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Original Research Article

Safety design of steel beams with time-dependent stability

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ARTICLE INFO

Article history:
Received 7 July 2012
Accepted 7 October 2012
Available online 12 October 2012

Keywords:
Steel beams
Steel sections
Lateral torsional buckling
Time-dependent survival probability
Reliability index

ABSTRACT

The expediency of using probability-based approaches in the analysis of rolled and welded steel beams subjected to lateral torsional buckling is discussed. Buckling resisting moments and their mechanical uncertainties for rolled and equivalent welded flanged steel sections as particular members are analyzed. Time-dependent statistics of resisting and bending moments of steel beams is considered and their essential features are determined. Instantaneous and time-dependent survival probabilities of deteriorating and non-deteriorating steel beams in consecutive and cascade levels are also considered. Probabilistic design of steel beams with time-dependent stability is based on the generalized reliability index concept. Differences in design computation results of rolled and welded steel beams under lateral torsional buckling obtained by using the consecutive and cascade approaches are illustrated by a numerical example.

1. Introduction

The phenomenon of elastic lateral buckling of I-section beams has been investigated extensively long period and are few books on it, such [1,2].

Steel structures are required to satisfy torsional–flexural buckling resistance requirements for rolled and welded I sections. The lateral torsional buckling occurs in particular structural types of steel beams with a deep I section, whose compression flanges are insufficiently restrained against flexural action effects about the major axes of their sections, caused by heavy gravity loads. Due to the buckled positions of the deformed beams with open cross-sections presented in Fig. 1, tensile and compressive stresses develop in their top and bottom flanges, respectively. The dangerously high values of these stresses caused by torsional and lateral flexure effects of vertically applied loads may cause horizontal buckling of beam flanges, which cannot be completely protected from it by the stability of beam webs. Beams with the sufficiently restrained compression flanges may be unsusceptible to lateral torsional buckling.

The reliability class (RC) for buckling steel beams must be designated in the same class of consequences as that for the entire structure of buildings and civil engineering works. Failures and collapses of deep I sections may be caused not only by the gross human errors of designers or erectors, but also by statistical uncertainties of variable loads or some conditionality of recommendations and directions presented in semi-probabilistic design codes and standards [3–5].

The lateral torsional buckling criteria for unrestrained beams may be generally expressed as the critical values of either their compressive bending stresses or ultimate bending moments. However, the buckling resistance of beams depends not only on their geometric parameters, support conditions and torsional properties but also on mechanical and statistical features of rolled and equivalent welded sections, which may exert a significant influence on their structural safety. Regardless of fairly developed concepts of the theory of buckling resistance

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1644-9665/$ - see front matter Published by Elsevier Urban & Partner Sp. z o.o. on behalf of Politechnika Wrocławska.
http://dx.doi.org/10.1016/j.acme.2012.10.002

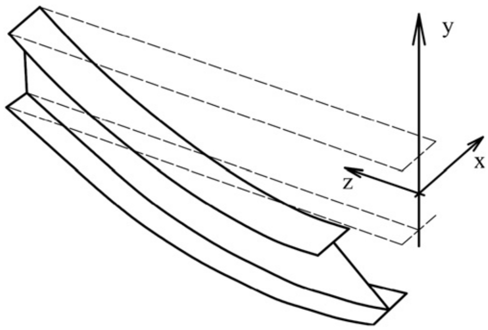

Fig. 1 – A schematic view of the buckled beams.

of beams, the relevant semi-probabilistic approaches of partial safety factor design yield different values of reliability indexes of the erected rolled welded steel sections.

When the performance of rolled members exposed to torsional–flexural buckling may be treated as perfectly sufficient, the structural safety of the equivalent welded beams may be insufficient. The probability-based assessment and prediction of time-dependent structural safety of non-deteriorating and deteriorating steel beams exposed to lateral torsional buckling may be inevitable in the cases, when their variable bending moments are caused by extreme, extraordinary or recurrent static and dynamic loads. The probabilistic analysis of buckling members subjected to sustained variable loads is fairly unsophisticated.

The time-dependent reliability index as a quantitative probabilistic parameter may enable engineers to make an easier and more accurate selection of optimal durability solutions for structures subject to aggressive environmental actions. The reliability index is indispensable for structural members exposed to recurrent regular and non-regular extreme actions.

The aim of the present paper is to assess the difference in reliability indexes of non-deteriorating and deteriorating rolled and welded beams with buckling flanges and to encourage designers to use the cascade resistance drop approach and probability-based methods for safety prediction in their design practice.

2. Buckling resisting moment

As was mentioned above stability problem of beams is investigated widely [2,6,7] and others. For practical purpose design codes are used. According to EN 1993-1-1 [8], the flanged beams loaded in the plane of the web and subject to major axis bending, which are shown in Fig. 1, should be verified against lateral torsional buckling as follows:

$$M_{Ed} < M_{b,Rd}, \tag{1}$$

where M_{Ed} is a design bending moment,

$$M_{b,Rd} = \chi_{LT} W_y f_y / \gamma_{M1} \tag{2}$$

is a design buckling resisting moment expressed by its reduction factor χ_{LT} and the appropriate section modulus W_y equal to $W_{pl,y}$, $W_{el,y}$ and $W_{eff,y}$ for cross-sections of class 1 or 2, 3 and 4. f_y is the nominal (characteristic) value of yield strength f_{yk}, while $\gamma_{M1}=1.0$ is its particular factor, and the design yield strength is equal to $f_y/\gamma_{M1}=f_{yk}$. Thus, a design buckling resisting moment is equal to its characteristic value, i.e. $M_{b,Rd}=M_{b,Rk}$.

For rolled and equivalent welded beams of constant cross-sections, the values of the reduction factors of buckling resisting moments may be determined from the following equation:

$$\chi_{LT} = \frac{1}{\Phi_{LT} + \left(\Phi_{LT}^2 - 0.75\bar{\lambda}_{LT}^2\right)^{0.5}}, \begin{cases} \le 1.0 \\ \le 1/\bar{\lambda}_{LT}^2 \end{cases}, \tag{3}$$

where the value for determining the reduction factor, Φ_{LT}, may be calculated as follows:

$$\Phi_{LT} = 0.5[1 + \alpha_{LT}(\bar{\lambda}_{LT} - 0.4) + 0.75\bar{\lambda}_{LT}^2] \tag{4}$$

It consists of the imperfection factor α_{LT} equal to 0.21, or 0.34 and 0.49, or 0.76 for rolled and welded sections, respectively, and their non-dimensional slenderness:

$$\bar{\lambda}_{LT} = \left(W_y f_y / M_{cr}\right)^{0.5}, \tag{5}$$

when M_{cr} is the elastic critical moment for the lateral torsional buckling. With the increase of the elastic critical moment, the non-dimensional slenderness of a beam, $\bar{\lambda}_{LT}$, decreases and the value of its buckling resistance moment may be improved.

In the case of a beam of uniform cross-section, symmetrical about the minor and major axes, the elastic critical moment for lateral torsional buckling is expressed by the formula:

$$M_{cr} = C_1 \frac{\pi^2 EI_z}{(kL)^2} \left\{ \left[\left(\frac{k}{k_w}\right)^2 \frac{I_w}{I_z} + \frac{(kL)^2 GI_t}{\pi^2 EI_z} + [C_2 z_g]^2 \right]^{0.5} - C_2 z_g \right\}, \tag{6}$$

where C_1, C_2 are factors, depending on the loading and end restraint conditions; k and k_w are the effective length factors; E is the modulus of elasticity; G is the shear modulus; L is the length of the beam between the points which have lateral restraint; I_t is the torsion constant; I_w is the warping constant; I_z is the second moment of the area about the minor axis, $z_g = z_a - z_s$, where z_a is the coordinate of the point of load application and z_s is the ordinate of the shear centre.

When the non-dimensional slenderness of beams by Eq. (5) is between 0.3 and 1.3, the coefficients of variation of the reduction factors may be defined as $\delta\chi_{LT,r} \approx 0.06-0.10$ and $\delta\chi_{LT,w} \approx \sqrt{\delta^2\chi_{LT,r} + \delta^2\chi_{\sigma_w}}$ where $\delta\chi_{\sigma_w} \approx 0.07-0.12$ is an additional component representing the effect of welding stresses σ_w [9].

The means and standard deviations of the variables χ_{LT} and f_y may be expressed as follows:

$$\chi_{LTm} = \chi_{LT} \text{ by Eq. (3)}, \quad \sigma\chi_{LT} = \delta\chi_{LT} \times \chi_{LTm} \tag{7}$$

$$f_{ym} = \frac{f_y}{1 - k_{0.05}\delta f_y}, \quad \sigma f_y = \delta f_y \times f_{ym} \tag{8}$$

The variables χ_{LT}, W_y and f_y are stochastically independent. Therefore, the means and coefficients of variation of buckling resisting may be written as

$$M_{b,R_r m} = \chi_{LT,rm} W_{ym} f_{ym} \tag{9}$$

$$\delta M_{b,R_r} = \sqrt{\delta^2\chi_{LT,r} + \delta^2 W_y + \delta^2 f_y} = 0.10-0.13 \tag{10}$$