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Characterization of Steel Joint Modeling Parameters for Progressive Collapse Stability Analysis

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Abstract
Progressive collapse occurs when the failure of a structural member due to abnormal loading results in instability of adjacent members, potentially leading to an extent of collapse that is disproportionate to the initial failure. From the 1960s to the present, standards for structural design have included general requirements intended to enhance structural integrity and improve structural stability under abnormal loading. However, even today they do not provide specific characteristics for accurately modeling steel joints when assessing the global stability behavior of the structure under the abnormal loading condition. Through this research program, a modeling approach is developed to simulate the response of the joints under a condition when a major load-carrying element is compromised. The model is intended to be simple to implement and suitable for a wide range of practical applications, in order to simulate the global stability behavior of the structure. The model is reliable for predicting both linear and nonlinear performance of the structure. A mechanical model that can predict connection response after column removal is validated using available test results. Through the use of this model, large multi-bay systems can be analyzed much more efficiently than the micro-modeling approaches used in previous studies. The model is used to study the influence of different factors such as connection geometry, span length, and number of bolts on the collapse resistance of the global system.

1. Introduction

Progressive collapse occurs when an unexpected localized failure of an individual structural member subjected to an abnormal loading condition results in instability of adjacent members, potentially leading to an extent of collapse that is disproportionate to the initial failure. From the 1960s to the present, standards for structural design have included general requirements intended to enhance structural integrity and improve structural stability under abnormal loading. For instance, the British code introduced integrity and structural robustness requirements after the partial collapse of a 22-story tower block, Ronan Point building, in 1968. Other codes added some provisions such as "tying force" or "notional member removal" to their regulations to enhance structural robustness. ASCE/SEI 7-10 Standard (ASCE 2010) provides guidance to ensure minimum structural integrity of all structural elements by using two different approaches:

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direct design, and indirect approach. In the direct method, the structure is designed to be capable of bridging over the removed element. On the other hand, the indirect method aims to satisfy minimum levels of integrity, continuity, and ductility of the structure.

Recently, the rate of research on progressive collapse has been intensified. Many studies have been conducted to address the behavior of steel connections under a column removal scenario (e.g., Thompson 2009; Main and Sadek 2012; Oosterhof and Driver 2012; Jamshidi and Driver 2012). Many studies have experimentally (Oosterhof and Driver 2014; Tan and Astaneh 2003) and numerically (Jamshidi and Driver 2014; Gong 2014; Khandelwal et al. 2008) investigated the response of the structure under progressive collapse.

However, even today available design and analysis methods do not provide specific characteristics for accurately modeling steel joints when assessing the global stability behavior of the structure under abnormal loading conditions. Through this study, a simplified modeling approach is developed to simulate the response of steel frame joints under a condition when a major load-carrying element is compromised. The model is reliable for predicting both linear and nonlinear performance of the structure. This paper focuses on the steel single-plate or "shear-tab" connection. A mechanical model that can predict connection response after column removal is validated using available test results. The model is used to study the influence of different factors such as connection geometry, number of bolts, and span length on the collapse resistance of the system.

2. Test Set-up and Specimen

To be able to propose a simplified mechanical model, it is necessary to have a clear insight about previous research work. An experimental test set-up at the University of Alberta (Oosterhof and Driver 2014) was employed to determine the response of steel shear connections under a progressive collapse scenario. Throughout this experimental process, three actuators were employed to apply different levels of moment, shear, and axial force to beam-to-column connections, independently. The magnitude, direction, and location of the forces applied to the beam were measured explicitly by load cells, clinometers, and cable transducers, respectively. Consequently, the moment, shear and axial forces were accurately monitored at the connection. Moreover, a digital image correlation system was implemented to monitor displacements and surface strains in the connection region.

The shear-tab connection specimens consisted of a single steel plate bolted to the beam web and welded to the column flange. Two different plate thicknesses (6.4 mm and 9.5 mm) and two connection depths (3 bolts and 5 bolts in a single vertical line) were tested. Moreover, weld sizes and bolt diameters were designed to be consistent with the plate thickness, and 19 and 22 mm (3/4 and 7/8 in.) diameter ASTM A325 bolts were installed without washers in standard holes for the 6.4 mm and 9.5 mm plate thicknesses, respectively (Oosterhof and Driver 2014). Also, different span lengths and load history arrangements were considered in the study. A pitch of 80 mm and an edge distance of 35 mm were considered for all shear-tab connections.

3. Experimental Test Results

Based on the experimental results, extensive local yielding occurred at bolt-bearing locations, with the average bearing deformation being 26.9 mm for the 6.4 mm specimens and 35.4 mm for the 9.5 mm specimens, before tear initiation. The main observed failure mechanism of the shear-tab specimens was bolt tear-out due to the transferred catenary force in the direction of the beam axis. In all cases, failure started from the extreme bolt with respect to the neutral axis, due to the nature of the applied load, and by increasing the load, failure progressed to each successive bolt.

In the previous experimental work (Oosterhof and Driver 2014), the maximum horizontal load was recorded right before the extreme bolt tear-out, and consequently by increasing the beam rotation, decreases occurred in a stepwise manner as a result of failure at successive bolt locations. Also, results indicated that axial stress is the dominant stress due to the catenary tension. On the other hand, moment was small enough to be neglected in column removal investigation.

As shown in Fig. 1, the load arrangement (distributed or concentrated load) does not have any significant effect either on the maximum horizontal load or the beam rotation at initial failure (θ_c). However, by increasing the span length, θ_c will be decreased, while the maximum horizontal load remains constant, as shown in Fig. 2. Moreover, the increase of maximum horizontal load for the thicker shear tab is far more significant than the observed increase of θ_c (see Fig. 3).

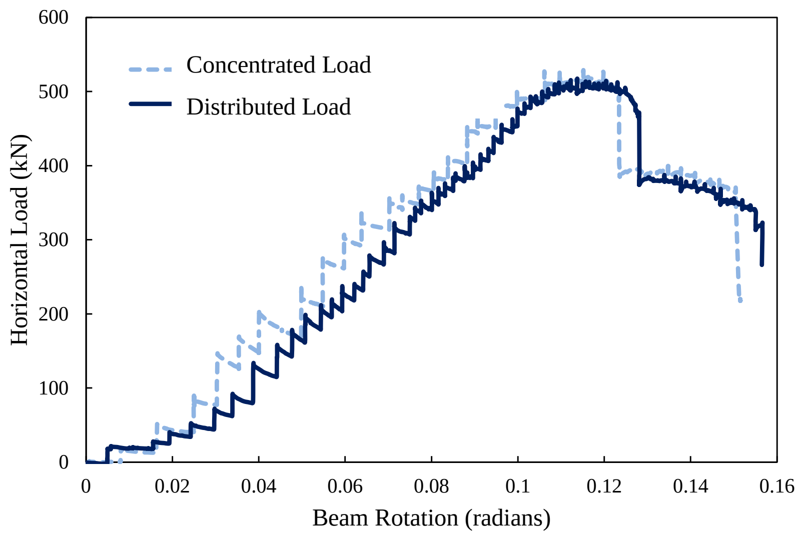

Figure 1: Effect of loading arrangement on load–rotation response of 9.5 mm shear tabs with 3 bolts

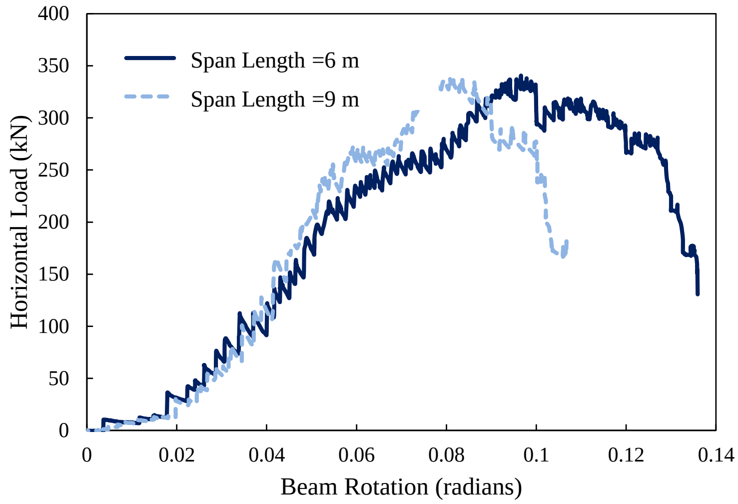

Figure 2: Effect of span length on load–rotation response of 6.4 mm shear tabs with 3 bolts

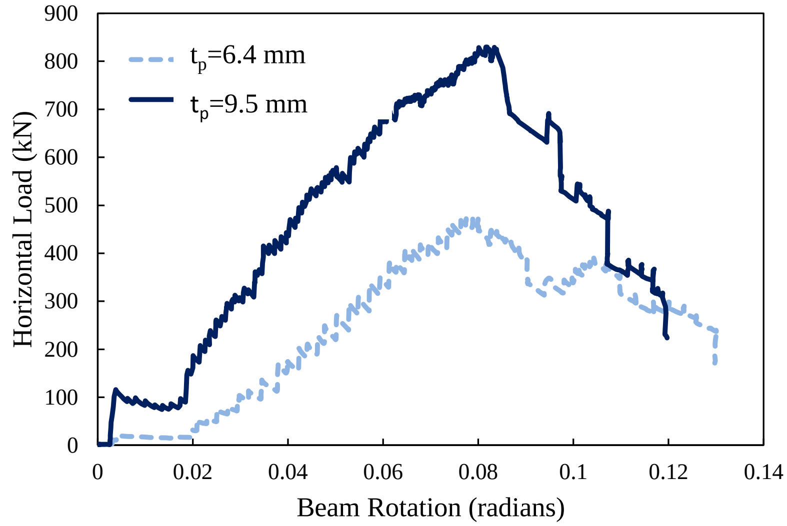

Figure 3: Effect of plate thickness on load–rotation response of shear tabs with 5 bolts

4. Mechanical Model

To be able to propose a simplified model for shear-tab connections under a column removal scenario, it is required to identify the ultimate rotation of the beam at complete bolt tear-out (θ_u). However, due to experimental restrictions, tests did not reach the stage of failure of the final bolt and θ_u therefore remained unknown. Therefore, it is of paramount interest to identify θ_u through a mechanical model in order to simulate the connection behavior through the full range of response.

There are several different models in the literature to predict the plastic deformation of shear tabs. In this work, a mechanical model based on bolt bearing capacity (Rex and Easterling 2003) was used to determine θ_u. As the first step, the accuracy of the model was validated with the experimental results of beam rotation at initial failure (θ_c). As a result, θ_u was identified using the validated model.

A single-bolt bearing deformation model for bolts loaded toward the free edge of a plate (Rex and Easterling 2003), as a function of bearing force (P_{br}) and normalized bearing displacement (Δ_n), was considered at each bolt location, as follows:

$$\frac{P_{br}}{R_{n,br}} = \frac{1.74\Delta_n}{\left(1+\Delta_n^{\ 0.5}\right)^2} - 0.009\Delta_n \tag{1}$$

where P_{br} is the plate load, and $R_{n,br}$ is the nominal bearing strength that can be calculated using published bearing equations (CSA 2014):

$$R_{n,br} = 1.2L_e t_p \frac{F_y + F_u}{2} \leq 3d_b t_p F_u \tag{2}$$

where L_e stands for the end distance of the plate to the center of the hole, t_p is the plate thickness, F_y is the yield stress, and F_u is ultimate stress of the material. The variable Δ_n in Eq. 1 is the normalized bearing deformation calculated as:

$$\Delta_n = \frac{\Delta_{br}\beta K}{R_{n,br}} \tag{3}$$

where Δ_{br} stands for the hole elongation or bearing deformation at the bolt location, β is a correction factor that is dependent on the elongation at rupture—which in the case of the current study was assumed as unity (30% elongation)—K is the bearing stiffness, and based on finite element models and experimental tests, it depends on the following three stiffness values:

$$K = \frac{1}{\frac{1}{K_{br}} + \frac{1}{K_{be}} + \frac{1}{K_v}} \tag{4}$$

where K_{br} is the bolt bearing stiffness, K_{be} is the bending stiffness of the segment of plate directly in front of the bolt hole, and K_v is shear stiffness of the same plate segment. In this work, the bolt bearing stiffness (K_{br}) alone was in good agreement with the overall bearing stiffness (K) leading to the neglect of the bending and shear stiffness contributions. Therefore, the mechanical model for axial stress was developed based on bolt bearing stiffness (Rex and Easterling 2003):

$$K_{br} = 120F_y t_p (d_b / 25.4)^{0.8} \tag{5}$$

where d_b is the bolt diameter in millimetres and the quantity in brackets is dimensionless.

To investigate the behavior of the connection under the progressive collapse scenario, a series of discrete springs, considering the aforementioned stiffness, was assumed at each bolt location. In addition, an average of 1.6 mm was assumed to take into account the slippage effect at each bolt location. Fig. 4 illustrates a comparison between the mechanical model and experimental results for a 6.4 mm shear-tab connection with five bolts. The comparison shows that the mechanical model is able to predict both the peak horizontal load and the beam rotation at initial failure of the connection with about 1% error. Therefore, due to the good agreement between the experimental results and the mechanical model, it is considered reliable to estimate θ_u based on the aforementioned method.

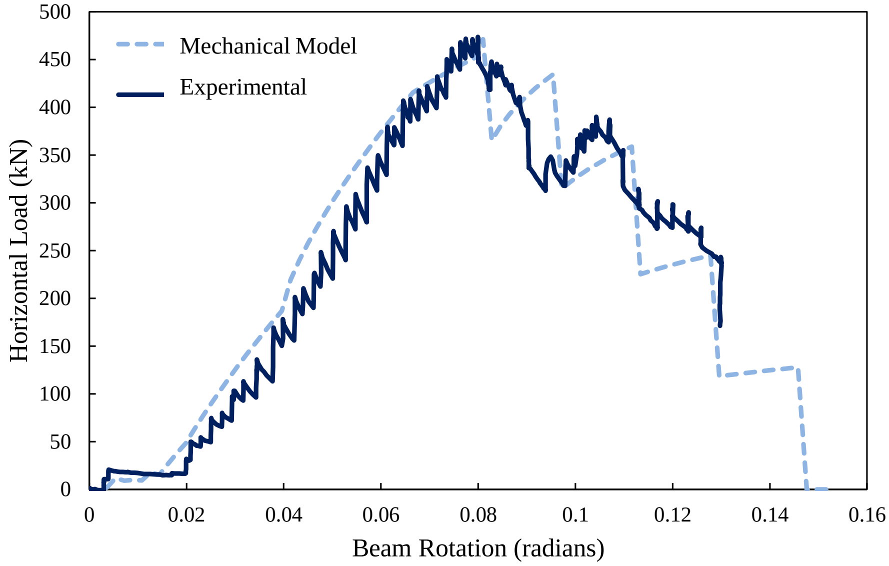

Figure 4: Horizontal load vs. beam rotation of mechanical model and experimental results (6.4 mm; 5 bolts)

5. Proposed Simplified Model

The main objective of this study was to develop a simplified method (SM) to simulate the response of steel frame joints under a condition when a major load-carrying element is compromised. To achieve this objective, based on structural analyses of the plate, θ_c and θ_u were calculated according to Eqs. 6 and 7. As shown in these equations, the rotations at initial and final failure are related to the end distance of the plate (L_e), eccentricity of the outermost bolt (e), beam span length (L), and the maximum bearing deformation of the plate (Δ).

$$\theta_c = \arctan\left(-\frac{1}{2}\frac{L}{e} + \frac{1}{2}\frac{(2\Delta + L_e)\left(L_e^{\,2} + 2L_e\Delta + 4\sqrt{\Delta^2 e^2 + L_e\Delta e^2 + e^4}\right)}{e\left(4\Delta^2 + 4L_e\Delta + L_e^{\,2} + 4e^2\right)}\right) \tag{6}$$

$$\theta_u = \arctan\left(\frac{1}{2}\frac{L}{e} + \frac{1}{2}\frac{(-2\Delta - L_e)\left(L_e^{\,2} + 2L_e\Delta - 4\sqrt{\Delta^2 e^2 + L_e\Delta e^2 + e^4}\right)}{e\left(4\Delta^2 + 4L_e\Delta + L_e^{\,2} + 4e^2\right)}\right) \tag{7}$$

where e is the vertical distance between the outer bolt and the center point of the bolt group.

The model is intended to be simple to implement and suitable for a wide range of practical applications, in order to simulate the global stability behavior of the structure. The model is reliable for predicting both linear and nonlinear performance of the structure. A bilinear curve

(Fig. 5) was employed to model the entire response of the shear-tab connection under a column removal scenario.

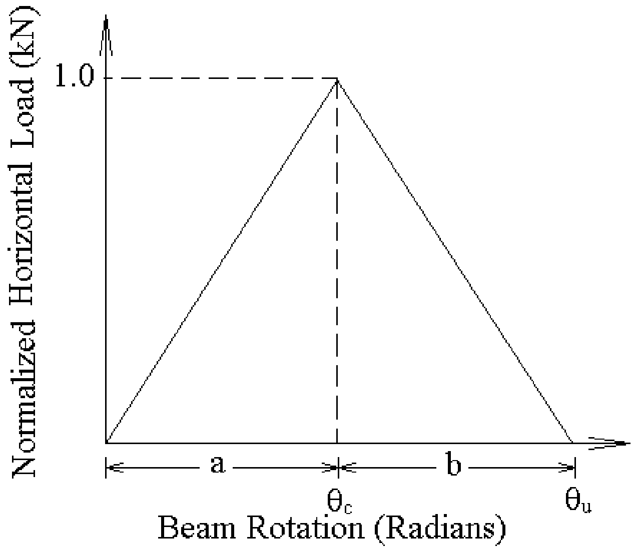

Figure 5: Proposed bilinear simplified model for shear-tab connections

Further simplifications were applied to θ_c and θ_u so that the use of the relatively complex Eqs. 6 and 7 could be replaced by parameters "a" (θ_c) and "b" ($\theta_u - \theta_c$) in Fig. 5. To accommodate the error introduced by the simplifying assumptions made in the derivations of θ_c and θ_u, some adjustments were implemented and, consequently, "a" and "b" were determined as follows:

$$a = 15.35n\left(\frac{t_p}{L}\right) + 0.2206\left(\frac{L_e}{d_b}\right) \tag{8}$$

$$b = \text{Constant} = 0.0494 \tag{9}$$

where n is the number of bolts in the connection.

In the next step, the developed bilinear simplified model was validated with the experimental results and the mechanical model discussed in the previous section, as shown in Fig. 6.

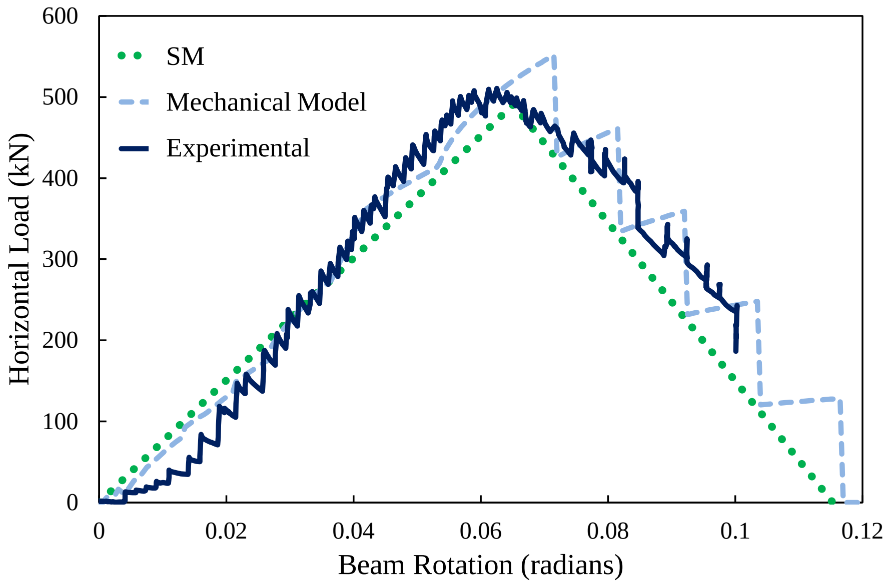

Figure 6: Horizontal load vs. beam rotation for experimental results, mechanical model, and SM (6.4 mm; 5 bolts)

The results of the simplified model and the experiments have been summarized in Tables 1 and 2. In the specimen descriptions, A and B represent the 9.5 mm and 6.4 mm shear tabs, respectively, and the preceding number represents the number of bolts in the connection. The final number is consistent with the nomenclature used in the experimental research program (Oosterhof and Driver 2014). In Tables 1 and 2, the negative values indicate that the SM underestimates the rotation, and the positive values represent overestimated rotations. These results indicate that the SM can estimate the rotational capacities of the beam at both initial and ultimate failure relatively well.

Table 1: Rotation of the beam at initial failure (radians)

Specimen	Experiment	SM	Error (%)
3A-1	0.1212	0.1272	4.95
3A-3	0.0969	0.1030	6.30
3B-1	0.0974	0.0990	1.64
3B-2	0.0810	0.0828	2.22
5A-1	0.0972	0.1172	20.58
5A-2	0.0818	0.0849	3.79
5B-1	0.0800	0.0855	6.88
5B-2	0.0650	0.0651	0.15

Table 2: Rotation of the beam at ultimate failure (radians)

Specimen	Experiment	SM	Error (%)
3A-1	0.1728	0.1721	-0.41
3A-3	0.1404	0.1473	4.91
3B-1	0.1530	0.1478	-3.40
3B-2	0.1242	0.1311	5.56
5A-1	0.1656	0.1666	0.60
5A-2	0.1296	0.1355	4.55
5B-1	0.1486	0.1362	-8.34
5B-2	0.1188	0.1153	-2.95

To investigate whether the linearization of the model has compromised any properties of the original nonlinear functions of θ_c and θ_u (Eqs. 6 and 7), a series of verifications were performed using the software MAPLE™ (MapleSoft 2013). As shown in Fig. 7, an increase in either the bearing deformation of the plate (Δ) or the span length (L), or both, results in an increase in the beam rotation at initial failure (θ_c); however, the span length has a greater effect. Moreover, a comparison of Figs. 7 and 8 shows that the span length has a greater effect on the ultimate rotation of the beam at complete bolt tear-out (θ_u) than on the beam rotation at initial failure (θ_c), leading to a higher degree of nonlinearity. Although there is some level of nonlinearity in the mechanical model, no severe nonlinearity has been observed. Therefore, the linear assumptions in the SM are considered valid as a simple approximation to the true behavior, but further work is required to examine the consequences of the nonlinear trends depicted in Figs. 7 and 8.

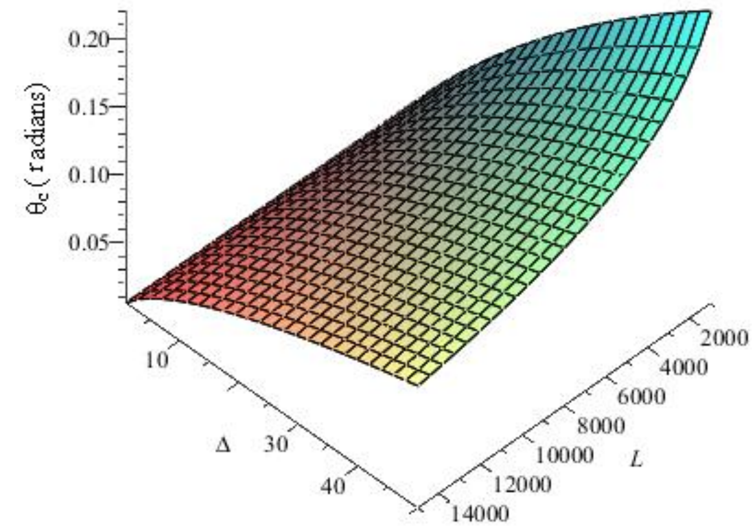

Figure 7: θ_c as a function of L and Δ (e: constant)

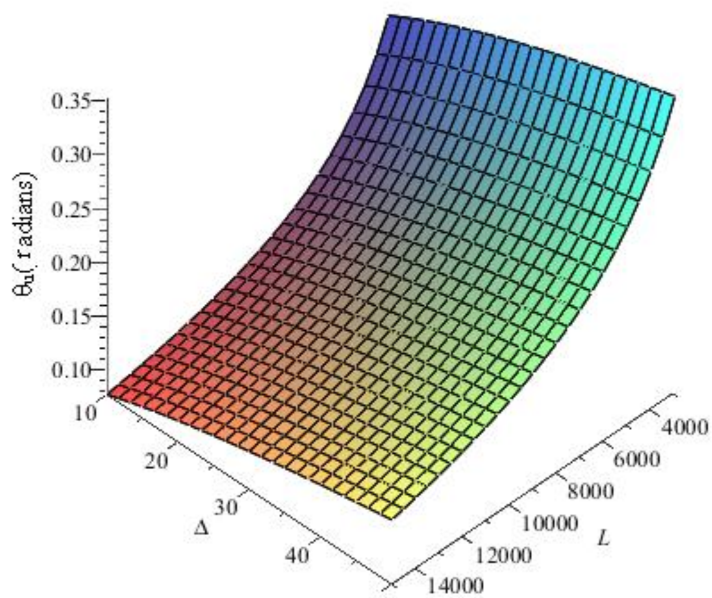

Figure 8: θ_u as a function of L and Δ (e: constant)

7. Conclusion

A bilinear model was developed as a simplification to a more comprehensive mechanical model to simulate the response of shear-tab connections under a column removal scenario, and it provides good agreement with the detailed model results. The simplified model was also validated using available experimental results and shown to be reliable to predict both linear and nonlinear response of the connection. The model was then used to study the influence of different factors such as connection geometry, number of bolts, and span length.

The developed model is very simple, yet accurate enough to be employed as joints representing shear-tab connections in a frame analysis to simulate the global stability behavior of the structure. Through the use of this model, large three-dimensional multi-bay systems can be analyzed much more efficiently than with the micro-modeling approaches used in previous studies.

Acknowledgments

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