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European column buckling curves and finite element modelling including high strength steels

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ABSTRACT

Eurocode allows for finite element modelling of plated steel structures, however the information in the code on how to perform the analysis or what assumptions to make is quite sparse. The present paper investigates the deterministic modelling of flexural column buckling using plane shell elements in advanced non-linear finite element analysis (GMNIA) with the goal of being able to reestablish the European buckling curves. A short comprehensive historical review is given on the development of the European buckling curves and the related assumptions made with respect to deterministic modelling of column buckling. The European buckling curves allowing deterministic analytical engineering analysis of members are based on large experimental and parametric measurement programs as well as analytical, numerical and probabilistic investigations. It is of enormous practical value that modern numerical deterministic analysis can be performed based on given magnitudes of characteristic yield stress, material stress–strain relationship, and given characteristic values for imperfections and residual stresses. The magnitude of imperfections and residual stresses are discussed as well as how the use of equivalent imperfections may be very conservative if considered by finite element analysis as described in the current Eurocode code. A suggestion is given for a slightly modified imperfection formula within the Ayrton-Perry formulation leading to adequate inclusion of modern high grade steels within the original four bucking curves. It is also suggested that finite element or frame analysis may be performed with equivalent column bow imperfections extracted directly from the Ayrton-Perry formulation.

1. Introduction

The European column buckling curves are based on extensive experimental research programs as well as theoretical, numerical and probabilistic investigations performed around the 1960s and early 1970s. Standardized buckling tests were performed at different laboratories and the gathered parametric information and results were analysed using both probabilistic and deterministic methods. The original experimental and theoretical (deterministic) basis for European buckling curves is respectively given by Sfinitesco [1] and Beer & Schulz [2]. An early proposal of a series of buckling curves based on a probabilistic approach was given by Bjorhovde [3]. Strating & Vos [4] demonstrated that buckling curves corresponding to a constant probability of failure can be determined from the distribution functions of the physical and geometric column parameters. This work was based on statistical information from the European test series on IPE160 sections, a theoretical nonlinear member theory and Monte Carlo simulation. They found a reasonable agreement with the experimental buckling curve (with the same confidence value). Strating & Vos [4] found that for a column with the length L the mean value m of the bow imperfection corresponded well to $m = 0.00085L = L/1176$ and that the mean plus 2 times the st. dev. s, corresponded to $m + 2s = 0.00125L = L/800$. Bjorhovde [3] used a randomly distributed bow imperfection between $L/1000$ and $L/10000$ corresponding to the limits of the 95% confidence interval for the distribution of the bow imperfection with a mean value of $L/1470$.

In 1975 Dwight [5] reports on the work towards incorporating the “Ayrton-Perry” approach including equivalent imperfections and a plateau corresponding to relative slenderness values lower than $\lambda_0 = 0.2$. The main results of the work of the European Convention for Constructional Steelwork (ECCS) was gathered in 1976 in the ECCS “Manual on Stability of Steel Structures” [6], which is a very thorough gathering of the academic state of the art of the European stability research at that time including references to many known related works. The resulting ECCS recommendations [7] came two years later. However, the five ECCS column buckling curves (a_o,a,b,c,d) were just tabulated and the related analytical formulations were not given in either of these references. This was due to the fact that the “Ayrton-Perry” type approach [8] in the form proposed by Robertson [9] (the so-called “Perry-Robertson” formula) was not fully developed with respect to equivalent imperfections for these curves. Investigations performed in 1978 by Maquoi & Rondal [10] with 7 proposals for the formulation of the equivalent imperfections made it possible to decide on an “Ayrton-Perry”

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approach. In the 1984 report on Eurocode 3 by Dowling et al. [11] the "Ayrton-Perry" approach was included for simple member verification as well as the possibility of numerical verification using 1/1000 of the buckling length, *L*, as initial bow imperfection with simplified linear residual stress distributions. Furthermore, an equivalent geometric imperfection dependent on the buckling curve of the cross-section could be used. The simplified residual stress distributions including tubes were also given in the ECCS report on sway frames [12]. In the 1992 draft for development of Eurocode 3 [13] the Ayrton-Perry approach was included with a somewhat cumbersome awkward mix of definitions of the equivalent imperfections dependent on safety factors and on whether strong or weak axis was being analysed. In the case of strong axis buckling the equivalent imperfection was more or less extracted directly from the "Ayrton-Perry" based buckling curve. In case of weak axis buckling an equivalent imperfection was given as a fraction of the buckling length, but including a correction factor removing the (correct) influence of the yield stress. In this preliminary Eurocode 3 the previous proposal of allowing the use of assessment with residual stress and related bow imperfection was not included.

From 1992 until 2005 when the (EC3) Eurocode 3 part 1–1 [14] finally became a harmonized European standard the buckling curve formulation remained nearly unchanged. In this period the major research was related to the stability treatment of beam-columns with combined compression and bending as thoroughly described in the ECCS publication [15] giving the background documentation and design guidelines. Thus the formulation of the buckling curves in the harmonized European standard Eurocode 3 part 1–1 is based on the "Ayrton-Perry" formulation of the five buckling curves and alternatively numerical treatments can be performed using a set of very conservative equivalent imperfections. However, Eurocode 3 part 1–5 [16] includes an Annex C on FE-methods which in a relatively vague formulation allows a more refined analysis of the geometric imperfections and residual stress that respectively may correspond to 80% of the geometric "plate" fabrication tolerance and a residual stress pattern using mean amplitude values. Since the reference is to "plate" tolerances, it does indeed seem to be the intention that the over conservative equivalent bow imperfections from part 1–1 are to be used. This does not correspond to a more refined analysis as shown later in this paper, it is simply too conservative. The commentary to part 1–5 prepared by Johansson et al. [17] gives recommendations on imperfections and residual stress, but does not shed light on the magnitude of the column bow imperfection. The ECCS document from 2006 [15] on the beam-column instability formulas in Eurocode 3 part 1–1 gives background documentation and references to papers in which numerical analysis has been used for verifying and evaluating the constants of the beam-column formulas. In these papers, found on the companion CD, the residual stress distributions have mainly been the simplified distributions described in [11] and [12], but different parabolic distributions (with residual stresses in the same order of magnitude) have also been used and results have been compared. The conclusion is that use of the simplified linearized residual stress distributions lead to similar results, which seem to be a bit more conservative compared to the parabolic distribution. A mean value for the residual stress magnitude should be used and this is often for I-sections set to 30% or 50% of the yield stress (for S235 steel) dependent on the height to width ratio. However, it is also stated, discussed and shown that the magnitude of the residual stress is independent of the steel grade, see the work of Alpsten [18] and the ECCS manual on stability [6] especially the section concerning compression members of high strength steel. Residual stress magnitudes and distributions have been measured in numerous applications and in a multitude of specimens, mainly during the 1970 (many included in the works of Alpsten). However, with the increasing strengths of high strength steel and their introduction on the market there is a need for residual stress measurements concerning these "new" grades of steel. Recent experimental studies [19] confirm that for welded members this is also the case for high-strength steels. Naturally, if the magnitude of residual stresses is wrongly assumed to be proportional to the yield stress, the column buckling capacity of steel members found by finite element simulation will be underestimated for higher grades of steel. Moreover, since most numerical simulations regarding both flexural and lateral torsional buckling which include residual stresses are performed on grade S235 steel, the fact or statement that the maximum residual compressive stress is defined as a percentage of the yield stress might be misleading and may represent one of the reasons why conflicting assumptions can be found throughout the literature regarding this issue. This was pointed out in a recent paper by Boissonnade, [20], where this topic was studied for the case of lateral torsional buckling. Placing different steel grades on the same buckling curve leads to an overly conservative design for higher strength steels was pointed out by Dwight, [5], even before the establishment of the first tabulated buckling curves. While it is true that Eurocode 3 prescribes higher buckling curves for S460, all other steel grades fall into the same buckling curve.

Variations in the cross-section geometry are not taken into account, since it is assumed that the influence is minimal and thus nominal or mean values are used for the geometric parameters. Therefore deterministic column analysis is to be performed using a characteristic value for the yield stress, corresponding to a 95% confidence level and all other parameters are taken as mean values. However, when introducing phenomena such as local imperfections and local buckling into the analysis, which is necessary for class 4 (slender) cross-sections, this may have to be reconsidered or calibrated by prescribing adequate levels of combined imperfections for example through the use of a square root of squares combination rule.

When it comes to finite element modelling of columns using shell elements, there are mainly two methods of analysis following the statements in the code and the development history. The first method (I) is to introduce mean bow-imperfections and adequate simplified mean residual stress distributions. The other method (II) is to introduce equivalent bow-imperfections that include all relevant effects. Of course within each of these methods there are a number of important choices, which have a great influence on the results.

In this paper results and comparisons are expected to be generic in character. The finite element analysis and analytical computations have therefore been limited to standard hot rolled profiles IPE160, and HEB300, which are not prone to local buckling. Thus class 4 sections are not discussed in this work. Furthermore, the computations have been limited to strong axis buckling. The investigations performed have also included IPE500 and HEB500, but it has been deemed to be unnecessary to include all these very similar results. However, in Section 6 results are included for the IPE500 in order to show that the benefits of the proposal made in this paper also apply for the IPE500 profile with a higher web slenderness.

In the following section the theoretical background for the Ayrton-Perry formulation of Eurocode 3 is briefly described and the theoretical imperfection formulation is clarified, so that the theoretical influence of the steel grade becomes clear.

Then in the next section the finite element modelling and boundary conditions are introduced, including subsections with discussions and brief descriptions on how the modelling of the steel material, the imperfections and the residual stress is performed.

With the finite element modelling in place the following section turn to the results and comparisons using method (I) with geometric bow imperfection and residual stress. The influence of residual stress distribution (linear or parabolic), the residual stress magnitude, the material stress–strain curve and the steel grade (yield stress) is investigated and discussed.

Then results and comparisons when using method (II) with equivalent imperfections are discussed and analysed. Two different magnitudes of the equivalent imperfections are investigated: 1) The equivalent imperfections taken as a fraction of the column length as stated in Eurocode 3 and the logical alternative 2) the equivalent