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Local web buckling of double-coped steel beam connections

Cheng Fang [a,*], Michael C.H. Yam [b], Angus C.C. Lam [c], Yunhao Liu [c], K.F. Chung [d]

[a] Department of Structural Engineering, School of Civil Engineering, Tongji University, Shanghai 200092, China
[b] Department of Building & Real Estate, The Hong Kong Polytechnic University, Hung Hom, Kowloon, Hong Kong SAR, China
[c] Department of Civil & Environmental Engineering, University of Macau, Macau SAR, China
[d] Department of Civil & Environmental Engineering, The Hong Kong Polytechnic University, Hung Hom, Kowloon, Hong Kong SAR, China

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ABSTRACT

This paper presents a comprehensive study on local web buckling behaviour of double-coped steel beam connections, cases which are common when beams of similar heights are joined. The study commenced with a series of full-scale tests on 11 specimens, covering a spectrum of cope lengths and cope depths. Local web buckling was observed as the main failure mode for most specimens. The ultimate load was found to decrease with increasing coped length and cope depth, and in addition, an increase of the rotational stiffness of the beam end connection could benefit the local web buckling capacity. A numerical study was subsequently performed enabling further interpretation of the test results. Good agreements were observed between the test results and finite element analysis predictions, and the stress conditions within the coped web panel at different loading stages were fully revealed. The numerical study also showed that the ultimate loads of some models were not sensitive to initial imperfection amplitudes, especially when the cope length was large (i.e. 450 mm or longer). It was believed that the imperfection insensitivity characteristic was due to the presence of post-buckling mechanism. Summarising the available test data, including the current test results and those previously reported by other researchers, design comments were made through comparisons against the existing design method. Conservative test-to-predicted ratios were generally shown, but unsafe predictions were obtained for some cases. A modification to the existing design approach was finally proposed for safer design of such connections.

1. Introduction

In practical design of steel structures, the grillage beams often need to be coped at the beam end to avoid interference of the connected structural members thus to ensure the same elevation of the flanges of the beam at member intersections. Depending on specific design requirements, either the top or both top and bottom flanges of the beam end need to be removed, as shown in Fig. 1, and in either case, the load carrying capacity of the beam is inevitably compromised [1]. Coped beams can have different failure modes from those observed in their uncoped counterparts, and local web buckling is one of the most common failure modes at the coped region. Significant stress concentration can also be induced near the cope corner, which may further decrease the load carrying capacity [2]. Another common local failure mode for coped beams is 'block shear' which is featured by a fully or partially tearing-out of a block of material near the connection zone. In addition, lateral torsional buckling, a global instability failure mode, can more easily occur for coped steel beams compared with uncoped ones if insufficient lateral constraint is provided.

The structural response of coped beams has been studied by various researchers through either full-scale tests or numerical investigations. Cheng and Yura [3] reported ten tests on local web buckling response of top flange coped beams. Based on a subsequent numerical study, a plate buckling calculation model was developed for elastic local web buckling design of top (single) flange coped beams, and it was claimed that the proposed design provisions could give conservative results. The above design equations have been adopted by the AISC Steel Construction Manual [4]. Through conducting more local web buckling tests and numerical studies, Yam et al. [5] improved the plate buckling formula proposed by Cheng and Yura [3] to additionally consider the shear buckling phenomenon at the coped region. The modified calculation method was shown to agree better with the test results. Aalberg and Larsen [6] conducted six tests to study the local buckling response of top flange coped beams, where a simply-supported condition of the coped beam end (which was seated in a small steel block supported by a cylindrical bearing) was considered. Aalberg [7] later revisited those testing details and presented additional FE analysis. Based on the parametric study, a new design model incorporating a reduction factor for coped beam capacity was finally developed to account for the effects of cope. Recently, more test data on the local web buckling behaviour of aluminium coped beams were provided by Aalberg [8], and it was found that the design model proposed by Yam et al. [5] could lead

* Corresponding author.
E-mail address: chengfang@tongji.edu.cn (C. Fang).

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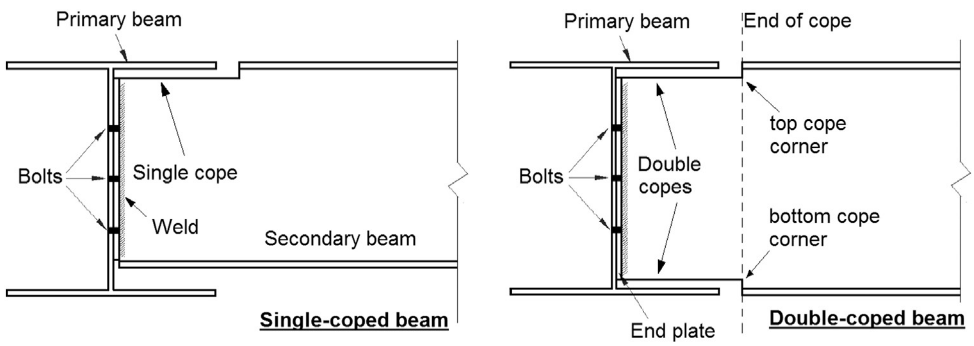

Fig. 1. Practical coping details of steel beams.

to better predictions than that proposed by Cheng and Yura [3]. Some researchers also proposed reinforcing strategies aiming to improve the local web buckling performance of coped beams through employment of various stiffener types, and those methods were found to be effective [9–10]. Apart from the studies on local web buckling, extensive investigations have also been conducted on the block shear [11–15] and lateral-torsional bucking [16–19] responses of coped beams. A comprehensive state-of-the-art review on local failures of coped beams was recently reported by Yam et al. [20].

However, it is noted that the emphasis of the previous research was mainly given to the response of top (compressive) flange single-coped beams, whereas there is limited information on the performance of double-coped beams. Double-coped beams are popular for the condition when beams of similar heights are joined, as illustrated in Fig. 1. Compared with the case of single-coped beams, more significant reduction of strength and stiffness may be induced in double-coped beams due to the removal of both beam flanges, and therefore, their behaviour needs to receive more attention. Cheng et al. [2] have undertaken a series of numerical studies on double-coped beams and proposed relevant design recommendations (hereafter named as the 'design method'). For practical design, it was suggested that the load carrying capacity (i.e. reaction) of double-coped beam connections could be sufficiently predicted by checking: 1) elastic local buckling capacity, 2) flexural yielding capacity, and 3) shear yielding capacity of the coped region, where the lowest value governs. Although the design recommendations were found to agree well with the numerical results, no physical test was conducted to further validate the design model until recently. Aalberg [8] conducted two tests on double-coped beams made of aluminium, one with welded end-plate and one with bolted double angle cleat connections. It was found that the connection type could have evident influence on the local web buckling capacity. Considering the design method [2], the test-to-predicted ratio for the coped beams with the two connection types were 0.83 and 1.36, respectively (flexural yielding of the coped web governed). The authors [21] also performed five preliminary local web buckling tests on double-coped beams, where relatively large cope lengths ($c \geq 300$ mm) were employed to facilitate the observation of local web buckling mode. It was found that the design method tended to be conservative. These test data will also be included in the discussion of the current paper.

The main objective of this study is to help fill the knowledge gap in the area of local web buckling behaviour and design of double-coped beam connections. A total of 11 full-scale tests were conducted, covering a spectrum of coping details (including varying cope lengths and cope depths). A numerical study was subsequently carried out to enable further discussions on the stress distribution, buckling mechanism, and imperfection sensitivity of the specimens. Preliminary design recommendations were finally made through comparing the available test data pool against the existing design method.

2. Experimental programme

2.1. Test specimens

Six S460 UB406 × 140 × 39 test beams were ordered, and a total of 11 full-scale tests on double-coped beam connections were conducted. The main test parameters included cope length (c), cope depth (d_c), and the effect of end-plate connection rotational stiffness. The nominal dimensions for the top and bottom copes were identical for each specimen. The typical geometric configurations and key symbols of the test specimens are illustrated in Fig. 2(a), and the key dimensions of the coped web are given in Table 1. The end-plate connection type (with 10 mm end-plate thickness) was employed for all the specimens, and double fillet welds were employed to connect the end-plate and the beam end. The depth of the end-plate (d_{ep}) was the same as the remaining web depth (h_o). For easy reference, each specimen was assigned with a test code, where 'C' represents nominal cope length (in mm), and 'dc' represents nominal cope depth (in mm). For specimen C550dc50R, the last letter 'R' represents the case of increased rotational stiffness of the end-plate connection, which was achieved through decreasing the gauge distance g (from 175 mm to 90 mm) of the bolt hole, as shown in Fig. 2(a). The radius of the rounded cope corner was 15 mm for all the specimens. Photos of typical test specimens are given in Fig. 2(b). In the following discussions, for ease of understanding, the section along the boundary line between the coped and uncoped web panels is named as the 'end-of-cope section', as illustrated in Fig. 1. The top and bottom cope corners are also clearly marked in the figure.

The test specimens can be divided into two groups: the first group included those with cope length c up to 150 mm, which realistically represents typical coping details in normal building structures that employ profiled I-sections; the second group incorporated cope lengths ranging from 450 mm to 550 mm, reflecting more extreme cases. It was expected that the specimens in the first group failed by inelastic local web buckling, while those in the second group were expected to fail by elastic local web buckling. The main purpose of covering both failure modes in the current test programme was to fully examine the design method proposed by Cheng et al. [2]. For each test beam, both ends were coped with different coping details and they were considered as separate test specimens. When the test on one end was completed, the test beam was turned around for another test at the opposite end. The dimension of a typical test beam is shown in Fig. 3. Tension coupon tests were carried out to obtain the material property of the test beams, and the tests were conducted in a universal testing machine according to the ASTM A370 standard [22]. The Young's modulus, yield strength, and ultimate strength of the coped webs of the test beams are listed in Table 2.

2.2. Test setup

The test arrangement is schematically shown in Fig. 3. The statically determinate condition of the test beam allows measurements of cope