DISPLACEMENT-BASED SEISMIC ASSESSMENT OF STEEL MOMENT RESISTING FRAME STRUCTURES

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ABSTRACT :

The primary focus of this study is on the development of a detailed assessment of the inelastic seismic behavior, response, and performance of typical ductile SMRF structures. At this aim, a multi-objective and multi-criteria performance evaluation of steel moment-frame buildings was realized with a combination of the performance criteria both for structural members and for non-structural components at the different limit state. Analytical models of various complexities are evaluated using nonlinear static pushover analysis and incremental dynamic analysis, to evaluate the potential for collapse in flexible SMRF structures due to the P-delta effect. In particular, as an alternative to incremental response history analysis, an incremental non-iterative nonlinear static procedure based on adaptive capacity spectra method was used for the displacement-based seismic assessment.

KEYWORDS: Steel frames, Adaptive pushover, Seismic Performance.

1. INTRODUCTION

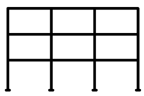
The steel moment resisting frames (SMRFs) are expected to be able to sustain large plastic deformations in bending and shear. However, structural damage and collapses during recent earthquakes have evidenced some critical aspects in the seismic behavior of steel structures even when designed according to the current design codes. The main limit of traditional design provisions is that the performance cannot be predicted because the seismic behavior of the structure is governed by phenomena which are not adequately captured in the simple design process. In particular, the formation of local plastic mechanism could not be avoided and the safety factor could change with the design level. On the other side, the ductility and the capacity design criteria 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. More advanced design procedures based on the second order plastic analysis proved to be effective to ensure a global plastic mechanism. However, these procedures require a great overstrength of steel members. Furthermore, the design strength of the structure is independent by the intensity level of the earthquake ground motion. Finally, the ultimate limit state verification is not sufficient to ensure the verification at the other limit states. The primary focus of this study is on the development of a detailed assessment of the inelastic seismic behavior, response, and performance of typical ductile SMRF structures. The objective is to estimate the effectiveness of design procedures in order to have the desired seismic performance at each intensity level of the input ground motion. Such objectives cannot be pursued through the comparison between local capacity and seismic demand. 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. The information generated through nonlinear dynamic analysis will form the basis for a simplified seismic demand estimation procedure in which the spectral displacement of the ground motion is related to the inelastic deformation demands for the structure.

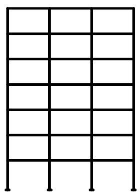
2. DESIGN OF SEISMIC RESISTANT STEEL FRAMES

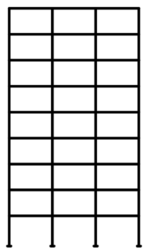
The aim of the conventional design approach is to satisfy the two requirements that a structural system must meet: a) under ordinary actions the structure must be stiff in order to minimize structural and non-structural damage; b) under severe earthquakes the structure must be safe from collapse even if some structural damage may be tolerable. The serviceability limit state (SLS) and the ultimate limit state (ULS) are checked with a linear elastic analysis. Undesirable brittle failure mechanisms are usually avoided with local ductility requirements and capacity design rule. Finally, full-strength connections concur to avoid the failure of beam-column joints. Three different approaches are generally used to apply the capacity design rule: 1) amplification of the acting moment in the seismic design situation (ECCS 1988; New Italian Code 2008); 2) check of hierarchy criterion after analysis (EC8 2003; UBC 1997 and AISC 1997; Lee 1996); 3) Plastic design (Mazzolani-Piluso 1997) based on the application of the kinematic theorem of plastic collapse. In this case, the beam section are designed under gravity loads. The column sections are determined by means of design conditions deriving from kinematic theorem of plastic collapse. Second-order effects are considered by means of the equilibrium curve of the collapse mechanism. In this paper, the plastic design method is modified increasing the design ultimate displacement (and, therefore, the ultimate plastic rotation capacity of the beams) till to satisfy the SLS verification of Italian Code (interstorey drift $d_{lim} \leq 0.005h$). Three different steel frames are considered in the analyses. Each frame is designed according to: 1) New Italian Code (IC08); 2) Eurocode 8 (EC8); 3) Plastic Design (PD); 4) Plastic Design with SLS Verification (PD-SLS). The design seismic action is defined with soil class A, damping ratio ξ=5 per cent, peak ground acceleration PGA=0.25g, behavior factor q=6.5. Steel members are made from Fe 430 steel. The interstorey height is 3.50 m for the first floor and 3.00 m for the other floors. The bay length is 5.00 m. In tab.2.1 the sections obtained with the different design procedures are reported. EC8 and IC08 gives very similar results. On the contrary PD gives a great overstrength of the steel members.

Table 2.1 Steel Frames and Design Sections

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. ADAPTIVE CAPACITY SPECTRUM METHOD

3.1. Adaptive and non-adaptive pushover analysis

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. Since such forces depend on the response history of the building, the lateral load pattern should be modified during the analysis as an effect of yielding. In other words, as the damage progresses, the structure changes its response from the original dynamic amplification, well represented by a modal force distribution. Therefore, deformation estimates obtained from a pushover analysis may be very inaccurate for structures in which higher mode effects are significant, local plastic mechanism occurs or shear forces vs. story drift relationships are sensitive to the applied load pattern. In fact, none of the invariant force distributions account for the higher modes contribution to the response, the redistribution of inertia forces because of structural yielding and the associated changes in the vibration properties of the structure. In this case, the distribution of localized demands in the MDOF system (story drifts and forces or component deformations) can differ from those associated with the equivalent SDOF system. The importance of this so-called "MDOF Effects" increases with the amount of inelasticity in the structure. To overcome these limitations, 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. Although response in each mode may potentially be nonlinear, the mode shapes and lateral force profiles are assumed invariant. Lateral load patterns based on the first three modes of vibration are used. Target displacement values are computed by applying equivalent nonlinear procedures with a SDOF system representative of each modal load pattern. Finally, response quantities, obtained from each modal pushover, are combined using 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 either the Square Root of the Sum of the Squares (SRSS), if the modes can be assumed as fully uncoupled, the Complete Quadratic Combination (CQC) method, if cross-coupling of the modes and respective viscous damping is to be considered. 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 through a SRSS combination weighted with the spectral displacement is considered. Alternative modal combinations without removing any sign such as the Direct Vectorial Addition (DVA) are proposed in literature.

3.2. Adaptive Capacity Spectrum Method

The result of the analysis is the pushover curve, which plot a deformation index (typically roof displacement d_{TOP}) against a force index (typically base shear V). This capacity curve (CC) is the starting point for all the NSPs 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 δ_j^i, as follows:

$$M_{eq}^i = \frac{\left(\sum_{j=1}^{N} m_j \cdot \delta_j^i\right)^2}{\sum_{j=1}^{N} m_j \cdot \delta_j^{i\,2}} \qquad K_{eq}^i = \frac{\left(\sum_{j=1}^{N} m_j \cdot \delta_j^i\right)^2}{\left(\sum_{j=1}^{N} m_j \cdot \delta_j^{i\,2}\right)^2} \sum_{j=1}^{N} F_j^i \cdot \delta_j^i \tag{3.1}$$

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:

$$\Delta S_a^i = \Delta V^i \cdot \frac{\sum_{j=1}^{N} m_j \cdot \delta_j^{i\,2}}{\left(\sum_{j=1}^{N} m_j \cdot \delta_j^i\right)^2} \qquad \Delta S_d^i = \Delta d_{TOP}^i \frac{1}{\delta_N^i} \frac{\sum_{j=1}^{N} m_j \cdot \delta_j^{i\,2}}{\sum_{j=1}^{N} m_j \cdot \delta_j^i} \tag{3.2}$$

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). In this paper the IDRS are not directly obtained through the nonlinear time-history analysis of the equivalent bilinear SDOF system, but they are indirectly computed scaling the 5%-damped Elastic Demand Response Spectra (EDRS) by means of ductility reduction factor R_μ. In particular, the inelastic pseudo-acceleration S_a and displacement S_d - which represents the coordinates of the IDRS in ADRS format - are characterized from the coordinates [S_{de};S_{ae}] of the EDRS, as follows:

$$S_a = \frac{S_{ae}}{R_\mu} \qquad S_d = \frac{\mu \cdot S_{de}}{R_\mu} \tag{3.3}$$

A great number of reduction rules are available in literature. Usually the reduction factor R_μ is an explicit function both of structural period and of characteristic periods of the earthquake. In this paper, a reduction factor depending on velocity and displacement elastic spectra was adopted. Starting from the reduction rule proposed by Ordaz et al. (1998) the following expression of the strength reduction factor is used:

$$R_\mu = 1 + \left(\frac{S_v(T)}{PGV}\right)^{\alpha(\mu)} \cdot \left(\frac{S_v(T)}{PGV}\right)^{\beta(\mu)} \cdot (\mu - 1) \tag{3.4}$$

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 which have to be obtained with a statistical data analysis. For seismic inputs consistent to Eurocode 8 type 1 elastic response spectrum for firm soil (class A) a very good fitting was found for the following functions (Ferraioli et al., 2004):

$$\alpha(\mu) = -0.1967 \cdot \log(\mu) + 0.454 \qquad \beta(\mu) = 0.2314 \cdot \log(\mu) - 0.0071 \tag{3.5}$$

The reduction factor R_μ depends from the ductility μ and, therefore, from the lateral displacement of the equivalent SDOF system. Consequently, an iterative graphic procedure is usually required to obtain the intersection between demand and capacity that is between IDRS and BCS. The intersection between the elastic stiffness of the equivalent system and the EDRS spectrum is the strength demand so that the structure remains in elastic range. If the EDRS spectrum intersects the BCS over the yielding point, then the structure behave inelastically under the input ground motion. In this case, eqs.3.3 gives the coordinates of EDRS spectrum from IDRS spectrum. However, the displacement reduction factor R_μ and the strength reduction factor R_μ/μ depend from the position of the point of performance (PP). Consequently, an iterative procedure is usually required in order to estimate the intersection between IDRS and BCS. Applied for the displacement-based assessment the adaptive capacity spectrum method may give some advantages. In fact, the acceptance criteria for performance-based assessment are usually based on the interstorey drift damage index (IDI) and the plastic rotations. As a consequence, the equivalence between MDOF and SDOF system gives the lateral displacement of the SDOF system at each performance level. Consequently, the position of the point of performance (PP) on capacity spectrum in ADRS format is defined. 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. As a consequence, the problems in convergence and accuracy of the iterative graphical procedures based on the Capacity Spectrum Method are avoided.

4. DISPLACEMENT-BASED SEISMIC ASSESSMENT

The building codes generally use strength as the main design criterion and they consider the lateral force procedure at the base of the earthquake resistant design. The displacement control usually plays a secondary role, and the deformation demand is generally checked at the end of the design process for the serviceability limit state. However, recent earthquakes have shown that structures may suffer irreparable or too costly to repair damages. Furthermore, inelastic behavior, indicating damage, is observed even during smaller earthquakes. As a consequence, the modern seismic design requires the application of performance-based concepts. In this way, multi-level objectives may be pursued and structures that perform appropriately for all earthquakes may be obtained. In this context, building codes in the United States and Japan are rapidly moving towards the adoption of performance based design procedures for earthquake resistance. The displacement-based design (DBD) and the performance-based design (PBD) seem to be more promising than the traditional force-based design (FBD). As a consequence, the fundamental design parameters are the displacements and the ductility demands, and the most suitable approach in modern seismic design is to ensure that these parameters will not be exceeded under the design-level earthquake ground motion. The defined level of damage during a specified earthquake ground motion should be ensured by performance-based criteria. These criteria should be selected such that at the specified levels of ground motion and with defined levels of reliability, the structure will not be damaged beyond certain limiting states. The PBD approach requires the execution of non-linear analysis. In this paper, 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. The model takes into account geometrical nonlinearity and material inelasticity. Sources of geometrical nonlinearity considered are both local (beam-column effect) and global (large displacement/rotation effects). The distributed plasticity fiber element model accounts for the spread of plasticity along the element and inside the cross section and so it allows for the accurate estimation of structural damage distribution. The sectional stress-strain state or beam-column elements is obtained through the integration of the nonlinear uniaxial stress-strain response of the fibers in which the section has been subdivided. A bilinear model with kinematic strain-hardening of 0.5% is used for steel. 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. Relatively short elements (four per structural member) are considered in the inelastic model. In this way, the numeric accuracy problems derived from the constant generalized axial strain shape function are avoided. The empirical method of Kato-Akiyama is used for the determination of local ductility in plastic hinge model. According to this method the rotational capacity of the members is described by means of a trilinear moment-rotation curve with parameters calculated taking into account the axial load of the member, the slenderness of the web and the flange, the flexural-buckling phenomena. The values of plastic rotation and residual strength are defined by FEMA 356 as a function of geometric and mechanical characteristics of steel members. According to FEMA 356 the modeling of nodal panel is avoided in the hypothesis that: 1) the expected shear strength of panel zones exceeds the flexural strength of the beams at a beam-column connection; 2) the stiffness of the panel zone is at least 10 times larger than the flexural stiffness of the beam. First of all a comparative evaluation of design procedures considering both building code provisions both innovative methods based on the limit analysis or on the second order plastic analysis was carried out. For the steel frames designed with the capacity design rule both plastic hinge formation in columns and undesirable story mechanism are not completely avoided. In fig.1 the comparison of capacity curves obtained with distributed plasticity fiber element model and plastic hinge model under first mode distribution of lateral load is reported. Results show that the plastic hinge model generally underestimates the lateral strength. However, the two models give very similar results. Then, six distributions of lateral forces 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, including sufficient modes to capture at least 90% of the total building mass. 5) Force-based adaptive pushover (FAP); 6) Displacement-based adaptive pushover (DAP). In fig.2 the comparison of capacity curves obtained with different lateral load patterns 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.

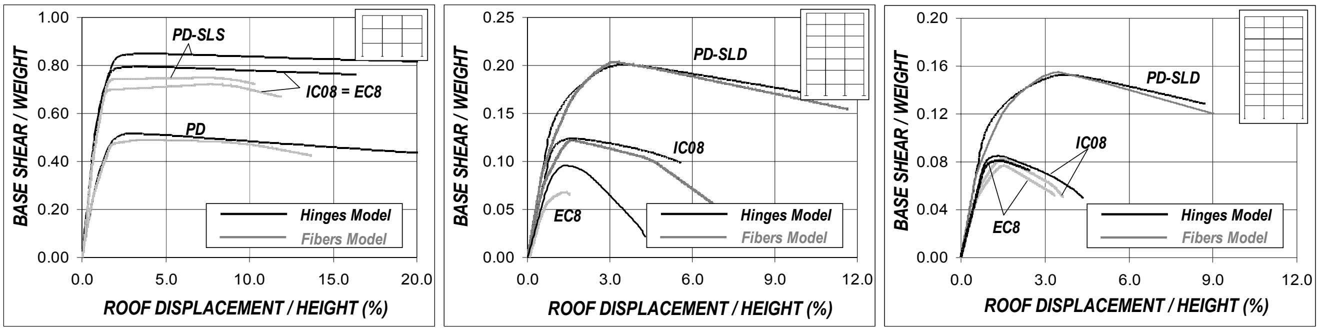


Figure 1 Base shear versus roof displacement curves – Variation with model

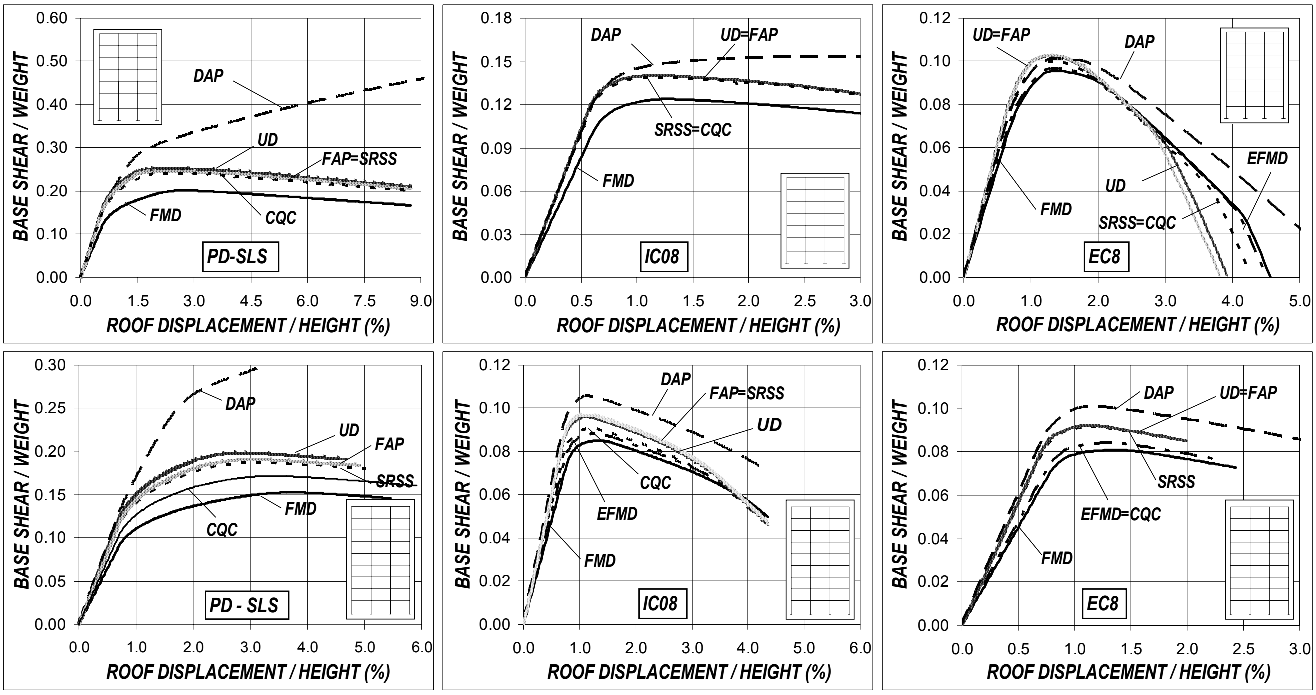


Figure 2 Base shear versus roof displacement curves – Adaptive and non adaptive lateral load pattern

Sensitivity analysis of steel moment frames 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 large set of artificial earthquakes. Particularly, a set of 10 input ground motions is generated to be consistent to 5%-damped Eurocode 8 elastic spectrum for soil class A. In fig.3 the comparison between pushover analyses and Incremental Dynamic Analyses (IDA) is shown. For the 9-storey frame designed with New Italian Code (IC08) the static pushover seems to be conservative since it understimates the capacity. On the contrary, for the frame designed with plastic design and SLS verification DAP analysis gives much greater lateral strength.

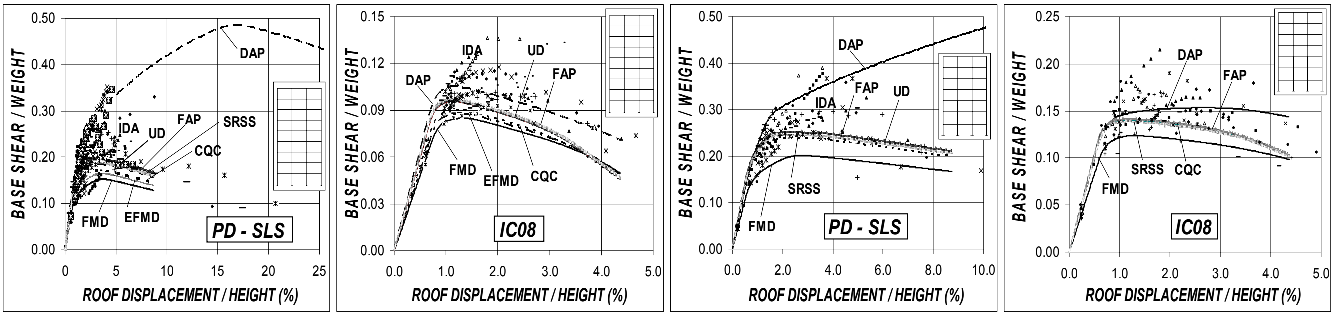


Figure 3 Base shear versus roof displacement curves – Pushover Analyses and Incremental Dynamic Analysis

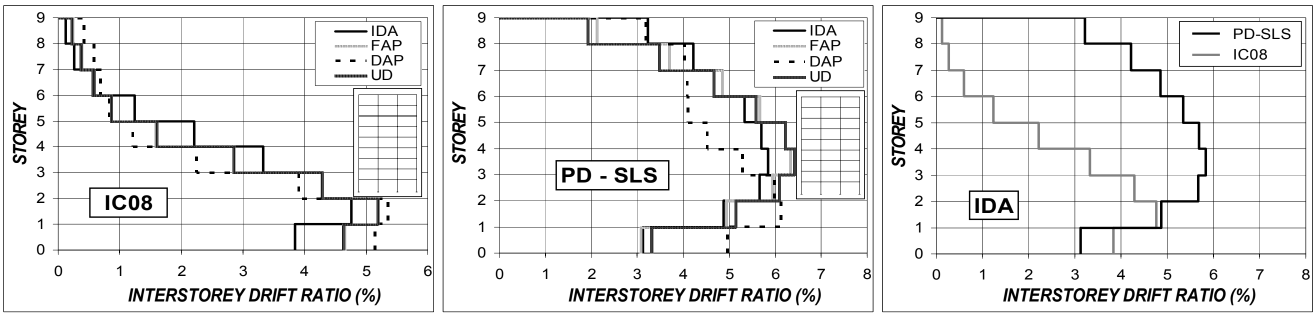


Figure 4 Interstorey drift profile – Pushover Analyses and Incremental Dynamic Analysis

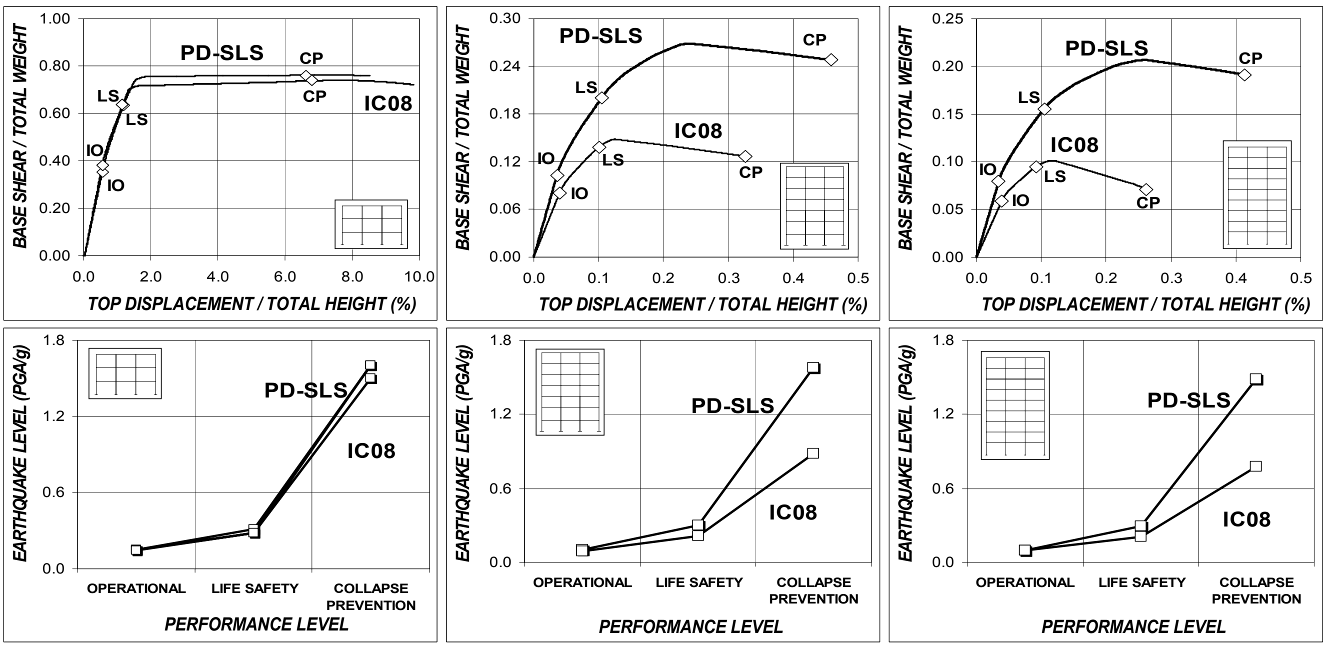


Figure 5 Earthquake intensity level versus performance level – Performance curves and performance matrix

In fig.4 the interstorey drift profiles obtained from static and dynamic pushover are compared. The results of IDA are referred to the 10 input ground motions scaled to have the same roof displacement. The mean value of the peak interstorey drifts is considered. The results of static pushover are referred to the step of analysis where the static roof displacement and the dynamic roof displacement are equal. The displacement-based assessment is carried out with three different levels of performance (Immediate Occupancy - IO, Life Safety – LS, Collapse Prevention – CP). Two control parameters are monitored to check the attainment of the different performance levels of the building (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. In fig.5 a comparative evaluation of design procedures in terms of performance curves and performance matrices is carried out with an incremental analysis. The plastic design with SLS verification (PL-SLS) gives a great increase of the safety factor at the different limit states (41% for IO, 64% for LS and 80% for CP).

5. CONCLUSIONS

A displacement-based comparative evaluation of design procedures considering both building code provisions both innovative methods based on the limit analysis or on the second order plastic analysis was carried out. At this aim, a multi-objective and multi-criteria performance evaluation of steel moment-frame buildings was realized with a combination of the performance criteria both for structural and for non-structural components at the different limit states. Analytical models of various complexities are evaluated using adaptive pushover analysis and incremental dynamic analysis. An adaptive version of capacity spectrum method based on inelastic demand spectra was used for the performance assessment of typical steel frames. The comparative evaluation confirmed the effectiveness of plastic design in governing the collapse mechanism, while the seismic codes were not able to ensure the suitable plastic mechanism. However, an iterative design process was required to avoid that the increase of structural overstrength produced by the damage limit state provisions leads to undesired collapse mechanisms. This often leads to a boundless increase of area sections. Finally, the displacement-based adaptive pushover may overestimate the lateral strength and underestimate the interstorey drift of intermediate floors.

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