Predicting displacement demand of multi-storey asymmetric buildings by nonlinear static analysis and corrective eccentricities

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SUMMARY:
Nonlinear static methods are not always effective in the assessment of multi-storey asymmetric structures because of the errors committed in the evaluation of the deck rotation. To overcome this shortcoming, some of the Authors have recently proposed the use of two nonlinear static analyses, characterised by lateral forces applied to different points of the deck. For each of the two analyses, the distance between the point of application of the lateral force and the centre of mass of the deck (corrective eccentricity) is given by relations resulting from a parametric study on a large set of single-story systems subjected to bidirectional ground motions. The corrective eccentricities depend on four parameters: the rigidity eccentricity e_r, the strength eccentricity e_s, the ratio Ω_θ of the torsional to lateral frequencies of the corresponding torsionally balanced system, and the ratio R_μ of the elastic strength demand to the actual strength of the system. In this paper, the effectiveness of the proposed method is evaluated on a set of r.c. framed multi-storey structures characterized by different values of e_r, e_s, Ω_θ and R_μ. The seismic response of these structures is evaluated by nonlinear time-history analyses and compared with that resulting from the application of the proposed nonlinear static method. To highlight the accuracy of the proposed method, a comparison is also made with the response obtained from the standard application of the nonlinear static method, i.e. without corrective eccentricities.

Keywords: Nonlinear static method, multi-storey asymmetric structures, corrective eccentricities

1. INTRODUCTION

The seismic assessment of structures requires the comparison between the displacement capacity, i.e. displacements and plastic deformations that the structure can undergo before the achievement of a given limit state, and the displacement demand, i.e. the displacements and plastic deformations caused by earthquakes. The most reliable tool for the evaluation of the displacement demand is the nonlinear time-history analysis. However, the difficulty in properly modelling the characteristics of the cyclic nonlinear behaviour of members and correctly simulating the seismic excitation makes this type of analysis accessible only to few experts. The need for a simple tool that explicitly considers the plastic deformations undergone by structural elements has led researchers to develop the so-called "nonlinear static methods" (Fajfar and Gašperšič, 1996; Freeman, 1998; Fajfar, 1999; Chopra, 2004). These methods are allowed by most seismic codes currently in force in the world, i.e. the Eurocode 8 (EC8, 2004), and provide results which are generally well approximated for planar frames, even if the differences with respect to the actual dynamic response are not always negligible (Fajfar and Gašperšič 1996; Bosco *et al.*, 2009). As remarked by many researchers (Fajfar *et al.*, 2005; Marusic and Fajfar, 2005; Lucchini *et al.*, 2009; De Stefano and Pintucchi, 2010; Kreslin and Fajfar, 2012), these methods are, however, less effective for the analysis of three-dimensional structures because of the errors committed in the evaluation of the deck rotation.

To improve the accuracy of the nonlinear static method, the Authors have recently proposed a new approach based on a double application of the nonlinear static analysis (Bosco *et al.*, 2012). In particular, for each direction considered of the seismic action, the nonlinear static analysis is applied

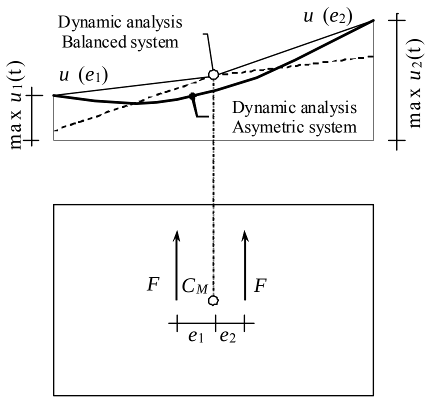


Figure 1. Maximum dynamic displacements vs. displacements determined by the nonlinear static method with corrective eccentricities

with reference to two different points of the deck (instead of one as suggested by seismic codes). This expedient is necessary because, owing to the simultaneous translation and rotation of decks, the maximum dynamic displacements of the vertical resisting members of the system are achieved at different times and thus the distribution of these displacements is nonlinear along the length of the deck. This consideration also explains that the in-plan distribution of the maximum dynamic displacements of the vertical resisting members of the system cannot be adequately approximated by a single nonlinear static analysis because the displacements resulting from this analysis are linearly variable along the length of the deck (if the deck is assumed to be rigid in its own plane). The points of application of the force have been properly defined in a previous study (Bosco *et al.*, 2012) through simple mathematical relations based on the investigation of the seismic response of a large set of asymmetric single-storey structural systems subjected to bidirectional ground motions. The distances e_1 and e_2 between the points of application of the forces and the centre of mass C_M are named "corrective eccentricities". As reported in (Bosco *et al.*, 2012) the use of the proposed method provides a suitable estimate of the maximum dynamic displacements of the two sides of the deck (Fig. 1). An exemplary application of the method to a multi-storey building is also shown in (Bosco *et al.*, 2012).

In this paper, the effectiveness of the relations for the determination of the corrective eccentricities is validated on reinforced concrete multi-storey framed structures. The analysed set of structures comprises seismic-resistant structures and structures designed to sustain gravity loads only. The maximum displacements demanded by the seismic events are first determined by nonlinear time-history analysis. These results are then compared to those obtained by the proposed method. Finally, to emphasise the benefit obtained from the use of the corrective eccentricities, a comparison is also made with the response obtained from the standard application of the nonlinear static method, i.e. without corrective eccentricities.

2. CORRECTIVE ECCENTRICITIES

The corrective eccentricities e_1 and e_2 are calculated by simple relations as a function of the parameters which mostly influence the lateral-torsional coupling of the seismic response of asymmetric buildings. These parameters are the rigidity eccentricity e_r (distance between the centre of rigidity C_R and C_M), the strength eccentricity e_s (distance between the centre of strength C_S and C_M), the ratio Ω_θ of the torsional to lateral frequencies of the corresponding torsionally balanced system (obtained by shifting C_M into C_R) and the ratio R_μ of the elastic shear force to the actual strength of the corresponding torsionally balanced system. The relations for the determination of e_1 and e_2 were obtained in a previous study (Bosco *et al.*, 2012) on the basis of a numerical investigation of asymmetric single-storey systems subjected to bidirectional ground motions.

Specifically, the corrective eccentricities e_1 and e_2 are obtained by the following relations

$$e_i = a_i\, e_s + b_i\, e_r \qquad i = 1 \text{ or } 2 \tag{2.1}$$

where the coefficients a_i and b_i depend on the ratios Ω_θ and R_μ. Note that e_s and e_r are negative or positive when C_S and C_R are on the left or right hand of C_M, respectively.

$$a_1 = \begin{cases} c_2^1\, R_\mu^2 + c_1^1\, R_\mu + c_0^1 & R_\mu \le 2 \\ c_2^2\, R_\mu^2 + c_1^2\, R_\mu + c_0^2 & R_\mu > 2 \end{cases} \tag{2.2a}$$

$$c_2^1 = -0.25\, c_1^1 \qquad\qquad c_2^2 = -0.25 c_1^2$$

$$c_1^1 = \begin{cases} -0.752\Omega_\theta + 1.373 & \Omega_\theta < 0.85 \\ 1.556\Omega_\theta - 0.589 & 0.85 \le \Omega_\theta \le 1.15 \\ -2.234\Omega_\theta + 3.770 & \Omega_\theta > 1.15 \end{cases} \qquad c_1^2 = 0.273\,\Omega_\theta - 0.182$$

$$c_0^1 = \begin{cases} 1.199\,\Omega_\theta - 0.834 & \Omega_\theta < 0.85 \\ -2.020\,\Omega_\theta + 1.902 & 0.85 \le \Omega_\theta \le 1.15 \\ 0.508\,\Omega_\theta - 1.004 & \Omega_\theta > 1.15 \end{cases} \qquad c_0^2 = c_0^1 + c_1^1 - c_1^2 \tag{2.2b}$$

$$b_1 = \begin{cases} \alpha\, R_\mu^\beta & R_\mu \le 2 \\ m\, R_\mu + t & R_\mu > 2 \end{cases} \tag{2.3a}$$

$$\alpha = \begin{cases} 0.756 & \Omega_\theta < 0.90 \\ -2.521\,\Omega_\theta + 3.025 & 0.90 \le \Omega_\theta \le 1.20 \\ 0 & \Omega_\theta > 1.20 \end{cases} \qquad \beta = -0.881\,\Omega_\theta - 0.015$$

$$m = \begin{cases} -0.085\,\Omega_\theta + 0.010 & \Omega_\theta < 1.00 \\ 0.373\,\Omega_\theta - 0.447 & 1.00 \le \Omega_\theta \le 1.20 \\ 0 & \Omega_\theta > 1.20 \end{cases} \qquad t = 2^\beta \alpha - 2\, m \tag{2.3b}$$

$$a_2 = \begin{cases} c_2^1\, R_\mu^2 + c_1^1\, R_\mu + c_0^1 & R_\mu \le R_{\mu V} \\ c_2^2\, R_\mu^2 + c_1^2\, R_\mu + c_0^2 & R_{\mu V} < R_\mu \le 5 \\ m\, R_\mu + t & R_\mu > 5 \end{cases} \tag{2.4a}$$

$$R_{\mu V} = -0.6\,\Omega_\theta + 2.86$$

$$c_2^1 = -0.50\,\frac{c_1^1}{R_{\mu V}} \qquad\qquad c_2^2 = -0.5\,\frac{c_1^2}{R_{\mu V}}$$

$$c_1^1 = \begin{cases} 0.946\,\Omega_\theta + 0.314 & \Omega_\theta \le 0.95 \\ 1.213 & \Omega_\theta > 0.95 \end{cases} \qquad c_1^2 = 0.606\,\Omega_\theta - 0.396$$

$$c_0^1 = \begin{cases} -0.171 & \Omega_\theta < 0.75 \\ -0.769\,\Omega_\theta + 0.406 & 0.75 \le \Omega_\theta \le 0.95 \\ -0.255\,\Omega_\theta - 0.083 & \Omega_\theta > 0.95 \end{cases} \qquad c_0^2 = c_0^1 + \frac{R_{\mu V}}{2}\left(c_1^1 - c_1^2\right) \tag{2.4b}$$

$$m = \begin{cases} 0.074 & \Omega_\theta \le 1.05 \\ -0.720\,\Omega_\theta + 0.831 & \Omega_\theta > 1.05 \end{cases} \qquad t = 25\, c_2^2 + 5\, c_1^2 + c_0^2 - 5\, m$$

$$b_2 = \begin{cases} c_2^1\, R_\mu^2 + c_1^1\, R_\mu + c_0^1 & R_\mu \le 3 \\ c_2^2\, R_\mu^2 + c_1^2\, R_\mu + c_0^2 & R_\mu > 3 \end{cases} \tag{2.5a}$$

$$c_2^1 = -\frac{c_1^1}{6} \qquad\qquad c_2^2 = -\frac{c_1^2}{12}$$

$$c_1^1 = \begin{cases} 9.91\,\Omega_\theta^2 - 14.6\,\Omega_\theta + 4.46 & \Omega_\theta \le 1.00 \\ -10.4\,\Omega_\theta^2 + 25.7\,\Omega_\theta - 15.5 & \Omega_\theta > 1.00 \end{cases} \qquad c_1^2 = \begin{cases} -2.59\,\Omega_\theta^2 + 3.28\,\Omega_\theta - 1.16 & \Omega_\theta \le 1.00 \\ 2.12\,\Omega_\theta^2 - 4.90\,\Omega_\theta + 2.31 & \Omega_\theta > 1.00 \end{cases} \tag{2.5b}$$

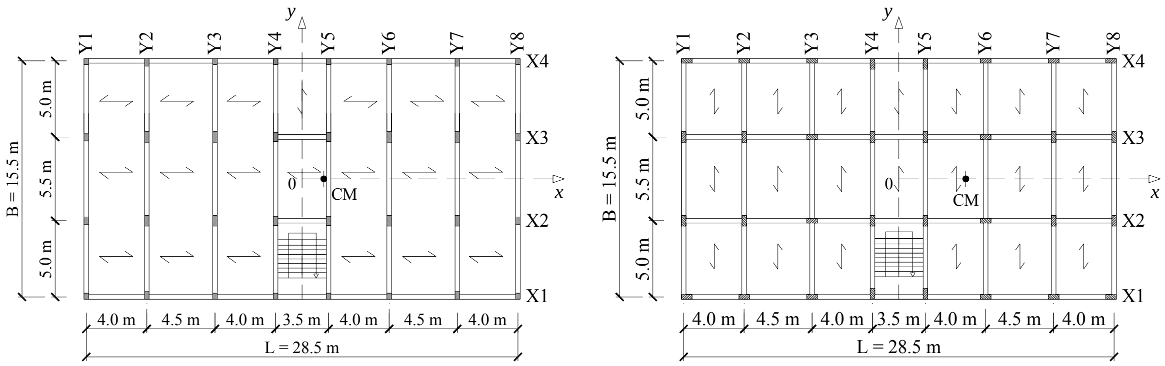
$$c_0^1 = \begin{cases} -15.6\,\Omega_\theta^2 + 22.2\,\Omega_\theta - 6.74 & \Omega_\theta \le 1.00 \\ 16.3\,\Omega_\theta^2 - 40.6\,\Omega_\theta + 24.1 & \Omega_\theta > 1.00 \end{cases} \qquad c_0^2 = c_0^1 + 3\left(\frac{c_1^1}{2} - \frac{3}{4}c_1^2\right)$$

3. ANALYSED STRUCTURES

The effectiveness of the proposed procedure is tested on six five-storey buildings with r.c. framed structure. The buildings have the same global dimensions but are designed differently to represent constructions built in different areas and periods of the last decades. In fact, out of all the buildings considered here, two (later named GL) are representative of systems designed to sustain gravity loads only and four (later named SR) are designed to resist gravity and seismic loads. Further, the SR systems are distinguished in two groups of two buildings each because of the procedure adopted for their design: specifically, in the first group (later named SR1) the seismic forces are applied to two separate planar models of the structure for the *x*- and *y*-directions while in the second group (SR2) the horizontal forces are applied at the centres of mass of the spatial model of the structure along *x*- and *y*-directions, separately considered. The two buildings belonging to the groups named GL, SR1 and SR2 are characterised by low (-L) and high (-H) levels of the rigidity eccentricity.

All the buildings are symmetric with respect to the *x*-axis (longitudinal direction) and experience lateral-to-torsional coupling of the seismic response only for ground motions acting along the *y*-direction (transverse direction). The deck is always rectangular in plan and has dimensions (B and L) equal to 15.5 m and 28.5 m (see Fig. 2). The centre of rigidity is always coincident with the geometrical centre of the deck. The centres of mass of the decks are lined up along a vertical axis. The position of this axis is shifted with respect to the centre of rigidity so as to obtain values of the rigidity eccentricity equal to $-0.05\,L$ and $-0.15\,L$. The position of the centres of mass is indicated in Figure 2a for the GL structure with $e_r = -0.05\,L$ and in Figure 2b for the SR structure with $e_r = -0.15\,L$. The mass and the radius of gyration of the mass r_m (calculated with respect to the center of mass of the floor) are equal to 440.6 t and 9.91 m (0.348 L) for the GL structures, equal to 477.7 t and 9.87 m (0.346 L) for the SR structures.

The plan layout of GL structure is shown in Figure 2a. The decks are chiefly sustained by eight three-bay frames (Y1 to Y8) along the *y*-axis; two seven-bay frames (X1 and X4) and two single-bay frames (X2 and X3) are arranged along the *x*-axis. Note that columns and beams are generally arranged along the transverse direction; this makes the GL structure flexible and weak against horizontal actions



(a) (b)

Figure 2. Plan layout of the analysed buildings: (a) GL structures; (b) SR structures

acting along the longitudinal direction. Beams and columns of this structure are made of concrete C20/25 (see Eurocode 2) with a characteristic compressive cylindrical strength $f_{ck} = 20$ MPa. Steel grade with a characteristic yield strength $f_{yk} = 375$ MPa is used for the longitudinal and transverse reinforcements. The characteristic value of the dead load is equal to 5.6 kN/m^2 for the deck and equal to 4.2 kN/m^2 for the staircase; the characteristic values of the live load of the deck and the staircase are 2.00 kN/m^2 and 4.00 kN/m^2, respectively. The characteristic value of the weight of the infill walls is 7.0 kN/m. The size of the structural members and the reinforcements are obtained according to the Italian Ministry Decree 14/02/1992. The allowable stresses method is used and the assumed values of the allowable stresses of concrete and steel reinforcement are 8.5 MPa and 215 MPa, respectively. The design internal forces of the structural members are determined considering gravity loads only. In particular, bending moments are ignored for the design of columns. Further, the minimum requirements stipulated by the aforementioned regulations for the cross-sectional area and steel reinforcement are taken into account. Specifically, the cross-sectional area of the columns is not smaller than

$$A_{c,nec} = \frac{N}{0.7\,\sigma_c\,(1 + n\,\rho)} \tag{3.1}$$

where N is the axial force due to the gravity loads determined according to the tributary area concept, σ_c is the allowable stress of concrete, n is the homogenization coefficient assumed equal to 10 and ρ is the reinforcement ratio assumed equal to 0.008. The minimum area A_s of the column reinforcement is

$$A_{s,\min} = \max\begin{cases} 0.003\,A_c \\ 0.008\,A_{c,nec} \end{cases} \tag{3.2}$$

where A_c is the actual cross-sectional area of the column.

The arrangement of the frames in the other two groups of buildings (SR1 and SR2) is typical of seismic-resistant structures (Fig. 2b). The decks are sustained by four seven-bay frames arranged along the x-axis (X1, X2, X3 and X4) and by eight three-bay frames arranged along the y-axis (from Y1 to Y8). Gravity loads are assumed to be the same as those of the GL buildings. Further, also the concrete used for structural members has the same compressive cylindrical strength (20 MPa). Instead, steel grade with a characteristic yield strength $f_{yk} = 430$ MPa is used for the reinforcement. The buildings designed to sustain gravity and seismic loads (SR1 and SR2) are designed according to the regulations in force in Italy from 1996 to 2008 (Ministry Decree, 16/01/1996; Eurocode 2, 1992). Note that for simplicity the cross-sectional area of the members is equal in the structures belonging to the groups SR1 and SR2. The area of the longitudinal and transverse reinforcements is differentiated instead as a function of the results of the design structural analyses. The fundamental periods of vibration of the corresponding planar system (structural model obtained by restraining the deck rotation of the building) in x- and y-directions, T_x and T_y, are reported in Table 3.1 for the structures belonging to the GL, SR1 and SR2 groups.

Table 3.1. Uncoupled periods T_x and T_y of the analysed structures

Structure	T_x	T_y
GL	1.108 s	0.712 s
SR1 and SR2	0.637 s	0.658 s

4. ACCELEROGRAMS

Seven artificial bidirectional ground motions, compatible with the EC8 elastic spectrum for soil type C and characterized by 5% damping ratio and peak ground acceleration a_g equal to 0.35 g, are used in this study for nonlinear time-history analysis. Each accelerogram is characterized by a total duration of 20 s and is enveloped by a “compound” function. The duration of the stationary part of the accelerogram is equal to 7.0 s and thus lower than the minimum value suggested by the Eurocode 8, i.e. 10 s. The adopted value has resulted from a previous investigation in which natural and artificial accelerograms were compared in terms of the input energy spectra, Arias intensity, frequency content and number of equivalent cycles (Amara, 2012). The mean of the peak ground accelerations of the generated accelerograms is not lower than the value stipulated by the Eurocode 8 and no value of the mean response spectrum is lower than 90% of the corresponding value proposed by the Eurocode 8. The SIMQKE computer program (1976) is used to generate the accelerograms.

5. VALIDATION OF THE CORRECTIVE ECCENTRICITIES

The use of the corrective eccentricities determined by the Equations of Section 2 is proposed to improve the estimate of the maximum displacements obtained by nonlinear static methods performed in compliance with current seismic codes, i.e. Eurocode 8. To test the effectiveness of the proposed formulae, the maximum storey displacements and drifts along the *y*-direction are first determined by nonlinear time-history analysis and the average values are assumed as the reference values. Then, the results of the standard and proposed nonlinear static methods are determined and compared with the aforementioned reference values. The analyses are performed by the OpenSEES computer program (Mazzoni *et al*., 2007). Beams and columns are modeled by means of elements with plastic hinges at the ends of the member (Scott and Fenves, 2006). The hysteretic behavior of the steel of the longitudinal reinforcement is represented by the model of Menegotto-Pinto, while the stress-strain curve proposed by Mander is adopted to model the unconfined and confined concrete.

5.1. Load pattern for pushover analysis

The use of corrective eccentricities proposed in this paper is finalized to improve solely the prediction of the torsional response. Unfortunately, nonlinear static methods can lead to errors, which are not negligible also for planar frames (Fajfar and Gašperšič 1996; Bosco *et al*., 2009). In this paper, in order to predict the storey displacements and eliminate the above mentioned errors committed also for planar systems, pushover analysis (PO) is carried out assuming an adaptive load pattern. The adaptive load pattern is defined so as to obtain heightwise distributions of the horizontal displacements proportional to that of the maximum dynamic displacements of corresponding planar structure. To define the load pattern of the pushover analysis PO, the maximum values of the dynamic displacements of the corresponding planar structure subjected to the *y*-components of the bidirectional ground motions are first determined. Second, a displacement-controlled pushover analysis (named PO-D) is performed by means of a displacement load vector which is (i) proportional to the maximum dynamic displacements of the corresponding planar structure and (ii) gradually increased up to the achievement of the top displacement of the corresponding planar structure. Third, the forces of the load pattern of the pushover analysis PO are defined as the difference of the storey shear forces corresponding to two successive steps of the pushover analysis PO-D.

The same procedure is used to define the adaptive load pattern for the prediction of storey drifts, but in this case the components of the displacement load vector adopted in the pushover analysis PO-D are proportional to the sum of the maximum storey drifts from the first storey to the storey under examination.

5.2. Evaluation of the corrective eccentricities

To evaluate the corrective eccentricities of the proposed method, the parameters e_r, e_s, Ω_θ and R_μ are first determined for each of the analysed asymmetric buildings. As the structure of the analysed buildings is symmetric with respect to the geometric centre of the deck C_G, the rigidity centres C_R are lined up along a vertical line passing through C_G. Further, also the centres of mass C_M are lined up along a vertical line. The *x*-component of the rigidity eccentricity e_r is equal to –1.43 m (–0.05 L) for structures with low level of rigidity eccentricity (GL-L, SR1-L and SR2-L) and –4.3 m (–0.15 L) for structures with high level of rigidity eccentricity (GL-H, SR1-H and SR2-H). Since both C_R and C_M lie on the x-axis, the y-component of e_r is always zero.

The centre of strength C_S of all the analysed buildings is located on the x-axis because the reinforcement is always symmetric with respect to this axis. Further, the reinforcement is symmetric also with respect to the y-axis for the structures named GL and SR1. The centre of strength C_S of these structures is coincident with the geometric centre of the deck C_G and with the centre of rigidity C_R. Consequently, the x-component of the strength eccentricity e_s is equal to the x-component of the rigidity eccentricity e_r and the y-component of e_s is zero. To evaluate the abscissa of the centre of strength C_S of the seismic-resistant structures SR2, the pushover analysis of the corresponding planar systems is performed in the y-direction and stopped when the top displacement provided by the nonlinear time-history analysis is achieved. The abscissa of the centre of strength of these structures is then calculated as that of the point of application of the total base shear in the y-direction at the end of the pushover analysis. The x-component of the strength eccentricity e_s is equal to –0.808 m (–0.028 L) for the seismic-resistant structure SR2-L and equal to –2.45 m (–0.086 L) for the seismic-resistant structure SR2-H. Since both C_S and C_M lie on the x-axis, the y-component of e_s is zero.

The parameters $\Omega_{\theta x}$ and $\Omega_{\theta y}$ are calculated as the ratios of the radii of gyration of the lateral stiffness r_{ky} and r_{kx} to r_m. The radii of gyration of the lateral stiffness r_{kx} and r_{ky} are calculated as suggested by Anastassiadis and Makarios (1998)

$$r_{kx} = \sqrt{\frac{u_y(C_R)}{\theta_z}}; \qquad r_{ky} = \sqrt{\frac{u_x(C_R)}{\theta_z}} \tag{5.1}$$

In these relations, the displacements u_y (C_R) and u_x (C_R) are calculated in the centre of rigidity of the 4-th storey and are caused by a set of lateral forces F_i applied at the centres of rigidity of the building along the y- and x-directions, respectively; θ_z is the deck rotation produced at the 4-th storey by the torsional couples $M_i = F_i \times 1$. The values of the radii of gyration of the lateral stiffness r_{kx} and r_{ky} and those of the ratios $\Omega_{\theta y}$ and $\Omega_{\theta x}$ are reported in Table 5.1 for all the analysed structures.

The R_μ factor is evaluated in the y-direction as the ratio of the required elastic base shear to the lateral strength of the structure. The elastic base shear V_{el} is calculated by modal response spectrum analysis of the corresponding planar system. The elastic response spectrum provided by the Eurocode 8 for the soil C and peak ground acceleration equal to 0.35 g is assumed as the seismic input. The lateral strength is calculated as the total base shear provided by the pushover analysis of the corresponding planar system when the top displacement equals the maximum value provided by the nonlinear time-history analysis. The values obtained for the elastic base shear $V_{el,y}$, for the lateral strength $V_{b,y}$ and for $R_{\mu,y}$ are reported in Table 5.2.

Based on the values of the parameters e_r, e_s, Ω_θ and R_μ, the corrective eccentricities are calculated by the Equations of Section 2. The values obtained are reported in Table 5.3.

Table 5.1. Ratios $\Omega_{\theta x}$ and $\Omega_{\theta y}$ of the analysed structures

Structure	r_{kx}	r_{ky}	r_m	$\Omega_{\theta y}$	$\Omega_{\theta x}$
GL	10.051 m	15.510 m	9.913 m	1.014	1.565
SR1, SR2	11.054 m	10.684 m	9.871 m	1.120	1.082

Table 5.2. $R_{\mu,y}$ of the analysed structures

		displacement estimation		drift estimation	
Structure	$V_{el,y}$	$V_{b,y}$	$R_{\mu,y}$	$V_{b,y}$	$R_{\mu,y}$
GL-L, -H	8435.4 kN	3514.8 kN	2.400	3489.9 kN	2.417
SR1-L, -H	12750.5 kN	4846.7 kN	2.631	4835.4 kN	2.637
SR2-L	12750.5 kN	4821.9 kN	2.644	4813.0 kN	2.649
SR2-H	12750.5 kN	4973.2 kN	2.564	4943.0 kN	2.579

Table 5.3. Corrective eccentricities of the analysed structures

	displacement estimation		drift estimation	
Structure	e_1	e_2	e_1	e_2
GL-L	-1.511 m	-0.686 m	-1.510 m	-0.685 m
GL-H	-4.536 m	-2.059 m	-4.529 m	-2.055 m
SR1-L	-1.230 m	-0.454 m	-1.229 m	-0.453 m
SR1-H	-3.691 m	-1.362 m	-3.690 m	-1.362 m
SR2-L	-0.747 m	0.130 m	-0.746 m	0.131 m
SR2-H	-2.280 m	0.363 m	-2.277 m	0.363 m

5.3. Prediction of the seismic response

In this section the seismic response is first shown in terms of the horizontal *y*-displacements developed along the length of the deck. A comparison is made between the mean value of the maximum displacements provided by the nonlinear-time history analyses and the displacements of the proposed nonlinear static method. To emphasise the accuracy of the proposed method the results provided by the standard application of the nonlinear static method are also considered.

The aforementioned displacements are reported in Figure 3 for the first, third and fifth storeys of the systems with rigidity eccentricity equal to 0.15 *L*. The comparison between the results of the nonlinear methods highlights that the corrective eccentricities generally provide accurate estimates of the horizontal displacements of the structures. Based on the results of the buildings considered, the proposed nonlinear method seems to underestimate slightly the displacements of the rigid side of the

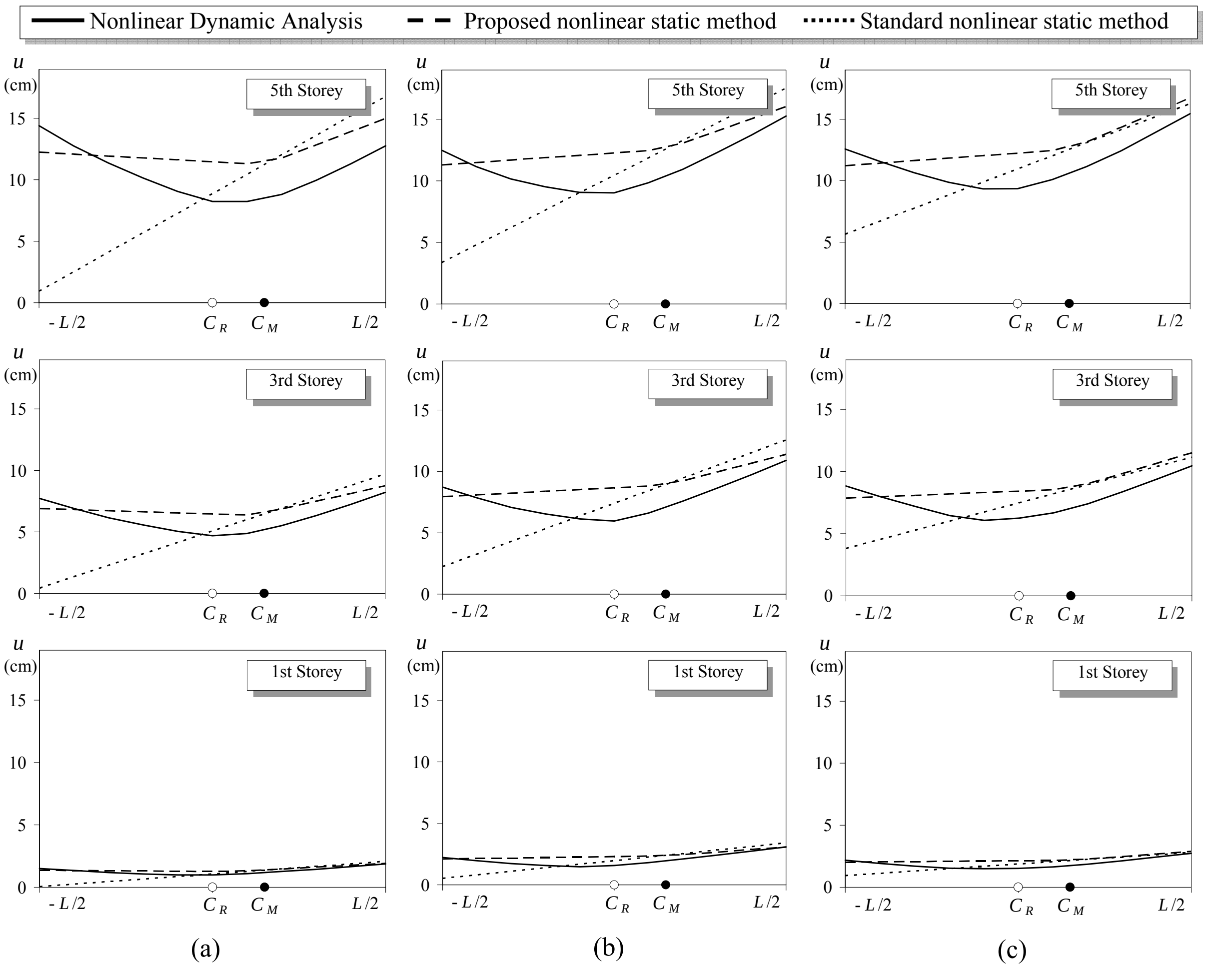


Figure 3. Displacements of the analysed buildings: (a) GL-H structure, (b) SR1-H structure, (c) SR2-H structure

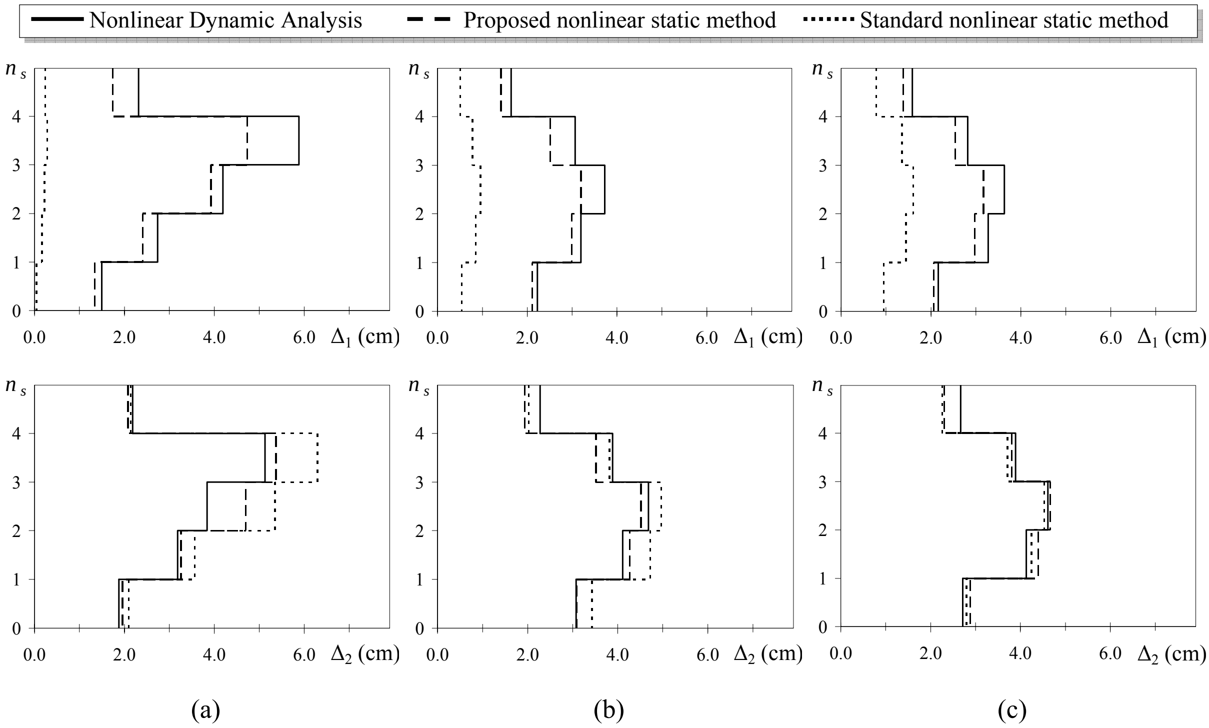


Figure 4. Storey drifts of the analysed buildings: (a) GL-H structure, (b) SR1-H structure, (c) SR2-H structure

building. The displacements of the flexible side are generally slightly overestimated. Instead, as is evident from the figure, particularly accentuated is the underestimation of the displacements of the rigid side when the standard application of the nonlinear static method is used.

Similar considerations apply to the heightwise distribution of the storey drifts at the rigid (Δ_1) and flexible sides (Δ_2) of the buildings, as shown in Figure 4.

6. CONCLUSIONS

The paper shows the validation of a nonlinear static method for the evaluation of the seismic response of asymmetric multi-storey systems. The method requires a double application of the nonlinear static analysis. The effectiveness of the proposed method is evaluated on a set of r.c. framed multi-storey structures characterized by different values of e_r, e_s, Ω_θ and R_μ. The seismic response of these structures is evaluated by nonlinear time-history analyses and compared with that resulting from the application of the proposed nonlinear static method. To highlight the accuracy of the proposed method, a comparison is also made with the response obtained from the standard application of the nonlinear static method, i.e. without corrective eccentricities.
The numerical investigation shows that the adoption of the proposed corrective eccentricities generally provides accurate estimates of the horizontal displacements of multi-storey structures. Based on the results of the buildings considered, the proposed nonlinear method seems to underestimate slightly the displacements of the rigid side of the building. Instead, the displacements of the flexible side are generally slightly overestimated. Particularly accentuated is the underestimation of the displacements of the rigid side when the standard application of the nonlinear static method is used.

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