PREDICTION OF PUNCHING SHEAR STRENGTH OF EDGE SLAB-COLUMN CONNECTIONS REINFORCED WITH FRP BARS

Salama, Ahmed[1], Hassan, Mohamed[2], and Benmokrane, Brahim[3]*
[1,2,3] Department of Civil Engineering, University of Sherbrooke, Sherbrooke, QC, Canada
* Corresponding author (Brahim.Benmokrane@USherbrooke.ca)

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ABSTRACT

This paper evaluates the accuracy of the available punching-shear equations for fiber-reinforced polymer (FRP) and steel reinforced edge slab-column connections without shear reinforcement. A database was populated with results from 13 FRP and 25 steel reinforced edge slab-column prototypes in the literature. The accuracy of the design equations was assessed through the comparison against the experimental results. The edge slab-column connections had an effective depth ranging from 57 to 160 mm and a concrete compressive strength ranging from 20 to 85 MPa. The slabs were reinforced with steel, glass FRP bars and carbon FRP grids with a reinforcement ratio ranging from 0.25 to 1.83 %. The CSA S806-12 (2012) showed good yet conservative predictions with an average V_{test}/V_{pred} of 1.26±0.20 and a corresponding COV of 16% while the ACI 440.1R-15 (2015) equation was consistently very conservative, giving average V_{test}/V_{pred} of 1.94±0.22 and COV of 11% for FRP-reinforced edge slab-column connections. On the other hand, JSCE (1997) and El-Gamal et al. (2005) showed the best prediction for edge slab-column connections, on average, with V_{test}/V_{pred} of 1.24±0.18 and 1.11±0.19 for FRP and 1.13±0.19 and 1.15±0.20 for steel, respectively.

1 INTRODUCTION

Flat slab system is a common structural system being used in reinforced concrete (RC) structures, such as parking garages. In cold regions, however, when flat slabs were used in parking structures, they are exposed to harsh environmental conditions, which results in steel corrosion and consequent deteriorations. The use of fiber-reinforced polymer (FRP) bars with its non-corrodible nature in parking garages, instead of steel, will eliminate corrosion related problems, reduce the maintenance costs, and increase the service life. Due to the difference in the mechanical properties between the FRP and steel bars, the punching-shear strength equations for the steel-reinforced concrete elements cannot be employed directly for the FRP-RC elements. Most of the current equations predicting the punching-shear strength of FRP-RC elements are modified forms of those for steel-reinforced elements. The modification included factors to account for the lower modulus of elasticity of FRP bars than that of steel. Until now, very limited research has been conducted for edge slab-column connections, especially with FRP. Further experimental tests are needed in order to assess the accuracy of existing design equations for FRP-reinforced two-way slabs supported by edge columns [1]. At the University of Sherbrooke, an extensive research project to investigate the behaviour of edge slab-column connections reinforced with steel and FRP bars is in progress. One of the main objectives of the project is to quantify the effect of the different parameters on the punching-shear strength and behaviour of edge slab-column connections reinforced with steel and FRP bars. The first step to achieve that was to evaluate the parameters included in the available punching-shear strength equations. Thus, the main objective of this paper is to evaluate the accuracy of the available punching-shear equations provided by codes and guides for steel and FRP [1]; [2]; [3]; [4]; [5] and those provided by researchers based on their experimental or theoretical investigations [6]; [7]; [8]. The accuracy was assessed through a comprehensive database includes 25 steel and 13 FRP-RC edge slab-column connections [9]; [10]; [11]; [6]; [12]; [13]; [14]; [15]; [16]. The prototypes had an effective depth ranging from 57 to 160 mm and a concrete compressive strength ranging from 20 to 80 MPa. The prototypes were reinforced with steel, glass and carbon FRP bars and grids with a reinforcement ratio ranging from 0.25 to 1.83 %. The details of the steel and FRP edge slab-column connections database are listed in Table 1.

Reference	Slab	C_1, (mm)	C_2, (mm)	t, (mm)	d, (mm)	f'_c, (MPa)	ρ (%)	RFT. Type	E_f or E_s (GPa)	M/V (m)	$V_{u\ test}$, (kN)	$M_{u\ test}$, (kN.m)
Zaglool et al.1971 [15]	Z-IV (1)	178	178	152.4	127	27.4	1.83	S	200	0.368	122.3	45.0
	Z-V (1)	267	267	152.4	127	34.3	1.41	S	200	0.393	215.3	84.6
	Z-V (2)	267	267	152.4	127	40.5	1.64	S	200	0.379	246.9	93.6
	Z-V (3)	267	267	152.4	125	38.8	1.77	S	200	0.386	268.2	103.6
	Z-V (6)	267	267	152.4	127	31.2	1.41	S	200	0.753	117.0	88.1
	ZVI-(1)	356	356	152.4	127	25.9	1.41	S	200	0.40	265.1	106.9
Regan 1981 [14]	SE1	300	200	125	98	35.5	1.08	S	200	0.200	197.9	39.6
	SE2	300	200	125	101	44.4	0.25	S	200	0.177	192.0	34.0
	SE 4	200	300	125	98	26.6	1.08	S	200	0.234	151.7	35.5
	SE5	200	300	125	98	44.9	0.81	S	200	0.235	164.0	38.5
	SE6	200	300	125	98	32.9	0.64	S	200	0.185	149.0	27.5
	SE7	200	300	125	98	39.8	0.86	S	200	0.247	129.0	31.8
	SE8	300	100	125	98	42.1	1.81	S	200	0.248	136.0	33.7
	SE9	250	250	125	98	41.9	0.54	S	200	0.290	123.2	35.7
	SE10	250	250	125	98	41.1	0.54	S	200	0.317	113.9	36.1
	SE11	250	250	125	98	51.5	0.54	S	200	0.287	137.9	39.6
Hanson and Hanson 1968 [16]	D15	152	152	76.2	57.2	31.1	1.5	S	200	0.867	12.1	10.5
Sherif 1996 [13]	S1-2	250	250	120	114	29.5	1.41	S	200	0.24	185.0	43.9
El-Salakawy et al. (1998) [12]	XXX	250	250	120	94.4	33	0.75	S	200	0.3	125.0	37.5
	HXXX	250	250	120	94.4	36.5	0.75	S	200	0.66	69.4	45.8
	SF1	250	250	120	83.1	33.0	0.75	S	200	0.3	115.0	34.5
	SF2	250	250	120	83.1	30.0	0.75	S	200	0.3	114.0	34.5
Zaghloul et al.2007 [6]	ZJES	250	250	140	119	27.9	1.4	S	200	0.265	188.1	49.8
	ZJEF1	250	250	140	120	25.0	1.37	NEF	100	0.265	188.3	49.9
	ZJEF2	250	250	140	120	26.2	0.94	NEF	100	0.265	155.9	41.3
	ZJEF3	250	250	140	120	56.8	1.37	NEF	100	0.415	210.1	87.5
	ZJEF5	250	250	140	81	28.4	1.37	NEF	100	0.265	97.1	25.7
	ZJEF7	250	420	140	120	27.8	1.37	NEF	100	0.265	196.2	52.0
Omar Ben sisi 2013 [11]	S-10	160	160	80	59	20.0	1.11	S	200	0.36	32.5	11.7
El-Gendy et al.2015 [10]	S-0.9-XX-0.4	300	300	200	160	41.0	0.84	S	200	0.4	306.4	122.6
	GSC-1.35-XX-0.4	300	300	200	160	41.0	1.28	SG	60.505	0.4	268.2	107.3
	GSC-1.8-XX-0.4	300	300	200	160	45.6	1.7	SG	60.505	0.4	276.9	110.8
	GSC-0.9-XX-0.2	300	300	200	160	37.7	0.85	SG	60.505	0.2	239.4	47.9
	GSC-0.9-XX-0.6	300	300	200	160	36.5	0.85	SG	60.505	0.6	159.1	95.5
	GRD-0.9-XX-0.4	300	300	200	160	41.0	0.85	GRD	59.877	0.4	191.2	76.5
Mostafa et al 2016 [9]	GSC-0.9-XX-0.4	300	300	200	160	81.0	0.85	SG	60.505	0.4	251.0	100.4
	GSC-1.35-XX-0.4	300	300	200	160	85.0	1.28	SG	60.505	0.4	272.0	108.8
	GSC-1.8-XX-0.4	300	300	200	160	80.0	1.7	SG	60.505	0.4	288.0	115.2

Where, C_1 is the shorter side of the column; C_2 is the longer side of the column; ρ_t is the average flexural tensile reinforcement ratio; f'_c is the concrete compressive strength; S= Steel, SG = Sand coated glass fiber bars; GRD= Ribbed glass fiber bars; NEF= NEFMAC 2-D carbon fiber

Table 1. Properties of steel and FRP-reinforced edge slab-column connections database

2 PUNCHING SHEAR STRENGTH EQUATIONS FOR STEEL AND FRP

2.1 ACI 440.1R-15 (2015) and ACI 318 (2014)

The punching-shear stress provided by concrete (v_c) for two-way slabs reinforced with FRP bars or grids is simply the ACI 318 [2]. Punching-shear equation for steel-reinforced slabs modified to account for the shear transfer in two-way slabs. The vc equation for steel modified by the factor ([5/2] k) accounts for the axial stiffness of the FRP reinforcement through the neutral-axis-depth term (kd). The following equations 1 and 2 are for steel and FRP two-way slabs, respectively.

$$v_c = Min\ of \left(0.33\sqrt{f_c^{'}}; 0.083\sqrt{f_c^{'}}\left[2+\frac{\alpha_s d}{b_{o,0.5d}}\right]; 0.083\sqrt{f_c^{'}}\left[2+\frac{4}{\beta_c}\right]\right) \quad \text{(for steel)}$$

(1)

$$v_c = \left(\frac{5}{2}\right) k * 0.33\sqrt{f_c'} \quad \text{(for FRP)} \quad (2)$$

where, c =k d; k = $\sqrt{2\rho_f\, n_f + (\rho_f\, n_f)^2} - \rho_f n_f$; ρ_f is FRP reinforcement ratio; and n_f is the modular ratio (E_f/E_c); $E_c = 4700\sqrt{f_c^{'}}$ (modulus of elasticity of concrete); α_s is a factor equals 40 for interior columns, 30 for edge columns, 20 for corner columns; $b_{o,0.5d}$ is the perimeter of the critical section for slabs at a distance of effective depth d/2 from the column face (mm); and β_c the ratio of long side to short side of the concentrated load or reaction area.

2.2 CAN/CSA S806-12 (2012) and CAN/CSA A23.3-14 (2014)

v_c is calculated by the smallest of Eqn. (3), which are essentially the CSA A23.3 [3] equations with modifications to account for FRP bars instead of steel.

$$v_c = Min\ of \left(0.38\sqrt{f_c^{'}}; 0.19\sqrt{f_c^{'}}\left[2+\frac{\alpha_s d}{b_{o,0.5d}}\right]; 0.19\sqrt{f_c^{'}}\left[1+\frac{2}{\beta_c}\right]\right) \quad \text{(for steel)} \quad (3)$$

$$v_c = Min\ of \left(0.056*(E_f \rho_f f_c^{'})^{\frac{1}{3}}; 0.147\left(0.19+\frac{\alpha_s d}{b_{o,0.5d}}\right)(E_f \rho_f f_c^{'})^{\frac{1}{3}}; 0.028\left(1+\frac{2}{\beta_c}\right)(E_f \rho_f f_c^{'})^{\frac{1}{3}}\right) \quad \text{(for FRP)} \quad (4)$$

where α_s is a factor equals 4 for interior columns, 3 for edge columns, 2 for corner columns.

2.3 JSCE (1997)

The punching-shear stress for steel and FRP-RC slabs is calculated as given in Eqn. (5) according to the JSCE [5]. The factor (β_p) is a factor to consider the difference in the elastic modulus between FRP and steel.

$$V_c = \beta_d\, \beta_p\, \beta_r\, f_{pcd} / \gamma_b \quad (5)$$

where $\beta_d=(1000/d)^{1/4}\leq 1.5$ (d in meters); $\beta_p=(100\, \rho_f E_{fu}/E_o)^{1/3}\leq 1.5$; $f_{pcd}=0.2\ (f'_c)^{1/2}\leq 1.5$ MPa; $\beta_r=1+1/(1+0.25\ u_o/d)$; u_o is the perimeter of reaction area of supporting column ; E_{fu} is Young's modulus of tensile reinforcement; E_o is standard Young's modulus (assumed to be 200 GPa); and ρ_f is average values for reinforcement ratio in both directions; γ_b is partial factor of safety equal to 1.3 or 1.5 for concrete strengths below and above 50 MPa, respectively, that was set to 1.0 to get an un-factored prediction of capacity; f_c' is cylinder concrete compressive strength (MPa), and d is the effective slab depth (mm).

2.4 Other Available Equations

El-Ghandour et al. [8] based on experimental tests of FRP-RC flat slabs, proposed modifying the ACI-318-05 (2005) equation by multiplying predicted punching-shear stress by $(E_f/E_s)^{1/3}$ as follows:

$$v_c = 0.33\sqrt{f_c'}\left(E_f / E_s\right)^{\frac{1}{3}} \tag{6}$$

El-Gamal et al. [7] proposed a new parameter (α) to the ACI 318-05 (2005) equation, which is a function of the flexural stiffness of the tensile reinforcement (ρE), the perimeter of the applied load, and the effective depth of the slab as shown in Eqn. (7).

$$v_c = 0.33\sqrt{f_c'}\alpha \tag{7}$$

$$\alpha = 0.5(\rho E)^{1/3}\left(1 + 8\frac{d}{b_{o.0.5d}}\right),\ \ E\left(GPa\right) \tag{7a}$$

Zaghloul [6] proposed two Eqns. (8,9) for estimating concrete shear stress of flat slabs reinforced with FRP bars and steel rebars.

$$v_c = 0.07\left(0.44 + 5.16\frac{\alpha_s d}{b_{o,0.5d}}\right)(E_f \rho_f f_c')^{\frac{1}{3}} \qquad \text{(for FRP)} \tag{8}$$

$$v_c = 0.054\left(0.44 + 5.16\frac{\alpha_s d}{b_{o,0.5d}}\right)(E_s \rho_s f_c')^{\frac{1}{3}} \qquad \text{(for steel)} \tag{9}$$

3 COMPARISON BETWEEN PREDICTED AND EXPERIMENTAL RESULTS

FRP and steel edge slab-column connections listed in Table 1 were analyzed according to the aforementioned provisions [1]; [2]; [3]; [4]; [5]; [8]; [7]; [6]; setting the safety factors included in all the punching-shear equations equal to 1.0. Table 2 and 3 provide the ratio between the experimentally measured and predicted punching-shear capacities (V_{test}/V_{pred}) for steel and FRP-RC edge slab-column connections. The relations between the predicted and measured punching-shear capacities for FRP and steel edge slab-column connections are shown in Figures 1 and 2, respectively. The maximum shear stress for a connection v_u transferring shear and unbalanced moment is calculated according to Eqn. (10). For edge slab-column connection when gravity load, wind, earthquake, or other lateral forces exist in the structure, a transfer of an unbalanced moment (M_u) between the slab and column will develop. A portion of this moment $\gamma_v M_{sc}$ is transferred by non-uniform shear stress at the perimeter of d/2 from the column face.

$$v_c = \frac{V}{b_{o,0.5d}\,d} + \frac{\gamma_v M_{sc} c}{J_c} \tag{10}$$

where (v_c) is the shear stress of concrete; c is the distance from the centroid of the critical section to the face of the critical section; γ_v is fraction of unbalanced moment resisted by shear; $b_{o,0.5d}$ is the perimeter of the critical section for slabs at a distance d/2 away from the column face; J_c is property of assumed critical section analogous to the polar of inertia; b_1, b_2 is width of the critical section for shear measured parallel and perpendicular to the direction of the unbalanced moment.

$$\gamma_v = 1 - \frac{1}{1 + (2/3)\sqrt{b_1 / b_2}} \tag{10a}$$

$$J_c = \frac{2d(c_1 + c_2 + d/2)^3}{12} + \frac{2(c_1 + d/2)d^3}{12} + (c_2 + d)dc_{AB}{}^2 + \left(\frac{d}{2}\right)d\left(\frac{c_1 + d/2}{12} - c_{AB}\right)^2 \tag{10b}$$

The punching-shear predictions for FRP edge slab-column connections reported in Table 2 and comparisons presented in Figure 1 show that the punching-shear equations yielded good yet reasonable conservative predictions, except ACI 440.1R [1] equation (Eqn.2) and Zaghloul et al. [6] equation (Eq. 8). The ACI 440.1R [1] equation showed very conservative predictions with an average $V_{test}/V_{pred\ of}$ 1.94 ±0.22 and a corresponding COV of 11.27 %. The very conservative predictions are referred to that Eqn. (2) employs the reinforcement ratio only in predicting the depth of the neutral axis. The other equations, however, include the effect of axial-reinforcement stiffness, such as $(\rho E_f/E_s)^{1/3}$. On the other hand, Zaghloul et al. [6] (Eqn. 8) yielded unsafe predictions, on average, for the test results with a V_{test}/V_{pred} of 0.9±0.13 with a COV of 14.26 %. The punching-shear equation of the CSA S806 [4] also showed good yet conservative prediction with a V_{test}/V_{pred} of 1.26±0.2 which was very close to that of the JSCE [5] equation which yielded V_{test}/V_{pred} of 1.24±0.18. The comparisons revealed also that El-Gamal et al. [7] equation showed the best prediction, on average, with a V_{test}/V_{pred} of 1.11±0.19.

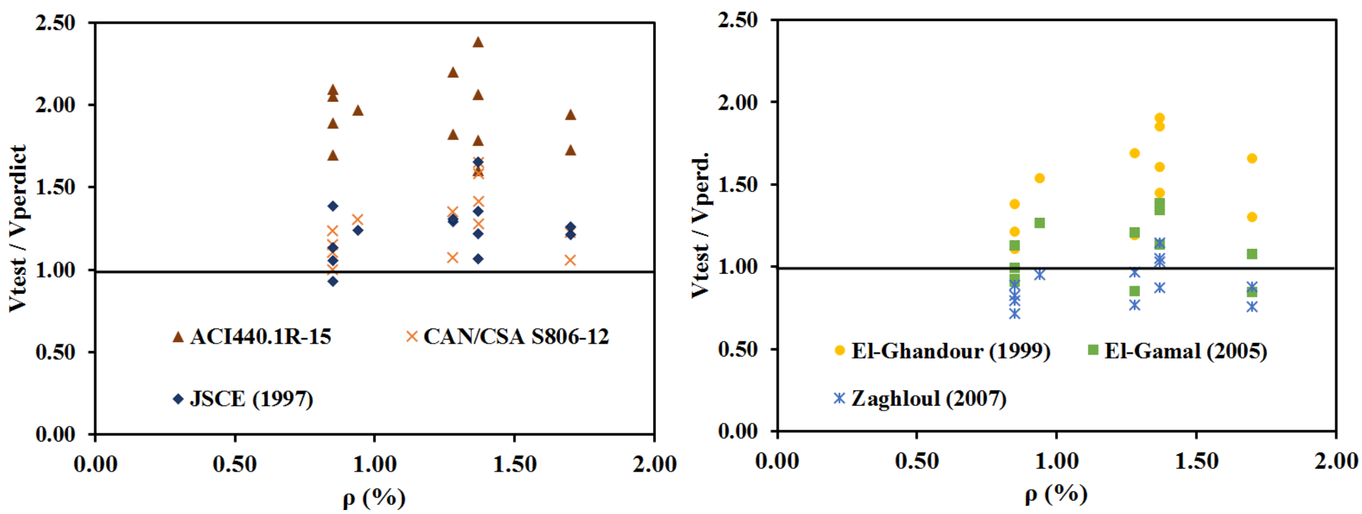

Figure 1. Tested-to-predicted punching-shear capacity relationships for FRP-RC edge slab-column connections

The punching-shear predictions for steel edge slab-column connections reported in Table 3 and comparisons presented in Figure 2 show that the punching-shear equations yielded good yet reasonable conservative predictions. The ACI 318 [2] and CSA A23.3 [3] equation showed the highest conservative level of predictions with an average V_{test}/V_{pred} of 1.39 ± 0.22 and 1.21 ± 0.19 and corresponding COVs of 16.01 % and 16.03 %. The high conservative predictions due to the absence of the reinforcement ratio in the punching shear equation. The comparisons revealed that JSCE [5] equation showed the best prediction, on average, with a V_{test}/V_{pred} of 1.13 ±0.19 and a COV of 16.4 %. On the other hand, the punching-shear equation of the El-Gamal et al. [7] showed good yet conservative prediction with a V_{test}/V_{pred} of 1.15±0.2 which was very close to that of the Zaghloul [6] equation which yielded V_{test}/V_{pred} of 1.18±0.2.

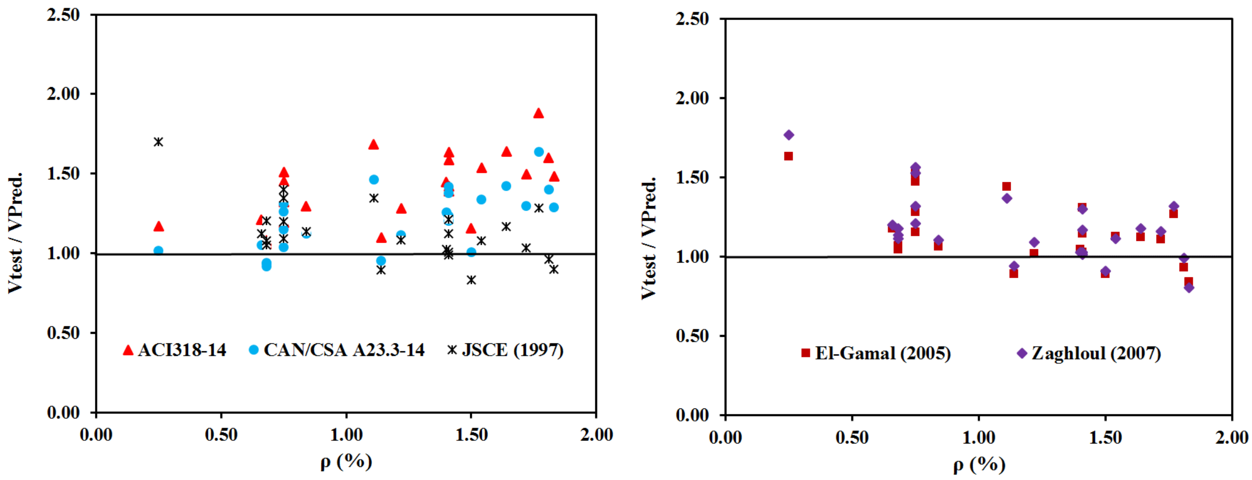

Figure 2. Tested-to-predicted punching-shear capacity relationships for steel-RC edge slab-column connections

Reference	Slab	V_{test}, kN	Tested-to-predicted punching-shear (V_{test}/V_{Pred})					
			JSCE (1997)	ACI 440.1R (2015)	CAN/CSA S806-12 (2012)	El-Ghandour et al. (1999)	Zaghloul (2007)	El-Gamal et al. (2005)
Zaghloul (2007) [6]	ZJEF1	188.34	1.36	2.06	1.41	1.90	1.03	1.38
	ZJEF2	155.87	1.24	1.97	1.30	1.54	0.95	1.26
	ZJEF3	210.87	1.65	2.38	1.58	1.85	1.15	1.34
	ZJEF5	97.1	1.22	1.78	1.65	1.60	1.05	1.34
	ZJEF7	196.16	1.07	1.60	1.28	1.45	0.87	1.13
El-Gendy et al. (2015) [10]	GSC-1.35-XX-0.4	268.2	1.29	2.20	1.35	1.69	0.97	1.21
	GSC-1.8-XX-0.4	276.9	1.21	1.94	1.23	1.65	0.88	1.07
	GSC-0.9-XX-0.2	239.4	0.93	1.70	1.00	1.11	0.72	0.91
	GSC-0.9-XX-0.6	159.1	1.14	2.09	1.24	1.38	0.89	1.13
	GRD-0.9-XX-0.4	191.2	1.06	1.89	1.11	1.21	0.79	0.99
Mostafa et al. (2016) [9]	GSC-0.9-XX-0.4	251	1.38	2.05	1.16	1.12	0.83	0.92
	GSC-1.35-XX-0.4	272	1.31	1.82	1.07	1.19	0.77	0.85
	GSC-1.8-XX-0.4	288	1.26	1.73	1.06	1.30	0.76	0.84
		Mean	**1.24**	**1.94**	**1.26**	**1.46**	**0.90**	**1.11**
		SD	0.18	0.22	0.20	0.27	0.13	0.19
		COV, %	14.46	11.27	15.63	18.45	14.26	17.47

Table 2. Tested-to-predicted punching-shear capacity for FRP edge slab-column connections

Reference	Slab	V_{test}, kN	Tested-to-predicted punching-shear (Vtest/VPred)				
			JSCE (1997)	ACI 318-14 (2014)	CAN/CSA A23.3-12 (2012)	El-Gamal et al. (2005)	Zaghloul (2007)
Zaglool et al. (1971) [15]	Z-IV (1)	122.3	0.90	1.48	1.29	0.84	0.80
	Z-V (1)	215.3	1.12	1.59	1.38	1.14	1.17
	Z-V (2)	246.9	1.17	1.64	1.42	1.12	1.18
	Z-V (3)	268.2	1.28	1.88	1.64	1.27	1.32
	Z-V (6)	117	1.00	1.42	1.23	1.03	1.03
	Z-VI (1)	265.1	1.21	1.64	1.42	1.31	1.30
Regan (1981) [14]	SE1	197.9	1.03	1.50	1.30	1.11	1.16
	SE2	192	1.70	1.17	1.02	1.63	1.77
	SE 4	151.7	1.08	1.54	1.34	1.13	1.11
	SE5	164	1.08	1.28	1.11	1.02	1.09
	SE6	149	1.13	1.21	1.05	1.18	1.20
	SE7	129	0.90	1.10	0.95	0.89	0.94
	SE8	136	0.96	1.60	1.40	0.93	0.99
	SE9	123.2	1.08	1.08	0.93	1.06	1.13
	SE10	113.9	1.05	1.06	0.92	1.05	1.11
	SE11	137.9	1.20	1.08	0.94	1.07	1.18
Hanson and Hanson (1968) [16]	D15	12.1	0.83	1.16	1.00	0.89	0.91
Sherif (1996) [13]	S1-2	185	0.99	1.39	1.21	1.02	1.01
El-Salakawy et al. (1998) [12]	XXX	125	1.20	1.32	1.15	1.28	1.32
	HXXX	69.4	1.09	1.19	1.04	1.16	1.21
	SF1	115	1.35	1.45	1.26	1.47	1.53
	SF2	114	1.40	1.51	1.31	1.53	1.57
Zaghloul (2007) [6]	ZJES	188.08	1.02	1.45	1.26	1.05	1.03
Omar Ben sisi (2013) [11]	S-10	32.5	1.35	1.68	1.46	1.44	1.37
El Gendy et al. (2015) [10]	S-0.9-XX-0.4	306.4	1.14	1.29	1.12	1.06	1.11
		Mean	**1.13**	**1.39**	**1.21**	**1.15**	**1.18**
		SD	0.19	0.22	0.19	0.20	0.22
		COV, %	16.44	16.01	16.03	17.80	18.24

Table 3. Tested-to-predicted punching-shear capacity for steel edge slab-column connections

4 SUMMARY AND CONCLUDING REMARKS

This paper evaluates the accuracy of the available steel and FRP-RC punching-shear equations through comparing their predictions against the measured punching-shear strength of 13 FRP and 25 steel RC edge slab-column connections. The edge slab-column connections had an effective depth ranging from 57 to 160 mm and a concrete compressive strength ranging from 20 to 85 MPa. The prototypes were reinforced with steel, glass FRP bars and carbon FRP grids with a reinforcement ratio ranging from 0.25 to 1.83%. Based on the discussion presented herein, the following concluding remarks can be drawn:

1. The CSA S806 [4] showed good yet conservative predictions with an average V_{test}/V_{pred} of 1.26±0.20 and corresponding COV of 16% while the ACI 440.1R [1] equation was consistently very conservative, giving average V_{test}/V_{pred} of 1.94±0.22 and COV of 11% for FRP-reinforced edge slab-column connections.

2. The JSCE [5] and El-Gamal et al. [7] showed the best prediction for edge slab-column connections, on average, with V_{test}/V_{pred} of 1.24±0.18 and 1.11±0.19 for FRP and 1.13±0.19 and 1.15±0.20 for steel, respectively.
3. Further research is needed to examine the punching shear design equations for FRP RC slabs supported by edge columns with a wide range of test parameters.

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