Journal of Constructional Steel Research 113 (2015) 146–155

Contents lists available at ScienceDirect

Journal of Constructional Steel Research

Experimental and numerical parametric study on the capacity of coped beam ends

A. Aalberg

Norwegian University of Science and Technology, Department of Structural Engineering, Richard Birkelandsveg 1a, N-7491 Trondheim, Norway

ARTICLE INFO

Article history:
Received 15 December 2014
Received in revised form 26 May 2015
Accepted 15 June 2015
Available online 21 June 2015

Keywords:
Coped beam
Web buckling
Seat connection
FE simulations
Design

ABSTRACT

Here, the reaction force capacity of I-section steel beam ends with top flange copes was investigated experimentally and numerically, with a focus on local web buckling. An experimental test series was carried out on unstiffened top coped beam ends that were subjected to reaction forces under the bottom flange. Experimental results are given for beam ends with five sizes of cope and for a reference beam without a cope. The investigated beam section was a standard IPE 300 in steel grade S355. The test results were compared with existing design models for coped beams and with models primarily developed for triangular stiffener brackets. A new design formula is proposed for top coped beams supported at the bottom flange, which allows the capacity to be determined as a function of capacity of uncoped beam end with a reduction based on the size of the cope. Finite element models were verified against the experiments and used to extend the experimental data and investigate parameters affecting the web buckling resistance, such as web slenderness and the restraints provided by connection devices.

1. Introduction

When detailing structural joints, such as in grillage beams, the coping of component ends may be necessary for the parts to fit together. For I-section beams, such coping typically implies cutting off one or both flanges at the end of one of the beams while leaving most of the web height intact. Fig. 1 shows three examples of coped beams, which can be made using either a connection plate welded to the main girder to connect a secondary beam, an end-plate welded to the web and flange of the secondary beam or a seat connection. In most cases, simply supported end connections are assumed when coped beam connections are used.

The coping reduces both the strength and stiffness of the beam end, leaving the reduced section in either a tee-section or rectangular shape. In addition, the cope introduces stress concentrations at the cope corner and reduces the stability of the beam web due to the lack of transverse restraint from the flange along the horizontal edge of the cope.

2. Previous studies and design recommendations

At least four distinct failure modes must be considered when designing unstiffened coped beams. These failure modes include (1) yielding of the reduced section due to bending moment and shear force, (2) local web buckling at the cope, (3) block shear failures in the web around groups of fasteners, and (4) lateral-torsional buckling (LTB) of the entire member in the case of a laterally unrestrained beam with reduced torsional and lateral-bending stiffness in coped regions.

LTB of end coped I-beams has been investigated by several authors [1–10]. These investigations range from classical energy solutions for general beams to FE studies on complete assemblies of beam and connections and cover cases with copes at one or both flanges and various connections, such as end-plates, partial end-plates and web angles. Design models use either an elastic LTB moment determined from interaction between the critical moments of the cross-section in the coped regions and the I-section of the uncoped length [5] or numerically determined reduction factors for the critical moment for various connection cases, followed by a design with the recommended buckling curves [9]. Coping has been shown to reduce the LTB resistance significantly, particularly for short beams and cases where the connection devices are not connected to the tension flange [9]. In addition, local web buckling in the coped region has been found to influence the LTB capacity significantly [7,10].

Cheng and Yura [11], Cheng [12] and Yam et al. [13] studied the resistance to local web buckling at the coped end. Cheng and Yura [11] presented FE simulations and experiments for laterally supported I-beams with top copes, connected with partial end-plate or angle cleat connections. They used their data to develop a plate buckling model. Their design recommendations were adopted by the AISC Manual of Steel Construction, Load & Resistance Factor Design (AISC-LRFD) [14]. Recently, Yam et al. [15] presented a comprehensive state-of-the-art review on the various possible local failures that can occur in coped steel beams.

E-mail address: arne.aalberg@ntnu.no.

http://dx.doi.org/10.1016/j.jcsr.2015.06.004
0143-974X/

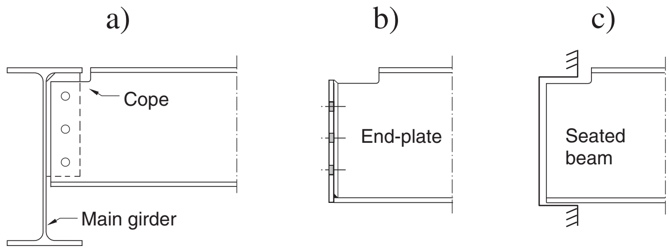

Fig. 1. Examples of coped beam connections.

Fig. 2a shows a schematic view of the cope geometry and load application of a beam with a top flange cope of cope length c and depth d_c. The reaction force R acts at a nominal distance e from the vertical face of the cope (line g–g). The actual position of R depends on the connection type and rotational restraint provided by the supporting member. A common simplification is to assume that the inflection point is located at the face of the supporting member. The linear bending stress distribution over the height h_0 of the tee-section, shown in Fig. 2a, is clearly a simplification, as the singularity at the cope corner will result in a complex stress field near the corner. Cheng and Yura [11] used a basic plate buckling model for the region of the web affected by the cope, represented by a rectangular model plate with three simply supported edges and one free edge, as shown in Fig. 2b. The loading was assumed to occur through linearly varying compressive stress over the height h_0. The design value of the buckling bending stress at the top of the web was given by

$$\sigma_{cr} = f_1 \frac{k_\sigma \pi^2 E}{12(1-\nu^2)} \left(\frac{t_w}{h_0}\right)^2. \qquad (1)$$

The factor f_1 is a factor that accounts for the effects of the stress concentration at the cope corner, the depth of the cope and the presence of shear stresses and the moment gradient along the cope length [11]. The factor k_σ is the buckling factor, which is derived from analytical solutions supplemented by numerical simulations. These expressions have been described previously [11,14,15]. The design value R_d of the reaction force is subsequently derived from the moment capacity of the tee-section, with a bending stress $\sigma = \sigma_{cr} \le f_y$ at the end of the cope. In addition, the mean shear stress $\tau = R/h_0 t_w$ in the web must not exceed the yield shear strength. For beams coped at both flanges, design recommendations are given in [12]. These recommendations are derived from the elastic lateral-torsional resistance of the web, which is considered as a component with a rectangular cross section and length c.

The experiments of Cheng and Yura [11] showed that for top coped beams where the web buckled in the elastic stress range, the predicted capacity (R_d) according to their model ranged from 49% to 75% of their experimental results. This was deemed satisfactory considering the simplifications of the model. Furthermore, they [11] did not explicitly model the residual stresses or web imperfections. For cases that buckled in the inelastic range, the test results showed that the capacity of the coped section determined by the lesser of yielding in bending or shear provided conservative values for the reaction force. The experimental values exceeded the predictions by 20% on average. Some end restraints provided by the connections, both in-plane and out-of-plane, likely contributed to these larger values. The plate buckling model of Cheng and Yura (Fig. 2b) is easy to use. However, it does not directly reflect the fact that for many cases, the observed buckle actually extends into the uncoped section of the beam and that this buckle is rotated by approximately 45° from the vertical line if shear buckling dominates. The test observations of Cheng and Yura [11], Yam et al. [13] and the present investigation indicate that shear buckling may occur for many cope geometries.

In view of this, Yam et al. [13] proposed a design model with a geometry and stress field as shown in Fig. 2c. The shear stress distribution along the edges was determined from FE simulations of a range of beam and cope geometries, giving distributions and magnitudes that varied along the edges. Bending stresses could be neglected for this model. The design value of the reaction force, R_d, for the elastic local web buckling of the top coped I-beam was given by

$$R_d = \tau_{cr} t_w (h-d_c) = k_s \frac{\pi^2 E}{12(1-\nu^2)} \left(\frac{t_w}{h_0}\right)^2 t_w (h-d_c) \quad \text{with} \quad k_s = a\left(\frac{h_0}{c}\right)^b. \qquad (2)$$

The parameters a and b vary with the ratio d_c/h [13]. For small ratios, the shear model predicted capacities ranging from 92% to 106% of the measured capacities [11,13], which represented an improvement compared to the model proposed by Cheng and Yura [11].

Yam et al. [16,17] and Yam and Chung [18] investigated the effect of strengthening the beam end by adding welded stiffeners. They demonstrated that horizontal stiffeners could not efficiently prevent web buckling and that vertical stiffeners had to be added to obtain a reasonable capacity. Both experiments and numerical investigations were performed.

Block shear failures may be particularly important for coped beams. The existing design formulas, such as in EN 1993-1-8 (Eurocode 3) [19], are adequate and are supported by experimental investigations. Kulak and Grondin [20] provided a thorough discussion of these models, and Aalberg and Larsen [21] provided tests extending the experimental basis to beams in high-strength steels and aluminum [22]. The capacity formulas of EN 1993-1-5 [23] for end patch loading [24–26] may serve as a basis for the design rules for coped beams.

The experimental results and test arrangement described in this paper were described previously in brief [27]. Here, the full details about the testing and test results are provided, along with an analysis of the various shapes of web buckles and how the buckling affects the resistance of the beam ends for concentrated reaction forces. Additional

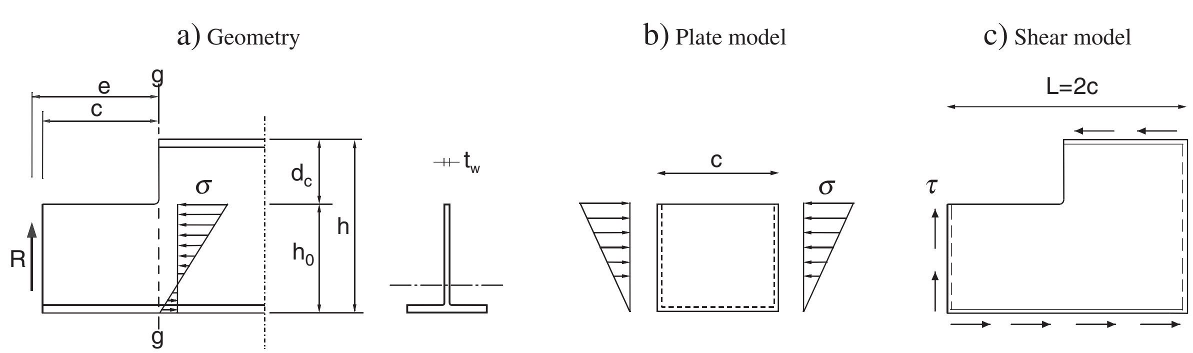

Fig. 2. I-beam with top cope, plate model [11] and shear model [13].