Structures 4 (2015) 91–104

Contents lists available at ScienceDirect

Structures

journal homepage: http://www.elsevier.com/locate/structures

On the influence of the load sequence on the structural reliability of steel members and frames

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ARTICLE INFO

Article history:
Received 20 July 2015
Received in revised form 13 October 2015
Accepted 14 October 2015
Available online 22 October 2015

Keywords:
Structural reliability
Steel structures
Eurocode
Stability
Load sequence

ABSTRACT

The influence of different load sequences on the deterministic resistance of steel structures (in the inelastic range) is a well-understood phenomenon. However, the impact of different load sequences on the reliability of members and frames made of steel has not been specifically studied in the past. Design rules for the stability and strength checks of such elements and structures are found, e.g., in Eurocode 3 [1]. The published background shows that load sequences and amplification patterns were not systematically included in the analysis of the reliability of the design rules for steel structures in Eurocode 3. In this paper, the impact of different load sequences on the reliability of three design rules or procedures (the resistance of plastic cross sections, of beam columns and of portal frame structures) is studied and illustrated by means of representative examples. The results show the significance of the load sequence at the level of scatter and non-exceedance probability of resistances. The paper finally discusses the implications of the study's findings for code-making, as well as the potential of accounting for the load sequence in the reliability assessment of, e.g., existing structures.

1. Introduction: overview, objectives and scope

1.1. Overview and objectives

Steel members and frames are commonly subjected to multiple load cases (e.g., dead, live and extraordinary loads) and internal forces (e.g., axial force, bending about either or both major bending axes). Contrary to basic, individual load cases and internal forces, for which the resistance of a given structural component can be expressed by a single quantity (e.g., as a resisting axial force for a column in pure compression), the resistance to these combinations of loads is thus represented by a multi-dimensional surface. An example for this is shown in Fig. 1, where the nominal cross-sectional resistance according to EN 1993-1-1 [1] of a plastic (class 1) or compact (class 2) I-shaped cross section against a design axial compression force N_E and bending moments $M_{y,E}$ and $M_{z,E}$ is represented by a three-dimensional surface.

Any point "R" on the resistance surface shown in Fig. 1 may be reached following an arbitrary load sequence. For example, in practical applications, a section may either (i) be first loaded by a certain axial load (represented by the normalized quantity "$n = N_E/Af_y = N_E/N_{pl,nom}$" in the figure) and then loaded in combined bending $M_{y,E} + M_{z,E}$ or (ii) be loaded by a proportionally increasing fixed ratio of $N_E + M_{y,E} + M_{z,E}$, in both cases until the failure surface is reached.

It is a well-known fact that whenever non-linearities and plasticity play a significant role, the *deterministic* strength of a steel component is influenced by the chosen load sequence, albeit this is not consistently reflected in common design rules. However, the *probabilistic* influence of the load sequence on the *reliability* of structural components made of steel is not commonly taken into account and is not specifically addressed in the Eurocode [2] rules. Specifically, the methodology included in EN 1990–Annex D for the assessment of the required resistance-sided partial safety factors γ_M to be used in combination with a given design rule to obtain the desired level of reliability does not address the question of load sequences.

Thus, as part of the European research project *Safebrictile [3]*, dedicated to the homogenization of the reliability level of Eurocode-based steel design rules, the authors set out to investigate the significance of the load sequence on the reliability of a number of steel structures design rules and to quantifiy its impact by means of relevant examples. In particular, the work focused on the changes caused by different load sequences to the – purely resistance-sided – *non-exceedance probability* of the nominal structural resistance according to the Eurocode. This can be described, in simple terms, as the probability of "encountering" structural components that – due to an inconvenient combination of geometric, material and imperfection quantitites – do not achieve the "nominal" resistance. The "nominal" resistance is thereby defined as the resistance given by the Eurocode design rules, omitting the codified partial safety factors γ_M (γ_{M0}, γ_{M1},...).

1.2. Scope of the study and paper

The preparatory and computational work for the study mentioned above was carried out in the context of a graduate thesis by the second

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http://dx.doi.org/10.1016/j.istruc.2015.10.007
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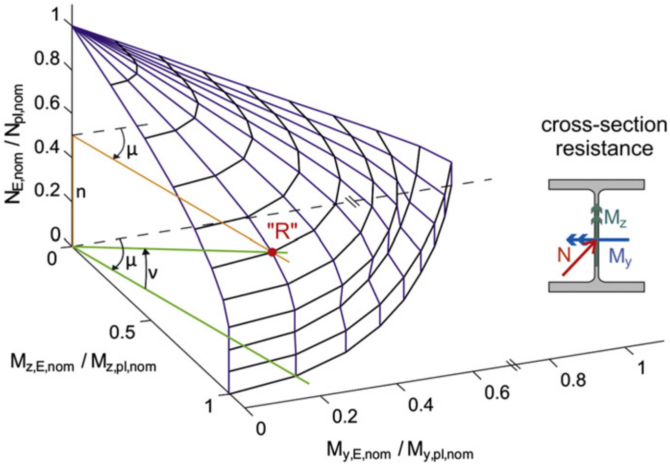

Fig. 1. Representation of the Eurocode 3 (EN 1993-1-1) plastic cross-sectional resistance function for axial force and bi-axial bending; two methods for the definition and calculation of the ultimate limit state load amplification factor: (i) amplification of all loads and internal forces simultaneously or (ii) amplification of bending moments only.

author [4], in which a variety of approaches for the structural reliability assessment of steel components and frames was discussed.

In the present paper, among the cases studied by Huemer [4], the three resistance functions and components/structures illustrated in Fig. 2 are treated, i.e.,

i. The plastic cross-sectional resistance of I- and H-sections loaded by axial compression N and bending M_y and M_z about both principal axes
ii. The in- and out-of-plane buckling resistance of I- and H-section beam columns subjected to axial compression N and strong-axis bending M_y
iii. The resistance of an exemplary planar portal frame structure subjected to various load combinations, causing normal stresses due to axial forces and major-axis bending moments in the individual components

In all cases, the resistance function was determined using the appropriate Eurocode 3 (EN 1993-1-1, [1]) rules. The exact formulae considered are reported in the pertaining sections of this paper.

It shall be noted that, in all calculations in this paper, only the epistemic uncertainty pertaining to the scatter of the basic input variables of the considered design resistance formulae was considered. The "model error" inherent to all design formulations, i.e., the difference between the "theoretical" resistance $r_{t.}$ and the real or experimental resistance r_e, was specifically omitted from the considerations made in this paper (see Fig. 3). In Huemer's study [4], the model uncertainty was included in the analysis of selected problems. However, the inclusion or omission of this additional uncertainty does not affect the validity of the results shown in this paper, as a comparative assessment of the difference in reliability levels is carried out here.

2. Reliability requirements of EN 1990 as used for steel structures

2.1. Probability of failure and reliability index β in EN 1990

The reliability of a structure or structural component corresponds to its ability to safely withstand the imposed actions and fulfil requirements of serviceability and durability. It is usually quantified by probabilistic measures, in particular, the probability of failure P_f and the reliability index β. Fig. 4 schematically illustrates the problem faced in determining structural reliability. The left-hand side of the figure shows two probability density functions (PDF) for the "effects of actions," for example, axial forces in a steel column caused by external dead and live loads, and "resistances," e.g., the maximum compressive strength (accounting for yielding and buckling) of this member. Both are represented schematically by PDFs for normally distributed variables. The member is "safe" as long as the resistance R exceeds the effect of actions E, R > E.

The right-hand side of the figure shows the combined PDF for the "safety margin," the so-called reliability function $g = R - E$. If this quantity is positive, the member or structure resists the considered effects of actions. If the quantity is negative, failure occurs. The probability of failure P_f (*Note:* by definition $0 \le P_f \le 1.0$) is given by the red area on the left of the Y-axis in the plot. For the assumption of a normally distributed quantity g, and for a mean value μ_g and standard deviation σ_g of g, the probability of failure P_f can also be expressed by "distances" $\beta \cdot \sigma_g$ of the mean value μ_g from the failure zone (where $g \le 0.0$). Finally, for normal distribution, the failure probability P_f can be expressed as follows:

$$P_f = \Phi(-\beta) \tag{1}$$

where Φ is the cumulative distribution function of the standardized normal distribution.

This means that any failure probability (for example, "accepted" values for code calibration) can equivalently be expressed by the "reliability index" β. The following Table 1 (see EN 1990 Annex C) shows some values for the relationship between β and the failure probability P_f:

Instead of directly stating recommended or "accepted" values of failure probability (for different reliability classes RC, see the next section), in the Eurocode—EN 1990, it was preferred to state recommended/accepted values of β. Note that this is however completely equivalent to a direct statement of accepted probabilities of failure.

EN 1990 differentiates between different reliability requirements by specifying three different levels of yearly accepted/recommended (notional) failure probability P_f, ranging from approximately 10^{-5} for the lowest RC1 and reaching 10^{-7} for RC3. For the most common class RC2, the yearly failure probability is $P_f \sim 10^{-6}$. The total recommended failure probability for the entirety of a design life corresponding $n = 50$ years (and thus corresponding to the reference return period for atmospheric actions) is approximately 50 times larger. EN 1990 specifies the above by providing – equivalently – values of the "reliability index β," as shown for the three RCs in Table 2.

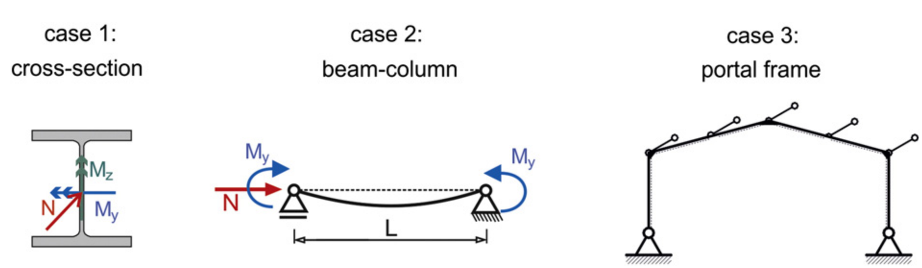

Fig. 2. Scope of the study: studied cases.