RESILIENT INFRASTRUCTURE
June 1–4, 2016

EFFECT OF CONCRETE STRENGTH ON THE PUNCHING SHEAR BEHAVIOUR OF GFRP-RC SLAB-COLUMN EDGE CONNECTIONS

Ahmed M. Mostafa
MSc Student, University of Manitoba, Canada

Mohammed G. El-Gendy
PhD Candidate, University of Manitoba, Canada

Ehab El-Salakawy
Professor and CRC, University of Manitoba, Canada

ABSTRACT

This paper presents the results of an experimental program carried out to investigate the effect of concrete strength on the punching shear behaviour of concrete slab-column edge connections reinforced with glass fibre-reinforced polymer (GFRP) bars. Six full-scale connections were constructed and tested to failure under gravity loads. Three connections were made of normal strength concrete (NSC), while the other three were made of high strength concrete (HSC). The dimensions of the slabs were 2,800×1,550×200 mm with a 300-mm square column extending 1,000 mm above and below the slab. All connections were reinforced with GFRP sand-coated bars without shear reinforcement. Three flexural reinforcement ratios were employed for each concrete strength; 0.90, 1.35 and 1.80% in the direction perpendicular to the free edge. All connections failed in a brittle punching mode. The HSC connections showed less deflections and strains in both reinforcement and concrete at the same load level than their NSC counterparts. Also, doubling the concrete strength (from 40 to 80 MPa) slightly increased the capacity by 10, 3 and 5% for connections with reinforcement ratios of 0.90, 1.35 and 1.80%, respectively. Moreover, the Canadian standard for FRP-reinforced concrete buildings provided reasonable predictions with an average experimental-to-predicted ratio of 1.29±0.05 and 1.22±0.05 for the NSC and HSC connections, respectively.

Keywords: edge connection; flat plate; GFRP; high strength concrete; punching shear; slab-column connection;

1. INTRODUCTION

Most parking garage structures in North America are constructed using reinforced concrete (RC) flat plate systems, which allow for more clearance for vehicles due to the absence of beams. Those parking garages are vulnerable to the corrosion of steel problem since they are exposed to harsh conditions such as freeze-thaw cycles, wet-dry cycles and de-icing salts. Recently, fibre-reinforced polymer (FRP) reinforcement is used in RC structures instead of the conventional steel reinforcement to overcome the steel corrosion problem; thanks to its superior performance in corrosive environments and its durable nature. Regardless of the reinforcement type, punching shear failure is a common type of failure in flat plate systems. This type of failure is very dangerous due to its brittle nature, which gives no warning before the failure occurs. In addition, the failure of one joint in the system may lead to a progressive collapse of the whole building. Punching shear failure occurs due to the high shear stresses resulting from the combination of direct shear forces and unbalanced bending moments transferred from the slab to the column at slab-column connections. This combination of shear and unbalanced bending moment is inevitable at slab-column connections as a result of unequal spans, unsymmetrical loading and the disruption of the slab at exterior connections. Moreover, slab-column edge connections are more critical to punching failure than interior ones due to the relatively higher moments transferred between the slab and the column at this location and the probable lack of slab reinforcement anchorage due to small column cross-sections and the disruption of the slab.

High strength concrete (HSC) is extensively used in bridges, buildings and other structures due to its superior strength and stiffness (El-Sayed et al. 2006). Tests on steel-RC slab-column connections made of HSC demonstrated a considerable increase in the punching capacity and significant enhancement in the pre- and post-cracking behaviour of the connections (Ghannoum 1998; Marzouk et al. 1998; Ozden et al. 2006). This is mainly attributed to the increased uncracked concrete contribution to the punching shear strength and the delayed initiation of cracks. On the other hand, little research was conducted to study the effect of concrete strength on slab-column interior connections reinforced with FRP bars (Hassan et al. 2013; Gouda and El-Salakawy 2016). However, to date, no research has been done to study the effect of concrete strength on FRP-RC slab-column edge connections. This paper aims to partially fill this gap.

2. EXPERIMENTAL PROGRAM

2.1 Test Connections

Six full-scale slab-column edge connections were constructed and tested to failure in the McQuade Structures Laboratory at the University of Manitoba. Three connections were made of NSC, while the other three were made of HSC. All connections were reinforced using GFRP bars with one orthogonal mesh in the tension side of the slab. Straight bars were used in the direction parallel to the free edge, while bent bars were used in the direction perpendicular to the free edge to provide sufficient anchorage. Three flexural reinforcement ratios were employed for each concrete strength; 0.90, 1.35 and 1.80% in the direction perpendicular to the free edge.

The dimensions and reinforcement distribution of the connections were defined by performing an elastic analysis of a typical parking garage building consisting of three 6.5 m-long bays in both directions. Each connection simulates a portion of the slab bounded by the slab free edge and the lines of contra-flexure located at a distance of 0.2*L* from the column centrelines, where *L* is the distance between the centrelines of the column. The typical dimensions of the slabs were 2,800×1,550×200 mm with two 300-mm square column stubs extending 1000 mm above and below the slab as shown in Figure 1. The control design was conducted using NSC with a compressive strength of 40 MPa, which resulted in a flexural reinforcement ratio of 1.8%. Then, two more reinforcement ratios 0.9 and 1.35% were selected to investigate the effect of concrete strength (either 40 or 80 MPa) at different reinforcement ratio. The columns were adequately reinforced with 4 No. 20M bars and No. 10M stirrups to prevent premature failure. The designation of the connections consists of a letter indicating the concrete strength (N for normal strength, H for high strength) and a number indicating the flexural reinforcement ratio in the direction perpendicular to the free edge (0.9 for ρ = 0.9%, 1.3 for ρ = 1.35%, 1.8 for ρ = 1.8%).

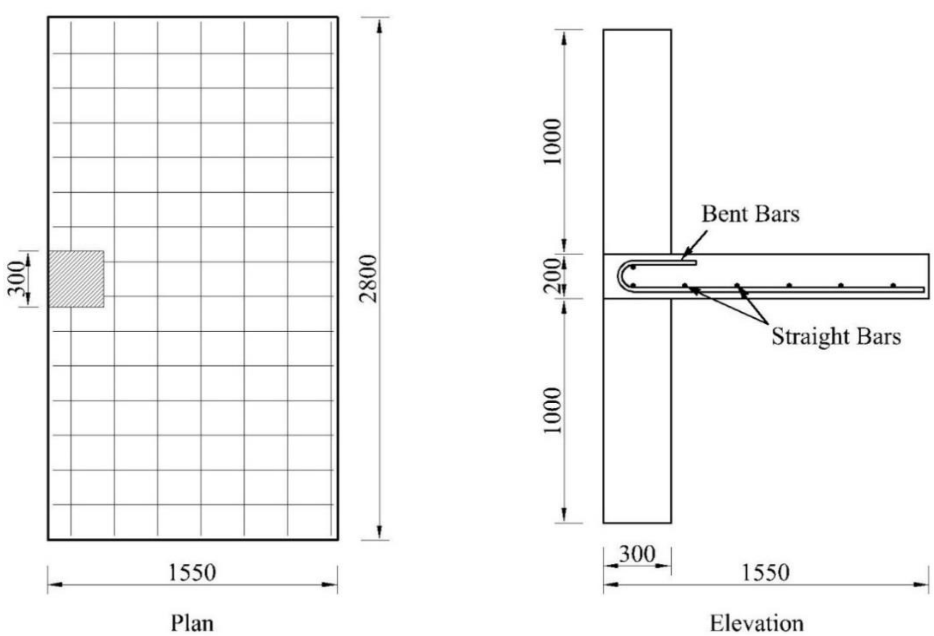


Figure 1: Dimensions of a typical connection and reinforcement distribution

2.2 Material Properties

All connections were cast using normal-weight ready-mix concrete with target compressive strengths of 40 and 80 MPa for the NSC and HSC connections, respectively. The actual concrete compressive strength of each connection was determined on the day of testing by testing three 100×200 mm concrete cylinders. The concrete strength of each connection is listed in Table 1. All connections were reinforced using No. 20 sand-coated GFRP bars. Table 2 shows the properties of the used GFRP reinforcing bars.

Table 1: Details of Test Connections

Connection	Concrete Compressive Strength (MPa)	Reinforcement Ratio Perpendicular to the Free Edge (%)
N-0.9	40	0.90
N-1.3	42	1.35
N-1.8	42	1.80
H-0.9	81	0.90
H-1.3	85	1.35
H-1.8	80	1.80

Table 2: Properties of Reinforcing Bars

Bar Type	Diameter (mm)	Area (mm^2)	Tensile Modulus (MPa)	Ultimate Strength (MPa)	Ultimate Strain (%)
Straight GFRP	19.1	285	65	1480	2.3
Bent GFRP (straight portion)	19.1	285	52	1230	2.3

2.3 Instrumentation

A total of 12 electrical strain gauges (6-mm long) were attached at critical locations to four bars passing through the column (two in each orthogonal direction) in order to measure slab reinforcement strains. Also, two electrical strain gauges (40-mm long) were used to measure concrete surface strains at the column face in both orthogonal directions. On the other hand, 12 linear variable displacement transducers (LVDTs) were used to measure the deflections of the slab at different locations. The applied loads were measured using load cells. All instrumentation was connected to a computerized data acquisition system (DAQ) to record the readings during the test.

2.4 Test Setup and Testing Procedure

Figure 2 shows the details of the test setup. The connections were tested in an upside-down position with respect to the position of a real structure, i.e., the vertical column load is applied from top to bottom. As a result, tension cracks appeared on the bottom face of the slab. Stiff I-beams were used to simply support the connections at three edges, while the fourth edge was left free. Also, neoprene strips were placed between the I-beams and the connections in order to have a uniform load distribution along the edges. A hydraulic actuator and two hydraulic jacks attached to a rigid steel frame were used to apply the vertical shear force and the lateral forces causing the unbalanced moment, respectively. The actuator and one hydraulic jack were placed at the tip of the upper column stub, while the other hydraulic jack was placed at the tip of the lower column. A moment-to-shear ratio of 0.4 m was kept constant during the whole test for all connections and the loading was paused every 20 kN to mark the propagation of cracks.

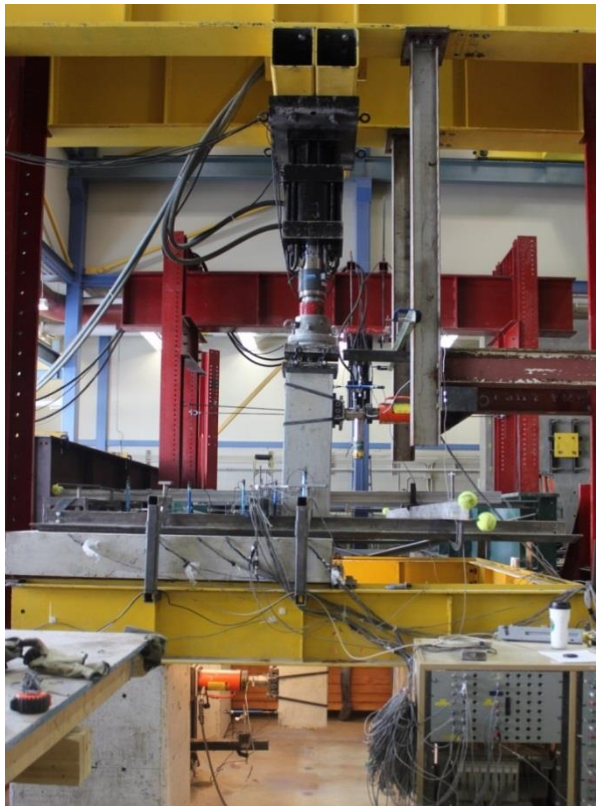


Figure 2: Test setup

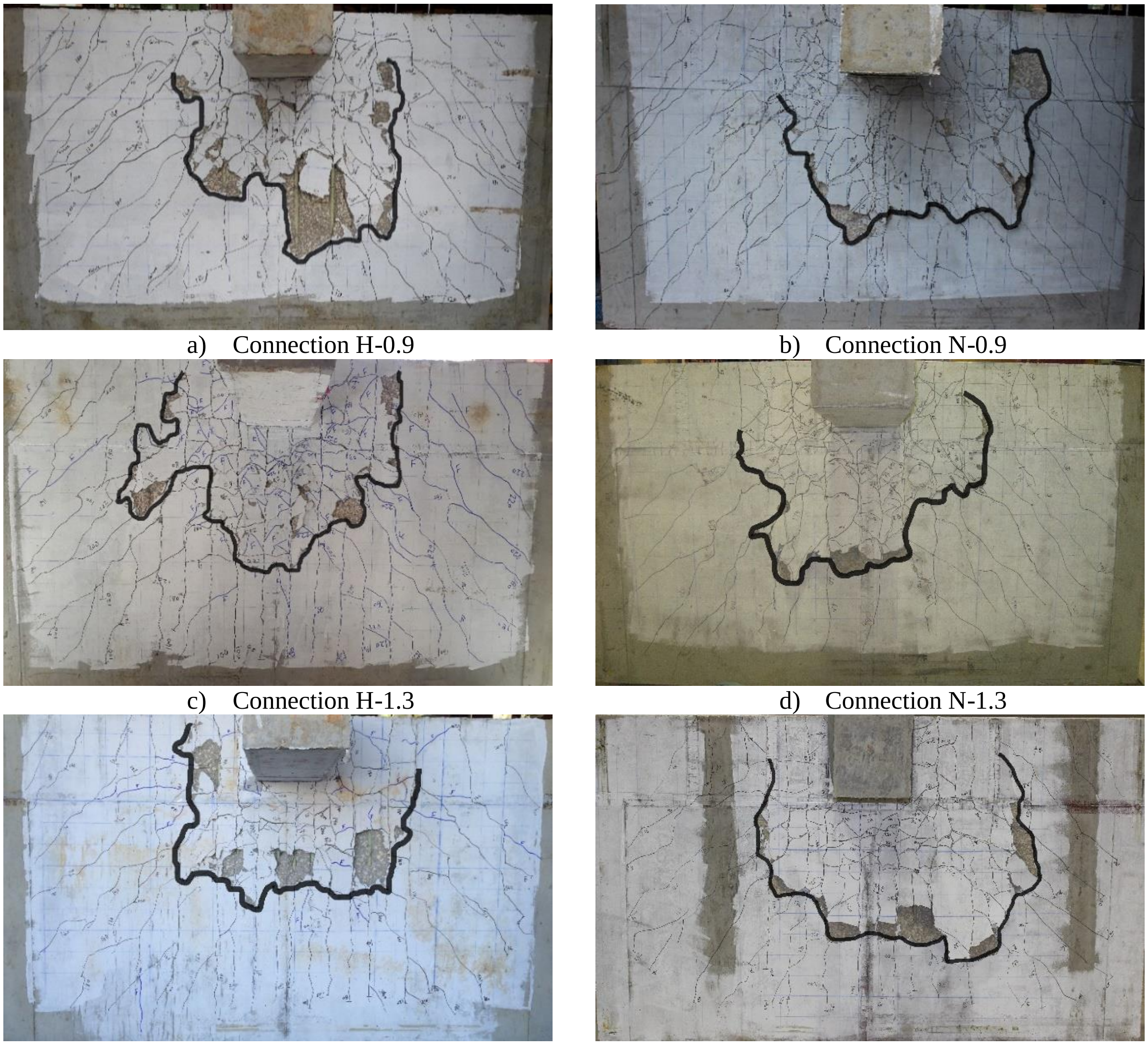
3. TEST RESULTS AND DISCUSSION

3.1 Mode of Failure and Cracking Pattern

All connections, regardless of the concrete strength and the reinforcement ratio, failed in a brittle punching shear mode, in which the applied load dropped suddenly with the formation of a wide circumferential crack and the penetration of column through the slab. However, as expected, the HSC connections had higher cracking load than their NSC counterparts. All connections showed similar cracking behaviour. Flexural cracks were observed first at the inner corners of the column at a vertical load of 32, 34, 36, 65, 60 and 63 for connections N-0.9, N-1.3, N-1.8, H-0.9, H-1.3 and H-1.8, respectively (Table 3). These flexural cracks then propagated around the column periphery reaching the slab free edge. As the load increased, radial cracks developed from the column faces towards the supports. Eventually, circumferential cracks formed and connected the radial cracks together. The cracking pattern on the tension face for the six connections is shown in Figure 3.

3.2 Deflections

The relationship between the vertical load and the deflection measured at 50 mm from the face of the column in the direction perpendicular to the free edge is shown in Figure 4. As expected, the use of HSC enhanced the uncracked stiffness of the connections. The HSC connections had higher uncracked stiffness factor, K_i, which is the slope of the load-deflection curve before cracking, than their NSC counterparts as listed in Table 3. Since the post-cracking stiffness, which is the slope of the load-deflection curve after cracking, is mainly dependent on the axial rigidity of the reinforcing bars ($\rho_f E_f$), the HSC connections had approximately the same post-cracking stiffness as their NSC counterparts. However, the HSC connections showed less deflection than their NSC counterparts at the same load level due to the increased cracking load and the enhanced uncracked stiffness.



a) Connection H-0.9 b) Connection N-0.9

c) Connection H-1.3 d) Connection N-1.3

e) Connection H-1.8 f) Connection N-1.8

Figure 3: Cracking patterns

3.3 Reinforcement and Concrete Strains

Figure 5 shows the relationship between the vertical load and the reinforcement and concrete strains measured at the column face in the direction perpendicular to the free edge. The maximum measured concrete strain was 1,640 με, which is considerably less than the theoretical concrete crushing strain defined by the CSA/S806-12 standard (3,500 με). This confirms that the connections exhibited a punching shear failure rather than a flexural failure. The HSC connections showed less reinforcement and concrete strains at the same load level than their NSC counterparts because of their high tensile strength, which delayed the formation of cracks. Figure 6 shows the reinforcement strain profile for connection H-1.3 in the direction perpendicular to the free edge at different loading levels of 25, 50, 75, 100% of the failure load. The measured strains were inversely proportional to the distance from the column face, which indicates a good bond behaviour between the GFRP and the surrounding concrete. This behaviour was observed in all tested connections and agrees with the findings of recent studies for both edge and interior connections (El-Gendy and El-Salakawy 2016; Gouda and El-Salakawy 2016).

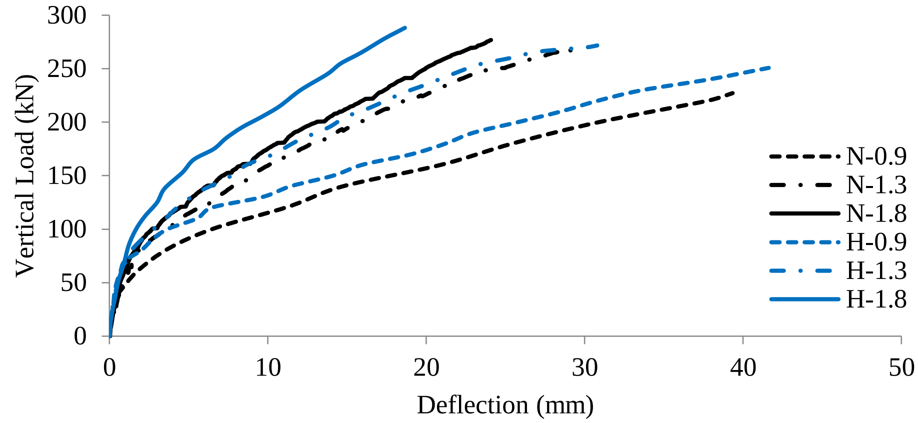


Figure 4: Load vs. deflection relationship

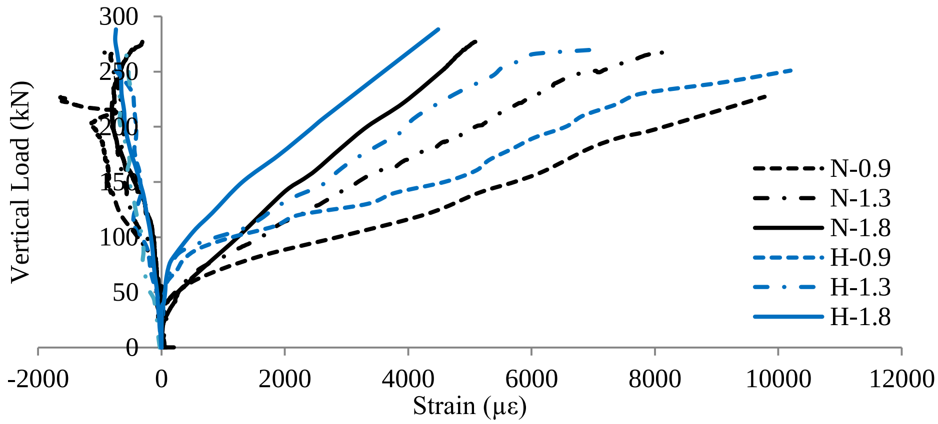


Figure 5: Load vs. reinforcement and concrete strain relationship

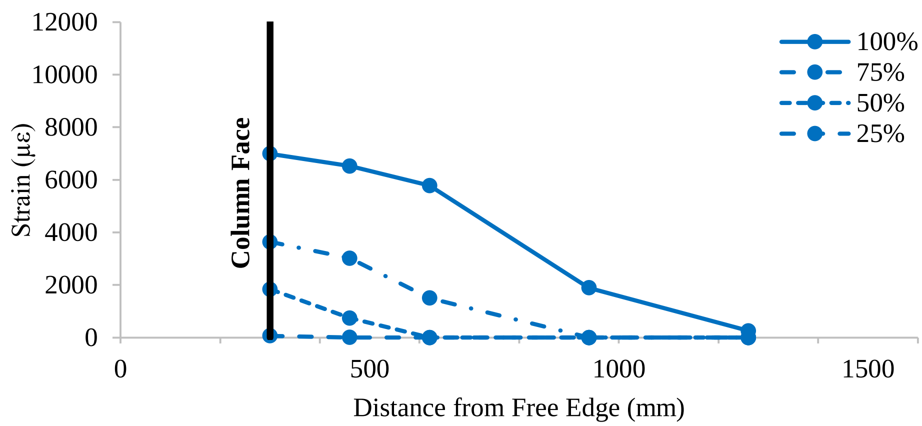


Figure 6: Reinforcement strain profile for connection H-1.3

3.4 Punching Shear Capacity

The failure loads of the NSC and HSC connections were multiplied by $\sqrt[3]{40/f_c'}$ and $\sqrt[3]{80/f_c'}$, respectively, to eliminate the effect of concrete strength variation within each type of concrete. The use of HSC slightly enhanced the punching shear capacity of the connections. Doubling the concrete compressive strength (from 40 MPa to 80 MPa) resulted in a 10, 3 and 5% increase in the capacity for connections with reinforcement ratios of 0.90, 1.35 and 1.80%, respectively. Similarly, Gouda and El-Salakawy (2016) found only 5% increase in the punching capacity when they increased the concrete strength from 42 to 70 MPa (67% increase) in GFRP-RC interior connections with flexural reinforcement ratio of 0.65% and subjected to eccentric loading. Furthermore, for steel-RC interior connections subjected to eccentric loading, Marzouk et al. (1998) reported only 5% increase in the punching capacity when the concrete strength was increased from 35 to 74 MPa (111% increase) in connections with flexural reinforcement ratio of 1.0%. These results raise an important concern about the accuracy of the current design standards in North America, which predicts the punching capacity to be proportional to the cubic root (CSA S806-12) and the square root (ACI 440.1R-15) of the concrete compressive strength. Further investigation is needed, however, to clarify this issue. On the other hand, while increasing the flexural reinforcement ratio by 50 and 100% increased the capacity by 14 and 21%, respectively, for the NSC connections, the increase in the capacity was only 7 and 15% for the HSC connections. Increasing the reinforcement ratio helps control the propagation of cracks, which results in increasing the aggregate interlock, the depth of uncracked concrete and dowel action. In HSC, however, cracks tend to pass through the aggregate particles instead of around it. As a result, crack surfaces in HSC are smoother than those in NSC, which reduces the contribution of aggregate interlock and, in turn, reduces the effect of increasing the flexural reinforcement ratio.

Table 3: Test Results

Connection	First Crack Load (kN)	Failure Load (kN)	Normalized Failure Load (kN)	Maximum Deflection at Failure (mm)	Uncracked Stiffness, K_i (kN/mm)	Reinforcement Strain at Failure (με)	Concrete Strain at Failure (με)
N-0.9	32	227	227	39	93	9,780	-1,640
N-1.3	34	264	260	28	80	8,170	-950
N-1.8	36	278	274	24	83	5,090	-800
H-0.9	65	251	250	42	127	10,200	-744
H-1.3	60	272	267	31	138	6,990	-683
H-1.8	63	288	288	19	120	4,485	-736

3.5 Code Comparison

The failure loads of the six connections were compared to the predictions of the CSA/S806-12 standard (CSA 2012) and the ACI 440.1R-15 guideline (ACI Committee 440 2015) as listed in Table 4. It is to be noted that the CSA S806-12 standard limits the maximum concrete compressive strength to be used in the punching shear equations to 60 MPa. The CSA S806-12 equations provided reasonable predictions with an acceptable safety margin yielding an average V_{Test}/V_{Pred} of 1.29±0.05 and 1.22±0.05 for the NSC and HSC connections, respectively. However, if the 60 MPa limit is ignored, the safety margin is reduced for the HSC connections and the average test-to-predicted capacities becomes 1.1±0.05. Similar results were obtained by Gouda and El-Salakawy (2016) and Marzouk et al. (1998) for GFRP-RC and steel-RC interior connections, respectively, subjected to eccentric loading. On the other hand, the ACI 440.1R-15 guideline highly underestimated the capacities with an average V_{Test}/V_{Pred} of 2.23±0.12 and 1.93±0.18 for the NSC and HSC connections, respectively.

Table 4: Codes Comparisons

Connection	Ultimate Capacity, V_{Test} (kN)	Punching Shear Capacity Predictions, V_{Pred}			
		CSA S806-12		ACI 440.1R-15	
		V_{Pred}	V_{Test}/V_{Pred}	V_{Pred}	V_{Test}/V_{Pred}
N-0.9	227	174	1.30	97	2.34
N-1.3	264	200	1.32	118	2.24
N-1.8	278	221	1.26	132	2.11
Mean			1.29		2.23
Standard Deviation			0.05		0.12
H-0.9	251	196	1.27	118	2.13
H-1.3	272	225	1.21	144	1.88
H-1.8	288	247	1.17	161	1.79
Mean			1.22		1.93
Standard Deviation			0.05		0.18

4. CONCLUSIONS

Based on the presented experimental results and discussion, the following can be concluded:

1. All tested connections failed in a brittle punching shear mode characterized by a sudden drop in the load accompanied by column penetration through the slab.
2. The use of HSC enhanced the deflection performance of the tested slabs. The HSC connections had higher uncracked stiffness and less deflection at the same load level compared to their NSC counterparts.
3. The use of HSC delayed the formation of cracks, which resulted in less reinforcement and concrete strains in the HSC connections compared to their NSC counterparts at the same load level.
4. The use of HSC slightly enhanced the punching shear capacity. Doubling the concrete strength (from 40 to 80 MPa) increased the punching capacity by 10, 3 and 5% for connections with reinforcement ratios of 0.90, 1.35 and 1.80%, respectively.
5. The CSA/S806-12 standard (CSA 2012) provided reasonable predictions with an average V_{Test}/V_{Pred} of 1.29±0.05 and 1.22±0.05 for the NSC and HSC connections, respectively, while the ACI 440.1R-15 guideline (ACI Committee 440 2015) highly underestimated the capacities with an average V_{Test}/V_{Pred} of 2.23±0.12 and 1.93±0.18 for the NSC and HSC connections, respectively.
6. The concrete strength limit of 60 MPa in the CSA/S806-12 standard (CSA 2012) provides acceptable safety margin for HSC slab-column edge connections.

ACKNOWLEDGEMENTS

The authors wish to express their gratitude to the Natural Sciences and Engineering Research Council of Canada (NSERC) through the Canada Research Chairs and Discovery Programs. Also, the assistance received from the technical staff at the McQuade Structures Laboratory is acknowledged.

REFERENCES

ACI Committee 440. 2015. *Guide for the Design and Construction of Structural Concrete Reinforced with FRP Bars*, American Concrete Institute, Farmington Hills, MI, USA.

Canadian Standards Association (CSA). 2012. *Design and Construction of Building Structures with Fibre Reinforced Polymers*, CAN/CSA S806-12, Canadian Standards Association, Toronto, ON, Canada.

El-Gendy, M. and El-Salakawy, E. 2016. Effect of Shear Studs and High Moments on Punching Behavior of GFRP-RC Slab-Column Edge Connections. *Journal of Composites for Construction*, ASCE, DOI: 10.1061/(ASCE) CC.1943-5614.0000668, 04016007.

El-Sayed, A., El-Salakawy, E. and Benmokrane, B. 2006. Shear Capacity of High-Strength Concrete Beams Reinforced with FRP Bars. *ACI Structural Journal*, 103(3): 383-389.

Ghannoum, C. 1998. *Effect of High-Strength Concrete on the Performance of Slab-Column Specimens*. MSc Thesis, McGill University, Montréal, QC, Canada, 91p.

Gouda, A. and El-Salakawy, E. 2016. Punching Shear Strength of GFRP-RC Interior Slab-Column Connections Subjected to Moment Transfer. *Journal of Composites for Construction*, ASCE, 20(1), DOI: 10.1061/(ASCE) CC.1943-5614.0000597, 04015037.

Hassan, M., Ahmed, E. and Benmokrane, B. 2013. Punching-Shear Strength of Normal and High-Strength Two-Way Concrete Slabs Reinforced with GFRP Bars. *Journal of Composites for Construction*, ASCE, DOI: 10.1061/(ASCE) CC.1943-5614.0000424, 04013003.

Marzouk, H., Emam, M. and Hilal, M. 1998. Effect of High-Strength Concrete Slab on the Behavior of Slab-Column Connections. *ACI Structural Journal*, 95(3): 227-236.

Ozden, S., Ersoy, U. and Ozturan, T. 2006. Punching Shear Tests of Normal- and High-Strength Concrete Flat Plates. *Canadian Journal of Civil Engineering*, 33(11): 1389-1400.