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Refined nonlinear finite element modelling towards ultimate bending moment calculation for concrete composite beams under negative moment

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ABSTRACT

In this study, a refined nonlinear finite-element (FE) model of steel-concrete composite beam (SCCB) capable of accommodating various effects such as the initial geometric imperfection, residual stress, interface slip, geometric nonlinearity, and material nonlinearity is developed to study the distortional instability of SCCBs under negative bending moment. The accuracy of the model is validated by publicly available results previously observed by experiments. On the basis of the refined nonlinear FE model, the ultimate bending moment of 54 groups of SCCBs with varying cross-section parameters is calculated. Also, the effects of the force ratio of the SCCB, width–thickness ratio of the compression flange, shear connection degree, and height–thickness ratio of the web on the ultimate bending moment stability coefficient of SCCB in negative bending moment area are investigated in detail. Results show that the effect of the degree of shear connection on the stability coefficient of the SCCB is insignificant, while the stability coefficient decreases with the increase of the remaining parameters, among which the effect of the height–thickness ratio of the web is proven to be the most significant. Finally, considering various effects aforementioned properly, practical formulas are proposed to compute slenderness ratio and ultimate bending moment stability coefficient of SCCB in a concise form, which are shown to be consistent with the FE modelling and experimental results, and thus are suitable for engineering design.

1. Introduction

In recent years, use of steel concrete composite beam (SCCB) has been on the rise [1–4]. As an important structural component designed to carry applied loads, SCCB consists of a concrete slab and a steel beam. The shear connections attached to the concrete slab and the steel beam enable the slab and the beam to work together to carry the loads. Since the torsional and the lateral flexural restraint stiffness of the concrete slab are relatively greater than that of the steel beam, the lateral and the torsional distortions of the top flange of the steel beam are therefore restricted by the concrete slab. As a result, the distortional buckling of a SCCB behaves very differently so that the well-known Vlasov's assumption that the cross-section remains undistorted is not valid.

As shown in Fig. 1, the modes of buckling are generally characterized by the lateral and torsion buckling in the compression zone of the flange and the out-of-plane distortion of the web [5–8]. Johnson [9,10] carried out a series of tests to investigate the failure mode of a continuous SCCB due to distortional buckling. It was found that the buckling failure mode can be classified as distortional buckling, local buckling and a combination of both. The results further showed that the ultimate strength of the test specimens was governed by the interaction between the distortional buckling and the local buckling and was strongly affected by the initial imperfections. Chen [11–14] also conducted research on the ultimate strength of an SCCB, which presented results consistent with those of Johnson [9,10].

Current studies for distortional buckling of an SCCB generally fall into two categories, i.e. theoretically simplified approach and nonlinear numerical analysis approach. The study of the elastic distortional buckling usually employs the theoretically simplified approach such as the elastic foundation beam method and the energy method. For light-gauge metal structures, the Swedish code [15] clarified the issue of considering the buckling analysis of an SCCB in the negative moment area as a stability problem of the elastic foundation beam subjected to a constant axial force. For the elastic foundation beam method, Svensson [16], Williams [17] and Goltermann [18] improved the method for a beam under a constant axial load by introducing the elastic foundation struts subjected to a varying axial force to represent the effect of the moment gradient. Zhou [19] further improved the method by putting forward an equivalent lateral and torsional restraint stiffness to the

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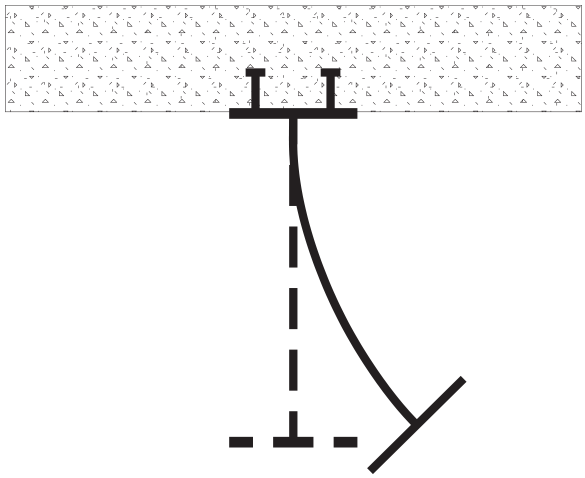

Fig. 1. A distortional buckling of a SCCB.

bottom flanges of an SCCB. In this way, the coupling effect between lateral/torsional restraint stiffness and the applied loads can be properly modeled. In terms of the energy method, the British Institute of Steel Structure [20] used the method to derive a formula that can be used to determine the critical moment of an SCCB under a negative moment. Jiang [21] also used the method to develop a model which can carry out the stability analysis of a steel-concrete composite box beam so as to determine the critical moments of the distortional and local buckling.

It is worth pointing out that abovementioned references analyzing the elastic distortional buckling of an SCCB could not consider the initial imperfections and the material nonlinearity, such as the residual strain, the initial geometric imperfection and the yield criterions of the steel and concrete. Both the residual strain and the initial geometric imperfection in a steel beam can cause deterioration in ductility of an SCCB, which inevitably leads to a decrease in its ultimate strength. In the design practice, these nonlinear influence factors are accounted for by using empirical instability curves, such as the Perry–Roberson curve. This curve is expressed in terms of the slenderness ratio and the reduction factor determined by full-scale testing of members with initial geometric imperfections and the residual stress. Currently, the codes that can predict the bending capacity of the SCCB mainly include the British Bridge Standard [20] and the Eurocode4 [22]. The major difference between these two codes is that the former uses the elastic foundation beam method, while the latter uses the energy method. These two codes share some common features in predicting the bending capacity of the SCCB by using the Perry-Roberson curve to assess the effect of the residual strain and the initial geometric imperfection.

FE modelling has gained widespread attention over the past years in the nonlinear analysis of concrete structure or component due to its versatility and efficiency [23]. The literature on the yield criterions of the steel and concrete indicates that the Drucker-Prager type model can be used by confined concrete, such as FRP-confined concrete, the William–Warnke type model can be employed by non-confined concrete, such as the concrete slab of SCCB, and the Mises type model can be used by steel beam of the SCCB [12,23–26]. Liu [24] developed a three-dimensional FE model to investigate the structural behavior of SCCBs with high-strength friction-grip bolts shear connectors. Concrete and steel were considered by using the William–Warnke type model and Mises type model, respectively. Material nonlinearities and the interaction of different structural components were properly included in the model. The accuracy and reliability of the proposed FE model were validated by comparing their results with available experimental results. Baskar [27] used a three-dimensional FE model to carry out the nonlinear analysis of SCCB under negative bending and shear loading. The comparison between the FE analysis and the corresponding experimental results showed that the proposed nonlinear FE model was capable of predicting the ultimate load behavior of SCCBs to an acceptable accuracy.

In addition to the work aforementioned, Bradford [28] presented an equivalent slenderness ratio and a formula for determining the bending capacity of an SCCB using an inelastic FE method of special purposes. While this method is simple, the SCCB was modeled as a steel beam fully restrained along their tension flanges and the initial geometric imperfection was not involved. As such, the model is not a good representation of a real-life beam. Lin [29–31] carried out an experimental study on the inelastic mechanical behavior of composite girders under hogging moment. The results showed that the current specifications such as EC4 could provide appropriate values for the ultimate strength of a composite girder under negative moment. Tong [32] also conducted eight experiments on the mechanical behavior of SCCBs with two span lengths, i.e. 3 m and 4.2 m, under monotonic negative bending moment. This study showed that the ultimate strengths of the SCCBs determined by EC4 were not accurate. He [33] further implemented an experimental study on the inelastic mechanical behavior of SCCBs under hogging moment, the results showed that up till the ultimate state, the test specimens can be assumed as a full composite section on the basis of the load and slip relationship of shear connectors, which illustrated that the effect of the degree of shear connection on the stability coefficient of SCCBs was insignificant. Su [34] carried out a study of a continuous composite box girder with a prefabricated prestressed-concrete slab in hogging moment region. The results showed that the performance of the prestressed member was better than the conventional member and the routine linear-elastic theory could be used to predict the load at initial cracking. A numerical investigation was carried out by Chen [12] on the inelastic buckling of prestressed continuous SCCBs with external tendons. The factors that affected the buckling moment resistance of the prestressed SCCBs, such as residual stress, initial geometric imperfection, and the slenderness ratio, were analyzed. The results showed that the tentative design method based on the Chinese Codified steel column design curve could be used in the assessment of the buckling strength of the prestressed SCCBs in a term of the defined slenderness ratio. However, this method has certain limitations. For example, the equations are complicated and the accuracy of the method needs further validation.

Owing to the difficulties in considering the initial imperfections and the material nonlinearity in calculating the ultimate bending moment of the SCCB under a negative bending moment, there is still significant space to improve the existing methods so as to formulate more accurate approaches beneficial and practical to engineering analysis. Further exploring the potential of numerical analysis, a highly refined nonlinear FE model of SCCB considering the initial geometric imperfection and residual stress was proposed and verified by experimentally testing data [32,33]. By changing the cross-section parameters, the refined FE model was employed to analyze the effect of different factors such as the force ratio of the SCCB in the negative bending moment area, width–thickness ratio of the compression flange, degree of shear connection, and height–thickness ratio of the web on the ultimate bending moment stability coefficient. On the basis of the analysis aforementioned, practical formulas of slenderness ratio accommodating the effects of different factors and ultimate bending moment stability coefficient of SCCB were proposed in a concise form, which were expected to be used in the future engineering design.

2. Refined nonlinear FE modelling

2.1. FE model generation

The primary concern of this section is modelling of a simply supported SCCB subjected to an in-plane force. The cross-sectional dimension of the model is shown in Fig. 2, where h_w is the height of the steel web, t_w is the thickness of the steel web, b_c is the width of the