DESIGN RULES FOR STEEL PORTAL FRAMES ACCORDING TO EC3 AND AISC 360 PROVISIONS

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ABSTRACT

Nowadays, it frequently happens that steel constructors design and erect civil and industrial constructions all over the world, because of the complex historical and economic dynamics of the global market: as a consequence, engineers should be familiar with the use of widely accepted steel design codes. In spite of this remark, a direct comparison between the design rules adopted by European (EU) and American (US) design provisions on typical framed configurations of interest for routine design should result interesting. This issue is the focus of the present paper: both the EU and US codes are shortly discussed with reference to the methods proposed for design analysis. Furthermore, a numerical parametric study on portal frames is presented by varying the geometry, load conditions and degree of continuity of beam-to-column joints. Research outcomes single out main differences between the methods proposed within each code and among the codes themselves.

SOMMARIO
REGOLE PROGETTUALI PER IL DIMENSIONAMENTO DI CAPANNONI IN ACCIAIO IN ACCORDO A EC3 E AISC 360

Le dinamiche storico-economiche che si sono ormai instaurate portano sempre più frequentemente i costruttori a realizzare opere civili ed industriali in acciaio in tutto il mondo; risulta dunque ricorrentemente richiesto agli ingegneri l'utilizzo delle diverse normative vigenti in ambito internazionale. Alla luce di questa considerazione viene nel seguito proposto un confronto diretto tra le regole di progettazione adottate in ambito europeo ed americano per strutture intelaiate in acciaio. In dettaglio, sono brevemente presentate entrambe le normative con particolare attenzione ai metodi di analisi proposti, sintetizzando poi i risultati relativi ad uno studio parametrico riguardante capannoni industriali, differenti per geometria, condizione di carico e grado di continuità dei giunti trave-colonna. Le differenze sostanziali sia tra i metodi di ogni normativa sia tra le normative stesse permettono di valutare criticamente la loro validità ed attendibilità.

KEYWORDS | PAROLE CHIAVE
Eurocode 3, AISC 360, second-order effects, elastic analysis, elasto-plastic analysis, semi-rigid connections, imperfections.
Eurocodice 3, AISC 360, effetti del secondo ordine, analisi elastica, analisi elasto-plastica, giunti semi-rigidi, imperfezioni.

One of the main results of the increasing globalization is that nowadays design, fabrication and erection of steel structures can take place at different locations, potentially separated by several thousand kilometers. Owners consequently might require the use of widely accepted steel design codes and designers should therefore be familiar with specifications that may be substantially different from one other. Two of the most widely used steel design standards are for civil and industrial buildings located in Europe (EU) and United States (US). The US provisions have been developed by the American Institute of Steel Construction (AISC) and the main reference design code is the ANSI/AISC-360 "*Specification for Structural Steel Buildings*" [1], herein referred to as simply as AISC. It deals with steel buildings in accordance with *Load and Resistance Factor Design (LRFD)* format but admits also the *Allowable Strength Design (ASD)* approach, which is outside the scope of this paper. In Europe, reference has to be made to the *EN 1993-Eurocode 3 "Design of Steel Structures"* (identified as *EC3*), that has been developed by the European Committee for Standardization. It allows exclusively for designing in accordance with the limit state design philosophy, corresponding hence to the AISC-LRFD approach. Furthermore, it should be noted that AISC provisions are complemented by a very exhaustive commentary [2], whereas EC3 consists of seven parts, each of them focused on particular structure typologies such as buildings, bridges, towers and silos. The main references for the design of conventional civil and industrial steel buildings are parts 1-1 [3], 1-5 [4] and 1-8 [5].

As expected, the requirements provided in both EC3 and AISC Codes differ significantly in terms of load combination rules, design approaches for the structural analysis, equations for the member/joint verification checks and safety factors accounting for material and verification approach uncertainties. Furthermore, it should be pointed out that each of these codes admits alternative design paths, differing in terms of the degree of refinement of the structural analysis and of rules regarding all the safety checks of components. As a final result, non-negligible differences in performance of design structures are expected, and this should be of great interest for designers, practitioners and constructors.

In a previous paper [6], a general comparison between the EU and US provisions was proposed stressing out similarities and differences related to cross-sections classifications, methods of analysis and limit state verifications checks. More recently, a numerical study was proposed [7] discussing research outcomes related to the design of unbraces multi-story frames. Now, attention is focused on the portal frame typology and the present paper reports on the design results associated with the alternatives admitted by codes, which have been referred to 480 cases of interest for routine design. In particular, the results of elastic and elastic-plastic analyses are compared to each other, by considering also the geometric non-linearities. Research outcomes allow for a direct appraisal of the differences in terms of load-carrying capacity, or equivalently of safety index of the frames.

THE METHODS OF ANALYSIS ACCORDING TO EU PROVISIONS

EC3-1-1 specifies, in its part 1-1 [3], the following methods for performing structural analysis, already identified for the sake of simplicity [6,7], as:

- EC3-DAM: Direct Analysis Method;
- EC3-RAM: Rigorous Analysis Method;
- EC3-FOM: First-Order analysis Method;
- EC3-GEM: GEneral Method.

Key features of these methods are summarized in table 1 and more details related to the code requirements can be found in the codes themselves and in ref. [8].

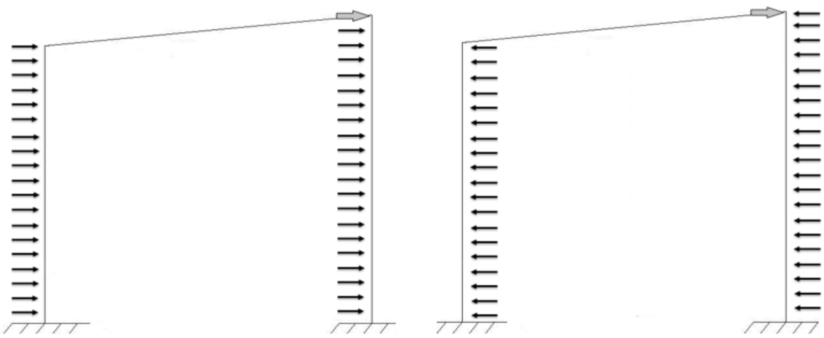
EC3-DAM approach

The Direct Analysis Method (DAM) takes into account all initial imperfections: out-of-straightness (bow) imperfections of members and lack-of-verticality (sway) imperfections of the whole structural system. Second-order effects have to be considered into analysis and only resistance verifications are required to designers.

Table 1. Comparison between European approaches.

Feature	Approach			
	EC3-DAM	EC3-RAM	EC3-FOM	EC3-GEM
Lack-of-verticality imperfections	Yes (direct modelling or notional loads)			
Out-of-straightness imperfections	Yes (direct modelling or notional loads)	No		
Member stability checks	No	Yes		No
Overall stability	No			Yes
Buckling length	Not required	System length (K=1)	Effective length ($K=K(N_{cr})$)	Effective length ($K=K(N_{cr})$)

As Eurocode 3 makes no prescription about the versus of application of the local imperfections, which are generally simulated via notional loads, many combinations are possible, and designer should find the worst from all the possible cases. In this paper, owing to the fact that only portal frames are considered, two possible combinations have been analyzed, which are presented in figure 1: the EC3-DAM(+) with local imperfections, increasing the effects of global imperfections and the EC3-DAM(-) with local imperfections opposite to the global imperfections.



1. Local and global imperfections used in the EC3-DAM approach.

EC3-RAM approach

The Rigorous Analysis Method (RAM) requires a second-order analysis considering only the lack-of-verticality imperfections. Member stability has to be checked and the code declares that the structure shall be considered as a no-sway frame, assuming buckling lengths equal to system (geometrical) lengths. EC3 formulas for stability verifications of beam-columns already account for geometric and material non-linearity on members by means of suitable buckling curves, via the reduction factors χ and χ_{LT}, which are later discussed in section 5.1.

EC3-FOM approach

The First-Order analysis Method (FOM) neglects imperfections and requires a first-order analysis. As to the member stability checks, the code declares that the effective buckling lengths has to be evaluated on the basis of global buckling mode of the frame, considered as a sway frame. EC3 does not provide any further detail regarding the more convenient and reliable approach to define the member buckling length (L_{eff}) or, equivalently, the effective length factor K. From the practical point of view, designers, in general, evaluate the critical load (N_{cr}^{i}) for the i^{th} member of frames via the equation:

$$N_{cr}^{i} = \alpha_{cr} \cdot N_{Ed}^{i} \tag{1}$$

where α_{cr} is the buckling overall frame multiplier obtained via a finite element buckling analysis and N_{Ed}^{i} is the design axial load acting on the considered member. Alternatively, reference should be made to suitable alignment-charts or to the well-established equations presented in literature. As an example, reference can be made to the previous edition of the EC3, that is the UNI ENV 1993-1-1 [9], which allows for a direct estimation of the K factor via well-established theoretical formulations.

EC3-GEM approach

Eurocode 3 in its part 1-1 proposes the General Method, that is quite innovative design approach, for the stability checks of structural components having geometrical, loading and/or supporting irregularities. Despite the fact that this method seems very promising, steel designers rarely use it. Overall buckling resistance is verified when:

$$\frac{\chi_{op}\alpha_{ult,k}}{\gamma_M} \leq 1 \tag{2}$$

where $\alpha_{ult,k}$ is the minimum load multiplier evaluated with reference to the component (member and joint) resistance, χ_{op} is the buckling reduction factor referring to the overall structural system and γ_M is the material safety factor.

The ultimate load multiplier for member resistance $\alpha_{ult,k,m}$ is determined, on the basis of the design axial load and bending moment, N_{Ed} and $M_{y,Ed}$, respectively, accounting for frame imperfections and determined via a second-order analysis, as:

$$\alpha_{ult,k,m} = \left(\frac{N_{Ed}}{N_{Rk}} + \frac{M_{y,Ed}}{M_{y,Rk}} \right) \tag{3a}$$

where N_{Rk} and $M_{y,Rk}$ are the squash load and the first yielding moment, respectively, of the most highly stressed cross-section.

Furthermore, indicating with $M_{j,Ed}$ and $M_{j,Rd}$ the design bending moment acting on the most stressed joint and its flexural resistance, respectively, the associated ultimate load multiplier $\alpha_{ult,k,j}$ is expressed as:

$$\frac{1}{\alpha_{ult,k,j}} = \frac{M_{j,Ed}}{M_{j,Rd}/\gamma_M} \qquad (3b)$$

Term $\alpha_{ult,k}$ in eq. 2) has consequently to be assumed as the minimum value between $\alpha_{ult,k,m}$ and $\alpha_{ult,k,j}$.

The buckling reduction factor χ_{op} depends on the relative slenderness λ_{op} of the whole structure defined as:

$$\lambda_{op} = \sqrt{\frac{\alpha_{ult,k}}{\alpha_{cr,op}}} \qquad (4)$$

where $\alpha_{cr,op}$ is the critical minimum elastic multiplier referred to sway mode.

THE METHODS OF ANALYSIS ACCORDING TO US PROVISIONS

AISC 360-10 allows the use of *any rational method of design for stability* and it suggests three methods for the design analysis, which are:

- AISC – DAM: *Direct Analysis Method,* defined in AISC sub-chapter C1.1;
- AISC – ELM: *Effective Length Method,* defined in AISC sub-chapter C1.2 and discussed in its Appendix 7.2;
- AISC – FOM: *First-Order Analysis Method,* defined in AISC sub-chapter C1.2 and discussed in its Appendix 7.3.

Key features of these methods are summarized in table 2 and are herein shortly discussed.

Table 2. Comparison between United States approaches.

Feature	Approach		
	AISC-DAM	AISC-ELM	AISC-FOM
Lack-of-verticality imperfections	Yes (direct modelling or notional loads)		
Out-of-straightness imperfections	No (already considered by formulas for member stability verification)		
Adjustments to stiffness	Yes (20% reduction)	No	
Member stability checks	Yes		
Buckling length	System length (K=1)	Effective length (K=K(N_{cr}))	System length (K=1)

AISC-DAM approach

The Direct Analysis Method (DAM) is the main suggested method that can be applied in every design case, without any kind of limitations. The AISC-DAM approach requires a second-order analysis, considering second order effects, together with flexural, shear and axial member deformations. In addition to the modelling of initial imperfections, the AISC-DAM approach prescribes also that all the steel properties contributing to the elastic stiffness should be reduced by multiplying them by 0.8 with the exception of member flexural rigidities, which must be reduced to 0.8τ_b. In particular, τ_b is equal to unity when the design axial force (P_r) is less than or equal to 50% of the axial yielding strength of the member (P_y); otherwise, τ_b assumes the value:

$$\tau_b = 4 \cdot \left(\frac{P_r}{P_y}\right)\left[1 - \left(\frac{P_r}{P_y}\right)\right] \qquad (5)$$

Once the designer has computed the required strength of each member of the frame by means of the second-order analysis, the available member strengths have to be defined assuming an effective length factor K equal to unity, i.e. corresponding to the system length of the member.

AISC – ELM approach

The Effective Length Method (ELM) can be applied if the structure supports gravity loads through vertical elements. Furthermore, in all stories, it is required that the maximum story drift evaluated via a second-order analysis (Δ_{II}) is lower than 1.5 times that obtained from a first-order analysis (Δ_I). A second-order analysis, like in AISC-DAM, is required but without stiffness reduction and imperfections are taken into account by notional loads. Available member strengths are based on the effective length factor K for moment resisting (sway) frames. In particular, when lateral loads are sustained by the flexural resistance of beams and columns, it is required that, if $\frac{\Delta_{II}}{\Delta_I} \geq 1.1$, verification checks are based on the system length (i.e. K = 1).

AISC – FOM approach

The First-Order analysis Method (FOM) can be applied if, in addition to the assumptions required by the AISC-ELM approach, the axial force demand (P_r) of all members whose flexural stiffness contributes to the structural lateral stability is less than or equal to 50% of the axial yielding strength of the member (P_y). For computing the required strength, AISC-FOM, unlike AISC-DAM and AISC-ELM, requires a first-order analysis, without any stiffness reduction but with additional lateral loads, $N_{i,FOM}$, accounting for P-Δ effects, which are defined as:

$$N_{i,FOM} = 2.1 \cdot \left(\frac{\Delta_i}{L}\right)_{\max} \leq 0.0042 Y_i \qquad (6)$$

where Y_i is the gravity load applied at the level *i*, Δ_I is the first-order inter-story drift and *L* is the height of the story.

In order to account for *P-δ* effects, designers can assume an effective length factor of *K =1.0* and have to apply a suitable B_1 amplifier factor to the beam-column moments.

COMPARISON BETWEEN THE EC3 AND AISC APPROACHES

As a general remark, it has to be pointed out that the AISC code defines the limits of applicability of the proposed approaches while Eurocode proposes its design alternatives without any type of limitations. With the exception of EC3-GEM, as shown in table 3, it is possible to note some similarities and differences between the EU and US methods that can be summarized as follows:

- EC3-DAM and AISC-DAM disagree for what concerns the stiffness reduction, imposed only by AISC and the imperfection requirements. Furthermore, EC3 requires only member resistance checks while, AISC-DAM recommends stability checks by assuming the system length (*K =1*);

- the same approach to evaluate the effective length as the system length is common to the EC3-RAM and the AISC-DAM approaches but a remarkable difference regards the stiffness reduction imposed only by the US Code;
- the EC3-RAM and AISC-ELM approaches can be directly compared when a second-order analysis is carried out. The main difference is related to the evaluation of the effective length for the stability checks: EC3 is based on the assumption of $K=1.0$ (i.e. the system length), while AISC requires a more accurate evaluation based on a buckling finite element analysis or, alternatively, on the use of suitable alignment charts;
- if a first-order analysis is carried out, the EC3-FOM and AISC-FOM approaches, differ mainly for the definition of the effective length factor K: more accurate according to EC3, depending on the overall frame stability, than to AISC equal to the system length.

Table 3. Comparison between the analysis methods admitted by the codes.

Feature	EC3-DAM	AISC-DAM	EC3-RAM	AISC-ELM	EC3-FOM	AISC-FOM
Analysis	Second-order		Second-order		First-order	
Lack-of-verticality imperfections	Yes		Yes		No	Yes
Out-of-straightness imperfections	Yes	No	No		No	Yes
Adjustments to stiffness	No	Yes	No		No	
Member stability checks	No	Yes (K=1)	Yes (K=1)	Yes ($K=K(N_{cr})$)	Yes ($K=K(N_{cr})$)	Yes (K=1)

SAFETY INDEX (SI): THE BEAM-COLUMN VERIFICATION CHECKS

Attention is herein focused only on members able to activate a plastic hinge, which correspond to the European class 1 profiles or to compact profiles according to the AISC classification criteria. The case of interest regards planar portal frames: therefore, bending moments on members act about the strong axis (identified as y- and x-axis according to the EU and US codes, respectively), while flexural buckling about the weak axis and flexural torsional buckling can be neglected into analyses being considered that are negligible for design purposes. The main symbols used for the verification checks by the EU and US codes differ to each other; therefore, reference can be made to the terminology specified in table 4.

Table 4. Terminology according to EU and US steel design codes.

EC3	Term	AISC
N_{Ed}	Axial force demand	P_r
N_{Rk}	Axial yielding member strength	P_y
$\chi\ N_{Rd}$	Design axial strength	P_n
$M_{y,Rk}$	Design flexural strength about strong axis	M_{cx}
δ	Displacement	Δ
N_{cr}	Elastic critical buckling load	P_{el}
W_{el}	Elastic section modulus about strong axis	S
F_i	Horizontal notational load	N_i
W_{pl}	Plastic section modulus about strong axis	Z
i_y	Radius of gyration about strong axis	r_x
f_y	Specified minimum yield stress strength	F_y
y-y	Strong axis	x-x

The European approach

Neglecting the resistance checks of beam-columns, already discussed in the introduction of the General Method, EC3 defines a quite complex criterion for member stability verification. It can be applied only to the case of uniform members with double symmetric cross-sections not susceptible to distortional deformation, such as those considered in the parametric analysis. In particular, in the case of members subjected to combined bending moment about the y-y axis, $M_{y,Ed}$, and axial compression N_{Ed}, the set of design equations for verification checks is reduced and remarkably simplified, requiring the following condition to be satisfied:

$$SI = \frac{N_{Ed}}{\chi_y \frac{N_{Rk}}{\gamma_{M1}}} + k_{yy} \frac{M_{y,Ed}}{\chi_{LT} \frac{M_{y,Rk}}{\gamma_{M1}}} \tag{7}$$

where χ_y is the reduction factor due to flexural buckling, χ_{LT} is the reduction factor due to lateral buckling and k_{yy} is the bending interaction factor.

In the portal frames herein considered, as already mentioned, only the flexural buckling along the strong axis has been considered. Assuming that lateral stability is suitably restrained, it results χ_{LT} = 1. The interaction factor k_{yy} depends on the design approach, which has to be selected from two options (table 5): the alternative method 1 and the alternative method 2, described in the Annex A and Annex B of EN 1993-1-1, respectively. The method 2 has been used in the proposed applications, being in general less complex and direct.

Table 5. Coefficient k_{yy} for members not susceptible to torsional deformations.

	k_{yy}
Alternative Method 1 (Annex A)	$C_{my} \frac{\mu_y}{1 - \frac{N_{Ed}}{N_{cr,y}}}$; $\mu_y = \frac{1 - \frac{N_{Ed}}{N_{cr,y}}}{1 - \chi_y \frac{N_{Ed}}{N_{cr,y}}}$; $C_{my} = C_{my,0}$
Alternative Method 2 (Annex B)	$C_{my}[1 + (\bar{\lambda}_y - 0.2) \cdot \frac{N_{Ed}}{\chi_y \cdot N_{Rk} / \gamma_{M1}}] \leq C_{my}\left(1 + 0.8 \frac{N_{Ed}}{\chi_y \cdot N_{Rk} / \gamma_{M1}}\right)$

The AISC approach

According to AISC design approaches, no distinction is considered between strength and stability verifications, being proposed a unified approach. In particular, with reference to the rules for doubly-symmetric members subjected to the bending moment about the strong x-axis (M_{rx}) and to the axial force (P_r), it is required that:

- If $\frac{P_r}{P_c} < 0.2$, $SI = \frac{P_r}{2P_c} + \left(\frac{M_{rx}}{M_{cx}}\right) \leq 1$ (8a)
- If $\frac{P_r}{P_c} \geq 0.2$, $SI = \frac{P_r}{P_c} + \frac{8}{9}\left(\frac{M_{rx}}{M_{cx}}\right) \leq 1$ (8b)

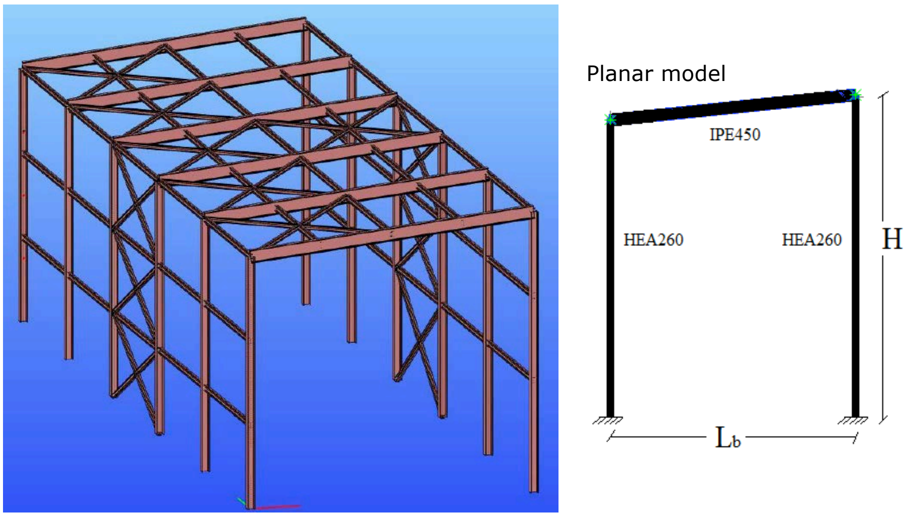
where $P_c = 0.9P_n$ is the resistance strength for compression, accounting for instability phenomena and $M_{cx} = 0.9M_{nx}$ is the resistance for bending.

It should be noted that for the cases of interest, M_{nx} coincides with the plastic moment capacity.

THE PARAMETRIC ANALYSIS

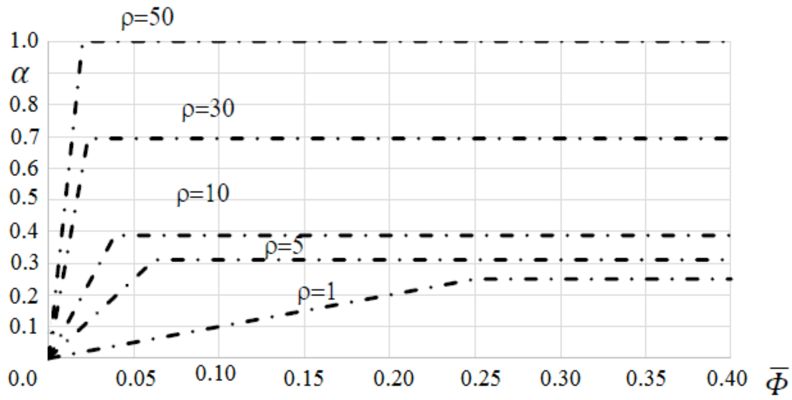
To allow for a direct appraisal of the differences associated with the aforementioned approaches according to the EU and US codes, a numerical study on cases of interest for practical design of steel structures has been developed on portal frames. In particular, the following parameters have been considered:

- *the frame configuration*. The reference spatial structure is composed by a sequence of portal frame (figure 2) with bracings in the plane of the roof and in the longitudinal direction. Owing to the need of reducing the number of variables affecting the research outcomes, a simplified planar model has been considered. To deeply investigate the influence of second-order effects, height (H) and bay length (L_b) have been varied by considering "FH10" (with L_b=10m and H=10m) and "FH13" (with L_b=13m and H=10m) cases, presenting both the same roof inclination that is equal to 1%. The columns are made by HEA260 profiles in S275 steel grade [10] while the beam is made by IPE450 in S355 steel grade;



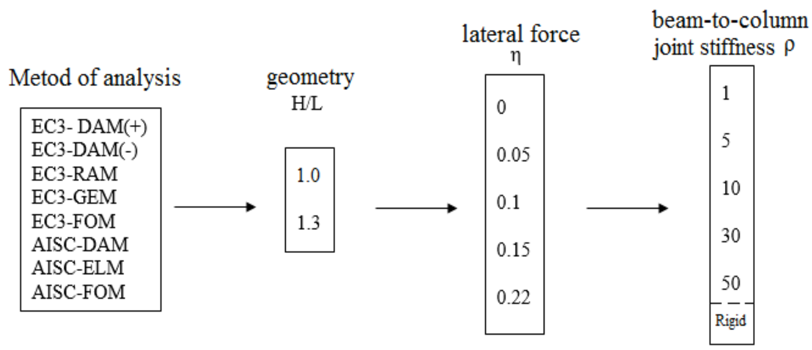
2. The case considered in the parametric study and the planar model.

- *the beam-to-column joint performance.* According to the criteria proposed in the part 1-8 of EC3 [5], both the cases of rigid and semi-rigid beam-to-column connections were analyzed. As to the latter, the selected values of the elastic rotational stiffness of beam-to-column joints $S_{j,btc}$ have been expressed as $\rho\left(\frac{EI_b}{L_b}\right)$ where I_b is the second moment of area of the beam and E is the Young's modulus of the steel. The ultimate strength of the joints, $M_{u,btc}$, is expressed as percentage of the plastic moment of the beam, $M_{pl,b}$, through the factor $\alpha \leq 1$, i.e. $M_{u,btc} = \alpha \cdot M_{pl,b}$. A linear relationship between ρ and α was supposed, imposing that for the lowest stiffness value ($\rho = 1$) α is assumed equal to 0.25 while for the highest ($\rho = 50$) α is equal to 1.0. In order to obtain results of interest for routine design because regarding the entire semi-rigid domain, the ρ values of 1, 5, 10, 30, and 50 have been assumed, and the associated non-dimensional joint relationships are presented in figure 3.



3. Semi-rigid beam-to-column joint cases in the $\alpha - \overline{\Phi}$ domain

• *the load combinations*. Both vertical and horizontal loads were considered acting on the portal frame, simulating the former the gravity loads and the latter the wind action or the effects of earthquakes via the well-established lateral force method of analysis. To avoid the complexity to define the limit state combinations prescribed in the codes [11, 12], a unique load combination was considered for each case. In particular, the uniform distributed vertical load on the girder was considered equal to q=27 kN/m for each cases and the horizontal loads were simulated thought a concentrated horizontal load applied on top of the right column, equal to $F=\eta q L_b$. The multiplier η assumes the values of 0.0 (gravity loads only), 0.05, 0.10, 0.15 and 0.22. The imperfections were applied on the frame following the prescriptions given by each considered design approach.
To reduce the number of variables affecting the research outcomes, columns have been always considered perfectly fixed to the foundation. A summary of the analyses performed in the study is sketched in figure 4, were all the key parameters are reported; in total 480 cases were analyzed.



4. The layout of the considered design cases.

Structural analyses have been carried out by using two different finite element software: SAP2000 [13] for elastic analyses and PEPmicro [14] for elasto-plastic analyses. It is worth noting that PEPmicro is a software developed by CTICM for linear and non-linear static analysis of planar steel structures. It takes into account for second-order effects and section plasticity, considering also bending moment-axial force interaction for semi-rigid member connections.

RESEARCH OUTCOMES

Research outcomes are summarized distinguishing the results related to rigid and semi-rigid joints in two separated sub-sections. Furthermore, to allow for a direct appraisal of the analysis results, reference can be made to the value of the safety index (SI), already defined in the previous section 5.

Frames with rigid joints

As to portal frames with rigid connections, the results associated with the design analyses for both FH10 and FH13 frames are reported in tables 6 and 7, related to the EC3 and AISC standards, respectively. In addition to the SI value associated with the most highly stressed column, the minimum value of which is highlighted, the maximum (Max) to minimum (min) ratio between the considered design alternatives is reported, too.

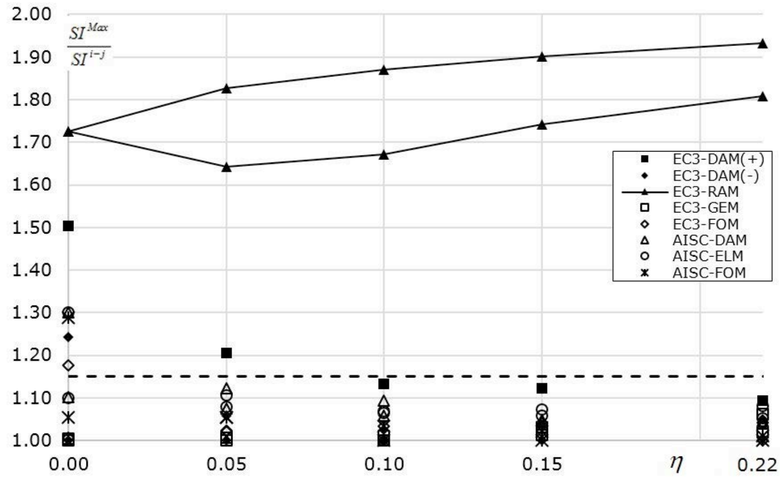
Table 6. Safety Index in according to EC3 standard.

	η	**EC3-DAM(+)**	**EC3-DAM(-)**	**EC3-RAM**	**EC3-GEM**	**EC3-FOM**	**Max/min**
FH10 frame	**0.0**	0.39	0.45	0.26	0.45	0.45	1.725
	0.05	0.51	0.57	0.31	0.57	0.56	1.827
	0.10	0.63	0.69	0.37	0.69	0.67	1.870
	0.15	0.75	0.81	0.43	0.80	0.78	1.902
	0.22	0.92	0.98	0.51	0.97	0.99	1.933
FH13 frame	**0.0**	0.33	0.45	0.29	0.49	0.46	1.726
	0.05	0.50	0.57	0.36	0.60	0.59	1.643
	0.10	0.66	0.73	0.45	0.75	0.75	1.671
	0.15	0.83	0.90	0.54	0.91	0.92	1.708
	0.22	1.10	1.14	0.67	1.13	1.15	1.735

Table 7. Safety Index in according to AISC standard.

	η	**AISC-DAM**	**AISC-ELM**	**AISC-FOM**	**Max/min**
FH10 frame	**0**	0.41	0.41	0.43	1.046
	0.05	0.53	0.53	0.54	1.025
	0.10	0.65	0.65	0.67	1.032
	0.15	0.78	0.77	0.80	1.037
	0.22	0.95	0.94	0.98	1.043
FH13 frame	**0**	0.38	0.38	0.38	1.011
	0.05	0.53	0.54	0.57	1.064
	0.10	0.69	0.71	0.75	1.093
	0.15	0.90	0.87	0.94	1.072
	0.22	1.15	1.11	1.20	1.081

The minimum EU value of the safety index is always associated with the EC3-RAM approach with differences from the other results comprised between 71% and 93%, approximately. The results given by EC3-DAM(+) approach are slightly "generous" with a SI greater than the EC3-RAM one but lower than the ones associated with the other approaches. As to the US results, AISC-FOM approach gives always the highest safety index values and the differences between the AISC-DAM and AISC-ELM approaches are very limited, never greater than 4%. Furthermore, it can be noted that the safety index values of the EC3-RAM approach are always significantly lower than the ones associated also with AISC approaches. Moreover, AISC results show a moderate variability of the values, as confirmed by the mean value of the Max/min ratio that is equal to 1.050 with a standard deviation of 0.026 (table 7) while for the EC3 is up to 1.774, with a standard deviation of 0.101 (table 6). To allow for a better appraisal of the contents of the tables, figure 5 can be considered, where the maximum SI over the SI obtained by each approach, i.e. the $\frac{SI^{Max}}{SI^{i-j}}$ ratio is sketched (superscript *i* indicates the standards and superscript *j* the approaches).



5. $\frac{SI^{Max}}{SI^{i-j}} - \eta$ relationship for all the considered cases.

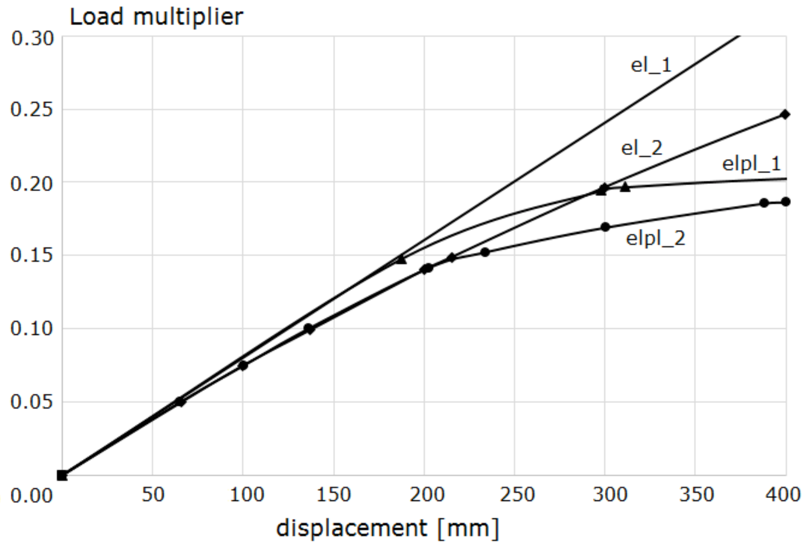
The maximum acceptable difference between methods, from a design point of view, should be, in the Authors' experience, reasonably fixed in 1.15, depicted as the dashed horizontal line in figure 5. It can be noted that the EC3-RAM approach is always significantly (and dangerously) far from this limit. The safety index ratios related to the gravity load case (η=0) are higher than this limit, while for high value of the horizontal forces, results are more homogeneous (differences lower 15%, excluding EC3-RAM results).

For each of the *j*-considered approach, the load carrying capacity of the frames can be assessed, starting from the value of the SI^j, as:

$$\tilde{q}_j = \frac{q}{SI^j} \tag{9}$$

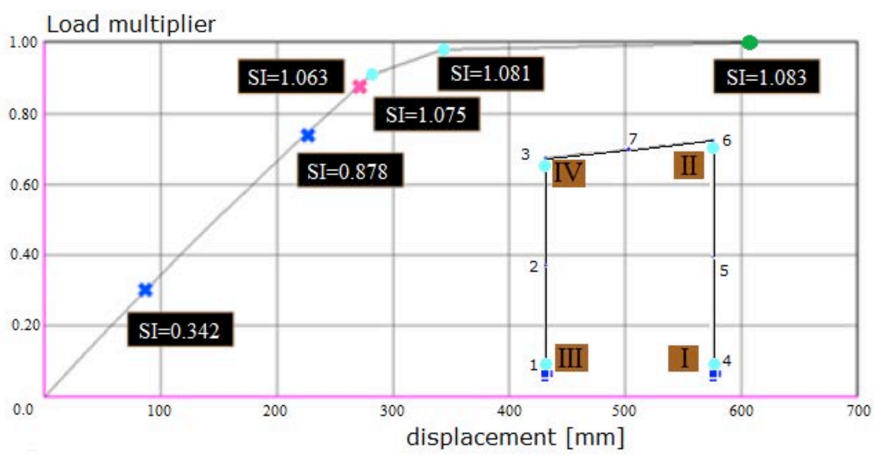
where q is the is the reference load applied to the portal frames, kept constant in all the considered cases (q=27kN/m).

Owing to the differences in the SI values, also differences between the associated $\tilde{q}_j$ value are expected both by using the approximation provided by eq. 9) or via a more accurate iterative procedure owing to the presence of second-order effects. A great interest for practitioners and engineers could be found in design procedures based on elastic-plastic analyses, which are nowadays commonly offered by several commercial finite element analysis packages. As an example, the typical incremental load multiplier-top displacement curves obtained from PEPmicro, are reported in figure 6 according to the EC3-DAM(+) load condition, considering both elastic (el) and elastic-plastic (elpl) analyses of the first (_1) and second (_2) order. The trends showed in figure 6 for the FH13 rigid portal frame, with $\eta=0$ are typical of all the considered cases. Initially, the curves are coincident, owing to the limited influence of second-order effects. Increasing the load multiplier, the differences between top displacement values increase. Furthermore, a quite wider extension of the plastic branch in the case of first-order analysis can be appraised, which has a collapse load higher than the one associated with second-order analysis, due to the interaction between instability and plasticity.



6. Load multiplier-top displacement curves for different types of analysis for the FH13 frame, $\eta=0$.

As required by the EU code, elasto-plastic analysis does not consider the second-order effects only for the EC3-FOM approach, while in the other approaches they have suitably to be taken into account. For all the considered cases, the value of safety index associated with the resistance has been evaluated in different points of the elastic-plastic curves. As an example, figure 7 can be considered, which is related to the EC3-DAM(+) approach applied to FH13 frame with $\eta=0.22$. In particular, the values of the resistance SI are proposed, as an example, for two load steps of the initial elastic branch (blue cross X) and for the step corresponding to a distributed vertical load equal to 27kN/m (purple cross X). Furthermore, in the same curve, SI values are specified also when plastic hinges form in members, allowing for following its sequence (brown box, light blue circle) up to collapse (green circle).



7. Load multiplier-displacement curve for FH13 frame with h=0.22, in according to the EC3-DAM(+) approach

The sequence of the plastic events has been monitored to have information about the collapse mechanism. The value of the SI proposed in the initial linear branch of the elastic-plastic curve never increases linearly, and this is due to the second-order effects and to the terms in the verification formula. By focusing on cases with η = 0, it can be noted that the first plastic hinge always originates in the middle of the beam; on the contrary, when η = 0.22 is considered, no plastic hinges are observed on the beam.
Independently of the collapse mechanism, it appears that the spread of plasticity in the frame with the consequences of a non-negligible bending moment redistributions leads to relevant benefits in terms of load-carrying capacity.
As expected, in the initial branch of the curves, the SI values are practically equal if evaluated with internal forces given by SAP2000 or PEPmicro, owing to the limited influence associated with the different finite element beam formulations and with the approaches to account for the second-order effects.
It is worth noting that, through elastic-plastic analyses it is possible approximate more accurately the frame performance that is to estimate the 'true' load carrying capacity q^u. The $\frac{q^u}{\tilde{q}_j}$ ratio is proposed in table 8 for all the European approaches, where the values lower than one are highlighted. It can be remarked that the value q^u just depends mainly on the applied forces and on the type of analysis.

Table 8. $\frac{q^u}{\tilde{q}_j}$ ratio for all the European approaches.

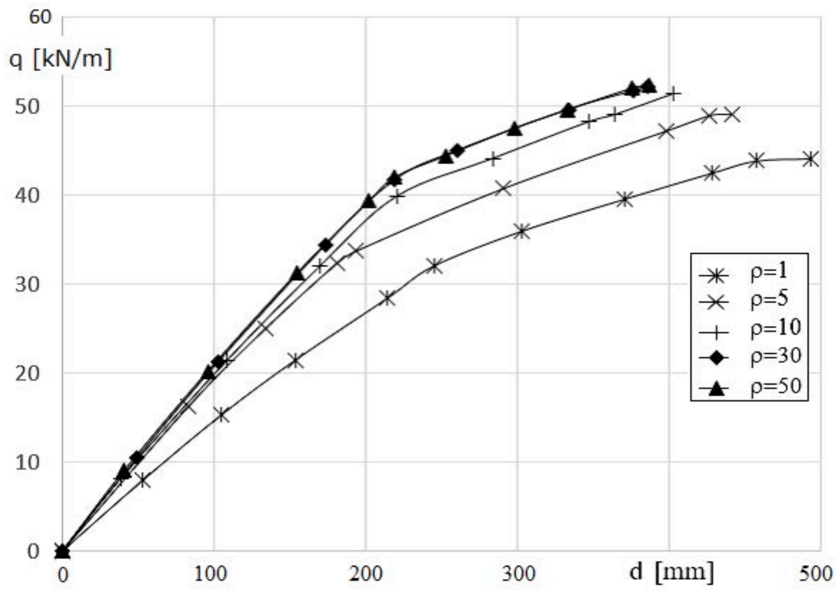
	η	EC3-DAM(+)	EC3-DAM(-)	EC3-RAM	EC3-GEM	EC3-FOM
FH10 frame	**0**	1.026	1.089	0.845	1.084	1.184
	0.05	1.089	1.088	0.825	1.086	1.210
	0.10	1.088	1.084	0.811	1.151	1.124
	0.15	1.075	1.074	0.745	1.144	1.214
	0.22	1.045	1.044	0.677	1.100	1.085
FH13 frame	**0**	1.046	1.121	0.850	1.144	1.189
	0.05	1.124	1.125	0.838	1.170	1.216
	0.10	1.126	1.126	0.821	1.230	1.251
	0.15	1.100	1.108	0.746	1.182	1.253
	0.22	1.083	1.087	0.687	1.132	1.198
	Mean	1.080	1.095	0.785	1.142	1.192
	dev.st	*0.033*	*0.026*	*0.065*	*0.045*	*0.052*

In all the considered cases, the EC3-RAM approach is always associated with values lower than unity: the EC3-RAM $\tilde{q}$ value is always significantly greater than the collapse load obtained by PEPmicro, confirming once again that this approach leads to a remarkable overestimation of the ultimate load carrying capacity also when assessed via elastic-plastic approaches. Excluding EC3-RAM, it appears that, in general, quite limited differences can be noted between 'approximate' and 'true' collapse load, up to 25%, confirming the benefits in design associated with the use of more refined type of analyses.

Frames with semi-rigid joints

In the following only elasto-plastic analyses are discussed. It can be noted that AISC-FOM approach has been neglected owing to the difficulty of defining $N_{i,FOM}$ for incremental analyses.

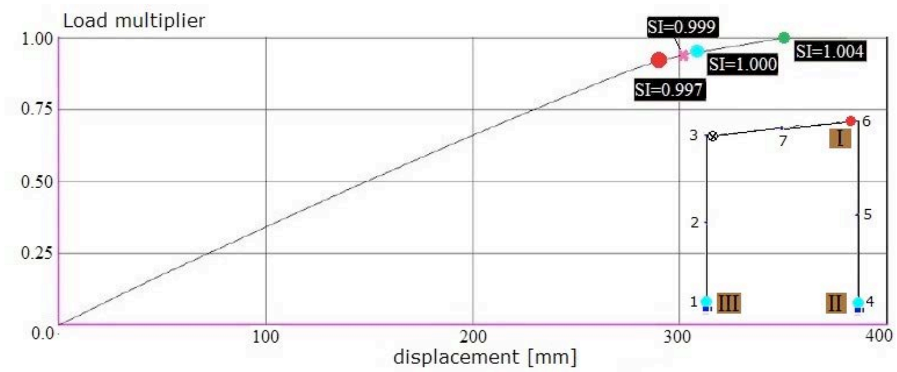
For semi-rigid joint frame, the plastic hinges are located at both column bases and beam-to-column joints. Therefore, also the value of flexural stiffness (ρ) and the associated yielding moment (α) greatly influence the structural response, as shown as example in figure 8, which is related to the FH13 portal frame with η=0.10 case.



8. Influence of degree of flexural continuity in the multiplier-displacement curve associated with the EC3-RAM approach for FH13 frame with η=0.10.

The results associated with the more rigid joints (ρ=30 and ρ = 50) are practically equal to each other for all the considered cases. From a practical point of view this means that the continuity of flexural strength and stiffness between joint and beam has already been achieved at ρ=30.

As in case of rigid frames, the SI values have been evaluated in different points of the elasto-plastic curves, as reported in figure 9, which is related to EC3-DAM(+) approach applied to FH13 frame with η=0.22 and ρ=10. Also in these cases, the sequence of the plastic events has been monitored in order to have information about the collapse mechanism and it can be noted that plastic events occur first in one beam-to-column connection and then at the column bases.



9. Load multiplier-displacement curve associated with FH13 frame having η=0.22 and ρ=10, in according to the EC3-DAM(+)approach.

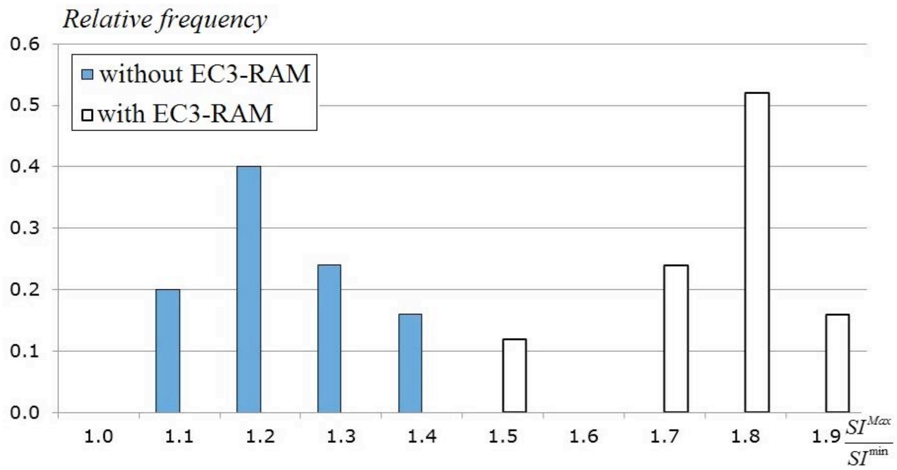
For the cases with the sole gravity load applied, it can be noted that the first plastic hinge appears always in the center of the beam. For the cases with ρ=30 and ρ=50 the same behavior is confirmed (that responses are similar) and the collapse occurs without the presence of plastic hinges in the beam-to-column joints.

Table 9 shows the values of the safety indexes evaluated at a load level equal to q = 27 kN/m, for all the considered cases of the FH13 frame. The lower safety index values are highlighted in each case. The remarks already proposed for rigid portal frames are totally confirmed and also for semi-continuous frames, the EU-RAM approach appears clearly inadequate for practical design purposed. Furthermore, it can be noted that the results associated with ρ=30 and ρ=50 are practically coincident, confirming the achievement of the complete joint flexural continuity.

Table 9. SI values evaluated via elastic-plastic analysis for the FH13 frame.

η	ρ	EC3					AISC		Max/min
		DAM(+)	**DAM(-)**	**RAM**	**GEM**	**FOM**	**DAM**	**ELM**	
0	**1**	0.26	0.28	0.21	0.34	0.26	0.27	0.36	1.720
	5	0.29	0.37	0.27	0.39	0.30	0.35	0.35	1.483
	10	0.31	0.39	0.27	0.41	0.31	0.36	0.36	1.493
	30	0.32	0.40	0.28	0.42	0.32	0.37	0.37	1.496
	50	0.32	0.40	0.28	0.42	0.32	0.37	0.37	1.495
0.05	**1**	0.41	0.42	0.30	0.47	0.54	0.46	0.57	1.900
	5	0.45	0.53	0.35	0.55	0.60	0.55	0.54	1.737
	10	0.47	0.55	0.36	0.57	0.62	0.56	0.56	1.732
	30	0.48	0.57	0.36	0.58	0.63	0.57	0.52	1.734
	50	0.48	0.57	0.36	0.58	0.63	0.57	0.57	1.735
0.10	**1**	0.66	0.57	0.37	0.61	0.64	0.57	0.69	1.855
	5	0.61	0.70	0.43	0.71	0.72	0.68	0.67	1.676
	10	0.63	0.72	0.44	0.73	0.74	0.70	0.69	1.677
	30	0.65	0.73	0.45	0.74	0.75	0.71	0.70	1.681
	50	0.65	0.74	0.45	0.74	0.72	0.71	0.71	1.668
0.15	**1**	0.68	0.67	0.44	0.74	0.65	0.68	0.83	1.873
	5	0.77	0.81	0.51	0.87	0.87	0.83	0.78	1.703
	10	0.80	0.88	0.52	0.89	0.89	0.87	0.86	1.704
	30	0.81	0.90	0.53	0.90	0.90	0.88	0.87	1.705
	50	0.82	0.90	0.53	0.91	0.90	0.88	0.87	1.705
0.22	**1**	0.79	0.79	0.48	0.80	0.73	0.80	0.81	1.695
	5	0.81	0.82	0.48	0.87	0.73	0.84	0.84	1.834
	10	1.00	1.00	0.61	1.05	0.90	1.00	1.02	1.717
	30	1.05	1.08	0.65	1.12	0.93	1.09	1.09	1.730
	50	1.06	1.08	0.65	1.12	0.93	1.09	1.09	1.728

Owing to the presence of the EU-RAM, it can be noted, once again, that the results, in terms of ratio between the maximum (Max) and the minimum, are very disperse, as shown by the Max/min ratio ranging from 1.483 to 1.900, with a mean value of 1.699 and 0.111 of the standard deviation. The great values are due to EC3-RAM allow to appraise for a frame performance significantly higher than the one associated with the other European results, such that the biggest value of the ratio is 1.834 for EC3 while it is 1.235 for AISC. These remarks are confirmed by figure 10, where the distribution of the $\frac{SI^{Max}}{SI^{\min}}$ ratio evaluated by considering (white bars) or neglecting (blue bars) the EC3-RAM approach.



10. Distribution of the $\frac{SI^{Max}}{SI^{\min}}$ ratio, with or without the EC3-RAM approach.

The ranges of distribution a very different: with reference to the EC3-RAM approach the values of the $\frac{SI^{Max}}{SI^{\min}}$ ratio range from 1.50 up to 1.90 with a great concentration of data around 1.70-1.80. Excluding this approach, the values of $\frac{SI^{Max}}{SI^{\min}}$ ratio decrease and data are located between 1.10 and 1.40, with a great amount of occurrences around 1.20-1.30. Once again, it is highlighted how excluding from the analysis the EC3-RAM methods the results associated with the remaining methods seems more reliable, accurate and comparable to each another.

CONCLUDING REMARKS

The methods of analysis admitted by the European and United States provisions for the design of steel framed buildings have been discussed and compared in this paper by considering also the different approaches admitted within each code. In total, 7 design paths have been identified allowing for a direct appraisal of their influence of the load-carrying capacity on conventional portal frames. Research outcomes are based on a parametric study comprising of 480 design cases of frames differing for geometry, load condition and degree of continuity of beam-to-column joints. Both elastic and elastic-plastic analyses have been executed by considering also second-order effects (when required). From the re-analysis of the numerical results, it can be concluded that:

- with the exception of the EC3-RAM, all the considered approaches lead to quite similar results, especially when high values of the lateral forces are applied;
- for what concerns the EC3-DAM approach, where local imperfections must be directly modeled into analysis, it can be noted that the direction of the equivalent imperfections has a great influence on the analysis results;
- the EC3-RAM approach often leads to a relevant (and dangerous) overestimation of the values of the load carrying capacity (figure 10).

Furthermore, it is worth noting that the critical aspects of the EC3-RAM approach are associated with the use of the system length instead of the effective length. In particular, this choice has great influence in:

i) the axial load capacity, that is generally overestimated;

ii) the definition of the bending interaction coefficient k_{yy}, which has double and not simple definition (Annex A and B of [3]), leading to two different and inaccurate results.

It is hence expected that, the EC3-RAM approach should be removed from the Eurocode as soon as possible, or its limits of applicability should be accurately re-defined. Otherwise, designers should significantly over-estimate the load carrying capacity of sway-frames, with very limited responsibility, having followed all the phases of the procedure recommended by the EU steel provisions.

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