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Stability of curved panels under uniform axial compression

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ABSTRACT

The use of curved panels for the construction of steel bridges becomes more and more popular. Their design is however made difficult by a lack of specifications, especially in European Standards. The present study aims thus at developing a method for predicting the ultimate strength of cylindrical unstiffened curved panels subjected to uniform axial compression. The methodology used in this study is based on the formal procedure recommended by Eurocode 3 for all types of stability verifications. A series of numerical simulations is first carried out to identify the fundamental characteristics of curved panels' elasto-plastic behaviour. Then on the basis of these numerical results, semi-empirical formulae for predicting the elastic buckling and ultimate strength are derived and illustrated on a practical example.

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1. Introduction

1.1. Engineering context

Typical examples of curved panels in the civil engineering domain are orthotropic decks in box-girder bridges and webs of in-plane curved I-girder. When designing such panels, the engineer will search for the best compromise between economy and safety, he will try to reduce as much as possible the thickness of the plate to lighten the structure and reduce its cost. Nevertheless, the stability verification of cylindrical curved members is not exactly covered by European Standards: EN 1993-1-5 [1] gives specifications for flat or slightly curved panels (with the condition $b^2/Rt<1$) and EN 1993-1-6 [2] deals with revolution cylindrical shells.

It will be seen in the following example that curved panels in bridges have characteristics exactly between these two cases. Fig. 1 shows such a bridge built over the large arm of the river Seine between Boulogne-Billancourt and the Seguin Island (France). The bridge is a steel box girder with a constant web spacing of 6.40 m. The radius of the bottom flange varies from 80 to 120 m. The flange thickness also varies from 14 mm at mid-span to 60 mm at the bridge's supports. These geometrical characteristics give a ratio b^2/Rt of 5.7 to 36.8, clearly out-of EN 1993-1-5 scope. EN 1993-1-6 is not applicable neither because these curved flanges are not revolution cylinders. In such a case, the use of finite element modelling (F.E.M.) becomes necessary. However it requires sophisticated models, time-consuming calculations and then careful analysis of results. Consequently, daily practitioners need a reliable and relatively simple hand calculation method which proposal is the aim of this work.

1.2. Scientific context

Papers related to the buckling theory of curved panels are not so numerous in comparison to the literature devoted to plate buckling or cylindrical shell buckling. Chronologically, in the first models, it was assumed that, for curved panels like for full revolution cylinders, the ultimate load is equal to the critical buckling load. For instance, the reference expressions developed by Redshaw [3] and Timoshenko [4] rely on this assumption. Stowell [5] used it in its proposition of a modified form of Redsaw's expression taking into account the influence of the boundary conditions. Then other expressions similar to Timoshenko's were achieved by using Donnell's equation [6] or the Schapitz criterion [7]. However, the experimental data obtained by Cox and Clenshaw [8], Crate and Levin [9], Jackson and Hall [10], Welter [11], and Schuette [12] exhibit significant differences with these theoretical values. These differences remain unexplained until the works of Batdorf et al. [13], who attempted to make a synthesis of the test data from Crate and Levin [9] and Cox and Clenshaw [8]. Their modified solutions are presented as abacus, also known as Batdorf's curves, which have been widely applied in the aeronautical fields till now [14].

In a second period, researchers wondered if the critical buckling load really represents the failure load for all curved panels, whatever their radius of curvature, because it could be expected that, at least for panels with small curvature (close to single plate), the ultimate load will increase after buckling. This was indeed verified by the

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Fig. 1. Renault Bridge — Seguin Island, France.

experimental work of Wenzek [15]. Some expressions including this post-buckling resistance were thus proposed by Pope [16], Sekine and Tamate [17] and Gerard [18]. These expressions are still approximations: from a mathematical point of view, the solution of the underlying differential problem is highly complicated by the shell curvature.

During last decades, the development of computers and finite elements codes make it possible to evaluate the stability of complex structures. Recognising a need for updating the N.A.S.A. structural stability monographs, Domb and Leigh [19] proposed an update of the currently used design curves. Other studies in the field of shipbuilding were also conducted [20]. In this last case, the plates are so thick that elastic buckling rarely occurs and that plastic buckling dominates which is exactly the opposite to the aeronautical field where only elastic buckling occurs. One should however notice that, for the ship industry, information on the elastic–plastic behaviour may be useful if a reduced thickness due to corrosion is considered. Further investigations were then conducted by Yumura et al. [21] who studied numerically the buckling/plastic collapse and by Park et al. [22–24] who provided a simplified method to estimate the ultimate strength based on Faulkner's formulae for a single plate with a newly defined slenderness parameter including curvature effects.

1.3. Objective and methodology

Despite abundant research, there is today no satisfying analytical expression for the estimation of the ultimate load of cylindrical curved panels. The present study aims thus at the development of such an expression. For simplicity reason, it will be limited to cylindrical, unstiffened square panels subjected to uniform longitudinal compression. Except the stiffening condition, this configuration is the most common one for bottom flanges in box girder bridges.

The general framework of this study is that of Eurocode 3 for all kinds of stability verifications [25,26]. Eurocode 3 proposes to use a procedure called $\chi-\bar{\lambda}$ method (Fig. 2), which consists of three steps:

- Calculate the linear elastic critical load (F_{cr}) for the ideal (imperfection free) panel and its elastic or plastic resistance (F_y) obtained when instability phenomena are absent.
- Calculate the relative slenderness $\bar{\lambda}$ of the panel, defined as $\bar{\lambda}=\sqrt{F_y/F_{cr}}$.
- Evaluate the reduction factor χ which depends on the relative slenderness $\bar{\lambda}$ and on the imperfections in the panel (initial geometric imperfections and residual stresses).
  χ is a semi-empirical parameter. Its mathematical expression comes from an analytical approach using standard strength of materials theory. It includes some numerical parameters which have to be calibrated with the available numerical and/or experimental data. These parameters cover the effects of the geometrical initial imperfections and material imperfections such as residual stresses in the panels.
- Calculate the ultimate resistance (F_u) by $F_u=\chi F_y$. Normally, F_u should be divided by γ_M. In the present paper, γ_M is assumed to be equal to 1.0, in order to be able to compare codes value of the resistance with values obtained by numerical simulations.

The article's structure will follow the three steps of the procedure mentioned above. The critical buckling will be investigated first and then the ultimate load. Afterwards a semi-empirical expression will be deduced and calibrated with previous numerical results. Finally a practical example will be shown to illustrate the design procedure and to verify the accuracy of the proposed formulae.

2. Finite element modelling

In the following, the studied cylindrical panels have square dimensions and uniform curvature (see Fig. 3). They are simply supported on all edges ($u_R=0$, no radial displacement): along the curved edges AC and BD (the loaded ones) longitudinal displacements are allowed but restricted ($u_Z=cste$) whilst the straight edges AB and CD are free to wave in the circumferential direction. These boundary conditions are similar to that usually used for the study of compressed plates. The external load is introduced by means of an external uniform compression in the longitudinal direction.

The numerical studies are conducted with the software ANSys version 11. The shell is modelled with the standard structural shell element, Shell 181 [27]. This element type is a quadrilateral 4-nodes

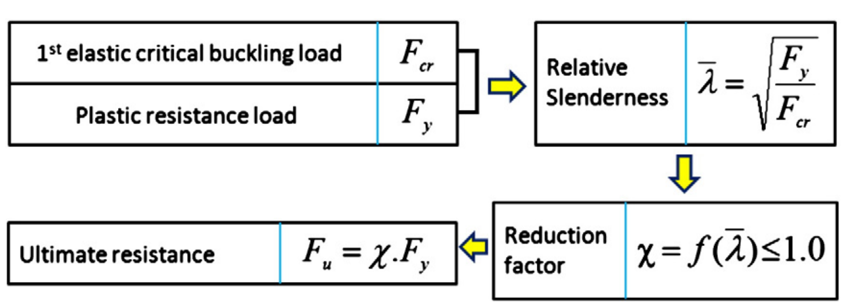

Fig. 2. The $\chi-\bar{\lambda}$ design method according to Eurocode 3 recommendations for stability verifications.

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