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Interaction of local and global buckling of box sections made of high strength steel

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ABSTRACT

While there are precise analytical model available to assess the calculation of critical and ultimate load for global and local buckling separately, the interaction of both modes prove to be difficult as membrane effects and imperfections are of major impact. In an experimental programme, thirteen tests on columns with high b/t-ratio were carried out on square welded box sections made of S500 and S960 steel material, varying the global slenderness. The experiments were re-calculated with the Finite-Element-programme Ansys. The calibrated numerical model was subsequently used for parametric studies. The study at hand provides additionally an analytic approach to determine a slenderness depending reduction factor to design box sections prone to coupled instability. This approach, subsequently denoted as "generalised slenderness approach (gs)" is still orientated on the Ayrton-Perry format, which is also the basis of the Eurocode design procedure. Local effects are not included by omitting parts of the cross-section in the gs-approach, but by adding an additional equivalent global imperfection. The amplitude of this imperfection is based on the effective width method, but design charts for box sections are developed to ease the application.

1. Introduction

1.1. Scope and background

The motivation of the study at hand is to clarify stability and material related issues exemplified on square welded box sections and provide designers with a procedure to assess coupled instability with an in terms of safety and calculation effort optimised routine. Box sections are commonly used as columns in industrial buildings, but more frequently in supporting structures in bridge design.

1.2. Codes and requirements for high strength steel

While steels up to S460 are included in the Eurocode 3 family part 1–11, higher grades between S500 and S700 are regulated in an additional part 12. However, it is intended by CENTC250-SC3 to omit part 12 and transfer the contend to the respective parts of part 1–11. The main obstacle is the assumed lower ductility and lower elongation at fracture, ε_u, of higher strength material which might not comply with the assumptions made in EC3 regarding e.g. re-distribution of stress and strain. Especially in plastic design, the usage of HSS might lead to unsafe structures. In consequence, in [1] three requirements were defined to guarantee sufficient ductility, and tightened in [2] as summarised in Table 1.

The table shows also the contradicting development of requirements: while the first and last requirement for the ratio of ultimate strength to yield strength f_u/f_y and the elongation at fracture are loosened for high strength steel to pay tribute to the actual material properties, the uniform elongation criterion is tightened. In terms of ductility, issues increase with increasing yield strength. However, for global buckling this might be of no influence, as the failure mode can be assessed purely elastic. When local effects have to be considered, the reduced strain hardening might influence the post-buckling behaviour, whenever only with small impact.

Aside from the ductility requirements of [1,2], for the fabrication process the standard EN 10149-2 [3] (technical delivery conditions for thermomechanically rolled steels), which was used for the steels investigated within this study, is valid. Compared with EC3, the minimum f_u/f_y-ratio is partly lower, with 1.02 for S960M and in some cases higher, e.g. 1.07 for S700M and 1.10 for S500M. Quenched and tempered steels, regulated in EN 10025-6 [4] have higher limits with 1.18 for S500Q, 1.12 for S690Q but an also lower value for S960Q with 1.02.

1.3. Concepts to solve coupled instability

Several approaches have developed over the years, using different

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Table 1
Ductility requirements in EC3 for mild and high strength steel.

	S235–S460 (EN 1993-1-1)	S500–S700 (EN 1993-1-12)
f_u/f_y	≥ 1.10	≥ 1.05
ε_u	$\geq 15 f_y/E$	$\geq 15 f_y/E$
A	$\geq 15\%$	$\geq 10\%$

aspects to take coupled instability of local and global behaviour into account. The multiplication method uses the product of the reduction factors for local and global buckling, which are calculated separately and multiplied afterwards. The method is considered to be rather conservative. The mechanical mechanism is very simplified and thus not satisfactorily represented. However, it was adopted in the former German design code DIN 18800-3. The model by Rubin [5] was developed for double- and single-symmetric I-sections with bending about the strong axis. Two approaches, one exact and one simplified, were introduced in the late DIN 18800-2, assuming that global buckling is dominant towards local buckling. Both approaches use pre-imperfections w_0 (whereas the magnitude is depending on the buckling curve) to amplify the stress resultants and the effective width method to reduce the cross-section. The design check equals a theory of second-order design check with reduced cross-section. For the Q-factor method, the reduction factor Q is determined as the ratio of the effective cross-section to the gross cross-section. The global slenderness is then assessed by multiplying the square root of Q with the global slenderness of the gross cross-section, and also applied on the plastic resistance of the member. The principle of this method is mirrored in the European design code [6]. The direct strength method was developed by Schafer and Peköz in 1998 [7] and adopted in the American Standard AISI S100-07 for the design of cold-formed structural steel members. In the direct strength method, the nominal axial P_n and flexural M_n strength resistance is determined for cold-formed columns and beams. Strict limitations in respect of geometric properties and relations are given, restricting the application to open sections. Firstly, the elastic buckling modes are calculated separately for each possible failure mode, e.g. local buckling, global buckling or distortional buckling. For the column design, the nominal axial strength is determined by the minimum of the separated calculated ultimate loads for global buckling P_{ne}, local buckling P_{nl} or distortional buckling P_{nd}. However, in [7], the authors assert that when looking at experimental data, it becomes apparent that when two buckling modes compete the final failure mode may not be consistent with the elastic minimum.

1.4. Eurocode design procedure

To design against coupled instability, Eurocode3-1-1 [1] allows to calculate the resistance with a reduction of plastic or elastic resistance on cross-section level. In dependence of boundary conditions and moment distribution along the column, occurring bending moments are increased by factors k. In the cases investigated within this paper, the moment distribution along one axis of the column was always constant, while the perpendicular axis has a moment of zero magnitude. As the cross-sections were all squared, the design equation simplifies to:

$$\frac{N_{Ed}}{\chi_c N_{Rk}} + k_{yy}\frac{M_{y,Ed}}{M_{Rk}} \leq 1 \tag{1}$$

with:

$$k_{yy} = C_{my}\left(1 + 0.6\cdot\bar{\lambda}_c\frac{N_{Ed}}{\chi_c N_{Rk}/\gamma_{M1}}\right) \leq C_{my}\left(1 + 0.6\frac{N_{Ed}}{\chi_c N_{Rk}/\gamma_{M1}}\right) \tag{2}$$

and $C_{my} = 1$ for a constant moment distribution along the column. The moment resistance M_{Rk} for cross-section Class 4 is given as the effective elastic resistance:

$$M_{Rk} = f_y \cdot W_{eff} \tag{3}$$

According to [1], a reduction of the cross-section has to be taken into account to capture the loss of stiffness due to plate buckling. With the local reduction factors an effective cross-section is calculated, which is used for the definition of the resistance and thus influences also the definition of the slenderness, see Eq. (4).

$$\bar{\lambda}_c = \sqrt{\frac{A_{eff} f_y}{N_{crit}}} \tag{4}$$

To calculate an effective cross-section due to local buckling, [8] offers a basic/ simplified procedure and a second possibility with a reduced slenderness $\bar{\lambda}_{p,red}$, under consideration of the actual loading conditions: clause 4.4(4) of [8]. For the basic procedure, the by ρ reduced plate-width for all four plates is assessed separately using the known local buckling reduction curve:

$$\rho = \frac{\bar{\lambda}_p - 0.055(3 + \psi)}{\bar{\lambda}_p^2} \tag{5}$$

ψ is in this context the stress ratio along each plate. The slenderness is characterised by:

$$\bar{\lambda}_p = \sqrt{\frac{f_y}{\sigma_{cr}}} = \frac{\bar{b}/t}{28.4\cdot\sqrt{\frac{235}{f_y}}\cdot\sqrt{k_\sigma}} \tag{6}$$

Using this simple approach, the plates of the cross-section are treated individually and any interaction of them is neglected.

In clause 4.4(4) of [8] a reduced slenderness under consideration of the actual stress distribution $\sigma_{com,Ed}$ is taken into account:

$$\bar{\lambda}_{p,red} = \bar{\lambda}_p\sqrt{\frac{\sigma_{com,Ed}}{f_y/\gamma_{M0}}} \tag{7}$$

Background information on EC3-1–5 can be found e.g in [9–11].

The global column reduction factor χ_c on the resistance is then assessed by:

$$\chi_c = \frac{1}{\phi + \sqrt{\chi^2 - \bar{\lambda}_c^2}} \tag{8}$$

ϕ is thereby calculated under consideration of the imperfection factor α, which equals in the study at hand 0.34, using buckling curve b.

$$\phi = 0.5[1 + \alpha(\bar{\lambda}_c - 0.2) + \bar{\lambda}_c^2] \tag{9}$$

The buckling coefficient α denotes the sensitivity of the cross-section towards imperfections. Structural and geometrical imperfections are of vital importance in stability design. Especially the structural imperfections are not easily to handle for analytical or hand calculations. Sophisticated numerical programmes would be needed, which contradicts the demand for efficient design. Therefore, the reduction curves for global as well as local buckling include structural as well as geometric imperfections from fabrication and assembling processes. EC3-1-1 distinguishes thereby 5 buckling curves for global buckling, where cross-sections are classified according to their sensitivity towards these imperfections. E.g for weak-axis bending of an I-section, the corresponding buckling curve is always worse than the curve for strong axis bending. Higher strength steel is generally better classified due to lower residual stress. The code provides Table 5.1 for the to the imperfection factor and analysis method corresponding initial curvature, from which the initial imperfection e_0 can be read. These imperfection factors are depending on the buckling curve and analysis method, elastic or plastic. In this paper, the definition for e_0 was used under the assumption of buckling curve b for welded box sections, see Tables 6.1 and 6.2 of [1], resulting in $e_0 = 1/250\cdot$Length and $\alpha = 0.34$.

Background information on EC3-1-1 is provided by the Technical Committee 8 – Stability of ECCS in [12].

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