Leadership in Sustainable Infrastructure

Leadership en Infrastructures Durables

Vancouver, Canada

May 31 – June 3, 2017/ *Mai 31 – Juin 3, 2017*

STABILITY OF EXTENDED SHEAR TAB CONNECTIONS

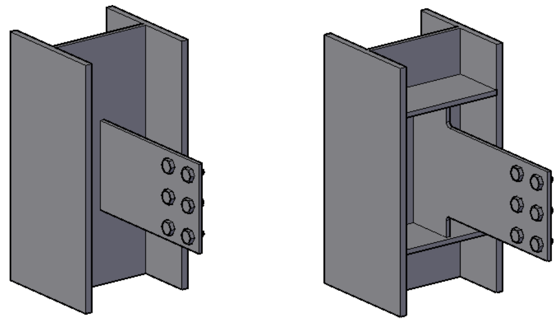
Buffam, Victoria[1,4], Driver, Robert[2], and Callele, Logan[3]
[1,2] Department of Civil and Environmental Engineering, University of Alberta, Edmonton, Canada
[3] Waiward Steel LP., Edmonton, Canada
[4] buffam@ualberta.ca

Abstract: Extended shear tab connections are efficient for both fabrication and erection where a beam would otherwise need to be coped to clear the flanges of the supporting member, and are therefore used extensively in industry. Stability issues can arise as the plate becomes longer and more slender, as may be required in skewed connections or other complex geometries. Commonly, designers resolve the concern of stability failure by increasing the plate thickness or adding stiffeners to the connection, which increases the required material and cost. Previous studies at the University of Alberta have experimentally investigated the behaviour of extended shear tabs and found that nearly all tested specimens reached their full cross-sectional plastic capacity without buckling. Therefore, this research aims to determine when stability of the extended shear tab governs the behaviour and capacity of the connection as opposed to strength, considering the interaction of shear and bending stresses. A parametric study using finite element simulations is being conducted in which the length of the extended shear tab is increased until instability of the plate becomes the governing failure mode. Preliminary results suggest that longer lengths can be used for shallow connections before stability occurs prior to achieving the full plastic strength of the cross-section, although deflections can be large at failure. Increased understanding of the behaviour of extended shear tabs derived from this research will be used to improve the associated design procedures by defining when instability becomes a potential design limit state in order to reduce design time, uncertainty, and unnecessary fabrication costs.

1 INTRODUCTION

Shear tab connections are commonly used in steel building construction to transfer gravity loads from beams to supporting members. The connection consists of a vertical plate welded to a column or girder and bolted to the supported beam; as such, they are efficient for both fabrication and erection and are extensively used in industry. Extended shear tabs have a similar configuration, but with an increased plate length in order to clear the flanges of the supporting member and eliminate the need for coping the beam. This reduces fabrication time and costs.

The behaviour of extended shear tabs differs from those with a conventional configuration, as the increased length of the plate introduces potential stability concerns. Since this behaviour is not well understood, designers have typically adopted a conservative approach where the plate thickness is increased or stiffeners are added to prevent plate buckling, eliminating the economic advantage of the connection. Figure 1 illustrates both configurations of the extended shear tab. The Steel Construction Manual (AISC 2011) includes a provision for stability where the double-coped beam design procedure is used to check for buckling in the plate. Additionally, the requirement for stiffeners is based on the lateral stability of double-coped beams, as proposed by Thornton and Fortney (2011).



(a) Unstiffened connection (b) Stiffened connection

Figure 1: Extended shear tab configurations

Recent research at the University of Alberta has aimed to investigate and improve the understanding of extended shear tab behaviour, particularly when the connections are subjected to simultaneous shear and axial force from the connected beam. Thomas et al. (2014) conducted full-scale experimental tests on both stiffened and unstiffened extended shear tab connections welded to a column web, similar to those depicted in Figure 1. The unstiffened connections' capacities were governed by weld or bolt rupture, whereas the capacities of those with stiffeners were limited by out-of-plane deformation. Extended shear tab connections with either rigid or semi-rigid supports were later investigated by Salem et al. (2016) through experimental tests and finite element simulations. Nearly all specimens developed their full cross-sectional plastic capacity prior to excessive out-of-plane deformations and failure. Additionally, it was determined that the rotational stiffness of the support has a significant effect on the behaviour of extended shear tabs; a higher stiffness corresponds to increased connection capacity.

Most research regarding extended shear tab connections has focused on the strength of the connection, the influence of a stiff or flexible boundary condition, or the addition of stiffeners. Limited research has addressed connection stability or plate length as an important geometric parameter. Abou-zidan and Liu (2015) conducted a numerical parametric study that varied the distance from the support to the first bolt line from 160 mm to 250 mm. The results showed a linear decrease in connection capacity as this distance is increased due to increased deformations and shear stresses in the bolts.

This paper outlines some results of the numerical part of a comprehensive study that focuses on the stability of extended shear tab connections. A parametric study has been conducted to determine when instability becomes the governing failure mode, as opposed to cross-sectional strength. Ultimately, this research aims to develop design procedures that address lateral stability for extended shear tab connections.

2 PARAMETRIC STUDY

2.1 Finite Element Modelling

A finite element model was developed using the structural analysis software ABAQUS. An extended shear tab model was created and validated by Salem et al. (2016) using results from three full-scale experimental tests conducted at the University of Alberta. This model was then modified in order to investigate stability of the connection.

Solid parts were used for all components, including the extended shear tab plate, bolts, and supported beam. The bolts are ASTM A325 with a 3/4-inch diameter and the beam was modelled to have a constant web thickness of 13.1 mm. The depth of the beam varied with the geometry of the extended shear tab and one such model is presented in Figure 2. A non-linear analysis was conducted using 8-node linear brick elements with reduced integration. Discrete rigid parts were used to create loading and support plates in order to distribute the loading and restraints evenly to the parts. Welding of the extended shear tab plate to the support was not modelled, but rather represented using a tie constraint. The general contact feature with a friction coefficient of 0.3 was used to simulate the interaction and slip between parts and allow for force transmission in the model.

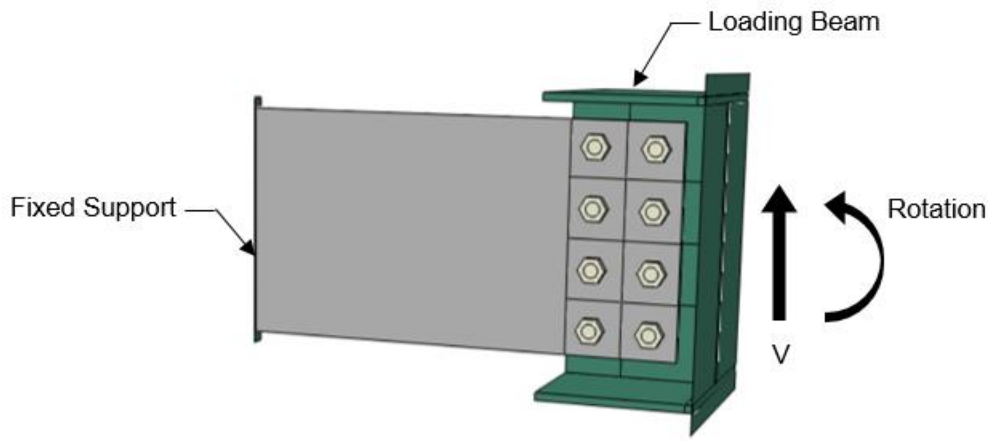


Figure 2: Model assembly in ABAQUS

Mesh size varied for the different parts of the model and was selected based on a balance between computational time and model behaviour. A mesh refinement study was undertaken in order to optimize the mesh density of the extended shear tab for the parametric study. Three different mesh sizes were investigated: 3 mm, 5 mm and 10 mm, representing the maximum dimension of the element. All models used three elements across the thickness of the plate to model bending deformations accurately. Figure 3 illustrates the relationship between mesh size and ultimate shear capacity of the three models. It was observed that decreasing the element size also decreases the shear load capacity, indicating that an improperly sized mesh can over-predict the capacity of this connection. The capacity obtained with the 3 mm and 5 mm meshes differ by less than one percent; therefore, an element width of 5 mm was selected for the remainder of the study. A larger mesh size was used for the beam, except around bolt holes in the web due to steep strain gradients at these locations. The bolts used a hex-dominated meshing scheme and an increased mesh density in the bolt shank.

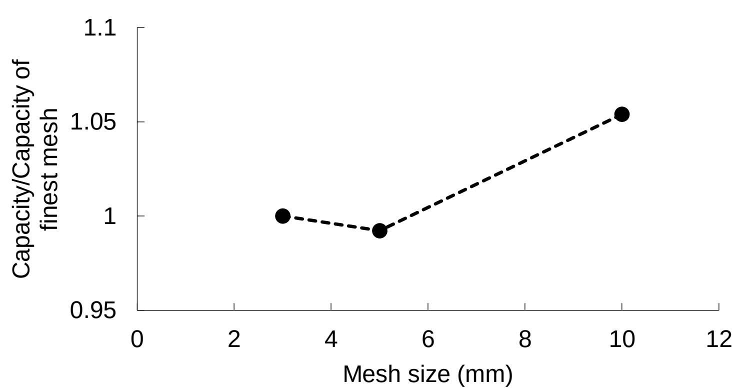


Figure 3: Mesh optimization results

2.1.1 Material Properties

Results from tension coupon tests previously completed at the University of Alberta were used to create a bilinear stress-strain curve, which was then used to model the material properties of the steel plate for the parametric study. A yield stress of 385 MPa was chosen, as it represents the probable strength of 350W steel. The slope of the strain hardening portion was chosen such that it was approximately parallel to the true stress-strain distribution of the tension coupons. This resulted in a slope of approximately 0.5% of the modulus of elasticity, which was taken as 200 GPa for this study (Salem et al. 2016). ASTM A325 bolts were also modelled using the stress-strain relationship developed experimentally by Salem et al. (2016).

2.1.2 Boundary Conditions and Loading

The parametric study was conducted assuming a fixed boundary condition for the extended shear tab (i.e., the support was restrained against translation and rotation in all directions). The flanges of the supported beam were laterally braced to examine localized failure in the extended shear tab.

Loading was applied in three separate steps. The first step applied a pretension force of 15 kN to the bolts in order to simulate a snug-tight condition. Next, a rotation of 0.03 radians was applied to the supported beam, which represents the upper limit on rotational demand of shear connections when the plastic moment is reached in the beam (Thomas et al. 2014). The bolt tensile force and the beam rotation were then held constant while the shear force was applied to the connection as a displacement at the end of the supported beam. Displacement control was used as it is more stable than force-controlled loading and is also able to capture the post-peak behaviour of the structure. The displacement was increased until the load in the connection began to decrease, signifying a failure of the extended shear tab.

2.2 Parameters

A parametric study using ABAQUS has been completed in order to investigate the behaviour of extended shear tabs and the effect of certain geometric parameters. The thickness, depth, and length of the plate can vary based on the demands of the connection, and this study aimed to determine how these parameters affect connection stability. Buckling resistance of a connection plate element is primarily based on the weak-axis moment of inertia of the plate (Dowswell 2016). Therefore, the depth and thickness of the plate are of primary importance and the values selected are presented in Table 1. The number in brackets corresponds to the number of horizontal bolt lines. No axial load was applied to the connection in this phase of the study in order to focus on the understanding of stability limits when these connections are loaded in shear only.

Table 1: Parametric study matrix

Depth (mm)	Thickness (mm)
150 (2B)	6.35
230 (3B)	9.50
310 (4B)	12.7

The objective of the parametric study was to determine the shortest length of plate that leads to a stability failure of this connection type. This was achieved by varying the plate length and analyzing the results to determine whether the plate reached its plastic cross-sectional capacity at the locations shown in Figure 4. Equation 1 (Neal 1961) was evaluated at both the support and the first bolt line, and used to determine whether full plasticity had occurred. This equation considers the interaction of bending and shear stresses in the connection plate. The original equation has been simplified, as no axial load is applied to the connection. In order to focus on a plate failure, it was assumed that both the weld at the support and the bolts were adequately designed.

$$[1]\ \frac{M_f}{M_p} + \left(\frac{V_f}{V_p}\right)^4 \leq 1.0$$

where:

M_f = applied moment on a particular cross-section of the plate

M_p = plastic moment resistance of the plate cross-section = $F_y \times Z_{Plate}$

V_f = applied shear

V_p = shear yield strength = $A_v \times 0.66 \times F_y$, where A_v is the plate cross-sectional area

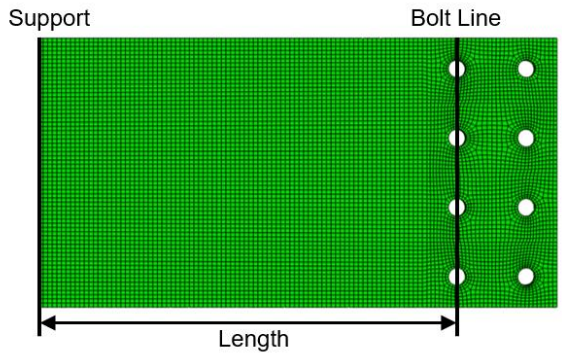


Figure 4: Locations of analyzed section cuts

For each model within the matrix, the total plate length was varied in 25 mm increments until a value close to 1.0 was obtained from Eq. 1 at either the support or bolt line. This was taken as the stability limit for the given connection geometry. Due to large computational processing times and the incertitude of finite element analysis, a length was accepted if the value was within 3% of the target.

The model ID, used later in this paper, identifies the specimens based on their varying geometric parameters. The first value represents the depth, specified by number of horizontal bolt lines, while the second number indicates the plate thickness rounded to the nearest whole number. The last number is the length of the extended shear tab, measured from the support to the first vertical bolt line, as shown in Figure 4.

3 RESULTS AND DISCUSSION

3.1 Connection Behaviour

It has generally been assumed in design that the extended shear tab connection transfers no moment to the supporting member, and therefore the moment would increase linearly from the support to the bolt line. However, previous tests have shown that this is inaccurate, and significant moment can be developed at the support. The magnitude of this moment is, however, dependent on the rotational stiffness of the support (Salem et al. 2016). In this study the support was fixed, creating a significant bending moment at this location. As a result, gross section yielding occurs and the first plastic hinge is formed at the support. The plateau observed in Figure 5 illustrates this softening of the system as the plastic hinge is formed and also corresponds to the onset of out-of-plane deformations. Based on the form of Eq. 1, bending stresses have a greater effect on section plasticity than shear stresses. Variations of the bending moments at both the support and bolt line were also plotted to analyze connection behaviour and are shown in Figure 6. The bending moments were normalized by dividing the value obtained from ABAQUS by the plastic moment of either the gross or net section, as appropriate. This graph further supports the contention that the plastic moment was reached at the support early in the analysis, at a low vertical displacement.

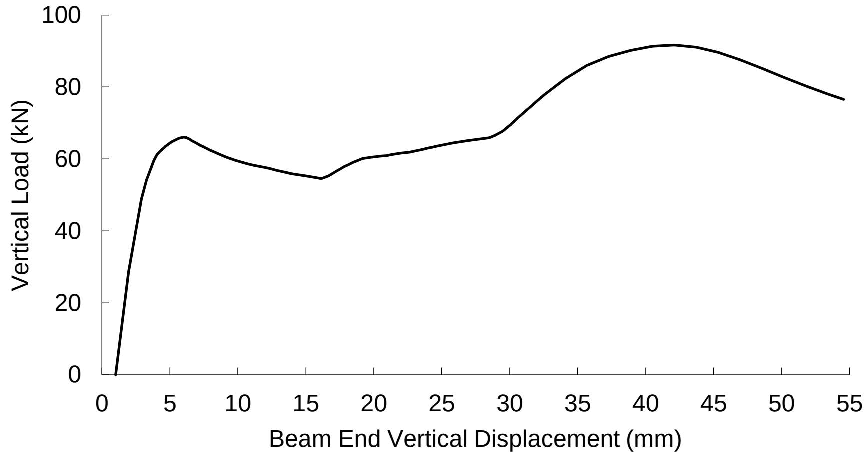


Figure 5: Load-displacement graph for 3B-10-685 model

After the reduction in stiffness caused by the formation of the plastic hinge at the support, the load is shed towards the bolt line and the plate goes into double curvature about the weak axis. This corresponds to an increase in connection strength, and therefore an increase on the load–displacement curve, as shown in Figure 5. However, due to the length of the extended shear tab, an out-of-plane deformation failure occurs prior to the formation of a second plastic hinge at the bolt line. As such, the extended shear tab reached a value of Neal's equation equal to 1.0 at the support and not the bolt line and therefore, the stability limit is governed by plasticity at the support. It is important to note that this occurs at the first peak in the load-displacement curve presented in Figure 5, rather than at the maximum ultimate load.

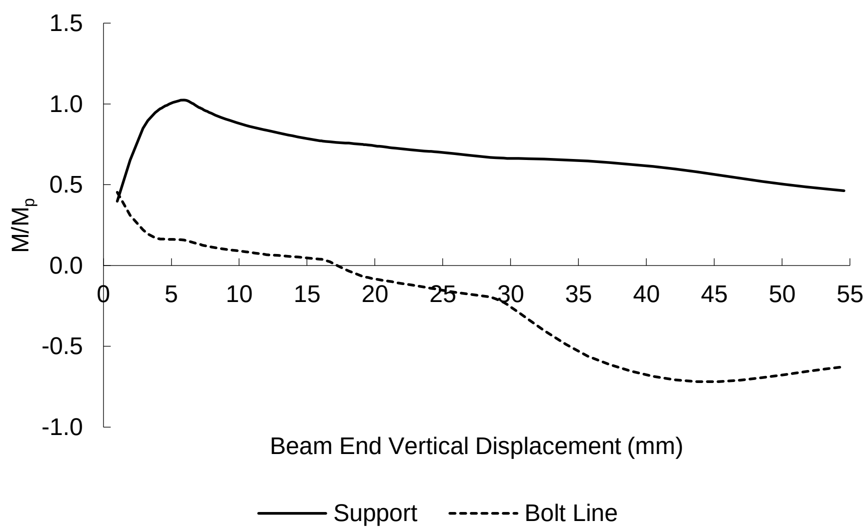


Figure 6: Bending moment variation for 3B-10-685 model

The purpose of this study was to investigate the effect of geometric parameters on the stability of extended shear tab connections. Therefore, the bolts and welds were assumed to be adequately designed and these failure modes were not considered. Out-of-plane deformation of the plate was observed early in the analysis and began at the top of the plate closest to the support. In some cases, this movement was caused by the applied rotation and then as the shear displacement increased, so did the out-of-plane deformation. At the peak load, the bottom of the plate began to deform out-of-plane in the opposite direction of the top and increased until the analysis was complete. This results in double curvature of the plate about the weak axis, as shown in the deformed shape in Figure 7.

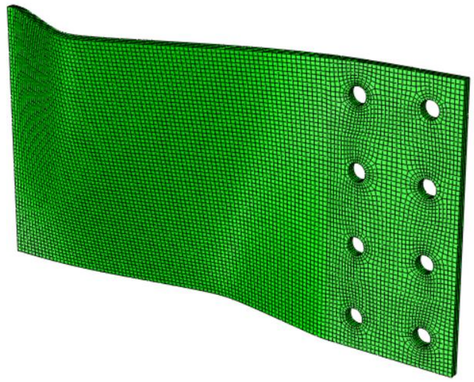


Figure 7: Deformed shape of 4B-10-485 model

3.2 Stability Limit and Capacity

Table 2 summarizes some key results of the parametric study. The length at the stability limit corresponds to the length of the extended shear tab at which the combination of stresses resulted in a value of Eq. 1 within the above-mentioned range. The length recorded is the length from the support to the first bolt line of the extended shear tab, as shown in Figure 4. In all cases, the stability limit is governed by plasticity at the support and not the bolt line. The shear capacity is taken as the peak load observed in the load-displacement response of each model.

Table 2: Parametric study results

Depth (mm)	Thickness (mm)	Length at Stability Limit (mm)	Peak Vertical Load (kN)
150 (2B)	6.35	535	30.4
	9.5	1385	20.1
	12.7	>2385*	15.7
230 (3B)	6.35	285	148.3
	9.5	685	91.7
	12.7	1435	57.1
310 (4B)	6.35	185	354.5
	9.5	485	232.7
	12.7	985	146.3

*The length at stability limit was not obtained for this model as it exceeds the practical range for the connection. However, the length is greater than the presented value.

The length of the extended shear tab at the stability limit is heavily dependent on both the depth and thickness of the plate. Plate length increases with an increased thickness, as expected. As previously stated, the failure mode in this study is out-of-plane deformation. Therefore, increasing the plate thickness has a large impact on the resistance of the connection. This is also demonstrated by Figure 8, which shows the approximately linear relationship between plate length and thickness at the stability limit. There is little difference in the plate length obtained for the three depths at a thickness of 6.35 mm; however, the spread increases with plate thickness. For thicker plates, the connection has an increased resistance to out-of-plane deformations and plate depth has a greater influence on the stability limit.

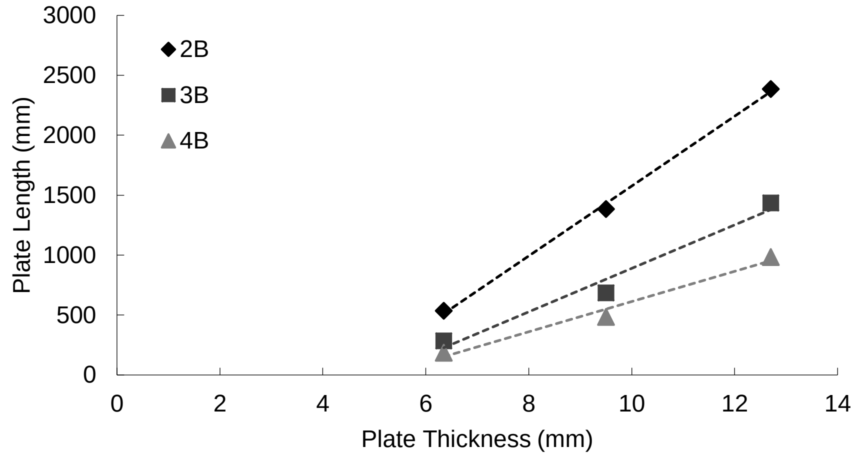


Figure 8: Effect of plate thickness on stability limit

Similarly, Table 2 shows that increasing the connection depth results in a decreased length at the stability limit. This relationship is represented in Figure 9; where a steeper slope is attributed to the thicker extended shear tab plates. There is a larger spread in the data at a connection depth of 150 mm, indicating that the stability of shallow connections is greatly affected by plate thickness. The data converges as the connection depth increases, indicating that depth begins to govern the behaviour of the connection.

The shear capacity of the connection also increases with increasing depth and shorter plate length. This is because less load is required for a shallow, long plate to reach a value of 1.0 using Eq. 2. The reason for this is two-fold: longer plates have a greater moment arm, creating a larger applied moment at the support for the same vertical load, and the plastic moment of the plate is smaller for a shallower section. The contribution of the shear term in Neal's equation is small, since plasticity occurs at the support very early in the analysis and at low levels of vertical displacement.

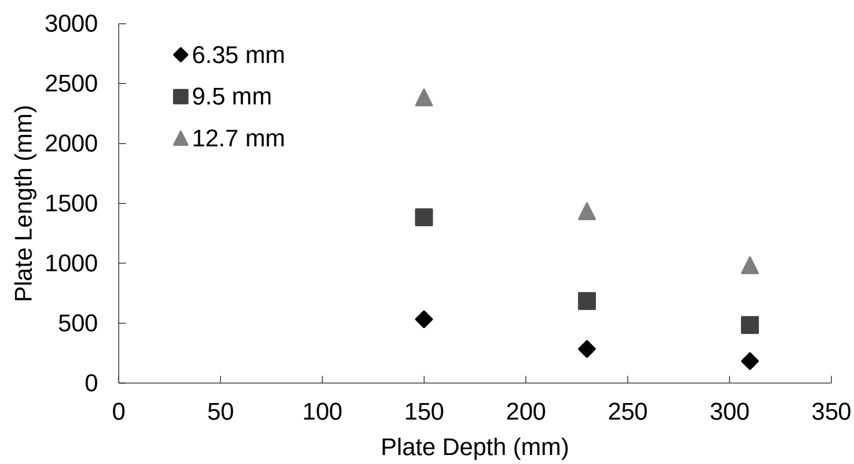


Figure 9: Effect of plate depth on stability limit

3.3 Beam Longitudinal Displacement

No restraint was imposed on the horizontal movement of the beam towards the support. This movement occurs as a result of the applied rotation and shear. This displacement was investigated in order to ensure that the model accurately represents a real-world application, where the supported beam is restrained due to its connection at the far end. The horizontal displacement was measured at the peak load, just prior to connection failure. A maximum displacement of 13.7 mm was observed with the 3B-13-1435 model, with a mean displacement over all models of 10.8 mm. This was deemed acceptable, as allowing the horizontal movement of the connection only exacerbates buckling failure of the extended shear tab.

3.4 Slenderness Parameter

Section F11 of the AISC Specification (AISC 2016) provides requirements for the flexural strength and stability of rectangular bars. A slenderness parameter, presented in Eq. 2, is used to determine whether lateral–torsional buckling will govern the behaviour. The equation uses the length between points braced against lateral displacement (L_b), which was taken as the length from the support to the first bolt line for this study. As previously stated, the behaviour of extended shear tab connections at the stability limit is governed by weak-axis buckling of the plate. As such, the data obtained in this study was analyzed using the AISC slenderness parameter to determine whether it accurately represents the behaviour of extended shear tab connections. Figure 10 plots this parameter with the shear capacity obtained from each model.

$$[2]\quad \lambda = \frac{L_b d}{t^2}$$

The data from this study is well represented by the AISC slenderness parameter because the failure mechanism of extended shear tabs at the stability limit is similar to lateral–torsional buckling, even though this may not be the case for shorter shear tabs. A curve was fit to the data using a power function and the majority of the model points are clustered around this curve. It should be noted that the 3B-6-285 point is aligned vertically with the 4B-13-985 point, as they have similar shear capacities, because the slenderness parameter does not differentiate between depth and length. Further refinement of the slenderness parameter is required for it to be applied in a design procedure for extended shear tab connections.

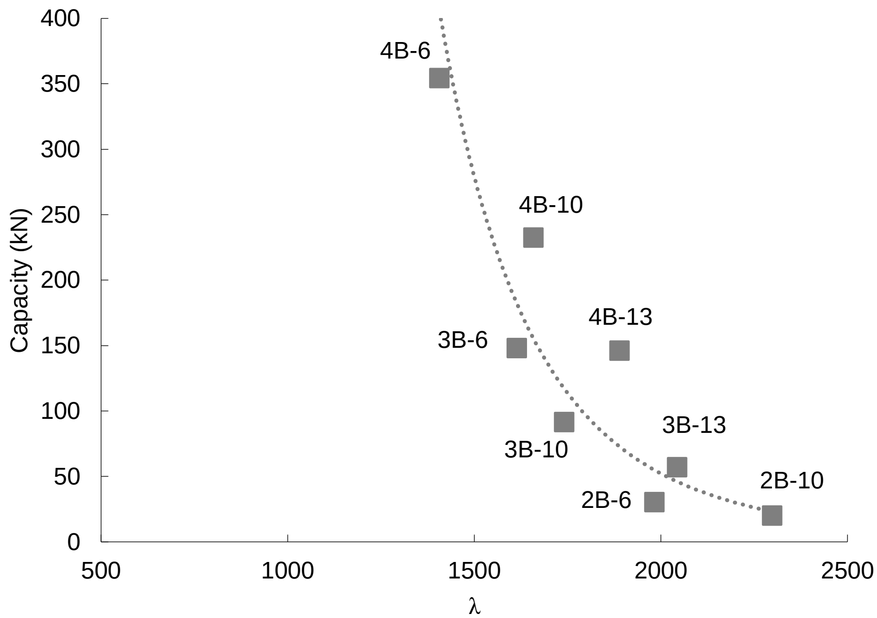


Figure 10: AISC lateral–torsional buckling slenderness parameter applied to model results

4 SUMMARY AND CONCLUSIONS

Extended shear tab connections are extensively used in industry; however, lack of research regarding the stability of this connection has led to the routine addition of costly stiffeners. A parametric study was conducted in ABAQUS finite element software to study the effect of certain geometric parameters on the stability of extended shear tabs. The model was developed and verified using experimental tests previously conducted at the University of Alberta. A comprehensive account of the tests and validation process can be found in Salem et al. (2016). The parameters chosen were thickness, depth, and length of the plate, with each connection consisting of two vertical bolt lines. The length of the plate was increased until a stability failure occurred prior to the development of cross-sectional plasticity in the plate. Plasticity was determined using Neal's equation to account for the interaction of bending and shear stresses.

The behaviour of the connection was analyzed and the first plastic hinge developed at the fixed support. As the shear force was increased, load was shed towards the bolt lines. Due to the length of the plates in this study, an out-of-plane deformation failure occurred prior to plasticity at the bolt line. This results in double curvature in the plate as both the top and bottom deform out-of-plane in opposite directions. Stability of extended shear tabs is dependent on plate thickness and depth; however, the lengths required to cause a stability failure are quite long, and in some cases longer than any practical application. Additionally, the shear capacity of the connection decreased as the length increased. As plate thickness increases, so too does the length of the plate at the stability limit. Conversely, deeper plates fail at shorter lengths than a shallow plate of the same thickness. It has also been observed that the stability limit lengths for thin (6.35 mm) plates are less sensitive to connection depth than the two thicker plates analyzed. The lateral-torsional buckling slenderness parameter given in the AISC Specification was investigated in relation to the data obtained in this study. The parameter fits the data relatively well; however, it is unable to distinguish between the influences of plate length and plate depth. Results from this study will be supplemented by an experimental testing program to help define the behaviour of extended shear tab connections with respect to stability.

Acknowledgements

This research was funded by the Natural Sciences and Engineering Research Council and the Canadian Institute of Steel Construction. Review of an advanced version of the manuscript by Graeme Johnston of Waiward Steel LP is greatly appreciated.

References

Abou-zidan, A. and Liu, Y. 2015. Numerical Study of Unstiffened Extended Shear Tab Connections. *Journal of Constructional Steel Research,* **107**(1): 70-80.

American Institute of Steel Construction. 2011. *Steel Construction Manual.* 14th ed., AISC, Chicago, IL, USA.

American Institute of Steel Construction. 2016. *Specification for Structural Steel Buildings.* AISC, Chicago, IL, USA.

Dassault Systemes. 2014. *ABAQUS/CAE User's Guide v. 6.14*. Providence, RI, USA.

Dowswell, B. 2016. Stability of Rectangular Connection Elements. *Engineering Journal,* AISC, **53**(4): 171-202.

Neal, B.G. 1961. The effect of shear and normal forces on the fully plastic moment of a beam of rectangular cross section. *J. App. Mech.,* **28**(2): 269-274.

Salem, P., Driver, R.G., and Callele, L. 2016. Unified Design Criteria for Steel Cantilever Plate Connection Elements*. Rep No. 308*, University of Alberta, Edmonton, Alberta, Canada.

Thomas, Kristin, Robert G. Driver, Logan Callele, and Steven Oosterhof. 2014. Design and Behaviour of Extended Shear Tabs under Combined Loading*. Rep. No 305*, University of Alberta, Edmonton, Alberta, Canada.

Thornton, W.A., and Fortney, P.J. 2011. On the Need for Stiffeners for and the Effect of Lap Eccentricity on Extended Single-Plate Connections. *Engineering Journal,* AISC, **48**(2): 117-126.