2019 SECED Conference
Earthquake risk and engineering towards a resilient world
9-10 September 2019, Greenwich, London

EC8 VS ASCE7 RULES ON P-DELTA EFFECTS: SOME APPLICATIONS TO STEEL MOMENT RESISTING FRAMES

Roberto TARTAGLIA[1], Mario D'ANIELLO[2] & Raffaele LANDOLFO[3]

Abstract: *EC8-compliant steel moment resisting frames (MRFs) are often overdesigned owing to the severe requirements for lateral deformability and P-Delta effects. This is not the case of MRFs designed in accordance with North American codes (e.g. ASCE7), which require different rules to consider the structural sensitivity to the P-Delta effects. In lines with the latter approach, the current draft version of the amended EN1998-1-1 introduces a different methodology to account to the structural lateral displacements. This work aims to investigate, by both static and dynamic non-linear analysis, the effectiveness of new rules as respect to the former version of EC8 and the current ASCE7. Both non-linear static and dynamic analyses have been carried out by accounting for the real dimension of beam-to-column joints and their behaviour, which was simulated by means of a non-linear spring properly calibrated against experimental tests. The results show that the structures designed according to the latest draft version of the EN1998-1-1 and those compliant with the North American code have a similar behaviour.*

Introduction

Moment resisting frames (MRFs) are largely used for steel multi-story low and medium rise buildings in seismic areas since they can provide a high ductile behaviour allowing the formations of plastic hinges at the beam ends, without interfering with architectural design (Montuori et al. 2015, 2017a, 2018, Nastri 2017, 2018, Elghazouli 2010, Francavilla et al. 2018, Giordano et al. 2017, Tartaglia et al. 2018).
However, MRFs are characterized by large lateral deformability, thus being highly sensitive to second-order P-Delta effects [Medina et al. 2005]. The current version of EN1998-1-1 (2005) recommends to verify the sensitivity of the structures to P-Delta effects by means of the Horne method, where the critical multiplier is estimated considering the secant stiffness of the structure, the latter evaluated assuming that the theoretical strength is set equal to the design base shear and the corresponding displacement is set equal to the seismic demand given by the equal displacement rule.
This approach does not consider that the actual yield strength of the structure can be larger due to (i) the selection of slightly larger members, (ii) the hardening related to the formation of plastic mechanism, and (iii) the expected strength of the materials, which are larger than the design values. All these aspects clarify the reasons why EC8-compliant MRFs are often overdesigned as shown by Tenchini et al. 2014, Cassiano et al. 2016.
In the light of these considerations, a possible criterion to improve the current rule of EC8 to compute the stability coefficient can be accounting for the sources of design overstrength.
In the United States of America (US), the rules given by ASCE7 (2016) codes also account for the design overstrength and the stiffness is evaluated at a displacement demand computed more efficiently and more accurately than the equal displacement rule adopted in EC8.
In order to investigate the influence on the design and performance of steel MRFs, the three methods (i.e. current EC8, draft of amended EC8 and ASCE7) to account for second order effects were applied to a set of structures and their seismic behaviour was also examined by means of both nonlinear static and dynamic analysis, against the three limit states defined in EN1998-3 (2005), namely: damage limitation (DL), severe damage (SL) and near collapse (NC).
The paper is organized into three main parts, namely (1) the introduction of the investigated parameters with the descriptions of the three design approaches; (2) the descriptions of the geometrical features of the examined structures and the modelling assumptions; (3) the discussion of the results.

[1] Phd, University of Naples Federico II, Naples, Italy, roberto.tartaglia@unina.it
[2] Prof., University of Naples Federico II, Naples, Italy
[3] Prof., University of Naples Federico II, Naples, Italy

EC8 vs ASCE 7 design rules for P-Delta effects

The current EN1998-1-1 (hereinafter also referred as “EC8-C”) recommends to calculate the stability coefficient (θ_{EC8-C}) for P-Delta effects as follows:

$$\theta_{EC8-C} = \frac{P_{tot} \cdot d_r}{V_{tot} \cdot h} \tag{1}$$

Where P_{tot} is the total vertical loads due to the gravity loads in the seismic loading combination, d_r is the design interstorey drift, V_{tot} is the total seismic shear and h is the interstorey height
If θ_{EC8-C} is smaller than 0.1, the second order effects can be neglected at design stage, while if θ_{EC8-C} is smaller than 0.2 the calculated internal effects should be magnified by the factor (α) equal to 1/(1- θ_{EC8-C}). If θ_{EC8-C} exceed 0.3 the structure should be re-designed.
The design interstorey drift (d_r) is evaluated as follows:

$$d_{r,i} = d_{s,i} - d_{s,i-1} \tag{2}$$

Where d_s is a structural displacements induced by the design seismic action and it is evaluated on the basis of the equal displacement rule by the following equation:

$$d_s = q \cdot d_e \tag{3}$$

Where q is the adopted behaviour factor and de is structural displacement evaluated by means a response spectrum linear analysis.
According to this method, the displacements are evaluated considering the secant stiffness of the structure by assuming that the ultimate resistance is equal to the design base shear, disregarding either any source of overstrength or the hardening effects associated with redundancy and plastic redistribution.
From these considerations a proposal to amended EC8 rule (hereinafter referred as “EC8-M”) may be computing the stability coefficient by increasing the secant stiffness as follows:

$$\theta_{EC8-M} = \frac{P_{tot} \cdot d_r}{V_{tot} \cdot h \cdot \gamma_{ov} \cdot \Omega_d} \tag{4}$$

Where γ_{ov} accounts for the randomness of the yield stress of the steel and Ω_d is equal to min($R_{d,i}$/$E_{d,i}$) being $R_{d,i}$ the plastic resistance of the i-th dissipative component and $E_{d,i}$ is the corresponding design effect.
This criterion is very similar to the proposal by Vigh et al. (2016), but it is more stringent because the overall hardening due to the redundancy of the structural system is not considered. The stability coefficient (θ_{ASCE}) recommended by ASCE7 is evaluated according to the following equation:

$$\theta_{ASCE} = \frac{P_x \cdot \Delta \cdot I_e}{V_x \cdot h_{sx} \cdot C_d} \tag{5}$$

Where P_x are the total vertical loads at level x (which perfectly corresponds to the P_{tot} in the European approach), I_e is the importance factor (which is assumed equal to 1 in this study), V_x is the seismic shear force acting between levels x and x – 1, h_{sx} is the storey height, C_d the deflection factor (that was assumed equal to 5.5) and Δ the design story drift occurring simultaneously with V_x. With this regard, the design interstory drift Δ are computed as the largest difference of the displacement δ_x and δ_{x-1}, where δ_x is estimated as follows:

$$\Delta_i = \delta_{x,i} - \delta_{x,i-1} \quad \rightarrow \quad \delta_x = \frac{C_d \cdot \delta_{xe}}{I_e} \tag{6}$$

where δ_{xe} is the displacement determined by an elastic analysis.
If the stability coefficient is larger of 1 an amplification factor should be introduced to take into account, in the design analyses, of the P-Delta influence. However, θ_{ASCE} should not exceed the θ_{max} (Equation (7)).

$$\theta_{\max} = \frac{0.5}{\beta \cdot I_e} \leq 0.25 \tag{7}$$

Where β can be assumed equal to 1 in conservatively way.

Reference structures

The influence of the second order effects was investigated with reference to six MRFs that were designed varying the number of storey (3 and 6) and the method to account for the P-Delta effects. In order to avoid misleading conclusions, all the structures were designed according to current Eurocodes (i.e. both the EN1993-1-1 and EN1998-1-1), with the only exception of the P-Delta effects.
The first set of structures were designed according to the current EC8 (i.e. EC8-C), the second set of structures were designed considering a modification of the actual EN1998-1-1 (i.e. EC8-M) and finally, in the last set of MRFs the P-Delta effects were accounted for according the ASCE7 approach.

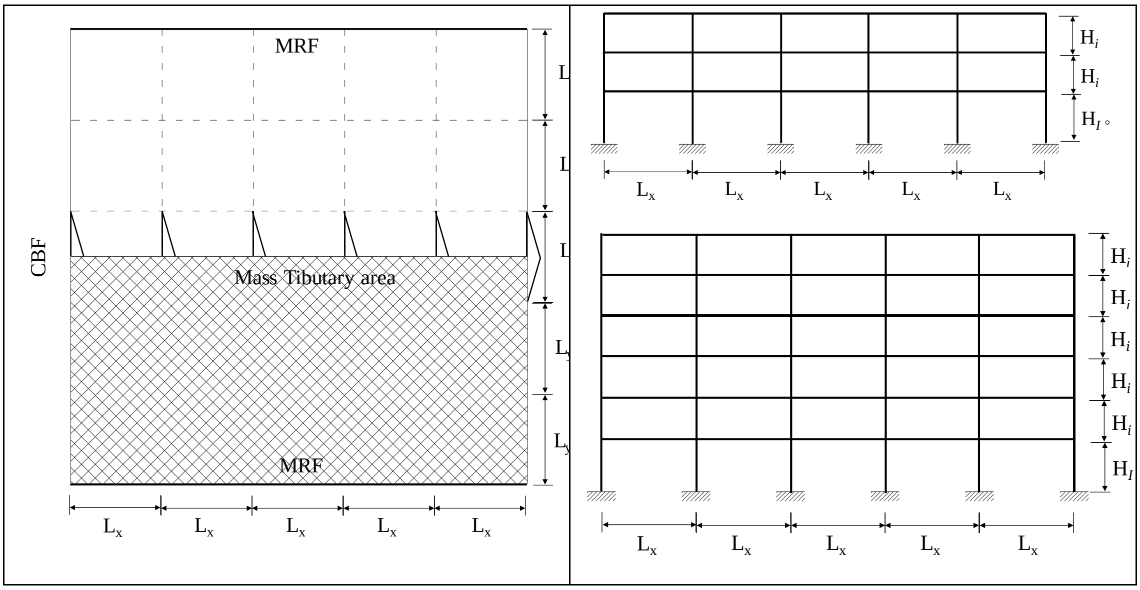

Figure 1: Geometrical features of the investigated structures

All structures have constant interstorey height of 3.5m except for the first level that is equal to 4.5m. The plane has a rectangular shape, with five MRF bays in X direction and three bays in Y direction with a concentric brace frame (CBF) system in the central bay (see Figure 1). The span length is constant for all the investigated structures and equal to 8m. Both the structural design and verification were done on a planar MRF system that was extracted from the 3D structure.
The vertical dead load was assumed equal to 4.5kN/m2 and the live loads were set equal to 2kN/m2. A peak ground acceleration (PGA) equal to 0.35g considering a soil type B and a damping ratio equal to 2% were assumed. According to the EN1998-1-1, in the seismic design, the torsional effect was taken into account by the introduction of the amplification factor δ assumed equal to 1.3.
As damage limitation requirement, the maximum interstorey drift (ISD) ratio was fixed equal to 1%, thus assuming non-structural elements fixed in a way so as not to interfere with structural deformations.
The design forces have been calculated by means of response spectrum analyses (RSA), according to EN1998-1-1, where all the vibration modes that contribute significantly to the global response were accounted for.
Full strength extended stiffened end-plate joints, designed according to the procedure described by D’Aniello et al. (2017), were adopted for all the investigated structures.
The results in terms of cross section are reported in Table 1 and Table 2 for the all the investigated structures.

Design Criteria	Floor	First bay		Second bay		Third bay	
		Column	Beam	Column	Beam	Column	Beam
EC8-C	I	HE500B	IPE500	HE550B	IPE500	HE550B	IPE500
	II	HE500B	IPE500	HE550B	IPE500	HE550B	IPE500
	III	HE500B	IPE500	HE500B	IPE500	HE500B	IPE500
EC8-M	I	HE400B	IPE500	HE450B	IPE500	HE550B	IPE500
	II	HE400B	IPE500	HE450B	IPE500	HE550B	IPE500
	III	HE400B	IPE500	HE400B	IPE500	HE500B	IPE500
ASCE	I	HE360B	IPE400	HE400B	IPE400	HE400B	IPE400
	II	HE360B	IPE400	HE400B	IPE400	HE400B	IPE400
	III	HE360B	IPE400	HE400B	IPE400	HE400B	IPE400

Table 1: Cross section of the 3-storey frames

Design Criteria	Floor	First bay		Second bay		Third bay	
		Column	Beam	Column	Beam	Column	Beam
EC8-C	I	HE300B	IPE400	HE650M	IPE750x147	HE800M	IPE750x147
	II	HE300B	IPE400	HE650M	IPE750x147	HE800M	IPE750x147
	III	HE300B	IPE400	HE550M	IPE750x137	HE650M	IPE750x137
	IV	HE300B	IPE400	HE550M	IPE750x137	HE650M	IPE750x137
	V	HE300B	IPE400	HE500B	IPE550	HE550M	IPE550
	VI	HE300B	IPE400	HE500B	IPE550	HE550M	IPE550
EC8-M	I	HE550B	IPE750x137	HE600M	IPE750x137	HE600M	IPE750x137
	II	HE550B	IPE750x137	HE600M	IPE750x137	HE600M	IPE750x137
	III	HE500B	IPE550	HE550B	IPE550	HE550B	IPE550
	IV	HE500B	IPE550	HE550B	IPE550	HE550B	IPE550
	V	HE400B	IPE450	HE500B	IPE450	HE500B	IPE450
	VI	HE400B	IPE450	HE500B	IPE450	HE500B	IPE450
ASCE	I	HE500B	IPE550	HE550B	IPE550	HE550B	IPE550
	II	HE500B	IPE550	HE550B	IPE550	HE550B	IPE550
	III	HE450B	IPE500	HE500B	IPE500	HE500B	IPE500
	IV	HE450B	IPE500	HE500B	IPE500	HE500B	IPE500
	V	HE360B	IPE450	HE450B	IPE450	HE450B	IPE450
	VI	HE360B	IPE450	HE450B	IPE450	HE450B	IPE450

Table 2: Cross section of the 6-storey frames

Modelling assumptions

The structures were modelled by means of Seismostruct (2018). Planar 2D models of the MRFs were adopted. The diaphragm constraint was considered at each floor to simulate the in-plane rigidity of the slab and to connect the frame to a leaning column, where both the concentrated masses at each floor and the gravity loads acting on the interior frames were applied. The leaning columns have no lateral stiffness and do not interact with the lateral stiffness of the frame.
All structural elements were modelled by force-based (FB) distributed inelasticity elements. These elements account for distributed inelasticity through integration of material response over the cross section and integration of the section response along the length of the element. The cross-section behaviour is reproduced by means of the fiber approach, assigning a uniaxial stress–strain relationship at each fiber. The Menegotto-Pinto (Menegotto et al. 1973) law was introduced to model the behaviour of all steel elements, while both the leaning column and the rigid links were modelled by an indefinitely elastic material.

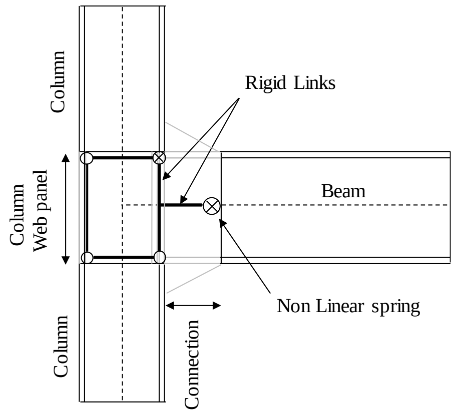

Figure 2: Joint modelling features

Since the MRF structural behaviour is strongly influenced by the beam-to-column joints (Montuori et al. 2016, 2017b, a nonlinear spring was introduced between the column and the beam.
The lumped springs having zero-length placed at the end of each elements were adopted to account for the plastic hinges behaviour (see Figure 2). In particular, the Ibarra-Medina-Krawinkler (IMK) modified model (Lignos and Krawinklerwas (2011)) was implemented in all the investigated structures; the model was developed mainly for the US profile and only a few European tests were taken into account. However, as mentioned by other authors (Tsitos et al (2017), Bravo-Haro (2018)), the IMK model was validated on a large number of experimental results considering a wide range of steel profile and beam-to-column connection; therefore, its robustness allows its use also for the European practice.

PushOver

The static push-over analyses were performed to investigate: the elastic stiffness (k), the maximum resistance at the base (V_{Max}) and the overall overstrength factor (Ω) of the structures.
The results (see Figure 3) show that independently from the number of story (3 or 6), the EC8-C structures are the more resistant and stiffer, showing a very large overstrength, while, a gradual decrease of stiffness and resistance can be observed from the others design criteria. The EC8-M and ASCE structures have a similar behavior, showing larger lateral displacement with a consequent smaller overstrength factor.
For what concern the overstrength factor, the structures designed according to the current version of EN1998-1-1 show the larger value of the Ω. This result is consistent with the design assumptions; indeed, the EC8-C structures have an overstrength factor 1.18 and 1.21 larger of the EC8-M and ASCE structures respectively. These differences are almost the same for all the investigated structures (see table 3).

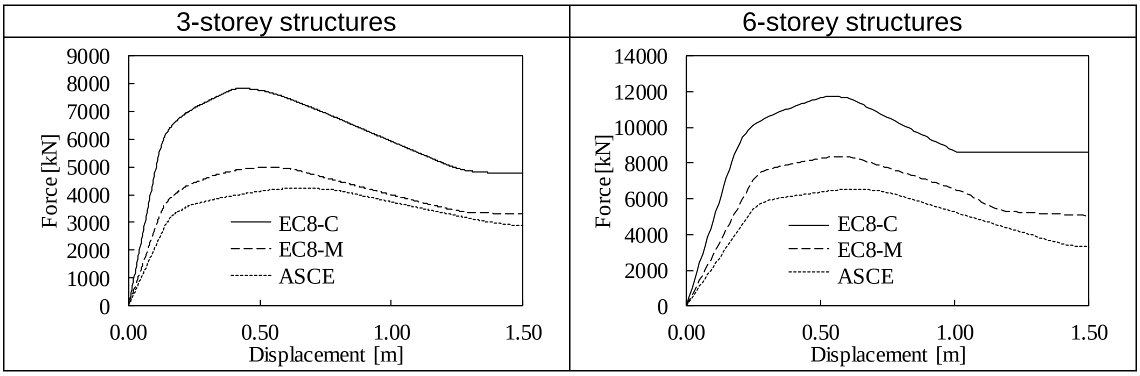

Figure 3: Pushover results in terms of force displacements curve

	Design Criteria	Elastic Stif. (k)	Design Res. (V_{Ed})	Shear at first Plastic hinge (V_y)	Max Capacity (V_{max})	V_{max}/V_y	V_y/V_{Ed}	OverStr. (Ω)
		kN/m	kN	kN	kN	-	-	-
3-storey	EC8-C	47500	1162	5100	7845	1.54	4.39	5.93
	EC8-M	27500	917	3150	4984	1.58	3.43	5.02
	ASCE	21000	814	2750	4244	1.54	3.38	4.92
6-storey	EC8-C	48300	1895	7200	11770	1.63	3.80	5.43
	EC8-M	28700	1654	6300	8084	1.33	4.05	5.38
	ASCE	22000	1537	4800	6543	1.36	3.12	4.49

Table 3: Push over results of 3-storey and 6-storey structures

Dynamic non-linear analyses

A selection of 14 natural earthquake acceleration records was considered for the dynamic time history analyses (DTH); the signals were obtained from the RESORCE ground motion database (Akkar et al. 2014) and selected according to procedure described in (Fulop, 2010). The data of the records are reported in Table 5 and the comparison with the elastic spectrum provided by EN1998-1-1 is reported in Figure 4. In order to evaluate the structural response at damage limitation (DL), severe damage (SD) and at near collapse (NC) limit states, the DTH analyses were performed for each of the investigated signals scaled respectively for 0.5, 1 and 1.73. Hence, the maximum intersorey drift (IDR) was evaluated at each storey (see Figure 5).

The average profiles of peak interstorey drift ratios (ISDR) per limit state are shown in Figure 5; focusing on the 3-storey structures, it can be observed the EC8-M structure has a similar behavior respect to the ASCE structure; while the EC8-M MRFs show the largest stiffness. Indeed, at the DL limit state all the structures behave elastically with the maximum joint rotation equal to 1% (for the ASCE structure); contrariwise, at SD limit state only the EC8-C remains in elastic range. The other two methods show larger lateral displacements, reaching a maximum of 2% of chord rotation. Small ISDR can be observed also NC limit state, where the ASCE and EC8-M structure reach up to 2.5% of rotation while the EC8-C MRF not overcame the 2%. This means that the structure designed according to the current version of the EN1998-1-1 behave almost elastically up to NC limit state.

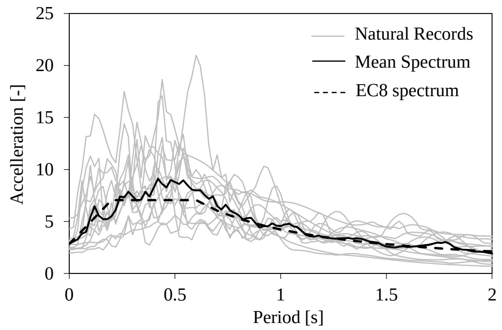

Figure 4: Comparison between the European elastic spectrum and the selected records

Earthquake	Date	Station Name	Country	Magnitude	Fault mechanism
Alkion	24.02.1981	Xylokastro-O.T.E.	Greece	6.6	Normal
Montenegro	24.05.1979	Bar-Skupstina Opstine	Montenegro	6.2	Reverse
Izmit	13.09.1999	Yarimca (Eri)	Turkey	5.8	Strike-Slip
Izmit	13.09.1999	Usgs Golden Station Kor	Turkey	5.8	Strike-Slip
Faial	09.07.1998	Horta	Portugal	6.1	Strike-Slip
L'Aquila	06.04.2009	L'Aquila V. Aterno Aquila	Italy	6.3	Normal
Aigion	15.06.1995	Aigio-OTE	Greece	6.5	Normal
Alkion	24.02.1981	Korinthos-OTE Building	Greece	6.6	Normal
Umbria-Marche	26.09.1997	Castelnuovo-Assisi	Italy	6	Normal
Izmit	17.08.1999	Heybeliada-Senatoryum	Turkey	7.4	Strike-Slip
Izmit	17.08.1999	Istanbul-Zeytinburnu	Turkey	7.4	Strike-Slip
Ishakli	03.02.2002	Afyon-Bayindirlik ve Iskan	Turkey	5.8	Normal
Olfus	29.05.2008	Ljosafoss Hydroeletric Power	Iceland	6.3	Strike-Slip
Olfus	29.05.2008	Selfoss-City Hall	Iceland	6.3	Strike-Slip

Table 4: Main features of the adopted accelerograms

Contrariwise, in case of the 6-Strorey structures the differences between the different P-Delta rules are less evident; in particular at DL limit state, the differences in terms of interstorey drift between the EC8-C and the others design criteria are 10% for EC8-M structures and -20% for ASCE ones. At SD limit state, due to the activation of some plastic deformation in both the beam and the column base, the ratio between the maximum lateral displacements of the investigated approaches show a smaller decrease. At NC limit state, the structures designed according to EC8-C show large plastic deformation exceeding the limit of 2% especially on the top storey; larger plastic deformation could be observed in the case of EC8-M and ASCE structures that reach 3% of rotation.

Conclusions

This work summarizes the results of a preliminary study devoted to investigate the limits and the potential improvements of the rules for the stability checks of EN1998-1 with application to steel moment resisting frames. On the basis of the results of both static and dynamic nonlinear analyses the following remarks can be drawn:

- The pushover analysis confirms that, independently from number of storey, the structures compliant the EN1998-1-1 are the stiffer and the stronger among those examined.

• At the damage limitation limit state, all the investigated MRFs show an elastic behaviour with small displacements (1% of chord rotation). Some damage is observed at severe damage limit state, where both the ASCE and the EC8-M structures show ISDR up to 2%. Contrariwise, only at the near collapse limit state the EC8-current structures have interstorey drift about 2%.

• The rule of the current Eurocode 8 is extremely conservative, thus leading to design structures that behave almost elastically up to the NC limit state.

• The modified rule that account for the design overstrength of the frame gives results comparable to those obtained for the frames designed according to ASCE-7.

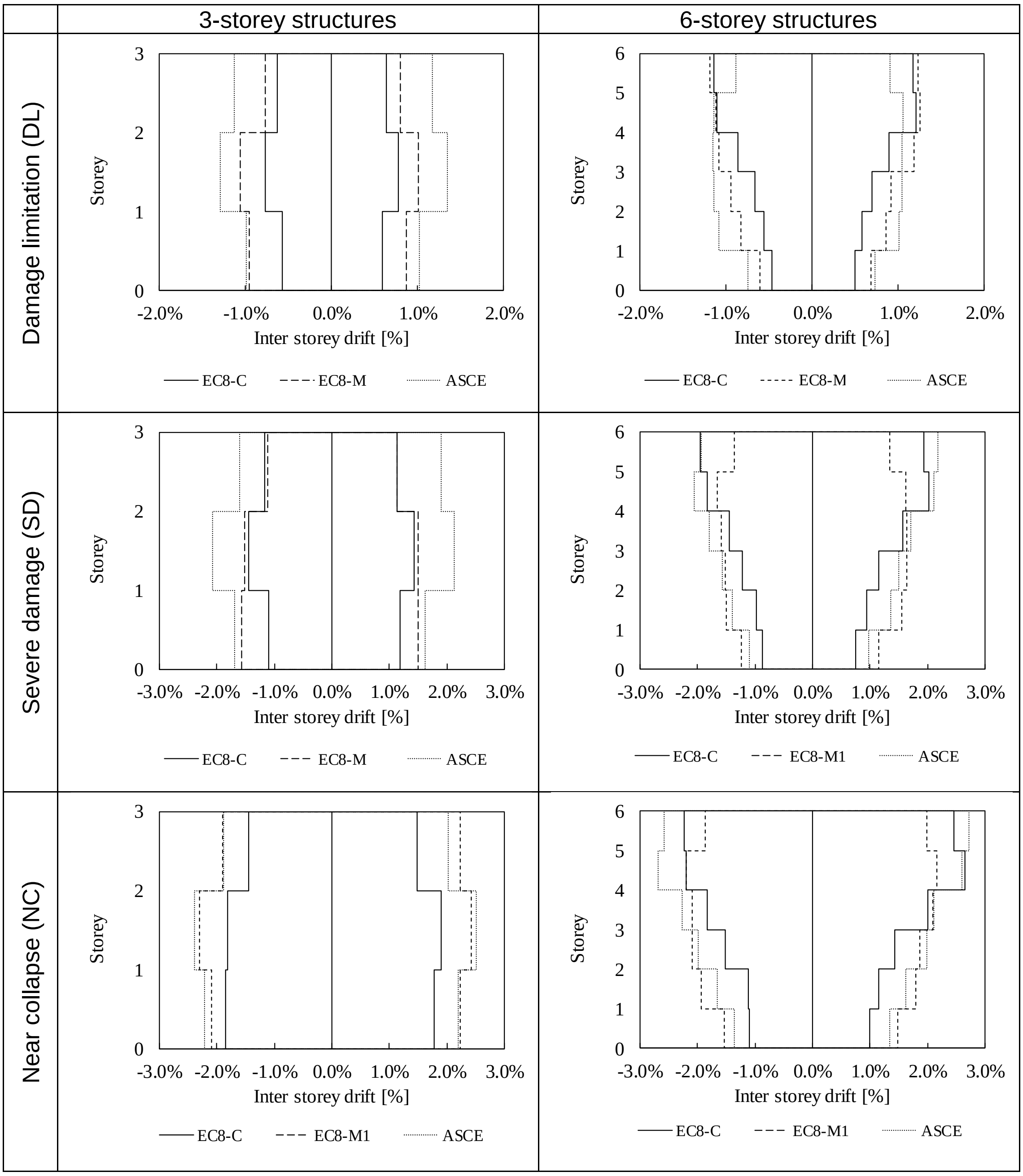

Figure 5: MRF-3-3-0.25 and MRF-3-5-0.25 interstorey drift results

References

Akkar S, Sandıkkaya MA, Şenyurt M, Azari Sisi A, Ay BÖ, Traversa P, Douglas J, Cotton F, Luzi, L, Hernandez B, Godey S (2014), Reference database for seismic ground-motion in Europe (RESORCE), Bulletin of Earthquake Engineering, 12(1): 311-339.

ASCE/SEI. Minimum design loads for buildings and other structures, ASCE7-16. American Society of Civil Engineers/Structural Engineering Institute, Reston, VA; 2016.

Bravo-Haro MA, Tsitos A, Elghzouli A (2017), Drift and rotation demands in steel frames incorporating degradation effects, Bulletin of Earthquake Engineering 16: 4919-4950

Cassiano D, D'Aniello M, Rebelo C, Landolfo R, da Silva L (2016), Influence of seismic design rules on the robustness of steel moment resisting frames, Steel and Composite Structures, 21(3): 479-500

D'Aniello M, Tartaglia R, Costanzo S, Landolfo R (2017), Seismic design of extended stiffened end-plate joints in the framework of Eurocodes, Journal of Constructional Steel Research, 128: 512–527

D'Aniello, M., Tartaglia, R., Costanzo S, Campanella G, Landolfo R, De Martino A (2018), Experimental Tests on Extended Stiffened End-Plate Joints within Equal Joints Project. Key Engineering Materials, 763: 406-413

Elghazouli AY (2010), Assessment of European seismic design procedures for steel framed structures. Bulletin of Earthquake Engineering, 8(1): 65–89

EQUALJOINTS – European pre-QUALified steel JOINTS: RFSR-CT-2013-00021. Research Fund for Coal and Steel (RFCS) research programme

EN 1993:1–1 (2005), Design of Steel Structures - Part 1–1: General rules and rules for buildings. CEN

EN 1998-1 (2005), Design of Structures for Earthquake Resistance - Part 1: General Rules, Seismic Actions and Rules for Buildings. CEN

EN 1998-3 (2005), Design of Structures for Earthquake Resistance - Part 3: Assessment and retrofitting of buildings. CEN.

EN 1993:1–8 (2005), Design of Steel Structures - Part 1–8: Design of Joints. CEN.

Francavilla AB, Latour M, Rizzano G, Jaspart J-P, Demonceau J-F (2018), On the robustness of earthquake-resistant moment-resistant frames: Influence of innovative beam-to-column joints, Open Construction and Building Technology Journal, 12: 101-111

Fulop L (2010), Selection of earthquake records for the parametric analysis, Research Report VTT-R-03238-10, (VTT, Espoo)

Giordano V, Chisari C, Rizzano G, Latour M (2017), Prediction of seismic response of moment resistant steel frames using different hysteretic models for dissipative zones, Ingegneria Sismica - International Journal of Earthquake Engineering, 34(4): 42-56

Lignos DG, Krawinklerwas HM (2011), Deterioration modelling of steel components in support of collapse prediction of steel moment frame under earthquake loading, Journal of Structural Engineering 137(11): 1291-1302

Medina AR, Krawinkler HJ, (2005), Evaluation of drift demands for the seismic performance assessment of frames, Structural Engineering, 131(7): 1003–13

Menegotto M and Pinto P E (1973), Method of analysis for cyclic ally loaded R.C. plane frames including changes in geometry and non-elastic behaviour of elements under combined normal force and bending, Proc. of Symposium on the Resistance and Ultimate Deformability of Structures Acted on by Well Defined Repeated Loads

Montuori R, Nastri E, Piluso V (2015), Advances in theory of plastic mechanism control: Closed form solution for MR-Frames, Earthquake Engineering and Structural Dynamics, 44(7): 1035-1054

Montuori R, Nastri E, Piluso V, Troisi M (2016), Influence of the cyclic behaviour of beam-to-column connection on the seismic response of regular steel frames, Ingegneria Sismica - International Journal of Earthquake Engineering, 33(1): 91-105

Montuori R, Nastri E, Piluso V (2017a), Influence of the bracing scheme on seismic performances of MRF-EBF dual systems, Journal of Constructional Steel Research, 13:, 179-190

Montuori R, Nastri E, Piluso V, Troisi M, (2017b), Influence of connection typology on seismic response of MR-Frames with and without 'set-backs', Earthquake Engineering and Structural Dynamics, 46(1): 5-25

Montuori R, Nastri E, Piluso V, Streppone S, D'Aniello M, Zimbru M, Landolfo R (2018), Comparison between different design strategies for Freedam frames: Push-Overs and IDA Analyses, The Open Construction and Building Technology Journal, 12 (1): 140-153.

Nastri E (2017), The influence of geometry, loads and steel grade for the development of a specific collapse type of MR-Frames, COMPDYN 2017 – Proc. of the 6th International Conference on Computational Methods in Structural Dynamics and Earthquake Engineering 2: 5014-5025

Nastri E (2018), Design and assessment of steel structures in seismic areas: Outcomes of the last italian conference of steel structures, Ingegneria sismica - International Journal of Earthquake Engineering, 35(2): 1-4

SeismoStruct 2018 – A computer program for static and dynamic nonlinear analysis of framed structures," available from http://www.seismosoft.com

Tartaglia R, D'Aniello M, Rassati GA, Swanson J, Landolfo R (2018a), Influence of composite slab on the nonlinear response of extended end-plate beam-to-column joints, Key Engineering Materials, 763: 818-825

Tartaglia R, D'Aniello M, Di Lorenzo G, De Martino A (2019), Influence of ec8 rules on p-delta effects on the design and response of steel MRF, Ingegneria sismica - International Journal of Earthquake Engineering, 3: 104-120.

Tsitos A, Bravo-Haro MA, Elghzouli A, (2017), Influence of deterioration modelling on the seismic response of steel moment frames designed to Eurocode 8, Earthquake Engineering. Structural Dynamics. 47:356–376.

Tenchini A, D'Aniello M, Rebelo C, Landolfo R, da Silva LS, Lima L (2014), Seismic performance of dual-steel moment resisting frames, Journal of Constructional of Steel Research, 101: 437-454

Vigh LG, Zsarnóczay A, Balogh JM, Castro JM (2016), P-Delta effect and pushover analysis: Review of Eurocode 8-1, Report N.1 of WG2 CEN/TC 250/SC 8