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Behavior of conventional and enhanced gravity connections subjected to column loss

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ABSTRACT

Gravity frames in steel buildings use simple connections designed to support gravity loads primarily in shear. Under column loss scenarios, large axial loads several times the shear demand can develop at these connections. Recent work has shown that these connections may not be capable of resisting these loads. In this paper, gravity connections studied experimentally under column loss scenarios are presented. Ten specimens of five different connection types were initially tested, including all bolted double angles, welded-bolted double angles, conventional and extended shear tab connections. Because the capacity of these connections was significantly below the required demand given by the ASCE 7 extreme event load combination (1.2D + 0.5L), two enhanced connections were developed and tested. These two connections consisted of a shear tab and a double angle connection, both reinforced with tension plates connecting the beam flanges to the column flanges. Of these two connections, the enhanced shear tab was capable of resisting the required demand.

1. Introduction

In recent years, a number of studies on steel gravity framing systems have explored their potential to support gravity loads under column loss scenarios. These have included large-scale experimental studies [10,12] and computational investigations [3,13]. The research has been complemented by component testing of bare-steel connections [8,14, 15,17,18] and composite connections [11,22]. The literature has confirmed that gravity framing systems have some inherent robustness, but also revealed that conventional details may not have sufficient resistance for demands due to column loss.

The robustness of gravity framing systems was the focus of a collaborative investigation that explored connection behavior, the contribution of the composite slab, and system response [21]. Component tests and computational investigations of the concrete on metal deck floor slab were full-scale [5–7], as were most of the bare-steel connection tests [17,18]. A number of half-scale connection tests were also conducted in support of the half-scale system tests that followed [12,16].

The vulnerability of gravity framing systems with conventional connection and composite slab details motivated development of enhanced gravity connections by Weigand and Berman [19,20]. One connection was a shear tab with an additional bolt in a separate, vertical 'column'. Another enhanced detail included plates with slotted holes at the beam flanges and was intended as a potential retrofit option. Computational simulation results confirmed improved strength and deformation capacity for the shear tabs with multiple columns of bolts, as well as for the enhanced steel gravity framing system [19]. The retrofit strategy also proved effective in simulations of column removal [16,20].

The overarching goal of this study was to add to the body of knowledge with respect to gravity frame connections and their behavior under large rotational and axial demands. The tests conducted would confirm existing connection test data as well as add new knowledge about enhanced connections for robustness of gravity frames. Specific objectives were:

(1) to provide additional knowledge (e.g., rotational capacity, level of catenary loads, failure modes) of the behavior of conventional gravity connections for evaluation of robustness; and
(2) to design and test enhanced gravity connections capable of resisting the extraordinary load combination demands [4].

The conventional connection specimens were modeled after the half-scale tests conducted by Johnson et al. [12]. The enhanced connection specimen details were inspired by the multiple-row shear tab and the retrofit connections studied analytically by Weigand [16]. The loading scenario, the scaling procedure, the test setup, the test specimens, the instrumentation and the loading protocol are explained in the following sections. Results are then presented and discussed.

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2. Experimental study

2.1. Load scenario and half-scale design

As for the large-scale system test by Johnson et al. [12], the full-scale prototype was a gravity framing system with 9.15 m bays and 3.05 m filler beam spacing. Connections were sized for the gravity loading and resulting girder-to-column forces at an edge column, identified in Fig. 1, without consideration for column loss scenarios. The connection demand was calculated based on a 4.6 kPa dead load and a 2.4 kPa live load. An additional cladding load of 2 kN/m was used for the exterior girders. The connection demand resulted in a design shear force of 140 kN for the full-scale gravity load scenario, and translated into a 35.1 kN demand for the half-scale designs, because the tributary area was one quarter of the full-scale tributary area. Half-scale elements were chosen with dimensions as close as possible to the half-scale ratio. These dimensions included the thickness of the beam web, the depth of the beam, the thickness and width of the beam flange. W21x50 [2] girders in the full-scale prototype building were scaled down to W8x10 sections in the half-scale specimens. Plates in shear tab connections were reduced to half the thickness and half the spacing between bolts, resulting in a quarter of the shear area of the full-scale prototype. Likewise, bolt diameter was halved, resulting in a quarter of the bolt area.

With the plate shear area and the bolt area at one quarter of the full-scale specimens, the demand/capacity ratios of the half-scale specimens correspond to those of the full-scale connections; as explained above, the half-scale demand was also one quarter of the full-scale connection. In the design of the half-scale connections, full-scale connections were designed first, and then half-scale connections were designed to follow the progression of limit states of the full-scale connections as closely as possible. In all cases the controlling limit state for the half-scale specimen matched the controlling limit state of the full-scale connection.

Although the connections were not designed for column loss scenarios, the demands under such a loading condition was also estimated and used to verify if the connections tested would have sufficient capacity. This demand was based on the extraordinary load combination [4] 1.2D + 0.5L. For the aforementioned dead and live load demands, but neglecting the cladding load, the edge column is subjected to a vertical force of 70.2 kN, or 23.4 kN from each beam/girder supported. Because the sub-assemblies tested only considered the two girders (i.e., do not include the transverse filler beam), the target demand for the experiments was taken as 46.7 kN total, assuming that the transverse beam would contribute an equal amount. It should be noted that the capacity of the different configurations studied, as explained in the following sections, was determined by applying a point load directly on the missing column; however, the actual demand on the missing column arises from floor loading (i.e., point loads and distributed loads) acting on the two girders and filler beam attached. The effects of this approximation should be negligible as studied by Main and Sadek [13].

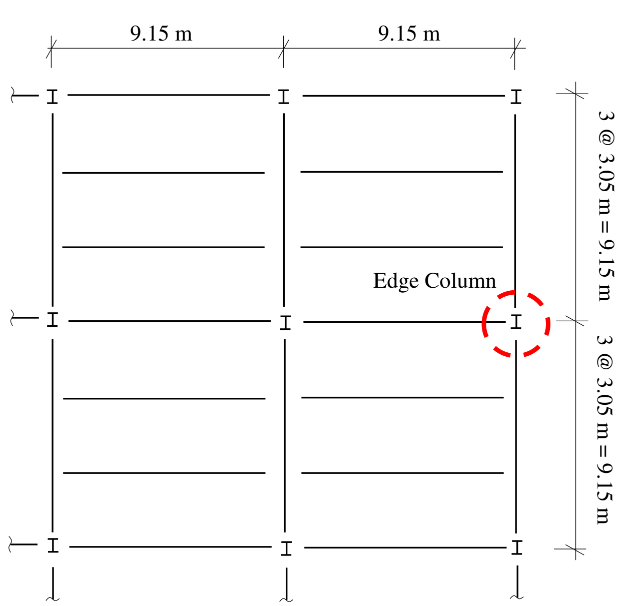

Fig. 1. Plan view of the full-scale prototype steel gravity framing system used for calculating demands.

2.2. Test setup

Specimens were tested in a self-reacting frame designed to remain elastic and to have negligible deformations under predicted loads. The self-reacting frame, shown in Fig. 2, is composed of a 12.2 m long A992 W24x162 beam, attached to two A992 W12x79 columns. Column base plates (A36) were bolted to anchor rods cast inside 0.76 m × 1.52 m × 0.46 m concrete foundation blocks. Diagonal braces (2 L 127 × 89 × 13 (L5″ × 3 ½″ × ½″) A36) were added to the frame to reduce deflections in the columns due to the expected large tension loads (due to catenary action) developed at the beam specimens. 12.7 mm A36 plate stiffeners were added to the test frame at all locations where concentrated forces were expected.

Specimens were restrained against out-of-plane deformations by four lateral braces. These braces, shown in Fig. 3, were made from 100 mm × 100 mm Southern Pine lumber. To reduce the possibility of friction from the contact of the specimen and the wood braces, a 1.5 mm smooth medium density fiberboard (MDF) panel was installed at the surface of the 100 mm × 100 mm brace. In addition, lubricant was added to the panel surface.

2.3. Specimens

Twelve half-scale specimens consisting of two 4.57 m center-to-center spans with a missing center column were tested. These specimens were composed of A992 W8x10 beams attached to A992 W8x24 column stubs (t_w = 4.3 mm) by means of shear tabs or double angle connections, with or without enhancements. Table 1 shows a test matrix describing the 7 different connection types tested along with the name assigned, the connection type, the number and type of bolts, and the number of specimens tested. Specimen names follow the format: connection type – single or double row and number of bolts – bolt diameter – enhanced (if applicable). For example, ST-S4-12.7-E is a shear tab with a single row of four bolts, 12.7 mm bolt diameter, enhanced with flange plates with slotted holes.

Connections 1, 2 and 3 used 9.5 mm hexagonal cap screw J429 grade 5 bolts. The minimum ultimate tensile strength (F_u) of these bolts is 827 MPa which is identical to that of A325 bolts. This bolt grade was also used in the 3-bay by 3-bay test performed by Johnson et al. [12]. 12.7 mm diameter A325 bolts were used for connections 4, 5 and 6. Connection 7 used 9.5 mm J429 grade 5 bolts at double angle connections and 12.7 mm diameter A325 bolts at the enhancement tabs. The initial intent was to install snug-tight bolts in all specimens. However, the smaller diameter bolts ruptured easily during this procedure. Therefore, for shear tabs and double angles, a 27 N-m torque was applied to all 9.5 mm diameter bolts and 67.8 N-m to all 12.7 mm diameter bolts to maintain uniform torque throughout and to prevent rupturing of the smaller 9.5 mm diameter bolts during installation. This torque resulted in approximately 50% of the minimum bolt pretension specified in the AISC Specifications [1]. Bolts used at enhancement plates were the larger, 12.7 mm diameter bolts, and were installed snug tight to avoid introducing significant rotational resistance to the connection.

2.3.1. Double angle connections

Connections BA-S3-9.5 and WA-S3-9.5 used double angle connections. Both types are similar, the only difference being that BA-S3-9.5