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Flexural buckling of extended shear tab connections under gravity induced shear force

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Abstract
Extended shear tab connections have been widely used to connect simply supported beams to the flange or web of supporting columns, as well as the web of steel girders. The American Institute of Steel Construction (AISC) introduced the use of extended shear tab connections in 1992 and first published their design procedure in 2005. Unlike conventional shear tab connections, flexural buckling of the shear plate was observed as one of the governing failure modes of extended shear tab connections in past experiments. The AISC adopted a design equation for doubly coped beams to compute the flexural buckling strength of the shear tab. This paper presents the findings from an experimental research program, conducted at McGill University, related to the performance of extended shear tab connections under gravity induced shear force. Emphasis is placed on the flexural buckling strength of such connections. Seven full-scale tests were conducted and flexural buckling of the shear plate dominated the behavior of such connections. The beam sizes ranged from W310 to W690; the shear tab eccentricities that were examined as part of the experimental program ranged from 190 to 318 mm. It is shown that the current AISC design equation to compute the flexural buckling strength of extended shear tab connections overestimates the predicted flexural buckling strength of the shear plate.

1. Introduction

Among the current types of simple shear connections, the single plate shear tab has been widely used due to its ease of fabrication and erection. As shown in Fig. 1, this connection consists of a steel plate which is shop welded to the supporting girder or column and bolted to the supported beam in the field (Astaneh-Asl et al. 1989). The configuration of a shear tab connection depends largely on the location and geometry of the supported and supporting structural members. The conventional configuration of a shear tab is used to connect a beam to the flange of a column. In the conventional shear tab (Fig 1a) the distance between the weld line and the single vertical row of bolts is less than 89 mm (3.5 inches) (AISC 2011). The extended shear tab is implemented to connect the supported beam to the flange or web of supporting columns, as well as to the web of steel girders. The increased length of the extended shear tab allows the beam to be connected to

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the web of a column or the girder web (Figs. 1d-f) without coping the beam flanges, which can be an expensive and time-consuming procedure. As shown in Fig. 3a, horizontal steel plates (stiffeners), welded to the column flanges, are used to increase the stiffness and strength of beam-to-column web shear tab connections. Regarding beam-to-girder shear tab connections, the shear plate can only be welded to the web of girder (Fig. 1d) or be extended to either the top flange (Fig. 1e) or both top and bottom flanges (Fig. 1f).

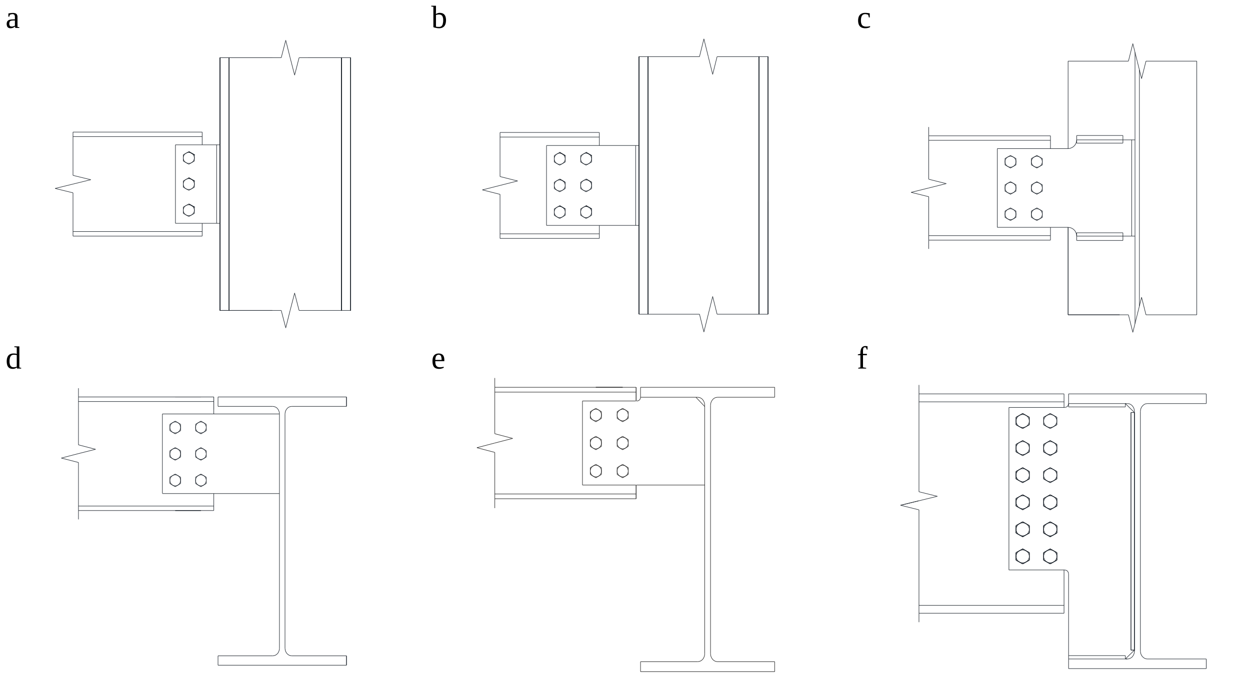

Fig. 1- Shear tab connections: a) Conventional beam-to-column shear tab connection, b) Extended beam-to-column shear tab connection, c) Extended beam-to-column shear tab (stiffened), d) Extended beam-to-column shear tab (unstiffened), e) Extended beam-to-girder shear tab (partial depth-stiffened), f) Extended beam-to-girder shear tab connection (full depth-stiffened)

Although the AISC Manual of Steel Construction has illustrated the usage of extended shear tabs since 1992, it contained for the first time in 2005 a design procedure (AISC 2005, Muir & Hewitt 2009). The AISC design method considers the combined flexural and shear yielding of the shear plate, bolt bearing, flexural buckling of the shear plate, shear rupture of the shear plate, block shear rupture of the shear plate, weld tearing, and bolt shear as failure modes of extended shear tab connection. To obtain a ductile response of the shear tab connection, the plate thickness and the weld throat are proportioned to develop yielding of shear plate prior to bolt shear and weld tearing, respectively. Regarding the flexural buckling of the shear plate, the AISC design method implements equations corresponding to the flexural buckling resistance of a doubly coped beam (Eq. 1) (Cheng et al. 1984).

$$V_r = \phi_b F_{cr} S_{net} \tag{1}$$

Where ϕ_b is the resistance factor for buckling (ϕ_b=0.9), S_{net} is the net section modulus of the coped section (here section modulus of the shear plate), and F_{cr} is the buckling stress of the shear plate. F_{cr} is determined based on Eq. 2 if the length of the coped section (here the horizontal distance between weld line and closest vertical row of bolts), c, is less than twice the depth of the beam, d, and the depth of the compression cope, d_c (here $d_c = \frac{d - d_p}{2}$), is less than or equal to 0.2d.

$$F_{cr} = 0.62\pi E \frac{t_w^2}{c h_0} f_d \tag{2}$$

Where E is the modulus of elasticity, t_w is the thickness of the web of the coped beam (here the thickness of the shear plate), h_o is the depth of the coped section (here the depth of shear plate), and f_d can be calculated based on Eq. 3.

$$f_d = 3.7 - 7.5(\frac{d_c}{d}) \tag{3}$$

Furthermore, the buckling stress of a shear plate can be conservatively calculated by implementation of the classical equation for plate buckling, Eq. 4.

$$F_{cr} = QF_y \tag{4}$$

Where Q is determined based on the slenderness of the coped section (here shear plate) based on Eq. 5.

$$\left\{\begin{array}{ll} 1 & \lambda \le 0.7 \\ 1.34 - 0.486\lambda & 0.7 < \lambda \le 1.4 \\ (\frac{1.30}{\lambda^2}) & 1.41 < \lambda \end{array}\right\} \tag{5}$$

Where λ is the slenderness of the shear plate, calculated based on Eq. 6.

$$\lambda = \frac{h_0 \sqrt{F_y}}{10 t_w \sqrt{475 + 280(\frac{h_o}{c})^2}} \tag{6}$$

Where F_y is the yield stress of structural steel. Other parameters are as defined in Equations 1 to 5.

The current AISC design procedure for extended shear tabs is based on a limited number of laboratory tests and finite element simulations of extended shear tabs under shear force. On the basis of laboratory tests on shear tab connections under shear force, Sherman and Ghorbanpoor (2002) determined the out of plane distortion of column web (web mechanism) as the primary failure mode of unstiffened shear tabs connecting the beam to the column web. In addition, they determined shear plate yielding and shear plate twisting as governing failure modes of the stiffened beam-to-column web shear tabs. Furthermore, Thomas (2014) tested beam-to-column shear tabs under combined axial and shear forces and observed that column web yielding occurred. However, she determined that weld tearing and bolt fracture are the governing failure modes of unstiffened beam-to-column web shear tabs. On the other hand, she determined that shear plate buckling is the governing failure mode in stiffened beam-to-column web shear tabs. Baldwin-Metzger (2006) tested four beam-to-column flange extended shear tabs. These connections failed due to weld tearing and/or beam failure. It should be noted that their weld size was smaller than that recommended by the AISC design procedure.

Sherman and Ghorbanpoor (2002) observed that yielding and twisting of the shear plate were the governing failure modes for partial depth shear tabs in beam-to-girder shear tab connections. In addition, plate buckling was observed as the failure mode of full depth beam-to-girder shear tab connections. Furthermore, Goodrich (2005) tested full depth beam-to-girder shear tabs and observed plate buckling as the governing failure mode.
The main objective of the global research program is to evaluate and improve the current design practice for extended shear tabs. To study the behavior of extended shear tabs, full-scale laboratory tests were implemented. Laboratory tests provided a base point to understand the complex and nonlinear behavior of these connections. In addition, this research will rely on finite element simulation of shear connections. Finite element analyses, calibrated on the basis of the full-scale connection tests, will allow for a better comprehension of their behavior and the influence of various parameters for a wider range of loading and connection configurations. However, this paper is limited to a discussion of the laboratory test program.

2. Laboratory Testing

2.1 Test Configurations

To understand the behavior of extended shear tab connections, several full scale laboratory tests were conducted at McGill University (Fig. 2) (Marosi 2011, Hertz 2014). The connection configurations were designed in collaboration with practicing structural engineers to be representative of the current design practice in North America. Among these tests, seven specimens showed flexural buckling of the shear plate under loading. Two specimens were beam-to-column extended shear tab connections and five of them were beam-to-girder extended shear tab connections. The beam-to-column shear tabs varied in the depth of the shear tab, the number of bolts and the eccentricity between bolts and weld line. It should be noted that configuration BG2 (BG: Beam-to-girder extended shear tab, BC: Beam-to-column extended shear tab) were tested twice. First it was tested without consideration of the concrete slab; then a nominally identical connection was tested with restriction of the vertical displacement of the top flange of the girder to be representative of the presence of a concrete slab. The girder and column were steel ASTM A992 Grade 50 (F_y=345 MPa). An ASTM A992 Grade 50 beam was connected to an ASTM A572 Grade 50 shear plates using snug tightened ASTM A325 bolts. To weld the shear tab to the supporting girder or column, E71T (F_u=490 MPa) electrode was used through the flux-cored arc welding process with additional shielding gas (CO_2).

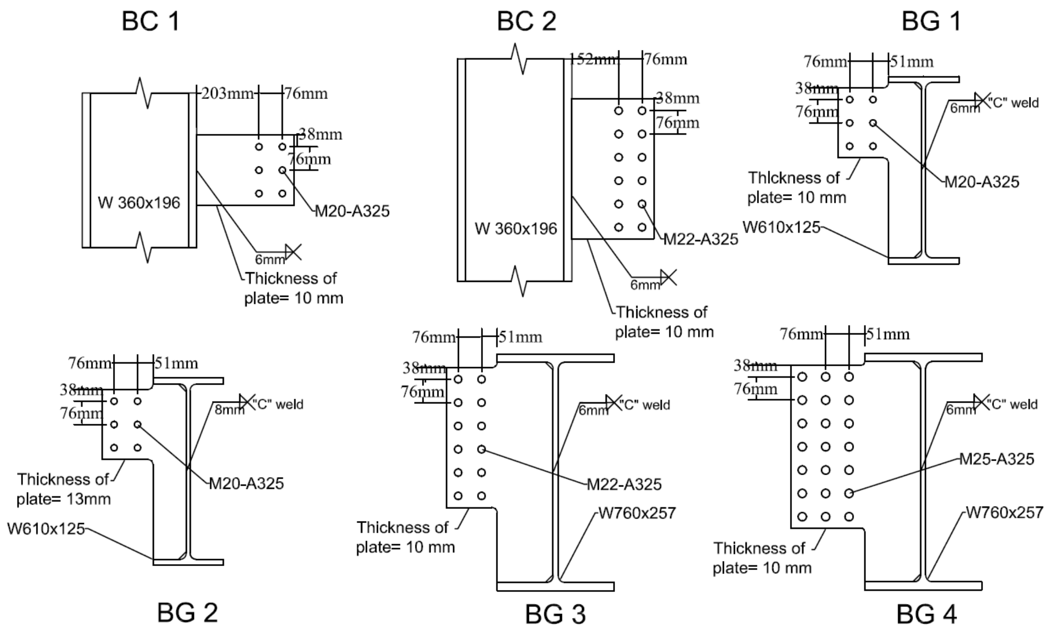

Fig. 2-Details of shear tab test specimens

On the basis of the AISC design procedure, the connection strengths corresponding to probable failure modes were determined (Table 1). The measured material properties of steel and the nominal properties of the bolts and welds were used to conduct the calculations.

Table 1-Predicted strength of shear tab test specimens

	Predicted Strength (kN)					
Failure mode	BC1	BC2	BG1	BG2	BG3	BG4
PB	---	---	---	---	---	535
PY	206	794	245	326	646	765
RN	482	909	482	643	909	996
BSH	225	1235	270	270	1074	1328
WT	233	1072	991	1239	1435	2227

PB=Buckling of shear plate, PY=Flexural and shear yielding of shear plate, RN=Rupture at net section of shear plate, BSH=Bolt shear, WT=Weld tearing

2.2 Test Setup, Instrumentation and Loading Protocol

The test setup consisted of a 12MN and a 445kN hydraulic actuator, a lateral bracing system for the steel beam, and supporting elements for the stub columns, or stub beams, as applicable. As shown in Fig. 3, the 445kN actuator located near the shear tab connection developed the main shear force in the connection. A half round steel cylinder, a roller and two steel plates were placed between the top flange of beam and the head of the 12MN actuator. The 445kN actuator, placed near the far end of the beam, was used to control the vertical displacement of the beam tip as well as the rotation of the connection. The lateral bracing system was implemented to restrict lateral displacement of the beam, without affecting its vertical displacement.

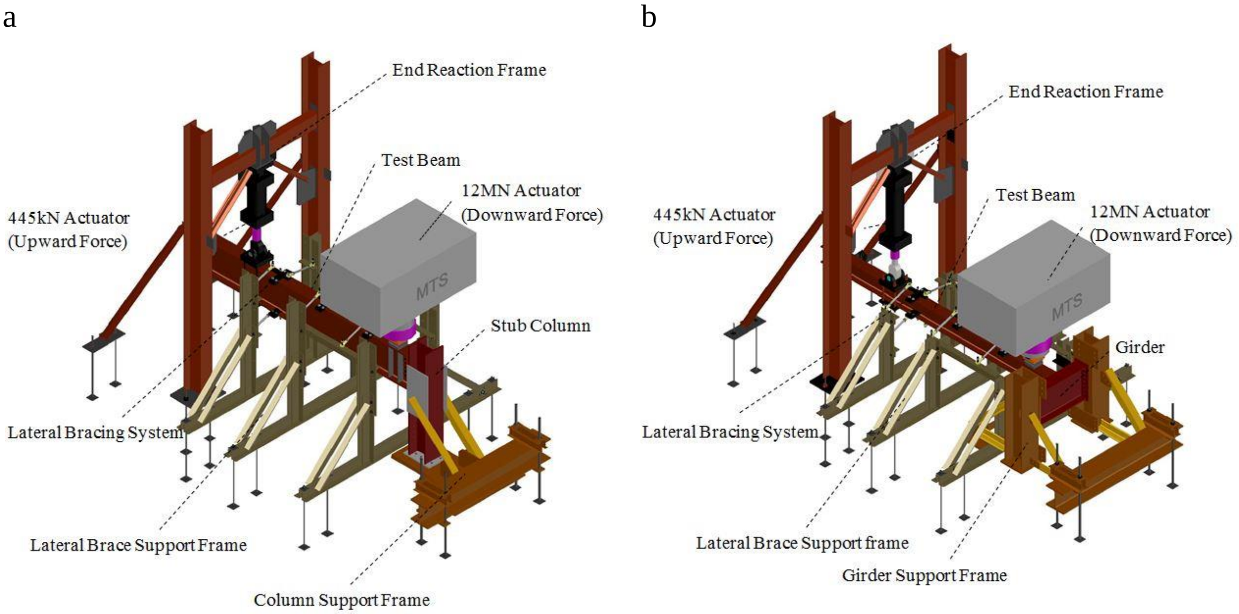

Fig. 3-Test setup for: a) Beam-to-column shear tab, b) Beam-to-girder shear tab

Strain gauges were installed on the shear plate and the girder web to detect yielding under the applied shear force and the developed flexural moment. String potentiometers were used to determine the vertical deformation of the girder, beam and shear tab. The lateral displacement of beam and shear plate were measured by using LVDTs. In addition, LVDTs were used to record the horizontal and vertical displacements of the column as well as the out-of-plane displacement of the girder. Inclinometers were used to measure the rotation of the beam, girder, column and shear tab (Marosi 2011, Hertz 2014).

To simulate the real situation that a gravity beam may experience in a single bay simply supported frame, a trilinear shear-rotation loading path, shown in Fig. 4, was used to load these specimens. The implemented loading protocol is based on that proposed by Astaneh-Asl et al. (1989). The probable ultimate shear resistance of the shear tab was assumed tobe achieved at 0.015 radians relative rotation of the supported beam with respect to the stub column or girder. This relative rotation is referred as the connection rotation in this paper. The probable ultimate shear resistance of the connection was the lowest resistance of the connection corresponding to its probable failure modes. The predicted resistances were calculated based on a resistance factor of 1.0 and probable material properties of $1.1F_y$ and $1.1\ F_u$ (Marosi 2011, Hertz 2014).

3. Test Results and Discussion

The shear force-rotation responses of the tested specimens are shown in Fig. 5. In addition, the results of the tests are summarized in Table 2. In this table the observed failure modes and their corresponding shear strength are reported. Furthermore, the observed buckling strength and the shear resistance of connections are compared with the expected values in Table 3. It should be noted that the dash line seen in Fig. 5 was representative of the stiffness variation observed at the beginning of the tests. This stiffness variation was due to the adjustment of the displacement rate of both actuators to achieve the desired stiffness of loading protocol.

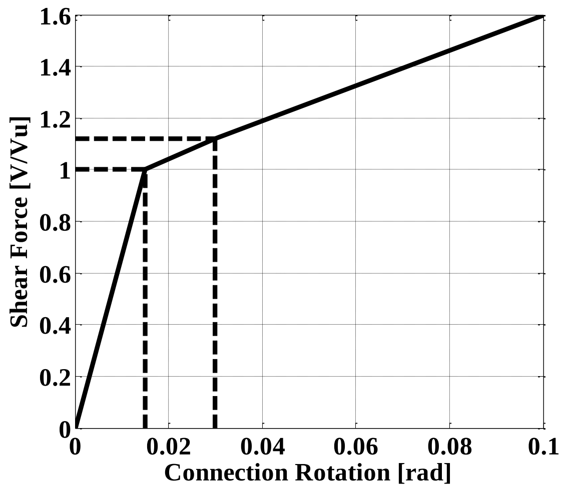

Fig. 4- Trilinear loading path

Table 2-Observed Failure modes of connections

	Observed Strength (kN)						
Failure mode	BC1	BC2	BG1	BG2	BG2-S	BG3	BG4
PB	210	550	100	200	150	350	290
PY	225	730	---	---	---	---	---
WT	240	830	---	---	---	---	---
GWY	---	---		250	270		
GWM	---	---	225	---	---	500	410

PY=Flexural and shear yielding of shear plate, PB=Buckling of shear plate, WT=Weld tearing, RN=Rupture at net section, GWY=Yielding of web of girder, GWM= Mechanism of girder web

3.1 Beam-to-Column Extended Shear Tab Connection

Flexural buckling was observed on the compressive edge of the shear tab connections as shown in Fig. 6a-b. The onset of buckling, however, did not result in a decrease of the connection resistance; rather the connections were able to tolerate greater shear load. This behavior can be attributed to the stress redistribution occurring at the shear plate due to the flexural behavior of the shear plate. It should be noted that the AISC design method overestimated the buckling strength of the connection 18% in average. Both Eqs. 2 4 showed F_{cr} value equal to F_y This implies that buckling did not occur prior to the yielding of shear tab connection. The buckling of the shear plate was followed by a combined flexural and shear yielding of the shear tab and ultimately by weld tearing. As shown in Tables 1 and 2, the observed combine flexural and shear yielding strength of the connections were close to the expected values. Initiation of weld tearing at the top of shear plate demonstrated that the interaction of flexural moment and shear force should be considered in the determination of the weld strength. This interaction may be considered by implementation of the Instantaneous Center of Rotation (ICR) method. Furthermore, the weld tearing behavior was characterized as being ductile. The resistance of the connection started to degrade when tearing of the weld propagated to a significant portion of its length. This ductile behavior is a result of the stress redistribution that occurs due to formation of a compression strut and tensile tie mechanism. In contrast, the predicted failure mode of bolt shear was not observed in the beam-to-column shear tabs. This behavior demonstrated that the actual shear strength of bolts was higher than the nominal material properties that were used to calculate the expected strength of the shear tab connections.

Table 3- comparison between predicted and measured strength of connection.

	Measured material properties		Experimental results			
Specimen	Predicted connection shear (kN)	Predicted failure mode (kN)	Buckling strength (kN)	Connection shear (kN)	Experimental to predicted buckling strength	Experimental to predicted connection shear
BC1	206	PY	210	240	1.02*	1.17
BC2	794	PY	550	1040	0.69*	1.31
BG1	245	PY	100	250**	0.41*	1.02
BG2	326	PY	200	520***	0.61*	1.60
BG2-S	326	PY	150	538***	0.46*	1.65
BG3	646	PY	350	500**	0.54*	0.77
BG4	535	PB	290	410**	0.54	0.77

*As expected buckling stress was equal to yield stress, the observed buckling strength was compared with expected connection shear.
**Test was stopped due to binding of the bottom flange of beam to the shear plate.
*** Test was stopped due to limitation of vertical displacement of lateral bracing system.

3.2 Beam-to-Girder Extended Shear Tab Connection

Flexural buckling was also observed in all extended beam-to-girder shear tab connection tests. As shown in Fig. 6c, buckling was seen on the neck of the shear tab where its depth increased to extend to the bottom flange of the girder. However, the shear tab connection was able to carry increased load after buckling. Regarding specimens BG1, BG2, and BG3, the AISC equations, Eq. 2 and Eq. 4, resulted in an F_{cr} value equal to F_y. This indicated that yielding occurred prior to the buckling of shear plate. As shown in Table 3, the observed buckling strength of specimen BG4 is approximately half the value predicted based on the AISC design equations. It should be noted that the presence of the girder flanges, and the effective reduction of the connection eccentricity, was not considered in the design procedure; a conservative assumption. This discrepancy demonstrated that the equations corresponding to the buckling strength of a doubly coped beam were not applicable to the extended beam-to-girder shear tab connections that were tested.

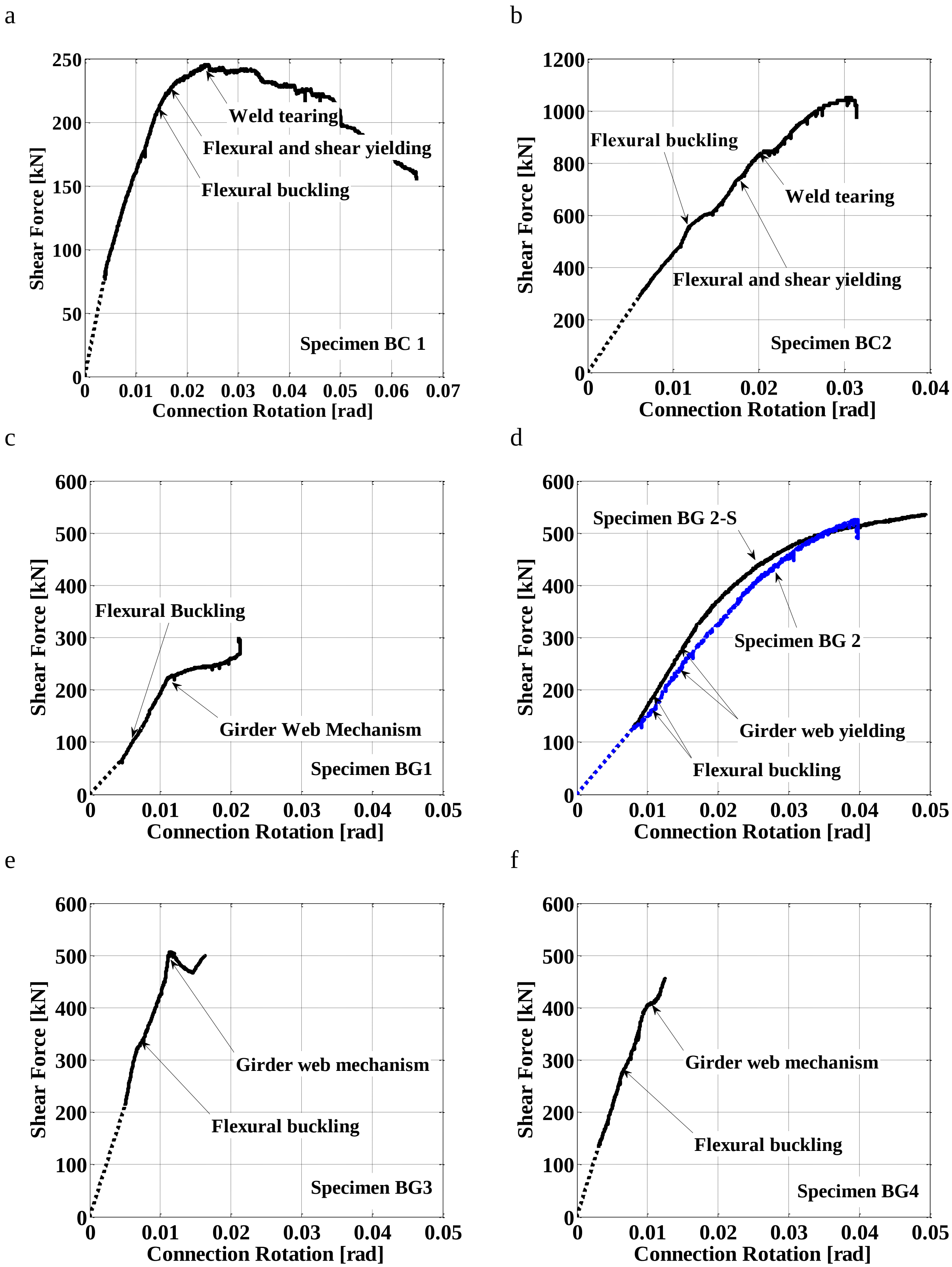

Fig. 5-shear force versus connection rotation of tested shear tab beam-to-column and beam-to-girder connections

Yielding of the girder web and a girder web deformation mechanism were observed after buckling of the shear plate. The girder web deformation mechanism is shown schematically in Fig. 7. The response to loading of specimens BG1, BG3, and BG4 demonstrated that the out of plane deformation of the girder web could be considered as a governing failure mode of the beam-to-girder shear tabs. Therefore, the out of plane resistance of the girder web should be considered as part of the design procedure. The girder web of specimens BG2 and BG2-S yielded, however, but the out of plane mechanism did not develop. Although the detailing of specimen BG1 and BG2 was the same, BG1 demonstrated small relative beam-to-girder rotation as compared with specimen BG2 (Figs. 3e-f). The absolute rotation of the two beams was in the same range, but the girder rotation of specimen BG1 was larger than that of BG2. The larger deformation of the girder web was attributed to the thinner shear plate of BG1 which lead to higher concentration of compressive stress on the girder web. This higher compressive stress resulted in larger deformation of girder web. A comparison between the buckling strength of BG1 and BG2 demonstrated that, as expected, the specimen BG2 which has a thicker plate, was able to carry higher loads than specimen BG1. Furthermore, comparison between the response of specimen BG2 and BG2-S showed the effect of the concrete slab on the reduction of the out of plane deformation of the web girder and consequently the rotation of the girder.

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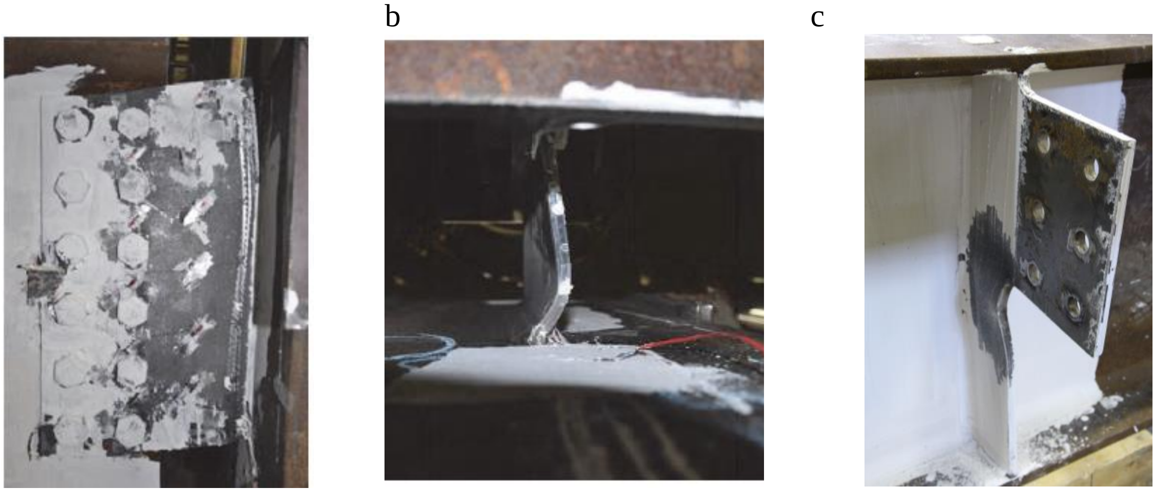

Fig. 6- The deformed shape of connections at the end of tests: a) For specimen BC2, the yielding and buckling of the shear plate, in addition to the weld tearing was observed, b) The buckled shape of shear tab from below beam for specimen BC2, c) The buckled shape of shear tab for specimen BG2-S

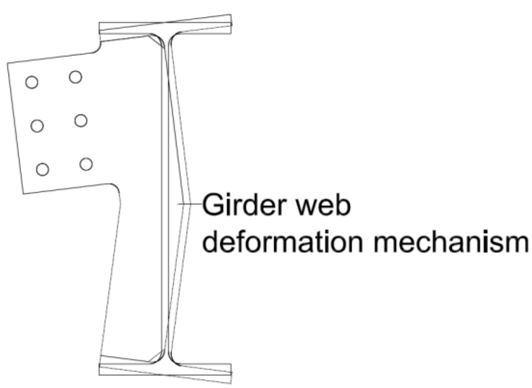

Fig. 7-Girder web deformation mechanism

4. Summary and conclusions

A series of two beam-to-column shear tab connections and five beam-to-girder shear tab connections were tested under simulated gravity induced shear forces. Flexural buckling was observed on the bottom edge of the shear tabs in all beam-to-column test specimens prior to the combined flexural and shear yielding of the shear tab. After comparing the observed and expected buckling strength based on the AISC equation for buckling of the doubly coped beam, it is concluded that this equation overestimates the buckling strength for beam-to-column shear tabs. Similar findings hold true for the extended beam-to-girder shear tabs.

Although the shear tab plate is extended to both top and bottom flanges of the girder, a web girder mechanism was observed as a primary failure mode in beam-to-girder shear tab connections. Therefore, the out of plane deformation of the girder web should be considered as a potential failure mode in the design procedure for extended beam-to-girder shear tab connections. On the basis of tests, the thickness and the height of shear plate adjacent to the connected beam, is directly related to the strength of the girder web mechanism. On the other hand, the eccentricity is inversely related to the web mechanism strength of shear tab connections. Restriction of the upward vertical deformation of the top flange of girders, representing the effect of the concrete slab, decreases the out of plane deformation of the girder web and consequently the corresponding girder rotation. The influence of these parameters will be the subject of future study by implementation of finite element simulations.

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