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Muhammad N. S Hadi
University of Wollongong, mhadi@uow.edu.au

Hua Zhao
hz99@uow.edu.au

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Experimental Study of High-Strength Concrete Columns Confined with Different Types of Mesh under Eccentric and Concentric Loads

Muhammad N. S. Hadi¹ and Hua Zhao²

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Abstract: A new idea is investigated in this study to reduce the cover spalling of high-strength concrete columns. This idea is to install relatively cheap materials such as household fly screen and wire mesh in the formwork of RC columns. Three materials were chosen in this study: fiberglass fly mesh (FGFM), standard aluminum fly mesh (SAFM), and 12.7 × 12.7 galvanized steel wire mesh (S12.7 WM). A total of 16 cylindrical specimens with a length of 925 mm and a diameter of 205 mm were cast and tested under concentric, eccentric, and pure bending loading. The testing results showed that S12.7 WM significantly improved the load-carrying capacity under both concentric and eccentric loading, but it did not significantly increase the ductility of the columns for each load case. FGFM and SAFM significantly improved the ductility of columns under concentric loading, but the significance decreased with the increase of eccentricity. DOI: 10.1061/(ASCE)MT.1943-5533.0000234. © 2011 American Society of Civil Engineers.

CE Database subject headings: High-strength concrete; Concrete columns; Fiberglass; Mesh generation; Ductility; Concentrated loads; Experimentation.

Author keywords: High-strength concrete column; External confinement; Fiberglass fly mesh; Standard aluminum fly mesh; 12.7 × 12.7 galvanized steel wire mesh; Ductility.

Introduction

High-strength concrete (HSC) has the advantage over normal-strength concrete (NSC) in strength and durability. However, NSC outperforms HSC in ductility. In addition, HSC columns are subject to early concrete cover spalling, which reduces the load-carrying capacity of the columns (Foster and Attard 2001).

Fiber-reinforced polymers (FRP) are widely used confinement materials for civil infrastructure applications because many theoretical and experimental studies have proved that FRP composite jackets can significantly increase the compressive strength and ductility of concrete columns; for example, Parvin and Wang (2001), Lam and Teng (2002), and Hadi (2007a, b). Among fiber-reinforced polymers, carbon-reinforced polymer (CFRP) is most widely adopted in practice. The effectiveness of other reinforced polymers such as glass-reinforced polymer (GFRP) and aramid-fiber-reinforced polymer (AFRP) have been addressed by researchers; for instance, Akguzel and Pampanin (2010) and Wu et al. (2009). In addition to FRP, stainless steel wire mesh composite (Choi 2008) and composite grid (Bentayeb 2007) have also been investigated as retrofitting methods to improve strength and ductility of concrete columns. However, owing to the high manufac-

turing and application costs of these materials, the need to investigate other possible wrapping materials has arisen.

This paper presents a new idea of using relatively cheap materials to confine HSC columns. The materials investigated in this study include fiberglass fly mesh (FGFM), standard aluminum fly mesh (SAFM), and 12.7 × 12.7 galvanized steel wire mesh (S12.7WM). In each case, the confining material was installed in the mold before casting. A total of 16 specimens with a length of 925 mm and a diameter of 205 mm were cast and tested under concentric, eccentric (25 and 50 mm), and pure bending loading.

The specimens were divided into four groups, as shown in Table 1. The specimens of C Group, labeled C0, C25, C50, and CB, were considered as reference specimens and no confinement mesh was provided, whereas one layer of FGFM was applied to the specimens of FG Group, labeled FG0, FG25, FG50, and FGB. The specimens of SA Group, labeled SA0, SA25, SA50, and SAB were confined with one layer of SAFM. Finally, specimens of the WM Group, labeled WM0, WM25, WM50, and WMB, were confined with one layer of S12.7WM.

From each group, one specimen (C0, FG0, SA0, and WM0) was tested concentrically, one specimen (C25, FG25, SA25, and WM25) was tested with a 25 mm eccentric load, and one specimen (C50, FG50, SA50, and WM50) was tested with a 50 mm eccentric load. The remaining specimen of each group (CB, FGB, SAB, and WMB) was tested as a beam under four-point loading.

Experimental Program

Ready-mix high-strength concrete was supplied by a local concrete supplier with target strength of 100 MPa. Owing to the lack of permission from this supplier, the concrete mix design is not provided. The complete experimental design included column geometry design, steel reinforcement design, formwork, and external confinement design. The specimens were cylindrical, with a height of

¹Associate Professor, School of Civil, Mining and Environmental Engineering, Univ. of Wollongong, Australia (corresponding author). E-mail: mhadi@uow.edu.au

²Ph.D. student, School of Civil, Mining and Environmental Engineering, Univ. of Wollongong, Australia. E-mail: hz99@uow.edu.au

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Table 1. Configuration of Specimens

Specimen	Diameter (mm)	Height (mm)	Internal reinforcement	Confining material	Test eccentricity (mm)
C0	205	925	Yes	None	0
C25	205	925	Yes	None	25
C50	205	925	Yes	None	50
CB	205	925	Yes	None	Bending
FG0	205	925	Yes	1-layer FG fly mesh	0
FG25	205	925	Yes	1-layer FG fly mesh	25
FG50	205	925	Yes	1-layer FG fly mesh	50
FGB	205	925	Yes	1-layer FG fly mesh	Bending
SA0	205	925	Yes	1-layer SA fly mesh	0
SA25	205	925	Yes	1-layer SA fly mesh	25
SA50	205	925	Yes	1-layer SA fly mesh	50
SAB	205	925	Yes	1-layer SA fly mesh	Bending
WM0	205	925	Yes	1-layer S12.7WM	0
WM25	205	925	Yes	1-layer S12.7WM	25
WM50	205	925	Yes	1-layer S12.7WM	50
WMB	205	925	Yes	1-layer S12.7WM	Bending

925 mm and a diameter of 205 mm. All specimens were internally reinforced with the same amount of steel reinforcement. In compliance with Standards Australia (2009), 6N12 bars (12 mm diameter deformed bars with nominal tensile strength of 500 MPa) were chosen for the longitudinal reinforcement, and R8 (8 mm diameter plain steel with nominal tensile strength of 250 MPa) with a pitch of 50 mm was adopted for the lateral reinforcement in the form of helices (spirals). Concrete cover was 20 mm thick on the circular surface and 20 mm at top and bottom. All tests were carried out in the Engineering Laboratories at the University of Wollongong.

The objectives of the experimental program were to provide fundamental knowledge of new potential confinement materials and to investigate the performance of HSC columns and beams confined with fly mesh or steel wire mesh.

Specimen Preparation

The molds were made of PVC pipe with an inner diameter of 205 mm. A vertical cut was made through the entire length of each mold that allowed it to be removed from the concrete specimen. Each mold was stabilized vertically by three galvanized steel straps and two hose clips. The inside of each mold was evenly greased with Vaseline before the steel reinforcement cage was placed.

To form longitudinal reinforcement, the N12 steel bar was cut into 885-mm-long pieces. The R8 steel bar was outsourced to a local steel fabrication company to be formed into 16 helices (spirals) with a pitch of 50 mm and an external diameter of 165 mm. These helices were then welded to the required length of 885 mm.

Six N12 steel bars were then fixed to each helix with steel wire to form a reinforcement cage.

Three types of mesh, FGFM, SAFM, and S12.7WM, were used to confine 12 specimens. All the confinement meshes were placed into the molds before casting. The mesh was overlapped at a length of 110 mm. To ensure that the overlap would not allow loosening or sliding, it was sewn in two rows through the whole length (910 mm) with a fishing line. The fishing line used had a diameter of 0.35 mm and its load capacity was 164.64 N. Consequently, the tensile strength of the fishing line was 1712 MPa. The length of each piece of the sewing was approximately 35 mm, as shown in Fig. 1.

Mesh S12.7WM was prepared in a slightly different way. The wire mesh was formed as a tube with an overlap of approximately 120 mm. To ensure that the overlap would not allow loosening or sliding, it was tied with steel wire at a spacing of 90 mm through the whole length (910 mm) of the mesh at each side of the overlap, as shown in Fig. 2.

The concrete was delivered by a local supplier to the engineering laboratory (concrete mix design was provided by the concrete supplier). Wheelbarrows were used to transfer the concrete from the truck to the molds in the laboratory. Then the concrete was placed using scoops. To achieve complete compaction, a 30 mm immersion vibrator was used during the casting. The surface of the concrete was leveled with a trowel to ensure a smooth and even finish. After casting, all the concrete columns were covered with wet hessian and plastic sheets on top to prevent moisture loss

Fig. 1. SAFM external confinement (photo courtesy of the writers)

Fig. 2. S12.7WM external confinement (photo courtesy of the writers)

Fig. 3. Formwork and curing (photo courtesy of the writers)

and allow continual hydration of the cement, as shown in Fig. 3. All the columns were watered during the weekdays until the molds were removed.

Placing the mesh tube into the mold prior to concrete casting made the confining system slightly different from what is called external confinement. The mesh was covered with a very thin layer of concrete, approximately 2-mm-thick, which may affect the integrity of the concrete specimens. However, as the concrete cover on the mesh is thin, the influence is limited. Such a thin layer was reported by Mu et al. (2002), where the fabric mesh was placed on the tension side of the beam specimens with 2 mm concrete cover. Polypropylene mesh with a 4.5×4.5 mm grid and glass mesh with a 5×5 mm grid were investigated in this work. Preventing exposure of the FRP to the environment, the concrete cover can protect the mesh from environmental effects and avoid the aging problem. However, it is noted that this proposed construction method is not appropriate for severe environmental conditions.

Mesh Confinement Testing

Three types of mesh, FGFM, SAFM, and S12.7WM, were investigated as potential wrapping materials. The first two materials were tested using the Instron 4302 with a capacity of 10 kN at a speed of 2 mm/min. The details of the testing results are summarized in Table 2, and Figs. 4–6 present the stress-strain diagram for each confinement material.

As shown in Table 2, the average ultimate tensile stress and strain of SAFM were 243.86 MPa and 0.02926, respectively, whereas FGFM had an average ultimate tensile stress of 972.90 MPa and an ultimate strain of 0.05395. S12.7WM specimens were tested using the Instron 8033 with a capacity of 500 kN at a speed of 2 mm/min. Its average ultimate tensile

Fig. 4. FGFM tensile strength test

Fig. 5. SAFM tensile strength test

Fig. 6. S12.7WM tensile strength test

Table 2. External Confinement Tensile Strength Testing Results

Sample	FG1	FG2	SA1	SA2	SA3	WM 1	WM 2	WM 3
Total length (mm)	300	300	300	300	300	500	500	500
Thickness (mm)	0.137	0.127	0.270	0.273	0.277	0.737	0.713	0.740
Width (mm)	20	18	18	22	23	70	70	70
Cross section (mm ²)	0.1613	0.1260	0.6295	0.7624	0.8412	2.9820	2.7961	3.0091
Ultimate load (kN)	0.164	0.117	0.16	0.18	0.21	1.774	1.794	1.706
Average ultimate load (kN)	0.141			0.183			1.758	
Ultimate stress (MPa)	1,016.85	928.95	249.41	236.09	246.07	594.74	641.60	566.96
Average ultimate stress (MPa)	972.90			243.86			601.10	
Ultimate strain	0.05526	0.05263	0.02989	0.02632	0.03158	0.01491	0.01743	0.01536
Average ultimate strain	0.05395			0.02926			0.01590	

strength and ultimate strain were 601.10 MPa and 0.01590, respectively. Obviously, FGM had the maximum value in strain, followed by SAFM and S12.7WM. The average ultimate strain for each type of material is much higher than the yield concrete strain of 0.003 that is specified in Standards Australia (2009).

For each sample, the cross-sectional area was calculated using the average area of each strand multiplied by the number of strands. The average area of each strand was based on the average diameter of each strand, which was measured from the top, middle, and bottom of each strand.

Preliminary Material Property Testing

The slump test was conducted complying with Standards Australia (1998). The result shows that the concrete had a slump of 155 mm. Concrete cylinder samples of 100 mm in diameter and 200 mm height were scheduled to be tested for compressive strength at 7, 14, and 28 days. The tests planned for 28 days were actually conducted at 36 days. The specimens were cast complying with Standards Australia (2000) and the test was conducted based on Standards Australia (1999). Hard rubber caps were used for the cylinders aged 7 days, and high-strength plaster called Hydrostone was used for the 14 and 36 day tests. The average compressive strength at 7, 14, and 36 days were 64.33, 76.72, and 92.55 MPa, respectively. The compressive strength of concrete at 28 days was calculated to be 86.08 MPa, with a correlation of 0.9524.

The steel specimens were placed under a monotonically increasing tensile load, and measurements of load and extension were recorded by the Instron 8033 tensile testing machine, which has a capacity of 500 kN. The average yield tensile strength and ultimate strength of R8 were 547 and 620 MPa, respectively. For N12 reinforcement, the average yield tensile strength and ultimate strength were 572 and 663 MPa, respectively.

Specimen Testing

All the columns and beams were tested using the Denison testing machine, which has an ultimate compressive load capacity of 5000 kN. The loading caps shown in Fig. 7 were adopted. Before the test, the column was capped with high-strength plaster on both bottom and top. Two hours were allowed for the plaster to dry. Four bolts were adopted symmetrically on each cap. A pair of knife edges was applied for eccentric loading. As shown in Fig. 8, the total height of the knife edge was 27 mm, and the edge plate was 18 mm high. Before the load was applied on the eccentrically loaded columns, knife edges were placed at the grooves at either 25 or 50 mm from the center line of the loading cap, as shown in Fig. 7.

Fig. 7. Column loading caps (eccentricity in mm) (photo courtesy of the writers)

Fig. 8. The bottom pair of knife edges (photo courtesy of the writers)

The laser measurement device $\mu\epsilon$ Micro-Epsilon (Class Two) was adopted to measure the midheight lateral deflections of the columns under concentric and eccentric loading. In the four-point bending test, this laser was also used to measure the midspan deflections of the beam specimens. An LVDT (Micro-Measurement LDC1000A) was adopted for measuring the axial displacement of the column specimens, as shown in Fig. 9.

Four specimens, CB, FGB, SAB, and WMB, were tested as beams. The details are shown in Fig. 10. A four-point loading

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Fig. 9. LVDT (photo courtesy of the writers)

Fig. 10. Beam test (photo courtesy of the writers)

Fig. 11. Load-deflection curves under concentric loading

system composed of two parts, top rig and bottom rig, was adopted for the test. The beam specimen was placed on the bottom rig, diagonally placed on the bottom plate of the Denison machine. There was a hole at the middle of the support beam and the laser was put right under the hole, facing the middle of the specimen. The laser provided the measurement of the deflection. The top rig included two steel cylinders, a square steel plate, and a steel beam. The top plate of the Denison machine was adjusted to the top of the steel cylinder. Both the beams and the columns were tested by displacement control and the ramp rate was set at 0.5–1 mm/min.

Results and Discussion

Reinforced Concrete Columns Under Concentric Loading

Four columns, marked C0, FG0, SA0, and WM0, were tested under a concentric load. The results of the four columns concentrically tested are presented in Fig. 11 and Table 3. Specimen WM0 had the largest ultimate load. This high ultimate load is because WM0 had the largest compressive strength of mesh-confined concrete, which depends on the tensile strength and the thickness of the confinement, as shown in Table 4. The ultimate load of Specimens FG0 and SA0 was lower than the reference Column C0, which may be the result of the constraint of the experiment, because it was difficult to ensure that the wrapping mesh was fully and uniformly covered with concrete. The bond between the mesh and the concrete is also questioned. As a result, surface imperfection was observed at the specimens. The ultimate load of Specimen FG0 was higher than that of Specimen SA0. The failure of the four specimens was presented by cover spalling, longitudinal bars buckling, and helix rupture. For Specimen WM0, no helix snapped in the exposed areas, despite two buckled longitudinal bars. Ductility gives warning of failure, which is a desirable trait of any structural material (Nawy 2003). Based on Pessiki and Pieroni's approach (1997), the equation $\mu_{85} = \delta_{85}/\delta_y$ was used for ductility analysis, where μ_{85} = ductility based on 85% of ultimate load; δ_{85} = post-yield axial deflection at 85% of the ultimate load; δ_y = axial

Table 3. Summary of Specimen Testing

Column specimen	C0	FG0	SA0	WM0
Ultimate load (kN)	2,544	2,321	2,293	2,876
Yield load (kN)	2,544	2,321	2,140	2,876
Axial deflection at yield load (mm)	3.42	3.13	3.07	3.94
Axial deflection at ultimate load (mm)	3.63	3.34	3.67	4.42
Ultimate axial deflection (mm)	28.85	30.20	49.07	21.81
Ductility	1.188	1.940	1.900	1.150
Column specimen	C25	FG25	SA25	WM25
Ultimate load (kN)	1,785	1,019	1,617	1,873
Yield load (kN)	1,785	993	1,617	1,873
Axial deflection at yield load (mm)	3.72	3.48	4.39	3.74
Axial deflection at ultimate load (mm)	3.86	4.06	4.32	4.00
Ultimate axial deflection (mm)	14.78	19.28	18.06	16.84
Ductility	1.070	1.192	1.178	1.092
Column specimen	C50	FG50	SA50	WM50
Ultimate load (kN)	841	1,058	1,012	1,103
Yield load (kN)	841	1,058	1,012	1,103
Axial deflection at yield load (mm)	2.84	3.05	3.11	3.13
Axial deflection at ultimate load (mm)	2.84	3.28	3.36	3.29
Ultimate axial deflection (mm)	8.66	13.55	6.10	5.09
Ductility	1.018	1.106	1.095	1.091
Beam specimen	CB	FGB	SAB	WMB
Ultimate load (kN)	262	256	286	Lost
Yield load (kN)	195	186	198	—
Axial deflection at yield load (mm)	4.11	3.97	4.29	—
Ultimate axial deflection (mm)	29.34	45.17	65.77	—
Ductility	2.380	2.759	2.761	—

Table 4. Compressive Strength of Confined Concrete

Confining Material	d_{col} (mm)	t (mm)	f_{FRP} (MPa)	$f_{FRP}(t)$ (MPa)	$2f_{FRP}(t)/df_1$ (MPa)	f'_{co} (MPa)	f_1/f'_{co}	$[1 + 2 \times (f_1/f'_{co})] \times f'_{co} f'_{cc}$ (MPa)
FGFM	205	0.1317	972.90	128.10	1.24974	86.08	0.0145	88.58
SAFM	205	0.2733	243.86	66.65	0.65028	86.08	0.0076	87.38
S12.7 WM	205	0.7300	599.32	437.50	4.26832	86.08	0.0496	94.62

Note: f_{FRP} and t are the tensile strength and the thickness of the mesh confinement, respectively.

deflection at the yield point. As shown in Table 3, Specimen FG0 had the highest value of ductility.

Reinforced Concrete Columns Under Eccentric Loading

A total of eight columns with different confinement were tested under eccentric loading. Four columns marked C25, FG25, SA25, and WM25 were loaded under 25 mm eccentricity. Then the last four columns, labeled C50, FG50, SA50, and WM50, were tested under 50 mm eccentricity.

Table 3 shows the experimental results for the columns under eccentric loading of 25 mm, and Fig. 12 illustrates the load-deflection curves for the four columns. All the specimens failed by lateral helix dilation and concrete cover spalling, although longitudinal reinforcement buckling was only observed in Specimen C25.

Fig. 12 and Table 3 show that Column WM25 has the highest ultimate load, followed by C25 and SA25. In contrast, FG25 had the lowest ultimate load, which was 57% of C25. The unexpected low ultimate load of Specimen FG25 may result from premature failure. Consequently, Specimen FG25 may not have been tested to its ultimate strength. Similarly, the ultimate load of SA25 was 91% of the reference Column C0 owing to premature failure, which may relate to significant imperfections occurring during the casting and/or curing. For 25 mm eccentric loading, Specimens FG25 and SA25 had relatively high ductility and Specimen FG25 had the highest ductility.

Specimens marked C50, FG50, SA50, and WM50 were tested under 50 mm eccentricity. The results of the four specimens are presented in Table 3 and the load-deflection curves are shown in Fig. 13. The lateral deflection of WM50 is not included owing to an error during the test.

Fig. 13 and Table 3 show that the reference Column C50 had the lowest ultimate load and Column WM50 had the highest value of 1,103 kN, which is 1.31 times that of C50. Column FG50 had the second highest value of 1,058 kN, which is 1.26 times that of C50, closely followed by SA50. Among the four 50 mm eccentrically loaded specimens, FG50 had the highest ductility.

The eccentric loading test showed that the wire mesh significantly increased column load capacity and slightly improved ductility. Although the load capacity was not improved for Specimens

Fig. 12. Load-deflection curves under eccentric loading, $e = 25$ mm

Fig. 13. Load-deflection curves under eccentric loading, $e = 50$ mm

FG25 and SA25 owing to premature failure, the load capacity of Specimens FG50 and SA50 were increased significantly. In terms of ductility, FGFM had the most significant influence, outperforming SAFM and S12.7WM.

Beam Specimen Testing

Four specimens marked CB, FGB, SAG, and WMB were loaded at four-point loading to determine the ultimate bending moment capacity of the four tested specimens. The results of Specimen WMB were lost. Fig. 14 shows the load-midspan deflection of the remaining beam specimens. Among the three specimens, the beam with standard aluminum fly mesh (SAB) had the highest load capacity.

The results of the three beam specimens are summarized in Table 3. This table shows that Specimen BFG had the lowest yield deflection and the lowest ultimate load among the three beams. This is properly related to imperfection. However, the ductility of the concrete beam was improved by applying FGFM and SAFM. Beam SAB had the highest value of ductility, closely followed by Specimen FGB.

Theoretical Calculation

The squash load of unconfined reinforced-concrete columns equals the force in concrete plus the force in steel. The

Fig. 14. Load-midspan deflection of specimens

following equation was adopted to calculate the axial concentric load capacity (Warner et al. 2007):

$$N_{uo} = \alpha_1 f'_c A_g + A_s f_{sy} \quad (1)$$

where N_{uo} = force in concrete plus the force in steel; f'_c = compressive strength of unconfined concrete; A_g = gross area of a cross section; f_{sy} = yield strength of steel reinforcement; A_s = area of steel reinforcement; $\alpha_1 = 1.0 - 0.003f'_{co}$, within the range $0.72 \leq \alpha_1 \leq 0.85$. In this case, because f'_{co} is approximately 85 MPa, α_1 is 0.745 ($= 1.0 - 0.003 \times 85$). The way to determine α_1 is consistent with Standards Australia (2009). For concrete columns confined with mesh, f'_c is replaced by the compressive strength of the confined concrete, f'_{cc} , as shown in Eq. (5). This adopted equation is slightly different from that proposed by Rocca et al. (2009), in which α_1 is taken as 0.85 and the cross section of the concrete member equals $A_g - A_s$.

For columns with a circular cross section, the gross area A_g was considered as the effective area A'_c , which was calculated using Eq. (2), proposed by Warner et al. (1998):

$$A'_c = \frac{1}{2} r^2 (\alpha - \sin \alpha) \quad (2)$$

$$b_o = 2\sqrt{[2\gamma d_n r - (\gamma d_n)^2]} \quad (3)$$

$$\alpha = 4 \tan^{-1} \left[\frac{2\gamma d_n}{b_o} \right] \quad (4)$$

where d_n = neutral axis depth; b_o = maximum width; r = radius of the section; α = angle subtended at the center of the circle by b_o .

Equations proposed for FRP-confined columns were adopted in this study. These equations were used as an approximation because there is no specific equation for mesh-confined concrete columns. According to Lam and Teng (2002), the compressive strength of the confined concrete is calculated using Eq. (5), and f_l is computed by Eq. (6):

$$\frac{f'_{cc}}{f'_{co}} = 1 + 2 \frac{f_l}{f'_{co}} \quad (5)$$

$$f_l = \frac{2f_{FRP}t}{d} \quad (6)$$

where f'_{cc} = compressive strength of the confined concrete, and in this study, f'_{cc} is adopted for computing the compressive strength of concrete columns confined with mesh [as shown in Eq. (1)]; f'_{co} = unconfined concrete; f_l = lateral confining pressure, and the value 2 is the confinement effectiveness coefficient suggested. In the original equations proposed by Lam and Teng (2002), f_{FRP} is the tensile strength of the FRP in the hoop direction; t is the total thickness of the FRP. However, in this study, f_{FRP} and t are the tensile strength and the thickness of each type of mesh confinement, respectively. The diameter of the confined concrete column is d .

The results of the compressive strength of the confined concrete are presented in Table 4. The unconfined concrete compressive stress at 28 days (f'_{co}) was chosen to compute the compressive strengths of the confined concrete (f'_{cc}).

A spreadsheet was used to create the theoretical section capacity line. The section capacity lines for each group are shown in Fig. 15. All specimens were tested to failure. For specimens tested at 25 or 50 mm eccentricity, the bending moment capacity, including the

Fig. 15. Theoretical interaction diagram

secondary moments, was calculated by multiplying the ultimate load capacity (P_{ult}) by the sum of eccentricity (e) and the lateral deflection (δ) at the ultimate load, as shown in Eq. (7). This equation was indicated in the work of Warner et al. (1998). The specimen test results are shown in Table 5, and the experimental curves are presented in Fig. 16:

$$M = P_{ult}(e + \delta) \quad (7)$$

The results for Group C, Group FG, and Group SA are included in Fig. 16. The results for Group WM are not included owing to data loss. The overall trend of the experimental interaction curves is similar to that of the theoretical curves, except in Group FG. There is an obvious discrepancy between the theoretical and the experimental interaction curves for Group FG. The curve of Group FG dropped dramatically because the experimental load capacity was lower than the estimated value. Among the three specimens, specimen SAB produced the largest increase of bending moment for the pure bending loading condition. For the specimens confined externally with FGFM and SAFM, the squash load was unexpectedly lower than that of the reference specimens marked Group C. These

Table 5. Experimental Results

Specimen	Ultimate load (kN)	Maximum moment (kN/m)	Lateral deflection at maximum load (mm)	Eccentricity (mm)
C0	2,544	0	0	0
C25	1,785	53.6	5.055	25
C50	841	45.7	3.613	50
Beam	262	37.0		
FG0	2,321	0	0	0
FG25	1,019	27.9	2.377	25
FG50	1,058	60.2	4.860	50
Beam	256	36.2		
SA0	2,293	0	0	0
SA25	1,617	45.6	3.194	25
SA50	1,012	56.3	3.914	50
Beam	286	40.4		
WM0	2,876	0	0	0
WM25	1,873	55.0	4.374	25
WM50	1,103	55.9	0.027	50
Beam	Lost	N/A

Fig. 16. Experimental interaction diagram

phenomena were predicted based on previous research and are likely to be associated with premature failure. Many studies have indicated that early spalling of cover concrete leads to lower than anticipated failure loads (Cusson and Paultre 1994; Foster et al. 1998; Bayrak 1999). According to Bae and Bayrak (2003), the loss of cover concrete before reaching the theoretical axial load-carrying capacity is a peculiar problem that can only be associated with HSC columns.

The experimental results of this study indicate that the current models, proposed to estimate the confined concrete strength of FRP-confined concrete columns, need to be adjusted for mesh confinement. As indicated by Eid and Paultre (2008), the current models have been developed primarily on the basis of experimental results or steel-confined concrete models, which are related to the form of the equation proposed by Richart et al. (1929) to define the confined concrete strength. Wire mesh and fly mesh are different from FRP wrapping materials in mechanical properties, geometry, and surface condition. No study has been found to use mesh as confinement material in the proposed manner. The development of a proper model for mesh-confined concrete will depend on experimental work with a large number of specimens.

Based on the results of C Group and WM Group under concentric loading, the value of α_1 , either computed by $1.0 - 0.003f'_c$ or just assumed as 0.85 (Eid and Paultre 2008), made no significant difference. However, these results do not imply that choosing a proper way to determine α_1 is not necessary. Further testing is required to fully understand the influence of this variable.

The discrepancy between the theoretical interaction curve and the experimental curve may be related to imperfect bonding between mesh and concrete, which affected the estimation of confined concrete strength. As a result, the experimental diagram is different from the theoretical interaction diagram. A similar discrepancy between the theoretical interaction diagram and the experimental diagram is also reported in the study carried out by Rocca et al. (2009).

Bond strength between mesh and concrete is considered as the key issue that affects the behavior of the concrete specimens in this experimental study. The mesh confinement materials were assumed to be bonded perfectly with the concrete until the load reached its maximum value. Therefore, the lateral confining pressure can be determined from the tensile stress of the mesh confinement materials. However, the results suggest that the bonding between the mesh and concrete is imperfect. Imperfect bonding is considered to accelerate premature failure.

Currently, no study is available on the modeling of bonding between mesh and concrete. The bond-slip model, proposed by Zhou et al. (2010) for externally bonded FRP joints, is not appropriate for the mesh application because the mesh was not applied to the concrete surface and the geometry and surface condition of FRP and mesh is completely different. Furthermore, the two predominant modes of bond failure between steel and concrete pull-out and splitting are not suitable for the mesh application either because mesh is different from steel bars in mechanical properties and geometry.

A range of parameters listed in the following are considered to influence the bonding between the mesh and concrete. The parameters associated with components of the concrete consist of the types of binding minerals, admixtures (concrete mix design was provided by the concrete supplier), and compressive and tensile concrete strength; other factors include concrete cover and mechanical and physical properties of mesh such as thickness, geometry and surface conditions. Additionally, temperature, corrosion level, environmental conditions, and loading history may also influence the bonding.

Further research is needed to investigate the bond strength between mesh and concrete. Because no existing study has been found on using mesh confinement in such a way, a large number of tests have to be conducted. The assumption about bond condition can be verified indirectly by comparing the experimental results of squash load with their counterpart of theoretical calculations. Cylinder specimens without steel reinforcement are recommended for this case.

The discrepancy is also probably related to the quality of concrete and mesh. Small voids appeared on the surfaces of specimens confined with fly mesh. To avoid the imperfection, concrete quality control is required, especially in terms of W/C ratio and

Table 6. Material Cost and Structural Performance

No.	Column specimen	Steel reinforcement (AU\$)	Concrete (AU\$)	Confining material (AU\$)	Cost AU\$/m ³	Ultimate load (kN)	Ductility
1	C0	93	28	0	3,893	2,544	1.188
2	C25	93	28	0	3,893	1,785	1.070
3	C50	93	28	0	3,893	841	1.018
4	FG0	93	28	6	4,093	2,321	1.940
5	FG25	93	28	6	4,093	1,019	1.192
6	FG50	93	28	6	4,093	1,058	1.106
7	SA0	93	28	10	4,219	2,293	1.900
8	SA25	93	28	10	4,219	1,617	1.178
9	SA50	93	28	10	4,219	1,012	1.095
10	WM0	93	28	7	4,130	2,876	1.150
11	WM25	93	28	7	4,130	1,873	1.092
12	WM50	93	28	7	4,130	1,103	1.091

Fig. 17. External confinement material cost—ultimate load

Fig. 18. External confinement material cost—ductility

compaction. The characteristics of the household mesh may vary from producer to producer. However, the use of continuous fiber mesh or wire mesh is attractive because of the economical feasibility of replacing FRP and the potential structural improvements. When such mesh confinement materials are adopted in the industry, certain standards will be in place to ensure the product's quality.

Economic Analysis

The material costs for each column included concrete, steel reinforcement, and external confinement. The costs excluded some miscellaneous expenses used for external confinement such as fishing line and steel wire. All the costs were calculated in Australian dollars. Table 6 shows cost, ultimate load, and ductility of each column. To help the readers visualize the cost difference between the columns confined with different types of mesh, Figs. 17 and 18 are provided. The cost per cubic meter of concrete is calculated by dividing the cost per column with the volume of the column. FGFM, SAFM, and S12.7WM increased the material cost by 5, 8, and 6%, respectively. Wire mesh is an efficient and economic material for increasing capacity because the cost of wire mesh was very close to FGFM, but wire mesh increased the load capacity 13% compared to the reference Column C0. Table 6 also shows that both FGFM and SAFM significantly increased the ductility of the columns. Moreover, FGFM is more economical in terms of ductility improvement.

Conclusions

On the basis of the experimental results of this study, the following conclusions are drawn:

- 1 S12.7WM significantly increases the load-carrying capacity of the column specimens for both concentric loading and eccentric loading. Columns with FGFM or SAFM did not show improvement in load-carrying capacity under concentric loading or 25 mm eccentric loading. The unexpected low load capacity

results are possibly related to premature failure because Specimens FG50 and SA50 had higher load capacity than Specimen C50.

- 2 On the basis of Pessiki and Pieroni's approach (1997), columns confined with FGFM and SAFM outperformed their counterparts confined with S12.7WM in ductility under both concentric loading and eccentric loading, but the significance decreased with the increase of eccentricity. The ductility values of columns confined with FGFM and SAFM were close to one other.
- 3 Among the three beam specimens, CB, FGB and SAB, Specimen SAB had the highest load capacity. FGFM or SAFM improved the ductility of the beam specimens. Again, Specimens SAB and FGB had close results in ductility.
- 4 Finally, the results of this study show that considerable increases in strength and ductility of concrete members can be achieved at modest costs. The results suggest that wire mesh is the most efficient and economic material for increasing the load capacity, and FGFM is the most economical material in terms of ductility improvement.

References

- Akguze, U., and Pampanin, S. (2010). "Effects of variation of axial load and bidirectional loading on seismic performance of GFRP retrofitted reinforced concrete exterior beam-column joints." *J. Compos. Constr.*, 14(1), 94–104.
- Bae, S., and Bayrak, O. (2003). "Early cover spalling in high-strength concrete columns." *J. Struct. Eng.*, 129(3), 314–323.
- Bayrak, O. (1999). "Seismic performance of rectilinearly confined high strength concrete columns." PhD dissertation, Univ. of Toronto, Toronto, 80–187.
- Bentayef, F., Tahar, K. A., and Chateaufneuf, A. (2008). "New technique for reinforcement of concrete columns confined by embedded composite grid." *Constr. Build. Mater.*, 22, 1624–1633.
- Choi, J. H. (2008). "Seismic retrofit of reinforced concrete circular columns using stainless steel wire mesh composite." *Can. J. Civ. Eng.*, 35(2), 140–147.
- Cusson, D., and Paultre, P. (1994). "High-strength concrete columns confined by rectangular ties." *J. Struct. Eng.*, 120(3), 783–804.
- Eid, R., and Paultre, P. (2008). "Analytical model for FRP-confined circular reinforced concrete columns." *J. Compos. Constr.*, 12(5), 541–552.
- Foster, S., Liu, J., and Sheikh, S. (1998). "Cover spalling in HSC columns loaded in concentric compression." *J. Struct. Eng.*, 124(12), 1431–1437.
- Foster, S. J., and Attard, M. M. (2001). "Strength and ductility of fiber-reinforced high strength concrete columns." *J. Struct. Eng.*, 127(1), 28–34.
- Hadi, M. N. S. (2007a). "Behavior of FRP strengthened concrete columns under eccentric compression loading." *Compos. Struct.*, 77(1), 92–96.
- Hadi, M. N. S. (2007b). "The behavior of FRP wrapped HSC columns under different eccentric loads." *Compos. Struct.*, 78(4), 560–566.
- Lam, L., and Teng, G. (2002). "Strength models for fiber-reinforced plastic-confined concrete." *J. Struct. Eng.*, 128(5), 612–623.
- Mu, B., Meyer, C., and Shimanovich, S. (2002). "Improving the interface bond between fiber mesh and cementitious matrix." *Cem. Concr. Res.*, 32, 783–787.
- Nawy, E. G. (2003). *Reinforced concrete: A fundamental approach*, 5th Ed., Prentice Hall, New York.
- Parvin, A., and Wang, W. (2001). "Behavior of FRP jacketed concrete columns under eccentric loading." *J. Compos. Constr.*, 5(3), 146–152.
- Pessiki, S., and Pieroni, S. (1997). "Axial load behavior of large-scale spirally-reinforced high-strength concrete columns." *ACI Struct. J.*, 94(3), 304–314.
- Richart, F. E., Brandtzaeg, A., and Brown, R. L. (1929). "The failure of plain and spirally reinforced concrete in compression."

- Bulletin No.190*, Engineering Experimental Station, Univ.of Illinois, Urbana, IL.
- Rocca, S., Galati, N., and Nanni, A. (2009). "Interaction diagram methodology for design of FRP-confined reinforced concrete columns." *Constr. Build. Mater.*, 23, 1508–1520.
- Standards Australia. (1998). "Determination of properties related to the consistency of concrete—Slump test." *AS1012.3.1*, Sydney, Australia.
- Standards Australia. (1999). "Methods of testing concrete—Determination of the compressive strength of concrete specimens." *AS1012.9*, Sydney, Australia.
- Standards Australia. (2000). "Method of making and curing concrete compression and indirect tensile test specimens." *AS1012.8.1*, Sydney, Australia.
- Standards Australia. (2009). "Concrete structures." *AS3600*, Sydney, Australia.
- Warner, R. F., Foster, S. J., and Kilpatrick, A. E. (2007). *Reinforced concrete Basics*, Person Prentice Hall, Sydney, Australia.
- Warner, R. F., Rangan, B. V., Hall, A. S., and Faulkes, K. A. (1998). *Concrete structures*, Addison Wesley Longman, Melbourne, Australia.
- Wu, H. L., Wang, Y. F., Yu, L., and Li, X. R. (2009). "Experimental and computational studies on high-strength concrete circular columns confined by aramid fiber-reinforced polymer sheets." *J. Compos. Constr.*, 13(2), 125–134.
- Zhou, Y. W., Wu, Y. F., and Yun, Y. (2010). "Analytical modeling of the bond-slip relationship at FRP-concrete interfaces for adhesively-bonded joints." *Composites, Part B*, 41, 423–433.

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