Displacement-based performance assessment of steel moment resisting frames

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ABSTRACT: The assessment of inelastic seismic behavior and performance of SMRF structures was developed. At this aim, some specific performance criteria both for structural members and for non-structural components at the different limit state were used. An incremental non-iterative nonlinear static procedure based on adaptive capacity spectra method was used for performance assessment. The results obtained were compared with incremental time-history response.

1 INTRODUCTION

The estimation of lateral displacement demands is of primary importance in performance-based earthquake-resistant design, especially, when damage control is the main quantity of interest. In particular, the steel moment resisting frames (SMRFs) are expected to be able to sustain large plastic deformations in bending and shear. However, so that the dissipative capacity of the structure can be completely activated, it is necessary to optimize the energy dissipation and guarantee the formation of a global plastic mechanism of collapse. Such objectives are persecuted not directly through nonlinear response history analysis, but indirectly through design procedures essentially based on capacity design. These traditional design provisions may be not effective to obtain a global plastic mechanism and to avoid that interruption or damage may far outweigh the cost of the structural system. It is, rather necessary a multi-level and multi-objective design procedure based on the estimation of the global behavior of the structure in terms of lateral displacement. At this aim, this study develops a simplified seismic demand estimation procedure in which the spectral characteristics of the ground motion are related to the inelastic deformation capacity for the structure.

2 DISPLACEMENT-BASED CAPACITY SPECTRUM METHOD

2.1 *Adaptive and multimodal pushover*

Static pushover analysis is usually employed to determine the deformation demands with acceptable accuracy without the intensive modeling and computational effort of a dynamic analysis. The lateral force distribution should be defined to reproduce the inertia forces deriving from the earthquake ground motion. As the damage progresses, the inertia forces are redistributed, the vibration properties of the structure change and local plastic mechanism may occur. As a consequence, also the original participation and dynamic amplification of the mode shapes changes, and higher mode effects may be significantly increased. Therefore, multimodal and adaptive pushover analyses may be required to improve the accuracy of the deformation estimates. Several researchers have proposed adaptive force distributions that attempt to follow more closely the time-variant distributions of inertia forces. These approaches can give better estimations of the inelastic response, but they are conceptually complicated and computationally demanding for application in structural engineering practice. The Modal Pushover Analysis (MPA) (Chopra and Goel, 2002) allows for the change in load distribution due to damage of the structure without resorting to an adaptive load pattern. Target displacement values are computed by applying equivalent nonlinear procedures with a SDOF system representative of each modal load pattern and, finally, response quantities are combined with the SRSS method. Other authors (Antoniou and Pinho 2004) proposed adaptive pushover procedures: Force-based adaptive pushover (FAP) and Displacement-based adaptive pushover (DAP). Particularly, in the force-based adaptive pushover approach (FAP), a modal analysis is performed step by step to update the force modal ratios. The lateral load distribution is continuously updated during the process according to modal properties, softening of the structure, its period elongation, and the modification of the inertial forces due to spectral amplification. The lateral load profiles of each vibration mode are then combined by using SRSS or CQC method. An incremental updating with increment of load calculated according to the spectrum scaling is applied at each analysis step. Despite its apparent conceptual superiority, the results obtained through FAP appear to be similar to those from conventional pushover analysis. Both types of analysis may give very poor prediction of deformation patterns. In the displacement-based adaptive pushover (DAP), the modal shape is directly imposed to the structure, using a displacement control analysis. The maximum interstorey drift values are obtained directly from modal analysis, rather than from the difference between not-necessarily simultaneous maximum floor displacement values. However, the use of SRSS or CQC rules to combine modal results lead to load vector shapes which neglect the possibility of sign change in storey displacements from different modes. Generally, the displacement-based adaptive pushover provides much improved approximation of highly irregular dynamic deformation profile envelopes, even if it assumes that all the interstorey drifts are maxima at the same time, which is of course not realistic. Two shortcomings of the modal combination rules can be pointed out: the first one is that the result obtained does not fulfill equilibrium; the second limitation is that signs are lost during the combination process eliminating the contribution of negative quantities. In other words, an "always-additive" inclusion of higher modes contribution is considered.

2.2 *Non-iterative Capacity Spectrum Method*

The result of the analysis is the pushover curve, which plot a deformation index (typically roof displacement δ_{TOP}) against a force index (typically base shear V). This capacity curve (CC) is the starting point for all the NSP_s based on Capacity Spectrum Method. In the case of adaptive pushover, the lateral load pattern is updated during pushover analysis according to variation in modal properties as the stiffness of the structure changes.

This leads to variation in lateral displacement pattern and in lateral force pattern. Therefore, also the equivalent SDOF system, which is representative of MDOF three-dimensional model of the building in the Capacity Spectrum Method, changes during pushover analysis. In order to consider such effect, an adaptive version of the Capacity Spectrum Method (ACSM) is considered in the analyses. At each step of the pushover analysis a different equivalent SDOF system is defined as a function of the actual lateral displacement pattern. Particularly, the mass M_{eq} and the stiffness K_{eq} of the equivalent SDOF system at the i^{th} step of pushover analysis can be expressed as a function of the j^{th} storey displacement, as follows:

$$M_{eq}^{i} = \frac{\left(\sum_{j=1}^{N} m_j \cdot \delta_j^i\right)^2}{\sum_{j=1}^{N} m_j \cdot \delta_j^{i\,2}} \qquad K_{eq}^{i} = \frac{\left(\sum_{j=1}^{N} m_j \cdot \delta_j^i\right)^2}{\left(\sum_{j=1}^{N} m_j \cdot \delta_j^{i\,2}\right)^2} \sum_{j=1}^{N} F_j^i \cdot \delta_j^i \quad (1)$$

where F_j^i is the j^{th} storey force at the i^{th} step. The transformation from Capacity Curve (CC) to Capacity Spectrum (CS) in ADRS format (Acceleration-Displacement Response Spectra) is carried out considering the following variation of the spectral coordinates to every step of pushover analysis:

$$\Delta S_a^i = \Delta V^i \cdot \frac{\sum_{j=1}^{N} m_j \cdot \delta_j^{i\,2}}{\left(\sum_{j=1}^{N} m_j \cdot \delta_j^i\right)^2} \quad \Delta S_d^i = \Delta d_{TOP}^i \frac{1}{\delta_N^i} \frac{\sum_{j=1}^{N} m_j \cdot \delta_j^{i\,2}}{\sum_{j=1}^{N} m_j \cdot \delta_j^i} \quad (2)$$

Finally, the CS is approximated with an elastic-perfect-plastic equivalent model (Bilinear Capacity Spectra – BCS). In particular, the elastic stiffness and the yielding displacement S_{dy} are defined from the point of the CS correspondent to 60% of the yielding acceleration S_{ay}. The seismic demand is generally represented by means of the Inelastic Demand Response Spectra (IDRS – S_d versus S_a). In this paper the IDRS are computed scaling the 5%-damped Elastic Demand Response Spectra (EDRS – S_{de} versus S_{ae}) as follows:

$$S_a = \frac{S_{ae}}{R_\mu} \qquad S_d = \frac{\mu \cdot S_{de}}{R_\mu} \quad (3)$$

A reduction factor depending on velocity and displacement elastic spectra is adopted (Ordaz et al. 1998):

$$R_\mu = 1 + \left(\frac{S_v(T)}{PGV}\right)^{\alpha(\mu)} \cdot \left(\frac{S_d(T)}{PGD}\right)^{\beta(\mu)} \cdot (\mu - 1) \quad (4)$$

where PGV is the peak ground velocity; PGD is the peak ground displacement; $S_d(T)$ is the elastic spectral displacement; $S_v(T)$ is the elastic spectral velocity; $\alpha(\mu)$ and $\beta(\mu)$ are functions obtained with a statistical data analysis on spectrum compatible earthquakes (Ferraioli et al., 2004). R_μ depends from the ductility μ and, therefore, from the lateral displacement of the equivalent SDOF system. Consequently, an iterative procedure is usually required in order to estimate the intersection between IDRS and BCS. Applied for the displacement-based assessment the capacity spectrum method may become non-iterative. In fact, the performance-based assessment is displacement-based since the performance parameters used in the acceptance criteria are the plastic rotations and the interstorey drift damage index (IDI). As a consequence, the performance levels may be associated to the displacement demand of the structure. Then, the equivalence between MDOF and SDOF system gives the lateral displacement of the SDOF system ($S_{d,O}$ $S_{d,LS}$ $S_{d,CP}$) at each performance level.

ance level. Consequently, the position of the performance point (PP) on capacity spectrum in ADRS format is defined (fig.1). This greatly simplifies the estimation of the intensity levels of the earthquake ground motion. In fact, the position of the PP gives the ductility ratio μ and ductility reduction factor R_μ without any iterative procedure. So the PGA may be increased until IDRS intersects BCS in PP. As a consequence, the problems in convergence and accuracy of the iterative graphical procedures based on the Capacity Spectrum Method are avoided.

3 DISPLACEMENT-BASED PERFORMANCE ASSESSMENT

3.1 *Study cases*

Three different steel frames are considered in the analyses (tab.1). The frames are designed according to four different design provisions: 1) New Italian Code - IC08 (2008); 2) Eurocode 8 - EC8 (2003); 3) Plastic Design – PD (Mazzolani et al., 1997); 4) Plastic Design with SLS Verification - PD-SLS. In the last approach the design ultimate displacement is increased till to satisfy the SLS verification of Italian Code (interstorey drift ratio=5‰). The design seismic action is defined with soil class A, damping ratio ξ=5%, peak ground acceleration PGA=0.25g, behavior factor q=6.5. Steel members are made from Italian S275 (f_y=275 MPa). The interstorey height is 3.5m for the first floor and 3.0m for the other floors. The bay length is 5.00 m. EC8 and IC08 gives very similar results. On the contrary, PD gives a great overstrength of the steel members. Both distributed plasticity-fiber element model and plastic hinge model implemented, respectively, in Seismostruct (SeismoSoft, 2004) and SAP2000 non linear computer programs are considered in the analyses. Sources of geometrical nonlinearity considered are both local and global. The spread of plasticity along the element derives from an inelastic cubic formulation with two Gauss points to use for numerical integration of the equilibrium equations. A bilinear model with kinematic strain-hardening of 0.5% is used for steel. The empirical method of Kato-Akiyama is used for the determination of local ductility in plastic hinge model. The plastic rotation and the residual strength are defined with FEMA 356.

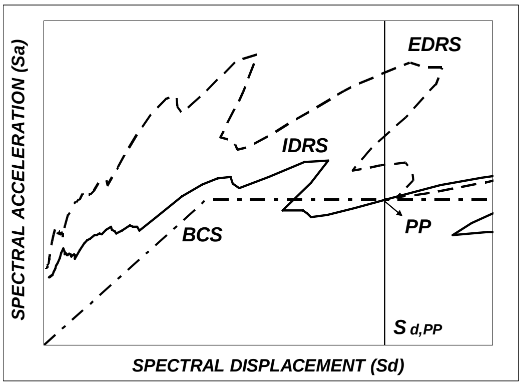
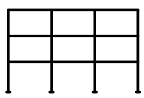
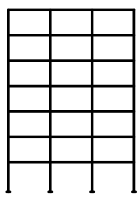
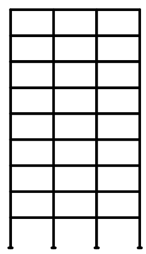


Figure 1. Non-iterative capacity spectrum method

Table 1. Study cases

Design	ELEMENT		3 STOREYS – 3 BAYS								
IC08	Level		1°	2°	3°						
	Beams		IPE270	IPE270	IPE270						
	Columns	Ext.	HE400B	HE400B	HE400B						
		Int.	HE400B	HE400B	HE400B						
PD	Level		1°	2°	3°						
	Beams		IPE300	IPE300	IPE300						
	Columns	Ext.	HE300B	HE280B	HE260B						
		Int.	HE280B	HE260B	HE240B						
PD-SLS	Level		1°	2°	3°						
	Beams		IPE300	IPE300	IPE300						
	Columns	Ext.	HE450B	HE340B	HE300B						
		Int.	HE400B	HE300B	HE260B						
EC8	Level		1°	2°	3°						
	Beams		IPE300	IPE300	IPE300						
	Columns	Ext.	HE400B	HE400B	HE400B						
		Int.	HE400B	HE400B	HE400						
7 STOREYS – 3 BAYS											
IC08	Level		1°	2°	3°	4°	5°	6°	7°	8°	9°
	Beams		IPE270	IPE270	IPE270	IPE270	IPE270	IPE270	IPE270	-	-
	Columns	Ext.	HE220B	HE220B	HE220B	HE220B	HE220B	HE220B	HE220B	-	-
		Int.	HE280B	HE280B	HE280B	HE280B	HE280B	HE280B	HE280B	-	-
PD-SLS	Level		1°	2°	3°	4°	5°	6°	7°	8°	9°
	Beams		IPE270	IPE270	IPE270	IPE270	IPE270	IPE270	IPE270	-	-
	Columns	Ext.	HE550B	HE450B	HE400B	HE400B	HE360B	HE320B	HE260B	-	-
		Int.	HE500B	HE450B	HE360B	HE360B	HE340B	HE300B	HE240B	-	-
EC8	Level		1°	2°	3°	4°	5°	6°	7°	8°	9°
	Beams		IPE270	IPE270	IPE270	IPE270	IPE270	IPE270	IPE270	-	-
	Columns	Ext.	HE160B	HE160B	HE160B	HE160B	HE160B	HE160B	HE160B	-	-
		Int.	HE240B	HE240B	HE240B	HE240B	HE240B	HE240B	HE240B	-	-
9 STOREYS – 3 BAYS											
IC08	Level		1°	2°	3°	4°	5°	6°	7°	8°	9°
	Beams		IPE270	IPE270	IPE270	IPE270	IPE270	IPE270	IPE270	IPE270	IPE270
	Columns	Ext.	HE220B	HE220B	HE220B	HE220B	HE220B	HE220B	HE220B	HE220B	HE220B
		Int.	HE280B	HE280B	HE280B	HE280B	HE280B	HE280B	HE280B	HE280B	HE280B
PD-SLS	Level		1°	2°	3°	4°	5°	6°	7°	8°	9°
	Beams		IPE270	IPE270	IPE270	IPE270	IPE270	IPE270	IPE270	IPE270	IPE270
	Columns	Ext.	HE500B	HE450B	HE450B	HE450B	HE400B	HE400B	HE400B	HE320B	HE260B
		Int.	HE500B	HE400B	HE400B	HE400B	HE400B	HE360B	HE340B	HE300B	HE240B

3.2 *Lateral load pattern effects*

For structures in which local plastic mechanism occurs the shear forces vs. story drift relationship may be very sensitive to the applied load pattern. To investigate the lateral load pattern effects six distributions are considered: 1) Uniform Distribution (UD). The lateral load distribution is proportional to the floor masses m_i. 2) First Mode Distribution (FMD). The vertical distribution is proportional to the floor masses and the shape of the fundamental mode. 3) Equivalent First Mode Distribution (EFMD). The lateral force distribution is proportional to an equivalent first mode defined from SRSS combination of sufficient modes to capture at least 90% of the total mass. 4) SRSS Distribution. The vertical distribution is proportional to the story shear distribution calculated by combining modal responses. 5) Force-based adaptive pushover (FAP); 6) Displacement-based adaptive pushover (DAP). In fig.2-5 the comparison of capacity curves is reported. The 9-storey frame designed according to EC8 or to Italian Code shows very little variation with the lateral load pattern. On the contrary, the steel frames designed with plastic design are very sensitive to the lateral load pattern. Particularly, DAP tends to overestimate the lateral strength if compared to other pushover analyses. This result derives from the higher mode contribution that in DAP analysis gives a reduction of the axial force in the external column of the first floor. Consequently, P-delta effects decrease and the plastic bending moment consequently increases.

3.3 *Static versus dynamic pushover analysis*

Sensitivity analysis of SMRFs loaded into the plastic response range, is more complicated and computationally intensive because the state of internal forces depends on the loading history. In order to verify the accuracy of non-linear static procedures and the sensitivity to input ground motion a series of FAP, DAP and conventional pushover analyses are compared with the predictions of inelastic dynamic analysis, employing a set of artificial earthquakes. Particularly, a set of 10 input ground motions is generated to be consistent to 5%-damped EC8 elastic spectrum for soil class A. In fig.2-5 the comparison between pushover analyses and Incremental Dynamic Analyses (IDA) is shown.

For the 9-storey frame designed with Italian Code (IC08) the static pushover seems to be conservative since it underestimates the capacity. On the contrary, for the frame designed with plastic design and SLS verification DAP analysis gives much greater lateral strength.

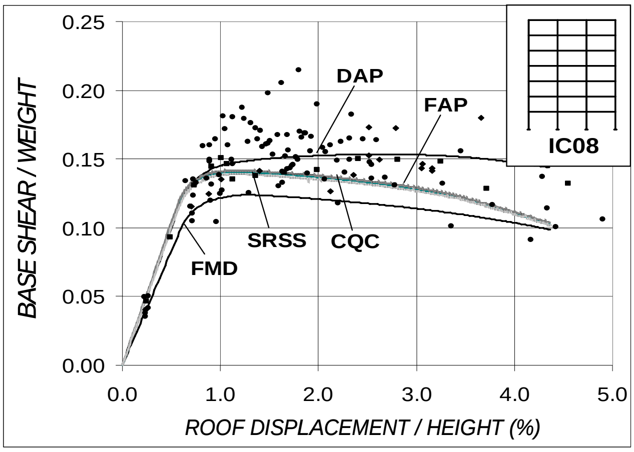


Figure 2. Base shear versus roof displacement – Pushover and Incremental Dynamic Analysis (7-storey, IC08 design)

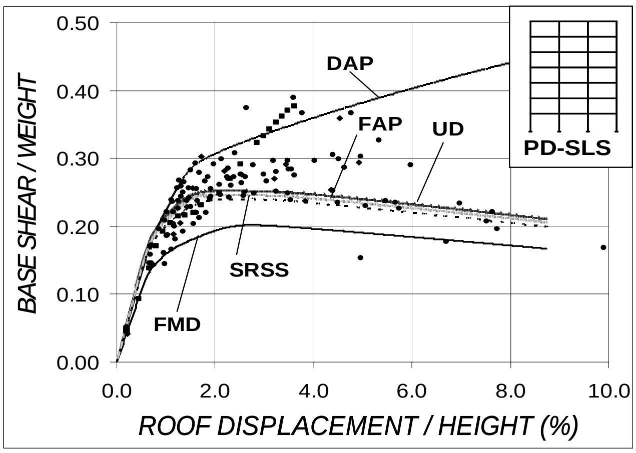


Figure 3. Base shear versus roof displacement – Pushover and Incremental Dynamic Analysis (7-storey, PD-SLS design)

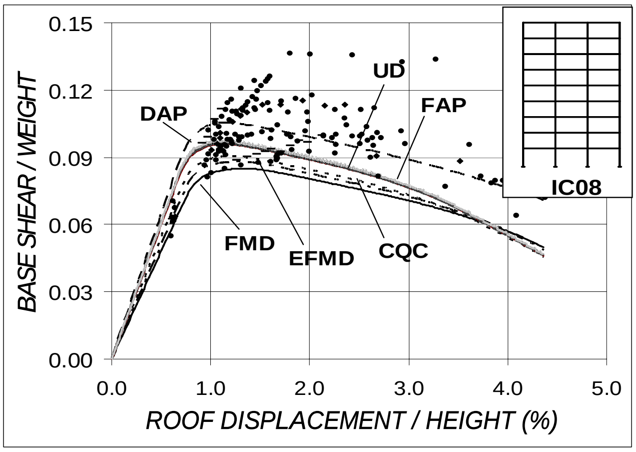


Figure 4. Base shear versus roof displacement – Pushover and Incremental Dynamic Analysis (9-storey, IC08 design)

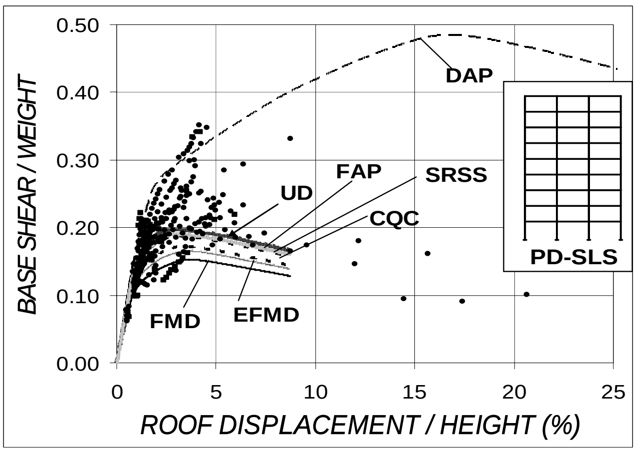


Figure 5. Base shear versus roof displacement – Pushover and Incremental Dynamic Analysis (9-storey, PD-SLS design)

3.4 *Performance-based assessment*

The displacement-based assessment is carried out for three different levels of performance (Immediate Occupancy - IO, Life Safety – LS, Collapse Prevention – CP). Two control parameters are monitored to check the acceptance criteria: 1) interstorey drift damage index (IDI); 2) plastic rotations in columns and beams. The plastic rotations are defined by tab.5.6 of FEMA 356. The limit values for the interstorey drift damage index are: 1) IDI=0.01 for IO limit state; 2) IDI=0.02 for LS limit state; 3) IDI=0.04 for CP limit state. Using the non-iterative Capacity Spectrum Method gives the peak ground accelerations corresponding to the three levels of performance (IO,LS,CP). In fig.6-9 these performance points are reported in the plane PGA versus total drift ratio (roof displacement/height). In particular, the performance points obtained from non-linear static analysis are compared with the performance points deriving from incremental dynamic analysis. In the same figures both the mean value and the maximum value of the responses obtained using the generated accelerograms are reported. The plastic design with SLS verification (PD-SLS) gives a great increase of the intensity level of the earthquake ground motion at collapse. On the contrary, the intensity levels of the earthquake ground motion corresponding to the other limit states enjoy very moderate increases.

3.5 *MDOF Effects*

The distribution of localized demands in the MDOF system can differ from those associated with the equivalent SDOF system, and the importance of this so-called "MDOF Effects" increases with the amount of inelasticity in the structure and with the occurrence of local plastic mechanism.
In order to investigate these effects the interstorey drift profiles obtained from pushover analyses are compared to the drift profiles from nonlinear time-history analysis (fig.10).
The drift profile from pushover analysis are referred to the CP limit state. The drift profile from dynamic analysis are referred to the input ground motion corresponding to the maximum total drift ratio.

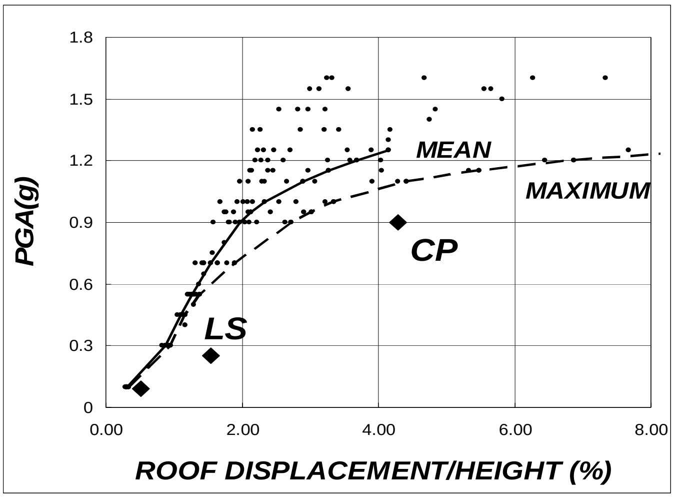


Figure 6. Base shear versus roof displacement – Pushover and Incremental Dynamic Analysis (7-storey, IC08 design)

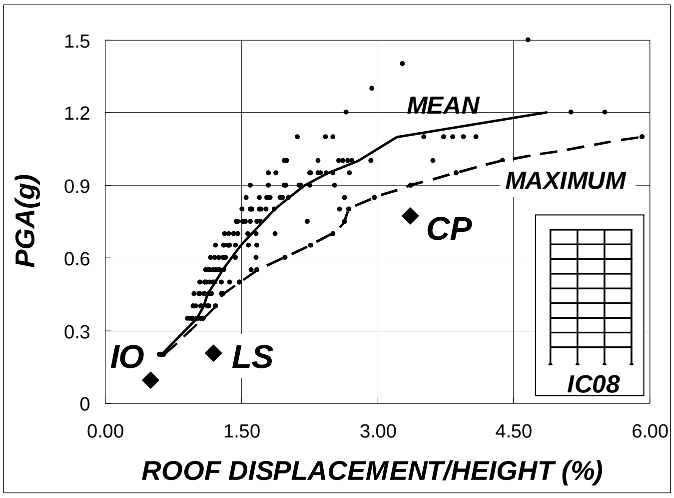


Figure 8. Base shear versus roof displacement – Pushover and Incremental Dynamic Analysis (9-storey, IC08 design)

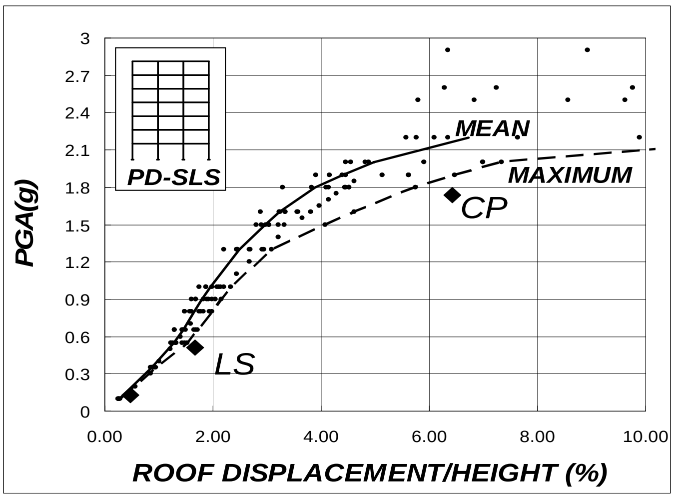


Figure 7. Base shear versus roof displacement – Pushover and Incremental Dynamic Analysis (7-storey, PD-SLS design)

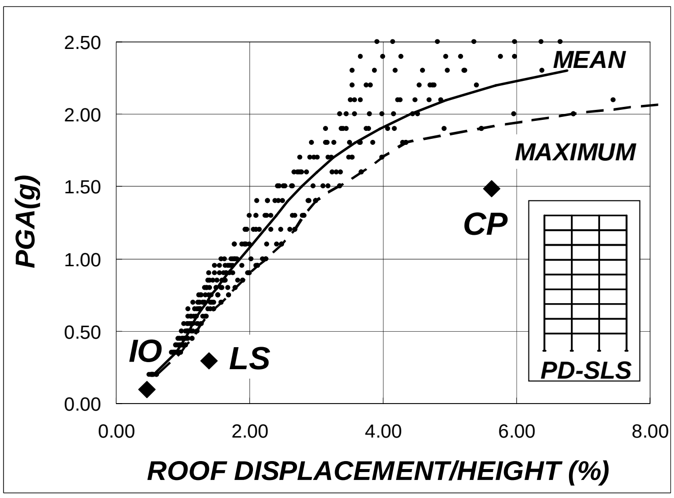


Figure 9. Base shear versus roof displacement – Pushover and Incremental Dynamic Analysis (9-storey, PD-SLS design)

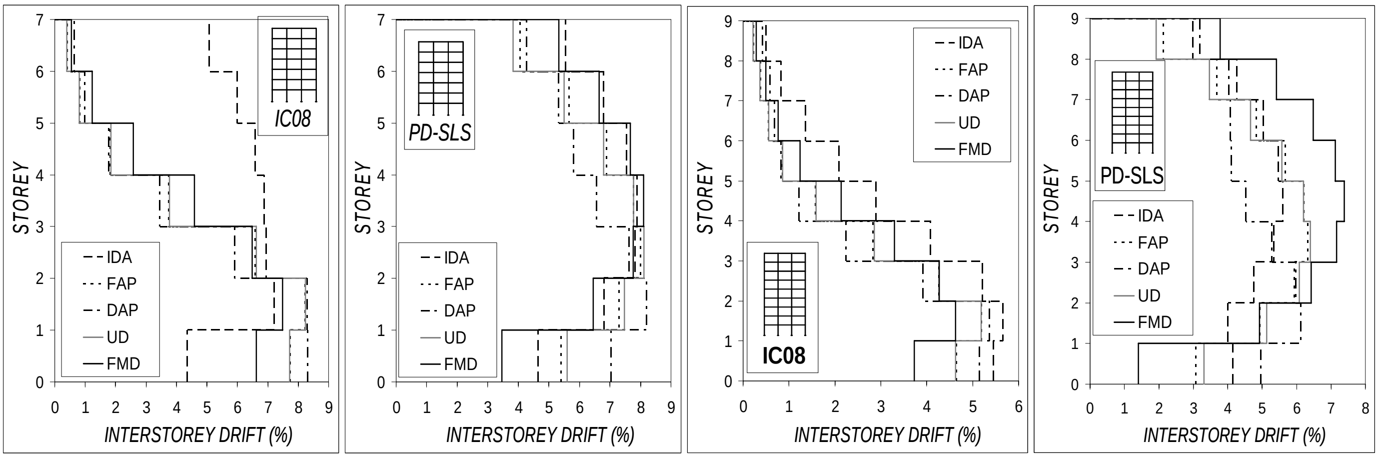


Figure 10. Comparison of drift profiles from Pushover Analyses with the maximum interstorey drift from Dynamic Analysis

This earthquake is scaled to have the same roof displacement obtained from pushover analysis at the collapse prevention limit state. The results obtained show that the DAP analysis systematically understimates the interstorey drifts if compared to dynamic analysis, while the FMD distribution seems to be more effective. All the nonlinear static procedures understimates the interstorey drifts of the 7-storey frame designed with the Italian Code. This result derives from the high strength required to the roof columns by the capacity design criteria.

4 CONCLUSIONS

The comparative evaluation of design procedures confirmed that also recent codes based on the local ductility condition and the capacity design rule are not able to assure the suitable plastic mechanism. On the other side, the results demonstrated the effectiveness of the plastic design in governing the collapse mechanism. The design procedure based on the control both of collapse mechanism and of lateral displacement require an iterative design process to avoid that the increase of structural overstrength produced by the damage limit state provisions leads to undesired collapse mechanisms. Furthermore, their cost is often an uncontrollable overstrength of the structure. However, the same safety factor for the other limit states (Immediate Occupancy and Life Safety) is not guaranteed. Finally, the overstrength generally increases the seismic demand and so reduces the benefits deriving from the control of collapse mechanism and lateral displacement.

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