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Negative bending capacity prediction of composite girders based on continuous strength method

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ABSTRACT

This paper investigates the ultimate negative bending capacity of composite plate girders, whose ultimate hogging flexural state is governed by the local buckling of the bottom flange of steel girders. The Continuous Strength Method, a deformation-based design approach, which employs a continuous relationship between cross-sectional slenderness and inelastic local buckling deformation capacity, is adopted to accurately predict the ultimate negative bending capacity of composite plate girders. A large number of composite cross-sections, which could represent the conventional cross-section of composite plate girder bridges with medium spans, are designed along with various combinations of steel girder strengths and rebar strengths. For each composite cross-section, its ultimate negative bending capacity based on CSM, ultimate elastic bending capacity, and rigid plastic bending capacity are calculated and compared mutually. Both linear and quadratic interpolation equations are proposed for simplifying the negative bending capacity prediction of the composite section. The analysis results indicate that the linear interpolation bending capacity predicting equation can produce relatively conservative results for most Class 2 and Class 3 cross-sections but the equation is quite concise for application. The quadratic interpolation bending capacity predicting equation can offer more accurate results, especially for cross-sections of which the bottom flange governs the ultimate elastic flexural state rather than the rebar in concrete slabs. The rebar in concrete slabs, the top flange and bottom flange of steel girders are supposed to enter into yield state simultaneously for improving the elastic negative bending efficiency of the composite section. Meanwhile, the proposed quadratic interpolation equation could also provide an accurate negative bending capacity prediction.

1. Introduction

Recent decades, steel-concrete composite bridges have been widely constructed around the world [1–5]. Their prevailing applications, especially in the area of medium span bridges, are owing to the advantages of their outstanding mechanical performance, their accelerated construction period, and their structural sustainability. With the development of composite superstructures, continuous composite twin-girder superstructures have become a popular bridge structural configuration in France, after then have been introduced into other regions [6]. Despite that the continuous composite superstructure could improve the vehicular smoothness and also decrease the depth-span ratio of steel girders compared with the identical span of simply supported composite superstructures, from the standpoint of the engineering mechanics, the composite girder at the internal support regions may be unreasonable since the hogging bending moment could lead to the cracking of the concrete slab and the premature local buckling of the bottom flange.

The structural performance of composite girders subjected to hogging moments has been examined by many researchers. Saadatmanesh et al. [7–10] experimentally and theoretically studied the performance of prestressed composite steel-concrete beams under negative bending moments. The results showed that prestressing a composite girder in the negative moment region could increase its stiffness by preventing the cracking of the concrete slab under service loads and also raise the ultimate bending capacity. Ryu et al. [11–13] implemented a systematic research on composite plate girders subjected to hogging bending moments, which included static flexural cracking behaviour of concrete slabs, the ultimate bending capacity prediction equation with Class 3 cross-sections, and the fatigue performance of prefabricated concrete slabs. He et al. [14] conducted static experimental tests on four composite girders with welded headed stud and perfobond connectors under hogging bending moments in order to investigate the reduction of the flexural stiffness and the inelastic performance after concrete slab cracking. Lin et al. [15,16] concerned the structural behaviour differences of composite girders under the monotonic and

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fatigue hogging bending moments, and the experimental program was developed to investigate various aspects of composite girders, including steel-fibre-reinforced concrete, shear connectors, and rubber-latex mortar coating. The test results illustrated that the current AASHTO LRFD specification provided typically conservative bending capacity prediction for composite girders subjected to hogging moments. Su et al. [17] tested two large continuous composite box girders with prefabricated prestressed concrete slabs under short-term loading to explore their structural performance. The test results indicated that, although the ultimate strengths of those two specimens were almost identical, the initial cracking load and serviceability limit state load of the specimen with a prefabricated prestressed concrete slab were 3.2 and 2.6 times of the specimen with a conventional concrete slab respectively.

As far as authors' knowledge, there were many literatures examined the serviceability performance of the composite girder under hogging bending moments, while less research focused on their ultimate bending capacity in negative bending regions. In addition, rather accurate ultimate bending moment prediction equations for composite plate girders subjected to sagging bending are specified both in EN 1994-2 [18] and AASHTO LRFD specification [19]. But for composite plate girders under hogging bending, the rigid plastic bending capacity or the elastic bending capacity would be chosen for predicting their ultimate negative bending capacity according to the cross-section classification of steel girders. Since the steel girder in hogging bending regions of composite superstructures is always designed as Class 2 (compact) or Class 3 (semi-compact) cross-sections, the elastic bending capacity would underestimate the actual ultimate bending capacity while the rigid plastic bending capacity may result in an overestimated prediction.

The primary objective of this paper is to evaluate the ultimate negative bending capacity of composite plate girders used in twin-girder composite bridges for medium spans. The Continuous Strength Method (CSM) is adopted to predict the critical local buckling strain of the bottom flange of steel girders according to the bottom flange width-to-thickness ratio. A large number of composite sections are designed, and their geometric parameters could represent the conventional composite section in the hogging bending region. The ultimate bending capacity based on CSM, ultimate elastic bending capacity, and rigid plastic bending capacity are calculated and compared mutually. Finally, simplified equations for predicting the ultimate negative bending capacity of the composite section are proposed.

2. Continuous strength method

The concept of cross-section classification is used in the current design codes for steel and steel-concrete composite structures. EN 1993-1-1 defines four classes of cross-sections including Class 1 (plastic), Class 2 (compact), Class 3 (semi-compact), and Class 4 (slender) [20]. For steel cross-sections subjected to bending, Class 1 cross-sections are capable of reaching and maintaining their full plastic moment, Class 2 cross-sections have a lower deformation capacity and could also reach their full plastic moment, Class 3 cross-sections could not reach their full plastic moment owing to the local buckling, and for Class 4 cross-sections, local buckling occurs prior to steel yielding.

The Continuous Strength Method (CSM) is a deformation-based steel structural design approach which provides an alternative treatment to cross-section classification, and the core of the method is a base curve related the inelastic local buckling deformation capacity of a cross-section to its slenderness. The CSM is originally developed by Gardner [21–26] for the structural design with stainless steel material, which exhibits a high degree of strain-hardening effects. It was shown to yield a high level of accuracy and consistency in the resistance predictions of stainless steel cross-sections under various loading conditions, including pure compression [21–26], pure bending [21,23–26] and combined loading [27–30]. Additionally, the CSM has been introduced into the steel structural design [31–35]. Comparisons with experimental and numerical data revealed that the CSM generally provides more accurate and consistent resistance predictions for hot-rolled steel cross-sections than the existing design provisions, especially for very stocky cross-sections and for Class 3 sections in bending [35].

2.1. Cross-sectional slenderness

The CSM design base curve provides a continuous relationship between the cross-sectional inelastic local buckling deformation capacity $\varepsilon_{LB}/\varepsilon_y$ and the cross-sectional slenderness $\bar{\lambda}_p$, where ε_{LB} represents the maximum strain that a cross-section can endure prior to failure by inelastic local buckling and ε_y is the yield strain of the steel equal to f_y/E, with f_y being the steel yield stress and E being the steel Young's modulus. Within the CSM, the cross-sectional slenderness $\bar{\lambda}_p$ is defined as the square root of the ratio of the steel yield stress f_y to the cross-sectional elastic critical buckling stress f_{cr} as given by Eq. (1).

$$\bar{\lambda}_p = \sqrt{\frac{f_y}{f_{cr}}} \tag{1}$$

The cross-sectional elastic critical buckling stress f_{cr} should be determined either using numerical methods or approximately analytical methods. As a conservative option, the cross-sectional elastic critical buckling stress f_{cr} could be determined as that of the most slender plate using the classical plate buckling expression as given by Eq. (2). The classical plate buckling expression assumes simply supported conditions at the edges of the adjoining plates, which neglects element interaction and generally results in a conservative prediction result.

$$f_{cr} = \kappa \frac{\pi^2 E}{12(1-\upsilon^2)}\left(\frac{t}{b}\right)^2 \tag{2}$$

In Eq. (2), κ is the elastic local buckling coefficient allowing for different loading and boundary conditions, E and υ are Young's modulus and Poisson's ratio which are equal to 210,000 MPa and 0.3 respectively, b and t are the width and thickness of the steel plate respectively. For the bottom compressive plate of I cross-section girder, the elastic local buckling coefficient κ equals 0.425.

2.2. CSM base curves

Gardner et al. [32] proposed a base curve for the CSM based on a series of stub column and beam tests with hot-rolled and cold-formed carbon steel cross-sections, and some tests on high-strength carbon steel cross-sections were also incorporated. The relationship between the normalized deformation capacity of the cross-section $\varepsilon_{LB}/\varepsilon_y$ and the cross-sectional slenderness $\bar{\lambda}_p$ was then obtained through regression analysis as Eq. (3).

$$\frac{\varepsilon_{LB}}{\varepsilon_y} = \frac{0.40}{\bar{\lambda}_p^{3.20}} \tag{3}$$

Since the base curve for the CSM illustrated by Eq. (3) could only describe the inelastic local buckling of steel cross-sections, the strain ratio $\varepsilon_{LB}/\varepsilon_y$ reflecting the deformation capacity of steel cross-sections should be greater than 1.0. Eq. (3) could only predict the deformation capacity of the cross-section with its slenderness $\bar{\lambda}_p$ satisfying the condition as given by Eq. (4).

$$\bar{\lambda}_p \le \sqrt[3.20]{0.40} = 0.75 \tag{4}$$

After gathering and analyzing the test data on stainless steel stub columns and four points bending tests from a series of existing test programs and combining with equivalent carbon steel test data, Afshan and Gardner [26] proposed a more general base curve for the CSM as given by Eq. (5). The application of the base curve was extended to the carbon steel components, and was proved to be much accurate over current design approaches to predict the ultimate capacity of structural

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