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Behaviour of stiffened extended shear tab connections under gravity induced shear force

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ABSTRACT

Stiffened extended shear tab connections (either in full-depth or partial-depth configurations) are widely used to connect simply supported beams to the web of supporting girders or columns. Full-scale laboratory tests of stiffened extended shear tab connections underscored the differences between their observed and expected design strength calculated according to current design specifications. In particular, the design procedure of such connections neglects the influence of the out-of-plane deformation of the supporting girder web on yielding and inelastic buckling of the shear plate. These are the main governing failure modes for the full-depth configurations of stiffened extended shear tabs, when placed on one side of a supporting girder or column. The research described in this paper aims to develop a better understanding of the load transfer mechanism and failure modes of extended beam-to-girder shear tab connections. The findings are based on finite element (FE) simulations validated with full-scale experiments on beam-to-girder shear tab connections. The influence of girder web flexibility on the behaviour of single-and double-sided shear tabs is assessed. The stiffened portion of the full-depth extended shear tabs yielded due to the interaction of horizontal shear and vertical axial force. Due to the flexibility of the girder web of the single-sided shear tab, its stiffened portion experienced much larger vertical axial force in comparison to that of the double-sided configuration.

1. Introduction

Extended shear tab connections are widely used in steel construction practice due to their ease of fabrication and erection. They consist of a steel plate, which is shop-welded to the supporting girder or column and then bolted to the supported beam in the field. The increased shear tab length allows the beam to be connected to the girder web without coping the beam's flanges (Fig. 1). The shear plate may be welded to the girder web alone, i.e. unstiffened configuration (Fig. 1a), or may be connected either to the top flange, i.e. partial-depth stiffened configuration (Fig. 1b) or to both the top and bottom flanges, i.e. full-depth stiffened configuration (Fig. 1c). Similarly, connection to the minor axis of a W-shape column can benefit from the use of an extended shear tab.

The potential failure modes of unstiffened extended shear tab connections are summarized in the 15^{h} Edition of the AISC Steel Construction Manual [1]. The plate thickness and the weld throat are proportioned to develop plate yielding prior to bolt shear and weld tearing such that a stable behaviour can be achieved for the imposed loading. The 15th Edition of the AISC Steel Construction Manual [1] uses the rectangular plate buckling model [2, 3] to account for flexural buckling of the shear plate, while the 14th Edition of the AISC Steel Construction Manual [4] implements equations corresponding to the flexural buckling resistance of a doubly coped beam [5, 6].

The AISC design method [1] was originally developed for unstiffened extended shear tabs connected to rigid supports. The same method was further applied to unstiffened extended shear tabs connected to flexible supports by considering the out-of-plane deformation of the supporting element's web (either girder or column) as a serviceability issue for the supported beam [7]. The AISC design method was not originally developed for use with the partial-depth or full-depth stiffened extended shear tab. The shear tab in this case, may impose higher rotational demands to the supporting member (girder or column), which are typically not considered in frame analysis. This raises concern about the desirability of using stiffened extended shear tabs [8]. Nonetheless, practicing structural engineers do use stiffened extended shear tabs, typically, when an increase in the thickness of the shear plate is not a reasonable option to address the need to stabilize either the beam or the shear plate itself. The stiffened detail may be chosen because an upper limit is placed on the thickness of the shear plate to ensure its yielding prior to shear fracture of the bolts. Hence, an increase in thickness of the shear plate to improve its stability may not be permitted.

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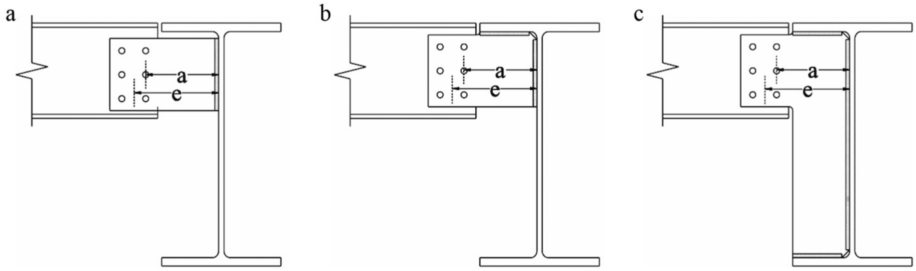

Fig. 1. Extended beam-to-girder shear tab connections: (a) partial-depth unstiffened, (b) partial-depth stiffened, (c) full-depth stiffened.

Further, specific to a beam-to-column connection, the column may also need continuity plates if there exists a fully restrained beam-to-column moment connection in the perpendicular direction. This allows for the possibility of attaching the extended shear tab to these plates as a lateral stability bracing. As well, even when continuity plates are not required, horizontal stabilizer plates may be added to laterally support the extended shear tab attached in the minor direction of a W-shape column. Moreover, if the supporting members are part of the primary lateral load-resisting system, their behaviour under gravity and lateral loads may be adversely affected by a potential out-of-plane deformation of the respective columns and/or girders. This may be particularly concerning when deep members are utilized in the lateral load resisting system [9]. Stiffened shear tab connections may also be chosen for this reason. Given these situations, in which extended shear tabs are stiffened, there exists the need to better understand their behaviour under load, and ultimately to ascertain whether existing design methods are appropriate. As a first step, the design method found in the AISC Manual [1] can be utilized to identify the potential failure modes of these shear tab connections.

In the design of extended shear tabs the current AISC Manual [1] suggests the inflection point to be located at the face of the supporting member, i.e. the girder web in this case (Fig. 2a). The design shear force and flexural moment for the bolt group (Figs. 2b and 3a) are the shear force at the beam end (R) and the resultant eccentric moment ($M = R \times e$), respectively. Furthermore, the vertical weld line, which connects the shear plate to the girder web (or the column web as shown in Fig. 3a), is designed to resist the shear force (R) alone. The horizontal weld lines, that connect the shear plate to the girder flanges (the stabilizer plates in Fig. 3a), are not considered as load carrying welds; as such, they are detailed having a minimum size. Of note, Figs. 2b and 3a show the symmetric configuration where the centreline of the supported beams is located midway between the girder flanges (the two stabilizer plates in Fig. 3a). This configuration may not be applicable if a supported beam is connected to a deeper supporting girder (Fig. 2c). Further, the symmetric configuration may not be applicable in the presence of continuity plates of a fully restrained moment connection joining a deeper beam to the column in the orthogonal direction (Fig. 3b).

For a girder or column, which supports a beam on both sides (Figs. 2d and 3c), each connection is designed for its corresponding shear force (R_R and R_L) and a portion of the net flexural moment ($M_R - M_L = R_R \times e_R - R_L \times e_L$) determined based on the engineer's judgement [1]. For the design of other connection elements, i.e. the shear plate and stabilizer plates, the current AISC Manual gives no explicit recommendations.

It is often the case that the design procedure of stiffened beam-to-girder shear tabs follows that of the unstiffened ones; the bolt group and the gross section of the plate are designed for the connection shear force (R) and the resultant eccentric bending moment ($R \times e$ and $R \times a$, respectively). This leads to either bolt shear fracture or yielding of the extended portion of the shear plate as the governing failure mode of the stiffened shear tab connection if the current AISC design approach [1] is followed. However, this is not consistent with the observed behaviour of such connections from laboratory tests [10–12].

Findings from past experimental and finite element studies [10–13] reveal that bolt shear fracture is not deemed to be critical in the context of the connection configurations that were evaluated. Plate buckling is the governing failure mode for stiffened full-depth configurations of either beam-to-girder [10] or beam-to-column shear tab connections [11, 12]. Notably, in stiffened extended beam-to-girder shear tabs with a partial-depth shear plate, shear plate yielding and twisting were the governing failure modes [10, 13]. Although the girder web mechanism was evident, it was a secondary failure mode that mostly occurred in deep connections, i.e. shear tab connections with a single vertical line of six or more bolts [10, 13].

In order to improve the current design provisions for full-depth stiffened extended shear tabs, Fortney and Thornton [14] recommended that the distance between the bolt line and the toe of a stabilizer plate should be used as the bolt group eccentricity for the design of extended shear tabs with stabilizer plates. Neither published laboratory tests nor finite element analyses were provided to fully explain this recommendation. Although the design calculations based on the aforementioned eccentricity result in a higher prediction for the bolt shear strength, they still overestimate the shear plate buckling strength, which is the governing failure mode observed in laboratory tests [10–12].

The test results of extended beam-to-girder shear tabs are limited to a few configurations with a single vertical row of bolts, although shear tabs with multiple bolt lines are common in current steel construction practice. Multiple bolt lines may decrease the shear plate buckling strength because the shear plate is loaded farther from its support, the weld line. Furthermore, most of the experimental studies on stiffened beam-to-column shear tabs [12] were limited to the configuration similar to that shown in Fig. 3a. Nevertheless, this configuration would need

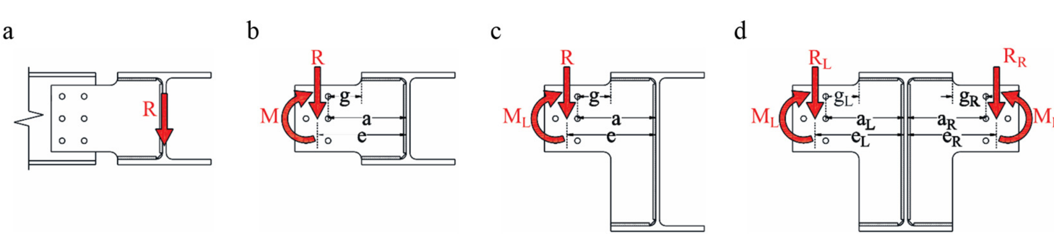

Fig. 2. Full-depth stiffened extended beam-to-girder shear tab: (a) location of inflection point, (b) single-sided (the beam and girder have the same depth), (c) single-sided, (d) double-sided.