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Numerical study of unstiffened extended shear tab connections

Ahmad Abou-zidan, Yi Liu *

Department of Civil and Resource Engineering, Dalhousie University, Halifax, NS, Canada

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ABSTRACT

This paper presents the results of a numerical study on the behavior and strength of unstiffened extended shear tab connections using finite element modeling. The model was verified with the experimental results reported in the literature. The parameters studied included the web slenderness ratio of supporting member, distance between the center bolt line and weld line, plate thickness, number of bolts, double-row of bolts, and beam lateral restraint. Results were analyzed to determine the effects of these parameters on the behavior and bolt shear strength and also used to assess the effectiveness of the AISC manual 2011 design procedure. It shows that the AISC design procedure for bolt shear fracture was overly conservative for extended shear tab connections with number of bolts less than 6 in a single row configuration. For connections with 6 bolts or higher or double-row bolts, the AISC design procedure provided reasonably good estimate. When the AISC design procedure was used but in combination with the finite element determined effective eccentricity, the estimate on bolt shear strength was markedly improved, especially for connections with less than 6 bolts in a single row configuration.

1. Introduction

Shear tab connections, also known as single plate connections, consist of a single rectangular steel plate commonly bolt-connected to the supported beam web and weld-connected to the supporting girder or column. The steel plate is often shop-welded to the supporting member and field bolted to the supported beam. Compared with traditional double-angle connections, single plate connections use less material and enable a safer and faster field erection. Hence it has become a popular alternative for light to moderate end shear connections. The connection detail leads to a distance between the weld and the centroid of the bolt group. This distance is commonly referred to as distance “a”. The research on shear tab connections dated back in the 1960s. Earlier research focused on connections where “a” is relatively small [1–6] and this type of shear tab connections is referred to as conventional shear tab connections. The results have shown that shear tab connections can develop some moment due to the inherent stiffness of the plate, and this moment is equivalent to an eccentric load on the bolts and welds. The magnitude of the eccentricity is based on the location of the point of inflection of the beam. As the load on the beam increases and the shear tab begins to yield, the inflection point moves toward the support and thus the magnitude of the eccentricity changes accordingly. The distance between the point of inflection in the beam to the bolt line at failure is defined as the effective bolt eccentricity, e_b, as seen in Fig. 1. The design of bolts in shear tab connections needs to consider the shear from the supported beam and a moment equal to the product of the shear and the effective bolt eccentricity [7]. The aforementioned studies also showed that the end moment developed in the connection depended on many factors such as the thickness of the plate; the number, size and arrangement of bolts; and the flexibility of the supporting member.

In recent shear tab application, shear tabs with long distance “a” have been used when it is desirable to extend the bolt line outside the supporting member flange, for example, in the beam-to-column web connections. Extending the shear tab beyond the supporting member flange eliminates the need to cope the supported beam. Additionally, the extension eases erection, as more space is available to fit bolt wrenches for fastening bolts, which in turn leads to reduction in costs. To distinguish from conventional shear tab connections, this type of shear tab connections is then referred to as extended shear tab connections. In the 2000s the research in extended shear tab connections began to attract attention. Some studies [8–10] have shown that factors influential for conventional shear tab (CST) connection also affected the behavior of extended shear tab (EST) connection. But the longer shear tabs in extended shear tab connections may result in different effective eccentricity and failure modes that were not observed in the conventional shear tab connections, for example, the twisting of the shear plate. Compared with the CST connections, the research on the EST connections has been limited and experimental studies and the test results are scarce in the reported literature.

For design practice in North America, the AISC manual 2011 [11] categorizes the shear tab connection as conventional when distance “a” is less than 90 mm and extended when “a” is greater than 90 mm. Unlike CST connections, the EST connection design does not place limitations on the distance from the weld line to the bolt line, the number of

* Corresponding author at: 1360 Barrington St, Halifax, NS, Canada. Tel.: +1 902 494 1509; fax: +1 902 494 3108.
E-mail address: yi.liu@dal.ca (Y. Liu).

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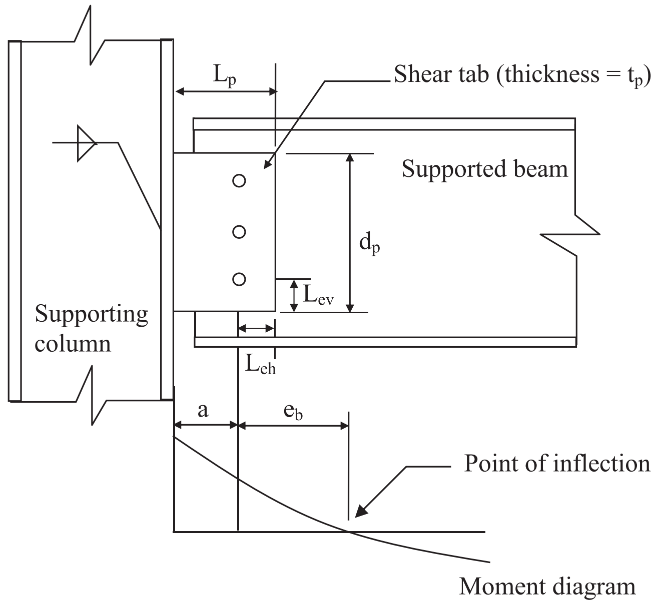

Fig. 1. Extended shear tab connection details and symbols.

rows or the number of bolts used in the connection. It states that the bolt eccentricity is to be taken equal to the distance from the weld line to the bolt line (distance "a"). The design moment of a bolt group, by means of using effective eccentricity, was developed by Crawford and Kulak [12,13]. However, this procedure combined with the AISC defined bolt eccentricity has shown to lead to conservative design [14–16]. To prevent the twisting of the plate, the beam is laterally braced at the connection location in the AISC procedure. The effect of the lack of lateral restraint is not considered. In view of this, more information on the behavior and strength of extended shear tab connections as affected by influential parameters is needed.

A study was hence motivated to investigate the behavior and strength of extended shear tab connections. A finite element modeling technique was used in this study. The model was verified using the test results available in the literature. A parametric study using the verified model was subsequently conducted to study the effects of several parameters on the behavior and strength of the connections. Lastly, the accuracy of procedure suggested in the AISC manual 2011 on the design of extended shear tab connections was examined.

2. Finite model development

2.1. Element description

The finite element model was developed using the commercial software ANSYS 13.0 [17]. The model consisted of a beam framing into the web of a supporting column with a single plate shear connection. All elements of the connections including the beam, column, shear tab plate and bolts were modeled using the ANSYS element SOLID185. The SOLID185 is a three-dimensional eight-node structural solid with three translational degrees of freedom per node, suitable for modeling large rotation and large strain nonlinearities. Targe170 and Conta174 are 4-node quadrilateral, surface to surface contact elements which overlay solid elements to simulate the physical transfer of force through contact and friction between the two bodies. Contact elements were used on the adjacent surfaces of the shear tab and beam web, the bolt shank and shear tab holes, the bolt shank and web holes, the bolt head and shear tab, and the nut and beam web surfaces. For normal contact stiffness factor, a value of 1.0 is recommended by ANSYS for general bulk deformation-dominated problems. To reflect the snug tightened bolt condition, a pretension force was applied normal to the cross-section of the bolt shank. The PRETS179 element was used to define the pretension section within the meshed bolt. This element has one translational degree of freedom.

2.2. Mesh refinement

To effectively simulate the localized behavior of the shear tab connection, the mesh density study was conducted with three mesh sizes having a maximum element width of 9.5, 4.8 and 2.4 mm, respectively for the shear tab. Connection capacity versus shear tab rotation curves using these three element widths are plotted in Fig. 2 and it shows that all three mesh sizes yielded practically the same ultimate connection capacity but the load versus beam web rotation responses with sizes 4.8 and 2.4 mm were smoother with expected ductility. The 4.8 mm mesh size was then chosen for all simulations to have a balance on the computational time and the acceptable accuracy level. The bolt, bolt head and nut were modeled as one solid body. The mesh size of the bolt was kept to be the same as the tab to ensure smooth contact between the bolt shank and the holes in both the shear tab and the beam web. A 2 mm gap was provided between the bolt shank and the holes to simulate the geometry of punched bolt holes.

Two mesh sizes were implemented for the beam. In the vicinity of the bolts a fine mesh was used to capture the high stress concentrations with element dimensions of 9.5 mm by 10 mm. Away from the bolt line, the mesh dimension increased to 17 mm by 19 mm. The supporting column had the same mesh configuration as the shear tab to enable the nodes along the end of the shear tab to be coupled with the nodes of the supporting member. Fig. 3 illustrates the fully assembled EST connection mesh detail at the beam-to-column web and the meshing for the bolt.

The column was assumed to be fixed at two ends where all nodes were restrained against translations in the x, y and z-directions. The full beam length was modeled and the support at the far end was modeled as a pin support.

2.3. Material properties

An elasto-perfect plastic material model was used for the beam, column and the shear tab. The modulus of elasticity, E, and the yield strength were taken as standard values for 350 W structural steel, 200 GPa and 350 MPa respectively. The Poisson ratio of 0.3 was assumed. As for the high strength bolts, the stress versus strain relationship for high strength A325 bolts experimentally determined by Rahman et al. [18] was adopted and shown in Fig. 4 where a yield and an ultimate strength of 620 and 945 MPa were assumed.

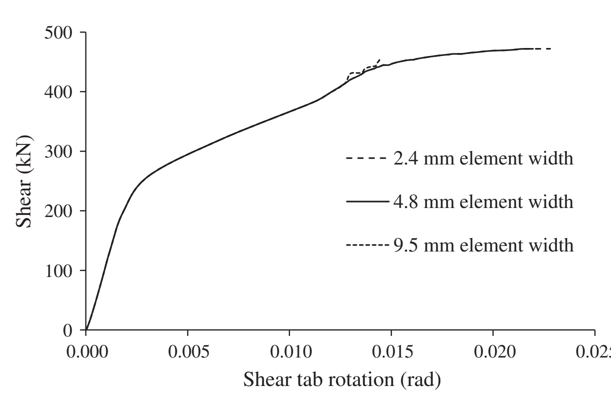

Fig. 2. Mesh density study on connection behavior.