Strategic Utilization of Fiber Reinforced UHSC in Slab-Column Connections

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ABSTRACT

This study reports on the structural characteristics of slab-column connections using an ultra-high-strength-fiber-reinforced concrete from new and retrospective data. The parameters investigated were the "puddling" of ultra-high-strength-fiber-reinforced concrete and the use of high-strength concrete in the slab. The effects of these parameters on the punching shear capacity, negative moment cracking, and stiffness of the two-way slab specimens are investigated. Furthermore, the ACI Code (2002), the CSA Standard (1994), the BS Standard (1985) and the CEB-FIP Code (1990) predictions are compared to the experimental results obtained from some slab-column connections tested in this experiment and those tested by other investigators. The beneficial effects of the ultra-high-strength-fiber-reinforced concrete puddling and of the use of high-strength concrete are demonstrated. It is also concluded that the punching shear strength of slab-column connections is a function of the flexural reinforcement ratio.

1. Introduction

Due to recent trends in the construction industry (i.e., construction of larger and taller structures), the need for high-strength concrete is growing larger. High-strength concrete is being used very effectively for structural elements which are subject to the axial loads, such as columns. However, structural research into the practical applications of high-strength concrete is not yet complete. Furthermore, as the use of columns constructed by concrete of considerably high strength like ultra-high-strength concrete is becoming more common, the investigation of structural stability and safety is required for those ultra-high-strength concrete columns in combination with normal strength concrete slabs.

The overall objective of this research project is to investigate the strategic use of ultra-high-strength-fiber-reinforced concrete in two-way slab construction to provide a practical means of improving performance. In this experimental test, the ultra-high-strength concrete was used for the puddling. In addition, steel fibers were added to the puddled concrete. And the retrospective data for the use of high-strength concrete in the slab were analyzed. This paper investigates the effects of the high-strength-fiber-reinforced concrete puddling and of the use of high-strength concrete on the punching shear capacity.

2. Details of Test Specimens

Two-way slab-column specimens were tested to investigate the punching shear capacity at McGill University. Each specimen had a different concrete compressive strength. The different specimens were identified as shown in Fig. 1. The normal strength concrete slab Specimen S1 provides the benchmarks with which to compare the responses of slab specimens puddled with high-strength concrete. Specimen S1 and S2 were tested at McGill University by McHarg (2000) and Ghannoum (1998), respectively. Therefore test results of Specimen S1 and S2 were analyzed by using the retrospective data. Specimen S3 contained the ultra-high-strength-fiber-reinforced concrete puddling over the entire depth of the slab, but only in the immediate vicinity of the column, to study the influence of strategically puddle ultra-high-strength- fiber-reinforced concrete on crack control, stiffness, and punching shear strength. One of

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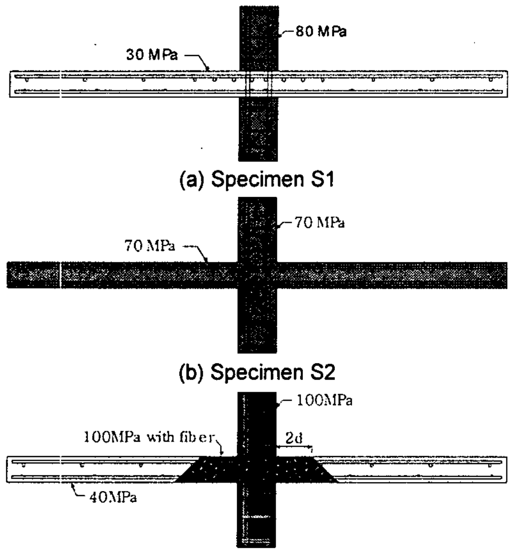

(a) Specimen S1

(b) Specimen S2

(c) Specimen S3

Fig. 1 Two-way slab test specimens

the main advantages of using ultra-high-strength-fiber-reinforced concrete is that this puddled concrete is stiffer and hence, much more able to maintain its shape while the normal strength concrete is being placed around it, thus eliminating the need for containing the puddled concrete. The normal strength concrete can then be placed in the remaining area of the slab followed by vibrating both concretes.

The test specimen shown in Fig. 2, consist of a slab that is 125 mm (5 in.) thick and 2.3 m (7.5 ft) square with 175 mm (7 in.) square reinforced high-strength-concrete column stubs extending 300 mm (1 ft) above and below the slab.

To ensure a shear failure the columns were chosen to be relatively small and the amount of reinforcement was distributed such that the slabs would have sufficient flexural strength to meet the codes requirements and avoid a flexural failure. In accordance to the requirements of the CSA Standard (1994), top reinforcing bars were distributed in a banded manner where the reinforcement was concentrated in the vicinity of the column. The two additional bars in the weak direction were placed to improve the slabs flexural strength in that direction.

The lower column stub of the slab-column specimen was placed on a steel supporting block, and the slab was loaded with eight equal concentrated loads around the perimeter to simulate a uniformly distributed load on the test specimen.

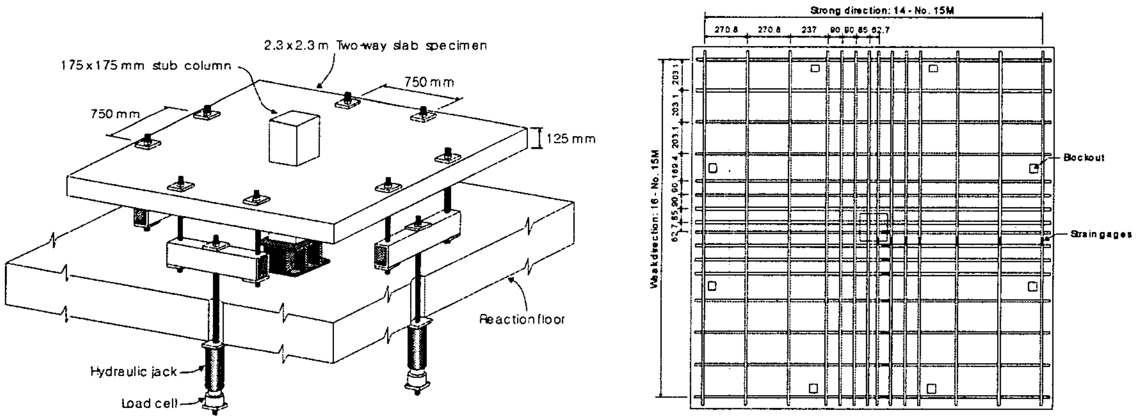

Fig. 2 Test setup and the layout of the top reinforcement

3. Test Results

3.1 Stress-Deflection Responses

Figure 3 compares the total shear stress versus average deflection responses for the three slab-column specimens. As shown in Table 1, it can be seen that the first cracking stresses increased with the increase in the concrete compressive strength of the slab test specimens. And the increase in compressive concrete strength in the slab test specimens resulted in increases in shear stress at first yielding corresponding to 13% for Specimen S2 and 54% for Specimen S3, when compared to Specimen S1. The ultimate capacity of the slab specimens was influenced directly by the increasing the concrete compressive strength in the immediate column region. In addition, the placement of fibers into the puddled concrete increased the load-carrying capacity of Specimen S3.

Table 2 includes the stiffness and the ductility of the slab specimens. In the load-deflection responses, the slope of straight line extended from the point of first cracking up to the point of first yielding in the top reinforcing mat represents the postcracking stiffness, K. Stiffness of Specimen S2 and S3 were 7% and 21% larger than Specimen S1, respectively. The ductility is usually quantifies as the ratio of the deflection at the peak load to the deflection at first yielding of the steel flexural reinforcement. It is noted that the use of higher strength concrete resulted in an increase in the ductility of the slab specimens.

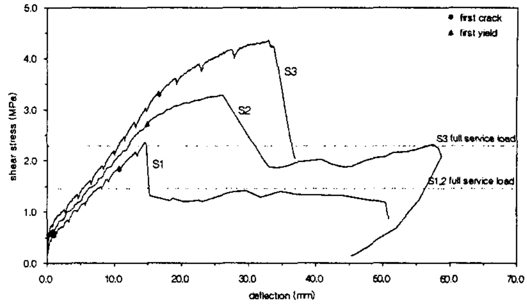

Fig. 3 Stress-deflection responses

Table 1 Summary of shear stress-deflection curves for punching shear tests

Specimen		First Cracking	Full Service Load	First Yielding	Peak Load
S1	stress (MPa)	0.53	1.45	1.85	2.37
	deflection (mm)	1.00	7.80	10.70	15.30
S2	stress (MPa)	0.61	1.45	2.30	3.29
	deflection (mm)	0.85	6.28	12.11	26.05
S3	stress (MPa)	0.55	2.28	3.33	4.37
	deflection (mm)	0.27	10.51	16.53	33.01

Table 2 Stiffness and ductility for punching shear test specimens

Specimen	Stiffness K (MPa/mm)	Ductility Δ_u/Δ_y
S1	0.14	1.43
S2	0.15	1.74
S3	0.17	2.00

3.2 Maximum Crack Widths

Table 3 shows the maximum crack widths at full service load for the slab specimens. Specimen S3 displayed excellent performance for crack control. Specimen S3 had not only the fewest cracks but also the smallest crack widths in the immediate column region. The crack control of Specimen S3 was greatly affected by the use of ultra-high-strength concrete and by the presence of the steel fibers. The steel fibers in the concrete matrix bridge the cracks that form and limit their growth as loading is increased.

Table 3 Maximum crack width at full service load for punching shear test specimens

Specimen	Maximum Crack Width at Full Service Load (mm)	
	Inside "immediate column region"	Outside "immediate column region"
S1	0.35	0.40
S2	0.40	0.33
S3	0.15	0.20

3.3 Comparison of Failure Loads to Code Predictions

In Table 4, the experimental results obtained for the punching shear strength of the slab specimens is compared to the nominal shear strength predicted by the ACI Code (2002), the CSA Standard (1994), the BS Standard (1985) and the CEB-FIP Code (1990). The predictions of the all codes underestimate the punching shear strength for all slab specimens except the punching shear strength of Specimen S1 predicted by the BS Standard (1985). The shear strength provision of the British codes can be unsafe under certain condition, particularly for banded reinforcing slabs with low concrete compressive strength.

Table 4 Comparison of failure loads to code predictions for punching shear test specimens

Specimen	f_c' (MPa)	V_{test} (kN)	$V_{test}/V_{ACI\,\&\,CSA}$	V_{test}/V_{BS}	$V_{test}/V_{CEB-FIP}$
S1	30	349	1.31	0.99	1.28
S2	67.1	485	1.22	1.25	1.50
S3	92.1	386	1.38	1.34	1.54

4. Conclusion

1) Providing puddled ultra-high-strength-fiber-reinforced concrete in the slab around the column for the distance of 2d (that is, 170 mm for Specimen S3) from the column face results in a significant improvement in performance. This includes an increase in the punching shear resistance, a significant increase in the shear stress at first yielding, greater postcracking stiffness, an increase in the ductility, and excellent performance for crack control. The increase in punching shear resistance due to the ultra-high-strength-fiber-reinforced concrete puddling in the immediate column region was 84% over Specimen S1.

2) Increasing the concrete compressive strength of slabs results in an improvement in their performance, with an increase in the punching shear resistance, a significant increase in the shear stress at first cracking, an increase in the shear stress at first yielding, greater postcracking stiffness, and an increase in the ductility.

3) The design expressions in the ACI Code (2002), the CSA Standard (1994), the BS Standard (1985) and the CEB-FIP Code (1990) give conservative predictions for the punching shear strength of the all slab specimens except Specimen S1. The BS Standard (1985) overestimates the punching shear strength of Specimen S1. The shear strength provision of the British codes can be unsafe under certain condition, particularly for banded reinforcing slabs with low concrete compressive strength.

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