DEFORMATION LIMITS AND ROTATIONAL CAPACITIES FOR CONNECTIONS UNDER COLUMN LOSS

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ABSTRACT

For structural steel framing systems subjected to column loss, the ultimate gravity load carrying capacity of the system is often controlled by the rotation and deformation capacities of the connections within the affected bays. Within current design specifications, the potential for disproportionate collapse in structural design is evaluated by notionally removing critical load supporting elements from the structure, and designing the structure to sustain the applicable gravity loads without collapse via the alternative load path analysis method. Steel gravity framing systems subjected to column loss undergo large vertical deflections under gravity loads that induce large rotation and axial deformation demands on the connections. However, the acceptance criteria currently used to evaluate connection failure in alternative load path analysis are based on results from seismic testing that do not reflect the large axial demands imposed on the connections. Recent experimental data have shown that application of these rotation limits to column loss scenarios in steel gravity framing systems can be non-conservative. In this paper, experimental data from steel single-plate shear connections tested under column loss are compared to the rotation limits provided in existing standards, and the factors influencing the widely varying levels of conservativism for different connection geometries are explained. A new approach for calculating acceptance criteria for steel gravity connections under column loss is also introduced. The new approach provides more risk consistency and accounts for the important influence of axial deformation demands on the connections.

INTRODUCTION

Some building owners in the United States (e.g., the General Services Administration and the Department of Defense (DoD)) and certain jurisdictions (e.g., New York City) currently require buildings to be designed to resist disproportionate collapse. To evaluate the potential for disproportionate collapse in design, engineers rely on the alternative load path analysis method (see e.g. DoD (2009)), in which individual load-bearing elements are notionally removed from the structure, and the remaining structure is designed to support the applicable gravity load combination without collapse. For steel frames designed to support only gravity loads, the steel shear connections play a critical role in ensuring the system robustness and stability. Large-scale tests of steel gravity framing systems under column removal (Johnson et al. 2014; Johnson et al. 2015) have shown that the system robustness depends on the capacity of the connections to resist axial loads after undergoing large rotation and axial displacement demands.

Current acceptance criteria for steel connections, which are used to evaluate connection failure in alternative load path analysis, take the form of rotation limits. These rotation limits are based almost entirely on results from seismic testing. In such seismic tests, the connections are typically subjected to rotation cycles of increasing magnitude until failure. These test conditions (1) result in low-cycle fatigue of the connection components, which is not relevant to column loss, and (2) do not reflect the large axial demands placed on the connections under column loss. While the increased plastic deformations associated with low-cycle fatigue may partially compensate for the lack of axial demands, recent experimental data have shown that direct application of seismic rotation limits to column loss scenarios in steel gravity framing systems can be non-conservative. Thus a better approach for calculating rotation limits for connections under column loss is needed.

In this paper, experimental data from steel single-plate shear connections tested under column loss (Weigand and Berman 2014) are compared to the rotation limits provided in standards for seismic evaluation and retrofit of existing buildings and for alternative load path analysis. Widely varying levels of conservativism are observed for different connections, and the factors contributing to this variability are discussed. To address the issues with the existing acceptance criteria, a new approach for calculating rotation limits for steel gravity connections under column loss is introduced. The new approach accounts for the influence of axial deformation demands on the connections and provides more risk consistency in the evaluation of connection failure under column loss.

COMPARISON OF EXPERIMENTAL DATA WITH CURRENT ROTATION LIMITS

Several existing specifications incorporate rotation limits for various types of steel connections; however, the applicability of these rotation limits to connections subjected to column loss needs to be considered carefully. The American Society of Civil Engineers (ASCE/SEI) 41-13 *Seismic Evaluation and Retrofit of Existing Buildings* (ASCE 2013) provides rotation limits for different connection types derived from tests of connection subassemblies under rotation cycles of increasing magnitude without axial restraint. These rotation limits depend only on the connection type and the depth of the connection bolt group, and thus have a number of deficiencies when considering their applicability to column loss, including the following:

1. they do not account for changes in connection geometry (e.g., changes in bolt diameter, thickness of the shear plate), which strongly influences the connection rotational capacity,

2. they do not include the effects of axial demands on the connections, and
3. they do not account for the effect of span length.

The Unified Facilities Criteria (UFC) 4-023-03 *Design of Buildings to Resist Progressive Collapse* (DoD 2009), which covers buildings under the jurisdiction of the DoD, adopted life-safety rotation limits from ASCE/SEI 41-13 for most connections, but provided reduced rotation limits for specific connection types including welded unreinforced flange, bolted web moment connections, reduced beam section moment connections, and single-plate shear connections. The rotation limits specified for these connection types in the UFC 4-023-03 were reduced relative to ASCE 41-13 based on a series of tests on connections subjected to blast and/or column removal conducted by the U.S. Defense Threat Reduction Agency and analyses performed by Myers, Houghton & Partners (Karns et al. 2008). Fig. 1 shows a comparison between the applicable acceptance criteria and measured rotational capacities for single-plate shear connections under column loss from Weigand and Berman (2014). The uncertainty in the experimental data was estimated as ±1 % (Weigand and Berman, 2016). The acceptance criteria, which are shown as dashed lines, include rotation limits from ASCE/SEI 41-13 for both life safety (labeled ASCE 41-LS) and collapse prevention (labeled ASCE 41-CP), as well as rotation limits from UFC 4-023-03 for primary members (labeled UFC-Primary) and for secondary members (labeled UFC-Secondary). The UFC-Secondary line is the same as the ASCE 41-LS line. The equations used to calculate the rotation limits are shown in Table 1.

Fig. 1 shows that the ASCE 41 rotation limits would be unconservative if applied directly to consider column loss (i.e., they would predict larger rotational capacities than single-plate shear connections can actually sustain). The UFC-Primary rotation limits are conservative when compared to the measured rotational capacities, but the amount of conservativism (i.e., the amount that the measured rotational capacities lie above the UFC-Primary line) vary widely for the different connections, which had different geometries (span, plate thickness, thread-condition, etc.). The connections with the least conservativism had either the longest spans or threads included in the shear plane.

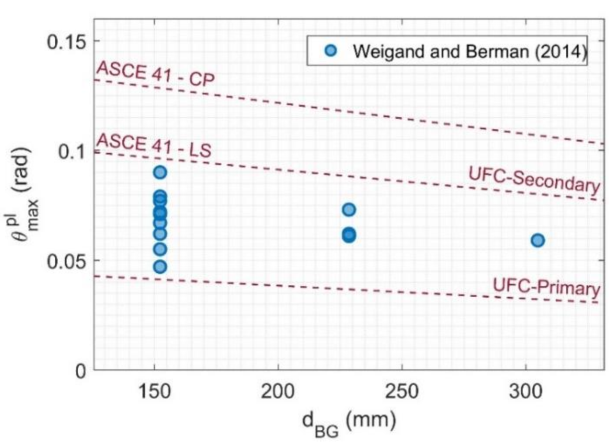


Figure 1: Comparison between rotation limits from ASCE/SEI 41 and UFC 4-023-03 and measured rotational capacities for connections subjected to column loss, as a function of the depth of the bolt group, d_{BG}.

Table 1: Specified Rotational Capacities for Single-Plate Shear Connections from ASCE 41-13 and UFC 4-023-03 (d_{BG} is the depth of the connection bolt group in mm).

Specification	Condition	Rotation Limit
ASCE 41-13	Life-Safety	$\theta_{\mathrm{max}}^{\mathrm{pl}} = 0.1125 - (0.0001063\ \mathrm{mm}^{-1})d_{BG}$
	Collapse Prevention	$\theta_{\mathrm{max}}^{\mathrm{pl}} = 0.1500 - (0.0001417\ \mathrm{mm}^{-1})d_{BG}$
UFC 4-023-03	Secondary Members	$\theta_{\mathrm{max}}^{\mathrm{pl}} = 0.1125 - (0.0001063\ \mathrm{mm}^{-1})d_{BG}$
	Primary Members	$\theta_{\mathrm{max}}^{\mathrm{pl}} = 0.0502 - (0.0000591\ \mathrm{mm}^{-1})d_{BG}$

A new standard, which is intended to specifically address disproportionate collapse, the *ASCE/SEI Standard for Mitigation of Disproportionate Collapse*, is currently under development. However, much of the existing guidance is still based on the ASCE/SEI 41-13 acceptance criteria. The new approach presented in this paper for calculating rotation limits for single-plate shear connections is based on the kinematics of connection response to column loss, and provides capabilities to overcome the deficiencies discussed above.

COMPONENT-BASED ANALYSIS OF FACTORS AFFECTING ROTATION LIMITS

Fig. 1 demonstrates that the amount of conservatism between the measured connection rotational capacities and the rotation limits specified in ASCE/SEI 41-13 and UFC 4-023-03 vary as a function of connection geometry. However, Fig. 1 only includes the specific connection geometries tested by Weigand and Berman (2014). Here, additional connection geometries are considered to answer two key questions:

1. What if the beam spans were longer?
2. What if the bolt threads were included in the shear plane?

Because experimental data are not available for these geometries, a component-based model for single-plate shear connections developed by Weigand (2016) is used to answer these two questions. In the component-based connection model, the connection is discretized into an arrangement of component springs that geometrically resembles the connection, where each component spring embodies a single bolt and characteristic-width segments of the shear plate and beam web (Fig. 2(a)). The component-based connection model was already validated against all 13 of the single-plate shear connection tests from Weigand and Berman (2014), and shown to predict their capacities within an average of 10 % (e.g., see Figs. 2(b) and 2(c)), using only the connection geometry, material properties, and applied loading (see Weigand (2014) for more details).

Fig. 3(a) shows the measured rotational capacities from Weigand and Berman (2014), with span length differentiated by marker color (all except one connection had threads excluded from the shear plane), Fig. 3(b) shows calculated rotational capacities for connections with 3.66 m (12 ft) longer spans (i.e., 12.8 m (42 ft) and 18.3 m (60 ft) spans) and all other connection geometry held constant, Fig. 3(c) shows calculated rotational capacities for connections with threads included in the shear plane, and Fig. 3(d) shows calculated rotational capacities for connections with threads included and with the 3.66 m (12 ft) longer spans. Comparison of Figs. 3(a) and 3(b) shows that increasing the span would reduce the rotational capacities for all connections, and that one 3-bolt connection rotational capacity would actually fall below the UFC-Primary acceptance criteria. Comparison of Figs. 3(a) and 3(c) shows that including threads in the shear plane would also reduce the rotational capacities for all connections, and that four of the connection rotational capacities would fall below the UFC-Primary acceptance criteria. Including threads in the shear plane had a larger influence on the connection rotational capacities than did increasing the span (inferred by comparing Figs. 3(b) and 3(c)). Fig. 3(d) shows that connections having both threads included in the shear plane and long spans are particularly vulnerable to having rotational capacities that are non-conservative, relative to the UFC-Primary acceptance criteria (seven out of the total thirteen connection rotational capacities fall below the UFC-Primary acceptance criteria). This demonstrated potential for the rotational capacities of realistic connection geometries to be predicted non-conservatively by the most stringent current acceptance criteria (UFC-Primary) motivates the need for a better approach for calculating connection rotational capacities.

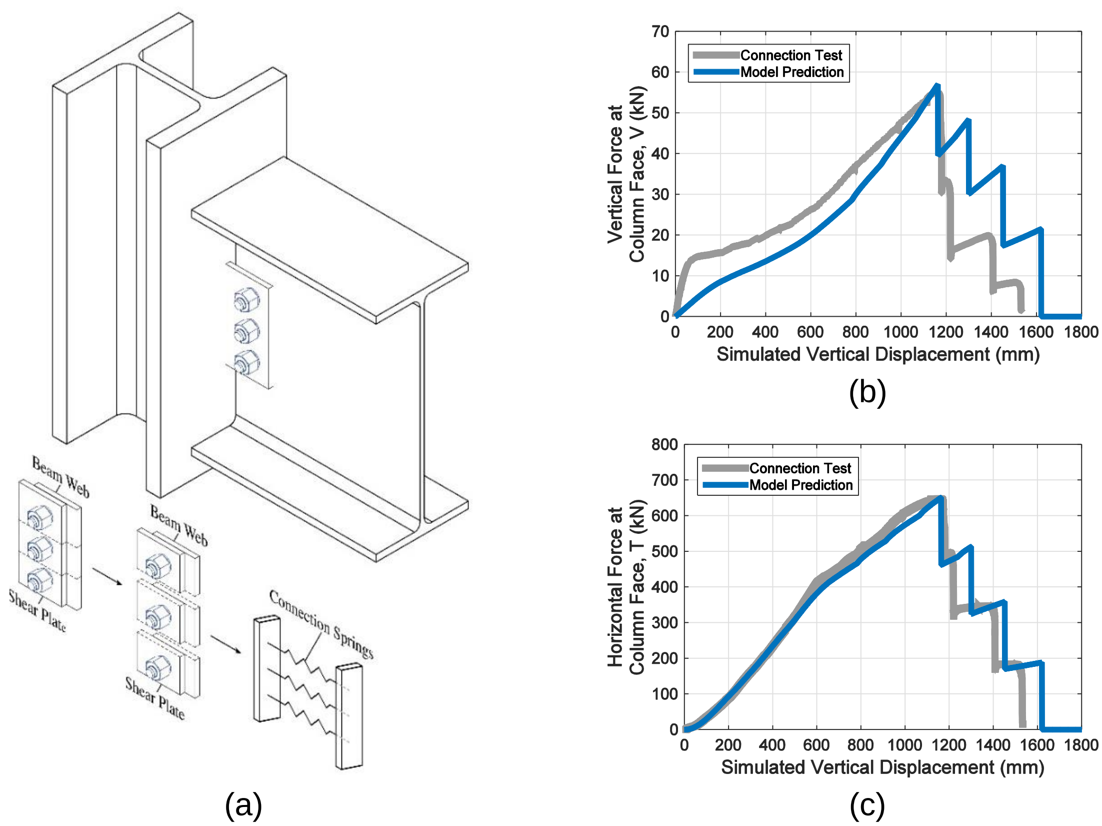


Figure 2: (a) discretization of single-plate shear connection into individual bolt-widths, (b) comparison of predicted vertical force-displacement response with connection data and (b) comparison of predicted horizontal force-displacement response from component-based model with connection data.

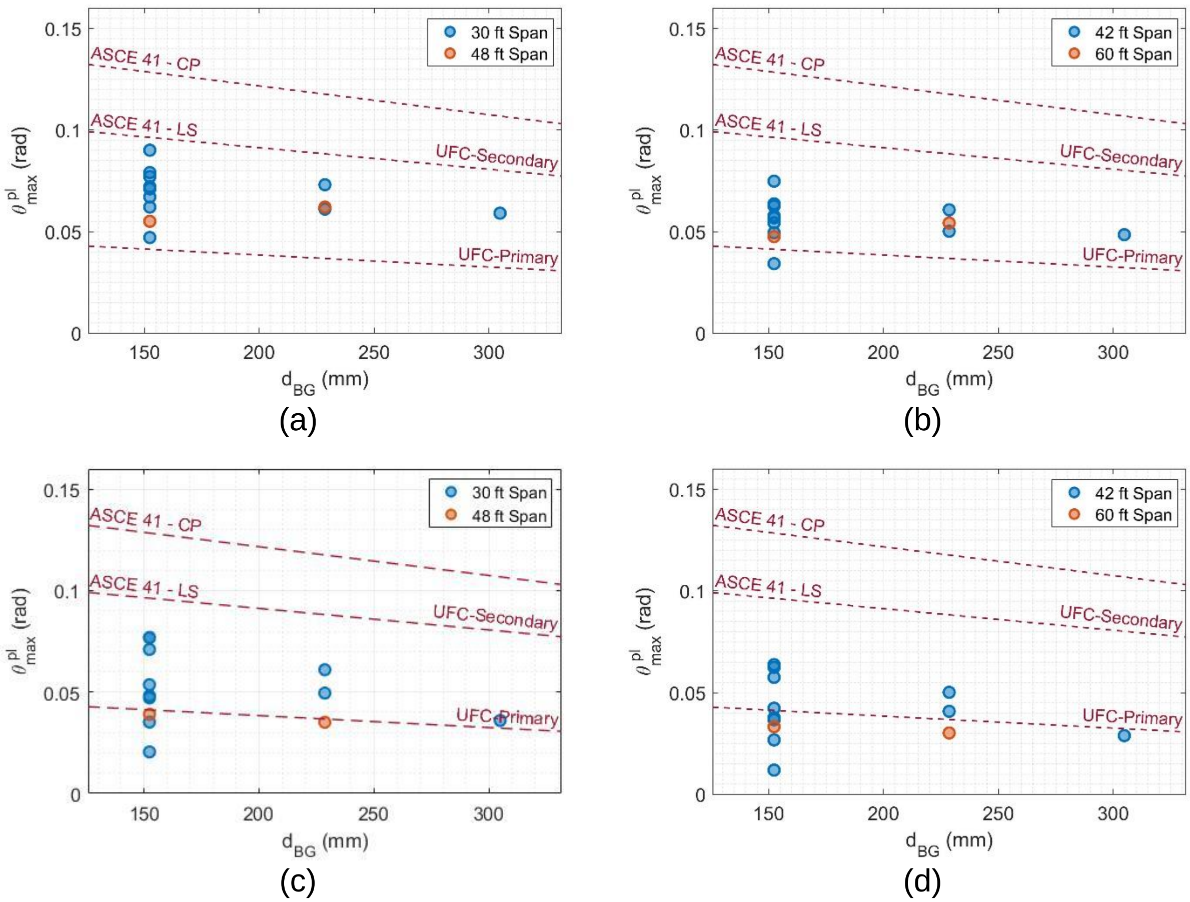
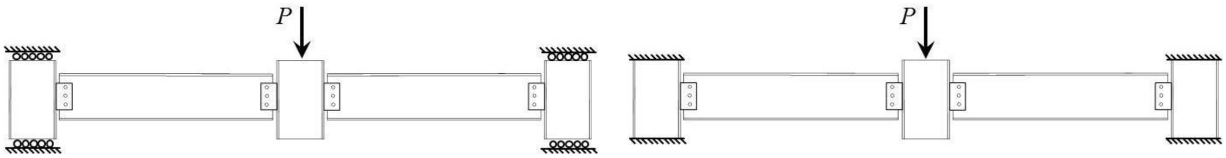


Figure 3: (a) Experimental data; calculated rotational capacities for, (b) increased span lengths and threads excluded from shear plane, (c) threads included in shear plane, and (d) increased span lengths and threads included in shear plane.

ANALYTICAL DERIVATION OF ROTATION LIMITS UNDER COLUMN LOSS

For frames designed to support gravity loads only, loss of a column results in large vertical deflections at the missing column that impose significant rotational demands on the connections. Significant axial demands can also be imposed, depending on the degree of restraint provided by the surrounding structure. Where the connections are discontinuous through the unsupported column (e.g., for a corner column loss scenario) minimal axial restraint is provided, and thus the axial demands are small. However, where the connections span continuously through the unsupported column (e.g., for an interior column loss scenario), the axial restraint provided by the surrounding structure can subject the connections to significant axial deformations in combination with large rotations. Fig. 4(a) illustrates the limiting case of no axial restraint, where the end columns are free to translate horizontally in-plane and the connections are subjected only to monotonically increasing rotation until failure. Fig. 4(b) illustrates the limiting case of perfect axial restraint, where translation of the column ends is prevented.



(a) (b)

Figure 4: Column loss scenarios with (a) no axial restraint and (b) axial restraint.

The rotational demands imposed on the connections can be calculated in terms of the vertical deflection of the missing column, Δ, as

$$\theta = \tan^{-1}\left(\frac{\Delta}{L_{\mathrm{r}}}\right) \,, \tag{1}$$

where L_{r} is the distance between the centers of the bolt groups at the ends of the framing members in the undeformed configuration (Fig. 5). For the condition without axial restraint (Fig. 4(a)), no axial demands are imposed on the connections. For the condition with axial restraint (Fig. 4(b)), an axial deformation δ is imposed on each connection (see Fig. 5), which can be calculated as follows:

$$\delta = \frac{L_{\mathrm{r}}}{2}\left[\sqrt{1+\left(\frac{\Delta}{L_{\mathrm{r}}}\right)^2} - 1\right] . \tag{2}$$

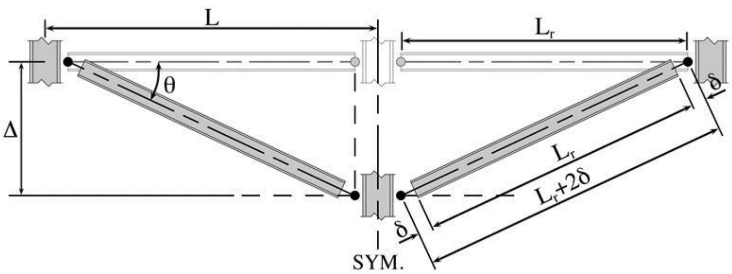


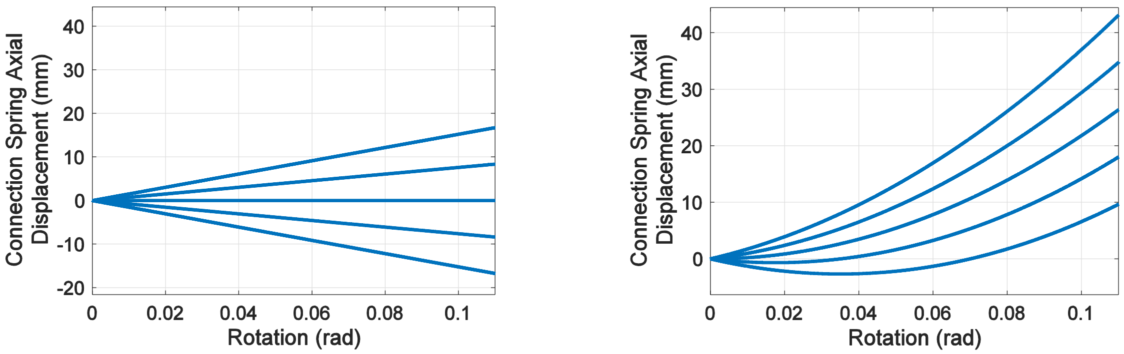
Figure 5: Connection demands based on vertical deflection of the missing column.

For both axial restraint conditions, the axial deformation of each connection spring (see Fig. 2(a)) can be calculated in terms of θ and δ as:

$$\delta_{\mathrm{j}} = \delta + y_{\mathrm{j}} \sin\theta \,, \tag{3}$$

where y_j denotes the vertical distance from the j^{th} connection spring to the center of the bolt group. Eqs. (1) - (3) are based on the assumption that the beams are rigid relative to the connections, so that the rotations and deformations localize in the connections. This rigid-body assumption is further discussed by Weigand and Berman (2014), including validation of the assumption through comparison with experimental measurements.

For the condition without axial restraint (i.e., $\delta = 0$), the connection spring deformations from Eq. (3) are essentially linear with increasing rotation (Fig. 6(a)). For the condition with axial restraint, δ is calculated from Eq. (2), and larger tensile deformations of the component springs are observed for a given level of rotation (Fig. 6(b)), relative to the case without axial restraint. Because of the dependence of Eq. (2) on the span length, larger span lengths result in increased tensile deformations of the component springs.



(a) (b)

Figure 6: axial deformations of connection segments, δ_{j}, with (a) $\delta = 0$, and (b) $\delta \neq 0$.

If the limiting axial deformation for a single bolt row of a connection, δ_{u}, is known, either from experiments or from computational modeling, then the corresponding rotation limit for the connection, θ_{u}, can be calculated. The rotation limit, θ_{u}, corresponds to the configuration in which the axial deformation of an extreme bolt row (lowest or highest) reaches δ_{u}. Setting $y_j = d_{\mathrm{BG}}/2$ in Eq. (3) for the extreme bolt row and introducing the small-angle approximation $\sin(\theta) \approx \theta$ allows Eq. (3) to be solved for θ_{u}, as

$$\theta_{\mathrm{u}} = \frac{2(\delta_{\mathrm{u}} - \delta)}{d_{\mathrm{BG}}} \quad . \tag{4}$$

For the condition without axial restraint (i.e., $\delta = 0$), Eq. (4) reduces to $\theta_{\mathrm{u}} = 2\delta_{\mathrm{u}}/d_{\mathrm{BG}}$. For the condition with axial restraint, substituting Eq. (2) into Eq. (4), introducing small-angle approximations, and neglecting higher-order terms (see Main and Sadek (2012) for more details), results in a quadratic equation for θ_{u} that yields the following positive root:

$$\theta_{\mathrm{u}} = 2\sqrt{\left(\frac{d_{\mathrm{BG}}}{2L}\right)^2 + \frac{\delta_{\mathrm{u}}}{L}\left(1 + \frac{\delta_{\mathrm{u}}}{L}\right)} - \frac{d_{\mathrm{BG}}}{L} \quad . \tag{5}$$

Fig. 7 compares solution curves calculated from Eq. (5) against measured rotation limits for single-plate shear connections subjected to column loss from Weigand and Berman (2014), for 3-bolt and 4-bolt connections with 9.1 m (30 ft) and 14.6 m (48 ft) spans. Fig. 7 verifies that the rigid-body model provides a close approximation to the measured rotation at connection failure, as demonstrated by the close proximity of the connection data (circular markers) to the solution curves. The solution curves are also shown to be slightly conservative relative to the experimental data (i.e., the solution curves are consistently below the experimental data), which is expected based on the rigid-body model and the assumed perfect axial restraint.

By using Eq. (5), along with average measured values of the limiting axial deformation δ_{u} for different groups of connections tested by Weigand and Berman (2014), rotational capacities can be calculated and compared with the experimental data (Fig. 8). Compared with the wide scatter of the experimental data relative to current rotation limits (Fig. 1), Fig. 8 shows that Eq. (5) provides significantly improved consistency with the experimental data. The improved consistency is achieved by accounting for the influences of axial restraint, span length, and connection geometry, factors which current rotation limits used in alternative load path analysis do not directly consider. Fig. 8 shows that Eq. (5) is conservative relative to the experimental data for all but one test, with the slight non-conservatism in that case resulting from the use of the average measured value for δ_u. Uncertainty in the deformation limit δ_u is the key factor affecting the uncertainty in the calculated rotational capacities. In selecting appropriate values of δ_u to use in design, uncertainty in the value of δ_u should be considered to ensure consistent reliability.

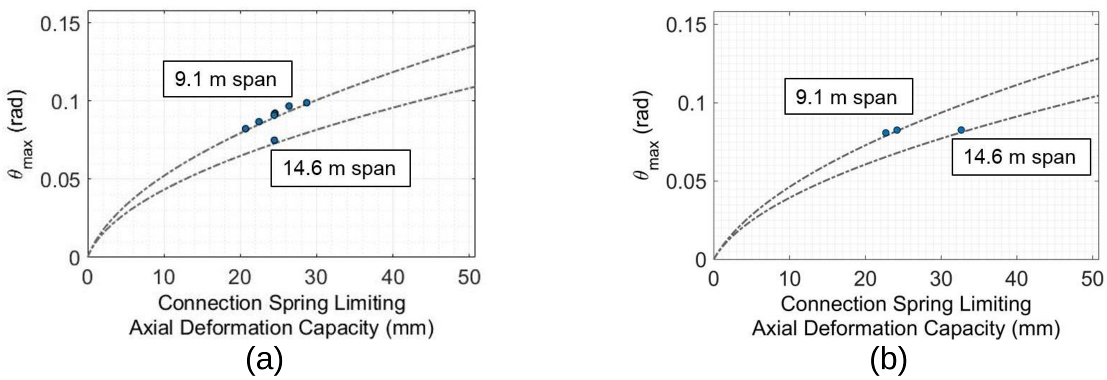


Figure 7: Comparison between Eq. (5) and measured rotational capacities for (a) 3-bolt single-plate shear connections and (b) 4-bolt single-plate shear connections.

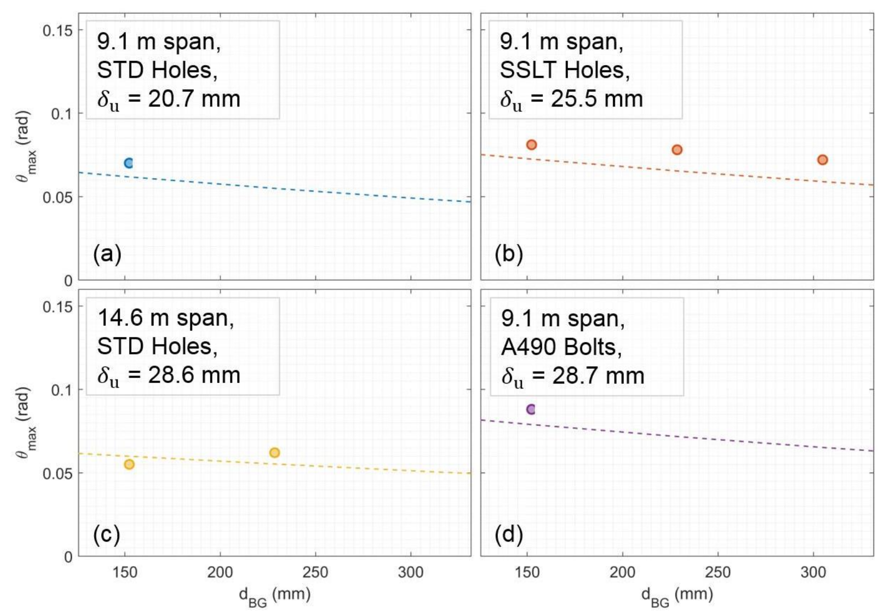


Figure 8: Comparison between Eq. (5) (dashed line) and measured rotational capacities for single-plate shear connections (circular markers).

SUMMARY AND CONCLUSIONS

When evaluating the performance of connections subjected to column loss, it is important to recognize that connections behave differently when subjected to seismic loads than when subjected to column loss. Acceptance criteria in existing specifications, which were developed based on results from seismic testing, may not be appropriate for column loss as they do not capture (1) differences in connection geometry (e.g., bolt diameter, plate thickness, thread condition), (2) the influence of axial deformation demands on the connections, and (3) the influence of span length. As a result of these deficiencies, current acceptance criteria are not risk-consistent for connections with different geometries, or frames which have different spans.

Results from component-based models of single-plate shear connections showed that there exist connection geometries in which even the most stringent currently specified acceptance criteria (the UFC 4-023-03 rotation limits for primary members) would not be conservative for disproportionate collapse. However, a new approach in which the axial deformation capacities of component-width segments of the connection are used to calculate rotational capacities for the connections can overcome these deficiencies to provide results that are both risk-consistent and capture the influence of axial deformation demands on the connection, including those resulting from span length.

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