Accuracy of nonlinear static procedures for estimating the seismic performance of steel frame structures

M. Ferraioli, A. Lavino, A.M. Avossa, A. Mandara

Seconda Università di Napoli, Dipartimento di Ingegneria Civile, via Roma 29 Aversa (CE), Italy

ABSTRACT: The effectiveness of nonlinear static procedures for estimating the seismic performance of steel frame structures was investigated. At this aim, the procedures proposed in FEMA 356, ASCE/SEI 41-04, ATC40, FEMA440-ATC-55 were applied for the displacement-based performance assessment. The results obtained were compared with incremental time-history response.

1 INTRODUCTION

The validity and applicability of the static pushover analysis have been extensively studied in literature. In particular, many researchers have compared the pushover curves with idealized envelopes obtained from incremental dynamic pushover analyses of structures subjected to artificial and natural input ground motions. Furthermore, the effectiveness of the Capacity Spectrum Method (CSM) has been validated by comparison with experimental results from pseudo-dynamic, cyclic and pushover tests. The Capacity Spectrum Method - by means of a graphical procedure - compares the capacity of the structure to resist lateral forces to the demands of earthquake response spectra. Different Nonlinear Static Procedures (NSPs) based on the CSM have been introduced in pre-standards reports and guidelines. Some of them were incorporated in the new generation of seismic codes to determine the deformation demand imposed on a building expected to behave inelastically.

A simplified procedure based on the pushover analysis and on the strength reduction factor was introduced in the Eurocode 8 (2003). Static Displacement-Based Procedures were developed by the Federal Emergency Management Agency (FEMA).

The Coefficient Method (CM) of FEMA-356 (2000) is based on a displacement modification procedure in which several empirically derived factors are used to modify the response of a linearly-elastic, single-degree-of-freedom (SDOF) model of the structure. In other words, the CM is a form of equivalent linearization.

Empirically derived relationships, that gives the effective period and damping as a function of ductility, are used to estimate the response of an equivalent nonlinear SDOF system.

ATC-40 Report (ATC-40, 1997) proposes three nonlinear static procedures (A,B,C) based on the Capacity Spectrum Method and on the High Damping Elastic Response Spectra (HDERS) with an equivalent viscous damping accounting for the energy dissipation due to yielding.

However several deficiencies were found by some authors. In particular, no physical principle justifies the existence of a stable relationship between the hysteretic energy dissipation and equivalent viscous damping, particularly for highly inelastic systems, and so the convergence of ATC iterative procedures is not assured. Moreover, the ATC-40 procedures require the calculation of the characteristic periods of earthquake spectrum which often are not easy to be estimated.

The N2 method proposed by Fajfar (2001) combines together the visual representation of the CSM and the superior physical basis of Inelastic Demand Response Spectra (IDRS) which are expected to be more accurate than HDERS, especially in the short period range and in the case of high ductility factors. Both CM and CSM were found to provide substantially different estimates of target displacement for the same ground motion and the same building (Aschheim et al., 1998; Akkar and Metin, 2007; Chopra and Goel, 2000; Goel, 2007; Miranda and Luiz-Garcia, 2002) and have proposed improved procedures for estimating the target displacement. Recently, FEMA-440 document (ATC-55, 2003) re-examined the existing NSPs and proposed improvements to both the CM and CSM. Recommendations in the FEMA-440 document have been adopted in the ASCE/SEI 41-06 standard (ASCE, 2007).

2 NON LINEAR STATIC PROCEDURES

The aforementioned nonlinear static procedures (FEMA-356, ATC-40, FEMA-440, ASCE/SEI 41-06, CSM-N2) usually require the pushover analysis. The structure is pushed statically to a target displacement at the control node to check for the acceptable structural performance. Different height-wise distributions of lateral loads are usually considered. In fact, the shear forces vs. story drift relationship may be very sensitive to the applied load pattern. Generally, different distributions of lateral load are considered. 1) Equivalent lateral force distribution (ELFD). The seismic forces at each floor level of the building are distributed according to the distribution of floor mass m_i and to the height h_i above the base. 2) Fundamental Mode Distribution (FMD). The vertical distribution is proportional to the floor masses and the fundamental mode shape.. 3) Response Spectral Analysis distribution (RSA): the vector of lateral forces is proportional to the story shear distribution calculated by combining the modal responses of sufficient modes to capture at least 90% of the total mass. 4) Uniform Distribution (UD). The lateral load distribution is proportional to the floor masses m_i. FEMA-356 requires development of the pushover curve for two height-wise distributions of lateral forces: UD and one selected from ELFD, FMD and RSA. ATC-40, FEMA-440, and ASCE/SEI 41-06 require only the fundamental mode distribution.

2.1 *FEMA-356 - Coefficient Method*

The Coefficient Method of FEMA 356 (ASCE, 2000) is based on an equivalent linearization. In particular, the maximum inelastic displacement (target displacement) is obtained from the linear elastic response of the equivalent SDOF system, as follows:

$$\delta_t = C_0 C_1 C_2 C_3 S_a \frac{T_e^2}{4\pi^2} g \tag{1}$$

S_a is the response spectrum acceleration at the effective fundamental vibration period and damping ratio of the building under consideration. g is the acceleration due to gravity. T_e is the effective fundamental period of the building in the direction under consideration, computed as follows:

$$T_e = T_i \sqrt{K_i / K_e} \tag{2}$$

where T_i is the fundamental vibration period defined from elastic eigenvalue analysis; K_i is the elastic stiffness of the building; K_e is the effective stiffness of the building obtained by idealizing the pushover curve as a bilinear relationship. The coefficient C_0 is the modification factor that relates the elastic response of a SDOF system to the elastic displacement of the MDOF building. The control node of the building may be defined from the first mode participation factor PF or selected from tabulated values in FEMA-356. C_1 is the modification factor that relates the maximum inelastic and elastic displacement of the SDOF system computed from

$$C_1 = \begin{cases} 1.0; & T_e \geq T_s \\ \left(1.0 + (R-1)\, T_s / T_e\right)/R\ ; & T_e < T_s \\ 1.5; & T_e < 0.1s \end{cases} \tag{3}$$

where R is the ratio of elastic and yield strengths and T_s is the corner period where the response spectrum transitions from constant pseudo-acceleration to constant pseudo-velocity.

The modification factor C_2 represents the effects of pinched hysteretic shape, stiffness degradation, and strength deterioration. C_2=1 for nonlinear analysis and tabulated as a function of framing system and performance level otherwise.

The modification factor C_3 accounts for the increased displacement due to P-delta effects, as follows:

$$C_3 = \begin{cases} 1.0; & \alpha \geq 0 \\ \left(1.0 + |\alpha| (R-1)^{3/2} / T_e\right); & \alpha < 0 \end{cases} \tag{4}$$

in which α is the ratio of the post-yield stiffness to effective elastic stiffness.

2.2 *ATC-40 - Capacity Spectrum Method*

The ATC-40 procedures are based on CSM Method. The basic assumption is the equivalent linearization by which the maximum inelastic deformation of a nonlinear SDOF system (target displacement) is approximated from the maximum deformation of a linear elastic SDOF system with equivalent period T_{eq} and equivalent damping ratio ξ_{eq}, expressed as follows:

$$T_{eq} = T_0 \sqrt{\frac{\mu}{1+\alpha\mu-\alpha}}\ ;\ \xi_{eq} = \xi_0 + \kappa \frac{1}{\pi} \frac{(\mu-1)(1-\alpha)}{\mu(1+\alpha\mu-\alpha)} \tag{5}$$

where T_0 is the initial period of vibration of the nonlinear system; α is the post-yield stiffness ratio; μ is the maximum displacement ductility ratio; κ is the adjustment factor to approximately account for changes in hysteretic behavior. In particular, the ATC-40 document defines three types of hysteretic behaviors: Type A with stable, reasonably full hysteretic loops. Type C with severely pinched and/or degraded loops. Type B between Types A and C. The equivalent linearization procedure requires prior knowledge of the displacement ductility ratio. ATC-40 provides three iterative procedures: (A, B, C). The procedures A and B are the most transparent and convenient for programming, whereas the Procedure C is purely a graphical method that is not suitable for programming.

2.3 *FEMA-440 - Coefficient Method*

FEMA 440 introduces some improvements in Coefficient Method of FEMA-356. In particular, the coefficient C_1 is given by

$$C_1 = \begin{cases} 1.0 + \dfrac{R-1}{0.04\,a}; & T_e < 0.2s \\ 1.0; & T_e > 1.0s \\ 1.0 + \dfrac{R-1}{a\,T_e^2}; & 0.2s < T_e < 1.0s \end{cases} \tag{6}$$

where a=130 for site class A and B, 90 for site class C, a=60 for site classes D, E, and F. The coefficient C_2 is given by

$$C_2 = \begin{cases} 1.0; & T_e > 0.7s \\ 1.0 + \dfrac{1}{800}\left(\dfrac{R-1}{T_e}\right)^2; & T_e \le 0.7s \end{cases} \tag{7}$$

Finally, the coefficient C_3 is eliminated and substituted by a limitation on strength to avoid dynamic instability. This limitation on strength is specified by imposing a maximum limit on R given by

$$R_{max} = \frac{\Delta_d}{\Delta_y} + \frac{|\alpha_e|^{-h}}{4} \; ; \; h = 1.0 + 0.15\,ln\left(T_e\right) \tag{8}$$

In eq.8 Δ_d is the deformation corresponding to peak strength, Δ_y is the yield deformation, and α_e is the effective negative post-yield slope given by

$$\alpha_e = \alpha_{P-\Delta} + \lambda\left(\alpha_2 - \alpha_{P-\Delta}\right) \tag{9}$$

Where α_2 is the negative post-yield slope ratio; $\alpha_{P-\Delta}$ is the negative slope ratio caused by P-Δ effects; λ is the near-field effect given: λ=0.8 for $S_1 \ge 0.6$ and λ=0.2 for $S_1 < 0.6$ (S_1 is defined as the 1-second spectral acceleration for the Maximum Considered Earthquake). The α_2 slope includes P-Δ effects, in-cycle degradation, and cyclic degradation.

2.4 *FEMA-440 - Capacity Spectrum Method*

The improved FEMA-440 CSM includes new expressions to determine the effective period and effective damping developed by Guyader and Iwan (2006). Consistent with the original ATC-40 procedure, three iterative procedures for estimating the target displacement are also outlined. Finally, a limitation on the strength is imposed to avoid dynamic instability. Improved formulas for the effective period and viscous damping are proposed as a function of several coefficients tabulated in Table 6-1 and Table 6-2 of FEMA-440 document. The equivalent linearization procedures applied in practice normally require the use of spectral reduction factors to adjust an initial response spectrum to the appropriate level of effective damping β_{eff}. This factor $B(\beta_{eff})$ is a function of the effective damping and is used to adjust the spectral acceleration ordinates as follows:

$$\left(S_a\right)_\beta = \left(S_a\right)_\beta / B\left(\beta_{eff}\right) \tag{10}$$

where the damping coefficient $B(\beta_{eff})$ is given by:

$$B = \frac{4}{5.6 - \ln \beta_{eff}} \tag{11}$$

2.5 *Capacity Spectrum Method -N2*

The CSM-N2 procedure is based on the Capacity Spectrum Method and the Inelastic Demand Spectra. The inelastic seismic demand is not represented by High Damping Elastic Response Spectra (HDERS), but it is represented by means of the Inelastic Demand Response Spectra (IDRS). The IDRS is not directly obtained through the nonlinear time-history analysis of the equivalent bilinear SDOF system, but it is indirectly computed scaling the Elastic Demand Response Spectra (EDRS). The IDRS is obtained by scaling the EDRS (with viscous damping ratio ξ=5 per cent) by means a 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 (ξ=5 per cent) as follows:

$$S_a = \frac{S_{ae}}{R_\mu} \qquad S_d = \frac{\mu \cdot S_{de}}{R_\mu} \tag{12}$$

In eq.12 μ is the ductility ratio and the ductility reduction factor R_μ is defined by the following espressions derived from statistical data analysis (Vidic et el. 1994):

$$R_\mu = \left(\mu - 1\right)T/T_0 + 1 \qquad T \le T_0 \tag{13}$$

$$R_\mu = \mu \quad \text{for } T \ge T_0 \text{ with } \; T_0 = 0.65\mu^{0.3} \cdot T_c \le T_c \tag{14}$$

where T_c is the corner period of the earthquake ground motion between the constant acceleration region and the constant velocity region of Newmark-Hall response spectrum.

3 ASSESSMENT OF CURRENT NONLINEAR STATIC PROCEDURES FOR PERFORMANCE EVALUATION OF BUILDINGS

3.1 *Study cases*

A 9-storey steel frame designed according to the Italian Code (2008) and Plastic Design (Mazzolani et al., 1997) are considered in the analyses. 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 steel (f_y=275 MPa). 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. The design sections of the two steel frames are reported in tab.1. In tab. 2 the modal properties are shown. A distributed plasticity-fiber element model implemented in Seismostruct (SeismoSoft, 2004) is 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 plastic rotation and the residual strength are defined according to FEMA 356. The following current nonlinear static procedures are considered in the analysis: 1) ATC 40 under first mode distribution (FMD); 2) ATC 40 under uniform distribution (UD); 3) FEMA 356 Coefficient Method (CM); 4) FEMA 440 Coefficient Method (CM); 5) Capacity Spectrum Method (CSM-N2) under first mode distribution; 6) Capacity Spectrum Method (CSM–N2) under uniform distribution; 7) FEMA 440 (CSM) under first mode distribution; 8) FEMA 440 (CSM) under uniform distribution. The NSPs are applied for the performance-based assessment of IC08 and PD-SLS steel frames. Three different levels of performance are considered: Immediate Occupancy – IO; Life Safety – LS; Collapse Prevention – CP. The acceptance criteria are based on the following performance parameters: 1) interstorey drift damage index (IDI); 2) plastic rotations in columns and beams. (defined by tab.5.6 of FEMA 356).

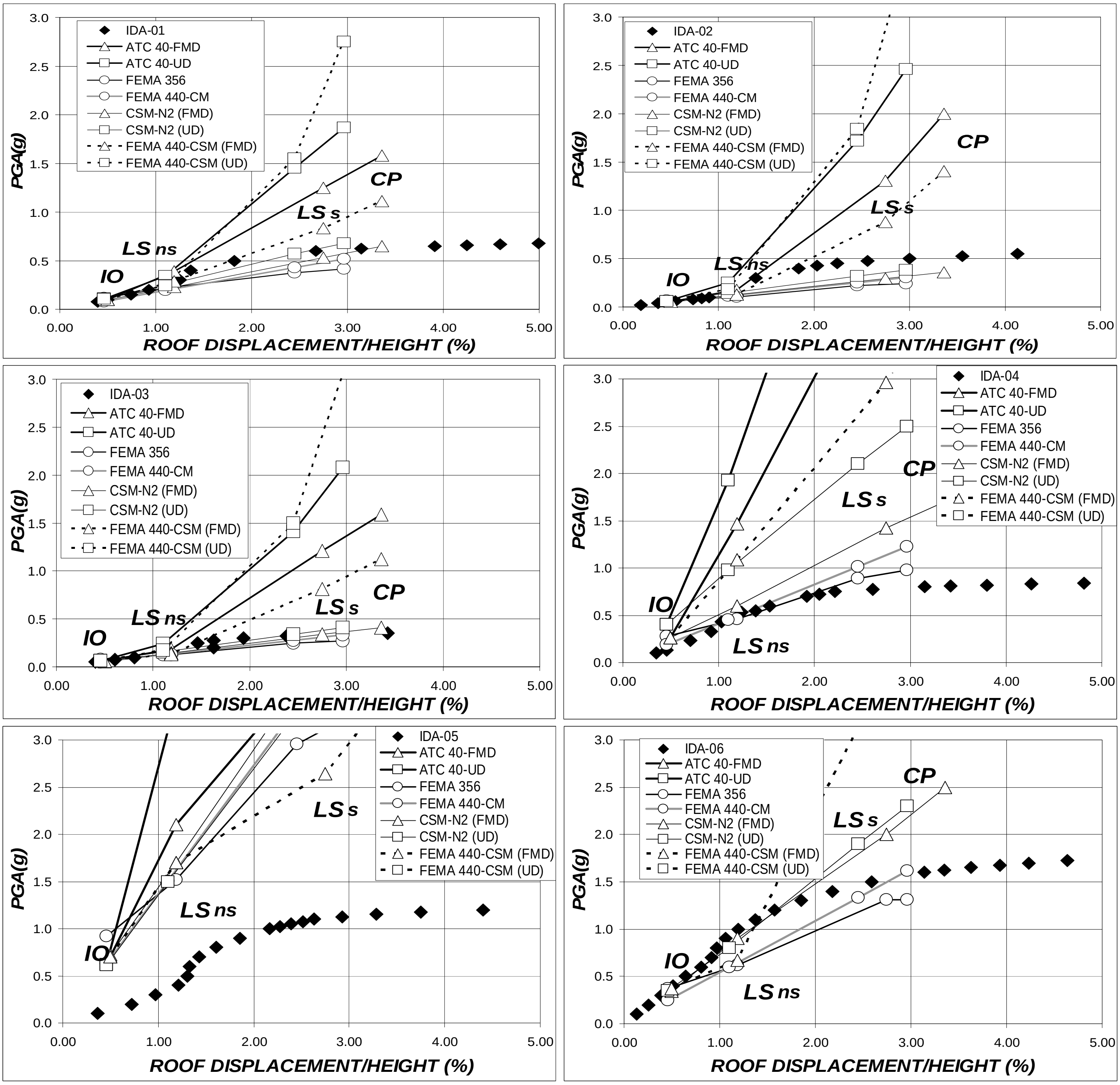

Figure 1 Peak ground acceleration versus roof displacement – Nonlinear Static Procedures and Incremental Dynamic Analysis (Italian Code 2008 design)

Table 1. Design Sections of Steel Frames

	9 Storeys 3 Bays - IC08			9 Storeys 3 Bays – PD-SLS		
N	BEAMS	EXT. COUMNS	INT. COUMNS	BEAMS	EXT. COUMNS	INT. COUMNS
1	IPE270	HE220B	HE280B	IPE270	HE500B	HE500B
2	IPE270	HE220B	HE280B	IPE270	HE450B	HE400B
3	IPE270	HE220B	HE280B	IPE270	HE450B	HE400B
4	IPE270	HE220B	HE280B	IPE270	HE450B	HE400B
5	IPE270	HE220B	HE280B	IPE270	HE400B	HE400B
6	IPE270	HE220B	HE280B	IPE270	HE400B	HE360B
7	IPE270	HE220B	HE280B	IPE270	HE400B	HE340B
8	IPE270	HE220B	HE280B	IPE270	HE320B	HE300B
9	IPE270	HE220B	HE280B	IPE270	HE260B	HE240B

Table 2. Fundamental periods and modal mass ratio

	1° mode		2° mode		3° mode	
	T(s)	α(%)	T(s)	α(%)	T(s)	α(%)
IC08	2.57	82	0.82	10	0.45	0.3
PD SLS	2.11	75	0.64	11	0.33	0.5

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. The assessment of current nonlinear static procedures is carried out by comparison against dynamic analysis. At this aim, six ground motions are selected to be consistent to EC8 8 elastic spectra for different soil classes (tab.3). In fig.3 the envelope values of the spectral acceleration are compared to EC8 elastic response spectra. In figs.1-2 the roof displacement for the limit states (Immediate Occupancy – IO; Life Safety for non-structural elements LS_{ns}; Life Safety for structural members LS_s, Collapse Prevention - CP) are related to the corresponding peak ground acceleration levels (PGA). Furthermore, also the peak roof displacement obtained from the incremental dynamic analysis is reported.

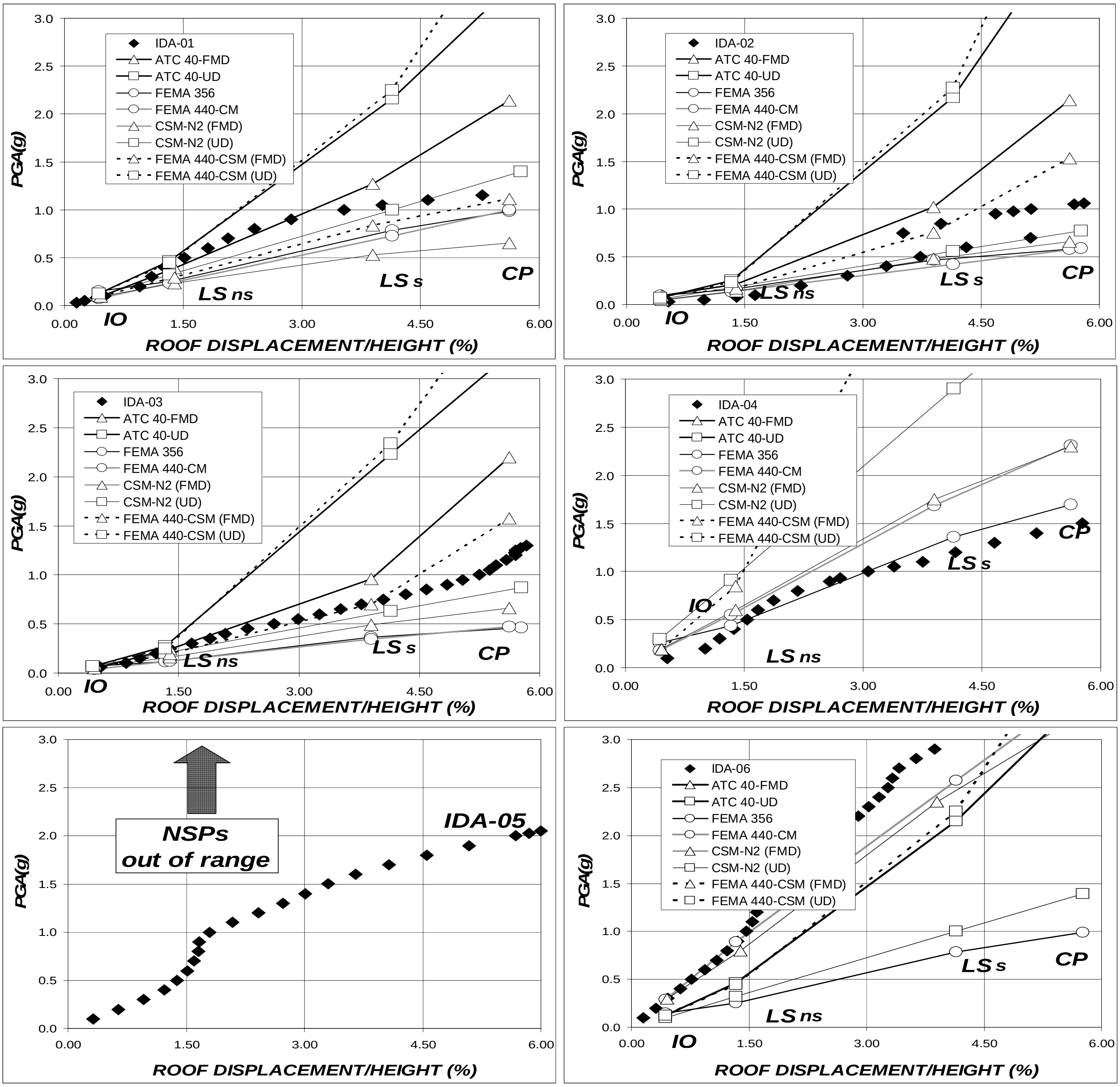

Figure 2. Peak ground acceleration versus roof displacement – Nonlinear Static Procedures and Incremental Dynamic Analysis (PD-SLS design)

FEMA 356 and FEMA 440 (Coefficient Method) give very similar performance points. This result derives from the value of the coefficient C_3 that is approximately equal to 1.0 in both cases. The procedures based on the High Damping Elastic Demand Response Spectra (HDERS) such as ATC 40 and FEMA 440 (ATC 55) give higher peak ground accelerations when compared to the other NSPs. Furthermore, the results are more sensitive to the lateral force distribution of the load pattern. In particular, the uniform distribution may give a non-conservative overestimation of PGA. This result derives from the differences between HDERS and IDRS defined by eqs.12-14. In order to show this effect in fig-4 the HDERS and the IDRS at collapse are compared. The PGA required to intersect the demand and the capacity spectrum in the performance point at collapse is 2.14 for HDERS and 1.2 for IDRS. For the El-Centro earthquake ground motion the nonlinear static procedures give values of PGA that are over the maximum value of the graphic. This result is a consequence of the acceleration response spectrum of El-Centro that quickly decreases for periods very close to the fundamental period of the framed structures considered.

Table 3. Earthquake ground motion characteristics.

N.	Input	Date	Dir.	Ms	PGA/g	t_R (s)	T_C(s)
1	Bevagna	26.09.97	NS	5.50	0.034	46.1	0.700
2	Sturno	23.11.80	EW	6.87	0.323	71.9	0.932
3	Gubbio	26.09.97	NS	5.90	0.099	106.03	0.652
4	San Fernando	09.02.71	NS	6.60	0.255	59.48	0.636
5	El Centro	18.05.40	NS	7.20	0.348	53.74	0.506
6	M. S. Severino	23.11.80	EW	6.87	0.139	72.31	0.471

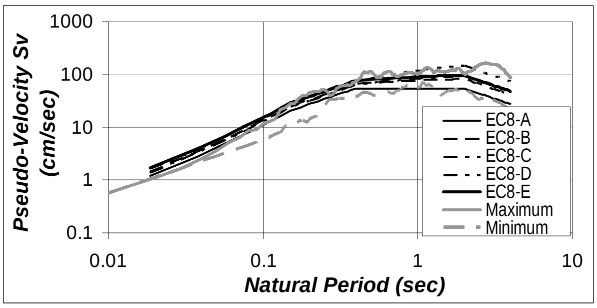

Figure 3. Envelope of ground motion response spectra and comparison with elastic response spectra of Eurocode 8.

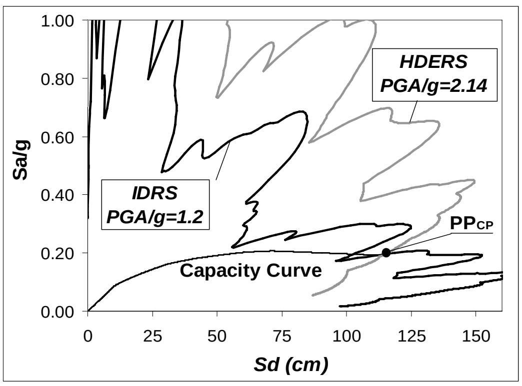

Figure 4. Comparison between ATC40 High Damping Elastic Demand Response Spectrum (HDERS) and CSM-N2 Inelastic Demand Response Spectrum at collapse. PD-SLS frame.

4 CONCLUSIONS

The accuracy of current nonlinear static procedures for estimating the displacement response of steel framed structures was investigated. The procedures based on the on the Capacity Spectrum Method and on the High Damping Elastic Response Spectra (ATC-40, FEMA 440) give deformation much different than the nonlinear response history analysis. On the contrary, the superior physical basis of Inelastic Demand Response Spectra (CSM-N2) proved to give more accurate estimation of the seismic performance, especially in the case of steel framed structures that usually exhibit high ductility factors.

REFERENCES

ASCE, 2000. Prestandard and Commentary for the Seismic Rehabilitation of Buildings. Report No. FEMA-356, Building Seismic Safety Council, Federal Emergency Management Agency, Washington, D.C.

ASCE, 2007. Seismic Rehabilitation and Evaluation of Existing Buildings, ASCE Standard No. ASCE/SEI 41-06, American Society of Civil Engineers, Reston, VA.

Aschheim, M.A., Maffei, J. & Black, E. 1998. Nonlinear Static Procedures and Earthquake Displacement Demands. Proceedings of 6th U.S. National Conference on Earthquake Engineering, Earthquake Engineering Research Institute, Seattle, WA.

Akkar, S. & Metin, A. 2007. Assessment of Improved Nonlinear Static Procedures in FEMA-440. *Journal of Structural Engineering*, 133(9): 1237-1246.

ATC-40. 1997. Seismic Evaluation and Retrofit of Con-crete Buildings. Report No. ATC-40, Applied Technology Council, Redwood City, CA.

ATC-55. 2003. Improvement of Inelastic Seismic Analysis Procedures. *Report No. FEMA-440*, Applied Technology Council, Redwood City, CA.

Chopra, A.K. & Goel, R.K. 2000. Evaluation of NSP to Estimate Seismic Deformation: SDF Systems. *Journal of Structural Engineering*, 126(4): 482-490.

CEN. 2003. Eurocode 8. Design of structures for earthquake resistance. European Committee for Standardization.

Ferraioli, M., Lavino, A., Avossa, A.M., Mandara, A. Displacement-based seismic assessment of steel moment resisting frame structures. 14th World Conference on Earthquake Engineering. October 12-17. Beijing, China. 2008.

Goel, R.K. 2005. Evaluation of Modal and FEMA Pushover Procedures Using Strong-Motion Records of Buildings." *Earthquake Spectra*, 21(3): 653-684.

Goel, R.K. & Chadwell, C. 2007. Evaluation of Current Nonlinear Static Procedures for Concrete Buildings Using Recorded Strong-Motion Data, Final Report, California Strong Motion Instrumentation Program, CDMG, Sacramento, CA.

Guyader, A.C. & Iwan, W.D. 2006. Determining Equivalent Linear Parameters for Use in a Capacity Spectrum Method. *Journal of Structural Engineering*, 132(1): 59-67.

Miranda, E. & Ruiz-Garcia, J. 2002. Evaluation of Approximate Methods to Estimate Maximum Inelastic Displacement Demands. *Earthquake Engineering and Structural Dynamics*, 31(3): 539-560.

SeismoSoft, 2008. SeismoStruct. A computer program for static and dynamic analysis for framed structures. (online) avaliable from URL: www.seismosoft.com.