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Buckling resistance of HSS box section columns Part II: Analytical study

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ABSTRACT

The accurate and economical consideration of the flexural buckling resistance of high strength steel (HSS) structures is highly important in the design. According to the previous research results the flexural buckling behaviour of HSS and NSS columns can be significantly different, which differences are not considered in the current design rules. The aim of the current study is to obtain a reliable design method for the flexural buckling resistance of HSS welded box section columns. The buckling resistance is previously determined by the authors based on an experimental research program and based on stochastic numerical simulations . The current paper focuses on the investigation of the theoretical background of the flexural buckling phenomena and on the implementation of the specialties regarding HSS materials into the buckling curve formulation. The analytically derived Ayrton-Perry type formulation is studied and modified based on the theoretical and numerical investigations implementing the effect of residual stresses and geometrical imperfections into the design method. Using the revised buckling curve, the flexural buckling resistance of welded box columns can be predicted with larger accuracy for steel grades between S420 – S960.

1. Introduction

The buckling behaviour is highly important in case of HSS structures because slender sections might be used due to their higher yield strength. The current EN 1993-1-1 [2] is applicable to steel materials up to S460. The EN 1993-1-12 [3] gives design proposals for high strength steel materials up to steel grades of S700. However, in case of the flexural buckling resistance of welded box section columns the same rules are given for NSS and HSS materials. Numerous previous research results [4–12] prove that the flexural buckling resistance of HSS structures is more favourable than for NSS structures. Therefore, it can be expected that higher column buckling curves are applicable for HSS than for NSS members. Moreover the effect of the residual stress is different in different slenderness regions, that is already shown by Szalai & Papp [13], Taras and Grainer [14] and Somodi & Kövesdi [9,15]. Due to this phenomenon the typical shape of the buckling curve should be different for HSS and for NSS members as well.

The current design rules for flexural buckling resistance in the EN 1993-1-1 [2] are based on the Ayrton-Perry type formulation taking the effect of the residual stresses and the geometric imperfections as generalized imperfections into account. The analytical method cannot be applied for HSS structures in the same form, thus the residual stresses are not implemented directly. To solve this theoretical problem there are two possible solutions. The first solution could be the modification of the imperfection parameter of the current Eurocode based buckling curves depending on the steel grade. The second possibility is the implementation of the residual stresses and the effect of the yield strength into the principle of the Ayrton-Perry type formulation, which could result in a more precise column buckling curve description. The authors studied both solution strategies and the results regarding the modified imperfection amplitude is presented in [1]. The current paper presents the results regarding the theoretically based column buckling curve improvement. This improved formulation could be theoretically applicable for hot-rolled, cold-formed and welded sections, as well as for NSS and HSS structures after validation.

The general aim of the current research program is to create a revised buckling curve that can be used to determine the flexural buckling resistance of welded box columns for steel grades between S420 – S960. In order to reach this goal a stochastic numerical analysis is executed for welded HSS box columns, published in [1]. The purpose of that analysis was to obtain buckling curves based on the characteristic buckling resistances using the real distributions of the input parameters. The distributions of the applied variables are determined based on own column buckling tests, residual stress and geometric imperfection measurements. The lower 2.3% quantile values of the buckling resistances are obtained using Monte Carlo simulations.

The current paper focuses on the implementation of the residual stresses of welded box sections into the Ayrton-Perry type formulation creating an improved column buckling curve. The influential parameters of the improved buckling curves are studied and evaluated. The

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effect of the residual stress magnitude on the buckling resistance is studied and compared to the numerical results. Based on the buckling curves obtained by the stochastic analysis an improved analytical column buckling curve is calibrated. The improved buckling curve is compatible with the general buckling curves of the EN 1993-1-1 [2], and applicable for welded box sections made from steel grades between S420 - S960. The objectives of the research program are achieved by the following research strategy:

1. literature review on the Ayrton-Perry type formulation and on the origin of the current flexural buckling curves of the EN 1993-1-1 [2],
2. literature review on the buckling behaviour of HSS welded box section columns,
3. experimental research program to determine the column buckling resistance,
4. stochastic numerical parametric study based on a detailed experimental program using Monte Carlo simulations,
5. determination of the characteristic buckling curves,
6. modification of the Ayrton-Perry type formulation implementing the effect of the residual stresses and geometrical imperfection,
7. calibration of the revised Ayrton-Perry type formula based buckling curve,
8. comparison of the results with the current buckling curves of the EN 1993-1-1 [2].

2. Literature overview

2.1. Origin of the European buckling curves

This section summarizes the steps of the development of the buckling curves currently found in the EN1993-1-1 [2] for NSS structures. The way of the buckling curve development has a large importance and needed to know if the structural behaviour of HSS member is compared to the structural behaviour of NSS structures. The buckling curves of the EN 1993-1-1 were achieved by the following steps according to [14,16].

Step 1: An extensive experimental research program was carried out in order to obtain a large database representing experimental results for the flexural buckling resistances [17,18]. These tests were supplemented by numerical FEM simulations [19]. Based on the statistical evaluation of the large database the lower characteristic values of the flexural buckling resistance are determined for different slenderness ranges.

Step 2: Numerical study was carried out by Beer and Schulz using advanced GMNIA simulations to extend the experimental database and to generate flexural buckling curves for a large number of different sections [20]. The assumptions used in the numerical simulations are presented in Fig. 1. The computed buckling curves were in good agreement with the 2.3% quantile values of the test results [17].

Step 3: The resulted column buckling curves from the mentioned numerical simulations are the bases of the European column buckling curves [16]. The theoretical representation of Rondal and Maquoi [21] [22] has been adopted in the analytical formulation of the European flexural buckling curves. The bases of this theory is the Ayrton-Perry type formulation [23] [24], which was calibrated to the best fit of the numerical buckling curves. The calibration procedure is illustrated in Fig. 2.

The *"generalized imperfection"* factor (η) depends on the cross-sectional properties A (area) and W (cross section modulus), as well as on the initial out-of-straightness e_0 of the analysed member. For the validation of the Ayrton-Perry type formulation in the plastic range, the elastic expression is replaced by a calibration factor α.

Step 4: The necessary level of safety was reached by determining the appropriate values of the partial safety factors γ_{M1} following the procedures of the EN1990 Annex D [25].

2.2. Buckling behaviour of HSS welded box section columns

The accompanying paper [1] gives a detailed overview in the topic of the buckling behaviour of HSS welded box section columns. The literature survey shows that previous experimental and numerical investigations prove that the box section columns made from HSS material provides significantly larger resistances than the columns having the same geometry but made from NSS. This observation indicates the improvement request of the current buckling curves make them applicable for HSS structures. There are two different ways for the evaluation of the column buckling curves. The first option is based on an experimental way performing real tests, or stochastic numerical simulations using Monte Carlo simulation and executing a statistical evaluation on the large database. The second option is to carry out numerical simulations applying residual stresses and initial geometric imperfection with amplitude of L/1000. It is proved in the international literature for NSS structures, that these assumptions taken in the numerical simulation provides a good prediction of a characteristic value of the buckling reduction factor [14].

All the previous investigations used deterministic numerical analyses to explain the differences of HSS and NSS structures and to obtain design proposals for HSS structures. In the accompanying paper [1] the authors designed and executed a stochastic numerical research program using Monte Carlo simulation based on a comprehensive experimental research program conducted by Somodi and Kövesdi [9]. The

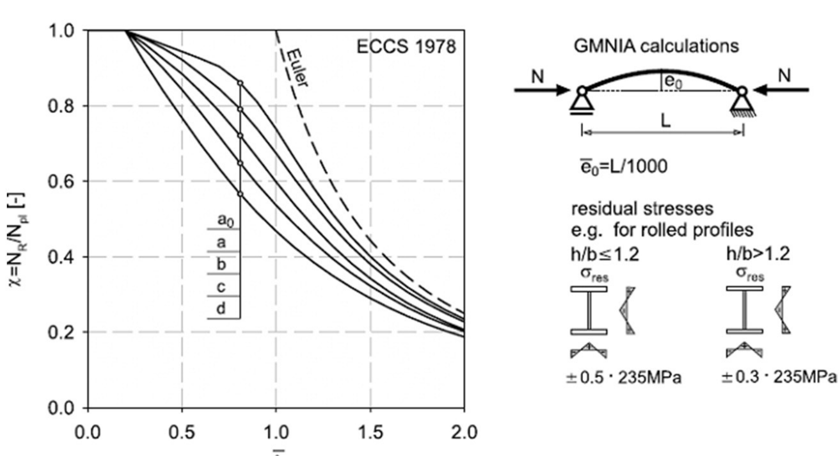

Fig. 1. Original tabulated ECCS column buckling curves [14].