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# Seismic assessment of existing r.c. framed structures with in-plan irregularity by nonlinear static methods

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**Abstract.** This paper evaluates the effectiveness of three nonlinear static methods for the prediction of the dynamic response of in-plan irregular buildings. The methods considered are the method suggested in Eurocode 8, a method previously proposed by some of the authors and based on corrective eccentricities and a new method in which two pushover analyses are considered, one with lateral forces applied to the centres of mass of the floors and the other with only translational response. The numerical analyses are carried out on a set of refined models of reinforced concrete framed buildings. The response predicted by the nonlinear static analyses is compared to that provided by nonlinear dynamic analyses. The effectiveness of the nonlinear static methods is evaluated in terms of absolute and interstorey displacements.

**Keywords:** asymmetric buildings; framed buildings; reinforced concrete; nonlinear static methods; assessment

## 1. Introduction

In several earthquake-prone countries, e.g., in Italy, many buildings have been designed by non-seismic regulations and others have been constructed according to old seismic codes. In the light of the performance objectives of modern seismic codes, the seismic performance of these buildings is expected to be inadequate. This is even more true of in-plan irregular buildings where the floor rotations may cause an increase of the ductility demands and induce a concentration of damage.

The development of intervention techniques able to safeguard the human life and reduce the structural damage has been particularly discussed in the scientific community (e.g., see Formisano *et al*. 2010). However, effective inelastic methods of analysis are expected to be available to validate the selected intervention of structural rehabilitation. The linear method of analysis used

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for the design of new buildings is not suitable for this purpose because the plastic collapse mechanism of existing structures, and thus the behaviour factor, is not known *a priori*. Owing to this, this method of analysis does not provide a reliable prediction of the response of existing structures. The nonlinear dynamic analysis considers explicitly the inelastic deformation of structural members and can predict effectively the seismic response of existing buildings. However, the complications of modelling of both the seismic ground motion and the nonlinear cyclic behaviour of structural members make this analysis not recommended for everyday use. The nonlinear static method of analysis represents a fair compromise between the elastic method of analysis and the nonlinear dynamic analysis. In particular, it implies the use of response spectra and requires that modelling efforts be focused only on the monotonic nonlinear behaviour of structural members. Provided that proper load patterns are applied (Chopra and Goel 2002, Gupta and Kunnath 2000, Mwafy and Elnashai 2001, Requena and Ayala 2000, Sasaki *et al.* 1998, Valles *et al.* 1996, Bracci *et al.* 1997), this method of analysis predicts accurately the plastic collapse mechanisms of low to moderate rise buildings and provides a satisfactory estimate of the demanded plastic deformations. This method of analysis is included in all the modern seismic codes and is nowadays the most popular method for the seismic assessment of existing buildings.

The nonlinear static method of analysis suggested in Eurocode 8 (EC8, CEN 2004) has been formulated by Fajfar and his research team for regular planar frames (Fajfar and Fishinger 1988, Fajfar 1999, Fajfar and Gaspersic 1996). It generally provides satisfactory results for planar frames (Bosco *et al.* 2009b, Giorgi and Scotta 2013) but usually does not yield an effective prediction of the deck rotation of in-plan irregular structures. This consideration also applies to similar nonlinear static methods of analysis reported in other seismic codes. To eliminate this drawback, some improved versions of the nonlinear static method of analysis have recently been proposed (Bento *et al.* 2010, Bosco *et al.* 2009a, 2012, Kreslin and Fajfar 2012, Fujii 2011, 2013).

In this paper the effectiveness of the nonlinear static method suggested in Bosco *et al.* (2009a, 2012) is deeply investigated and a simplified nonlinear static method is proposed. Their prediction is compared with that of the nonlinear static method of analysis suggested in Eurocode 8 and with the results of nonlinear dynamic analyses. The basics of the method suggested in Bosco *et al.* (2009a, 2012), later called *corrective eccentricity method*, come from the observation that in asymmetric structures the in-plan distribution of the maximum dynamic displacements is nonlinear and that very often this distribution cannot be approximated properly by the results of a single pushover analysis. For this reason, the authors suggested that the pushover analysis should be performed twice, with lateral forces applied to two different sets of points of the decks. The distances of these points from the centres of mass were named *corrective eccentricities* and defined in such a way that, on either side of the building, the maximum displacement obtained by the two pushover analyses matched the maximum displacement obtained by nonlinear dynamic analysis. The corrective eccentricities were calibrated in Bosco *et al.* (2012) based on a numerical investigation on single-storey systems endowed with shear-type resisting elements with bilinear force-displacement relationship. The effectiveness of the corrective eccentricity method was demonstrated in Bosco *et al.* (2012, 2013a) by comparison with the results of other nonlinear static methods of analysis. The systems considered in these past studies were single-storey systems with the exception of only a few multi-storey framed systems. In the latter cases, however, the response of the members was obtained by very simple models. In particular, (1) beams and columns were modelled by means of elastic elements with plastic hinges lumped at their ends; (2) the moment-rotation relationship of the plastic hinges was assumed to be elastic perfectly plastic and applied with reference to the two principal bending planes of the cross-section, independently considered;

and (3) the interaction between the bending moments and the axial force was neglected. Like the corrective eccentricity method of analysis, the one proposed in this paper requires a double application of the pushover analysis. Specifically, while in the first pushover analysis the lateral forces are applied to the centres of mass, in the second a purely translational response is considered. This second pushover analysis aims to avoid the underestimation of the displacement demand usually predicted by the Eurocode 8 method on the rigid side of reinforced concrete framed buildings. For this reason, this method of analysis is later referred to as the *improved Eurocode 8 method*.

Past comparisons between results from single- and multi-storey systems highlighted significant differences in the prediction of the seismic response of asymmetric building (Anagnostopoulos *et al.* 2010, De Stefano *et al.* 2006, Ghersi *et al*. 2007, Stathopoulos and Anagnostopoulos 2003, 2005). For this reason, the effectiveness of the abovementioned three non linear static methods in the prediction of the seismic response of r.c. framed buildings is investigated in this paper by means of refined multi-storey models in which the abovementioned limits of modelling are overcome. The structural type under examination is very common worldwide (e.g., most of the buildings constructed in Italy in the last decades belong to this type). The buildings are designed for either gravity loads or gravity and seismic loads according to the regulations in force in Italy from the 1970s (Italian Ministry of Public Works 1971, 1974a, b, 1996). For each building, the prediction of the nonlinear static methods of analysis is compared to the response obtained by nonlinear dynamic analysis.

## 2. Nonlinear static methods

### *2.1 The Eurocode 8 method*

The method reported in Eurocode 8 is characterised by a single pushover analysis with lateral loads applied to the centres of mass. This method is accurate in the prediction of the response of torsionally rigid systems, but fails in the other cases (Bosco *et al.* 2012, Chopra and Goel 2004, Fajfar *et al.* 2005, De Stefano *et al.* 2013, 2014).

### *2.2 The corrective eccentricity method of analysis*

The response parameter is obtained, for each direction of the seismic action, as the maximum between the values provided by two pushover analyses. The lateral forces are applied to two points of the deck that are generally different from the centre of mass $C_M$. The corrective eccentricities $e_i$ are defined for the two pushover analyses as a function of four parameters: the rigidity eccentricity $e_r$ (distance between the centre of rigidity $C_R$ and $C_M$), the ratio $\Omega_\theta$ of the uncoupled torsional to lateral frequencies, the strength eccentricity $e_s$ (distance between the centre of strength $C_S$ and $C_M$) and the ratio $R_\mu$ of the elastic strength demand to the actual strength of the corresponding planar system. While $e_r$ and $\Omega_\theta$ influence the torsional response of the system in both the elastic and inelastic range of behaviour (Hejal and Chopra 1987, Goel and Chopra 1990, Palermo *et al*. 2013), $e_s$ and $R_\mu$ influence only the inelastic torsional response (Goel and Chopra 1990). The relations which defines the corrective eccentricities are

$$e_\mathrm{i} = a_\mathrm{i}\, e_s + b_\mathrm{i}\, e_\mathrm{r} \qquad \text{i=1 or 2} \tag{1}$$

where the parameters $a_i$ and $b_i$ are reported in Bosco *et al.* (2012) as a function of $\Omega_\theta$ and $R_\mu$. In particular, $e_1$ is the eccentricity used to predict the displacements on the rigid side of the building whereas $e_2$ is that used for the flexible side.

### *2.3 The improved Eurocode 8 method of analysis*

This nonlinear static method (Ferrara 2012) considers two pushover analyses. Unlike the corrective eccentricity method of analysis, the improved Eurocode 8 method does not entail any effort for the identification of the points of application of the lateral forces. In the first pushover analysis the lateral forces are applied to the centres of mass (3D analysis). In the second, instead, the deck rotations are restrained (2D analysis) in keeping with the provisions of some seismic codes, e.g., Uniform Building Code (ICBO 1997), for the design of torsionally unbalanced buildings. The first pushover analysis aims to equal the static displacement on the flexible side of the deck and the dynamic response on the same side. The second analysis aims to capture the response on the rigid side of the building.

## 3. Analysed buildings

The effectiveness of the methods of analysis considered is evaluated on ten reinforced concrete mass eccentric multi-storey buildings. All the buildings are endowed with hollow clay block-cement mix decks and are served by longitudinally-supported stairs, i.e. by stairs designed as one-way reinforced concrete slabs and supported by beams at the top and bottom of the flights. The geometric characteristics (i.e., total height, interstorey height, length and width of the deck) are equal for all the buildings. In particular, the buildings examined are five-storey high and are characterised by an interstorey height equal to 3.20 m. The deck is rectangular shaped with maximum dimension $L$ equal to 28.5 m and minimum dimension $B$ equal to 15.5 m (Fig. 1).

The lateral stiffnesses of the resisting members are symmetric with respect to the principal axes of the deck. Consequently, the elastic axis of the building exists and overlap the vertical axis passing through the geometric centres of the decks. Masses are assumed to be lumped at deck level and distributed in the same manner within every deck of the building. The centres of mass of the storeys are lined up on a single vertical axis and belong to the $x$-axis because masses are assumed to be symmetric with respect to this axis. The centres of mass are distant from the $y$-axis of a quantity $e_m$ named *mass eccentricity*. Two values of the mass eccentricity are considered. These values are equal to $0.05L$ and $0.15L$ and are later named *low* and *high mass eccentricity*. As shown in Fig. 1, the centres of mass are on the right side of the elastic centres. Therefore, the left side of the deck is later referred to as the rigid side while the right one is called the flexible side.

The characteristic values of the permanent loads of decks and stairs are equal to 5.6 and 4.2 kN/m$^2$, respectively, while the characteristic values of the variable loads are 2.0 and 4.0 kN/m$^2$. The characteristic permanent load transferred by claddings to beams is equal to 7.0 kN/m.

The buildings are designed according to different building regulations and are expected to be representative of buildings constructed in Italy in the last forty years. Specifically, two buildings are designed to support only gravity loads and are later identified by means of the label GL; all the other buildings are designed to resist gravity and seismic loads and are identified by the character S. These latter buildings are subdivided further into two groups. The buildings of the first group (denoted as ST) are designed by seismic forces applied to two separate planar models along the $x$-

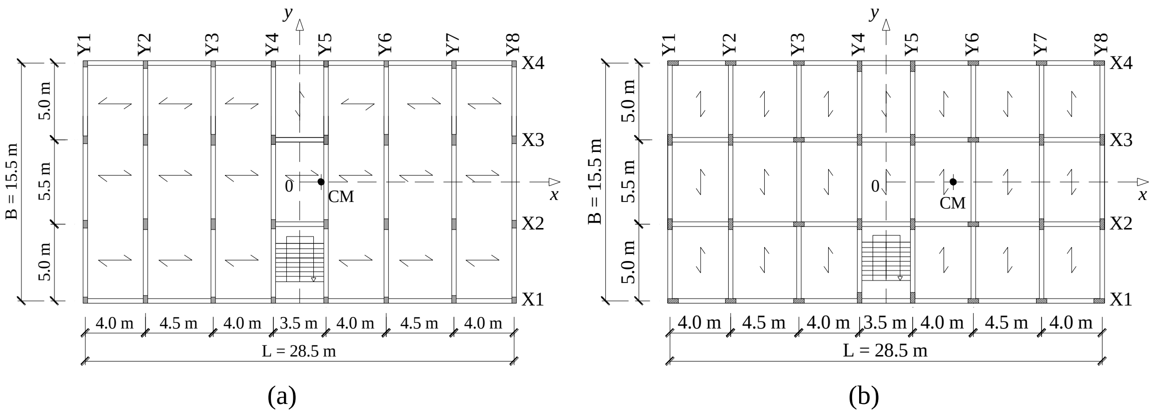


Fig. 1 Plan of the building: (a) type GL; (b) type SR

and $y$-directions. The seismic forces considered for the buildings of the second group (labelled as SR) are applied to the centres of mass of the spatial model along the $x$- and $y$-directions, separately considered.

Fig. 1(a) shows the floor plan of the buildings designed to support only gravity loads. Decks are supported by eight three-bay plane frames (Y1 to Y8) in the $y$-direction, two seven-bay plane frames (X1 and X4) and two one-bay plane frames (X2 and X3) in the $x$-direction. The lack of transverse beams in the $x$-direction (with the exception of frames X1 and X4, beams lying in the $x$-direction are present only in the central bay of the perimeter frames) and the orientation of the column cross-sections (stretched in the $y$-direction) make buildings type GL particularly flexible for seismic forces acting in the $x$-direction. These buildings are characterised by lateral stiffnesses that are significantly different in the $x$- and $y$-directions, as common in buildings constructed prior to the enforcement of seismic regulations. The mass of each floor is equal to 440.6 t while the mass radius of gyration is 9.91 m (0.348 $L$). The characteristic compressive cylinder strength of concrete $f_{ck}$ is equal to 20.75 MPa. The transverse and longitudinal reinforcements consist of deformed bars (steel type FeB38k) with a characteristic yield strength equal to 380 MPa. The structure is designed according to the allowable stress design method, as reported in the building code issued in Italy in the early 1970s (Italian Ministry of Public Works 1974b). The allowable stresses considered for concrete and steel bars are 8.5 MPa and 215 MPa, respectively. It should be noted that columns are designed on the basis of the sole axial force and that this internal force is calculated on the basis of tributary areas. In any case, concrete and reinforcement cross-sectional areas are not lower than the minimum values considered in the abovementioned Italian code.

Fig. 1(b) shows the floor plan of the buildings designed to resist gravity and seismic loads. Decks are supported by four seven-bay frames in the $x$-direction (X1, X2, X3and X4) and eight three-bay frames in the $y$-direction (Y1 to Y8). These buildings are endowed with beams along both the $x$- and $y$-directions. The storey mass $m$ is equal to 477.7 t and the radius of gyration of the mass $r_m$ is 9.87 m (0.346$L$). The characteristic value of the compressive strength of concrete $f_{ck}$ is 20.75 MPa and the characteristic value of the yield strength of the deformed bars (steel type FeB44k) is equal to 430 MPa. The seismic design loads are evaluated according to the Italian Code (Italian Ministry of Public Works 1974a, 1996) assuming that the building is located in areas prone to moderate intensity earthquakes (II category, base shear coefficient equal to 0.07).

Table 1 Uncoupled periods of vibration

| Building type | Triplet | $T_x$ [s] | $T_y$ [s] | $T_\theta$ [s] |
|---|---|---|---|---|
| GL | 1 | 1.1078 | 0.7119 | 0.7042 |
| | 2 | 0.3680 | 0.2436 | 0.2389 |
| | 3 | 0.2187 | 0.1572 | 0.1419 |
| ST and SR | 1 | 0.6370 | 0.6580 | 0.5882 |
| | 2 | 0.2084 | 0.2125 | 0.1906 |
| | 3 | 0.1204 | 0.1203 | 0.1092 |

Table 2 Coupled periods of vibration

| Building type | $e_m/L$ | Triplet | $T_x$ [s] | $T_{y\theta}$ [s] | $T_{\theta y}$ [s] |
|---|---|---|---|---|---|
| GL-L | 0.05 | 1 | 1.1078 | 0.7606 | 0.6591 |
| | | 2 | 0.3680 | 0.2591 | 0.2245 |
| | | 3 | 0.2187 | 0.1533 | 0.1329 |
| GL-H | 0.15 | 1 | 1.1078 | 0.8760 | 0.5723 |
| | | 2 | 0.3680 | 0.2981 | 0.1954 |
| | | 3 | 0.2187 | 0.1763 | 0.1152 |
| ST-L and SR-L | 0.05 | 1 | 0.6370 | 0.6547 | 0.5488 |
| | | 2 | 0.2084 | 0.2115 | 0.1777 |
| | | 3 | 0.1204 | 0.1200 | 0.1015 |
| ST-H and SR-H | 0.15 | 1 | 0.6370 | 0.7680 | 0.5040 |
| | | 2 | 0.2084 | 0.2483 | 0.1632 |
| | | 3 | 0.1204 | 0.1412 | 0.0924 |

The building is designed for residential purposes and is founded on soils of medium compressibility. The beam and column concrete cross-sections are equal in all the buildings designed to resist seismic loads. The reinforcements are, instead, different among the buildings and evaluated by the limit state design according to Eurocode 2 (1993). More details regarding the design of all the buildings are reported in Ferrara (2012).

The uncoupled periods of vibration of the systems are grouped in triplets and are reported in Table 1. Each triplet contains the periods of the modes of vibration (translational in the *x*- and *y*-directions and rotational) characterised by the same number of changes in the sign of the modal components (Hejal and Chopra 1987). The (coupled) periods of vibration of the asymmetric buildings are in Table 2. Note that buildings belonging to the two groups ST and SR have equal elastic characteristics and only differentiate because of the reinforcements in beams and columns.

## 4. Numerical model

The numerical analyses are carried out by means of the OpenSees program (Mazzoni *et al.* 2007). The buildings are represented by means of a 3D centreline model with rigid diaphragms. Beams and columns are modelled by means of elements (*Beam With Hinges Elements*) which are elastic in the middle and inelastic at the ends within parts of finite length. In the ending parts the cross-section is discretized by means of fibres subjected to uniaxial stresses. The mechanical properties of the fibres are assigned so as to simulate the response of longitudinal reinforcement and concrete. To consider the effect of confinement on concrete, distinction is made between cross-section section core (confined concrete) and cover (unconfined concrete). The length of the member where the inelastic behaviour is expected to concentrate is assumed equal to the maximum dimension of the member cross-section. Given the high aspect ratio of the members, the shear contribution to the structural response is not considered. Torsion stiffness is neglected.

The response of the longitudinal steel bars is simulated by means of the Giuffrè-Menegotto-Pinto model (1970, 1973), as modified by Filippou (1983) and implemented in OpenSees as "Steel02". Referring to this model, note that the isotropic hardening is neglected in this study and that only the kinematic hardening is considered (the parameter $b$ responsible for the kinematic hardening is assumed equal to 0.003).

The uniaxial model adopted for concrete is implemented in OpenSees as "Concrete04". When fibres are subjected to negative axial deformations (shortening), the monotonic response of this model is equal to that proposed by Mander *et al.* (1988). Loading and unloading paths follow the rules proposed by Karsan and Jirsa (1969), i.e., they are linear but their slope decreases with the increase in the uniaxial deformation. The response under positive axial deformations (elongation) is described by an initially linear function and by a nonlinear softening branch with a degradation exponential function. The loading and unloading paths are linear with slope equal to the secant stiffness at the point where the unloading path takes place.

The axial (compressive and tensile) strengths considered in the numerical analyses are equal to the mean values of the axial strengths of the assumed materials. The yield strength of the longitudinal reinforcement $f_{ym}$ is equal to 400 MPa in buildings type GL (steel type FeB38k) and equal to 450 MPa in buildings type ST and SR (steel type FeB44k). The mean compressive strength of (unconfined) concrete $f_{cm}$ is equal to 28 MPa in all the examined buildings. The tensile strength $f_{ctm}$ is evaluated as a function of the compression strength $f_{cm}$ by means of the following relation

$$f_{\mathrm{ctm}} = 0.62 \cdot \sqrt{f_{\mathrm{cm}}} \tag{2}$$

where $f_{cm}$ and $f_{ctm}$ are in MPa. The ultimate axial deformation of the unconfined concrete $\varepsilon_{cu}$ is assumed equal to 0.004.

The compressive strength of confined concrete is evaluated, as proposed by Mander *et al.* (1988), by means of the relation

$$f_{\mathrm{cm}}^{\mathrm{c}} = f_{\mathrm{cm}}\, k \tag{3}$$

where the parameter $k$ depends on the mechanical percentage of the transverse reinforcement $\omega_{st}$

$$k = 2.254\sqrt{1+3.97\alpha\omega_{\mathrm{st}}} - \alpha\omega_{\mathrm{st}} - 1.254 \tag{4}$$

In the relation above, the effectiveness factor $\alpha$ is assumed equal to 0.80, in keeping with Priestley *et al.* (2008) who suggested using effectiveness factors in the range from 0.75 to 0.85. As proposed by Priestley *et al.* (2008), the increase in the ultimate deformation is evaluated by

$$\varepsilon_{\mathrm{cu}}^{\mathrm{c}} = \varepsilon_{\mathrm{cu}} + \frac{1.4 \cdot \omega_{\mathrm{st}}}{k} \cdot \varepsilon_{\mathrm{su}} \tag{5}$$

where the ultimate deformation of the hoops $\varepsilon_{su}$ is assumed equal to 0.075.

The values $f^{c}_{cm}$ and $\varepsilon^{c}_{cu}$ are calculated for all the beams and columns and their average values are considered for the whole structure. In particular, the compressive strength of the confined concrete is assumed equal to 32 MPa in buildings type GL and equal to 34 MPa in buildings type ST and SR. The ultimate deformation of confined concrete is assumed equal to 0.01 in all the buildings.

In regard to the modelling of the building, the writers note that the presence of rigid diaphragms may cause high axial forces in beams if cross-sections are modelled by means of fibres. To avoid this shortcoming "*Zero Length Section*" elements are added at one end of each beam. These zero length elements have very low axial rigidity and very high rigidity in flexure. $P$-$\Delta$ effects are not considered in all the analyses in that the examined buildings are moderate height (16 m). Further details are reported in Ferrara (2012).

## 5. Seismic analyses

The examined buildings are analysed by means of nonlinear dynamic analyses and nonlinear static analyses. All the seismic analyses are carried out starting from a structural configuration in which the structural members have already been deformed by the gravity loads of the seismic design situation.

To investigate the effectiveness of the analysed nonlinear static methods in the assessment of buildings with moderate or high inelastic response, different values of the peak ground acceleration $a_g$ are considered. In particular, peak ground accelerations equal to 0.25 g and 0.43 g are considered for the buildings type SR and ST. According to Eurocode 8, these values are representative of medium seismicity zone earthquakes with probabilities of exceedance of 10% and 2% in 50 years. A single value of the peak ground acceleration is taken into account for the buildings type GL. This value, equal to 0.20 $g$, is lower than those considered for buildings type SR and ST because higher values have proved to lead to numerical instability.

### *5.1 Dynamic analyses*

Seven ground motions are considered for the nonlinear dynamic analyses. The single ground motion consists of two accelerometric components along the $x$- and $y$-directions. The components of each ground motion are different from each other and compatible with the elastic response spectrum proposed in Eurocode 8 for soft soil (type C) and equivalent viscous damping ratio equal to 0.05. They have been generated by means of the SIMQKE program (1976). The single accelerogram is defined by a stationary random process modulated in amplitude by means of a compound intensity function consisting of three parts: the first part is described by a power function, the second by a constant function (*strong motion phase*), and the third by a function with exponential decay (Ferrara 2012). The total length of the accelerogram is equal to 20 s while that of the strong motion phase is equal to 7 s, i.e., slightly lower than the minimum value suggested in Eurocode 8. Details about the choice of this envelope intensity function and the procedure for the determination of the lengths of the parts of the compound function are described in Amara *et al.* (2013). As required in Eurocode 8, no value of the mean spectrum of the seven accelerometric signals is more than 10% below the corresponding value of the elastic response spectrum of the code; further, the average of the pseudo-accelerations of the constant acceleration range and the average related to the null period are not smaller than the corresponding values in the code spectrum.

The Rayleigh formulation is used to introduce damping. Mass and stiffness coefficients are defined so that two modes of vibration of the structures are characterised by an equivalent viscous damping ratio equal to 0.05. For planar systems, the considered modes of vibration are the first and second modes of vibration in the direction of the seismic input (Table 1). For asymmetric systems, the first mode of vibration is that corresponding to the maximum period of coupled modes of vibration within the first triplet of modes while the second is that corresponding to the maximum period of coupled modes of vibration within the second triplet (Table 2).

The equations of motion are integrated by means of the Newmark method with coefficients $\gamma$=0.5 and $\beta$=0.25. The step of integration of the time history is equal to 0.01 s. The algorithm used for the solution of the nonlinear equilibrium equations is the Krylov-Newton algorithm (Scott and Fenves 2010).

### *5.2 Pushover analyses*

The pushover analyses of the asymmetric systems are displacement-controlled. The lateral forces are applied along the *y*-direction and their intensity is increased until a target displacement is reached at the centre of mass of the top floor (this point is later called *reference point*). The target displacement of the asymmetric buildings is assumed equal to the mean value of the dynamic displacements caused by the seven artificial accelerometric signals at the reference point of the corresponding planar systems.

A particular load pattern is defined for the pushover analyses of the asymmetric systems in order to reduce the errors in the heightwise distribution of the response parameters corresponding to the achievement of the target displacement at the reference point of the building. To make these errors virtually equal for all the examined nonlinear static methods, the load pattern is considered equal for all the methods. As described later, the load pattern is derived from the analysis of the response of the planar systems corresponding to the asymmetric systems under examination and is defined so as to return, at all the floors of the planar systems, a *target displacement profile*. Two target displacement profiles are considered here because attention is focused on the prediction of absolute and interstorey displacements. The first profile is provided by lateral displacements equal to the mean values of the maximum dynamic displacements in the planar systems; the second is provided by lateral displacements that are equal to the sum of the mean values of the maximum dynamic interstorey displacements at the storeys below the storey under consideration.

The load pattern above is obtained by means of a preliminary pushover analysis of the planar system. In this analysis (later called pushover P-0) the structure is subjected to a lateral displacement pattern that is proportional to the target displacement profile. The lateral displacements of the structure are increased until the target displacement is reached at the reference point. At each step of the pushover analysis P-0, the storey shear force $V^{(0)}$ and the displacement $u^{(0)}$ at the reference point are recorded. Then, the lateral forces $F^{(0)}$ are derived as the difference between the storey shear forces corresponding to the storey under examination and the storey above, i.e.

$$F_{\mathrm{i,j}}^{(0)} = V_{\mathrm{i,j}}^{(0)} - V_{\mathrm{i+1,j}}^{(0)} \tag{1}$$

where *i* is the storey under examination and *j* is the step of the pushover analysis.

During the pushover analysis P-0 the magnitude and heightwise distribution of the lateral forces $F^{(0)}$ change with the increase in the displacement of the reference point of the building. At each step of the pushover analysis, the incremental lateral forces $\Delta F^{(0)}$ are obtained as the difference between the lateral forces calculated at the current step and those calculated at the previous step, i.e.

$$\Delta F_{\mathrm{i,j}}^{(0)} = F_{\mathrm{i,j}}^{(0)} - F_{\mathrm{i,j-1}}^{(0)} \tag{2}$$

The lateral forces $\Delta F^{(0)}$ are finally scaled so as to define load patterns $\Delta F$ characterised by a unit value of the base shear. Each of these patterns refers to a specified range of displacements of the reference point from $u_{\mathrm{j}-1}^{(0)}$ to $u_{\mathrm{j}}^{(0)}$.

The load patterns $\Delta F$ are able to reproduce the target displacement profile when they are applied to the planar systems. If asymmetric systems are considered, these load patterns do not return at $C_M$ the target displacement profile exactly. However, for the buildings examined in this paper the differences are negligible.

## 6. Corrective eccentricities

In this section the corrective eccentricities $e_1$ and $e_2$ of all the examined buildings are calculated as a function of the rigidity eccentricity $e_{\mathrm{r}}$, ratio of the uncoupled torsional to lateral frequencies $\Omega_\theta$, strength eccentricity $e_s$ and parameter $R_\mu$.

The rigidity eccentricity $e_r$ and the ratio of the uncoupled torsional to lateral frequencies $\Omega_\theta$ can be evaluated rigorously for single-storey systems and regularly asymmetric multi-storey systems (Hejal and Chopra 1987). However, some approximate methods are also available in the literature for non-regularly asymmetric systems (Bosco *et al.* 2013c, Calderoni *et al.* 2002, Doudomis and Athanatopoulou 2008, Georgoussis 2010, Makarios and Anastassiadis 1998a, b, Makarios 2008, Marino and Rossi 2004, Moghadam and Tso 2000). In all the buildings examined the location of the centres of rigidity $C_R$ is equal to that of the geometric centres of the deck. Therefore, the rigidity eccentricity is equal to the difference between the *x*-coordinates of the geometric centre and centre of mass $C_M$. Two values of $e_r$ are considered: -1.43 m (-0.05*L*) for the buildings characterised by low mass eccentricity (GL-L, ST-L, SR-L) and -4.28 m (-0.15*L*) for the buildings with high mass eccentricity (GL-H, ST-H, SR-H). The uncoupled torsional to lateral frequency ratios ($\Omega_{\theta x}$ and $\Omega_{\theta y}$) are calculated as the ratios of the rigidity radii of gyration $r_{ky}$ and $r_{kx}$, evaluated according to Makarios and Anastassiadis (1998a, b), to the mass radius of gyration $r_m$. The uncoupled torsional to lateral frequency ratios of the examined buildings are reported in Table 3. Note that the response in the *y*-direction is coupled and influenced by the ratio $\Omega_{\theta y}$. All the values of this parameter are close to unity, i.e., to the value that marks the watershed between torsionally flexible and torsionally rigid systems. This value is usual in buildings with RC framed structure.

The location of the centres of strength $C_S$ is identified by means of the pushover analysis of the planar system. At each step of this analysis, the location of the centre of strength of the first storey is assumed to be representative of the location of the centre of strength at all the other storeys. In particular, $C_S$ is identified as the location of the centroid of the base shear forces transmitted by the columns of the first storey. Furthermore, the position of $C_S$ changes with the increase of the roof displacement because of the gradual yielding of members. In this paper, $C_S$ and the corresponding strength eccentricity $e_s$ are calculated at the achievement of the target displacement of the reference point. Note that all the structures are pushed well into the plastic range of behaviour and that at this stage the centre of strength reaches a stationary position.

As previously reported, the parameter $R_\mu$ is the ratio of the required elastic base shear $V_{by,el}$ to the actual lateral strength $V_{by,u}$ of the structure. Both shear forces are calculated on the corresponding planar system. Specifically, the elastic base shear is obtained by modal response spectrum analysis and the actual lateral strength by pushover analysis at the achievement of a roof displacement equal to the target value. A numerical example of the calculation of $e_s$ and $R_\mu$ can be found in Bosco *et al.* (2013b).

The values of $e_s$ and $R_\mu$ are evaluated for each peak ground acceleration because this parameter influences the target displacement and elastic strength demand. In total, ten cases are considered. The obtained values of $R_\mu$ and $e_s$ are reported in Table 4 along with the base shear forces $V_{by,el}$ and $V_{by,ul}$. The index in the label of buildings type ST and SR (i.e., 1 or 2) is representative of the level of the seismic intensity considered ($a_{g1}$=0.25 $g$ or $a_{g2}$=0.43 $g$). Still in Table 4, two values of $e_s$ and $R_\mu$ are evaluated for each case, one for the prediction of the absolute displacements and the other for the interstorey displacements (by means of the two corresponding load patterns described in Section 5.2). The strength eccentricity $e_s$ is equal to the rigidity eccentricity in the case of buildings type GL and ST. In these systems, in fact, the distribution of the lateral strength is symmetric with respect to the $y$-axis. In buildings type SR, instead, the design procedure (based on a 3D model) leads to an asymmetric distribution of the lateral strength. Owing to this, $C_S$ is between $C_R$ and $C_M$ and the magnitude of the strength eccentricity is lower than $e_r$.

Table 5 reports the corrective eccentricities for the prediction of absolute and interstorey displacements. As is evident, the corrective eccentricities to be used for the prediction of either absolute or interstorey displacements are virtually equal. Owing to this, the same corrective eccentricities could be used for the prediction of absolute and interstorey displacements.

Table 3 Uncoupled torsional to lateral frequency rations $\Omega_{\theta x}$ and $\Omega_{\theta y}$ of the examined buildings

| Building type | $r_m$ [m] | $r_{kx}$ [m] | $r_{ky}$ [m] | $\Omega_{\theta y}$ | $\Omega_{\theta x}$ |
|---|---|---|---|---|---|
| GL | 10.051 | 15.510 | 9.913 | 1.014 | 1.565 |
| ST and SR | 11.054 | 10.684 | 9.871 | 1.120 | 1.082 |

Table 4 Parameter $R_\mu$ and strength eccentricity $e_s$

| | | absolute displacements | | | | interstorey displacements | | | |
|---|---|---|---|---|---|---|---|---|---|
| Building | $V_{el,y}$ [kN] | $V_{b,y}$ [kN] | $R_{\mu,y}$ | $e_s$/L | $e_s$ [m] | $V_{b,y}$ [kN] | $R_{\mu,y}$ | $e_s$/L | $e_s$ [m] |
| GL-L | 8435.4 | 3514.8 | 2.400 | -0.0500 | -1.425 | 3489.9 | 2.417 | -0.0500 | -1.425 |
| GL-H | 8435.4 | 3514.8 | 2.400 | -0.1500 | -4.275 | 3489.9 | 2.417 | -0.1500 | -4.275 |
| ST-L1 | 12750.5 | 4846.7 | 2.631 | -0.0500 | -1.425 | 4835.4 | 2.637 | -0.0500 | -1.425 |
| ST-H1 | 12750.5 | 4846.7 | 2.631 | -0.1500 | -4.275 | 4835.4 | 2.637 | -0.1500 | -4.275 |
| ST-L2 | 21803.4 | 5237.3 | 4.163 | -0.0500 | -1.425 | 5172.8 | 4.215 | -0.0500 | -1.425 |
| ST-H2 | 21803.4 | 5237.3 | 4.163 | -0.1500 | -4.275 | 5172.8 | 4.215 | -0.1500 | -4.275 |
| SR-L1 | 12750.5 | 4821.9 | 2.644 | -0.0283 | -0.807 | 4813.0 | 2.649 | -0.0283 | -0.807 |
| SR-H1 | 12750.5 | 4973.2 | 2.564 | -0.0864 | -2.462 | 4963.8 | 2.579 | -0.0859 | -2.448 |
| SR-L2 | 21803.4 | 5228.7 | 4.324 | -0.0281 | -0.801 | 5156.5 | 4.384 | -0.0274 | -0.781 |
| SR-H2 | 21803.4 | 5384.0 | 4.050 | -0.0855 | -2.437 | 5353.0 | 4.074 | -0.0854 | -2.434 |

Table 5 Corrective eccentricities

| | absolute displacements | | | | interstorey displacements | | | |
|---|---|---|---|---|---|---|---|---|
| Building | $e_1$/L | $e_2$/L | $e_1$ [m] | $e_2$ [m] | $e_1$/L | $e_2$/L | $e_1$ [m] | $e_2$ [m] |
| GL-L | -0.0530 | -0.0241 | -1.511 | -0.687 | -0.0530 | -0.0240 | -1.511 | -0.684 |
| GL-H | -0.1591 | -0.0722 | -4.534 | -2.058 | -0.1589 | -0.0721 | -4.529 | -2.055 |
| ST-L1 | -0.0432 | -0.0159 | -1.231 | -0.453 | -0.0432 | -0.0159 | -1.231 | -0.453 |
| ST-H1 | -0.1295 | -0.0478 | -3.691 | -1.362 | -0.1295 | -0.0478 | -3.691 | -1.362 |
| ST-L2 | -0.0343 | 0.0081 | -0.978 | 0.231 | -0.0339 | 0.0092 | -0.966 | 0.262 |
| ST-H2 | -0.1029 | 0.0244 | -2.933 | 0.695 | -0.1016 | 0.0277 | -2.896 | 0.789 |
| SR-L1 | -0.0262 | 0.0046 | -0.747 | 0.131 | -0.0262 | 0.0046 | -0.747 | 0.131 |
| SR-H1 | -0.0800 | 0.0127 | -2.280 | 0.362 | -0.0799 | 0.0127 | -2.277 | 0.362 |
| SR-L2 | -0.0192 | 0.0261 | -0.547 | 0.744 | -0.0185 | 0.0274 | -0.527 | 0.781 |
| SR-H2 | -0.0628 | 0.0649 | -1.790 | 1.850 | -0.0624 | 0.0660 | -1.778 | 1.881 |

## 7. Results of numerical analyses

In this section a comparison is carried out between the seismic response obtained by means of the described nonlinear static methods and that obtained by nonlinear dynamic analysis. The comparison is carried out in terms of in-plan and in-elevation distributions of absolute and interstorey displacements. To provide a measure of capability of the nonlinear static methods to predict the results of the nonlinear dynamic analysis, the percentage error committed in the estimate of the dynamic displacement is calculated as

$$\mathrm{Err}\ (\%) = \frac{u^{\mathrm{st}}\left(\mathrm{P}\right) - u^{\mathrm{dyn}}\left(\mathrm{P}\right)}{u^{\mathrm{dyn}}\left(\mathrm{P}\right)} \tag{4}$$

In this relation the character P represents the generic point of the floor, $u^{dyn}$(P) is the mean value of the maximum displacements obtained by nonlinear dynamic analysis and $u^{st}$(P) is the displacement obtained by means of the nonlinear static method of analysis. Note that positive values of the error indicate overestimates of the dynamic response while negative values indicate underestimates of the dynamic response. Errors are evaluated at each storey, on both flexible and rigid sides of the building.

Note that the sole results relative to buildings with high rigidity eccentricity ($e_r$=−0.15$L$) are discussed in detail later. A synthetic comparison is reported, instead, for all the other cases. On commenting on results, the five buildings with high eccentricity are subdivided into two groups: buildings with moderate plastic response (i.e., with parameter $R_\mu$ approximately equal to 2.5) and buildings with high inelastic response (i.e., with parameter $R_\mu$ nearly equal to 4.0).

### *7.1 Buildings with moderate inelastic response*

Buildings belonging to this group are those which have been previously labelled as type GL and those labelled as type ST and SR subjected to ground motions with peak ground acceleration equal to 0.25 g (i.e., $a_g$=$a_{g1}$). As an example, the in-plan absolute displacements of the fifth and

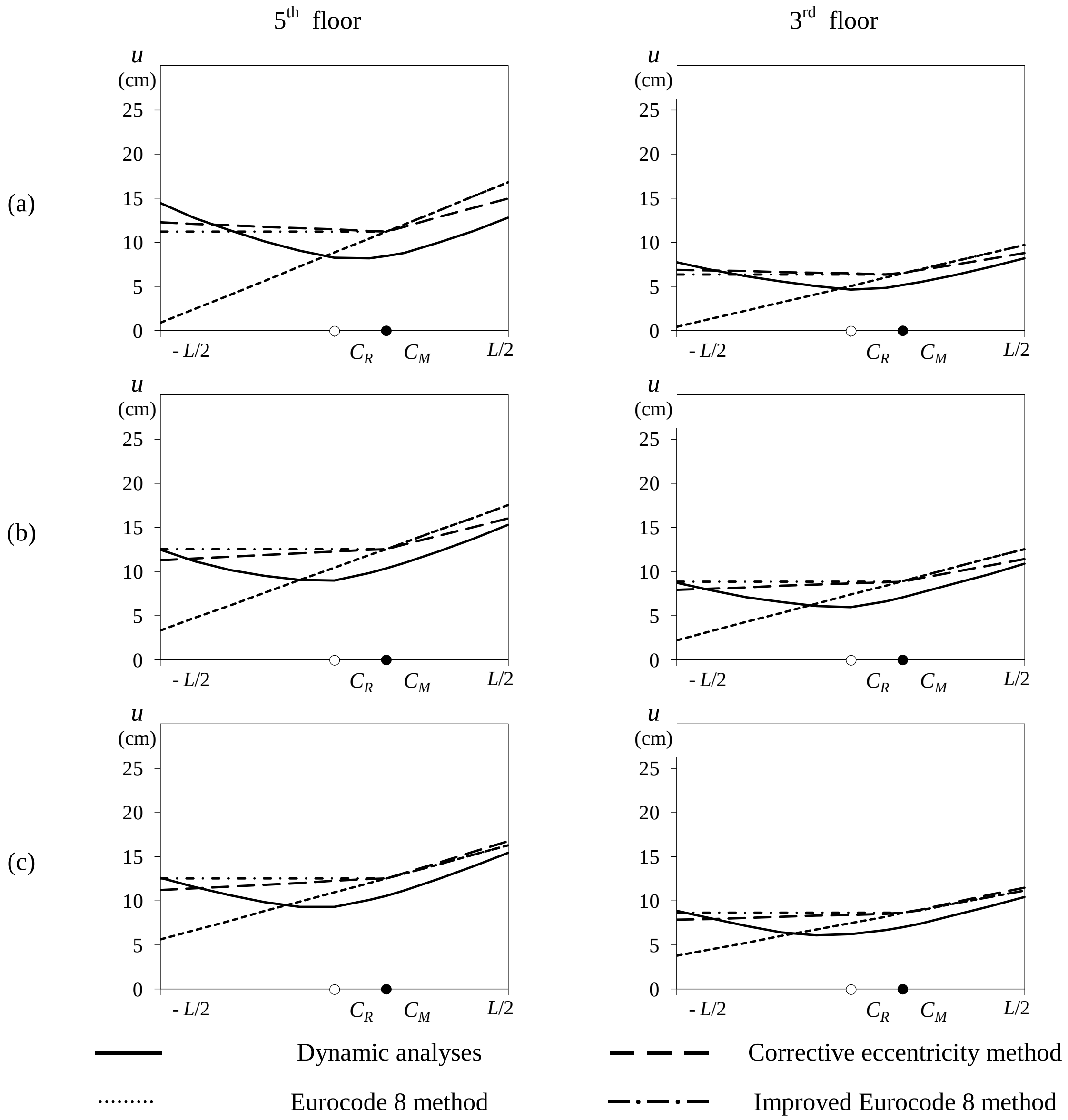


Fig. 2 Absolute displacements of buildings (a) GL-H; (b) ST-H1; (c) SR-H1

third floors of the structures GL-H, ST-H1 and SR-H1 are plotted in Fig. 1. In particular, the displacements of the fifth floor are shown on the left side of the figure while those of the third floor are shown on the right. The figure illustrates the results obtained by nonlinear dynamic analysis (continuous line), Eurocode 8 method of analysis (dotted line), corrective eccentricity method (dashed line) and improved Eurocode 8 method (dashed dotted line).

The building GL-H, characterised by a value of the parameter $\Omega_\theta$ close to unity ($\Omega_\theta$=1.014), shows dynamic displacements higher than those of the corresponding planar system. As usual in single-storey systems with similar properties, this occurs on both flexible and rigid sides of the building. Buildings ST-H1 and SR-H1, characterised by a value of $\Omega_\theta$ higher than unity ($\Omega_\theta$=1.120), exhibit instead different trends. In fact, whereas the displacements on the flexible side of the asymmetric building are higher than those of the planar system, the displacements of the rigid side are virtually equal to those of the planar system.

The results of each nonlinear static method are similar at all the storeys of the examined buildings (see left and right columns of Fig. 2). Therefore, only those at the top floor are discussed here.

The method suggested in Eurocode 8 leads to estimates of the dynamic displacements which are significantly unconservative on the rigid side of the building: the percentage errors at the fifth floor are approximately equal to -95% for the building GL-H, -75% for the building ST-H1 and -55% for the building SR-H1. Conversely, on the flexible side of the building the prediction is fairly satisfactory: the building GL-H is characterised by percentage errors equal to 31%; the building ST-H1 shows percentage errors equal to 15%; the building SR-H1 shows percentage errors equal to 7%.

The in-plan displacements returned by the corrective eccentricity method is the envelope of two linear diagrams, each one relative to a single pushover analysis. The prediction of the displacements is conservative on the flexible side of the building, similarly to that returned by the method proposed in Eurocode 8. However, the response anticipated by means of the corrective eccentricity method is less conservative than that obtained by the method proposed in Eurocode 8. The major benefit of the corrective eccentricity method can be inferred from the prediction of the seismic response on the rigid side of the building. The underestimates of the dynamic displacements are significantly reduced with respect to those produced by the method proposed in Eurocode 8. The underestimates of the corrective eccentricity method are equal to -15% for the building GL-H, -9% in the building ST-H1 and approximately equal to -11% in the building SR-H1.

The in-plan displacements provided by the improved Eurocode 8 method are still obtained by the envelope of two linear diagrams. In particular, on the flexible side of the building, the displacements are equal to those provided by the method reported in Eurocode 8 and thus the same considerations previously reported for this latter method hold. On the rigid side of the floor, the diagram of the lateral displacements is horizontal, i.e., equal to that of the corresponding planar system. On this side, the lateral displacement of the building GL-H is slightly underestimated with error equal to -22% (this error is much lower than that produced in the same building by the Eurocode 8 method). The errors in buildings ST-H1 and SR-H1 are virtually null.

Fig. 3 illustrates the distribution in elevation of the interstorey displacements of the buildings GL-H, ST-H1 and SR-H1. In particular, the left side plots refer to the rigid side of the buildings whereas the plots on the right refer to the flexible side. As is evident, the method suggested by Eurocode 8 provides a satisfactory estimate of the interstorey displacements only on the flexible side. Instead, both the methods based on two pushover analyses significantly improve the prediction on the rigid side and retain the effectiveness of the method suggested in Eurocode 8 on the flexible side.

### *7.2 Buildings with high inelastic response*

The buildings considered in this section are those labelled ST or SR and subjected to ground motions characterised by peak ground acceleration equal to 0.43 g (i.e., $a_g$=$a_{g2}$). As an example, Fig. 4 shows the in-plan distribution of the absolute displacements obtained at the fifth and third

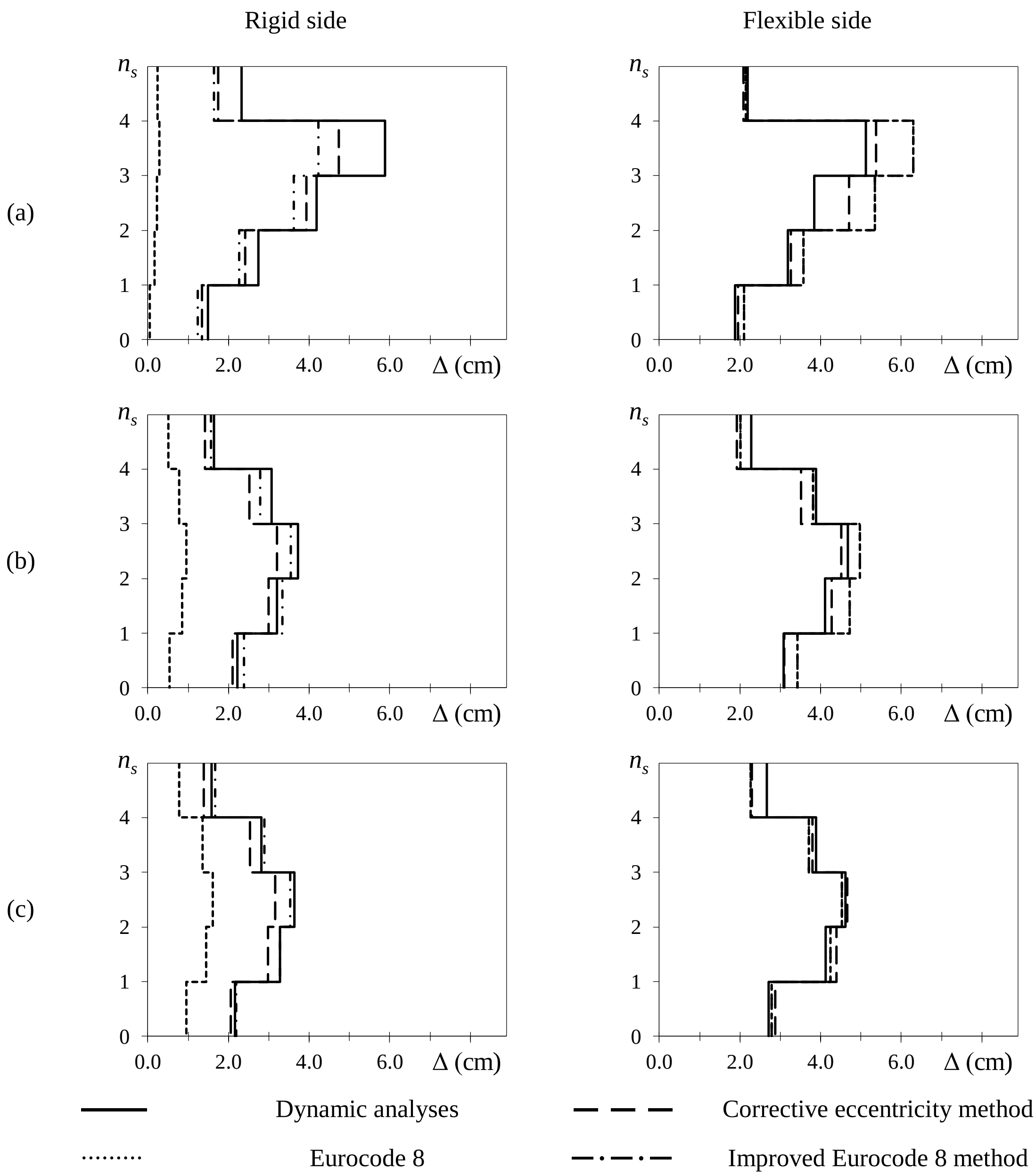


Fig. 3 Interstorey displacements of buildings (a) GL-H; (b) ST-H1; (c) SR-H1

floors (left and right sides of the figure, respectively) for buildings ST-H2 and SR-H2. Again, a comparison is carried out between nonlinear dynamic analysis, Eurocode 8 method, corrective eccentricity method and improved Eurocode 8 method. As evident, the in-plan distribution of the dynamic displacements is significantly nonlinear.

A comparison with the results of buildings ST-H1 and SR-H1 (characterised by the same elastic parameters $e_r$ and $\Omega_\theta$, virtually equal values of $e_s$ but much lower values of $R_\mu$) highlights

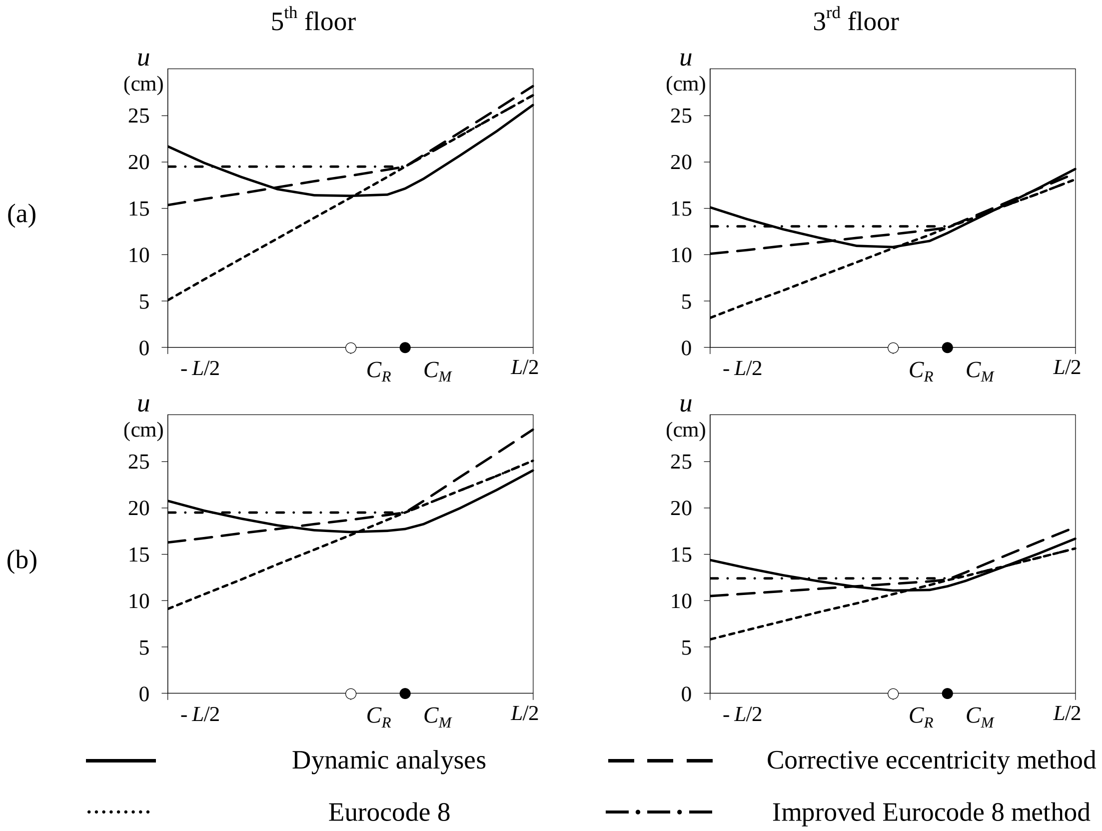


Fig. 4 Absolute displacements of buildings (a) ST-H2; (b) SR-H2

that the response of buildings ST-H2 and SR-H2 is not less rotational than that of buildings ST-H1 and SR-H1 (Fig. 2). This is in contrast to general findings on single-storey systems. In fact, in the light of past studies on single-storey systems, a flat trend of the in-plan displacements would be expected for buildings with pronounced plastic response. Further, unlike the response of buildings ST-H1 and SR-H1, the response of buildings ST-H2 and SR-H2 highlights that also the displacements on the rigid side of these buildings are higher than those of the corresponding planar systems.

Referring to the prediction of the dynamic response by means of the Eurocode 8 method, the same considerations made in regard to buildings with moderate plastic response hold. The Eurocode 8 method leads to noteworthy predictions of the dynamic displacements on the flexible side of the buildings (percentage errors equal to 4% at the fifth floor) and to unacceptable estimates on the rigid side (percentage errors equal to -80% for building ST-H2 and -60% for building SR-H2).

On the flexible side, the corrective eccentricity method is a little more conservative, but substantially tantamount to the Eurocode 8 method. On the rigid side, the corrective eccentricity leads to significant reductions of the unconservative errors produced by the Eurocode 8 method, as already observed for buildings subjected to moderate plastic response. The magnitude of these errors is, however, still not negligible and higher than that observed for buildings ST-H1 and SR-

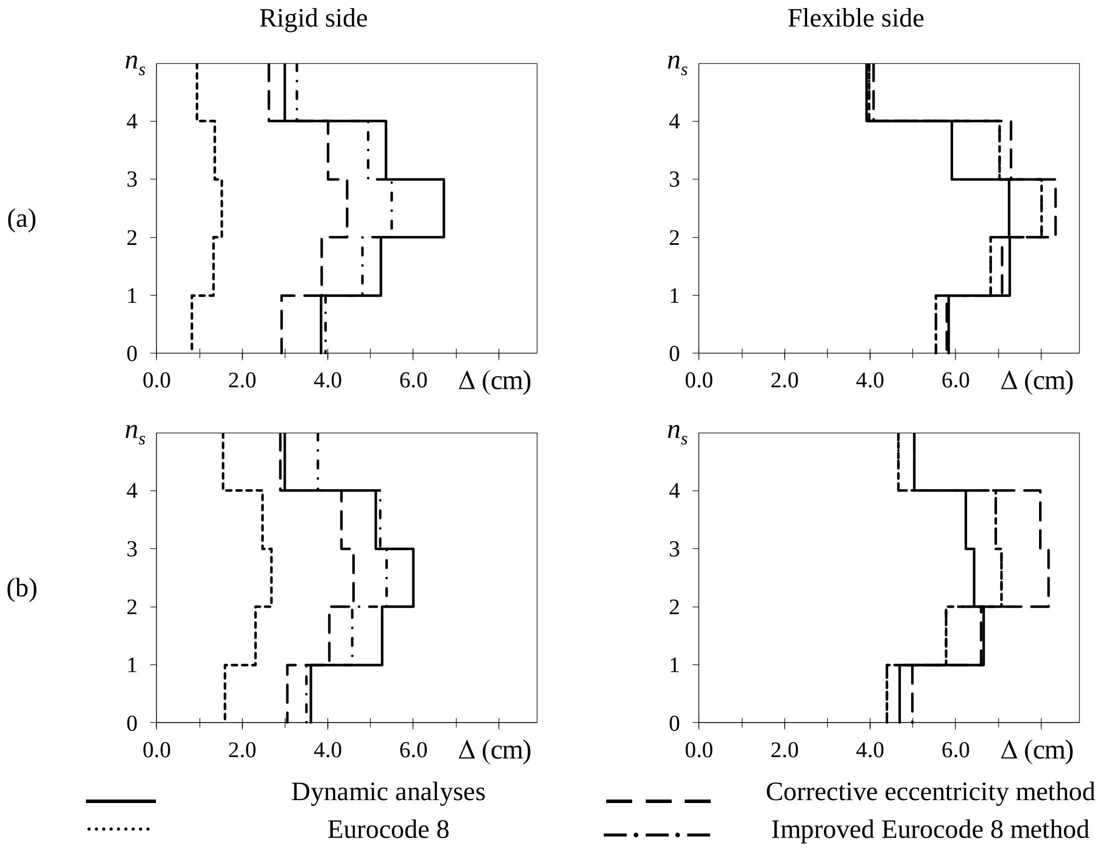


Fig. 5 Interstorey displacements of buildings (a) ST-H2; (b) SR-H2

H1 (percentage errors equal to -30% for building ST-H2 and about -22% for building SR-H2).

As reported above, on the flexible side of the building the improved Eurocode 8 method of analysis leads to results that are equal to those of the Eurocode 8 method. On the rigid side, it provides the best estimates and the lowest unconservative errors (percentage errors close to -10% for both the buildings).

These trends are substantially confirmed at all the storeys of the buildings. Similar observations are valid with reference to the interstorey displacements, which are shown in Fig. 5 for buildings ST-H2 and SR-H2. Again, the interstorey displacements of the rigid side are plotted on the left side of the figure whereas those of the flexible side are on the right side of the figure.

### *7.3 General considerations on all the buildings examined*

Comments on the prediction of the seismic response of all the examined buildings are made based on some synthetic plots. In particular, the mean of the percentage errors obtained at the different storeys is determined and plotted in Fig. 6 for the ten analysed buildings. Here, the mean errors of both the absolute and interstorey displacements are illustrated. The mean errors on the rigid and flexible sides of the building are reported on the left and right sides of the figure.

The figure shows that similar results are obtained on either the flexible or rigid side in terms of

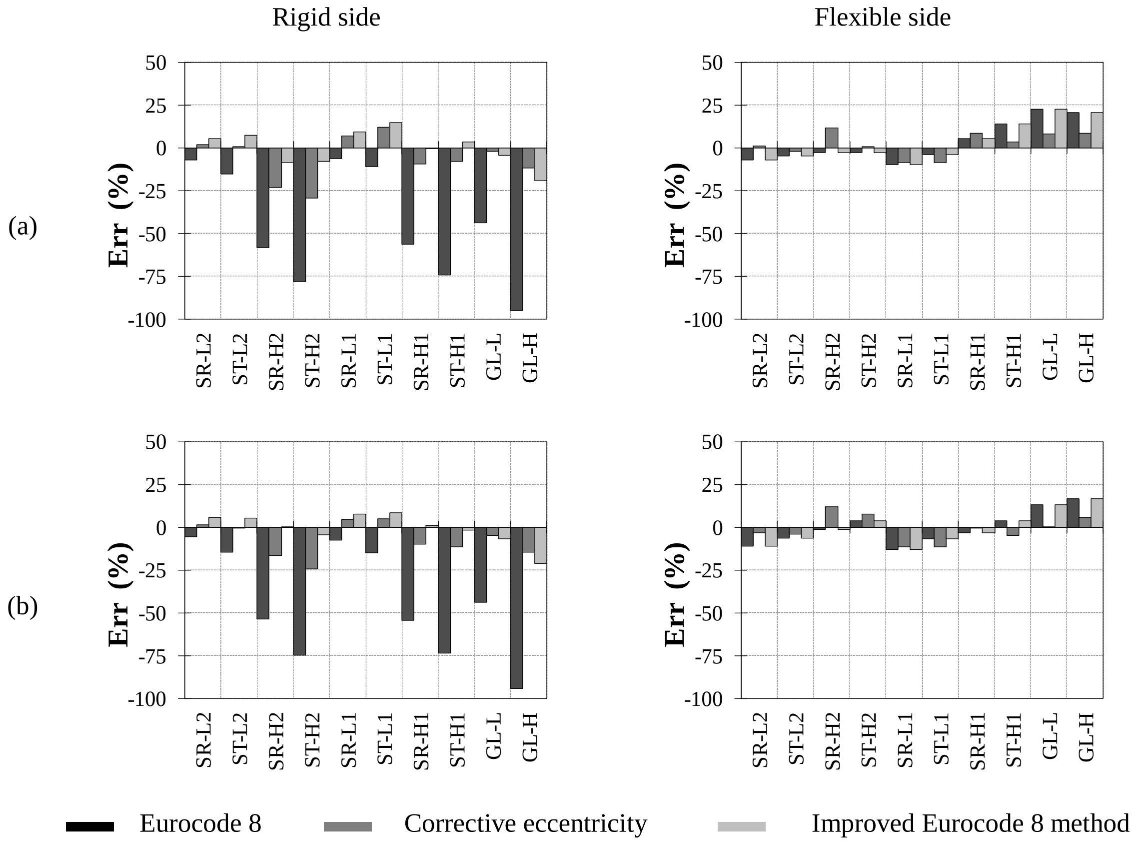


Fig. 6 Mean values of the percentage errors in the prediction of the absolute (a) and intestorey (b) displacements on the flexible and rigid sides of the examined buildings

absolute and interstorey displacements. Further, similar trends may be noted in buildings with low rigidity eccentricities (GL-L, ST-L and SR-L) and high rigidity eccentricities (GL-H, ST-H and SR-H). However, in the former cases, the less rotational character of the response causes more reduced percentage errors in the estimate of the dynamic displacements.

Referring to the rigid side, the Eurocode 8 method always provides an unconservative prediction, especially for buildings with high rigidity eccentricity. Of all the buildings with low rigidity eccentricity, the seismic resistant buildings (SR-L and ST-L) are characterised by acceptable errors while that designed to sustain gravity loads only (building GL-L) is subject to significant errors (-43%).

The use of the nonlinear static methods based on two pushover analyses improves significantly the prediction of the response. The corrective eccentricity method leads to mean unconservative errors lower than -15% with the exception of buildings SR-H2 and ST-H2, where the error is close to -25%. The maximum value of the mean unconservative error is always lower than -10% in the case of the improved Eurocode 8 method with the exception of building GL-H, where the error is close to -20%.

On the flexible side, the nonlinear static methods considered are either conservative or unconservative, but the percentage errors are always low.

## 8. Conclusions

This paper evaluates the effectiveness of three nonlinear static methods for the prediction of the dynamic response of in-plan irregular buildings. The methods considered are the method suggested in Eurocode 8, a method previously proposed by some of the authors and based on two pushover analyses with corrective eccentricities and a new method, referred to as the improved Eurocode 8 method, entailing two pushover analyses but no calculation for the corrective eccentricities. The effectiveness is assessed on a set of reinforced concrete framed buildings in terms of absolute and interstorey displacements. The buildings are designed by means of different code regulations and schematized by means of refined numerical models.

The numerical investigation leads to the following conclusions:

• The nonlinear static method suggested in Eurocode 8 provides a noteworthy estimate of the displacements on the flexible side of the building but unacceptable, unconservative errors on the rigid side.

• The corrective eccentricity method leads to a better prediction of the dynamic response. The estimates on the rigid side of the building are much better than those provided by the method suggested in Eurocode 8. However, the unconservative errors are still not negligible for buildings with high eccentricity and high inelastic response.

• The improved Eurocode 8 method proves to be particularly useful for reinforced concrete framed buildings (these buildings are usually characterised by an uncoupled torsional to lateral frequency ratio close to unity). The displacements on the rigid side of the examined buildings are well approximated by the response of the corresponding planar systems. The prediction is also noticeable on the flexible side. The method does not require any calculation of the corrective eccentricities and is suggested by the writers for the assessment of the seismic response of reinforced concrete framed buildings.

• The improved Eurocode 8 method provides reasonable results with few exceptions; for instance, it is slightly less accurate than the corrective eccentricity method for the building GL-H. However, the improved Eurocode 8 method is expected to fail in the prediction of the seismic response of highly torsionally flexible or rigid systems. This is because the response on the rigid side of these systems is usually characterised by a significant amplification or reduction of the lateral displacements with respect to those of the corresponding planar system. In these cases the use of the corrective eccentricity method is suggested.

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