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Effectiveness of different standard and advanced pushover procedures for regular and irregular RC frames

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Abstract. The purpose of the research presented in this paper was to investigate the effectiveness of several conventional, multi-modal and adaptive pushover procedures. In particular, an extensive numerical study was performed considering eight RC frames characterized by a variable number of storeys and different properties in terms of regularity in elevation. The results of pushover analyses were compared with those of nonlinear dynamic analyses, which were carried out considering different earthquake records and increasing values of earthquake intensity. The study was performed with reference to base shear-top displacement curves and to different storey response parameters. The obtained results allowed a direct comparison between the pushover procedures, which in general were able to give a fairly good estimate of seismic demand with a tendency to better results for lower frames. The advanced procedures, in particular the multi-modal pushover, provided an improvement of the results, more evident for the irregular frames.

Keywords: RC frames; pushover analysis; irregular structures; modal pushover; adaptive pushover; nonlinear dynamic analysis; incremental dynamic analysis

1. Introduction

In recent years the application of nonlinear static procedures experienced a significant increase not only for research but also for practical purposes. The prediction and control of the inelastic response is a fundamental aspect of performance-based design and the traditional linear method may be not effective in limiting the damage levels. The pushover analysis is less onerous than nonlinear dynamic analysis since it does not require the monitoring of cyclic inelastic response of structural members and it avoids the dependence on the input motion. The nonlinear static procedure, however, is affected by several levels of approximation, and the reliability of results depends also from the type of structure. Particular attention should be paid to the prediction of seismic demand and to the definition of a lateral load distribution which is able of reproducing the inertia forces during the seismic response (Fajfar 1999, Fajfar 2002, BSSC 2005, Diotallevi and Landi 2005, Shayanfar *et al*. 2013, Gholi Pour *et al*. 2014).

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Usually the pushover procedures provided by seismic guidelines or regulations, as the ATC-40 (ATC 1996), FEMA 273 (BSSC 1997), FEMA 356 (BSSC 2000) and Eurocode 8 (CEN 2003), are called also conventional or standard procedures and are based on single lateral load distributions which are invariant during the analysis. More advanced procedures have been proposed and applied with the purpose of including higher mode effects (Chopra and Goel 2002), or modification of lateral load distribution during analysis in the inelastic range (Antoniou and Pinho 2004a, Papanikolau and Elnashai 2005). These effects may be significant especially for high-rise or irregular structures in elevation and in plan.

The multi-modal pushover analysis initially proposed by Chopra and Goel (2002) for considering the effects of higher modes has been subsequently verified and modified (Chintanapakdee and Chopra 2003, Chopra *et al*. 2004, Goel and Chopra 2005). It is substantially an extension of the standard linear modal analysis to buildings with inelastic behaviour. The procedure requires to perform different pushover analyses, one for each considered vibration mode. The results relative to each mode are then combined through common modal combination rules. The critical aspects of the method regard the modal uncoupling operation, proposed in the inelastic range, and the combination rule. Moreover, the square based combination rules neglect the sign of modal forces. In this sense several procedures have been proposed with the purpose to consider a direct combination of modal parameters (Kunnath 2004, Kunnath and Kalkan 2004, Moghadam and Tso 2002). Recently the multi-modal pushover has been upgraded to take account of the cyclic response of special moment resisting frame buildings (Bobadilla and Chopra 2008) and it has been also applied for probabilistic evaluations (Han *et al*. 2010). Other authors proposed further improvements of the procedure (Poursha *et al*. 2009, Jiang *et al*. 2010).

An increasing excursion in the inelastic range may cause redistribution of inertia forces and significant effects of higher modes. In this condition the possibility of continuously updating the load shape during the analysis, as in the adaptive procedures, seems to be an effective improvement of the pushover analysis. In several adaptive procedures the lateral load pattern is updated at each loading increment considering a combination of the instantaneous mode shapes associated to the inelastic periods of the structure. The mode shapes are recalculated at each step of analysis considering the current stiffness properties of the structure. The modal combination may be applied to lateral forces associated to each mode or also to the effects of these forces (Aydinoglu 2003). Within adaptive approaches it is possible to distinguish also procedures based on application of a displacement vector rather than a force vector (Antoniou and Pinho 2004b). Other interesting procedures have been proposed in scientific literature (Colajanni *et al*. 2008, Elnashai 2001, Gupta and Kunnath 2000, Kalkan and Kunnath 2006, Kalkan and Kunnath 2007). The major issues related to the use of adaptive pushover analysis is still linked to the modal uncoupling operation performed in the inelastic range, and to the assumption of correspondence between the “pathways of damage” followed by the structure subjected to seismic input and during the pushover analysis.

The purpose of the present study was to perform a very extensive numerical investigation in order to evaluate the effectiveness of different conventional and advanced procedures. A modified adaptive technique was also proposed. The verification of the pushover procedures was made by comparison with nonlinear dynamic analyses. The study was developed with reference to a set of RC frames characterized by a variable number of storeys and different properties in terms of regularity in elevation. In this way it was possible to examine the influence of higher modes and of irregularity on the effectiveness of the considered procedures.

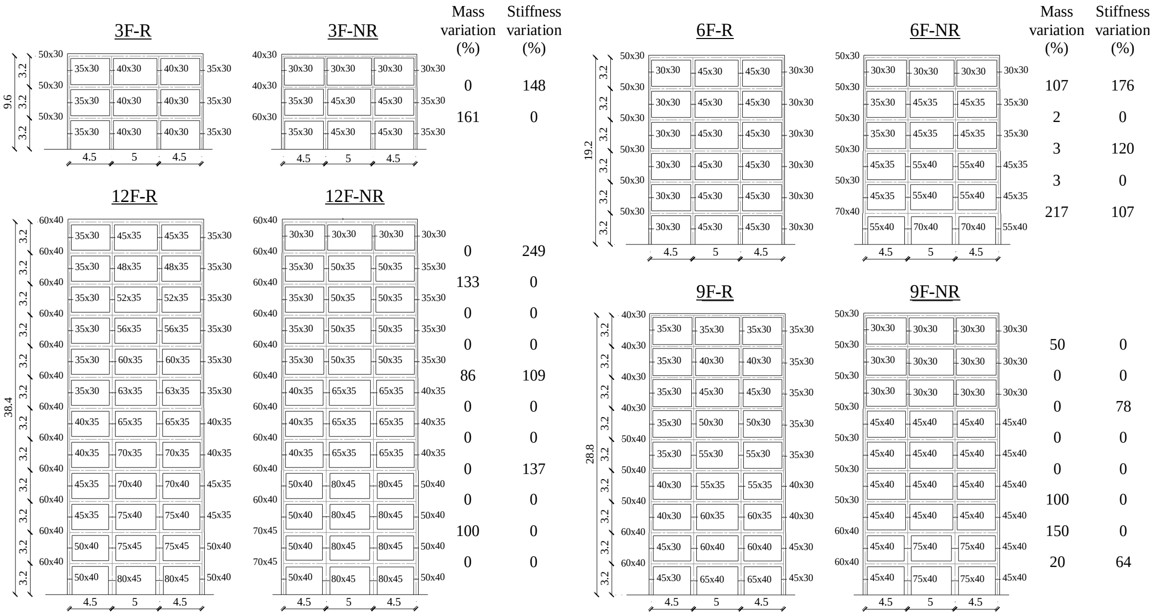

Fig. 1 Structures under study (length and height in m, cross sections in cm)

2. Structures under study and nonlinear model

The structures under study are eight RC plane frames characterized by three spans and by a number of storeys equal to three, six, nine and twelve (Fig. 1). Two structures are associated to each number of storeys, one regular and the other irregular in elevation. The design was performed according to Italian Seismic Code (Min. LL. PP. 2008), inspired to Eurocode 8 (CEN 2003). The response spectrum for medium soil condition and for a peak ground acceleration (PGA) equal to 0.35g was considered. Provisions and rules for the high ductility class as well as capacity design criteria were followed in the design. Assumed mechanical properties of materials are: concrete cylinder strength equal to 30 Mpa and steel yield strength equal to 430 Mpa. The regular structures, denoted with letter R, were dimensioned according to code criteria for regularity in elevation. In particular, variations of mass and stiffness between adjacent storeys lower than 20% were considered. Values of these variations much larger than code limits for regularity (Fig. 1) were adopted for the irregular structures, denoted with letter NR. From Table 1 it is evident the importance of the second and third mode of the irregular structures.

Nonlinear static and dynamic analyses were carried out using OpenSees software (McKenna and Fenves 2005, Mazzoni *et al*. 2007). Each structural member, column or beam, was modelled with a single distributed plasticity finite element. Five control sections were adopted, two located at the ends and the other along the element. Their response was studied by means of a fibre model (Diotallevi and Landi 2006, Diotallevi *et al*. 2008). A bilinear stress-strain relationship with hardening was adopted for the steel fibres. A constitutive law, similar to that proposed by Mander *et al*. (1988), which includes the effect of confinement due to stirrup and the stiffness degradation due to cyclic loading was considered for the concrete. The geometrical nonlinearity was considered both in dynamic and pushover analyses in terms of P-delta effects.

Table 1 Periods and effective modal masses for the structures under study

	3F-R		3F-NR		6F-R		6F-NR		9F-R		9F-NR		12F-R		12F-NR	
Mode	*T*[s]	*M*[%]	*T*[s]	*M*[%]	*T*[s]	*M*[%]	*T*[s]	*M*[%]	*T*[s]	*M*[%]	*T*[s]	*M*[%]	*T*[s]	*M*[%]	*T*[s]	*M*[%]
1	0.61	83	0.53	65	1.21	80	0.74	54	1.46	68	1.23	45	1.90	74	1.28	61
2	0.20	14	0.21	31	0.40	12	0.27	18	0.57	19	0.49	30	0.66	12	0.52	16
3	0.11	3	0.11	4	0.22	4	0.17	14	0.32	6	0.31	16	0.38	4	0.31	10
4	-	-	-	-	0.15	3	0.12	12	0.22	4	0.22	3	0.25	3	0.23	4

3. Examined pushover procedures

Eight pushover procedures were evaluated in this research. Four are based on invariant load shapes, one is the multi-modal procedure and three are based on adaptive load vectors. The following paragraphs illustrate these procedures and give the expressions of the forces to be applied at each storey. Before the application, these forces have to be normalized and multiplied by the current load increment. The load vector increment in a current step of analysis is defined through a normalization based on the value of base shear

$$\Delta P_i = \frac{F_i}{\sum_{j=1}^{N} F_j} \Delta\lambda_F \tag{1}$$

where ΔP_i is the increase of the force at the storey i in the current step of analysis, F_i is the not-normalized expression of the forces at the storey i , N is the number of storeys and $\Delta\lambda_F$ is the load multiplier for the step under consideration. In case of a procedure based on the imposition of a displacement vector, the imposed displacement vector increment in a current step of analysis is defined as follows

$$\Delta D_i = \frac{d_i}{d_{top}} \Delta\lambda_d \tag{2}$$

where ΔD_i is the displacement increment imposed at the storey i, d_i is the not-normalized displacement, d_{top} is the top displacement and $\Delta\lambda_d$ is the displacement multiplier for the step under consideration.

Load vectors normalization is useful to simplify the calculation procedure in terms of iterations required to achieve the convergence at each step of analysis, but it is not determinant on the results. Considering different methods of normalization does not lead, in fact, to different results in the cases without problems of convergence. For this reason, in order to simplify the reading of the formulas, the not-normalized expressions of forces and displacements applied at each storey of the structure are shown in the following.

The four considered procedures based on invariant load shapes and the corresponding load distributions are

$$\text{- Uniform} \quad F_i = m \tag{3}$$

$$\text{- Code} \quad F_i = m_i h_i \tag{4}$$

$$\text{- 1st mode} \quad F_i = m_i \phi_{i1} \tag{5}$$

$$\text{- SRSS} \quad F_i = \sqrt{\sum_{j=1}^{N_m} \left(m_i \phi_{ij} S_a \left(T_j \right) \Gamma_j \right)^2} \tag{6}$$

In the previous equations F_i is the force applied at the storey i, m_i is the mass of the storey, h_i is the height of the storey from the base of the building, ϕ_{ij} is the modal deformation of mode j, S_a is the spectral acceleration corresponding to the vibration period T_j of mode j, N_m is the number of considered modes and Γ_j is the modal participation factor

$$\Gamma_j = \frac{\sum_{i=1}^{N} m_i \phi_{ij}}{\sum_{i=1}^{N} m_i \phi_{ij}^2} \tag{7}$$

In the invariant SRSS load distribution the modal shapes are combined before to execute the pushover analysis. In the multi-modal procedure (MPA) the individual modal patterns are applied to perform different pushover analyses, one for each considered mode j (Chopra and Goel 2002)

$$\text{- Multi-modal} \quad F_i = m_i \phi_{ij} \tag{8}$$

The modal deformations are calculated before the pushover analyses and are assumed to be invariant. For the structures under study, as much modes as to activate altogether more than 85% of total mass and individually more than 5% of total mass were considered. Three modes were considered for all frames except for 6F-NR, since in this case four modes were used. The MPA procedure requires the calculation for each mode of the displacement demand. In this work the displacement demand $d_{top,j}$ at the top of the building of each mode j was calculated directly by using the displacement coefficient method proposed in FEMA 440 (BSSC 2005)

$$d_{top,j} = C_0 C_1 C_2 S_a \left(T_{e,j} \right) \frac{T_{e,j}^2}{4\pi^2} \tag{9}$$

where $T_{e,j}$ is the elastic period associated to the bilinear idealization of the pushover curve of mode j and S_a is the elastic spectral acceleration. C_0 relates the spectral to the top displacement, C_1 relates the inelastic to the elastic spectral displacement while C_2 represents the effect of hysteretic shape. When the displacement demand for each mode is known, the corresponding response parameters are extracted. The response parameters due to a proper number of modes are then combined using a modal combination rule as SRSS or CQC. Since the comparative study was made at a displacement $d_{top,NDA}$ equal to that obtained from nonlinear dynamic analyses, the MPA procedure was modified in order to obtain at the top the same displacement $d_{top,NDA}$. In particular, considering the SRSS rule, the displacement demand of each mode was multiplied by a scaling factor λ

$$\lambda = \frac{d_{top,NDA}}{\sqrt{\sum_{j=1}^{N_m} d_{top,j}^2}} \tag{10}$$

Finally, three adaptive procedures were considered: two are based on forces (FAP), one is based on displacements (DAP). Forces and displacements profiles for a mode j and for a step n are defined as follows

$$\text{- FAP1} \quad F_{ij}^{(n)} = m_i \phi_{ij}^{(n)} S_a\left(T_j^{(n)}\right)\Gamma_j^{(n)} \tag{11}$$

$$\text{- FAP2} \quad F_{ij}^{(n)} = m_i \phi_{ij}^{(n)} S_d\left(T_j^{(0)}\right)\omega_j^{(n)2}\Gamma_j^{(n)} \tag{12}$$

$$\text{- DAP} \quad d_{ij}^{(n)} = \phi_{ij}^{(n)} S_d\left(T_j^{(n)}\right)\Gamma_j^{(n)} \tag{13}$$

In these equations $S_a(T_j^{(n)})$ is the elastic spectral acceleration associated to the current period $T_j^{(n)}$ of mode j. The current periods $T_j^{(n)}$ and modal deformations $\phi_{ij}^{(n)}$ are recalculated at each step by considering the current stiffness properties of the structure. The procedures FAP1 and DAP were proposed in literature (Antoniou and Pinho 2004a, Antoniou and Pinho 2004b). Eq. (12) represents the modified adaptive procedure proposed by the authors and may be derived by replacing in Eq. (11) the elastic spectral acceleration associated to the current period $T_j^{(n)}$ with the elastic spectral displacement associated to the initial period $T_j^{(0)}$ multiplied by the square of the current circular frequency $\omega_j^{(n)}$. Some authors proposed, in fact, particular adaptive procedures where, among other features, the spectral acceleration is expressed by the product of the square of the current circular frequency with the inelastic spectral displacement (Aydinoglu 2003). This modification was introduced in order to better account for the nonlinear response of the structure. On the basis of such considerations, with the purpose to obtain also a simple improvement of the procedure FAP1, in the present paper it is proposed the above described modification. In order to reduce the complexity, on the basis of the equal-displacement rule, in place of the inelastic spectral displacement it was considered the elastic spectral displacement calculated at the initial period.

The total forces at each storey are obtained by combining the contributions of a proper number of modes using a rule as SRSS or CQC. For the SRSS rule

$$F_i^{(n)} = \sqrt{\sum_{j=1}^{N_m} F_{ij}^{(n)2}} \tag{14}$$

$$d_i^{(n)} = \sqrt{\sum_{j=1}^{N_m} d_{ij}^{(n)2}} \tag{15}$$

In the application of Eq. (14) and Eq. (15) for the structures under study, the same number of modes as for the MPA was considered.

4. Comparison between pushover and nonlinear dynamic analyses

Ten ground motion records (Table 2) characterized by an average response spectrum compatible with the design one (Fig. 2) were selected for the nonlinear dynamic analyses. They were applied to the eight structures under study. The nonlinear dynamic analyses were repeated considering for each record increasing values of seismic intensity in terms of PGA.

The response of each nonlinear dynamic analysis may be characterized by a point, whose coordinates are the maximum values, during the earthquake, of base shear and top displacement.

Table 2 Selected ground motions for the time-history analyses

Ground Motion	Year	Station	PGA [g]
Imperial Valley	1940	El Centro	0.341
Kern	1952	Taft	0.178
Parkfield	1966	Cholame#2	0.479
Iperial Valley	1979	El Centro D Array	0.480
Irpinia	1980	Sturno	0.358
Kalamata	1986	Kalamata	0.275
Loma Prieta	1989	Capitola	0.529
Northridge	1994	Newhall	0.591
Northridge	1994	Rinaldi (RRS)	0.838
Kobe	1995	KJMA (KJM)	0.821

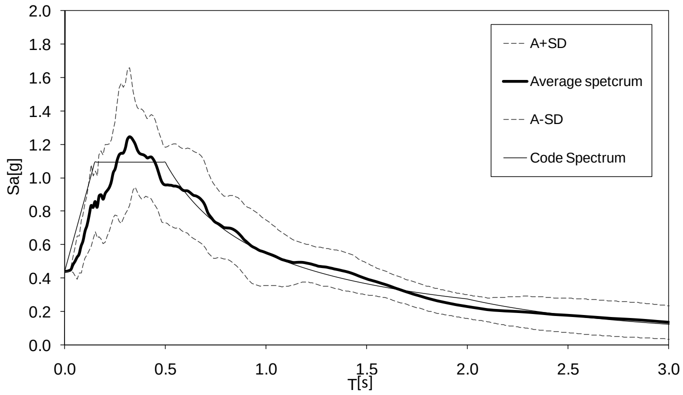

Fig. 2 Average spectrum of the selected ten ground motions and design spectrum

Through the incremental dynamic analysis (IDA) it is possible to build for each earthquake record a "dynamic pushover curve" and to compare it with the "static pushover curve" (Mwafy and Elnashai 2001, Papanikolau and Elnashai 2005, Papanikolau *et al*. 2006). The comparison of the pushover curves, however, does not give information about distribution of forces and deformations between the storeys of a building. Therefore, also comparisons regarding storey response parameters, as inter-storey drift and storey shear, were carried out. In this case the response parameters were evaluated at the storey level and the results obtained with nonlinear dynamic and pushover analyses were compared with reference to the earthquake records scaled to the design PGA. The comparison of the storey response parameters was made considering the results of pushover analyses corresponding to values of top displacement equal to those of nonlinear dynamic analysis. With regard to the evaluation of the storey response parameters, 160 comparisons for each of the considered pushover procedures were performed, for a total of 1280 comparisons.

Fig. 3 shows the comparison between the IDA and the pushover curves by distinguishing invariant from modal and adaptive procedures, and the regular from irregular frames. The curve corresponding to modal procedure was built from the pushover curves of the single modes by combining values of displacement demand and base shear associated to increasing values of

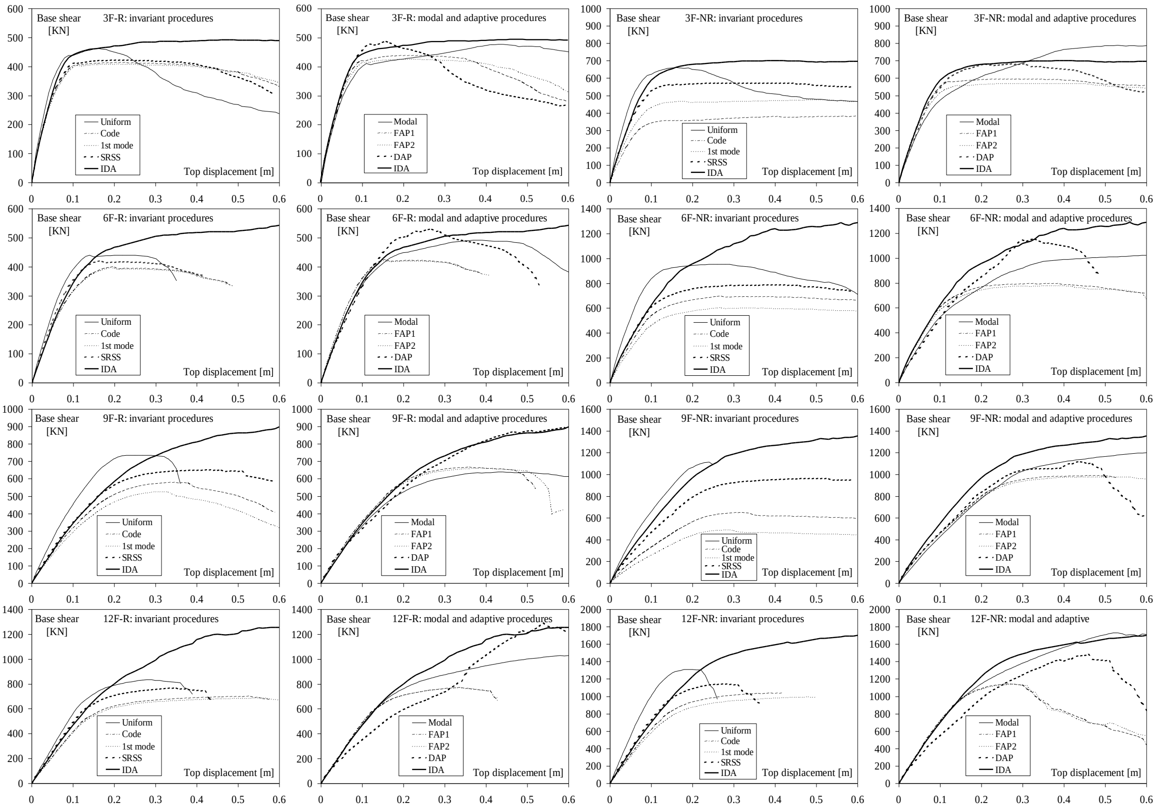

Fig. 3 Base shear-top displacement curves

seismic intensity. Some of the pushover curves in Fig. 3 were obtained with load shapes correlated to the spectral acceleration. In these cases the average response spectrum of the ten earthquake records was considered. With regard to the IDA curves, the average of the ten curves obtained with the individual earthquake records is shown in the figure. The IDA curves used as reference curves in the comparison study were derived considering the absolute maximum values of base shear and top displacement, neglecting the fact that these values may occur at different time instants (Papanikolau and Elnashai 2005). This approach may correspond better to the response of a structure during pushover analysis, where the maximum values of base shear and top displacement occur simultaneously. The pushover curves derived with uniform load shape are characterized by larger stiffness and values of base shear than IDA and other pushover curves, especially in the elastic range. The curves obtained with other invariant procedures are quite close to IDA curves in the elastic range. In the inelastic range they show lower values of base shear than IDA curves. Among these procedures a good agreement with IDA analyses was found with the SRSS load shape. For all structures the code and first mode distributions provided significantly lower values of base shear than SRSS load shape. This result is particularly evident in the inelastic range and for irregular structures. The smallest values of base shear were obtained with the first mode load shape. The invariant load distributions produced in general better results for lower than for higher structures. All the curves obtained with invariant load shapes are characterized by a peak point and by a subsequent horizontal or descending branch.

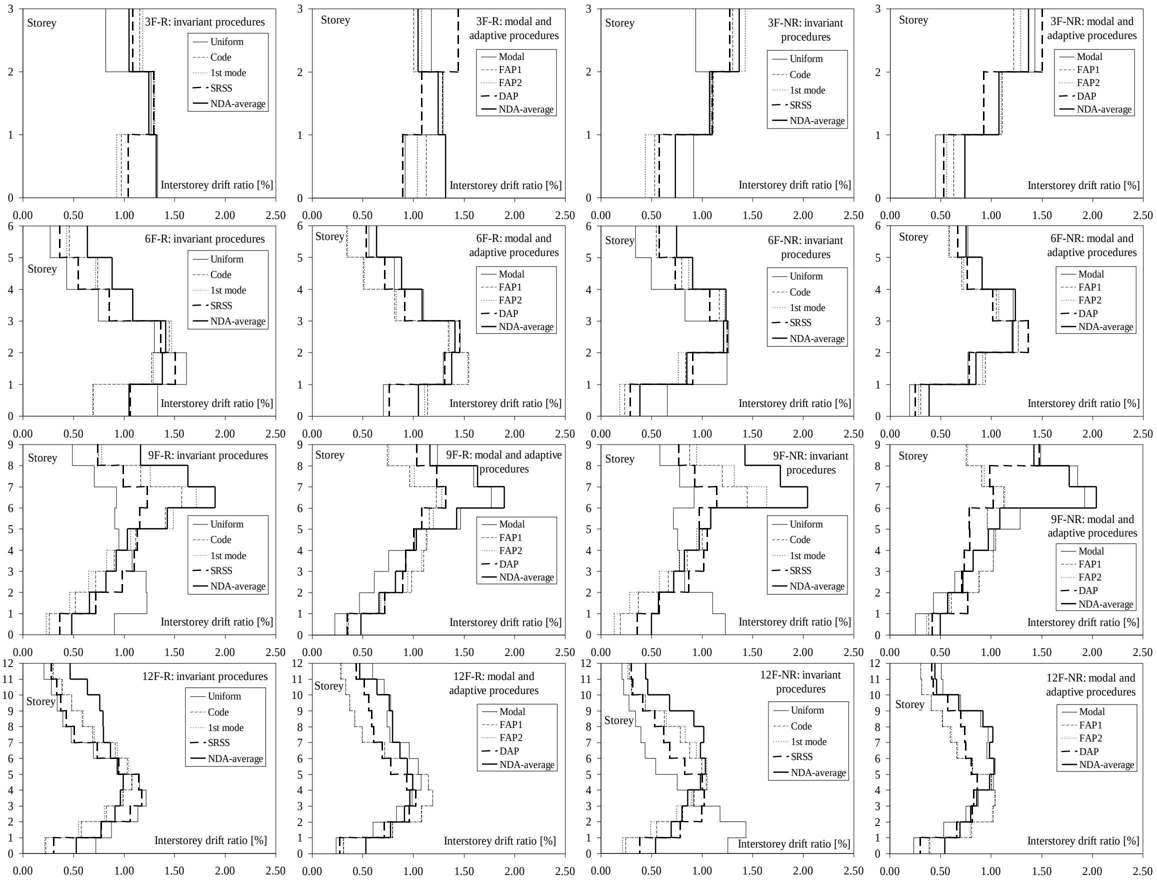

Fig. 4 Inter-storey drift profiles

On the contrary the IDA curves of the considered buildings are characterized by monotonically increasing values of base shear, in particular for higher buildings. The pushover curves obtained with modal pushover show a good agreement with IDA curves. In the elastic range they are close to IDA curves, in the inelastic range they are characterized by lower values of base shear. However this underestimation of base shear is smaller than that obtained with invariant SRSS load shape. The pushover curves determined with force-based adaptive procedures are similar to those obtained with invariant SRSS load shape, especially in terms of base shear in the inelastic range. The differences between the two FAP procedures are negligible. In comparison with FAP procedures, the DAP gave curves characterized by lower stiffness in the elastic range and by values of base shear closer to those calculated with IDA analyses in the inelastic range.

The comparison between the drift profiles determined with dynamic and pushover analyses is illustrated in Fig. 4. With regard to dynamic analyses, the average values of the inter-storey drifts obtained with the ten earthquake records scaled to design PGA are shown. The inter-storey drifts in the figure are normalized to the storey height. The maximum drift ratio was about 1.5% for the frames with three and six storeys, 2% for the frame with nine storeys and 1% for the frame with twelve storeys. The uniform load shape yielded a drift profile significantly different from the envelope of dynamic analyses, especially for higher frames. With this procedure, larger values at

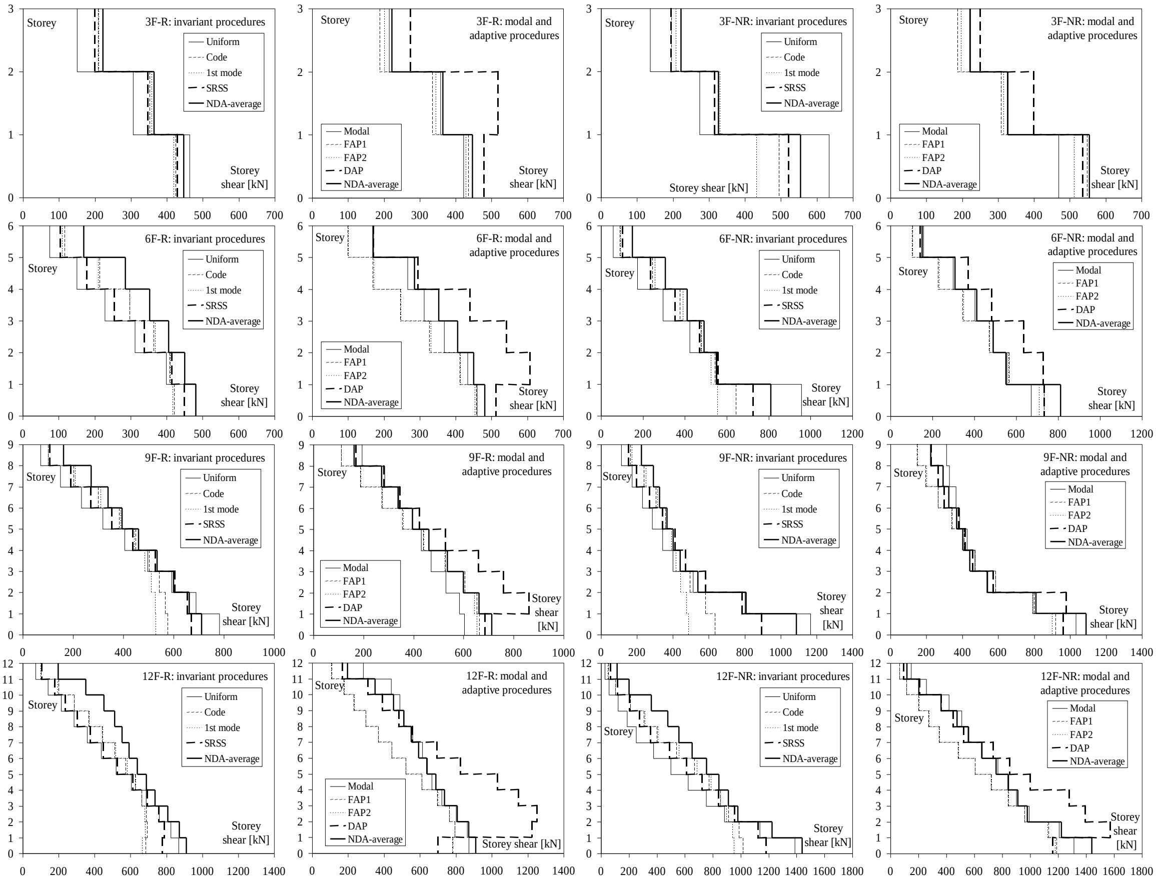

Fig. 5 Storey shear profiles

the lower storeys and lower values at the upper storeys than with dynamic analyses were obtained.

The drift profiles determined with code and first mode invariant distributions are similar to the envelope of dynamic analyses, in particular for frames with three and six storeys. These two procedures provided in almost all cases values of drift lower than those calculated with dynamic analyses. Anyway they gave a quite good estimate of position and value of maximum drifts. The SRSS invariant distribution, compared with the code and the first mode, produced values more close to the dynamic envelope at the lower storeys, but less close at the upper storeys. The best agreement with the envelope of dynamic analyses in terms of shape and of maximum values was obtained with multi-modal pushover. With this procedure the tendency to a slight underestimation at the lower storeys was observed. The force-based adaptive procedures gave results similar to those obtained with SRSS distribution. The FAP2 procedure yielded slightly better estimates than the FAP1. In general the DAP procedure gave good estimates of drift profile at the lower storeys and better estimates than other adaptive procedures at the upper storeys.

The comparison between the envelopes of storey shear obtained with dynamic and pushover analyses is illustrated in Fig. 5 for the frames under study. The results at the base reflect what observed about the force-displacement curves. All the invariant procedures, except the uniform distribution, gave values lower than the dynamic analyses. In fact, the uniform shape provided in

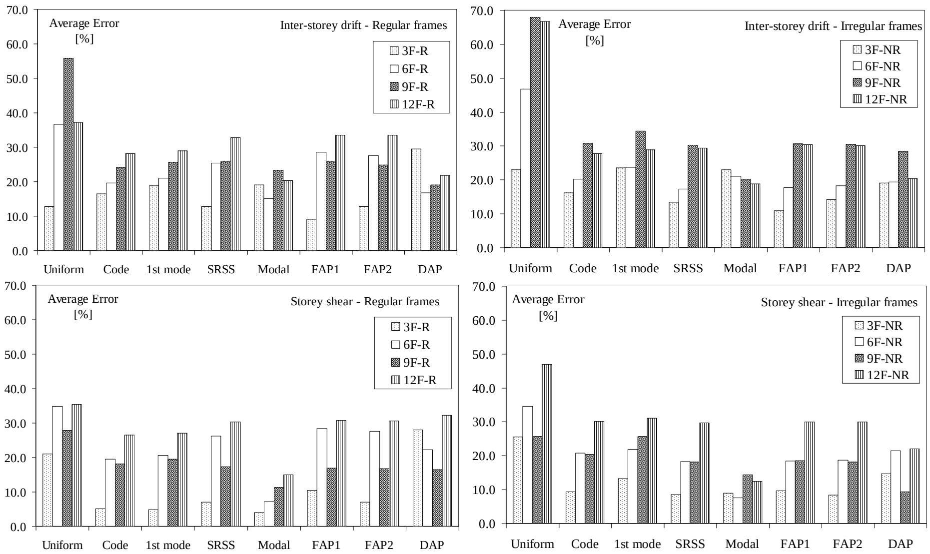

Fig. 6 Average error in the prediction of the inter-storey drift and storey shear

almost all cases an overestimation of storey shear at the lower storeys and an underestimation at the upper storeys. With the SRSS distribution the best estimates, among the invariant procedure, were obtained at the lower storeys, where the values of storey shear are larger. The modal pushover provided a storey shear envelope quite similar to that of dynamic analyses. The force-based adaptive procedures provided results similar to those of SRSS invariant procedure at the lower storeys, and an underestimation at the upper storeys. On the contrary the DAP procedure gave a good estimate at the upper storey and a significant overestimation at the lower storeys, except at the first one.

An average of the error in the prediction of the storey response parameters was evaluated as follows

$$s_i = \frac{r_{NDA,i} - r_{POA,i}}{r_{NDA,i}} \tag{16}$$

$$E = \sqrt{\frac{\sum_{i=1}^{N} s_i^2}{N}} \tag{17}$$

where $r_{NDA,i}$ is the average response parameter from nonlinear dynamic analyses for the storey i, $r_{POA,i}$ is the value of the same parameter from pushover analysis and N is the number of storeys. Figure 6 shows the values of the error E as defined in Eq. (17) for the different pushover procedures with regard to inter-storey drift and base shear. A general tendency to an increase of the error with the number of storey was observed. Also the irregularity influenced the error, but this influence resulted lower than that of the number of storeys and it was evident mainly for the frame with nine storeys. With regard to the inter-storey drift, values of E around 25% were obtained for higher frames and for all procedures except for the uniform one, which provided significantly larger values. The modal procedure yielded lower values of *E* than other procedures, especially for higher frames. Also with DAP procedure low values for higher structures were obtained. The error of pushover analyses was slightly lower for storey shear than for inter-storey drift. It resulted around 20% for all procedures except for the uniform distribution. Also for the storey shear the lower values were obtained with modal pushover.

5. Conclusions

In this work an extensive numerical investigation was carried out in order to compare invariant, modal and adaptive pushover procedures with nonlinear dynamic analyses. The study regarded eight RC frames with variable number of storeys and different properties in terms of regularity in elevation.

In the evaluation of base shear-top displacement curves the best agreement with results of nonlinear dynamic analyses was obtained with the procedures which account for contribution of higher modes, as the SRSS invariant, the modal and the adaptive ones. This result was particularly evident in the inelastic range and for the irregular structures. The invariant load distributions produced in general better results for lower than for higher structures. The multi-modal procedure provided better results than SRSS invariant distribution, thus indicating that modal combination applied on response parameters was more effective than that applied on lateral forces.

To assess the effectiveness of the procedures, also storey response parameters were examined. With regard to the inter-storey drift, the best agreement with the envelope of dynamic analyses in terms of shape and of maximum values was obtained with the multi-modal pushover. The invariant procedures, except the uniform, yielded a drift profile similar to the envelope of dynamic analyses, even if characterized by lower values of drift. The force-based adaptive procedures gave results similar to SRSS invariant distribution. The modification introduced in the force-based adaptive procedure produced better results, but the improvement was very low. With regard to the storey shear, as for the force-displacement curves, better results were obtained by considering higher modes in the pushover. Also for the storey shear envelope the results of modal pushover were close to those of dynamic analyses. The irregularity influenced the error in the prediction of the results of dynamic analyses, but the improved procedures limited this influence. The error of the estimate given by pushover procedures was particularly affected by the number of storeys.

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