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Unbraced steel frame design according to EC3 and AISC provisions

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ABSTRACT

One of the main results of increasing globalization is that nowadays design, fabrication and erection of steel structures can take place at different locations potentially separated by several thousand kilometres. As a consequence, owners might require the use of widely accepted steel design codes and designers should therefore be familiar with alternative specifications that may be substantially different from one another.
The paper deals with unbraced steel frames, and presents a comparison between the rules adopted by the European and United States design provisions. After a brief discussion on the key features of both codes, attention is focussed mainly on the different approaches to structural analysis. Furthermore, the results of a parametric study are described, highlighting the differences associated with the permissible design alternatives in terms of load carrying capacity. Reference is made to planar frames with differing geometry, load conditions, imperfection modelling and degree of the rotational stiffness of beam-to-column joints. Finally, Appendix A proposes a benchmark example reporting all details associated with the application of the design approaches under consideration.

1. Introduction

Independent of the finite element analysis packages available in engineering offices, routine design of steel structures is usually carried out following two separate steps:

- Structural analysis of the overall frame, aimed at evaluating the set of displacements, internal forces and moments and stress distributions for each frame component;
- Member safety checks, strictly depending on suitable criteria regarding deformability, resistance and stability. Particular care must also be paid to the verification of joints, not only in terms of strength but also with reference to the displacement and rotation requirements.

An increasing number of cases where design, fabrication and erection of steel structures take place at different locations has been observed in the last years, as a result of rapid globalization. Owners require the use of widely accepted steel design codes, regardless of the location where the structure has to be built; as a consequence, structural engineers are now faced with the challenge of being familiar with various design specifications, which could present substantial differences between one another. Two of the most widely used steel design specifications are for civil and industrial buildings located in Europe (EU) and the 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 [1], herein referred to as simply 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 was developed by the European Committee for Standardization. It allows exclusively for designing in accordance with the limit state design philosophy, corresponding to the AISC-LRFD approach. Furthermore, it should be noted that the AISC provisions are an integral document, complemented by a very exhaustive commentary [2], whereas EC3 consists of seven parts, each of them focused on a particular structural typology such as buildings, bridges, towers, and silos. General criteria for the design of buildings are specified in Part 1 of EC3, which is divided into 11 subparts and among these, part 1.1 (General rules and rules for buildings [3]), part 1.5 (Plated structural elements [4]) and part 1.8 (Design of joints [5]) are the main references for the design of traditional civil and industrial steel buildings.

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 to be used for the member/joint verification checks and safety factors accounting for material uncertainties. Furthermore, it should be pointed out that each of these codes permits alternative design paths, differing in terms of the degree of refinement of the structural analysis and for the rules regarding safety checking of components. As a result, differences in performance of designed structures are expected.

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From the designer's point of view, a great interest might be in the direct comparison between these options to evaluate differences in the load carrying capacity of steel frames of interest for routine practice. It is worth mentioning that several researchers have recently focused on the differences between EU and US requirements but only key phases of the design were separately analysed. Lacking in literature are complete and exhaustive overviews concerning all the design aspects, from the structural analysis phase to the member verification checks. In particular, Yong et al. [6] analysed the EU and US design approaches for isolated members subjected to combined axial compression and bending moment, showing on the basis of a comparative study, how these standards may disagree appreciably, especially for gradient moment distributions. Furthermore, Sousa and Barros [7] proposed a comparison for both concrete and steel framed buildings concerning the assessment of the effective length for the stability verification checks and concluding that no great differences could be observed between EU and US codes of practice. Finally, it is worth mentioning that a comparative study has been developed also with reference to cold-formed channels, focussing attention on the definition of the load carrying capacity of isolated columns [8].

This paper summarizes a research study aimed at analysing the design procedures according to EU and US practice for unbraced steel frames comprised hot-rolled members. Owing to the interest of the discussed topic for structural engineers familiar with the EU and/or US design provisions, it has been decided, in the following, to make reference to the symbols reported on each of these codes. Furthermore a comparative table is presented in the Appendix B, where for the main parameters governing design, both the EU and US associated symbols are proposed. In particular, key code requirements are briefly discussed and directly compared with one another, focussing attention not only on the structural analysis approaches but also on the geometrical imperfection modelling, which is an important aspect for routine steel design. A parametric study is conducted considering two planar frame configurations differing in geometry, load condition and degree of flexural continuity of beam-to-column joints. Research outcomes allow for a direct appraisal of the differences in terms of load-carrying capacity, or equivalently, of safety index of the frames. Furthermore, a complete design example is presented in Appendix A, where main results are reported to offer a benchmark to steel designers, highlighting, at the same time, differences associated with the considered design paths permitted by both codes.

2. The structural imperfections for unbraced frame design

The behaviour of steel structures is significantly affected by the presence of structural imperfections, which can be ascribed to various causes. Between them, the variability in lengths of the framing members, the lack of column verticality and beam horizontality and the possible errors in the location of the column bases and in the placement of the connections are worth mentioning. Furthermore, the member production processes, the variation of the yield strength and the residual stress distribution in the member cross-sections contribute to the definition of the load carrying capacity. Despite the fact that several refined approaches have been proposed to consider the effects of geometrical imperfections in advanced structural analyses, as clearly stated by Gu and Chan [9] and Shayan et al. [10], in the following attention is focused on the rules currently adopted for routine design of unbraced steel frames.

2.1. The European approach

In the case of frames subjected to large horizontal forces, geometric imperfections can usually be neglected as the magnitude of their effects is limited with respect to those associated with lateral loads. From a practical point of view, this occurs when:

$$H_{Ed} \geq 0.15 Q_{Ed} \tag{1}$$

where H_{Ed} represents the resultant of the horizontal forces at the base of all columns in one floor and Q_{Ed} is the resultant of the vertical force acting at the base of all columns in that floor.

The European design approach distinguishes between geometric imperfections in members and overall imperfections of the framed systems.

When it is necessary to include in structural analysis *member* (*initial out-of-straightness*) *imperfections*, a suitable system of self-balanced equivalent horizontal forces should be used as an alternative to the modelling of curved elements. It is worth noticing that the effects of local bow imperfections of members are incorporated within the formulas given for buckling resistance checks. Defining e_0 the maximum out-of-straightness defect (bow) imperfection with respect to the straight configuration (Fig. 1), it is possible to make reference to an equivalent uniformly distributed loads q_δ, generating a maximum bending moment $N_{Ed} \cdot e_0$, defined as:

$$q_\delta = \frac{8N_{Ed}}{L^2} e_0 \tag{2}$$

Depending on the analysis method used (i.e. elastic or plastic) and on the choice of the stability curve, the EC3 reference values for this type of imperfection have to be obtained from Table 1. An open problem, which is very important for routine design, is how to account in practice for member imperfections. Nowadays, very refined FE analysis packages are available and some of them, such as the Hungarian software ConSteel [11] used by the authors for the numerical study herein presented, allow for the direct definition of imperfections when meshing the frame. Alternatively, as shown by Rasmussen and Gilbert [12] discussing the new Australian rack design provisions [13], it appears sufficient to concentrate the geometric imperfections only in the two lower storeys, introducing at least two additional nodes per member to approximate the initial deformed (curved) shape of the columns via straight beam elements.

In the case of relatively high compression applied to the columns, the effects of out-of-straightness imperfections must always be accounted for in the structural analysis. In particular, with reference to sway frames, this occurs when:

$$N_{Ed} > 0.25 N_{cr} \tag{3}$$

in which N_{Ed} is the axial force acting on the element and N_{cr} is the critical elastic buckling load for the member.

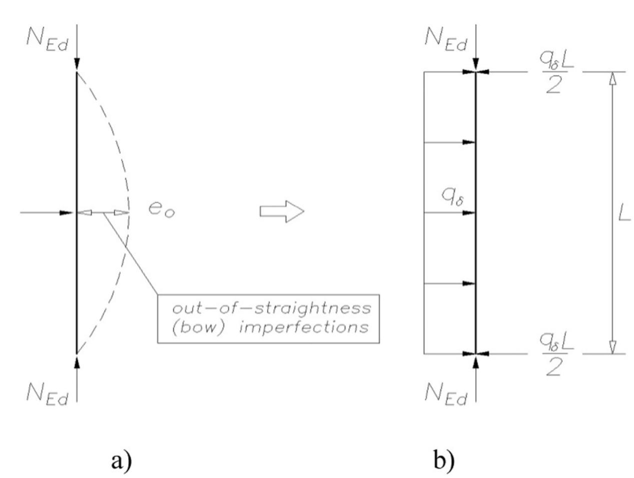

Fig. 1. Horizontal forces equivalent to the out-of-straightness imperfection (bow) according to EC3.