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Behavior of high-strength steel welded rectangular section beam–columns with slender webs

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ABSTRACT

This paper develops a double nonlinear finite element model that can take account of both geometric and material imperfections. On verified the numerical model the behavior and ultimate carrying capacity of eccentrically loaded welded thin-webbed rectangular section columns made from high strength steel with a nominal yield stress of 460 MPa are analyzed, and further the influences of the slenderness ratio, web depth-to-thickness ratio, flange width-to-thickness ratio, and relative eccentricity ratio on the ultimate carrying capacity are investigated. On the basis of these, the simple calculation formulas, which use the gross cross-section properties, for predicting the in-plane maximum strength of high strength steel beam–columns with large depth-to-thickness ratios are proposed. It shows that the developed finite element model can simulate the local–overall interaction buckling behavior of the eccentrically loaded welded box-section compression members. A brittle failure characteristic is found with a relatively steeper drop just after the peak in the load–displacement curve. A nonlinear and complex stress distribution, in the longitudinal direction, when reaching the ultimate capacity, seriously deviates from an elastic stress distribution, on the basis of which, the effective width is determined. The non-dimensional ultimate bearing capacity is approximately linear with the slenderness, depth-to-thickness ratio and width-to-thickness ratio, respectively. The interaction curves between the axial force and flexural moment for the high-strength steel thin-webbed rectangular section beam–columns are nearly linear. After introducing a steel yield strength modification factor, the formula based on the edge fiber yielding criterion can precisely predict the local–overall interactive buckling strength of high strength steel beam–columns.

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1. Introduction

For a high strength steel member, the limiting value of width-to-thickness ratio against local plate buckling is smaller compared to that made from a common structural steel, and thus the local buckling occurs more easily. If it is a medium-long member, it is more likely to fail in the interaction between local and overall buckling. As a consequence, the local–overall interactive buckling is of vital importance for a member fabricated from high-strength steel.

Over the past three decades, a large number of studies have been reported on the local–overall interactive buckling behavior of welded box- and I- shaped section columns [1–12] and beam–columns [1–4,13–21]. However, only few of them [1,2,21] investigated about the high-strength steel welded box section beam–columns. Usami and Fukumoto [1,2] tested a total of 11 welded rectangular box specimens with the nominal yield stress of 460 MPa and 3 of 690 MPa subjected to eccentric loads to study the local–overall interaction buckling, and presented the strength calculation formulas. Shen and Liu [21] analyzed the ultimate bearing capacities of Q460 steel welded square-box section members loaded with eccentricity by finite element method. All these still need to improve. The complexity of the calculation formulas proposed by Shen and Liu [21] hinders their application. The height–width ratio of the cross-section was taken as a constant value of 0.75 [1,2], and the flange width is greater than the web depth, leading to the flange buckling before the web buckling. It is just one case of the slender rectangular sections: only with slender flanges, there is still the other case: only with slender webs. Usually, for a column subjected to combined axial load and uniaxial bending, in order to enhance the in-plane bending resistance, a welded rectangular profile consisting of high and thin webs together with thick and solid flanges is adopted. In other words, the limit value of web height–thickness ratio is generally increased to permit the local web buckling ahead of the global member buckling, while the flange buckling isn't allowed. To this end, in this paper the local–overall interaction buckling behavior of high-strength steel (nominal yield strength of 460 MPa) welded thin webbed rectangular section beam–columns is analyzed by commercial software ANSYS 8.0 [22,23], and the effect of various factors on the ultimate load carrying capacity is also investigated. Further, on the basis of this, the formulas for predicting the

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local–overall interactive buckling strength of high strength steel beam–columns are presented.

2. Finite element model and verification

2.1. Finite element model

A column loaded with equal eccentricity at both ends is studied in this research and the cross-section of which is a welded rectangular box as plotted in Fig. 1 (x is the flexural axis). A double nonlinear finite element model for such a beam–column considering the geometrical and material imperfections is developed by using the ANSYS program 8.0 [22,23].

The geometric imperfections include both local and global initial deflections. The overall initial deflection is taken as a half sine-wave curve with an amplitude of $l/1000$ (l is the length of a beam–column) as recommended in Chinese Standard GB 50017-2003 [24]. Only the local imperfections of the webs not flanges are taken into account. The local web initial curvature is assumed as follows [9–12]:

$$\omega = \omega_0 \sin \frac{m\pi z}{l} \cos \frac{\pi y}{h} \quad \left(x = \pm \left(\frac{b}{2} + \frac{t}{2} \right) \right) \tag{1}$$

where $\omega_0 = h/1000$ [9], b and h are the flange width and web depth, respectively, m is the buckling half-wave number along the axial direction (z-direction) of a member.

Residual stress measurements of the welded square box section were made by Usami and Fukumoto [1]. SM58 steel plate with a thickness of 4.5 mm was used, the nominal yield strength of which was 460 MPa and measured yield strength was 568 MPa. The width-to-thickness ratio of compression flange, b/t, was taken as 29, 44 and 58. Measured residual stress patterns were all similar in shape to the well-known pattern, tensile stresses about 80% of the measured yield strength (i.e., $0.8 \times 568 = 454$ MPa, which is very close to the nominal yield strength of 460 MPa) were measured near the corners and nearly constant compressive stresses were observed over the central portion of each plate. The average values of measured compressive stresses were 32%, 22% and 15% of the measured yield strength for specimens with $b/t = 29$, 44 and 58, respectively. Also a similar residual stress distribution model was found in a welded rectangular cross-section [4]. Therefore, the simplified pattern of residual stress distribution as shown in Fig. 1 is adopted, where tensile residual stresses are designated as positive and compressive stresses as negative. The maximum value of tensile residual stresses $\sigma_{rt} = 454$ MPa and those of compressive residual stresses, σ_{rcf} and σ_{rcw}, along the flange and web, respectively, may be obtained according to the equilibrium condition.

Shell181 element is selected to consider the influence of local buckling. The stress–strain relationships measured for SM58 steel were similar to those for mild steel [1], so the steel material is assumed to be elastic-perfectly plastic. The measured material properties: yield strength $f_y = 568$ N/mm^2, Young's modulus $E = 213{,}000$ N/mm^2, and Poisson's ratio $\nu = 0.225$, are used.

In order to take account of geometric imperfections, the direct modeling method developed by Shen [9] is used. Residual stress is treated as initial stress, which is a loading and must be exerted at all integration points of Shell181 elements at the first sub-step of the first load step [9] just as required in ANSYS software [22,23].

An end plate with a thickness of 30 mm is attached to both ends of a member, on which boundary conditions are applied, to ensure that the two ends of the beam–columns are hinged [9]. Eccentric load on each end is equivalent to combined axial compression and bending moment, which are exerted at the central points of both end-plates. Three lateral braces are used to avoid the out-plane instability. A typical finite element model is shown in Fig. 2. The arc-length method iterative is adopted to acquire the descending segment of a load–displacement curve and to accelerate the convergence [25].

2.2. Verification of the finite element model

The finite element model was verified with the experimental results. 35 experimental specimens, of which 21 were centrally loaded, 14 were eccentrally loaded by Usami and Fukumoto [1,2] were simulated. Table 1 gives a comparison of the numerical and experimental results, in which φ_{test} and φ_{FEM} represent the strength reduction factors obtained from the experiment and numerical simulation, respectively, and equal to the corresponding ultimate carrying capacity, P_u, divided by the product of the gross cross-section area, A, and the measured yield stress of steel, f_y. The ratios of the numerical and experimental ultimate strength, $\varphi_{\text{FEM}}/\varphi_{\text{test}}$, change from 0.901 to 1.084 with an average value of 1.012 and a standard deviation of 4.53%. Comparison shows that the numerical results agree very well with the experimental ones, indicating that the finite element model provided in this paper can accurately predict the local and overall interaction buckling strength of welded box columns and beam–columns.

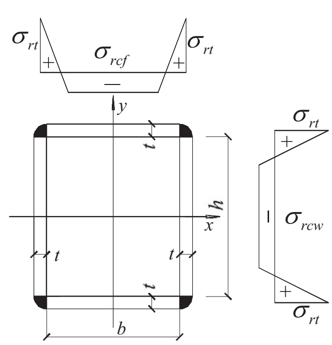

Fig. 1. Welded rectangular section and residual stress distribution.

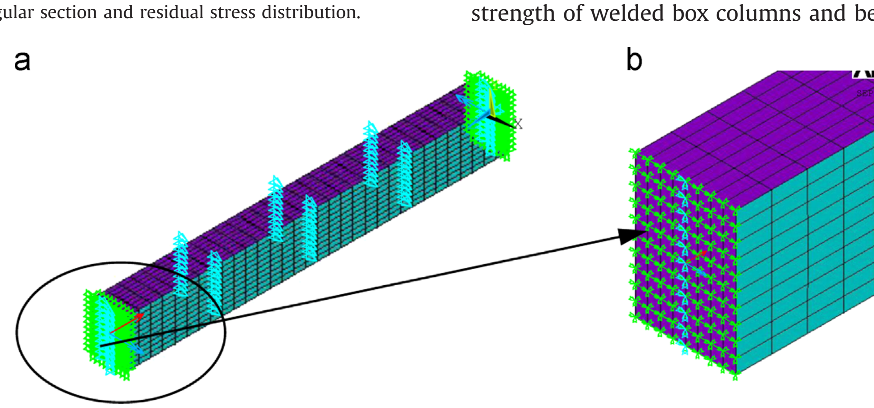

Fig. 2. Typical finite element model: (a) overall model; (b) end loads and boundary conditions.

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