Acceptance Criteria for Nonlinear Alternative Load Path Analysis of Steel and Reinforced Concrete Frame Structures

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ABSTRACT

Alternative load path analysis is the primary approach for evaluating the potential for disproportionate collapse in structural design. In this approach, individual load-bearing elements are notionally removed from a structure, and the remaining structure is required to sustain the applicable gravity loads without collapse. Column loss in steel and reinforced concrete frame structures can result in large vertical deflections that subject the beams and their connections to significant axial deformations in addition to large rotations. Failure of members and connections in alternative load path analysis is evaluated by comparing the plastic rotations of these components to acceptance criteria, defined as rotation limits, based largely on data from seismic tests. Axial demands on members and connections, which are important in column loss scenarios, were not relevant in this seismic testing, and thus the corresponding rotation limits may not be appropriate for column loss. This paper compares current acceptance criteria for alternative load path analysis with experimental data reported in the literature under column loss scenarios for steel gravity frames with single-plate shear connections and for reinforced concrete moment frames. Significant variability is observed in the level of conservatism of these acceptance criteria, and factors contributing to this variability are discussed. A new approach for defining acceptance criteria is summarized, which provides improved risk-consistency by directly accounting for the interaction of axial and rotational demands on the connections under column loss scenarios.

INTRODUCTION

Alternative load path analysis (ALPA) is the primary approach used to evaluate the potential for disproportionate collapse in structural design. In ALPA, various notional column loss scenarios are considered, and the capacity of the remaining structure to sustain the applicable gravity loads is evaluated. Failure in ALPA is evaluated by comparing the rotations developed in members and connections with acceptance criteria specified as rotation limits. Currently, the primary documents governing alternative load path analysis in the United States are the Unified Facilities Criteria (UFC) 4-023-03 *Design of Buildings to Resist Progressive Collapse* (DoD 2009), applicable to military buildings, and the General Services Administration (GSA) *Alternate Path Analysis & Design Guidelines for Progressive Collapse Resistance* (GSA 2013), applicable to civilian government buildings. The acceptance criteria in

UFC 4-023-03, which were subsequently adopted in the GSA Guidelines, were based primarily on seismic acceptance criteria specified in the American Society of Civil Engineers (ASCE/SEI) 41-13, *Seismic Evaluation and Retrofit of Existing Buildings* (ASCE 2013). Seismic acceptance criteria were adopted for use in ALPA because of the much more extensive experimental data and modeling guidance available for structural systems under seismic loading than under column loss.

While current acceptance criteria used in ALPA are based primarily on data from seismic testing, there are significant differences between the demands imposed on members and connections under seismic loading and under column loss. Low-cycle fatigue is an important issue in seismic loading that is not relevant to column loss. To account for this effect, seismic testing imposes rotation cycles of increasing magnitude on members and connections (e.g., Clark et al. 1997). As a result of this low-cycle fatigue, rotation limits based on seismic testing can be overly conservative for column loss. On the other hand, column loss can impose significant axial demands on beams and connections that are not relevant to seismic loading. The combination of axial and rotational demands can in some cases result in earlier failure than under purely flexural loading (e.g., for steel single-plate shear connections). In other cases, however, the development of catenary action can result in peak vertical loads that are achieved at much larger rotations than under purely flexural action (e.g., for reinforced concrete moment frames). In recognition of differences between seismic loading and column loss, some of the acceptance criteria in UFC 4-023-03 were modified relative to those in ASCE/SEI 41-13, in some cases based on additional experimental data or computational simulations specific to column loss. However, the modified acceptance criteria still consider only rotational demands and do not consider factors such as the span length and the axial restraint conditions, which can strongly influence the axial demands under column loss.

This paper presents a comparison of the acceptance criteria (rotation limits) specified in the UFC 4-023-03 with experimentally measured rotational capacities from structural assemblies tested under column loss. In the years since the development of the acceptance criteria in the UFC 4-023-03, numerous structural assemblies have been tested under column loss scenarios, providing an opportunity to evaluate the suitability of the current criteria. This paper focuses on steel gravity frames with single-plate shear ("shear tab") connections and on reinforced concrete moment frames. For single-plate shear connections and for reinforced concrete moment frames, the acceptance criteria in UFC 4-023-03 were modified relative to those in ASCE/SEI 41-13. Comparisons with the original acceptance criteria in ASCE/SEI 41-13 are also presented, with the recognition that these criteria are intended for analysis under seismic loading, not column loss. Significant variability is observed in the level of conservatism of the acceptance criteria in the UFC 4-012-03, largely because of the effect of axial demands, which are influenced by factors not considered in the current acceptance criteria, such as the span length. A new approach for defining acceptance criteria is summarized, which provides improved risk-consistency by directly accounting for the interaction of axial and rotational demands on the connections under column loss scenarios.

ACCEPTANCE CRITERIA IN CURRENT SPECIFICATIONS

Table 1 presents acceptance criteria from ASCE/SEI 41-13 and from UFC 4-023-03 for steel single-plate shear connections. The GSA Guidelines adopted the same acceptance criteria as in UFC 4-023-03, and therefore these are not listed separately. Equations for the limiting connection rotations are presented as functions of the depth of the connection bolt group, d_{BG}. Acceptance criteria from ASCE/SEI 41-13 are presented for two performance objectives: life safety (LS) and collapse prevention (CP). Acceptance criteria from UFC 4-023-03 are presented for both primary components (components whose contribution to structural resistance is included) and for secondary components (components whose contribution to structural resistance is neglected). In general, the UFC 4-023-03 adopts life safety acceptance criteria from ASCE/SEI 41-13 for both primary and secondary components, unless otherwise specified. However, Table 1 shows that the rotation limits for primary components in UFC 4-023-03 were reduced relative to the life safety criteria in ASCE/SEI 41-13. This reduction was based in part on numerical simulation of a single-plate shear connection subjected to column loss (Karns et al. 2008).

Table 1: Rotational capacities for single-plate shear connections from ASCE/SEI 41-13 and UFC 4-023-03.

Specification	Condition	Rotation Limit (rad)
ASCE/SEI 41-13 (ASCE 2013)	Life Safety	$\theta_{\max}^{\mathrm{pl}} = 0.1125 - (0.0001063\ \mathrm{mm}^{-1})d_{\mathrm{BG}}$
	Collapse Prevention	$\theta_{\max}^{\mathrm{pl}} = 0.1500 - (0.0001417\ \mathrm{mm}^{-1})d_{\mathrm{BG}}$
UFC 4-023-03 (DoD 2009; GSA 2013)	Primary Components	$\theta_{\max}^{\mathrm{pl}} = 0.0502 - (0.0000591\ \mathrm{mm}^{-1})d_{\mathrm{BG}}$
	Secondary Components	$\theta_{\max}^{\mathrm{pl}} = 0.1125 - (0.0001063\ \mathrm{mm}^{-1})d_{\mathrm{BG}}$

NOTE: The depth of the connection bolt group, d_{BG}, has units of mm.

Table 2 presents acceptance criteria from ASCE/SEI 41-13 and from UFC 4-023-03 for beams in reinforced concrete moment frames. The GSA Guidelines adopted the same acceptance criteria as in UFC 4-023-03, and therefore these are not listed separately. In Table 2, ρ is the reinforcement ratio, ρ' is the reinforcement ratio of the compression steel, ρ_{bal} is the balanced reinforcement ratio, V is the design shear force, b_w is the beam width, d is the distance from the extreme compression fiber to the centroid of the tension reinforcement, and f_c' is the compressive strength of concrete (see ASCE (2013) for further details on these definitions). Values between those listed in Table 2 are to be determined by linear interpolation.

Table 2 shows that for reinforced concrete beams, the rotation limits for primary components in UFC 4-023-03 were increased by a factor of approximately 2.5 relative to the LS acceptance criteria in ASCE/SEI 41-13. Similarly, the rotation limits for secondary components in UFC 4-023-03 were increased by a factor of 2.0 relative to the CP acceptance criteria in ASCE/SEI 41-13, or by a factor of 4.0 relative to the LS acceptance criteria. These increases indicate that reinforced concrete beams were considered to be capable of sustaining significantly larger rotations under column loss than under seismic loading.

Table 2: Rotational capacities for reinforced concrete beams from ASCE/SEI 41-13 and UFC 4-023-03.

Specification	Condition	$\frac{\rho - \rho'}{\rho_{\text{bal}}}$	$\frac{V}{b_w d \sqrt{f_c{}'}}$	Rotation Limit, $\theta_{\text{max}}^{\text{pl}}$ (rad)
ASCE/SEI 41-13 (ASCE 2013)	Life Safety	≤ 0.0	≤ 0.25	0.025
		≤ 0.0	≥ 0.50	0.020
		≥ 0.5	≤ 0.25	0.020
		≥ 0.5	≥ 0.50	0.015
	Collapse Prevention	≤ 0.0	≤ 0.25	0.050
		≤ 0.0	≥ 0.50	0.040
		≥ 0.5	≤ 0.25	0.030
		≥ 0.5	≥ 0.50	0.020
UFC 4-023-03 (DoD 2009; GSA 2013)	Primary Components	≤ 0.0	≤ 0.25	0.063
		≤ 0.0	≥ 0.50	0.050
		≥ 0.5	≤ 0.25	0.050
		≥ 0.5	≥ 0.50	0.038
	Secondary Components	≤ 0.0	≤ 0.25	0.100
		≤ 0.0	≥ 0.50	0.080
		≥ 0.5	≤ 0.25	0.060
		≥ 0.5	≥ 0.50	0.040

NOTE: V has units of N, b_w and d have units of mm, and $f_c{}'$ has units of MPa.

COMPARISON WITH EXPERIMENTAL DATA

In this section, experimental data reported in the literature under column loss scenarios are compared with the acceptance criteria from UFC 4-023-03, with a focus on evaluating the level of conservatism provided by the acceptance criteria for column loss. Seismic acceptance criteria from ASCE/SEI 41-13, from which the UFC criteria were adapted, are also presented for comparison.

Steel Gravity Frames with Single-Plate Shear Connections

Figure 1(a) shows a comparison of measured rotational capacities from typical single-plate shear connections tested under column loss scenarios by Weigand and Berman (2014) with the acceptance criteria from UFC 4-023-03 and ASCE/SEI 41-13. The uncertainty in the experimental data was estimated as ±1 % (Weigand and Berman 2016). As illustrated in Figure 2 for a typical vertical load vs. beam chord rotation curve, the measured rotational capacities correspond to the rotation at the peak vertical load. Elastic rotations were deducted from the rotational capacities presented in Figure 1 for consistency with the acceptance criteria. Figure 1(a) shows that the acceptance criterion for primary components in UFC 4-023-03 was conservative for all 13 of the connections tested by Weigand and Berman (2014), although the level of conservativism varied widely, with measured rotational capacities in some cases of more than double the acceptance criterion. Figure 1(a) also shows that the LS acceptance criterion from ASCE/SEI 41-13 was non-conservative for all of the connection geometries tested by Weigand and Berman (2014), confirming that the reduction of the rotation limits for primary components in UFC 4-023-03 relative to the seismic LS acceptance criterion is warranted (see Table 1).

A recent study by Weigand and Main (2016) further compared the acceptance criteria in UFC 4-023-03 to calculated rotational capacities for bare-steel single-plate shear connection configurations beyond those tested by Weigand and Berman (2014). Weigand and Main (2016) found that single-plate shear connections that used bolts with threads included in the shear plane, or those with longer-spans (between 12.1 m (40 ft) and 18.3 m (60 ft)), had sufficiently small rotational capacities that even the UFC 4-023-03 acceptance criteria for primary components would not be conservative. Main and Sadek (2012) identified the presence of a composite slab as another factor that is potentially detrimental to the rotational capacities of connections subjected to column loss, although the presence of the composite slab does provide increased capacity to sustain vertical loads. At the removed column, the slab response is very stiff in compression relative to the tensile stiffness of the connection bolt group, biasing the neutral axis of the connection toward the top of the beam flange and increasing the tensile deformations of the connection components for a given chord rotation. Figure 1(b) shows rotational capacities for single-plate shear connections, incorporating the effect of the composite slab on steel deck by locating the center-of-rotation of the connection at the top flange of the beam. These rotational capacities were calculated using the component-based model for single-plate shear connections formulated by Weigand (2016). A comparison between Figure 1(a) and Figure 1(b) shows that the effect of the composite slab would reduce the rotational capacities for all connection geometries, and Figure 1(b) demonstrates that the rotational capacities of three 3-bolt (152 mm-depth) connections would fall below the UFC 4-023-03 acceptance criteria for primary members. The demonstrated potential for the UFC 4-023-03 acceptance criteria to be non-conservative for single-plate shear connections motivates the need for a new approach for defining acceptance criteria such as that described by Weigand and Main (2016), summarized below.

Reinforced Concrete Moment Frames

Figure 3 presents measured rotational capacities from 23 reinforced concrete frame assemblies, tested under simulated column removal, and compares these with the rotation limits specified in UFC 4-023-03 and ASCE/SEI 41-13. As illustrated in Figure 4 for a measured vertical load vs. beam chord rotation curve, reinforced concrete moment frames typically exhibit an initial peak vertical load associated with flexural-arching action, followed by a drop in the vertical load associated with concrete crushing and plastic hinge formation at the beam-column joints, followed by a subsequent increase in load associated with the development of catenary action, with a final peak load that may or may not exceed the initial peak load. Red markers in Figure 3 represent rotational capacities associated with the initial flexural-arching-action stage of the response, as illustrated in Figure 4. Blue markers in Figure 3 represent rotational capacities associated with the catenary-action stage of the response, also illustrated in Figure 4. Blue markers are presented only when the peak vertical load associated with catenary action exceeded the peak vertical load associated with flexural-arching action.

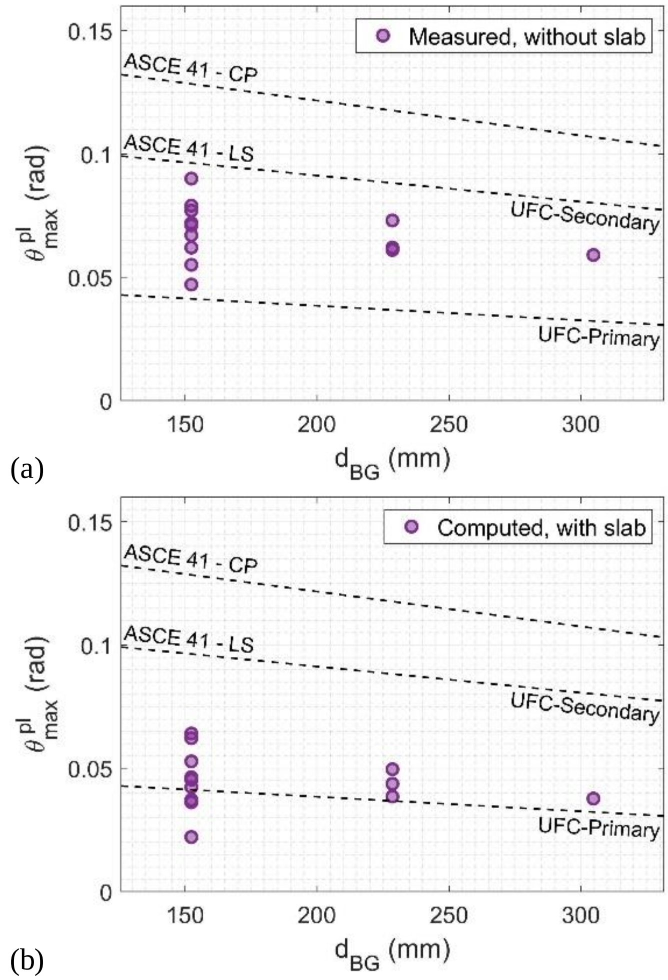

Figure 1: Comparison of acceptance criteria for single-plate shear connections from ASCE/SEI 41-13 and UFC 4-023-03 with rotations at peak load under column loss: (a) measured rotations for connections without slab (Weigand and Berman 2014) (b) computed rotations for connections with slab.

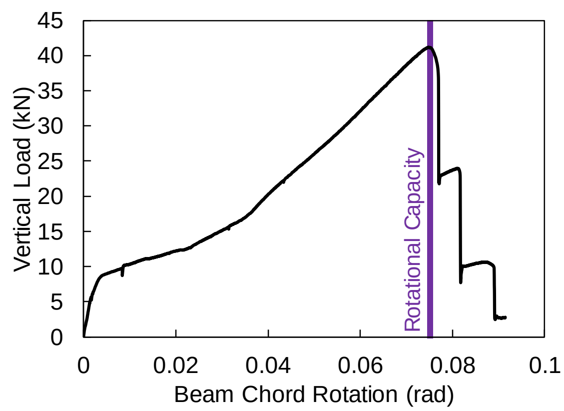

Figure 2: Vertical load vs. beam chord rotation for single-plate shear connection (specimen sps3b|STD|34|38|48L from Weigand and Berman 2014) with rotational capacity indicated.

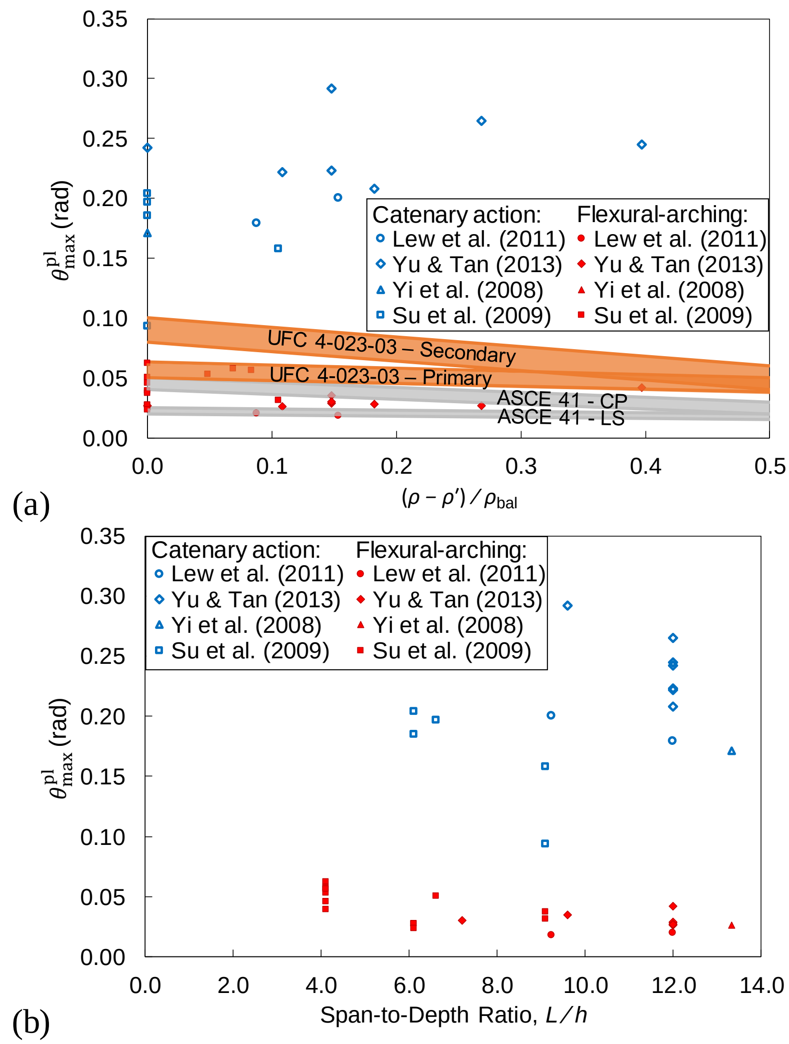

Figure 3: Plastic rotation limits for reinforced concrete beams as a function of (a) $(\rho - \rho')/\rho_{\text{bal}}$ and (b) span-to-depth ratio.

Figure 3(a) presents the measured rotational capacities as a function of the ratio $(\rho - \rho')/\rho_{\text{bal}}$. Because the rotation limits from ASCE/SEI 41-13 and UFC 4-023-03 depend on the normalized shear demand, $V/(b_w d\sqrt{f_c'})$, as well as on $(\rho - \rho')/\rho_{\text{bal}}$ (see Table 2), the rotation limits are plotted as shaded bands in Figure 3(a). The upper and lower bounds of each shaded band were obtained by linear interpolation from Table 2, with the lower bound corresponding to $V/(b_w d\sqrt{f_c'}) \geq 0.50$ and the upper bound corresponding to $V/(b_w d\sqrt{f_c'}) \leq 0.25$. The design shear demand for test specimens is generally not reported in the literature, so evaluating the effect of $V/(b_w d\sqrt{f_c'})$ was not possible. All of the rotational capacities associated with flexural-arching action were below the UFC 4-023-03 upper-bound rotation limit for primary components, which indicates that the UFC acceptance criteria are non-conservative for flexural-arching action. However, all except one of the rotational capacities associated with flexural-arching action were above the lower-bound ASCE/SEI 41-13 Life Safety (LS) rotation limit, which indicates that the LS acceptance criterion from ASCE/SEI 41-13 is generally conservative for flexural-arching action under column loss. All except one of the rotational capacities associated with catenary action exceeded the UFC 4-023-03 upper-bound rotation limit for secondary components, which indicates that the UFC acceptance criteria generally are conservative for cases in which catenary action is developed. However, the level of conservatism varies widely, with measured rotational capacities in some cases of more than five times the acceptance criterion for primary components.

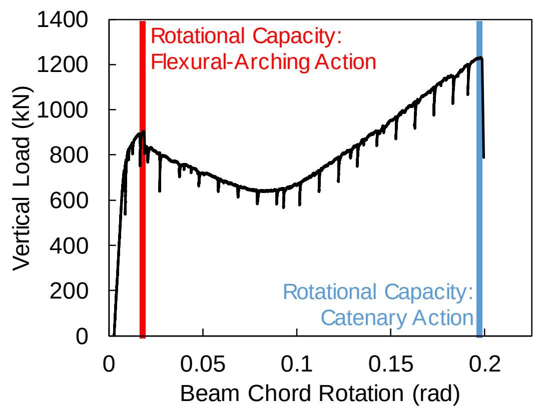

Figure 4: Vertical load vs. beam chord rotation for reinforced concrete special moment frame assembly from Lew et al. (2011) with rotational capacities associated with flexural-arching action and catenary action indicated.

Figure 3(b) shows data from the same 23 reinforced concrete frame assembly tests as in Figure 3(a), but presented as a function of the span-to-depth ratio, L/h. Figure 3(b) shows that reinforced concrete frames with span-to-depth ratios of less than 6 achieved their peak vertical capacity during the initial flexural-arching-action stage of the response and were unable to develop sufficient catenary action in the subsequent response to reach or exceed this initial peak. This implies that for reinforced concrete frames with span-to-depth ratios of less than 6, flexural-arching action may be the only response mechanism available for resisting disproportionate collapse. It is also important to recognize that development of catenary action requires axial restraint of the beams, which in some cases is not provided by the surrounding structural system (e.g., in a corner column loss scenario). When catenary action cannot be developed, much smaller rotations at peak load are attained, with the experimental data showing a maximum plastic rotation associated with flexural-arching action of 0.062 rad. As noted above, the UFC acceptance criterion for primary components is not conservative in such cases, but the LS acceptance criterion from ASCE/SEI 41-13 generally is conservative. As will be presented in future publications, improved risk consistency can be achieved by defining acceptance criteria for reinforced concrete moment frames using an approach like that of Weigand and Main (2016), which directly accounts for the combination of axial and rotational demands in a column loss scenario.

ROTATION LIMITS BASED ON DEFORMATION CAPACITIES

Weigand and Main (2016) demonstrated the effectiveness of a new approach for defining acceptance criteria for column loss that provides significantly improved risk consistency, relative to ASCE/SEI 41-13 and UFC 4-023-03. In this approach, rotation limits for single-plate shear connections were calculated via the following equation:

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

where θ_{u} is the rotation limit, L is the span length, d_{BG} is the depth of the connection bolt group, and δ_{u} is the axial deformation capacity of a single bolt row of the connection, known either from experiments or from computational modeling. Eq. (1) assumes full axial restraint. For the condition with no axial restraint, Eq. (1) reduces to $\theta_{\mathrm{u}} = 2\delta_{\mathrm{u}}/d_{\mathrm{BG}}$.

The dependence of Eq. (1) on both the span length L and the depth of the bolt group allows these important aspects of the system to directly influence the calculated rotation limit under column loss. In addition, since Eq. (1) was derived by comparing the axial deformations imposed on the connection directly against the axial deformation capacity, δ_{u}, Eq. (1) allows for consideration of more aspects of the connection geometry, such as the shear plate thickness, bolt diameter, and bolt thread-condition (i.e., threads included or excluded from the shear plate) by selecting appropriate values of δ_{u} for each condition. Using rotation limits calculated from Eq. (1), Weigand and Main (2016) showed that improved consistency could be achieved for single-plate shear connections, relative to the ASCE/SEI 41-13 or the UFC 4-023-03 acceptance criteria, by accounting for the influences of axial restraint, span length, and connection configuration. A similar approach will be presented in future publications for steel moment frames and reinforced concrete moment frames.

SUMMARY AND CONCLUSIONS

This paper presented acceptance criteria for alternative load path analysis from UFC 4-023-03 (DoD 2009), which have also been adopted in the GSA Guidelines (GSA 2013) and are used in structural design of U.S. military and civilian government buildings to mitigate disproportionate collapse. The acceptance criteria were compared with measured rotational capacities under column loss scenarios for steel gravity frames with single-plate shear connections and for reinforced concrete moment frames. Comparisons with seismic acceptance criteria from ASCE/SEI 41-13, from which the UFC criteria were adapted, were also presented.

For steel gravity frames with single-plate shear connections, the UFC 4-023-03 adopted a rotation limit for primary components that was reduced relative to the life safety acceptance criterion in ASCE/SEI 41-13, in recognition of the increased axial demands imposed on the connections in a column loss scenario. Comparison with the experimental data confirmed that such a reduction was warranted, as the ASCE/SEI 41-13 life safety acceptance criterion was non-conservative for all 13 of the connection tested under column loss, while the UFC acceptance criterion for primary components was conservative for all of the connections. However, axial demands are not directly considered in the UFC rotation limits, and consequently, the level of conservatism of the acceptance criteria varied widely, with measured rotational capacities in some cases of more than double the acceptance criterion for primary components. Computational analyses also indicated that the presence of a composite slab could cause the rotational capacities of some connections to fall below the UFC acceptance criterion for primary components. A new approach proposed by Weigand and Main (2016) for defining acceptance criteria would allow for improved risk-consistency by directly accounting for the combination of axial and rotational demands in a column loss scenario and enabling consideration of factors such as the influence of a composite slab.

For reinforced concrete moment frames, the UFC 4-023-03 adopted rotation limits for primary and secondary components that were increased relative to the life safety and collapse prevention acceptance criteria, respectively, in ASCE/SEI 41-13. For peak loads associated with flexural-arching action, comparison with the experimental data showed that this increase was not warranted. All of the rotational capacities associated with flexural-arching action fell below the UFC upper-bound rotation limit for primary components, whereas the ASCE/SEI 41-13 life safety acceptance criterion was generally conservative for flexural-arching action. The experimental data showed that the increased rotation limit was warranted for peak loads associated with catenary action, for which the UFC acceptance criterion for primary components was always conservative. However, the level of conservatism varied widely, with measured rotational capacities of more than five times the acceptance criterion in some cases. Importantly, it was noted that in some cases flexural-arching action may be the only response mechanism available for resisting disproportionate collapse. The experimental data showed that catenary action was not developed for span-to-depth ratios of less than six. Axial restraint of the beams is also required for the development of catenary action, which is not provided in cases such as a corner column loss scenario.

DISCLAIMER

The policy of the National Institute of Standards and Technology (NIST) is to include statements of uncertainty with all NIST measurements. In this paper, however, measurements of authors outside of NIST are presented, for which uncertainties were not reported and are unknown.

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