New Steel Gravity Connection Details for Enhanced Integrity

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ABSTRACT

Recent large-scale tests have found that Steel Gravity Framing Systems (SGFSs) with conventional connections may not be adequate to support the specified gravity load combinations under column removal scenarios. To address this issue, detailed connection models were validated against experimental results and then used to investigate alternative gravity connection details to enhance the integrity of SGFSs. A procedure was developed to design new single plate shear connection details with multiple columns of bolts so that they had design shear capacities equal to or greater than corresponding conventional configurations. Analyses of the connections under column removal showed that improvements in both connection strength and deformation capacity can be achieved using the multi-bolt-column configurations.

INTRODUCTION

Both large-scale experiments (Johnson et al. 2014, 2015) and computational studies (Sadek et al. 2008; Alashker et al. 2010; Main and Sadek 2012) have shown that steel gravity framing systems (SGFSs) may be vulnerable to collapse when subjected to column removal. In order to support the gravity loads after a column is removed or damaged, the system must form an alternate load path to redistribute the load that was previously supported by the lost or damaged column. Two primary modes exist by which SGFSs can form an alternate path to resist gravity loads under column removal: composite flexural arching and catenary action. At relatively small vertical deflections, composite flexure and compressive arching action dominate the system response. This mode depends on the composite action developed between the slab and the framing members, and may be particularly sensitive to the concrete slab details such as the slab thickness, reinforcement details, number and diameter of the shear studs, attachment method between the steel deck and the framing (e.g., puddle welds), and steel deck sidetap details.

If the gravity loads cannot be supported by arching action, then the concrete deck and gravity connections may be subjected to large rotation and axial extension demands as the system undergoes large vertical deflections. Under these catenary-type deformations, the system may be less sensitive to the concrete slab details and the
gravity connections and steel framing provide a significant contribution to the integrity of the SGFS.

Several significant research efforts have helped to advance the understanding of the integrity of SGFSs with conventional gravity connections, particularly under column removal scenarios. Jahromi et al. 2012 tested a half-scale floor system under interior and edge column removal scenarios. Oosterhof and Driver 2012 and Jamshidi and Driver 2014 tested steel gravity connection sub-assemblies under column removal without and with the inclusion of a composite concrete deck, respectively. Bare-steel interior column sub-assemblies with bolted single plate shear (i.e., “shear tab”) and bolted WT connections were tested under column pushdown by Thompson 2009 and Friedman 2009, respectively. Other interior column sub-assemblies with composite concrete deck and gravity connections were tested by Yang and Tan, 2013.

In addition, a collaborative research project was concluded in 2014 with four column removal tests on a half-scale SGFS. The specimen had steel frames, shear tab and bolted angle connections, and included a concrete slab on steel deck (Johnson et al., 2014). The project also included tests of bare-steel shear tab and bolted angle connections (Weigand and Berman 2014, 2015) and composite slab coupon tests (Francisco et al. 2014).

In order to address the potential vulnerabilities of SGFSs to column removal, and building on the results and observations of the studies listed above, detailed FE connection models were developed by the authors to investigate alternative gravity connection details to enhance the integrity of SGFSs. The new details included (i) shear tab connections with closely-spaced large diameter bolts, (ii) shear tab connections with multiple columns of bolts, and (iii) a retrofit strategy for existing SGFS connections that included slotted plates welded to the column flange and bolted to the beam flanges. This paper describes the new multi-bolt-column configurations (i.e., “reconfigured connections”) and presents analysis results from the four-bolt reconfigured connection case. The results show improvements in both the connection load and deformation capacity under column removal, relative to the conventional configuration.

DETAILED GRAVITY CONNECTION SUB-ASSEMBLAGE MODELS

Prior to investigating the alternative gravity connection details to enhance the integrity of SGFSs, detailed models of conventionally-configured connections were created and validated against experimental results from Weigand and Berman 2014. Each connection sub-assemble model consisted of a finely meshed assembly of connection components (e.g., bolts, shear plate, etc). Fig. 1 shows a schematic overview of a sub-assemble model for a conventional three-bolt shear tab connection. The simulations were conducted using ABAQUS-STANDARD version 6.12-3 (Simulia, 2012), a general purpose commercially available finite-element (FE) software package. First-order 8-node solid continuum elements with reduced integration were used for all connection components.
Figure 1 - Detailed connection sub-assemblage model overview.

The model geometry included a column stub and a 457 mm (18.0 in) segment of the beam stub. The remaining length of the beam stub had few deformations and was modeled as rigid. The bolts were modeled as cylindrical with nuts rigidly connected to the bolt shaft, and threads on the bolts were not included. The column stub flange opposite the connection was fixed, and out-of-plane lateral restraint was provided at the beam stub flanges. The shear plate welds were modeled as rigid. Surface contact definitions were included between all connection components expected to come into contact in the analysis.

The steel material properties used in the simulations were determined based on measured plate tension coupon data from Weigand 2014 and bolt tension test results from Kulak et al. 2001. The estimated uncertainty in the measured coupon data was ±1 %, based on the repeatability of the measured coupon data (Weigand 2014). For input into ABAQUS, the material curves were specified in terms of true stress and true plastic strain. Fig. 2 shows the true stress-true plastic strain curves used in the simulations.

The material curves were projected as perfectly-plastic after reaching their ultimate plastic strains and ABAQUS damage algorithms (Simulia 2012) were used to model the softening responses of the bolts and shear plates. Calibration of the bolt and plate responses with different mesh densities was achieved by adjusting the ultimate strain values (i.e., the mean nucleation strain at material fracture) at which damage initiated as a function of their element sizes.
The analyses were conducted in two steps. In the first step, pretension was applied to the bolts by uniformly decreasing their temperatures (i.e., the bolt material definition included a non-zero coefficient of isotropic thermal expansion, and the bolt temperatures were decreased to achieve the desired pretension). The second step included application of the rotation and axial extension displacement demands to the beam end reference point, as shown in Fig. 1. The demands exactly mirrored those applied to the physical connection sub-assemblages in Weigand and Berman 2014, on which the model was based.

Validation of the presented modeling approach is not included in this paper. However, a detailed comparison of the model results with connection sub-assemblage data is available in Weigand 2014. Based on that comparison, the model was able to adequately capture the connection behaviors and force-displacement response.

**SHEAR TAB CONNECTION CONFIGURATIONS WITH MULTIPLE COLUMNS OF BOLTS**

To address one of the critical vulnerabilities of SGFSs subjected to column loss, the connection sub-assemblage simulations were extended to consider alternative gravity connection details with enhanced integrity. New shear tab connection configurations with multiple columns of bolts were designed so that they had design shear capacities greater than corresponding conventional configurations tested in Weigand and Berman 2014, using connection limit state equations from the Steel Construction Manual (AISC 2010). Fig. 3 shows an example conventional configuration with its corresponding multi-bolt-column configuration. The conventional connection had a single vertical line of three 19.1 mm (3/4 in) diameter A325 bolts and a 9.53 mm (3/8
in) thick A36 shear plate. The reconfigured connection used four 22.2 mm (7/8 in) diameter A325 bolts and had an 11.1 mm (7/16 in) thick A36 shear plate. The reconfigured connection exceeded the shear capacity of the conventional connection by 8%. The column and beam stub sections used in the reconfigured connection were the same as those used in the conventional connection.

The reconfigured connection shown in Fig. 3 was subjected to the same displacement demands as were used in the conventional connection simulation. In the following section, results from the 4-bolt multi-bolt-column connection simulation are presented.

**RESULTS AND DISCUSSION**

Fig. 4(a) shows a comparison of the vertical force-displacement response of the 4-bolt reconfigured connection simulation to that of a tested 3-bolt conventional connection. The reconfigured connection analysis was terminated due to failed convergence at the first instance of element erosion in one of the bolts. The reconfigured connection achieved a 26% larger simulated vertical displacement (where “simulated vertical displacement” refers to the vertical displacement at the missing column, which was simulated in the connection experiments (see Weigand and Berman 2014)), a 62% larger axial deformation, and a 123% larger vertical force at connection failure than did the conventionally configured connection.
The increased deformation capacity of the reconfigured connection was due in part to an expanded distribution of yielding and deformations in the shear plate and beam web, relative to the conventional connection. Fig. 4(b) compares the deformed meshes of the shear plates from simulations of the conventional and reconfigured connections at their peak vertical loads. The areas shaded as grey in the simulation meshes correspond to elements that had reached stresses in excess of the plate yield stress at their integration points. While the shear plate in the conventional connection yielded mostly in the vicinity of the bolt holes, the shear plate in the reconfigured connection had more uniform and widespread yielding across the entire plate depth. As a result of the greater proportion of the connection deformations occurring in the shear plate and beam web, the shear deformations in the bolts were effectively reduced. This allowed the bolt group to reach a more uniform tensile configuration (i.e., a configuration where all of the bolts resisted load primarily along the longitudinal axis of the beam) prior to failure of the first bolt.
ESTIMATING THE BARE-STEEL FRAMING CONTRIBUTION TO THE SGFS RESISTANCE

The contributions of the framing and gravity connections to the vertical resistance of a SGFS were estimated as a static vertical load intensity, \( w \), using Eq. (1) and the connection sub-assemblage data and numerical simulation results:

\[
w = \frac{1}{L^2} \left( \sum_{i=1}^{n_{\text{cont}}} \frac{2V_i}{\cos^2 \theta_i} + \sum_{j=1}^{n_{\text{cant}}} \frac{2V_j^\theta}{\cos^2 \theta_j} \right).
\]  

(1)

\( L \) is the system span (assuming square bays), \( n_{\text{cont}} \) is the number of framing members that span “continuously” through the unsupported column (i.e., that can develop tension), \( V \) is the vertical force at the column face developed by the connections, \( n_{\text{cant}} \) is the number of framing members not continuous through the unsupported column (i.e., that behave as cantilevered and cannot develop tension), \( \theta_i \) and \( \theta_j \) are connection rotations, and \( V^\theta = V - T \sin \theta \) is introduced as the proportion of the vertical force at the column face due to the connection moment resistance only. A schematic of the application of Eq. (1) to a 9.1 m (30.0 ft) span prototype floor system (described in more detail in Weigand et al. 2012), is shown in Fig. 5. The connections for the different framing member types were selected from the configurations tested in Weigand and Berman 2014, so that their orientations were appropriate to the system geometry (see Fig. 5).

Eq. (1) was used to approximate the framing contributions to the SGFS resistance for four individual column removal scenarios for the prototype system: a corner column, edge column parallel to the intermediate framing, edge column perpendicular to the intermediate framing, and interior column. Fig. 6(a) shows the resulting SGFS framing vertical load contributions as vertical load intensities normalized by the ASCE 7-10 gravity load combination for extraordinary events (1.2D + 0.5L (ASCE 2010)) for the prototype SGFS with conventional gravity connections. Fig. 6(b) shows the same normalized vertical load intensities for the prototype SGFS with substituted multi-bolt-column shear tab connections. The expected gravity load intensity on the SGFS (i.e., 1.05D + \( L_{\text{survey}} \) (Main and Sadek, 2012)) is also shown as a grey dashed line.
Fig. 6(a) shows that the contributions of the bare-steel framing with conventional connections alone were not sufficient to support the ASCE 7-10 gravity load combination for extraordinary events for any of the four column removal scenarios. However, Fig. 6(b) shows that the improved performance of the reconfigured connections allowed the bare steel framing to exceed the ASCE 7-10 gravity load combination for extraordinary events for the interior column removal case, and to exceed the expected gravity loading on the system for the edge column removal case perpendicular to the intermediate framing.

Figure 5 – Schematic of SGFS prototype with conventional (grey) and reconfigured (green) vertical connection responses.

Figure 6 – Normalized Vertical Load Intensities for Experimental Prototype, with (a) Conventional and (b) Reconfigured Shear Tab Connections.
The peak values of the bare-steel prototype system capacity estimates demonstrate that for the column removal cases where tensile catenary action can be expected to develop in the steel framing (i.e., the interior and edge column removal perpendicular to the intermediate framing cases), the connection reconfiguration strategy significantly enhanced the framing contribution to the system capacity. Moreover, the framing contribution was enhanced to an extent that, at least under pseudo-static column removal, the expected gravity loading on the system could be carried by the framing and gravity connections alone. However, for the edge column parallel to the intermediate framing and corner column cases, where tension cannot be developed, the ASCE 7-10 gravity load combination for extraordinary events could not be supported with either conventional or reconfigured connections. Due to the cantilever-like behavior of the framing for those cases, the gravity connections could support little vertical load due to their limited moment resistance. Thus, for edge column parallel to the intermediate framing and corner column removal cases, moment-resisting connections may be required for the framing alone to carry the integrity gravity load combinations.

**SUMMARY AND CONCLUSIONS**

New single plate shear (i.e., shear tab) connection details with multiple columns of bolts, designed to have shear capacities equal to or greater than corresponding conventional configurations, were shown to offer improvements in both connection strength and deformation capacity relative to conventional configurations. The improved connection performance was also shown to significantly enhance the steel framing contribution to the SGFS integrity for column removal scenarios in which the framing could be expected to develop catenary action (i.e., tension in the framing members).

For the corner column and edge column parallel to the intermediate framing cases, all or most framing is not continuous through the unsupported elements and the capacity depends on the moment strength of the connections. This occurs because no axial tension can be developed in the framing, as it can in the interior and edge perpendicular cases. For the cases where no axial tension can develop, moment resisting connections would be required for the framing alone to support the integrity gravity load combinations.

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REFERENCES


