Effects of Capacity Design Rules on Seismic Performance of Steel Moment Resisting Frames

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SUMMARY:
The current paper deals with the seismic design of 9-Storey office building using Eurocode 8 and *AISC* (American Institute of Steel Construction) provisions, where the seismic load resisting system is composed of either spatial or perimeter moment resisting frames. According to EC8, Ductility Class High (DCH) and Ductility Class Medium (DCM) with behaviour factor of 6.5 and 4.0 respectively, are used. Whereas in the case of AISC code, only Special Moment resisting Frame (SMF) with response modification factor of 8 is employed. In order to shed light on the pros and cons of the design criteria and thus the influence on the capacity design rules of the two aforementioned codes, designed frames are analysed by non-linear static analysis. The frame performances are measured in terms of overstrength and redundancy factors, strength demand to capacity and drift demand to capacity ratios, allowing interesting conclusions to be drawn.

Keywords: Seismic codes, Moment resisting steel frames, Seismic resistance, Pushover analysis

1. INTRODUCTION

To control global structural behaviour, codes give the so called criterion of capacity design where non-dissipative members are designed for comparatively higher seismic forces than dissipative members and where dissipative members are kept at such locations that will fail before the brittle members and subsequently will protect non-ductile elements by overstressing. Capacity design has been initially recommended in the seismic code of New Zealand. In particular, (Paulay and Priestley, 1992) and (Priestley, 2003) proposed weak beam and strong column concept in the design of moment resisting frames by suggesting of providing reduce stiffness of beams than columns. (Nassar and Krawinkler, 1991) and (Miranda and Bertero, 1994) examined the force reduction factors, providing a detailed discussions and improvements on the ductility reduction factors. Further, (Bertero, 1991) discussed the influence of overstrength factor on the performance of structures designed according to the codified formulations. Also (Sanchez-Ricart and Plumier, 2008) investigated overstrength factors for frames, highlighting the concept of capacity design. (Rahgozar and Humar, 1998) assessed the extent of reserve strength attributable to redistribution in steel frames. (Ballio et al., 1988) provided extensive studies to justify definition values of the reduction factor in ECCS Manual 1988 (Design of steel structures in seismic zones). Further, (Hasegawa et al., 2000) assessed the perimeter frame designed according to U.S. procedure and spatial frame according to Japanese codes to evaluate the major differences between the two configurations. (Elghazouli, 2010) extensively contributed in the assessment of European seismic design procedures and philosophies for several lateral load resisting systems, especially concerning moment resisting frames due to their paramount inelastic behaviour. The presented paper is aimed at providing useful information for readers and technicians who are involved in the design of MRFs according to the European and American codes.

2. CAPACITY DESIGN OF MRFS: EUROPEAN VS. AMERICAN SEISMIC CODES

In order to provide a comparison of the capacity design rules in Eurocodes ((EN-1993-1-1, 2005)-

(EN-1998-1, 2005) and AISC-ASCE (ANSI/AISC-341-10, 2010)-(ASCE/SEI-7-10, 2010) for the design of MRF, the noticeable features provided by the relevant codes are illustrated briefly in the synoptic comparative scheme given in Table 1 (Naqash et al., In Press).

Table 1. Seismic related factors and checks for EC3-EC8 and AISC-ASCE provisions

Description	*Eurocodes (EC3/EC8)*	*AISC/ASCE*	*Remarks*
Energy dissipation philosophy	Prescribed by means of DCL, DCM and DCH	Given by OMF, IMF and SMF	*IMF and OMF are restricted to limited heights in high seismic categories*
Seismic load reduction factor	A behaviour factor (q) equal to 4 for DCM and $5\alpha_u/\alpha_1$ for DCH is provided.	A response modification factor (R) equal to 4.5 for IMF and 8 for SMF is given	*An almost same criterion is considered*
Cross section limitations	For $q > 4$ only class 1 sections are allowed, for $2 < q \leq 4$ class 1 and class 2 and for $1.5 < q \leq 2$ class 1, 2 and 3 are allowed	Limits λ_p *to* λ_{ps}, i.e. to use seismically compact section and is obtained by modified slenderness ratio	*Class 1 and seismically compact sections are unaffected by local buckling*
Rotation capacity (local ductility concept)	Plastic hinge rotation is limited to *35 mrad* for structures of DCH and *25 mrad* for structures of DCM	SMF and IMF are designed to accommodate plastic hinge rotation of *30mrad* and *10mrad,* respectively with inter-storey drifts in the range of 0.04 and 0.02 radians, respectively	*For high seismicity it is recommended by both codes to apply ductility concept*
Overstrength factor	$\Omega = \frac{M_{pl,Rd,i}}{M_{Ed,i}}$	Ω_o equal to 3 for MRFs	Ω_o *in EC8 is* $(1.1\gamma_{ov}\,\Omega)$
Strength checks for dissipative elements (Beam checks)	$\frac{M_{E,d}}{M_{pl,Rd}} \leq 1.0,\ \frac{N_{E,d}}{N_{pl,Rd}} \leq 0.15\ \frac{V_{E,d}}{V_{pl,Rd}} \leq 0.5$	No additional checks are required except strength checks using AISC/LRFD	*Additional checks to be carry out for the seismic conditions*
Non dissipative elements (e.g. Columns checks in MRFs)	$N_{Ed} = N_{Ed,G} + 1.1\gamma_{ov}\Omega N_{Ed,E}$ $M_{Ed} = M_{Ed,G} + 1.1\gamma_{ov}\Omega M_{Ed,E}$ $V_{Ed} = V_{Ed,G} + 1.1\gamma_{ov}\Omega V_{Ed,E}$	Verification of strength with loads computed from special load combinations having Ω_o	*Stability checks are normally employed for these conditions*
Strong column weak beam (SCWB) philosophy	$\sum M_{Rc} \geq 1.3 \sum M_{Rb}$	$\frac{\sum M^*_{pc}}{\sum M^*_{bc}} \geq 1.0$	*EC8 accounts 1.3, while AISC considers a factor 1.1*R_y *to increase the nominal beam strength*
Panel Zone philosophy	Strong-PZ with weak beam is recommended	Both weak/intermediate or strong PZ with weak beam are allowed	*Intermediate PZ is preferred in order to have high dissipative capacity*
Panel Zone (PZ) (Stability check)	$\frac{h_w}{t} \leq \frac{72\varepsilon}{\eta}$ *with* $\varepsilon = \sqrt{\frac{235}{f_y}}$ where f_y is in Mpa, and η is a factor with 1.2 as recommended value.	$t \geq \frac{d_z + w_z}{90}$ where d_z, w_z and t are length, width and thickness of PZ respectively	*EC8 refers to EC3 for stability check of PZ. (Brandonisio et al., 2011)*

According to Table 1, *DCL* is Ductility Class Low, *DCM* is Ductility Class Medium and *DCH* is Ductility Class High; SMF is Special Moment resisting Frames, IMF is Intermediate Moment resisting Frames and OMF is Ordinary Moment resisting Frames. In EC8 as mentioned in Table 1, the multiplier α_u/α_1 with behaviour factor (q) stands for redundancy factor. In Strong Column Weak Beam (*SCWB*) criteria as mentioned in Table 1 of EC8, ΣM_{Rc} and ΣM_{Rb} are the sum of the design values of moments of resistance framing the joint of the columns and beams, respectively. However, in *SCWB* criteria of AISC, ΣM^*_{pc} is the sum of moments in the column above and below the joint at the intersection, and ΣM^*_{pb} is the sum of moments in the beams at the intersection of the beam and column centrelines as defined by AISC.

For second order criteria, described in Table 2 in Eurocode, P_{tot} is the total vertical load acting on the level under consideration; d_r is the design story drift resulting from V_{tot}, where V_{tot} is the total seismic storey shear force, h is the inter-storey height. In AISC-ASCE the C_d factor is introduced, it being called deflection amplification factor, while Δ is the storey drift resulting from V_x, V_x is seismic shear acting between levels x and x-1 and h_{sx} is the story height below level x, P_x is the total gravity load at and the above storey in the seismic design situation.

Table 2. Deformability related parameters and checks for EC3-EC8 and AISC-ASCE provisions

Description	*Eurocodes (EC3/EC8)*	*AISC/ASCE*	*Remarks*
Second order effects	A simplified procedure is allowed by amplifying computed seismic forces and displacements by a factor *1/(1- θ),* where $\theta = \frac{P_{tot} \times d_r}{V_{tot} \times h}$ but 0.1< θ ≤0.2. In any case, θ may not exceed 0.3	$\theta = \frac{P_x \times \Delta}{V_x \times h_{sx} \times C_d}$ and $\theta_{\max} = \frac{0.5}{\beta C_d}$ if $\theta > 0.1$, use θ_{max}, where β is the ratio of shear demand to shear capacity (conservatively it can be taken as 1.0)	*The factor θ is used to classify the structures into sway and non-sway frames*
Drift philosophy (Reduction)	Spectrum is reduced by 2.0 and 2.5 for importance classes I & II, and III &IV, respectively	Reduction factor is (C_d/R)(5.5/8=1.45) for SMF and (4.5/4=1.125) for IMF	*Overall EC8 check for drift is more stringent*
Drift criteria for MRFs (Limit)	0.005*h*, 0.0075*h and* 0.01*h*, where *h* is the storey height	0.02*h*, where *h* is the storey height	

3. THE CASE STUDY

3.1. Building description

In order to investigate the design criteria and thus the capacity design rules of moment resisting frames according to the two codes, case study is conducted on 9-storeys office building using typical floor plan of SAC 9-storey building, measuring 45.75m in both directions.

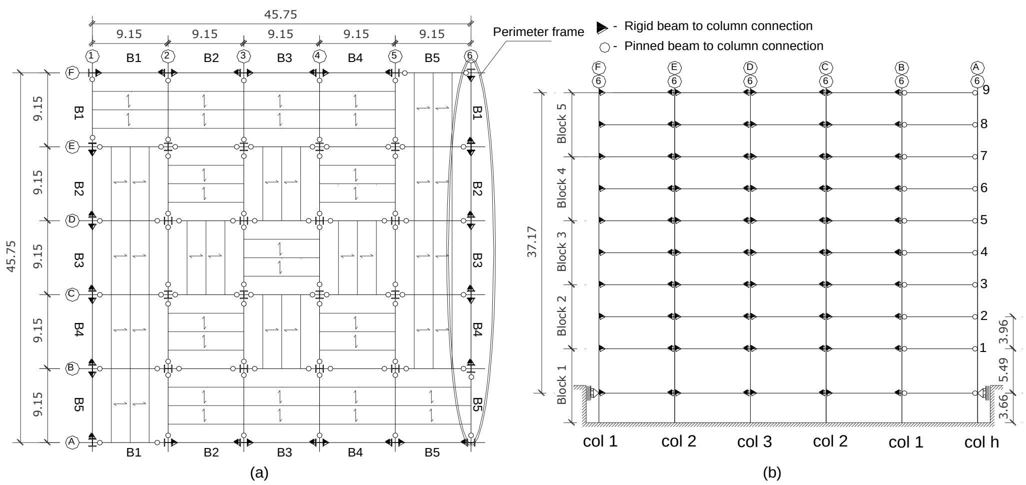

Figure 1. (a) Typical floor plan of the building with perimeter MRFs and (b) perimeter frame elevation

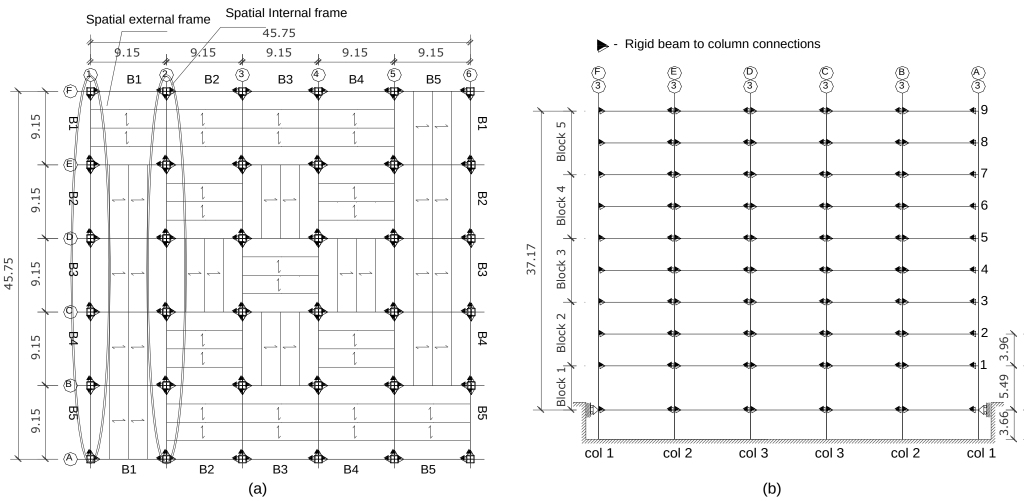

Figure 2. (a) Typical floor plan of the building with spatial MRFs and (b) spatial frame elevation

The typical floor plan of the building with the indication of perimeter frame is shown in Fig. 1a, and its elevation in Fig. 1b. Similarly spatial frame is shown in Fig. 2a, and its elevation in Fig. 2b. The columns of both frame configurations (perimeter and spatial) are designed considering five blocks. The inter-storey height of the ground floor is 5.49m whereas it is 3.96m for the rest of storeys, thus giving rise to an overall height of 37.17m of the building. Since the contribution of exterior spatial frame to gravity loading is less than the interior frame, as well the exterior frame contribute more to the lateral loading due to the torsional effects, therefore these two frames are designed separately. The exterior frame at grid 1-1 is designed, whereas the interior frame at grid 2-2 is designed as shown in Fig.2a. The three frame configurations with two aforementioned codes give 9 cases (see Table. 3), which are designed and analysed.

Table 3: Analysed cases

No.	*Frame design with EC3/EC8*	*No.*	*Frame design with EC3/EC8*	*No.*	*Frame design with AISC/ASCE*
1	Perimeter-DCH	4	Perimeter-DCM	7	Perimeter-SMF
2	External Spatial-DCH	5	External Spatial-DCM	8	External Spatial-SMF
3	Internal Spatial-DCH	6	Internal Spatial-DCM	9	Internal Spatial-SMF

3.2. Design criteria

Vertical loads acting on the structure are evaluated according to EC0 (EN-1990, 2002) and EC1 (EN-1991-1-1, 2004), providing as a result a total gravity loading (structural and non-structural) equal to 4.6 kN/m^2 for roof and 7.8 kN/m^2 for typical floor; these includes imposed load of 0.4 kN/m^2 and 3.0 kN/m^2 for roof and typical floor, respectively. The flooring system is composed of COMFLOR-46 (COMFLOR-46, 2012), using A252 mesh, and is comprised of 145mm thick concrete slab and 0.9mm steel sheeting. The masses according to EC8 for spatial and perimeter frames at typical floor level are 193 kN-sec^2/m and 589 kN-sec^2/m, respectively, while for roof these are 164 kN-sec^2/m and 491 kN-sec^2/m, respectively. In the case of ASCE the corresponding masses at typical floor for spatial and perimeter frames are 197 kN-sec^2/m and 592 kN-sec^2/m respectively, while they are 153 kN-sec^2/m and 459 kN-sec^2/m for roof. Based on the provisions of EC3 and EC8, the primary beams are designed in order to satisfy both the ultimate and serviceability limit states using steel grade S-275 (see Table. 4). Accordingly, when using the provisions of AISC/LRFD (ANSI/AISC-360-10, 2010) together with ASCE for the combination of gravity loads, in order to have the same effects on the beams, the same loads are assumed as defined by EC1.

Table 4. Designed primary beams for spatial and perimeter frames using EC3/EC8 and AISC/ASCE

Frame	*Floor*	*Beam B1*	*Beam B2*	*Beam B3*	*Beam B4*	*Beam B5*
Perimeter-AISC/ASCE	9,8,0	IPE600	IPE600	IPE600	IPE600	Hinge
	7,6,5,4,3,2,1	HE700A	HE700A	HE700A	HE700A	Hinge
Perimeter-EC3/EC8	9,8,0	IPE600	IPE600	IPE600	IPE600	Hinge
	7,6,5,4,3,2,1	HE700A	HE700A	HE700A	HE700A	Hinge
Spatial-AISC/ASCE (External)	9,8,7,6,5,0	IPE450	IPE450	IPE450	IPE450	IPE600
	4,3,2	IPE600	IPE600	IPE450	IPE600	IPE600
	1	HE600A	HE600A	IPE600	HE600A	HE600A
Spatial-EC3/EC8 (External)	9,8,7,6,0	IPE500	IPE500	IPE500	IPE500	IPE600
	5,4	IPE500	IPE500	IPE500	IPE500	HE600A
	3,2	HE600A	HE600A	HE600A	HE600A	HE600A
	1	HE700A	HE700A	HE700A	HE700A	HE700A
Spatial-AISC/ASCE (Internal)	9	IPE550	IPE400	IPE400	IPE400	IPE550
	8,7,6,5,4,3,2,0	HE450A	IPE550	IPE550	IPE550	HE450A
	1	HE450A	HE450A	HE450A	HE450A	HE450A
Spatial-EC3/EC8 (Internal)	9	IPE600	IPE500	IPE500	IPE500	IPE600
	8,7,6,5,4,3,2,0	HE500A	IPE600	IPE600	IPE600	HE500A
	1	HE600A	HE600A	HE600A	HE600A	HE600A

The beams for perimeter frame and spatial external frames in both codes are mostly designed for seismic conditions, whereas all the beams for the spatial internal frames in EC3/EC8 are designed for gravity loads while some of the beams in AISC/ASCE are designed for seismic condition as well. The reference frames are designed according to EC8 with DCH (q=6.5) and DCM (q=4.0), assuming type C soil stratigraphic profile (dense sand or gravel or stiff soil), important class II (γ_I=1.0), type 1 elastic response spectrum and 0.25g peak ground acceleration. In order to allow an apparent comparison and to have the same seismic intensity, an equivalent response spectrum for AISC/ASCE is adopted, using importance factor 1.0, and considering soil type B with S_s and S_1 as 1.07g and 0.57g, respectively. According to ASCE, a seismic category needs to be assigned for the structure, which is found to be in category D (High seismic category) from SDS (0.713) and SD1 (0.38) with the assumed site class.

4. FRAME DESIGN AND ANALYSIS

Initially, a linear modal dynamic analysis (SAP2000, 2010) is developed for the purpose of seismic design of the frames; then pushover analysis is used in order to check the performance of the frames. The fundamental period of vibration from the codified formulation is found 1.3*sec* which, is almost 50% lower than the modal response spectrum analysis (see Table 5).

Table 5. Fundamental period and design base shear following EC3/ EC8

Frame	*Ductility*	*Mass/frame [kN-sec^2/m]*	*T(modal) [sec]*	$V_{d\text{-}static}$ *[kN]*	V_d *[kN]*	Ω	$1.1\,\gamma_{ov}\,\Omega$
Perimeter-DCH	High	5790	2.41	4632	3380	1.50	2.07
Perimeter-DCM	Medium	5790	2.41	4816	3740	1.41	1.94
Spatial-external (1.6)-DCH	High	1930	2.09	1544	1118	1.81	2.49
Spatial-external (1.6)-DCM	Medium	1930	2.09	1635	1293	1.68	2.33
Spatial-internal (1.36)-DCH	High	1930	2.17	1312	864	1.74	2.39
Spatial-internal (1.36)-DCM	Medium	1930	2.14	1390	1107	1.67	2.30

When using the AISC/ASCE code, the fundamental period obtained from the codified formulation is found to be 1.30 *sec*, which is definitely lower than the one obtained by modal analysis (see Table. 6); in such circumstances code specifies that scaling factors for the design forces and drift have to be applied, as shown in Table 6.

Table 6. Fundamental period and base shears following AISC/ASCE

Frame	*Mass [kN-sec^2/m]*	*T(modal) [sec]*	$V_{d\text{-}static}$ *[kN]*	$V_{d\text{-}scaling}$ *[kN]*	V_d *[kN]*	*Scaling factors*		V_d *[kN]*
						Force	*Drift*	
Perimeter	5787	2.71	3333	2406	1340	1.53	1.53	2050
Spatial-external (1.60)	1926	2.63	1109	802	462	1.48	1.48	684
Spatial-internal (1.36)	1926	2.69	943	682	373	1.56	1.56	582

Design static base shear (V_d) is calculated using ASCE criteria, for which the minimum seismic response coefficient (C_s) is found equal to 0.036g. It is important to note that for calculating scaling factors, period is computed using C_uT_a (1.8 sec) and thus C_s is found to be 0.026g. Further it is to be highlighted that in order to take into account the torsion effect, the response spectra in both the codes are amplified by a factor δ (according to the simplified formulation of EC8). This leads to amplify the seismic forces on perimeter and external spatial frames by 1.6 and the internal spatial frame by 1.36. Minimum values of Ω are used according to EC8 which are smaller than those recommended by AISC/ASCE (as Ω_o is 3). The obtained columns cross sections using S-275 steel grade of spatial and perimeter frames following EC8/EC3 and AISC/ASCE prescriptions are shown in Table 7. For the inter-storey drifts, as per EC8 limit, 0.01h is considered, while according to AISC/ASCE, as recommended, a limit of 0.02h is used as shown in the corresponding graph. Almost the same profiles are obtained for all the frames designed according to EC8 with DCH and DCM (stress level of columns designed with DCM are higher than DCH). Only in the case of internal spatial frame, DCM influences the dimension of the col2 and col3 at the second block.

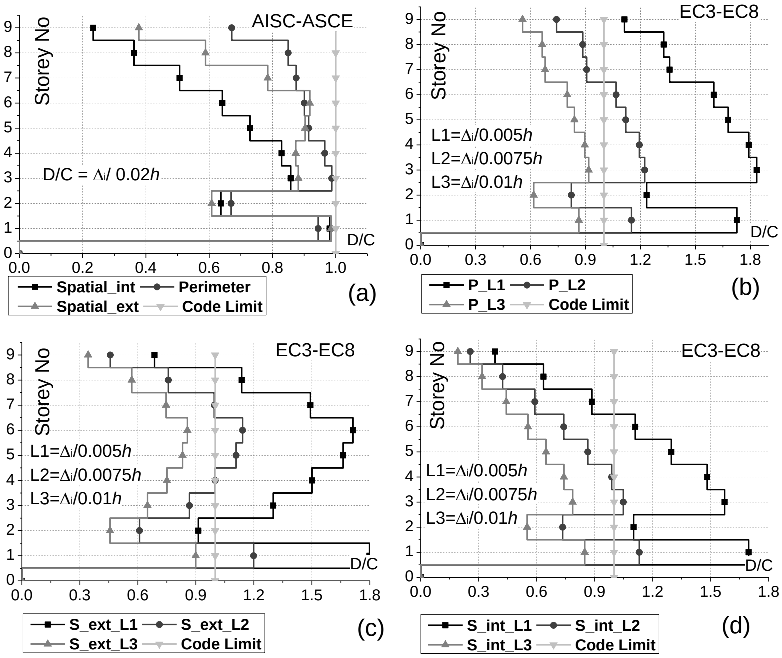

Figure 3. Design requests with respect to drift: (a) AISC/ASCE frames, (b) EC3/EC8 perimeter frame, (c) EC3/EC8 spatial external frame and (d) EC3/EC8 spatial internal frame

Fig. 3(a) shows *D/C* ratios according to drifts for all three frames configurations designed according to AISC/ASCE. The perimeter frame is optimized for satisfying drift requirement at some storeys. Fig. 3(b) and Fig. 3(c) show external and internal spatial frame designed according to EC3/EC8; it is evident that the strict limits (0.005*h* and 0.0075*h*) of EC8 are not respected, while the frames accomplish with design limit 0.01*h* (De Matteis, 2005). In Fig. 4 and in Fig. 5 the *D/C* ratios of the entire designed frames according to strength are depicted.

Table 7. The obtained columns profiles of designed frames

		Perimeter frame		*Spatial frame (External)*		*Spatial frame (Internal)*	
Col	*Block*	*AISC/ASCE (SMF)*	*EC3/EC8 (DCH/DCM)*	*AISC/ASCE (SMF)*	*EC3/EC8 (DCH/DCM)*	*AISC/ASCE (SMF)*	*EC3/EC8 (DCH/DCM)*
1	1	[1]HE1200	[1]HE1200*	[3]CR-600M	[3]CR-550M	[3]CR-600M	[3]CR-550M
	2	[2]HE1100	[2]HE1100	[3]CR-550B	[3]CR-550B	[3]CR-550B	[3]CR-500B
	3	[2]HE1100	[2]HE1100	[3]CR-500B	[3]CR-500B	[3]CR-500B	[3]CR-450B
	4	HE1000M	HE1000M	[3]CR-500B	[3]CR-450B	[3]CR-400B	[3]CR-400B
	5	HE900B	HE800B	[3]CR-500B	[3]CR-400B	[3]CR-400B	[3]CR-400B
2	1	[1]HE1200	[1]HE1200*	[3]CR-550M	[3]CR-550M	[3]CR-600M	[3]CR-550M
	2	[2]HE1100	[2]HE1100	[3]CR-550B	[3]CR-550M	[3]CR-600B	[3]CR-500B[DCM1]
	3	[2]HE1100	[2]HE1100	[3]CR-500B	[3]CR-500M	[3]CR-500B	[3]CR-500B
	4	HE1000M	HE1000M	[3]CR-500B	[3]CR-450B	[3]CR-450B	[3]CR-500B
	5	HE900B	HE800B	[3]CR-500B	[3]CR-450B	[3]CR-450B	[3]CR-500B
3	1	[1]HE1200	[1]HE1200*	[3]CR-550M	[3]CR-550M	[3]CR-600M	[3]CR-550M
	2	[2]HE1100	[2]HE1100	[3]CR-550B	[3]CR-550B	[3]CR-600B	[3]CR-500B [DCM2]
	3	[2]HE1100	[2]HE1100	[3]CR-500B	[3]CR-500B	[3]CR-500B	[3]CR-500B
	4	HE1000M	HE1000M	[3]CR-500B	[3]CR-450B	[3]CR-450B	[3]CR-500B
	5	HE900B	HE800B	[3]CR-500B	[3]CR-450B	[3]CR-450B	[3]CR-500B

Note: *[1]HE1200 means(in mm) depth 1200, width 352, web thickness 40, flange thickness 85 ,[2]HE1100 means depth 1100, width 325, web thickness 40, flange thickness 80, [3]CR denotes cruciform profile where the European standard profiles are welded orthogonally,*
** denotes steel grade S420, [DCM1] and [DCM2] denotes column profile CR-500M and CR-550B as they are required when dealing with DCM*

Nominally, an optimal design should provide *D/C* ratios just less than unity; however, this is not possible due to limited number of available profiles and also because these frames are designed considering five blocks therefore causing over-sizing of the profiles at the next storeys. In addition, drift criteria, capacity design rules with *SCWB* produce and overstrength also reflect on the *D/C* ratios.

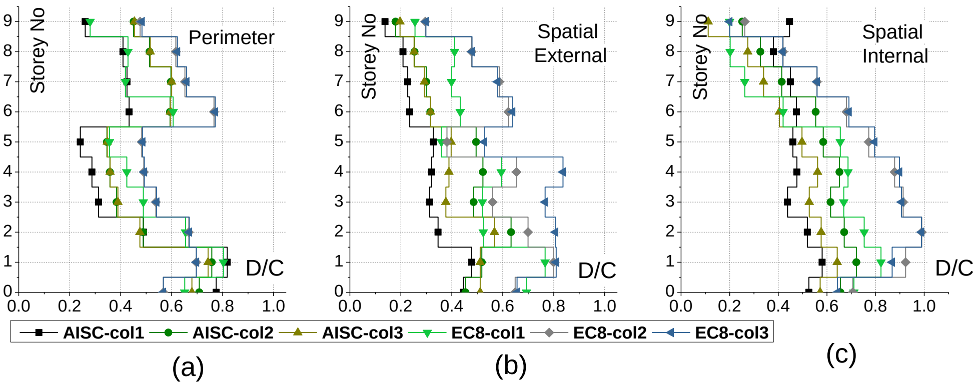

Figure 4. Design requests with respect to strength according to EC8 (DCH only) and AISC (a) Perimeter frames, (b) Spatial external frame and (c) Spatial internal frame

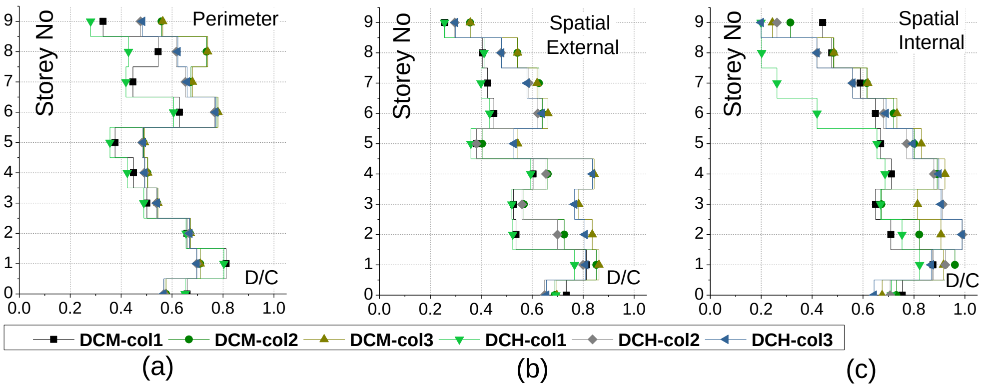

Figure 5. Design requests with respect to strength according to EC8 using DCH and DCM (a) Perimeter frames, (b) Spatial external frame and (c) Spatial internal frame

5. PUSHOVER ANALYSIS

Static pushover analysis has been carried out for checking the lateral load resisting performance of the frames. For this reason triangular distribution (unit load at roof level) of static incremental loads has been applied and the displacement at the roof level has been controlled. FEMA 356 (FEMA, 2000) acceptance criteria for non-linear procedure are adopted here. The obtained structural capacity curves are plotted in Fig. 6 for all the analysed frame configurations in terms of total base shear (V_b) versus top displacement (D_t).

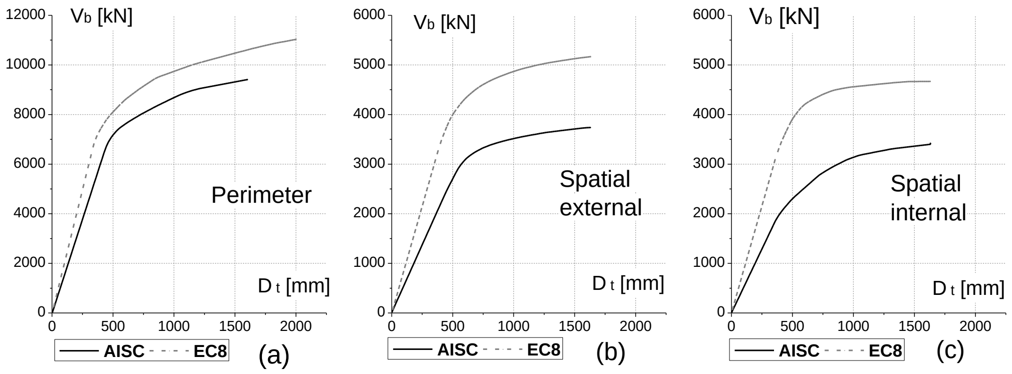

Figure 6. Pushover curves of frames: (a) Perimeter, (b) Spatial external and (c) Spatial internal

The frames designed with EC3/EC8 (DCH only) show higher performance than the ones of

AISC/ASCE. It is mainly due to the fact that under the same forces the EC8 drift limit (0.01h) results more stringent than the AISC drift limit (0.02h), also considering that in EC8 the elastic spectrum is reduced by 2.0 to allow for the lower return period of the seismic event related to the damageability limit state, whereas in AISC the elastic spectrum is reduced by 1.45 excluding the drift scaling factor. The same results had been obtained previously by the authors for a different frame configuration where drift limit of EC8 were found to be very stringent.

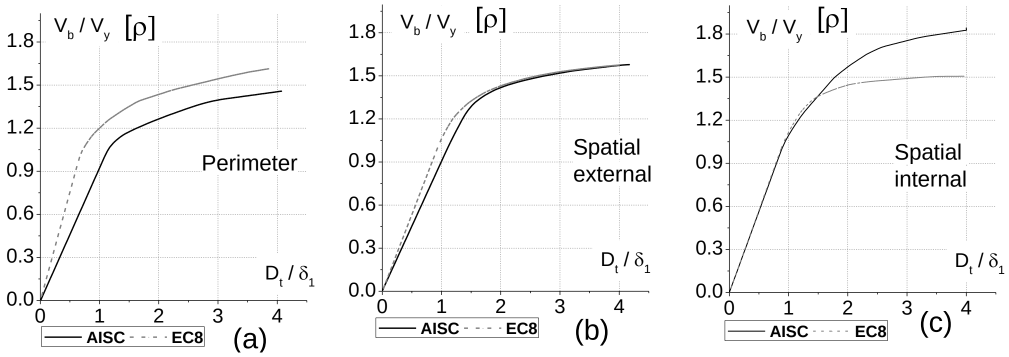

Figure 7. Pushover curve of frames normalized to V_y (a) Perimeter, (b) Spatial external and (c) Spatial internal

Also, the internal spatial frames shows a lower performance than the external ones, due to the fact that in the case of internal frames gravity loads has determinant effect on the design of beams whereas the column dimension remains almost the same. Fig. 7 shows the redundancy factors of the analysed frame configurations, where the maximum redundancy factor is about 1.5.

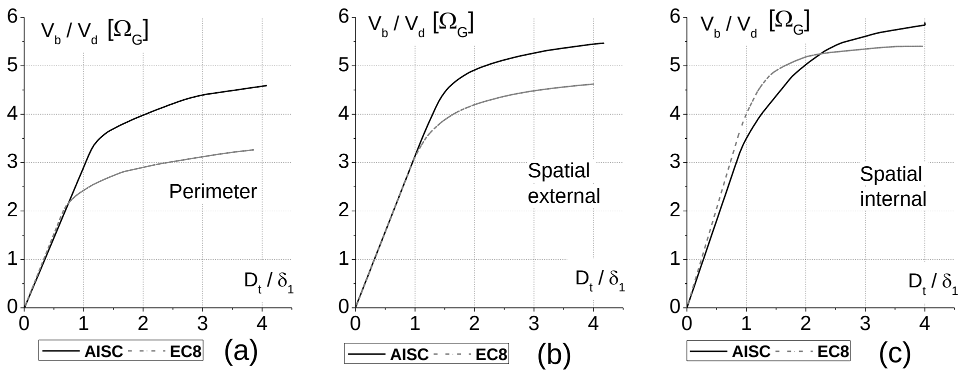

Figure 8. Pushover curve of frames normalized to V_d (a) Perimeter, (b) Spatial external and (c) Spatial internal

Fig. 8 shows the overstrength factor of the frames, which increases as the influence of gravity loading governing the design of frame increases. It is low as 3 to 4.5 for perimeter frames whereas it is in the range of 5.0 to 6.0 for internal spatial frames. In fact, since gravity loadings control the design of beams in EC3/EC8, these beams are larger than the beams obtained with AISC/ASCE loading conditions, also helping to satisfy drift criteria. In general, the high overstrength factor in spite of the material overstrength factor in both codes proves the increase of member dimensions due to flexibility of the frames (drift control and period control) as well as to the application of the *SCWB* criteria.
Fig. 9a show the value the design base shear forces where Fig. 9b show overstrength and redundancy factors of the analysed frames, which for a better comparison are also reported in Table 8.
It can be noted that the redundancy factor (ρ) is higher for perimeter frame when designed with EC8 while the elastic overstrength (Ω_E) is lower. The same result can be observed for spatial frames where AISC frames yield high overstrength.

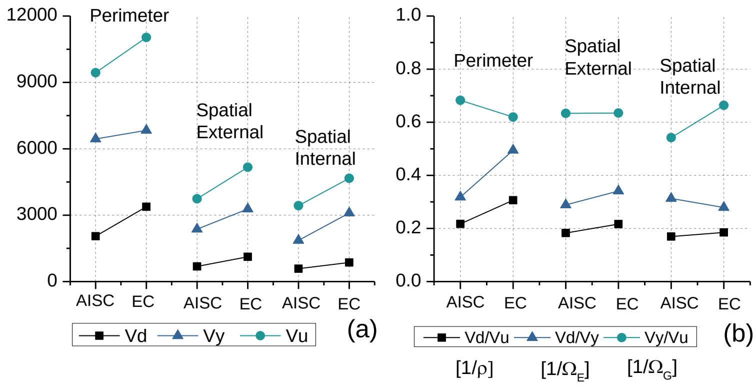

Figure 9. Comparative parameters in the two codes: (a) Base shear [kN], (b) D/C of base shear among different frame configurations

This is due to the fact that the seismic forces are reduced by 8 in AISC rather than by 6.5 in EC8 and at the same time the drift limit are not so stringent in AISC in comparison to EC8. In both codes, the design of beams had drastic effect on the performance of perimeter frames. In EC8 the beams are mainly designed for seismic combination as the seismic forces are reduced by 6.5 (V_d = 3380kN); then these forces are increased for columns by an overstrength of 2.07. On the contrary, in AISC seismic forces have a lower effect on the design of the beams as a reduction factor equal to 8 is assumed (V_d = 2050kN); then the seismic forces are increased for the design of columns by an overstrength factor of 3.

Table 8: EC8 and AISC redundancy and overstrength factors

Frame	V_d *[kN]*	V_u *[kN]*	V_y *[kN]*	V_u/V_y *[ρ]*	V_y/V_d *[Ω_E]*	V_u/V_d *[Ω_G]*
Perimeter-AISC/ASCE	2050	9440	6445	1.50	3.14	4.60
Perimeter-EC3/EC8-DCH	3380	11030	6835	1.60	2.02	3.30
Perimeter-EC3/EC8-DCM	3740	11030	6835	1.60	1.83	2.90
Spatial-AISC/ASCE (External)	684	3740	2369	1.60	3.46	5.50
Spatial- EC3/EC8-DCH (External)	1118	5167	3279	1.60	2.93	4.60
Spatial- EC3/EC8-DCM (External)	1293	5167	3279	1.60	2.54	4.00
Spatial- AISC/ASCE (Internal)	582	3431	1860	1.80	3.20	5.90
Spatial- EC3/EC8-DCH (Internal)	864	4668	3098	1.50	3.59	5.40
Spatial- EC3/EC8-DCM (Internal)	1107	4765	3186	1.50	2.88	4.30

Note: *V_u/V_y shows redundancy factors whereas V_u/V_d shows overstrength factors [$\Omega_G = \rho \times \Omega_E$]*

6. CONCLUSIONS

The main outcomes of the case study may be synthesised as follows:
a) The perimeter frame of EC8 gives higher performance as the design base shear is higher compared to AISC/ASCE. b) Together with the overstrength factors, V_d in the case of EC8 resulted to be 6997 kN (for the design of columns), whereas it is equal to 6150 kN in the case of AISC/ASCE; therefore the columns for EC8 are heavier than AISC, which in return provide higher performance for the frames designed according to EC8. c) As the beams dimensions are practically the same for both codes, being the columns heavier in EC8, the AISC frame configuration provides lower elastic base shear V_y (6455 kN) and ultimate base shear V_u (9440 kN) compared to EC8, for which V_y and V_u are recorded as 6834 kN and 11030 kN, respectively. This yields to give less redundancy factor (9440/6455=1.46) for AISC than EC8 (11030/6834=1.61). d) The global overstrength for AISC perimeter frame is 4.6 (=9440/2050), which is significantly greater than the one obtained for EC8 frame (3.2=11030/3380). Therefore, based on the obtained results, the following general conclusions may be drawn:

- The drift limits of EC8 are very stringent, thus influencing the capacity design approach, even though the largest limit (0.01*h*) of EC8 is applied;
- In both codes perimeter frame gives higher performance than the spatial frames;

- Internal spatial frames gives lower performance than the external spatial frames, as normally gravity load governs the design of beams;
- The gravity loading has great influence on the overstrength factors, it increasing as the influence of gravity loading governing the design of frame increases;
- AISC/ASCE frames show higher overstrength due to high *R* factor and less stringent drift limits;
- The ductility class of EC8 has an insignificant influence on the member cross section dimensions especially for perimeter frames.

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