.4 g where it matches the *timehistory* median response. The RSA leads to the same results as the extended N2 procedure in both sides of the building through all the seismic intensities.

For this building one can see that once again the N2 method always provides a linear estimation of the torsional motion for all intensity levels.

In Figs. 17 and 18 are plotted the torsional behavior of the eight storey building.

From Figs. 17 and 18a, it is clear that the extended N2 method perfectly reproduces the response of column S9 in the X direction. It does not consider the de-amplification of displacements due to torsion in column S69, so the method overestimates the response on this side of the building. The original N2 underestimates the displacements in column S9 and overestimates the response of column S69. Note that the response in the X direction of the original and the extended N2 in column S69, gets closer as the ground motion intensity increases. The RSA leads to the same results as the *timehistory* in both sides of the building in the X direction, through all the seismic intensities.

In the Y direction the extended N2 method reproduces perfectly the response of column S9. It slightly overestimates the results of column S69 for 0.4 g, Fig. 18b.

The original N2 slightly underestimates the response in column S9 for 0.2 and 0.4 g, and it captures the real motion of this column for 0.1 g. In column S69 the method provides non-conservative results for 0.1 and 0.2 g and it gets close to the nonlinear dynamic analysis for 0.4 g. Along the lines of what happened in the previous two analyzed buildings, in the eight storey building the N2 method provides a linear estimation of the structural torsional motion for all intensity levels analyzed.

The results of the RSA match the extended N2 method in column S69 in the Y direction for all intensity levels. The method underestimates the *timehistory* results in column S9, once again for all seismic intensities.

The obtained results lead to the conclusion that torsional effects are generally higher for lower ground motion intensities. For increasing seismic intensities, one can understand a flattening on the normalized top displacements. This can be seen in all the buildings analyzed. This conclusion confirms the idea that torsional effects are generally smaller in the inelastic range compared to what happens in the elastic one.

The plots clearly show that the RSA estimates an upper bound of the torsional amplification on the flexible side of the buildings, both in the elastic and in the inelastic range.

The extended N2 method reproduces in a very good fashion the nonlinear dynamic results for all the buildings analyzed and through all the seismic intensities tested. This method shows, for these case studies, a much better performance in estimating the torsional behavior of the buildings than the original N2 method. Generally the last one is not capable to reproduce the torsional response of the buildings.

The aforepresented plots show that the extended N2 method reproduces in a very accurate way the torsional amplification on the flexible edge in all the buildings analysed through all the increasing intensities. This good performance is justified because this extended procedure 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 N2 method generally underestimates the torsional amplification of the displacements on the flexible side.

From the plots it is evident that both RSA and the original N2 consider the torsional de-amplification on the stiff side of the buildings, leading in some cases to underestimated results. Figure 13 (column C8), Fig. 15 (column S23), Fig. 16a (column S23) and Fig. 18b (column S9) illustrate the cases where the RSA leads to normalized top displacements smaller than one on the stiff edge, being these results non-conservative when compared with the *timehistory*. Therefore, whenever the RSA leads to normalized top displacements smaller than one, the extended N2 method considers this value to be equal to one. This recommendation avoids the extended method to produce non-conservative results on this stiff edge.

The N2 method always provides a linear estimation of the torsional motion from one side of the building to the other, through all the seismic intensities. The extended N2 method does not consider any de-amplification of displacements due to torsion, leading in some cases to very accurate results and in others to conservative responses on the stiff edge of the buildings.

The results obtained herein seem quite optimistic regarding the implementation of this extended procedure in Eurocode8. However, one should be aware that the interplay among ground motion, inelastic amplification or de-amplification of displacements and structural system is complex. Therefore, more studies in different buildings should be developed in order to consolidate this nonlinear static approach.

6 Conclusions

In this paper, three real RC buildings with three, five and eight storey were assessed for several ground motion intensities. The extended N2 method was compared with the original N2, recommended in Eurocode8, and with the *timehistory* median response. The study presented in this endeavour aims to continue the work developed by Peter Fajfar and his team for the development of the extended N2 method.

This procedure consists on the application of a correction factor to the pushover analysis results determined by the original N2 method. These correction factors are calculated based on a linear elastic analysis and on a pushover analysis.

In the first part of the study, the seismic response was assessed in the centre of the analysed buildings. The torsional effects were not so evident in such location, therefore the results obtained with both original and extended N2 method were pretty much the same. The results in the centre of mass and in columns near this point, were generally overestimated for all seismic intensities analysed. This trend was justified based on the features of the procedure. Despite this fact, the method seems to gain in simplicity and its use is certainly worth, mainly in design offices where the time saving is considerable when compared with the *timehistory* analysis.

In the second part, the seismic assessment was performed in the elements located on the edges of the buildings in order to evaluate their torsional response. The results obtained from this study showed that torsional effects are in general higher for lower ground motion intensities. In fact, for increasing seismic intensities, one can notice a flattening on the normalized top displacements of each building. This confirms the idea that torsional effects are generally smaller in the inelastic range than in the elastic stage.

The extended N2 method performed in a much more accurate way than its original counterpart in estimating the torsional behavior of all buildings analyzed through all the seismic intensities tested. It generally captured in a very precise way the torsional amplification in terms of displacements on the flexible side of the buildings. The extended N2 method does not take into account any de-amplification of displacements due to torsion. Therefore, the response on the stiff side of the buildings was in some cases estimated in a very precise way by the method, and overestimated in others.

The N2 method is not capable in general to reproduce the torsional motion of the buildings, usually leading to a linear estimation of the torsional motion from one side of the building to the other. The original method considered the de-amplification on the stiff side of the buildings, underestimating their response through all the seismic intensities tested. On the flexible side, the normalized top displacements were also generally non conservative in respect to the *timehistory* results.

The proposed extended N2 method should be also tested considering the effect of masonry infill panels. This effect has been omitted in this work for reasons of a more thorough research. The non-uniform (not simultaneous) failure of masonry infill panels during the non-linear seismic response of a structure, may increase its torsional behavior and, thus, should be taken into account. This may increase the asymmetry and consequently increase the torsional effects. Therefore, the effect of infills from modelling till their non-uniform failure may become important.

Recently, several procedures have been proposed taking into account torsion in simplified nonlinear static procedures, however definitive answers have not yet been reached. This paper certainly does not present a breakthrough, but it does make a step forward. The results obtained herein added to the ones already published, confirm the idea that the extended N2 method has potential to be implemented in the next version of Eurocode8 in order to correctly estimate the torsional response in real plan-asymmetric RC buildings through the use of pushover analysis.

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