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DESIGN OF BOLTED CONNECTIONS SUBJECT TO TORSION AND SHEAR IN THE ULTIMATE LIMIT STRESS STATE

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Steel connections transferring axial and shear forces in addition to bending moment and/or torsional moment are widely used in steel structures. Thus, design of such eccentric connections has become the focal point of any researches. Nonetheless, behavior of eccentric connections subjected to shear forces and torsion in the ultimate limit state is still ambiguous. Most design codes of practice still conservatively use the common elastic analysis for design of the said connections even in the ultimate limit states. Yet, there are some exceptions such as the design method proposed by CAN/CSA-S16-14 which gives tabulated design aid for the ultimate limit state design of these connections based on an empirical equation that is derived for ¾ inch diameter A325 bearing type bolts and A36 steel plates. It was argued that results can also be used with a margin of error for other grade bolts of different sizes and steel of other grades. As such, in this paper, the performance of bolted connection subject to shear and torsion is experimentally investigated. The behavior, failure modes and factors affecting both are scrutinized. Twelve connections subject to shear and torsion with different bolts configurations and diameters are experimentally tested to failure. The accuracy of the currently available design equations proposed is compared to the outcomes of these tests.

Keywords: Eccentric connection, Shear center, Instantaneous center, Shear failure, Steel structures.

1 INTRODUCTION

The majority of steel connections are eccentrically loaded. Thus, design of these eccentric connections has become the focal point of many researches: for such connections, the moment-induced stresses must be taken into considerations besides the stresses induced due to normal and/or shear force. By large, design of bolted concentric connection subject to bending moment has been extensively investigated. In contrast, behavior and design of eccentric connections subjected to shear forces and torsion in the ultimate limit state are still ambiguous: current researches still focus on using the elastic analysis in its design. Most codes of practice (e.g. ECP 205 (2008) and BS EN 1993-1-1 (2005)) still use this elastic analysis for design of the said connections even in the ultimate limit states. Yet, there are some exceptions such as the design method proposed by CAN/CSA-S16-14 (2014) and AISC (2017) which give tabulated design aid for the ultimate limit state design of these connections based on an empirical equation that was derived for ¾ inch diameter Grade 4.8 bearing type bolts and A36 steel plates; it was argued that results can also be used with a margin of error for bolts of different sizes/grades and other type steel. As such, in this research, the performance of bolted connection subject to shear and torsion is experimentally investigated in order to either verify the currently (and barely) available limit states design methods or recommend new ones. An ongoing research experimentally scrutinizes the behavior, failure modes and factors affecting the capacity of these connections. Herein, the results of this experimental program are used to investigate the accuracy of the currently available limit state method proposed for by CAN/CSA S-16-14 (2014).

2 ELASTIC ANALYSIS

Figure 1 shows a schematic of a bolted connection subject to shear and torsion with bolts' shear areas and loads shown separately from the column and bracket plate. The eccentric load P can be replaced with the same load value acting at the bolts' centroid plus a couple M = PL, where L is the load eccentricity (Figure 1). As such, each bolt is assumed to resist an equal share of the load, which is given by $P_V = P/n$, where n is the number of bolts. Each bolt's force resulting from the couple M can also be assumed based on the distance between this bolt and the bolts' centroid. Based on this assumption, the forces acting on each bolt due to the couple M can be found from Eq. (1) (Sayed-Ahmed and Elserwi 2017),

$$P_{mx} = \frac{M \cdot X_i}{\sum_1^n \left(X_i^2 + Y_i^2\right)} \qquad and \qquad P_{my} = \frac{M \cdot Y_i}{\sum_1^n \left(X_i^2 + Y_i^2\right)}$$
$$d_i = \sqrt{X_i^2 + Y_i^x} \tag{1}$$

Where d is the distance from the centroid of the bolt to the bolts centroid. The total force acting on any bolt due to the shear force and the couple M is thus given by Eq. (2);

$$P_i = \sqrt{\left(P_{mxi} + P_{vx}\right)^2 \left(P_{myi} + P_{vy}\right)^2} \tag{2}$$

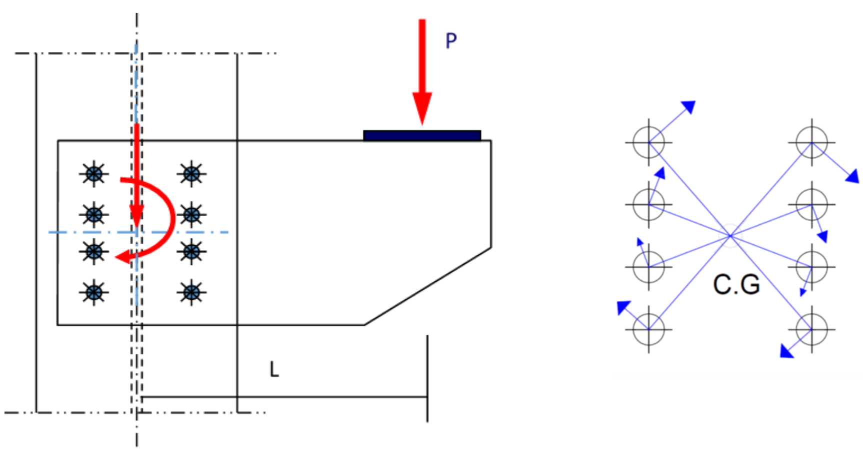

Figure 1. Parameters adopted in the elastic analysis of connections subject to shear and torsion.

3 INELASTIC ANALYSIS

A method of analysis for eccentric connections subject to shear and torsion is described by Kulak *et al.* (1987) and Kulak and Gilmore (2011) and adopted by CAN/CSA-S16 (2014). At the ultimate load, it is assumed that bolt furthest from the instantaneous center (IC) just reaches its failure load (Figure 2); Brandt (1982) presented in details the theoretical approach behind the IC method. Each bolt resistance is assumed to act on a line perpendicular to the radius joining the bolt to the instantaneous center; and, displacement Δ of each bolt is assumed to vary linearly with the length of that radius. The resistance of each bolt is calculated according to the load-deformation relationship of the bolt as Eq. (3):

$$R = R_u (1 - e^{-\mu\Delta})^{\lambda} \tag{3}$$

Where R is the bolt load at any given deformation, R_u is the ultimate bolt load, Δ is the shear, bending and bearing deformation of the considered bolt, μ and λ are regression coefficients and e is the base of natural logarithms. At the ultimate load, $\Delta = \Delta_{max}$ for the bolt furthest away from the instantaneous center.

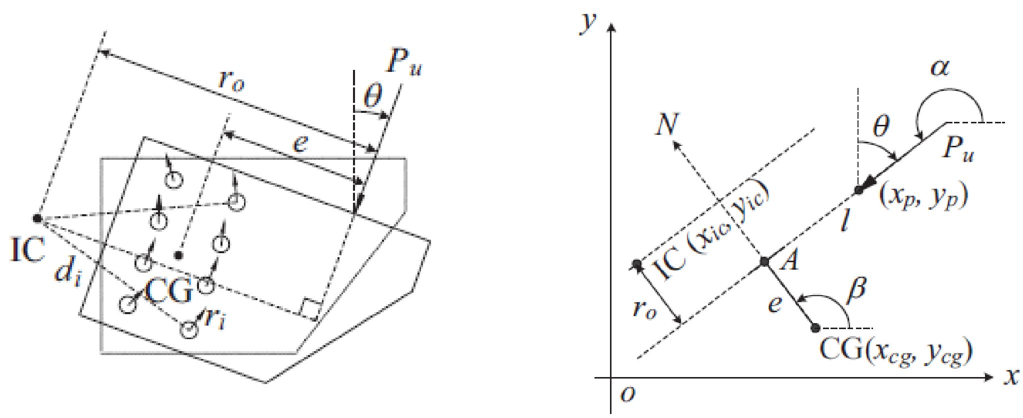

Figure 2. IC method (left) and perpendicular distance r_o from IC to a line *l*.

Tables have been developed using this method with normalized tabulated values for a coefficient C, which may be used for bolts of any diameter and/or grade. In determining C, the following values were used: R_u=329 kN, μ=10, λ=0.55, Δ_{max}=8.64 mm; these values were obtained experimentally for ¾ inch diameter A325 bolts and are reported by Crawford and Kulak (1971). Thus, the ultimate load for each bolt group and eccentricity can be computed from the tabulated values and then divided by the maximum value of R when $\Delta = \Delta_{max}$ to obtain the corresponding value of C.

AISC (2017) design manual provides two practical methods for the design of these bolted connections. The first method is essentially an elastic method and is regarded as a conservative method; the second method is based on the IC concept providing more realistic results. The C coefficient is listed for six inclination angles of the load (θ = 0°, 15 °, 30 °, 45 °, 60 °, and 75 °). Design Engineers tend to interpolate linearly the C coefficient for a nonspecific θ value. However, doing so is not entirely justified. Additionally, the direct implementation of the IC method is difficult because it involves a tedious trial and error process.

4 THE EXPERIMENTAL PROGRAM

Twelve specimens were designed to investigate behavior and failure mode of bolted connections subject to shear and torsion. Two and three rows of bolts (Grade 4.8) with 10 mm, 12 mm, and 16 mm bolt diameters were adopted in the tested connections. Test set-up and instrumentation are shown in Figure 3 while Table 1 shows details of the tested specimen with L and S symbols in the specimen names indicating long (80 mm) and short (60 mm) bolts, respectively.

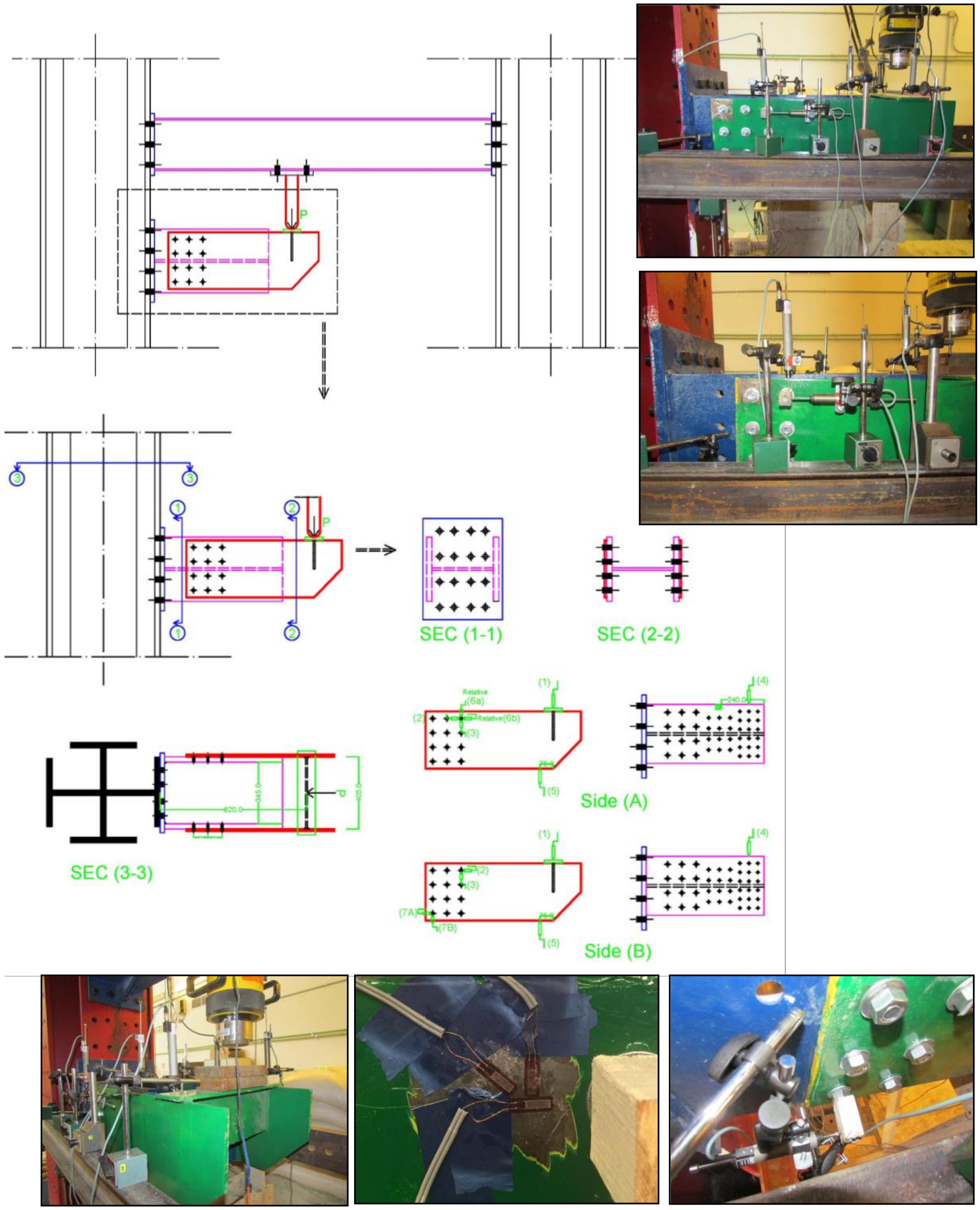

Figure 3. Test set-up and details of the tested connections.

The moving plates (brackets) were bolted directly to a bracket (fixed part), which was rigidly connected to the testing frame column (as a cantilever) using 27 mm diameter G8.8 bolts. Instrumentations were set, as shown in Figure 3 via 14 LVDTs and six strain gauges. The load was applied on a top plate connected the two brackets by an automatically controlled hydraulic jack, which was adjusted to maintain constant loading rate. All instrumentations and load cell were connected to an automatic data acquisition system, which was connected to a computer to record all data acquired by the system software.

Table 1. Details and results of all the tested connections.

Specimen ID	Bolt Dia.(mm)	No of rows	Exp. failure load P_{Exp} (kN)	Failure load Eq. 3 $P_{Eq.3}$ (kN)	$P_{Exp.}/P_{Eq.3}$	Elastic Load Eq. 2 $P_{Eq.2}$ (kN)	$P_{Exp}/P_{Eq.2}$
M(10)-2R-e1-S	10	2	60	70	0.86	43	1.39
M(10)-2R-e1-L	10	2	62		0.89		1.43
M(10)-2R-e2-S	10	2	52	60	0.87	35	1.47
M(10)-2R-e2-L	10	2	53		0.88		1.50
M(12)-2R-S	12	2	60	70	0.86	43	1.40
M(12)-2R-L	12	2	60		0.86		1.40
M(12)-3R-S	12	3	90	98	0.92	61	1.47
M(12)-3R-L	12	3	83		0.85		1.36
M(16)-2R-S	16	2	95	101	0.94	65	1.46
M(16)-2R-L	16	2	100		0.99		1.53
M(16)-3R-S	16	3	132	145	0.91	95	1.39
M(16)-3R-L	16	3	130		0.90		1.37
Average ± St. Dev.					0.9±0.04		1.4±0.05

5 TEST RESULTS

Table 1 provides a summary of the failure loads of all tested connections and compares these loads to both the elastic design approach and the inelastic design one adopted CAN/CSA-S16-14 (2014). Samples of failed connections are shown in Figure 4.

Table 1 reveals that the experimental failure load of the tested connections is about 90%±4% of that predicted by the inelastic design Eq. (3) while it is about 140%±5% of that predicted vial the elastic design approach Eq. (2). As such the elastic design tends to be very conservative and significantly underestimate the failure load of connections subjected to shear and torsion. On the other hand, the inelastic design predicted closer value to those recorded experimentally; however, it still needs some adjustment as it overestimates the failure load by about 10%.

6 CONCLUSIONS

An experimental program was conducted on twelve steel bolted connections, which are subjected to shear and torsion. The results of the experimental investigation were compared to the currently adopted elastic and inelastic design techniques.

The comparative investigation revealed that the elastic approach significantly underestimates the connection capacity and tends to be very conservative and uneconomic. On the other hand, the inelastic approach almost correctly predicted the connection capacity with about 10% error, which indicates that it is still in need of some adjustment since it currently overestimates the connection capacity by the said marginal error.

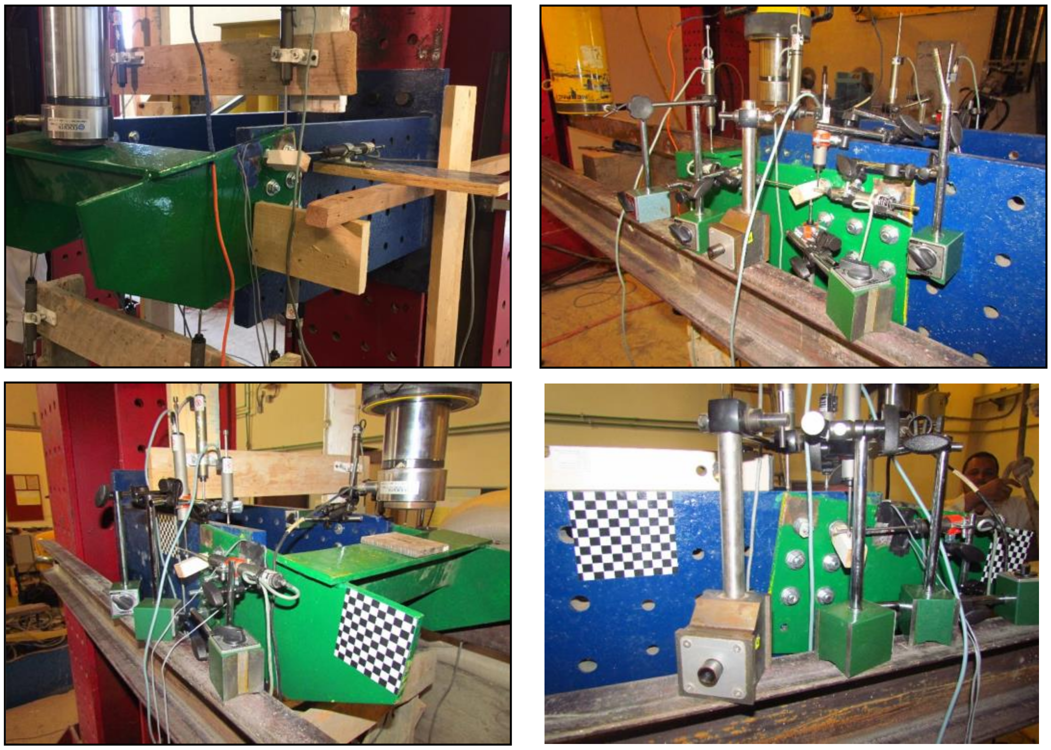

Figure 4. Sample of failed connections.

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