European and United States approaches for steel storage pallet rack design Part 1: Discussions and general comparisons

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ABSTRACT

A very common application of cold-formed thin-walled steel members regards the industrial storage systems for goods and products, such as pallet racks, which are the core of the present paper. Owing to the increasing needs associated with globalization as well as the importance of the logistics, nowadays, rack manufactures must be highly competitive in the most industrialized countries: as a consequence, it is frequently required that engineers design in accordance with standards of the country where the storage rack will be in service. Due to peculiarities associated with the requirements of each code, different values of the rack load carrying capacity are expected and their quantification should be of great interest for commercial reasons: this is an important open question for designers and manufactures that up to now is without any practical response.

A two-part paper has been written to summarize the main results of a comparative study on pallet rack design provisions currently adopted in Europe and the United States. *Part 1* describes the key features of both codes, discussing similarities and differences associated with the admitted design procedures with reference to the member verification procedures and to the alternative analysis approaches. Furthermore, practical comparisons related to isolated members under compression and bending are proposed to allow a direct appraisal of structural performance.

The companion paper (*part 2: Practical applications*) reports on an exhaustive analysis with regards to the contents herein introduced and applied to the design of 216 medium-rise semi-continuous pallet racks that are unbraced in the longitudinal direction.

1. Introduction

Among the different industrial solutions to store goods and products, steel storage pallet racks represent one of the most commonly used solutions [1], which are often made by cold-formed thin-walled members. As shown in Fig. 1, they consist of a regular sequence of upright frames connected to each other by pairs of pallet beams carrying the stored units. From a structural point of view, racks are generally braced only in the transverse (cross-aisle) direction, owing to the impossibility to locate longitudinal (down-aisle) bracing systems without reducing the storage capabilities. Stability in the down-aisle direction is hence provided solely by the degree of flexural continuity associated with beam-to-column joints and base–plate connections, which have to be modeled as semi-rigid joints.

An increasing number of cases where design, fabrication and erection of rack structures are separated by large distances has been observed in recent years, as a result of rapid globalization and of the modest costs associated with the transportation of these very light-weight structures. Owners require the use of widely accepted steel design codes regardless of the location where the structure is going to be built; as a consequence, struc-tural engineers are now faced with the challenge of being com-petent with design specifications, which could present substantial differences between one another. Attention is herein focused only on the pallet rack design for static loads: in Europe (EU), the reference is the EN15512 specification, *"Steel static storage systems-Adjustable pallet racking systems-Principles for structural design"*[2]. This code, which is in the process of being updated in the next few years, is the evolution of the recommendations FEM 10.2.08 [3], published by the technical committee Working Group 2 of the "Federation Europeenne de la Manutention" (FEM). In the United States (US), the design of industrial steel storage racks is carried out according to the Rack Manufacturers Institute (RMI) specification "*Specification for the Design, Testing and Utilization of Industrial Steel Storage Racks*" [4], which is tied closely to the AISI specification [5] for the cold-formed steel design (*North American specification for the design of cold-formed steel structural member*). In the framework of a recent study on the analysis approaches for traditional steel frames constructed using hot-rolled members

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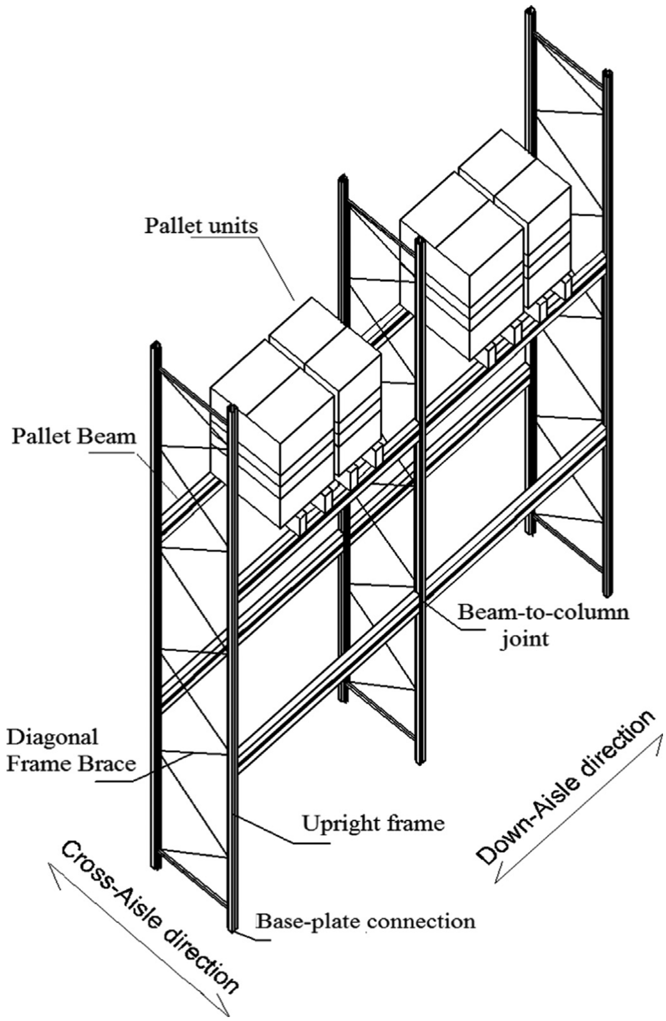

Fig. 1. Typical steel storage pallet rack.

[6], it has been demonstrated that non-negligible differences can be observed with reference to the frame performances predicted according to the EU [7] and the US [8] permissible design alternatives. As a consequence, owing to the extensive use of cold-formed members for logistic applications, the Authors decided to analyze also the design approaches for steel storage pallet racks, which differ from the more traditional steel frames due to the use of thin-walled components and for the significant influence of second-order effects.

Research results are summarized in a two-part paper, which has been focused on bi-symmetric cross-section uprights, being the important effects associated with non-coincidence between the centroid and the cross-section shear center already in-vestigated in previous research [9–11]. Despite the fact that usually upright cross-sections present one axis of symmetry (Fig. 2a), an increasing number of solutions recently proposed for the industrial storage market is characterized by the use of boxed closed cross-section (Fig. 2b), which present, in several cases, two axes of symmetry. In this *part 1*, key features of both the EU and the US design codes are introduced and discussed, focusing at-tention on the evaluation of the effective geometric properties and on the verification design procedures for columns and beam–col-umns. Furthermore, the permitted approaches for structural analysis are discussed highlighting the similarities and differences. Attention is mainly paid to the upright design for two reasons:

1. The difference due to the choice of the method of analysis according to both the considered codes reflects mainly on the design of these vertical elements. No alternatives are available for the design of the other key rack components (i.e., pallet beams, upright lacing and joints).
2. The importance of these vertical members with reference to the total weight of the industrial storage systems.

The companion paper (*part 2: Practical applications*) [12] reports the evaluation of the load carrying capacity according to the considered design alternatives, basing the proposed research outcomes of a set of 216 racks differing in configuration, geometry of the components and degree of rotational stiffness of beam-to-column joints and base–plate connections.

2. Effective cross-section properties

The theory of thin-walled cold-formed members was well-established several decades ago [13–15] and now steel specifications [5,16] propose very refined design approaches able to account for local, distortional and overall buckling phenomena as well as for their mutual interactions. Theoretical design procedures have been completely defined only for few types of cross-section, such as channel, angle and hat cross-sections, which are the most commonly used in framing, metal buildings and lightweight housing systems [17]. These cross-sections are often different from the ones typically used in the structural systems to store goods and products. In several cases, open mono-symmetric perforated cross-sections (Fig. 2a) are used but also hollow square/rectan-gular cross-sections are employed, the closure of which is some-times obtained by overlapping and clamping to each other the lateral edges of the strip coils (Fig.2b).

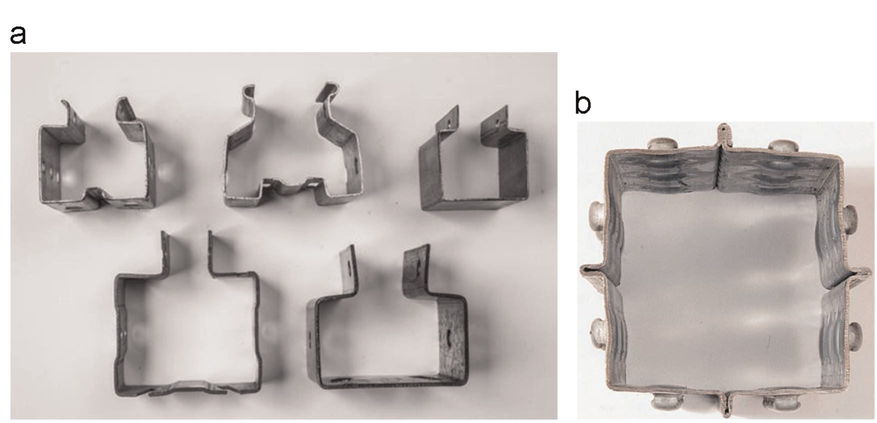

Fig. 2. Typical mono-symmetric (a) and bi-symmetric (b) cross-section uprights used in pallet racks system.

As clearly summarized by Baldassino and Zandonini [18], rack design is based on tests on the key structural components: the existing rules to design racks are therefore empirical and their validity is only in the range of the investigated parameters. It should be noted that several research projects are currently in progress on the development of innovative approaches for the design of such types of uprights, the most promising of which seems to be the US direct strength method [19–21], which probably, in the next few years will be included among the European design options [22]. However, focusing attention on the strategies already codified in steel rack provisions [2,4,23] and currently adopted by manufacturing engineers, a design assisted by a testing procedure is required to evaluate the effective cross-section geometric parameters. For the sake of simplicity, suitable Q factors are considered in routine design, that are expressed by a number, never greater than unity, accounting for the reduction of the considered geometric parameter (usually, area, second moments of area and section modulii) due to the use of thin-walled members.

In the following, reference is made to boxed cross-section but results maintain their validity also when one or no axis of symmetry are present on the cross-section uprights. The design approaches are, in fact, based on the use of equations accounting for the section behavior via suitable reduction factors and this allows them to cover the range of cases usually encountered in routine rack design independent of the cross-section form.

2.1. The EU approach

Effective cross-section properties of uprights to be used for European verification checks must be based on component tests, which are accurately described in the Appendix A of the EN 15512 standard [2]. Owing to the scope of the present paper, attention is focussed only on the essential tests, i.e. the ones necessary to design members under compression and/or bending. In particular, the well-known stub column test allows for the evaluations of the effective area accounting for perforations, cold manufacturing processes, connection points/zones and overlapping, local and distortional buckling phenomena. The typical specimen is composed of a stub upright, at each end of which a thick steel plate is welded (Fig. 3). The design failure load (R_d) is obtained from the statistical elaboration of the experimental results related to a set of tests of nominally equal specimens under axial load: the effective area, A_{eff}^{EU}, is however limited to be not greater than the gross area (A) and is evaluated as

$$A_{eff}^{EU} = \frac{R_d}{f_y} = Q_{EU}^{N} \cdot A \tag{1}$$

where f_y is the yielding strength of the base material before the cold working processes and Q_{EU}^{N} is the reduction factor accounting for buckling on stocky thin-walled members.

In a similar way, bending tests allow for the prediction of the upright flexural performance about major and minor axes of flexure (Fig. 4). In particular, the bending resistance of the uprights is derived from the values of the maximum load applied to the specimen during the test while the flexural stiffness is assessed via the initial branch of the load–displacement experimental response. When the flexural behavior about the axis of symmetry has to be investigated, a complete upright frame is usually tested instead of an isolated upright. The main result of these tests is the load–midspan displacement curve, from which the values of the effective second moments of area and of the elastic modulii can be

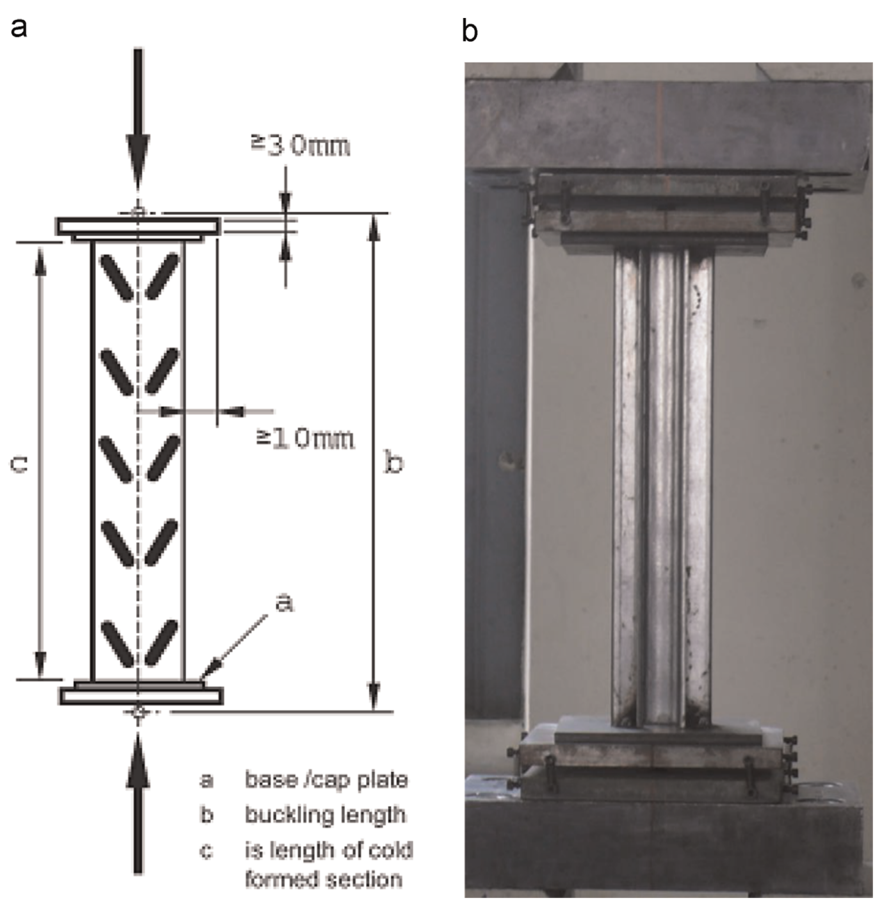

Fig. 3. Stub column test: (a) the specimen according to CEN, EN 15512 [2] and (b) the specimen during test.

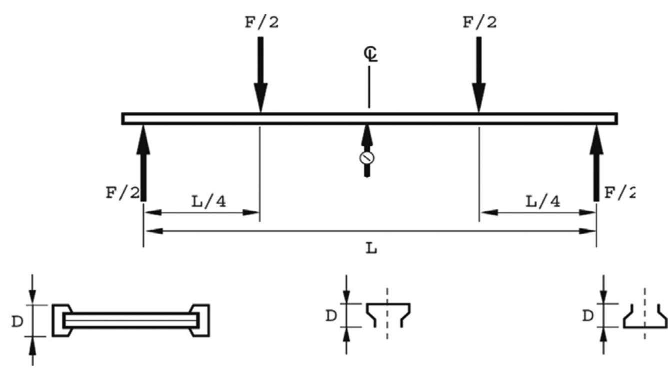

Fig. 4. Experimental arrangements for bending test on uprights.

directly estimated.

2.2. The US approach

The RMI specifications require only the stub column tests: the US and the EU procedures are practically equivalent but the reduction factor Q_{US}^{N} is evaluated making reference to the minimum cross-sectional area ($A_{net,min}$) instead of the gross area, i.e.:

$$Q_{US}^{N} = \frac{R_d}{F_y \cdot A_{net,\,min}} \tag{2}$$

For the evaluation of the effective cross-section area, A_{eff}^{US}, the following equation, strictly depending on the member slenderness, is proposed:

$$A_{eff}^{US} = \left[1 - (1 - Q_{US}^{N})\left(\frac{F_n}{F_y}\right)^{Q_{US}^{N}}\right] \cdot A_{net,\,min} \tag{3}$$

where F_n is the nominal buckling stress and F_y is the tensile yielding stress of virgin material, corresponding to f_y according to the EU symbology.

As to the bending design, the values of the effective cross-section moduli are based on the only reduction factor for axial load Q_{US}^{N}, distinguishing the cases of resistance and stability checks, according to the more general AISI [5] procedure for cold-formed members. In particular, with reference to flexure along one principal axis:

- for the resistance check, the effective cross-section modulus $S_{e,eff}$ is defined as

$$S_{e,eff} = \left(0.5 + \frac{Q_{US}^{N}}{2}\right) \cdot S_e \tag{4}$$

where S_e is the elastic section modulus of the net cross-section for the extreme compression fiber at the stress F_y.

- for the stability checks, the effective cross-section modulus $S_{c,eff}$ is defined as

$$S_{c,eff} = \left[1 - \frac{\left(1 - Q_{US}^{N}\right)}{2}\left(\frac{F_n}{F_y}\right)^{Q_{US}^{N}}\right] \cdot S_c \tag{5}$$

where S_c is the elastic section modulus of the net cross-section for the extreme compression fiber at F_c, that is the elastic critical lateral–torsional buckling stress.

For hollow rectangular cross-sections, which are the core of the applications proposed in this two-parts paper, the members are not influenced by lateral–torsional buckling, if the member length is not greater than a limit value L_u defined as

$$L_u = \frac{0.36 C_b \pi}{F_y S_f} \cdot \sqrt{E I_y G I_t} \tag{6}$$

where C_b is a bending coefficient depending on the moment gradient, S_f is the elastic section modulus of a fully unreduced cross-section relative to the extreme compression fiber, E and G are Young's and the shear modulus, respectively, and I_t is the Saint-Venant torsion constant.

2.3. The EU and the US comparison

Independent of the considered cross-section geometry, a comparison in general terms between the approaches adopted to evaluate the effective cross-section performance appears to be of great interest for designers. As a consequence, attention has been initially focused on the assessment of the effective area. Making reference to the reduction factor for compression (Q^N), the EU reduction factor (Q_{EU}^{N}) results are never lower than the US one (Q_{US}^{N}):

$$Q_{EU}^{N} = Q_{US}^{N} \frac{A_{net,\,min}}{A} = Q_{US}^{N} \cdot \beta \tag{7}$$

where term β has been introduced to express the ratio between the net and the gross area of the cross-section (i.e. $A_{net,\,min}/A = \beta$).

Furthermore, the effective area is constant only according to EU specifications; from the previously introduced equations, the ratio between A_{eff}^{US} and A_{eff}^{EU} is independent of the value of the perforated area (factor β), as given by the expression:

$$\frac{A_{eff}^{US}}{A_{eff}^{EU}} = \frac{\left[1 - (1 - Q_{US}^{N})\left(F_n/F_y\right)^{Q_{US}^{N}}\right] \cdot A_{net,\,min}}{Q_{EU}^{N} \cdot A} = \frac{\left[1 - (1 - Q_{US}^{N})\left(F_n/F_y\right)^{Q_{US}^{N}}\right]}{Q_{US}^{N}} \tag{8}$$

Without perforations (i.e. $\beta = 1$), the ratio $A_{eff}^{US}/A_{eff}^{EU}$ is plotted in Fig. 5 versus the parameter F_n/F_y, which is considered in the sole US approach to account for member slenderness. It can be noted that the trend of the plotted curves is approximately linear and their slope depends on the Q^N value. The value of decreases with the increase of F_n/F_y. The US effective area is always greater than the EU one: this difference, which is null only when $Q^N = 1$, increases with

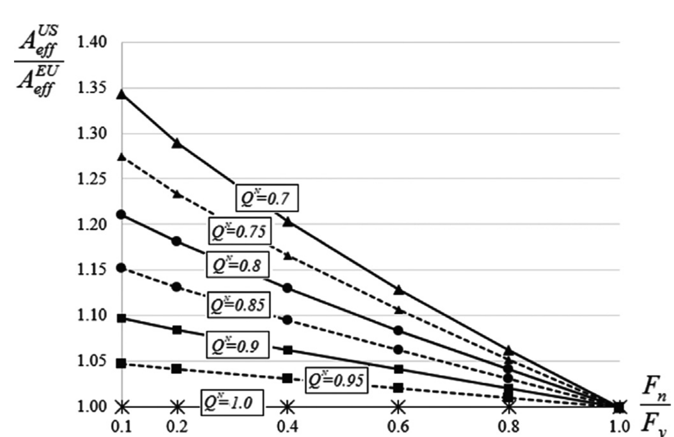

Fig. 5. $\left(A_{eff}^{US}/A_{eff}^{EU}\right) - \left(F_n/F_y\right)$ relationships for the case of cross-section without perforations.

the decrease of Q^N and appears absolutely nonnegligible, especially for stocky and moderately slender members.

A direct comparison with reference to the effective second moments of area or section moduli according to both codes is difficult, owing to the different approaches proposed by codes.

3. Verification rules for columns

Despite the fact that the case of pure axial load is quite rare in rack design, it appears necessary to briefly introduce the verification rules for columns, being them used to design uprights, i.e. members under axial load and bending moments.

3.1. The EU approach

According to the European design approach for class 4 columns in traditional steel frames [7] as well as for rack uprights [2], the axial load carrying capacity is evaluated as

$$N_{b,Rd} = \chi \cdot A_{eff} \frac{f_y}{\gamma_{M1}} = N_D^{EU} \tag{9}$$

where A_{eff} is the effective cross-sectional area, f_y is the yielding strength of the material, γ_{M1} is the partial safety factor and χ is a suitable reduction factor defined as

$$\chi = \frac{1}{\varphi + \sqrt{\varphi^2 - \bar{\lambda}^2}} \text{ with } \chi \le 1 \tag{10}$$

Being imperfection coefficient α=0.34, term φ is expressed, for rectangular cold-formed hollow cross-sections as

$$\varphi = 0.5 \cdot \left[1 + 0.34(\bar{\lambda} - 0.2) + \bar{\lambda}^2 \right] \tag{11}$$

The relative slenderness $\bar{\lambda}$ for class 4 (i.e. slender) members is defined as

$$\bar{\lambda} = \sqrt{\frac{A_{eff} \cdot f_y}{N_{cr}}} = \bar{\lambda}_{EU} \tag{12}$$

where N_{cr} is the elastic critical load for the appropriate buckling mode (flexural, torsional, or flexural–torsional) evaluated on the basis of the well-established theoretical approaches applied to the gross cross-section proprieties.

3.2. The US approach

No rules are directly given by the RMI specifications to evaluate the axial strength of compressed members and reference has to be made to the more general AISI approach. In particular, the nominal column resistance (P_n) is expressed as

$$P_n = 0.9 \cdot A_{eff} \cdot F_n = N_D^{US} \tag{13}$$

where F_n is the critical stress depending on the slenderness factor $\bar{\lambda}_C$ defined as

$$\bar{\lambda}_C = \sqrt{\frac{F_y}{F_e}} = \bar{\lambda}_{US} \tag{14}$$

with F_y representing the tensile yielding stress of virgin material and F_e is the lesser of the flexural, torsional or flexural–torsional elastic buckling stresses.

In particular:

- if $\bar{\lambda}_C \le 1.5$, $F_n = \left(0.658^{\bar{\lambda}_C^2}\right) F_y$ (15a)

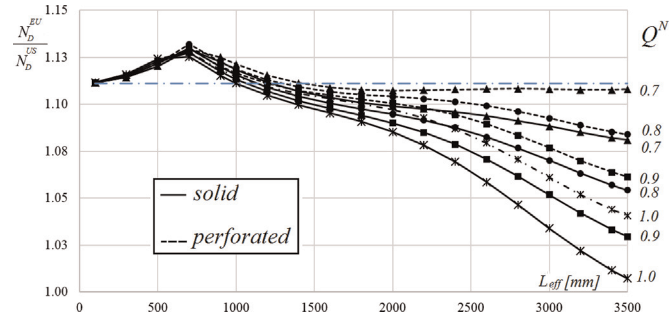

Fig. 6. $\left(N_D^{EU}/N_D^{US}\right) - L_{eff}$ relationships for a compression member for the case of $\beta = 1$ (solid line) and $\beta = 0.95$ (dashed line).

- if $\bar{\lambda}_C > 1.5$, $F_n = \left(\frac{0.877}{\bar{\lambda}_C}\right) F_y$ (15b)

3.3. Comparative applications

The member slenderness values according to both codes can be directly compared only when the net area coincides with the gross cross-section one (i.e. no perforations or, equivalently, β=1). In this case, the previously observed overestimation of the US effective area with respect to the EU one is balanced by the definition of the member slenderness, which is less severe according to the EU code, depending directly on the $Q^N (= Q_{EU}^N = Q_{US}^N)$ factor via the relationship:

$$\bar{\lambda}_{EU} = \bar{\lambda}_{US} \cdot \sqrt{Q^N} \tag{16}$$

Completely different approaches are followed by the codes for the evaluation of the stability strength. Non-negligible differences are hence expected in the column load carrying capacity, due also to the previously discussed differences in the evaluation of Q^N as well as of the effective area. As an example, which is sufficiently representative of the cases commonly encountered in routine rack design, reference should be made to Fig. 6 where the ratio between the EU (N_D^{EU}) and US (N_D^{US}) strength is plotted versus the effective column length L_{eff} for the cases of solid cross-section (solid line for β=1), and cross-section with a perforated area corresponding to 5% of the gross-one (dashed line for β=0,95). It can be noted that:

- the plotted curves start approximately from 1.11 for the lowest effective length (i.e. 100 mm, that is related to very stocky columns), independent of the Q^N value;
- the trend is similar for all the plotted curves: slightly increasing up to 650 mm, approximately, corresponding to the EU transition from stocky to slender columns (i.e. $\chi < 1$ for $L_{eff} > 650$ mm). Corresponding to this value, the maximum differences can be observed between the EU and the US predicted strengths: the ratio N_D^{EU}/N_D^{US} is practically constant, ranging from 1.125 (Q=1 and β=1) and 1.132 (Q=0.7 and β=0.95). When the EU χ factor decreases from unity, the N_D^{EU}/N_D^{US} ratio decreases significantly, too;
- the EU strength is always greater than the US one. Only for very slender member, with $L_{eff} > 4000$ mm, out of interest for the design of unbraced pallet racks, the US rules give greater values of the axial strength;
- reducing the Q^N value, the ratio N_D^{EU}/N_D^{US} increases remarkably with the increase of L_{eff} and for $Q^N \ge 0.7$ it tends again to the value of 1.11.
- increasing the value of the perforated area (i.e. reducing from unity the β parameter) the difference between the predicted strength performance in general increases and the US approach

is remarkably more severe when the cross-section is perforated.

It is worth mentioning that the aforementioned limit of 1.11 is given by the ratio 1/0.9 depends on the safety factors contained in the verification equations.

4. Verification rules for beams

As in the case of pure compression, also the rules for bending have to be introduced, owing to their importance for the verification of beam–columns. Attention is paid solely to the resistance, neglecting the serviceability (deflection, vibration and torsion) checks as well as the checks associated with lateral buckling, out of interest in the present study focused on tubular members.

4.1. The EU approach

No rules are directly reported in EN15512, which makes reference to the EN 1993-1-3 provisions [16]. In particular, for class 4 members, the design moment resistance of a cross-section for bending about one principal axis $M_{c,Rd}$ is determined as

$$M_{c,Rd} = \frac{W_{eff} \cdot f_y}{\gamma_M} = M_D^{EU} \tag{17}$$

where W_{eff} is effective section modulus, f_y is the yielding strength of the material and γ_M is the material safety factor.

4.2. The US approach

The beam moment resistance (M_n) for slender members in the case of lateral–torsional buckling prevented is evaluated as

$$M_n = 0.9 \cdot (S_e \cdot F_y) = M_D^{US} \tag{18}$$

where S_e is effective section modulus and F_y is the material yielding stress.

5. Verification rules for beam–columns

In addition to the axial force, N_{Ed}, uprights are generally subjected to bending moments acting in the down-aisle plane, $M_{y,Ed}$, due to the degree of flexural continuity of beam-to-column joints. Furthermore, also the moment $M_{z,Ed}$ has to be considered, which acts in the cross-aisle direction owing to the eccentricity of the connections of the bracing members forming the upright frames. In the following, attention is focused on design cases in which lateral buckling cannot occur, owing to the considered hollow cross-section types.

5.1. The EU approach

According to European rack specifications [2], uprights are designed correctly if the safety index (SI^{EU}) fulfils the condition:

$$SI^{EU} = \frac{N_{Ed}}{\frac{\chi_{min} \cdot A_{eff} \cdot f_y}{\gamma_{M1}}} + \frac{k_y \cdot M_{y,Ed}}{\frac{W_{eff,y} \cdot f_y}{\gamma_{M1}}} + \frac{k_z \cdot M_{z,Ed}}{\frac{W_{eff,z} \cdot f_y}{\gamma_{M1}}} \le 1 \tag{19}$$

where A_{eff} and W_{eff} indicate the area and the section modulus of the effective cross-section, respectively, f_y is the material yield strength, subscripts y and z identify the principal axes of the cross-section and γ_{M1} is the material safety factor.

The term k_j (where the subscript j corresponds either to the y- or to the z-axis) is defined as

$$k_j = \min\left[1.0;\ 1 - \frac{\mu_j \cdot N_{Ed}}{\chi_j \cdot A_{eff} \cdot f_y}\right] \tag{20a}$$

where the non-dimensional term μ_j is evaluated as

$$\mu_j = \min\left[0.9;\ \bar{\lambda}_j \cdot (2\beta_{Mj} - 4)\right] \tag{20b}$$

The coefficient β_{Mj} in Eq. (20b) takes into account the bending moment distribution along the longitudinal upright axis. In the case of linear bending distributions, which is frequently observed along the uprights in pallet racks, if $M_{j,Ed,Max}$ and $M_{j,Ed,min}$ indicate the bending moments at the member ends (with $M_{j,Ed,Max} > M_{j,Ed,min}$), the term β_{Mj} is given by the expression:

$$\beta_{Mj} = 1.8 - 0.7 \frac{M_{j,Ed,\min}}{M_{j,Ed,Max}} \tag{20c}$$

Furthermore, it should be noted that this design approach was already proposed in the previous ENV version of EC3 [24] but it has been removed from the updated EN version [7], due to its inaccuracy not only for doubly-symmetric beam–columns but also for members having mono-symmetric cross-section [25]. A similar action has not been adopted for the EN 15512, despite the fact that this code was updated later than EC3.

5.2. The US approach

The equation for the verification of members under compression and bending according to the US code is reported in the AISI provisions. It is worth to mention that both the EU and the US codes adopt different symbols to identify the cross-section principal axes: y and z in the former and x and y in the latter. For the sake of simplicity, the EU symbols are in the following always used for both codes. In particular, on the basis of the values of the design axial load (P) and bending moments (M_y and M_z), taking into account that racks are often analyzed via second-order analysis, it is required that:

$$SI^{US} = \frac{P_r}{0.9 \cdot P_n} + \frac{M_x}{0.9 \cdot M_{nx}} + \frac{M_y}{0.9 \cdot M_{ny}} \le 1 \tag{21}$$

where P_n is the already defined nominal compression member capacity (Eq. (13)) and M_{nx} and M_{ny} are the nominal bending flexural capacities (Eq. (18)).

This equation is very similar to the EU one but a remarkable difference is in the bending coefficients that according to the US procedure are constant instead of a function of the moment distribution and member slenderness when Eq. (19) is used.

5.3. Comparative applications

From the designer's point of view, a direct comparison between the EU and US predicted performance for beam–column uprigths should be of great interest. The case of axial load and bending moment due to the beam-to-column joint semi-continuity (i.e. moments acting along one axis) have been considered, which is of interest for routine design. As a consequence, the axial load (N_D^j)–bending moment ($M_{y,D}^j$) design domains have been evaluated, where superscript j identifies the design code of interest. It should be noted that $M_{y,D}$ is related to the y-axis in accordance with EU standard, which, in accordance with the US code is identified as the x-axis. Reference has been made to an isolated member having a square cross-section and the same effective length along both principal axes (L_{eff}) but the validity of the proposed research outcomes extends to all cases of interest for routine design. A constant axial load has been applied to the member together with a gradient moment expressed by means of parameter ψ, defined as the

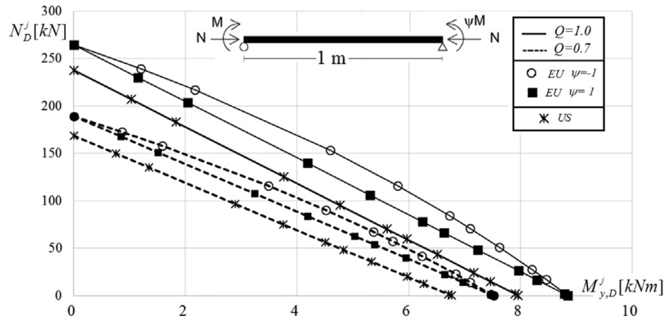

Fig. 7. Axial load (N^j_D)–bending moment ($M^j_{y,D}$) resistance domains for a beam–column having an effective length of 1 m ($\beta = 1$).

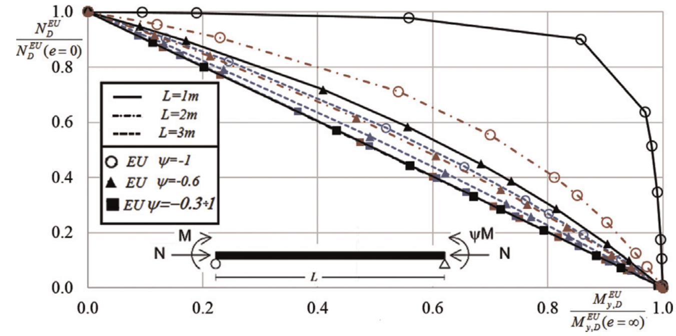

Fig. 9. EU axial load ($N^{EU}_D / N^{EU}_D(e=0)$)–bending moment ($M^{EU}_{y,D} / M^{EU}_{y,D}(e=\infty)$) non-dimensional domains for a beam–columns without perforations ($\beta = 1$).

ratio between the minimum and the maximum end moment: three representative values of ψ (1, 0 and -1) and three different values of L_{eff} (1 m, 2 m and 3 m) have been considered. Eccentricity (e) of the axial load with respect to the cross-section centroid has been varied from zero (column) to infinity (beam) and bending moment has been defined as $M^j_{y,D} = N^j_D \cdot e$.

In the case of uprights without perforations, the N^j_D– $M^j_{y,D}$ domains, which can be determined via Eqs. (19) and (21) for the EU and US approaches, respectively, are proposed in Fig. 7 ($L_{eff} = 1000$ mm) and Fig. 8 ($L_{eff} = 3000$ mm). Each of them is related to the cases of uniform moment (ψ=1) and gradient moment (ψ=0 and -1) for the cases of $Q^N = 1$ (solid line) and $Q^N = 0.7$ (dashed line). It can be noted that:

- the trend of the plotted domains is quite independent of the considered Q^N value;
- with reference to the US approach, the $N^{US}_D - M^{US}_{y,D}$ domains are always linear and independent of the moment distribution, owing to the linearity between axial load and bending moment and to the absence of moment distribution coefficient;
- the US domains are always the more severe ones and the differences from the EU domains increase with the decrease of ψ;
- the moment distribution influences remarkably the EU domains: in particular, when ψ =1 and 0 the associated domains are always quite linear and coincident with each other: in the case of opposite end equal moments ($\psi = -1$) the domain results concave. The discrepancy from linearity increases with the decrease of ψ and the increase of L_{eff}, as it appears clearly from Fig. 9 focused on solely the EU domains. In particular, the additional case of $\psi = -0.6$ has been considered together with $\psi = -1$ and the associated domains are proposed in non-dimensional form dividing the axial resistance $N^{EU}_D(e)$ for the column strength, $N^{EU}_D(e = 0)$, and the bending resistance $M^{EU}_{y,D}(e)$ for the beam strength, $M^{EU}_{y,D}(e = \infty)$. Decreasing ψ, the concavity of the domain increases significantly and this difference depends strongly on the L_{eff} value.

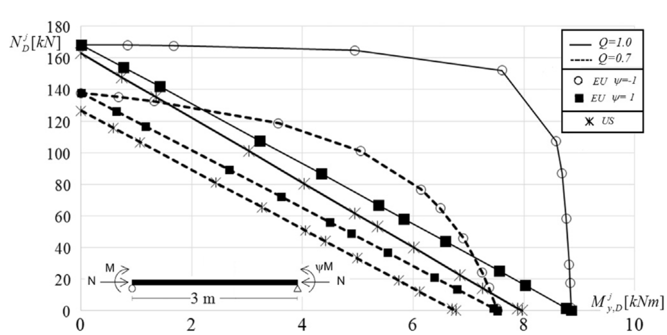

Fig. 8. Axial load (N^j_D)–bending moment ($M^j_{y,D}$) resistance domains for a beam–column having an effective length of 3 m ($\beta = 1$).

It has to be pointed out that, in the framework of a previous study on mono-symmetric beam–columns subjected to lateral buckling [11], the European domains have always been presented as linear, independent of the ψ value. It is due to the values, practically constant, assumed by the lateral buckling moment coefficient k_{LT}, to be used instead of k_y, adopted in the present analysis.

To investigate more accurately the differences of the beam–column performances, a suitable parameter expressing the member load carrying capacity (LCC^j) has been defined (Fig. 10) as

$$LCC^j = \sqrt{\left(n^j_D(e)\right)^2 + \left(m^j_{y,D}(e)\right)^2} \tag{22}$$

where j identifies the code (i.e. j=EU or US), $n^j_D(e)$ and $m^j_{y,D}(e)$ are the non-dimensional axial load and bending design values, respectively, defined with respect to the more severe US code, as

$$n^j_D(e) = \frac{N^j_D(e)}{N^{US}_D(e = 0)} \tag{23a}$$

$$m^j_{y,D}(e) = \frac{M^j_{y,D}(e)}{M^{US}_{y,D}(e = \infty)} \tag{23b}$$

Tables 1 and 2 are related to the cases of Q^N=1 and 0.7, respectively: in each table, for the three considered different values of the effective length, the ratio between the EU and US load carrying capacity (i.e. LCC^{EU}/LCC^{US}) is reported for representative values of ψ and considering both cases of β=1 and 0.95. A unique value is proposed for ψ ranging from 1.0 to -0.3, because the EU strength results constant for each moment distribution within this range, owing to the fact that the moment coefficients k_y do not

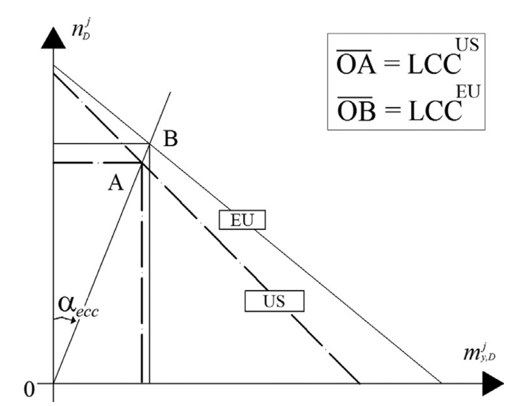

Fig. 10. Definition of the load carrying capacity (LCC^L) for the generic value of the eccentricity angle α_{ecc}.

Table 1

Values of the $^{LCC^{EU}}/_{LCC^{US}}$ ratio when $Q^N=1.0$ for $\beta=1.0$ and 0.95 (eccentricity e in millimeters).

L_{eff}	e	$\beta=1.0$					$\beta=0.95$				
		ψ									
		−1.0	−0.8	−0.6	−0.4	−0.3 ÷ 0.1	−1.0	−0.8	−0.6	−0.4	−0.3 ÷ 0.1
1 m	0	1.111					1.061				
	5	1.156	1.142	1.130	1.118	1.112	1.161	1.131	1.103	1.078	1.067
	10	1.184	1.162	1.141	1.122	1.112	1.259	1.191	1.137	1.091	1.071
	30	1.221	1.185	1.154	1.126	1.113	1.611	1.345	1.210	1.118	1.082
	50	1.216	1.182	1.153	1.126	1.113	1.793	1.390	1.230	1.128	1.088
	80	1.199	1.172	1.147	1.124	1.113	1.688	1.374	1.227	1.131	1.093
	100	1.190	1.165	1.143	1.122	1.113	1.593	1.350	1.220	1.131	1.095
	150	1.171	1.153	1.136	1.120	1.112	1.445	1.298	1.201	1.128	1.098
	300	1.146	1.136	1.126	1.116	1.112	1.280	1.217	1.165	1.121	1.102
	500	1.133	1.127	1.121	1.115	1.112	1.211	1.176	1.144	1.116	1.103
	5000	1.114	1.113	1.112	1.111	1.111	1.116	1.113	1.110	1.107	1.105
	∞	1.111					1.105				
2 m	0	1.085					1.097				
	5	1.170	1.145	1.121	1.100	1.090	1.177	1.153	1.130	1.109	1.099
	10	1.239	1.189	1.146	1.110	1.093	1.242	1.194	1.153	1.117	1.101
	30	1.372	1.265	1.187	1.126	1.100	1.365	1.264	1.189	1.130	1.104
	50	1.378	1.270	1.191	1.130	1.103	1.370	1.267	1.191	1.131	1.105
	80	1.338	1.251	1.184	1.129	1.106	1.331	1.247	1.182	1.129	1.106
	100	1.311	1.236	1.177	1.128	1.107	1.304	1.232	1.175	1.127	1.106
	150	1.261	1.208	1.164	1.126	1.108	1.255	1.204	1.161	1.123	1.106
	300	1.195	1.168	1.143	1.120	1.110	1.190	1.163	1.139	1.116	1.106
	500	1.163	1.147	1.132	1.117	1.110	1.158	1.142	1.127	1.113	1.106
	5000	1.117	1.115	1.113	1.112	1.111	1.111	1.109	1.108	1.106	1.105
	∞	1.111					1.105				
3 m	0	1.034					1.061				
	5	1.138	1.107	1.079	1.054	1.042	1.161	1.131	1.103	1.078	1.067
	10	1.240	1.171	1.115	1.069	1.049	1.259	1.191	1.137	1.091	1.071
	30	1.631	1.338	1.197	1.104	1.067	1.610	1.345	1.210	1.118	1.082
	50	1.887	1.390	1.222	1.117	1.077	1.792	1.390	1.230	1.128	1.088
	80	1.735	1.378	1.224	1.125	1.086	1.687	1.374	1.227	1.131	1.093
	100	1.623	1.355	1.218	1.126	1.089	1.594	1.351	1.220	1.131	1.095
	150	1.459	1.302	1.201	1.126	1.095	1.445	1.298	1.201	1.128	1.098
	300	1.287	1.221	1.168	1.122	1.102	1.280	1.217	1.165	1.121	1.102
	500	1.217	1.181	1.148	1.119	1.106	1.211	1.176	1.144	1.116	1.103
	5000	1.122	1.119	1.115	1.112	1.111	1.116	1.113	1.110	1.107	1.105
	∞	1.111					1.105				

change for these moment distributions. Furthermore, the ratio $^{LCC^{EU}}/_{LCC^{US}}$is plotted for these cases in Fig. 11 (L_{eff}=1 m) and 12 (L_{eff}=3 m) versus eccentricity angle α_{ecc} ranging from 0° (only axial load, no bending moment) to 90° (only bending moment, no axial load) for $\psi=-1$, -0.6 and ψ comprised between 1 and -0.3, distinguishing $Q^N = 1$ (solid line) and $Q^N = 0.7$ (dashed line). Both tables and figures show that:

European approaches lead always to a less conservative evaluation of the load carrying capacity and the differences between LCC^{EU} and LCC^{US} decrease with the increasing of the eccentricity angle.

- for $e=0$ and ∞ (or equivalently $\alpha_{ecc} = 0°$ and 90°), each curve tends to the value of 1.11, corresponding, as already introduced, to 1/0.9;
- with the exception of the data associated with the case $Q^N = 0.7$ and $\psi \in [-0.3, 1.0]$, the trend of the $\left(^{LCC^{EU}}/_{LCC^{US}}\right) - \alpha_{ecc}$ relationships is similar, rapidly increasing with a maximum generally in the range of eccentricity between 30 mm and 80 mm, otherwise it decreases monotonically;
- the ratio $^{LCC^{EU}}/_{LCC^{US}}$ depends strongly on the effective length values and assumes values non-negligible from the design point of view up to 1.22 ($Q^N = 1$) and 1.19 ($Q^N = 0.7$) for L_{eff}=1 m and up to 1.89 ($Q^N = 1$) and 1.55 ($Q^N = 0.7$) for L_{eff}=3 m (Fig. 12);
- the presence of perforations increases the difference of the load carrying capacity, especially for the lowest values of L_{eff}: as it can be noted from the tables, the maximum values are up to 1.79 ($Q^N = 1$) and 1.53 ($Q^N = 0.7$) for L_{eff}=1 m and up to 1.79 ($Q^N = 1$) and 1.53 ($Q^N = 0.7$) for L_{eff}=3 m.

6. The methods of analysis

Steel storage pallet racks are structures very sensitive to lateral loads, owing to the slenderness of the uprights, to the modest degree of rotational stiffness of beam-to-column joints and base–plate connections [26] and to the absence of longitudinal bracing the down-aisle direction. As a consequence, a second-order analysis Is often required in routine design, which could sometimes be developed also via approximated approaches. moreover, owing to the extensive use of thin-walled cold-formed members, the traditional design methods of analysis proposed for the traditional frames made by hot-rolled members cannot be directly adopted.

6.1. The EU approach

Structural analysis of racks could be carried out via the following methods, identified for the sake of simplicity, as:

- EU-DAM: *Direct Analysis Method* (specified in EN15512 subchapter 10.1.3);

Table 2

Values of the ${}^{LCC^{EU}}\!/_{LCC^{US}}$ ratio when Q^N=0.7 for β=1.0 and 0.95 (eccentricity e in millimeters).

L_{eff}	e	β=1.0					β=0.95				
		ψ									
		−1.0	−0.8	−0.6	−0.4	−0.3 ÷ 1.0	−1.0	−0.8	−0.6	−0.4	−0.3 ÷ 1.0
1 m	0	1.119					1.108				
	5	1.150	1.140	1.131	1.123	1.118	1.186	1.162	1.140	1.119	1.110
	10	1.171	1.155	1.139	1.125	1.118	1.256	1.207	1.164	1.128	1.111
	30	1.202	1.175	1.150	1.128	1.117	1.460	1.313	1.216	1.143	1.113
	50	1.202	1.175	1.150	1.127	1.116	1.531	1.343	1.229	1.147	1.114
	80	1.191	1.167	1.145	1.125	1.115	1.504	1.332	1.225	1.146	1.114
	100	1.183	1.162	1.142	1.123	1.114	1.464	1.316	1.218	1.144	1.114
	150	1.168	1.152	1.136	1.121	1.114	1.380	1.276	1.200	1.139	1.113
	300	1.145	1.136	1.126	1.117	1.113	1.261	1.210	1.167	1.129	1.112
	500	1.133	1.127	1.121	1.115	1.112	1.204	1.174	1.148	1.123	1.112
	5000	1.114	1.113	1.112	1.112	1.111	1.120	1.117	1.115	1.112	1.111
	∞	1.111					1.110				
2 m	0	1.099					1.107				
	5	1.161	1.142	1.125	1.109	1.101	1.165	1.148	1.132	1.116	1.109
	10	1.210	1.175	1.144	1.116	1.103	1.212	1.179	1.149	1.122	1.110
	30	1.308	1.235	1.176	1.128	1.107	1.304	1.235	1.179	1.132	1.111
	50	1.321	1.243	1.181	1.131	1.108	1.317	1.243	1.183	1.134	1.112
	80	1.299	1.231	1.176	1.130	1.110	1.297	1.231	1.177	1.132	1.112
	100	1.281	1.221	1.171	1.129	1.110	1.279	1.221	1.172	1.131	1.112
	150	1.244	1.199	1.160	1.126	1.111	1.243	1.199	1.161	1.127	1.112
	300	1.189	1.164	1.142	1.121	1.111	1.188	1.164	1.142	1.121	1.111
	500	1.160	1.145	1.131	1.118	1.111	1.160	1.145	1.131	1.118	1.111
	5000	1.116	1.115	1.113	1.112	1.111	1.116	1.114	1.113	1.111	1.111
	∞	1.111					1.110				
3 m	0	1.088					1.108				
	5	1.169	1.145	1.122	1.101	1.092	1.186	1.162	1.140	1.119	1.110
	10	1.244	1.193	1.149	1.111	1.094	1.256	1.207	1.164	1.128	1.111
	30	1.464	1.307	1.206	1.132	1.101	1.460	1.313	1.216	1.143	1.113
	50	1.544	1.341	1.222	1.138	1.104	1.531	1.343	1.229	1.147	1.114
	80	1.514	1.332	1.219	1.139	1.106	1.504	1.332	1.225	1.146	1.114
	100	1.472	1.315	1.213	1.138	1.107	1.465	1.316	1.218	1.144	1.114
	150	1.383	1.275	1.197	1.135	1.109	1.380	1.276	1.200	1.139	1.113
	300	1.261	1.209	1.165	1.127	1.110	1.261	1.210	1.167	1.129	1.112
	500	1.204	1.174	1.147	1.122	1.110	1.204	1.174	1.148	1.123	1.112
	5000	1.121	1.118	1.115	1.112	1.111	1.120	1.117	1.115	1.112	1.111
	∞	1.111					1.110				

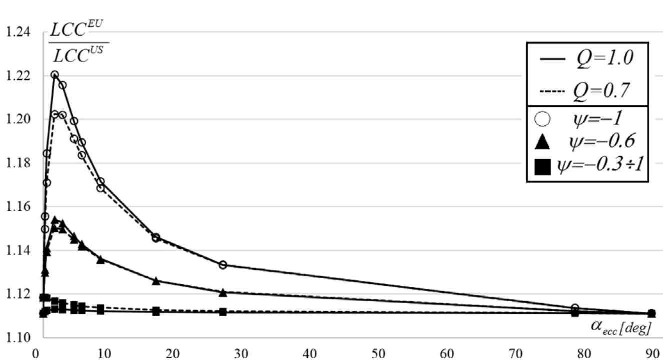

Fig. 11. ${}^{LCC^{EU}}\!/_{LCC^{US}} - \alpha_{ecc}$ relationships (L_{eff}=1 m).

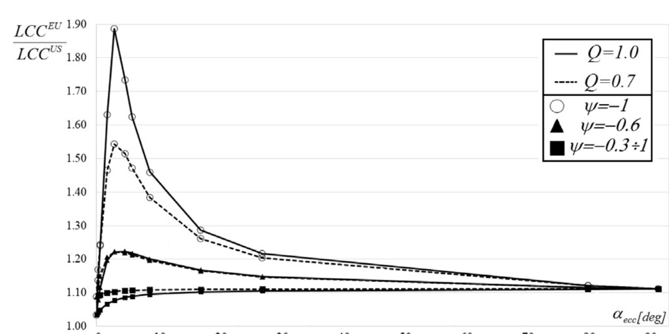

Fig. 12. $\frac{LCC^{EU}}{LCC^{US}} - \alpha_{ecc}$ relationships (L_{eff} =3 m).

- EU-RAM: *Rigorous Analysis Method* (specified in EN15512 sub-chapter 9.7.6);
- EU-GEM: *General Method* (specified in EN1993-1-1 sub-chapter 6.3.4).

Furthermore, the Improved Rigorous Analysis Method (*IRAM*) appears as an efficient alternative for rack design, which is actually not included among the design alternatives. It is worth mentioning that attention in previous research [10] had been paid only to the EU-RAM and the EU-IRAM approaches and hence the application of EU-DAM and EU-GEM approach to rack design is quite innovative. Key features of the four methods are summarized in Table 3.

6.1.1. *EU-DAM*

The *direct analysis method (DAM)* is an advanced three-dimensional analysis described in EN 15512. An accurate finite element spatial rack model is required including both overall rack and member imperfections and joint eccentricities, where relevant. A more detailed discussion related to the geometric imperfections is proposed in the companion paper [12]. Furthermore, as clearly stated in the code, rack design has to be based on the results of a second-order structural analysis carried out via refined

Table 3
Key features of the EU approaches for pallet rack design analysis.

Feature	Approach			
	EU-DAM	EU-RAM	EU-IRAM	EU-GEM
Lack-of-verticality (sway) imperfections	Yes (direct modeling or notional nodal loads)	Yes	Yes	Yes
Out-of-straightness (bow) imperfections	Yes (direct modeling or notional nodal loads)	No	No	No
Member (local) second-order effects	Yes by direct analysis	No (already considered by formulas for member stability verification)		
Member stability checks	No	Yes	Yes	
Overall stability	No	No	No	Yes
Buckling length	Not required	System length with $K=1$.	Effective length with $K=K(N_{cr})$.	

finite element analysis packages able to capture accurately the flexural–torsional buckling of the members and the influence of warping deformations on torsional buckling, warping torsion and shear center eccentricity. The coincidence between centroid and shear center, which characterizes the hollow cross-sections considered in the present research, simplifies structural analysis and, as a consequence, resistance checks. Neglecting warping torsion, it is in fact required that:

$$SI^{EU-DAM} = \frac{N_{Ed}}{N_{Rk}} + \frac{M_{y,Ed}}{M_{y,Rk}} + \frac{M_{z,Ed}}{M_{z,Rk}} \leq 1 \tag{24}$$

6.1.2. EU-RAM

The *Rigorous Analysis Method* (RAM) takes into account the lack-of-verticality imperfections neglecting in the structural analysis the effects of out-of-straightness of members; EU-RAM is the most preferred approach by rack designers because of the less conservative values of the load carrying capacity. The EN15512 code declares in fact that the *structure shall be considered a no-sway frame and buckling lengths shall be put equal to system (geometrical) lengths*. Effective length for member stability checks results hence independent of the degree of rotational stiffness of beam-to-column joints and base–plate connections, without any distinction between braced and unbraced racks.

6.1.3. EU-GEM

Eurocode 3 in its part 1-1 [7] proposes an innovative [27–29] design approach, the so-called *general method* (GEM), appropriate also for structural components having geometrical, loading and/or supporting irregularities. Overall buckling resistance of the whole skeleton frame is verified when:

$$\frac{\chi_{op}\alpha_{ult.k}}{\gamma_M} \geq 1 \tag{25a}$$

With reference to the symbols already presented for the other design approaches, this verification criterion can be more conveniently expressed as

$$SI^{EU-GEM} = \frac{\gamma_M}{\chi_{op}\alpha_{ult.k}} \leq 1 \tag{25b}$$

where $\alpha_{ult,k}$ is the minimum load multiplier evaluated with regard to the cross-section resistance, χ_{op} is the buckling reduction factor referring to the overall structural system and γ_M is the material safety factor.

For routine design, upright cross-sections in classes 3 or 4 guarantee that plastic hinges do not form in the members. Overall failure is generally due to the interaction between upright instability and plasticity in beam-to-column joints and base–plate connections, as expected because of the use of thin-walled members and also confirmed by recent experimental research which included several full scale push-over rack tests [30–32]. The ultimate load multiplier for resistance, $\alpha_{ult,k}$ is determined as

$$\frac{1}{\alpha_{ult,k}} = \frac{N_{Ed}}{N_{Rk}} + \frac{M_{y,Ed}}{M_{y,Rk}} + \frac{M_{z,Ed}}{M_{z,Rk}} \tag{26a}$$

where N_{Rk} is the squash load and $M_{y,Rk}$ and $M_{z,Rk}$ are the first yielding moments of the perforated cross-section.

The different combinations of imperfections included in the DAM (sway and member imperfections) and GEM (only sway imperfections) approaches should hence lead to the different values of the internal forces and moments and, as a consequence, despite Eqs. (24) and (26a) being nominally equal, it results that:

$$\frac{1}{\alpha_{ult,k}} \neq SI^{EU-DAM} \tag{26b}$$

6.1.4. EU-IRAM

An open problem related to rack design is associated with the selection of the value of the effective length for stability verification checks, for which few alternatives are possible [8,9].

As shown on the basis of a parametric analysis [10] on a set of medium-rise pallet racks similar to the ones considered in the present research, rack performance should be quite different, owing to the influence of both the warping torsion and the choice of the effective length. The last method considered for rack design is the Improved Refined Analysis Method (IRAM), which differs from the RAM only for what concerns the evaluation of the effective length for the stability verification checks. As recommended also by the AS rack code [3], the IRAM approach is based on a second-order analysis and the effective sway length from a buckling analysis has to be used instead of the system lengths.

6.2. The US approach

RMI states that all computations for safe loads, stresses and deflections, have to be made in accordance with conventional methods of structural design, as specified in the AISI document [5]. As to the design of medium-rise pallet racks, as clearly stated also by Sarawit and Pekoz [33], two methods can be adopted, i.e.:

- US–NOLM: *Notional Load Method* (defined in AISC360 subchapter C1.1);
- US–ELM: *Effective Length Method* (defined in AISC360 subchapter C1.2 and discussed in Appendix 7.2).

Key features of these methods are summarized in Table 4 and herein shortly discussed.

6.2.1. US–NOLM

The notional load method Is the main suggested method that can be applied in every design case, without any kinds of limitations. this approach requires a second-order analysis, considering

Table 4
Key features of the US approaches for pallet rack design analysis.

Feature	Approach	
	US-NOLM	US–ELM
Lack-of-verticality (sway) imperfections	Yes (direct modeling or notional nodal loads)	
Out-of-straightness (bow) imperfections	No (already considered by formulas for member stability verification)	
Member (local) second-order effects-(P–δ)	Yes by direct analysis	No
Adjustments to stiffness	Yes (reduction to 80%)	No
Member stability checks	Yes	Yes
Buckling length	System length (K =1).	Effective length. With K from alignment chart or K=$K(N_{cr})$.

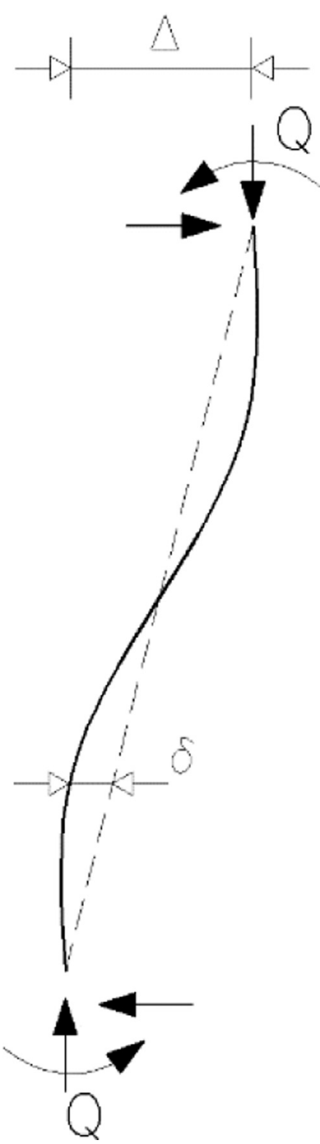

Fig. 13. Definition of the P–Δ and P–δ effects [5].

both P–δ and P–Δ effects (Fig. 13), together with flexural, shear and axial member deformations. All the P-δ effects are usually neglected in the analysis, but they must be adequately taken into account in strength evaluation for individual members subjected to compression and flexure.

Initial imperfections have to be accounted for and all steel properties contributing to the elastic stiffness should be multiplied by 0.8, i.e. a reduction of the structural stiffness to lateral load is imposed, mainly to account for the simplified approach used to evaluate the effective length of uprights, which is based on the system length. It should be noted that the same analysis approach is permitted also by the Australian rack standards [23,34].

6.2.2. US–ELM. the Effective Length Method can be applied to racks: a second-order analysis, like in US-NOLM, is required but the stiffness reduction has not to be applied and the imperfections have to be taken into account via notional loads. The evaluation of the member strengths is based on the effective length factor K for moment resisting (sway) frames or on the use of finite element buckling analysis.

6.3. Comparisons

As a general remark, it has to be noted that Eurocode proposes design alternatives without any type of limitations, meanwhile the US code provides the limits of applicability for the US-ELM method. With the exception of EC3–GEM, it is possible to note some differences, similarities and correspondences between the European and United States approaches, as it appears from Table 5. All the approaches require that the lack-of-verticality (sway) imperfections be taken into account via notional loads or the modeling of geometrically imperfect racks. Furthermore, neglecting the differences between the values of the sway imperfections, it can be noted that:

- EU-DAM and the US-NOLM approaches differ mainly for what concerns the upright verification: US stability checks are based on the system length while, according to EU code, only resistance checks are required;
- EU-RAM and US-NOLM assume the same value of the effective length but different performances are expected due to the stiffness reduction required by the sole US approach;
- significantly different values of the effective length should be also associated with EU-RAM and US-ELM approaches, which allow for the use of the system length and the buckling sway length, respectively;
- the EU-IRAM and US-ELM approaches present similarities for what regards major key design features. However, the equations used for the verification checks differ for what concerns the bending moment coefficients, and this reflects directly on the member performances, as it appears from the design domains of beam-columns discussed in Section 5.3.

7. Preliminary conclusions

A study on the rack design according to the European and United States provision has been developed and summarized in a two-parts paper: the first one is the present paper dealing with the general concepts and rules for isolated members.

Design assisted by testing is required by both codes: the effective cross-section properties according to the US practice depend on the member slenderness while the EU procedure allows the adoption of constant values obtained from suitable component tests. Furthermore, the codes are characterized by a different degree of complexity regarding the equations for verifying beam–columns: more parameters have to be determined according to EU design, which is based on a complex approach however removed from the European code for traditional steel buildings because unsafe [35]. In general, the EU performance of isolated members

Table 5
Similarities and differences between the EU and US design approaches.

Feature	EU – DAM	US – NOLM	EU-RAM	US-ELM	EU-IRAM	US – ELM
Lack of-verticality (sway imperfections)	Yes	Yes	Yes	Yes	Yes	Yes
Member out-of straightness imperfections	Yes	No	No	No	No	No
Stiffness reduction	No	Yes	No	No	No	No
Stability checks(K=effective length factor)	No	Yes (K =1)	Yes (K =1)	Yes ($K = K(N_{cr})$)	Yes ($K = K(N_{cr})$)	Yes ($K = K(N_{cr})$)

always results less conservative than the US ones and in particular, remarkably different values of the load carrying capacity can be observed, depending on the Q^N value, member length and moment distribution.

Finally, as to the permitted approaches for the structural analysis, four EU and two US alternatives have been discussed and compared in general terms, underlining similarities and differences. Their application is proposed in the companion paper [12], allowing for a concrete appraisal of the different guaranteed performances, which are directly associated with the cost and the competitiveness of the racks on the market.

Appendix A. List of symbols

Latin upper case letters

A gross cross-section area
A_{eff}, A_{eff}^{EU} A_{eff}^{US} effective cross-section area
$A_{net,min}$ minimum net cross-section area
AISC American Institute of Steel Construction
AISI American Iron and Steel Institute
ANSI American National Standards Institute
C_b bending moment distribution coefficient depending
D depth of the upright
DAM Direct Analysis Method
E Modulus of elasticity of steel
EC3 EN 1993-1-1 Eurocode 3 “Design of Steel Structures”
ELM Effective Length Method
RMI Rack Manufacturers Institute
EU Europe, European
FEM Federation Européenne de Manutention
F_n critical stress
F_e elastic buckling stress
F_y yielding strength
G shear material modulus
GEM GEneral Method
L member length, story height
LCC load carrying capacity
L_{eff} effective buckling length
L_u member length for flexural buckling instability
I_t Saint-Venant torsion constant
I_y, I_z second moment of area
K effective length factor
IRAM improved rigorous analysis method
M_D, M_D^{EU}, M_D^{US} design bending resistance
$M_{Ed}, M_{y,Ed}$, $M_{z,Ed}$ design bending moment
$M_{j,Ed,\,\mathrm{min}}$ or $M_{j,Ed,Max}$ minimum or maximum design bending moment
M_n, $M_{y,n}$, $M_{z,n}$ nominal bending resistance
M_{Rk} characteristic bending resistance
N_D, N_D^{EU}, N_D^{US} design axial resistance
N_{cr} critical load for the i-member
N, N_{Ed} member axial load
N_{Rk} characteristic axial resistance
$N_{b,Rd}$ axial stability resistance
P_c design axial strength
P_n nominal resistance strength for compression
Q,Q_{EU}^N, Q_{US}^N reduction factor for axial load
RAM Rigorous Analysis Method
R_d resistance
SI, SI^{EU}, SI^{US} design safety index
$S_{c,eff}$ effective cross-section modulus
S_c elastic section modulus of the net section for the extreme compression fiber
$S_{e,eff}$ effective cross-section modulus
S_e elastic section modulus of the net section for the extreme compression fiber
$SI^{j\text{-}k}$ safety index associated with the j-code and the k- design approach
S_f elastic gross section modulus relative to the extreme compression fiber
US United State of America
$W_{eff}, W_{eff,y}$ $W_{eff,z}$ effective cross-section modulus

For the sake of clarity, reference should be made also to Table B1. owing to the different symbols used in EU and US provisions to identify cross-section data.

Latin lower case letters

e_0 maximum out-of-straightness defect (bow) imperfection.
e eccentricity
h frame building height
k_j, k_z, k_y bending interaction factor
Max maximum value
min minimum value
f_y specified minimum yield stress strength
$n_D^j(e)$ non-dimensional axial load
$m_{y,D}^j(e)$ non-dimensional bending moment.

Greek case letters

α imperfection coefficient associated with the relevant buckling curve
α_{ecc} eccentricity angle
α_{cr} buckling overall frame multiplier obtained via a finite element buckling analysis
$\alpha_{ult,k}$ minimum load multiplier evaluated with reference to the cross-section resistance
β ratio between the perforated and the gross cross-section area
β_{Mj} bending moment distribution coefficient
δ bow imperfection displacements
Δ sway imperfection displacement
ψ gradient moment coefficient
$\bar{\lambda}_{op}$ relative slenderness of the whole structure

Table B1
Comparison between EU and US codes terminology.

EU	Term	US
N_{Ed}	axial force demand	P_r
$N_{b,Rd}$	design axial strength	P_n
$M_{y,Rk}$, $M_{z,Rk}$	design flexural strength about centroidal axes.	M_{cx}, M_{cy}
N_{cr}	elastic critical buckling load	P_e
W_{eff}	elastic section modulus of effective cross-section	S_e
LCC^{EU}	load carrying capacity	LCC^{US}
i_y, i_z	radius of gyration about symmetry centroidal axes.	r_x, r_y
Q_{EU}	reduction factor	Q_{US}
f_y	specified minimum yield stress strength	F_y
y–y,z–z	cross-section axes	x–x, y–y

$\bar{\lambda}_C$ slenderness factor
$\bar{\lambda}_{EU}$, $\bar{\lambda}_{US}$, $\bar{\lambda}$ relative slenderness
μ_j non-dimensional term for beam-column verification check
$\rho_{j,btc}$ parameter to define the elastic rotational stiffness of beam-to-column joints
χ reduction factor for the relative buckling curve
χ_{LT} reduction factor due to lateral buckling
χ_{op} buckling reduction factor referred to the overall structural system
$\gamma_M = \gamma_{M1}$ material safety factor

Appendix B

See Table B1.

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