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Assessing the reliability of local buckling of plates for mild and high strength steels

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ABSTRACT

In the current Eurocode 3-1-5 [1] for local buckling, the resistance curve used to represent the reduction factor of plated elements due to local failure is based on the so-called Winter-curve, derived on a semi-empirical approach by George Winter in 1947. This design curve represents the mean reduction values achieved in the experiments conducted by Winter and other researchers.

However, when applying the safety concept of EN 1990 [2], an additional safety factor γ_M is necessary to ascertain a defined level of failure probability. Currently, this factor is set to 1.0 for applications in building structures. In this paper 34 stub column tests on welded, squared box sections of steel grade S500 up to S960 are described. In combination with an experimental database on stub column tests summarised in Ref. [3], a new, optimised resistance curve is derived which could act as an alternative to the Winter curve. Additionally, both functions are evaluated in regard to the safety standard EN 1990 [2] with focus on the resulting γ_M.

As γ_M represents the safety factor for the actual material and geometric properties, which are not known by the designer, the more decisive safety factor is γ_M^*. This factor is used throughout Eurocode and refers to the nominal material and geometric properties. Its derivation and the influencing parameters are discussed and evaluated in the study at hand.

1. Introduction

1.1. Studies on local buckling behaviour of various steel grades

While there is no clear guidance on how to use the term “high strength steel”, it is common practice to start with grades of a nominal yield strength of 460 N/mm^2 and higher. S460 is still designable with Ref. [4], while steel grades between S500 and S700 are covered by additional rules given in Ref. [5].

The study at hand deals mainly with high strength steel material of S500 up to S960. The tests performed by the authors [6] and [7] suggest the existing local buckling resistance curve to be very optimistic, which is supported by further studies, e.g. Refs. [8–11]. However, looking at test results from steel grades below a yield strength of S500 and mild steels up to S420, the prediction by Eurocode proved to be optimistic as well [8,10,12–14]. In consequence, the problem of safe assessment of local buckling resistance is not confined to high strength steels. Therefore, for evaluation purposes various steel grades were included. The data included in this study is based completely on squared stub columns. Additionally, the research of Usami and Fukumoto [11] comprises squared and rectangular specimens. A complete compilation of experiments used for this study can be found in Ref. [3].

1.2. Historical derivation of buckling curve

While previous research of local buckling resistance was based on elastic utilisation ([15], first published in 1930) the need for higher utilisation rates in terms of plastic buckling, especially in aeronautical structures, led in 1932 to the development of the effective width method by von Kármán [16].

This method assumes that a plate buckling under compression loses its carrying capacity over a certain width, where the plate is out-of-plain deflected. Taking the whole load, the stress distribution over the areas adjacent to the corners is uniform and can be loaded

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up to the yield strength. The original differential equation can be written as:

$$\frac{Et^2}{12\,(1-\mu^2)}\left(\frac{\partial^4 w}{\partial x^4}+2\frac{\partial^4 w}{\partial x^2 \partial y^2}+\frac{\partial^4 w}{\partial y^4}\right)-\sigma\frac{\partial^2 w}{\partial x^2}=0 \tag{1}$$

for a plate under uniform compression in longitudinal direction x, not loaded in width direction (y) and deflection out-of-plane (w).

Further assuming a double sinusoidal shape of the deflected plate, we can derive the deflection amplitude:

$$2w=\frac{\pi}{\sqrt{3\,(1-\mu^2)}}\sqrt{\frac{E}{\sigma}}t \tag{2}$$

It is worth noticing that this approach is width independent. For steel material, where $\mu = 0.3$, the calculation of ultimate load could then be written as:

$$P_{\text{ult}}=1.90\sqrt{\frac{E}{\sigma}}t^2\cdot f_{\text{y}}. \tag{3}$$

Load capacity beyond the elastic limit was suggested to be taken into account, by replacing the Young's Modulus E with the slope of stress-strain curve E'. Based on this theoretical approach, Sechler and Donnell replaced the constant 1.90 by the factor C. To assess a lower bound for C, instead of a sinusoidal deflection Figure, they assumed w to be of a straight shape, see Fig. 1, resulting in $C = 1.24$. They expected the experimental results consequently to be between 1.90 and 1.24. Different metals and thickness values were investigated; however, no definite C-value was given in the end. Although a decrease in C with increasing slenderness was observed, it was attributed to the increasing influence of flexibility of the testing rig. von Karman et al. [16] nonetheless showed the study that metal structures can be treated in the same manner to assess the buckling resistance.

In 1947, Winter published his work on "Strength of Thin Steel Plates Compression Flanges" [17] which is still the fundamental basis for the resistance curve in many current design codes, e.g. the Eurocode [1]. He conducted two test series on thin-walled steel members, first on inverted U-sections, and second on two bolted U-sections, forming an I-section. He changed the C-factor to be dependent on the t/b-ratio and used a best fit function to match the average of results of his own tests and the tests conducted by Sechler. For C, he obtained:

$$C=1.90-1.09\sqrt{\frac{E}{f_{\text{y}}}}\left(\frac{t}{b}\right). \tag{4}$$

The effective width can then be written as:

$$b_{eff}=1.90t\sqrt{\frac{E}{f_{\text{y}}}}\left[1-0.574\left(\frac{t}{b}\right)\sqrt{\frac{E}{f_{\text{y}}}}\right] \tag{5}$$

The formula can be rewritten as:

$$\frac{b_{eff}}{b}=\frac{1}{\bar{\lambda}}\left(1-0.30\frac{1}{\bar{\lambda}}\right). \tag{6}$$

The factor 0.3 was revised several times, until, based on experimental results, it was finally changed to 0.22 [18]. This value was also used for the resistance calculation in Ref. [1]. The resulting reduction factor using the Winter formula yields to:

$$\rho=\frac{1}{\bar{\lambda}}\left(1-0.22\frac{1}{\bar{\lambda}}\right). \tag{7}$$

This reduction factor is applied on the plate width, hence the design check is handled as a cross-section check, using γ_{M0}. The tests on these cold-formed sections imply also certain hardening effects in the corners of the profiles, which are difficult to take into account and not mirrored in the chosen approach [6], leading to optimistic results. Sophisticated models were developed recently, see e.g. Refs. [19] and [20] to give guidance for design of thin-walled, cold-formed profiles. However, for a general approach to calculate the buckling resistance of plates, welded box sections are far more appropriate to evaluate the theoretical model, as the specimen is approximately in conformity with the model. In terms of safety, it can be concluded that the approach according to Winter is not in conformity with the Eurocode Standard. Either a γ_M evaluation has to be made to assess the necessary safety-level or the resistance curve should be derived as a lower envelope of the experimental results. In this paper, both approaches are explored and summarised.

2. Experiments on stub columns

In this section, the experiments on welded sections conducted by the first author are described. Special attention was paid on the measurement and evaluation of intended and unintended eccentricities. While the data collected from previous research claim a concentric loading of the specimens, it could be observed during the tests that small eccentricities are inevitable. For the assessment of the load prediction of Ref. [1], the eccentricity was taken into account leading to a reduction of scatter in the results. The procedure is explained in Section 3.

2.1. Design and fabrication

The welded sections were provided with matching welding strengths, i.e. the yield strength of the seams were similar to the yield strength of the specimens. The specimens were designed such that they covered a significant range of slenderness. The length was taken to 3*width + 50 mm (in accordance to Ziemian [21]) to avoid global buckling behaviour and allow for a representative residual stress distribution in the specimens. After sawing, the specimens were milled flat at the ends providing a best possible even surface. Welded end plates were avoided to introduce no further residual stresses. All the plate material was fabricated according to EN 10149-2:2013 [22].

2.2. Test matrix and actual dimensions

34 stub column tests were carried out. They covered the steel grades S500 up to S960, and a respective plate slenderness from 0.64 to 1.55. The denomination of specimens contains the steel grade (e.g. 960), the dimensions (e.g. 170-6 means a width of 170 mm and a thickness of 6 mm) and the sequential number (−4 means the 4th test of this specimen configuration). For each configuration (steel grade and slenderness), 4 to 5 tests were conducted, with different eccentricities of load introduction. A complete overview is given in

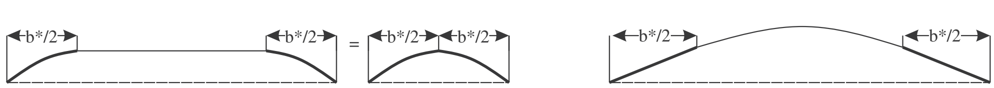

Fig. 1. Cross-sections of buckled plates [16] , according to von Kármán (left) and Sechler/Donnell (right).