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LOCAL BUCKLING BEHAVIOR OF DOUBLE-COPED STEEL BEAMS

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Local web buckling is one of the most common failure modes for coped steel beams. While several studies have been undertaken focusing on the behavior of top-flange/single-coped beams, double-coped beams have received little attention. To fill this knowledge gap, this paper presents experimental and numerical studies on local web buckling behavior of double-coped steel beams. Five full-scale tests were conducted, and the main test parameters were cope length and cope depth. Local web buckling was observed as the main failure mode for all of the five specimens, and the buckling resistance was found to decrease with increasing cope length and cope depth. A FE study was subsequently conducted, where the response of the FE models agreed well with the test results, especially in terms of buckling mode and buckling resistance. The test and FE results were compared with those predicted by an existing design approach. The design results were found to be quite conservative, and hence further investigation may be required to achieve a more accurate design approach.

Keywords: Local web buckling, Failure mode, Deflection, Test, FE analysis, Design.

1 INTRODUCTION

In steel structures, the same elevation is often required for beams at member intersections to satisfy architectural purposes. In this case, the secondary beams are usually coped to avoid interference of the connected structural members, such that sufficient clearance can be provided for the connections. From a structural point of view, however, the removal of part of the flange/web can lead to a reduced load-carrying capacity, where local web buckling is one of the most common failure modes for coped beams (Milek 1980). The local buckling response of coped beams has been studied by researchers through full-scale tests and numerical investigations (Cheng *et al*. 1984, Yam *et al*. 2003). However, the emphasis of those investigations was mainly on the response of top-flange/single-coped beams, whereas available information on double-coped beams is rare. Double-coped beams are usually employed under the condition where identical elevations of both top and bottom flanges of the connected beams are required. Although Cheng *et al*. (1984) have undertaken a numerical study on double-coped beams, and proposed a design approach that was found to agree well with the numerical results, no physical test data are available to further validate their study. Therefore, the main objective of this study is to perform experimental and numerical investigations on local web-buckling behavior of double-coped beams. The results obtained from the design equation proposed by Cheng *et al*. (1984) are also discussed through comparisons against the test and FE results.

2 TEST PROGRAM AND RESULTS

2.1 Test Specimens

Five full-scale tests were conducted, and the main test parameters were cope length (c) and cope depth (d_c). Nominal geometric configurations of the test specimens are illustrated in Figure 1, and the measured dimensions are given in Table 1. For easy reference, test codes are allocated for each specimen, as given in Table 1, where 'C' represents nominal cope length (in mm), and 'd' represents nominal cope depth (in mm). A 10mm-thick end plate was welded to the end of the coped beam web. S355 UB356×127×33 steel beams were employed for conducting the tests. Tension coupon tests were carried out to obtain the material properties of the test beams. The mean values of the yield strength, ultimate strength, and Young's modulus of the web material were 366.8 MPa, 482.2 MPa, and 207.4 GPa, respectively.

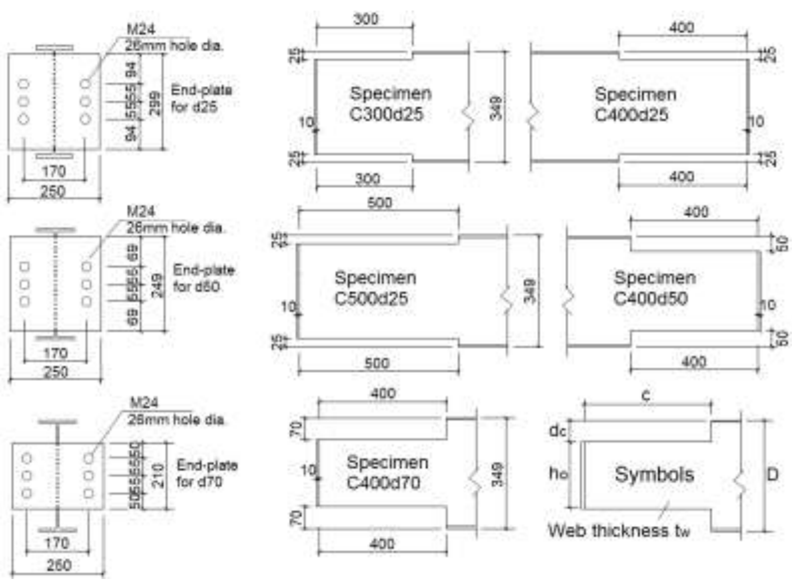

Figure 1. Nominal geometric configurations for specimens.

Table 1. Measured dimensions of specimens (mm).

Test code	Web thickness (t_w)	Flange thickness (t_f)	Cope length (c)	Cope depth (d_c)
C300d25	5.73	7.91	298.5	24.8
C400d25	5.75	7.97	400.0	25.0
C500d25	5.75	7.97	502.5	24.3
C400d50	5.76	7.94	400.8	48.8
C400d70	5.77	7.97	400.8	70.8

2.2 Test Setup, Instrumentation, and Procedure

The test arrangement is schematically shown in Figure 2. The statically-determinate condition of the test beam allowed measurements of cope-end reaction via simple force equilibrium. Lateral bracings were employed to prevent lateral-torsional buckling of the test beam, and to avoid lateral movement of the compressive beam flange near the coped end. The beam specimens were loaded by a hydraulic jack with a maximum capacity of 1000 kN. The distance between the loading point and the end plate was around 900mm, such that the influence of localized load bearing on the stress distribution in the coped region was negligible. M24 Grade 8.8 bolts were used to connect the end plate and the column flange. Additional bolt washers were placed between the end plate and the supporting column face, to eliminate pry action, and to avoid the contact between the end plate and the column face. This ensured the rotational restraint provided by the coped end connection could be minimal. A roller support was used at the other end of the beam.

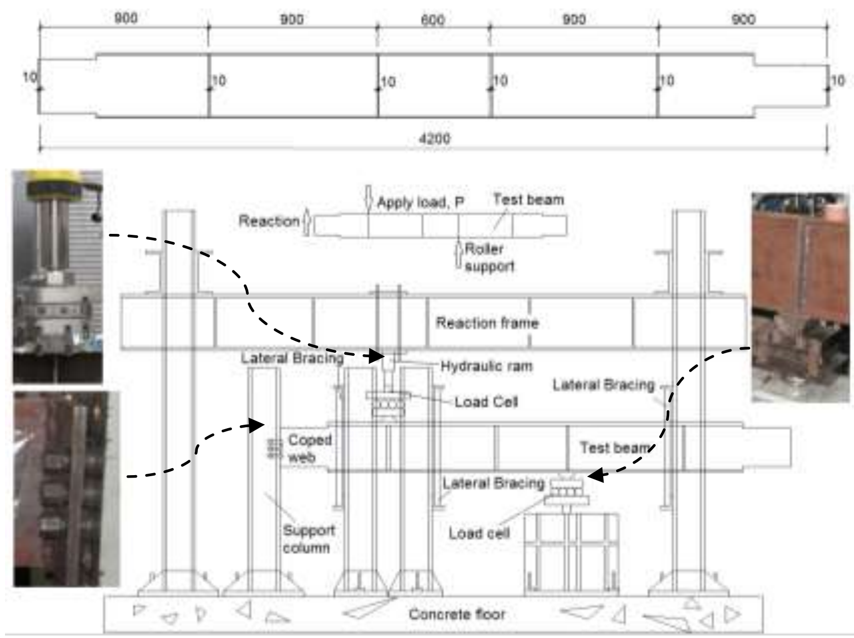

Figure 2. Test setup and beam dimensions.

The instrumentation details for the test specimens can be found in Wang (2011). The coped region was whitewashed for detecting any buckling line during the test. The loading process was mainly divided into two stages: load control and stroke control. In the early stage, load control was used. When nonlinear response of the test specimen was initially featured, the second loading stage, stroke control, was used to more reliably capture the nonlinear load-deflection response. The same test procedure was employed for all the specimens.

2.3 Test Results

The main test results, including the ultimate load P_u, the ultimate reaction R_u, the vertical deflection δ at the loading position when P_u was achieved, and the failure mode, are summarized in Table 2. Local web buckling was observed as the main failure mode for all the specimens. Typical buckling shapes of the specimens are shown in Figure 3, where the buckling line can be clearly found. The buckling line was extended from the top edge of the coped web towards the bottom edge, with an orientation of an approximately 25º angle from the vertical line. The lateral deflection at the top edge of the coped web was more significant than that at the bottom edge, although the latter was also shown to be evident.

Figure 3. Typical buckling mode – test and FE results.

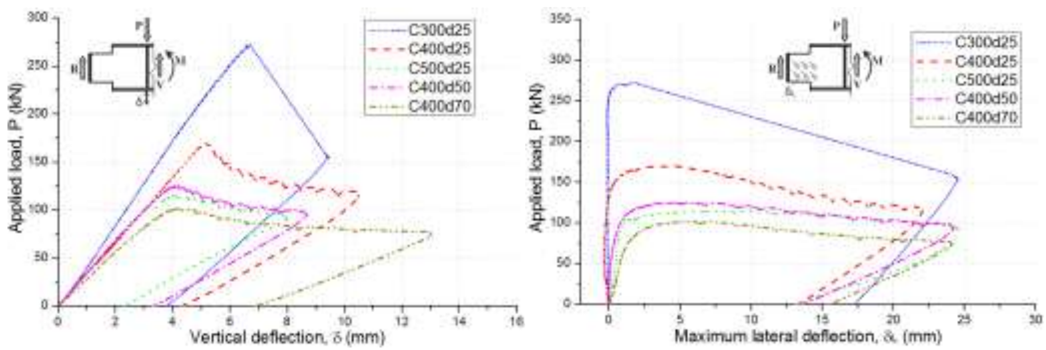

Figure 4. Load vs. deflection responses.

Table 2. Summary of test results.

Specimens	Ultimate load P_u (kN)	Ultimate reaction R_u (kN)	In-plane deflection δ (mm)	Failure mode
C300d25	272.1	171.9	9.4	Local web buckling
C400d25	170.2	107.8	10.5	Local web buckling
C500d25	114.8	73.1	8.0	Local web buckling
C400d50	125.3	79.4	8.7	Local web buckling
C400d70	101.7	64.2	13.0	Local web buckling

The load versus vertical in-plane deflection and the load versus maximum web lateral deflection responses for the five test specimens are shown in Figure 4. For the load versus vertical deflection curves, a linear response was generally observed until the load reached at least 95% of the ultimate load, quickly after which the load carrying capacity started to drop. The almost linear behavior prior to the ultimate load implies that the elastic-load web buckling mainly governed the buckling mode. For the load versus maximum measured lateral deflection curves, slight lateral deflections were observed with increasing loads, which could be due to the imperfections of the coped webs. The lateral deflections started to increase quickly in a nonlinear manner when approximately 80% of the ultimate loads were achieved. After reaching the ultimate loads, the lateral deflections increased substantially.

3 FE ANALYSIS AND DESIGN

3.1 FE Analysis

The nonlinear FE analysis package ABAQUS (2011) was employed for the numerical study. Four-node quadrilateral shell elements (S4R) with reduced integration were used to discretize the members. At the coped beam end, the bolt holes were constrained in all the three degrees of freedom. For the uncoped end, only the vertical degree of freedom was constrained to simulate the roller support. An isotropic elastic–plastic material model considering the von Mises yield criterion was employed. The stress-strain response was obtained directly from the coupon tests (Wang 2011) and was then converted to the true stress and true strain used for the FE model. Two steps were adopted to obtain the nonlinear buckling behavior of the coped beams. The first step was an eigenvalue analysis, mainly used to obtain the lowest elastic buckling mode. In the second step, a concentrated load was applied at the loading position, and a nonlinear Riks analysis was used to trace the nonlinear buckling response of the coped beams. The initial geometric imperfection shape obtained from the first step was incorporated in this step, and the amplitude was taken according to the measured value (Wang 2011). Figure 3 shows the typical buckling mode of the FE models, where the location and orientation of the buckling line are well predicted. Table 3 gives the ultimate reactions R_u predicted by the FE models, and reasonable agreements are generally shown. The discrepancies, which may attribute to the deviations of material properties and boundary conditions, are generally within 10%.

Table 3. Comparison of ultimate reaction R_u among test, FE, and design results.

Specimens	Test (kN)	FE (kN)	Design (kN)
C300d25	171.9	154.8	129.2
C400d25	107.8	99.3	72.6
C500d25	73.1	67.1	46.5
C400d50	79.4	73.4	49.5
C400d70	64.2	60.1	34.2

3.2 Design

The local web buckling strength of double-coped beams was first investigated by Cheng *et al*. (1984) via numerical studies, upon which a design approach was proposed. The design approach considered a simple lateral-torsional plate buckling model with an unbraced length of c, and a linearly-increased bending moment diagram. The critical moment M_{cr} was determined by:

$$M_{cr} = f_d \left(\frac{\pi}{c}\right)\sqrt{EI_y GJ} \tag{1}$$

where E = Young's modulus, $I_y = h_o t_w^3/12$, G = shear modulus, $J = h_o t_w^3/3$, f_d = an adjustment factor taken as $f_d = 3.5 - 7.5(d_c/D)$ to account for other parameters such as stress concentration, cope depth, and shear stress. Relevant geometric symbols are illustrated in Figure 1. The ultimate reactions obtained from Eq. (1) are given in Table 3, which are found to be quite conservative. This is probably because that the FE models used by Cheng *et al*. (1984) assumed an idealized simply-supported condition at the coped end, whereas in actual conditions end-plate connections were used. The end-plate connections inevitably produced a certain level of rotational restraint which can lead to increased local web buckling resistance.

4 CONCLUSIONS

This paper presented experimental and FE studies on local web buckling behavior of double-coped steel beams. Local web buckling was observed as the main failure mode for all the specimens, and the buckling resistance was found to decrease with increasing cope length and cope depth. A FE study was subsequently conducted, and an existing design approach was also examined. Reasonable agreements were observed between the test and FE results, and therefore the FE modelling strategy employed in this paper was validated. The design approach proposed by Cheng *et al*. (1984) was found to be conservative, was mainly due to the idealized simply-supported boundary condition initially assumed in Cheng's numerical model. The conservative design results also implied that the rotational restraints offered by the coped end connection could support the local web buckling resistance of double-coped beams. The test and FE results presented in this paper can also be used as benchmarks for further parametric studies.

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