

# Performance of Rigid Steel Frames under Adequate Soil Conditions Using Seismic Code Provisions

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## Abstract

This study is focused on nonlinear analysis and design of spatial and perimeter moment resisting frames for a 9-storeys office building having 9.15 m span. Seismic design criteria of Eurocode 8 Ductility Class High (DCH) with behavior factor ( $q$ ) of 6.5 and AISC/ASCE code, Special Moment resisting Frame (SMF) with response modification factor ( $R$ ) of 8 were employed. The design outcomes are expressed in terms of frame performance (non-linear analysis), section profiles (code recommendations), strength-demand to capacity ratios, drift-demand to capacity ratios and structural weight. The consequences of the research compare two codes in term of weights and design performances. This will aid professional engineers and researchers to select effective design criteria and capacity design rules efficiently.

## Keywords

Eurocodes, AISC, ASCE, FEMA, Moment Resisting Frames, Seismic Resistance, Pushover Analysis

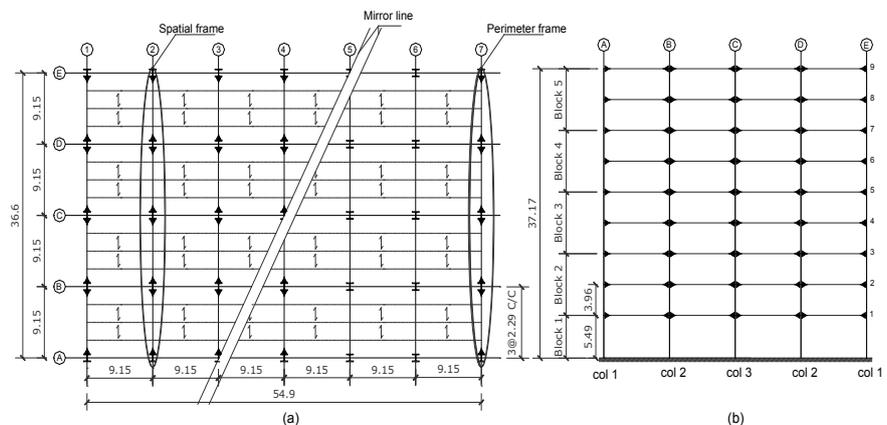
## 1. Introduction

Conventional structural design of buildings considers two limit states, the Ultimate Limit State (ULS) and the Serviceability Limit State (SLS). The ULS is used for strength criteria whereas SLS for deflection vibration and human factors. Furthermore, for structures vulnerable to strong or moderate seismic action, modern building codes [1] [2] [3] allow inelastic deformations but strictly limiting the development of unreliable mechanisms that could impair the global mechanisms of the structure. This all can be achieved through the smart use of capacity design approaches. For obtaining global ductility and better performance

of a structural system, dissipative (structural fuses) and non-dissipative zones are defined by the codes. This obliged the structural design that non-dissipative zones should remain in the elastic field whereas the dissipative ones should undergo large inelastic deformations. To control such a global structural behavior, codes provide the so-called criterion of capacity design where non-dissipative members are designed for comparatively higher seismic forces than dissipative members and dissipative members are kept at such locations that will fail before the brittle members and subsequently will protect non-ductile elements by overstressing [4]. Many researches have been investigated these factors, such as Uang *et al.* [5]; who established  $R$  (or  $R_w$ ) and  $C_d$  factors for building seismic provisions. Rahgozar and Humar [6], assessed the extent of reserve strength attributable to redistribution in steel frames. Bruneau *et al.* [7] and Mitchell *et al.* [8] included material effects caused by higher yield stress compared with the nominal value  $R_{yield}$ . In view of the above MRFs are designed in such a way that inelastic behavior is accommodated through plastic hinging within the spans of the beams rather than the columns, allowing the dissipation of energy in the beam ends and therefore the internal forces of columns members are increased by an over strength factor [4] [9]. The columns therefore behave elastically during a seismic event and allowed to carry the gravity loadings during a post seismic event [10].

## 2. Design Case

In order to investigate the design criteria of moment resisting frames according to the two codes the current study is conducted on 9 stories office building. The building has a rectangular plan measuring 54.9 m by 36.6 m in longitudinal and transversal direction, respectively. The typical floor plan of the building with the indication of spatial and perimeter frame is shown in **Figure 1(a)**. The elevation of frame is given in **Figure 1(b)** where the outer columns are named as “col1” and the inner columns as “col2” and “col3”. The columns are designed considering five blocks; the first block consists of single story whereas the rest of the



**Figure 1.** (a) Typical floor plan of the building and (b) Frame elevation.

four blocks consist of 2 stories each. The inter story height of the base floor is 5.49 m whereas the other stories are 3.96 m in height giving rise to an overall height of 37.17 m [11] [12] [13].

Vertical (gravity) loads acting on the structure are evaluated according to EC0 [14] and EC1 [15] providing as a result a total gravity loading (structural and non-structural) equal to  $6.0 \text{ kN/m}^2$  and an imposed load of  $2.0 \text{ kN/m}^2$ . The secondary beams are simply supported having a bay width of 2.29 m oriented along the longitudinal direction of the building. The masses at each floor level according to EC8 [1] for spatial and perimeter frames are found  $221 \text{ kN-sec}^2/\text{m}$  and  $664 \text{ kN-sec}^2/\text{m}$  respectively, whereas these correspond to  $218 \text{ kN-sec}^2/\text{m}$  and  $655 \text{ kN-sec}^2/\text{m}$  respectively in the case of ASCE [2]. Based on the provisions of EC3 [16] and EC8, the primary beams are designed for ultimate and serviceability limit states (see Table 1) using steel grade S-275. These primary beams are initially designed for gravity loading and then checked with respect to the seismic loading condition.

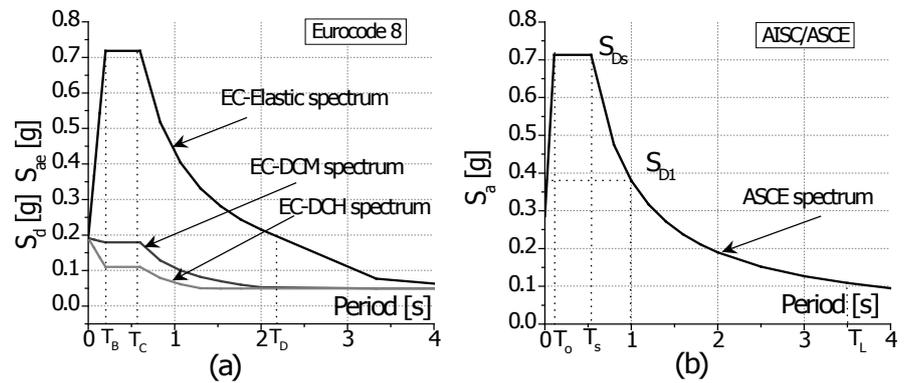
Accordingly for the sake of comparison when considering the provisions of AISC/LRFD [17] with AISC/ASCE for the combination of gravity loads (in order to have the same effects on the beams) the same loads are assumed as evaluated according to EC1.

The reference frames are designed according to EC8 with DCH ( $q = 6.5$ ) assuming type C soil stratigraphic profile, important class II ( $\gamma_I = 1.0$ ), type 1 elastic response spectrum and 0.25 g 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  $S_I$  as 1.07 g and 0.57 g, respectively. Selected soil conditions for both codes and respective geotechnical descriptions [18] are expressed in Table 2. While, the corresponding response spectra for EC8 and ASCE is shown in Figure 2.

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  $S_{Ds}$  (0.713) and  $S_{D1}$  (0.38) with the assumed site class. This code limits the use of multi-story

**Table 1.** Designed primary beams for spatial and perimeter frames.

Type of frame	Ductility	EC8/EC3		ASCE/ASCE			
		Floor	Type of profile	Type	Floor	profile	
Spatial	High	1~9	IPE600	SMF	1~9	IPE600V	
		1	IPE600O		1		IPE600O
		2~4	IPE600V		2~4		IPE600V
Perimeter	High	5,6	IPE600O	SMF			
		7	IPE600		5~9	IPE600	
		8,9	IPE500				



**Figure 2.** Response spectra for: EC8 (a) and ASCE (b).

**Table 2.** Soil Conditions for Design criteria.

Building Code	Code Specified Soil Type	Code Specified Soil Description	Geotechnical Description & (Unified Soil Classification)
Eurocode 8	C	Deep deposits of dense or medium dense and, gravel or stiff clay with thickness from several tens to many hundreds of metres.	Medium to dense Gravels, silty gravels and silty-clayey Gravels (GP,GM,& GC-GM) or
			Medium to dense Sands, silty Sands and silty-clayey Sands (SP,SM & SC-SM) or Stiff to hard Silts (ML) or Stiff to Hard Lean Clays and silty-Clays (CL & CL-ML)
ASCE	B	Rock with Medium Strength	Several possibilities, such as Weathered Shale, Limestone and Sandstone

IMFs in seismic category *D* up to 10 m height therefore the design is carried out considering only SMF ( $R = 8$ ).

### 3. Design Results

Linear modal dynamic analysis [19] is developed for the seismic design of the frames. It has to be observed that the fundamental period of vibration from the codified formulation is found 1.2 sec which is almost 50% lower than the modal response spectrum analysis (see Table 3). The reduction in the period is due to the fact that simplified formulae given by seismic codes tend to underestimate the fundamental period of vibration, being based on empirical evaluation. The codified formulae globally accounting also for stiffening effects of non-structural elements e.g. partition walls and infills etc. These effects are obviously of major importance for steel frames which exhibits relatively low horizontal stiffness. The underestimation of the natural period (considering only short branch of spectrum) leads to conservative design assumptions e.g. higher design acceleration (consequently high seismic base shear) and in turn larger inter story drifts [10].

When using the AISC/ASCE code the fundamental period obtained from the codified formulation is found to be 1.14sec, definitely lower than the one obtained by modal analysis (see **Table 4**). In this case code specifies scaling factors to be applied for the design forces and drift calculation as shown in **Table 4**. Design static base shear ( $V_d$ ) is calculated using ASCE criteria where minimum seismic response coefficient ( $C_s$ ) is found equal to 0.036.

The over strength factor ( $\Omega$ ) is 1.24 for spatial frame; it is 1.13 for perimeter frame. In EC8  $\Omega$  depends on the plastic resistance of beams and on the applied seismic forces. The obtained “ $\Omega$ ” values from EC8 design are further increased with 1.10 and  $\gamma_{ov}$ . A minimum value of  $\Omega$  is used according to EC8 which are quite smaller than those recommended by AISC/ASCE (as  $\Omega_o$  is 3).

The obtained columns cross sections using S-355 steel grade of spatial and perimeter frame following EC8/EC3 and AISC/ASCE prescriptions are shown in **Table 5**. It should be noted that when using EC8 the serviceability limit state condition governs the design. With reference to the AISC/ASCE provisions strength criteria are more stringent, especially due to the fact that the seismic forces are increased by an over strength factor equal to 3. In addition in order to assure the SCWB criteria both codes provide a supplementary check for frame structures which had some influences on the column profiles, this check is normally not satisfied for the top story of the building.

An elastic analysis with DCL ( $q = 1.5$ ) for EC8 and  $R = 3$  for AISC/ASCE will allow more consistent and lighter design (by avoiding the capacity design criteria and SCWB criteria) but both the codes are not of the opinion to allow the use of this criterion (elastic analysis) for the assumed seismic zone ( $PGA = 0.25$  g for EC8 and seismic category  $D$  for AISC/ASCE).

In order to allow an assessment between the analyzed frame configurations **Figure 3(a)** and **Figure 3(b)** show the “demand” to “capacity ratios” ( $D/C$ ) of columns strength and  $D/C$  ratios of drift for both perimeter and spatial frame configurations designed according to EC8 and AISC/ASCE. Nominally, an optimal design should provide  $D/C$  ratios just less than unity; nonetheless this is not possible due to limited number of available profiles and also because these

**Table 3.** Fundamental period and design base shear following EC8.

Frame	Ductility	Mass [kN-sec <sup>2</sup> /m]	T(modal) [sec]	$V_d$ [kN]
Spatial	High	1989	2.4	821
Perimeter	High	5976	3.7	2346

**Table 4.** Fundamental period and base shears following AISC/ASCE for SMF

Frame	Mass [kN-sec <sup>2</sup> /m]	T(modal) [sec]	$V_b$ (modal) [kN]	$V_{static}$ [kN]	Scaling factors		$V_d$ [kN]
					Force	Drift	
Spatial	1962	1.9	449	693	1.3	1.3	584
Perimeter	5895	3.2	680	2082	2.6	2.6	1768

**Table 5.** The obtained columns profiles.

Col	Block	Perimeter frame		Spatial frame	
		AISC/ASCE	EC8	AISC/ASCE	EC8
1	1	HE1000M+4	HE1000M	HE1000M	HE800M
	2	HE1000M	HE700B	HE900B	HE650B
	3	HE1000B	HE550B	HE800B	HE550B
	4	HE900B	HE500B	HE700B	HE500B
	5	HE500B	HE450B	HE600B	HE400B
2	1	HE1000M+4	HE1000M	HE1000M+2	HE1000M+2
	2	HE1000M	HE700B	HE900B	HE650B
	3	HE1000B	HE550B	HE800B	HE550B
	4	HE900B	HE500B	HE700B	HE500B
	5	HE500B	HE450B	HE600B	HE400B
3	1	HE1000M+4	HE1000M	HE1000M+2	HE1000M+2
	2	HE1000M	HE700B	HE900B	HE650B
	3	HE1000B	HE550B	HE800B	HE550B
	4	HE900B	HE500B	HE700B	HE500B
	5	HE500B	HE450B	HE600B	HE400B

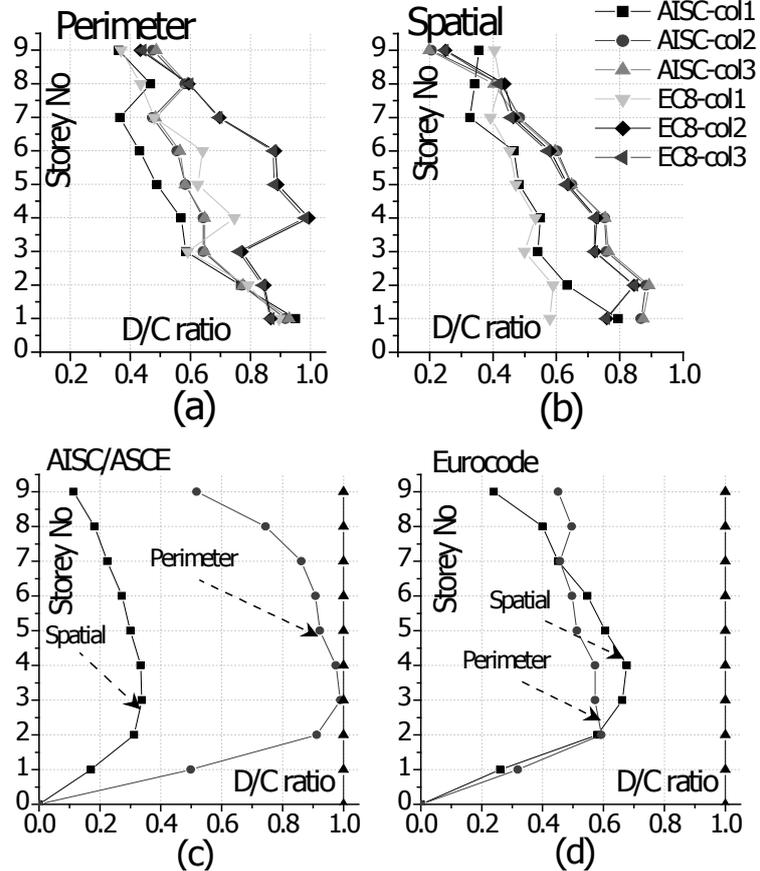
Note: HE1000M+4 and HE1000M+2 means an additional plate of 4 cm and 2 cm, respectively, to be welded with flanges of HE1000M.

frames are designed considering five blocks which causes over sizing of the profiles at the next stories. In addition drift criteria and capacity design rules produce over strength of the members reflecting on the D/C ratios as well.

For the inter story drifts as per EC8 limit 0.0075 h (for buildings having ductile non-structural elements) is considered, while according to AISC/ASCE a limit of 0.02 h is used. The drift criterion is more stringent for perimeter frame in the case of AISC/ASCE as the check is more close to the limit at certain stories. Generally the drift limits are quite far for spatial frames in the case of EC8 proving that drift controls the design in the case of perimeter frame when designed according to AISC/ASCE. **Figure 3(c)** and **Figure 3(d)** demonstrates the design requests in term of D/C ratios of drift for spatial and perimeter frames designed according to EC8 and AISC/ASCE [20] [21].

#### 4. Non Linear Analysis

In order to check the lateral performance of the frames static pushover analysis has been carried out using FEMA-350 [22] recommendations. Triangular distribution (unit load at roof level) of static incremental loads (continues from the gravity load case) has been applied and the displacement at the roof level has been controlled. Mechanical non-linearity of the members has been assumed to



**Figure 3.** Design requests according to strength (a) perimeter frames, (b) spatial frames, and drift (c) AISC/ASCE code (d) Eurocode 8.

be concentrated in plastic hinges at the ends of the beams and at the column bases.

The obtained structural capacity curves are plotted in **Figure 4(a)**, **Figure 4(b)** for all the main analyzed frame configurations in terms of total base shear ( $V_b$ ) versus top displacement ( $D_t$ ). In addition in **Figure 4(c)**, **Figure 4(d)** the ordinate  $V_b$  is normalized with respect to  $V_y$  (the lateral load producing the first plastic hinge), and in **Figure 5** the ordinate  $V_b$  is normalized with respect to  $V_d$  (the design base shear), while the abscissa  $D_t$  is normalized with  $\delta_1$  (the lateral displacement corresponding to the first initial yielding). All the relevant values are provided in **Table 6** and in **Figure 4** and **Figure 5**, where  $V_u$  represents the lateral load and  $\delta_u$  represents the corresponding displacement at failure. From **Figure 4(a)**, **Figure 4(b)**, it is obvious that AISC gives high base shear compared to EC8 as the structure is much stiffer mainly due to the introduction of force and drift scaling factors in the design procedure.

From **Figure 4(c)**, **Figure 4(d)**, it is evidenced that spatial frames are more redundant than perimeter frames. The over strength factors and ductility ratios measured in terms of displacement obtained by the pushover analysis are very high in the case of spatial frames. The high over strength factor in spite of the

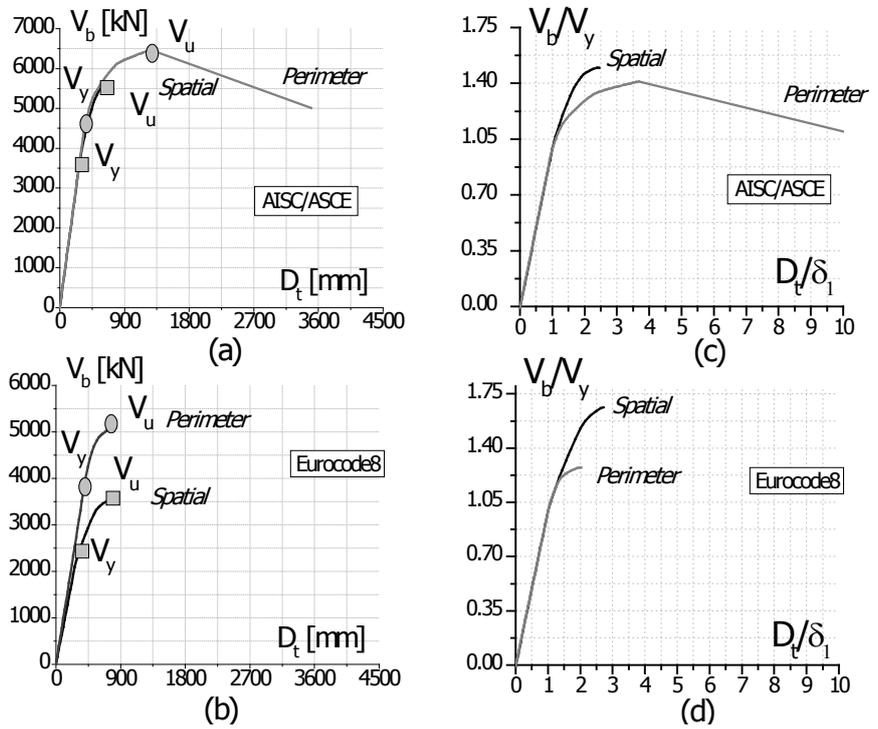


Figure 4. Pushover curves-9 stories (a) (b) Pushover curves normalized to  $V_y$  (c) (d).

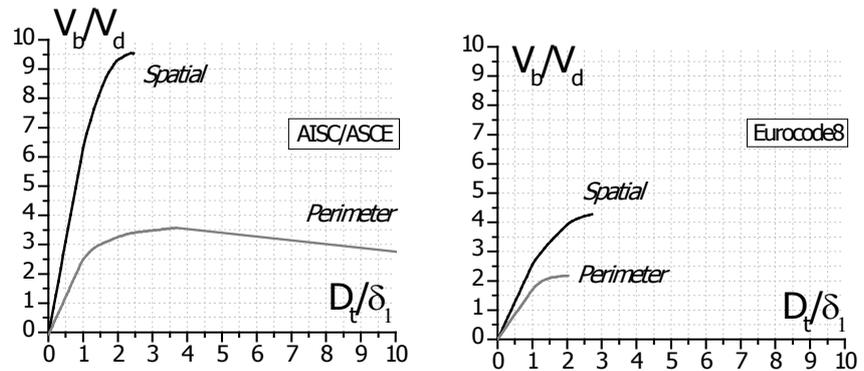


Figure 5. Pushover curves normalized to  $V_d$  for AISC/ASCE-SMF (left) and EC8-DCH (right) frame configurations.

Table 6. Redundancy and over strength factors as per EC8 and AISC.

Parameters	$V_d$ [kN]	$V_u$ [kN]	$V_y$ [kN]	$\delta_u$ [mm]	$\delta_1$ [mm]	$\delta_u/\delta_1$	$V_u/V_y$	$V_u/V_d$
P-EC8	2346	5101	3980	831	409	2.0	1.3	2.2
S-EC8	821	3505	2100	710	262	2.7	1.7	4.3
P-AISC	1768	6468	4552	1272	345	3.7	1.4	3.7
S-AISC	584	5622	3732	689	281	2.5	1.5	9.6

material over strength factor in both the codes demonstrates the increase of members dimensions due to flexibility of the frames (drift control and period

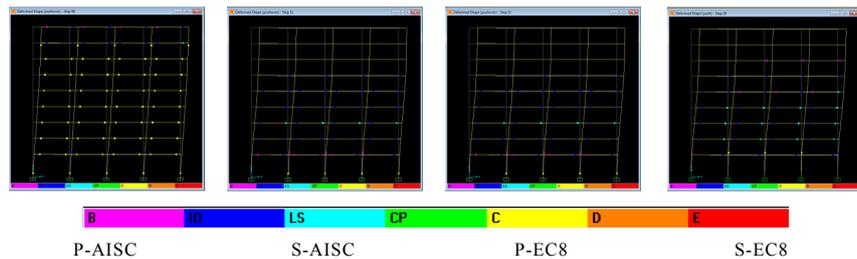
control), SCWB criteria and due to high “ $D/C$  ratios” of the members.

The abbreviations B, IO, LS, CP, C means basic, immediate occupancy, life safety, collapse prevention and collapse respectively. **Figure 6** shows the performance of the analyzed frames when plastic hinges develops. It is evident that perimeter frame designed according to AISC/ASCE gives higher performance than the spatial frame, whereas the spatial frame designed as per EC8 gives comparatively higher performance than the perimeter frame.

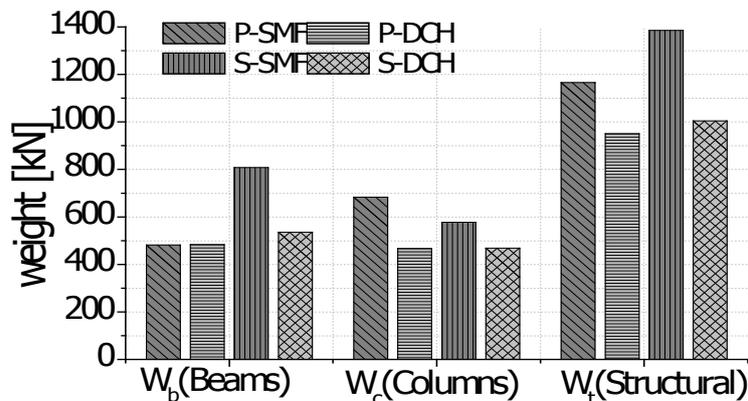
All the above aspects are also reflected in the structural weight for the designed profiles, which for the sake of comparison are provided in **Figure 7** considering separately the weight of columns ( $W_c$ ), the weight of beams ( $W_b$ ) and the total structural weight ( $W_t$ ). The variation of the weight are presented in the histograms where the overall difference in the weight of beams and columns of the two codes is due to the scaling factors adopted in AISC/ASCE, as the initial period obtained from response spectrum modal analysis was strongly connected by such a factor. As a result, from the analyzed case study it can be highlighted that AISC/ASCE gives costlier basic structural configurations.

### 5. Conclusions

The paper had dealt with the design of spatial and perimeter frames for a 9-storey office building according to the prescription of EC8 and AISC/ASCE. From the present study it is concluded that spatial frames are more redundant than perimeter frames. Furthermore spatial configuration results in higher over



**Figure 6.** Performance of perimeter and spatial frames.



**Figure 7.** Structural weight for the analyzed frames.

strength compared to the perimeter framing configuration. The AISC/ASCE gives a stiffer design solution compared to EC8 which is due to the influence of high  $\Omega$  as given by the code. This high  $\Omega$  affects the size of the column and consequently the weight of columns. It can be concluded as a whole that the design of moment resisting frames with AISC/ASCE appeared to be more consistent although it resulted in a heavy structural solution compared to the counterpart EC8. The present work is restricted to study the effect of over strength and the behaviour factors only on a limited case. Nevertheless the effect of over strength factor and compatible behaviour factor with the damageability limits need to be focused in the future studies. Such studies are intended to be based on dynamic nonlinear time history analysis.

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