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Behavior of Circular Concrete Columns Repaired with Steel Bars and Wire Mesh-Mortar

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Abstract In order to increase the strengthening efficiency of steel bar mat-mortar (BM) jacket and wire mesh-mortar (WM) jacket around existed circular concrete columns, an attempt to strengthen the columns with hybrid bar mat-wire mesh-mortar (HBWM) jacket was proposed. A comparatively experimental study on axial compression behaviors of concrete columns wrapped with three different strengthening systems, namely BM, HWBM and carbon fiber reinforced polymer (CFRP) was performed. The experiment results showed that much more cracks appeared in HWBM columns compared with those in BM columns. As a result, on the premise that the concrete compressive strength of the HWBM columns increased 90% compared with that of the BM columns, the ductility of the HWBM columns reached about twice as that of the BM columns. The increase of the concrete compressive strength of CFRP strengthened columns was higher than those of HWBM and BM strengthened columns. The ductility of CFRP strengthened columns, however, was obviously lower than that of HWBW columns.

Keywords: Steel bar-wire mesh-mortar, concrete columns, ductility, ultimate load bearing capacity

Introduction

Existed concrete columns may lack lateral confinement and energy absorption capacity. The strength and ductility of such existed columns may be enhanced by constructing external reinforced concrete or wire mesh mortar (WM, also called Ferrocement) cage around existing columns (Waliuddin and Rafeeqi 1994, Takiguchi 2001, Kubaisy 2005, Kondraivendhan and Pradhan2009), or wrapping fiber reinforced polymer (FRP)(Mirmiran and Shahawy 1997, Porter and Harries 2007). WM is a type of thin wall concrete. Because the transverse wires of mesh form preferential locations for cracks the crack spacing is generally equal to the spacing of two transverse wires (i.e. 6-25 mm(ACI 549 1999), leading to a much higher energy absorption capacity. Waliuddin and Rafeeqi (1994), Kondraivendhan and Pradhan (2009) reported that by providing external circular confinement using WM jackets to the original concrete columns, the ductility could be improved significantly. It should be noted that due to the small diameter of the wire mesh, WM jacket can not provided very significant confinement to concrete column (e. g. the area of thirty six 1-mm diameter wires is equal to that of one 6 mm diameter steel bar). Noticing this, Shang and Jiang (2005) proposed a bar (4-8mm diameter) mat-mortar (BM) strengthening system. Their experimental study showed that the concrete compressive strength might be increased greatly by using BM. The ductility of the BM strengthened columns, however, was remarkably lower than that of WM strengthened column; because the former crack spacing (80-100mm) was much larger than that of the latte (6-25mm).

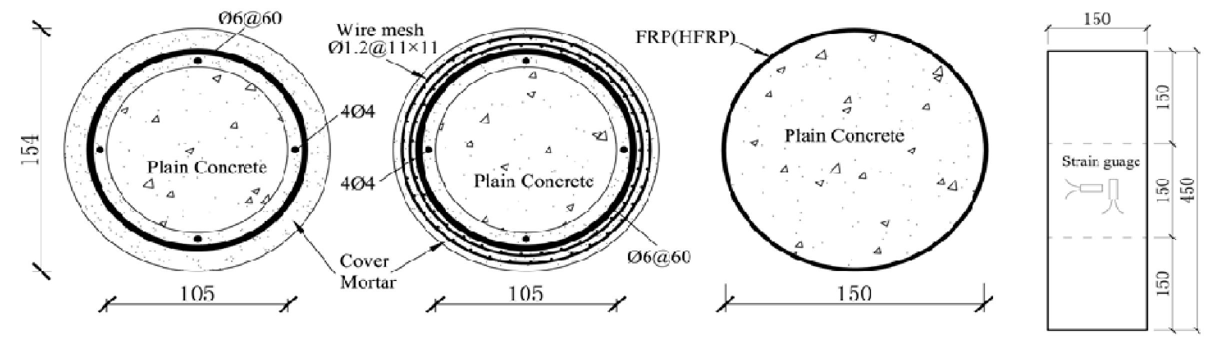
In order to elaborate the advantages of both WM and BM confinement simultaneously, an idea to develop a hybrid bar mat-wire mesh-mortar (HBWM) jacket was proposed for strengthening concrete columns. It was expected that the proposed HWBM confinement might significantly increase both the concrete compressive strength and ductility of the strengthened columns. A comparison experimental study including three strengthening (jacket) systems, namely BM, HWBM and carbon fiber reinforced polymer (CFRP) was performed under axial compression from strength and ductility of point of view.

Experimental Program

Materials Ordinary Portland cement (Chinese Standard GB175-1999, analogy with ASTM C150 Standard Specification) was used. The coarse aggregate consisted of crushed stone with a maximum size of 20 mm and the medium river sand had a modulus fineness of 2.98. The composition of the concrete mixes was 0.44: 1: 1.50: 2.41(water: cement: sand: stone). The 28-day tested concrete cube strength was 38.8MPa. The composition of the repair mortar mixes was 0.4: 1: 2 (water: cement: sand). The 28-day tested mortar cube strength was 35.7MPa. 6-mm diameter steel bars with yield strength of 403MPa and a elastic modulus of 210GPa were used as original and additional stirrups. 4-mm steel bars with a yield strength of 288MPa and an elastic modulus of 210GPa were used as maintain bars. Galvanized weld mesh of 11×11×1.2-mm-diameter was used. The yield strength of the 1.2-mm diameter wires was 350MPa. The nominal thickness of the CF sheet was 0.167 mm. The fiber characteristics were supplied by the manufacturer; the tensile strength and modulus of the CF were 3256MPa and 218GPa, respectively.

Casting A total of 21 concrete cylindrical specimens (15 with a diameter of 105 mm and the other 6 with 150 mm) with a height of 450mm were cast. The test specimens were cast on polyvinyl chloride (PVC) tubes. Immediately after casting, all of specimens were wet-cured by covering with wet burlap for 24 hours at a temperature of about 20^0C. The specimens were then de-molded and transferred to the curing room for further curing of 27 days (Chinese Standard GBJ 81-85, analogous with ASTM C 192). After curing, every PVC tube was dismantled by using an electric iron to cut (melt) a narrow gap in the tube. The fifteen 105 mm diameter specimens were used as specimens to be strengthened with BM or HWBM. The six 150 mm diameter specimens were used as specimens to be strengthened with CFRP laminates.

Strengthening of Concrete Columns The process of applying BM and HWBM to concrete columns involved surface preparation, steel bar application, priming, wire mesh application(for HWBM only), and a mortar plastering with a reference to ACI 549 (1999)(refer Figure 1). All of the strengthened columns were wet-cured by covering with wet burlap for an additional 28 days before testing. The process of applying CF sheet to concrete involved surface preparation, priming, resin undercoating, CF sheet application, and resin over-coating with a reference to ACI 440 (2002).



a. BM strengthening *b. HWBM strengthening* *c. FRP strengthening* *d. Elevation of specimen*

Figure 1: Cross section and elevation of specimens (unit: mm)

Specimen Groups The 21 specimens were divided into four types (namely P, BM, HWBM and FRP) according to different strengthening systems, as shown in Table 1. The four types of specimens were further divided into 7groups (each group has 3 specimens). Groups P1 and P2 were tested as control specimens. Group BM were strengthened with bar mat only. Groups HWBM1, HWBM2 and HWBM 3 were strengthened with bar mat and wire mesh simultaneously and were nominated according to the different layers of wire mesh wrapped. CFRP employed two layers of CF sheet only.

Table 1: Main parameters of specimens

Specimen type	*Specimen group*[a]	*Diameter [mm]*		*Reinforcement ratio[%]*		*Remarks*
		Before strengthening	*After strengthening*	*Stirrup*	*Wire*	
P	P1	105	-	-	-	Plain concrete
	P2	150	-	-	-	Plain concrete
BM[b]	BM	105	150	1.67	-	One layer of bar mat
HWBM[c]	HWBM2	105	148	1.67	0.658	One layer of bar mat + two layers of wire mesh
	HWBM 3	105	154	1.67	0.987	One layer of bar mat + three layers of wire mesh
	HWBM 4	105	160	1.67	1.316	One layer of bar mat + four layers of wire mesh
CFRP[d]	CFRP	150	154	-	-	Two layers of CF

[a] Three specimens for each group.
[b] S = steel bar mat.
[C] SW = steel bar mat and wire mesh.
[d] CFRP = carbon fiber reinforce polymer.

Loading method and Measurements All specimens were loaded in uniaxial compression until failure, using a hydraulic test machine. The load was applied at a constant rate of 0.3MPa/sec. The applied load was measured using a load cell. Axial and lateral strains were measured using electronic strain gages. The Axial and lateral strain gages were installed at the centre point of the cylinders. Moreover, for measurement of average axial strains (especially after the strain gauges reached their readability limits at 2% strain level), three linear variable displacement transducers (LVDTs) were placed at 120^o apart around the specimens. All of the measurements were automatically recorded through a data logger.

Results and Discussion

Load-Deformation behaviors and Failure Modes Typical load-displacement (*P-Δ*) curves for three groups of HWBM specimens are shown in Figure 2. Each curve could be roughly divided into three stages, representing the tension steel pre-yield (including mortar pre-cracking and post-cracking), tension steel post-yield with a yield plateau, and a very slow descending stage. The *P-Δ* curves of HWBM specimens exhibited an almost flat portion at the second stage and a very slow dropping tendency at the third stage, mainly because the steel bars and wire mesh had a bilinear ductile stress-strain behavior and a great number of cracks appeared in the repair mortar (Figure 3). Due to the high specific surface provided by wire mesh (ACI 549 1999), much more cracks developed in the HWBM specimens, both vertical and transverse crack spaces (Figure 3)were nearly equal to the mesh opening size(11×11mm). Bugling at the mid-height of the specimens occurred, followed by spalling of mortar cover at the third stage (ref. Figure 2 and 3). It should be noted that all spalling pieces were quite small and was about the size of mesh opening before failure (Fig 3). Failure in all HWBM strengthened specimens occurred owing to rupture of steel bars and wire mesh, followed by breaking of the core concrete of the middle part of the columns. Evidence indicated that the HWBM columns could undergo much greater deformation before failure, and such a failure was ductile and gave warning of the impending failure.

The shape of *P-Δ* curves of BM specimens was similar to that of HWBM specimens (Figs. 2). The crack number of BM, however, was much fewer than that of HWBM, and the spalling pieces of the mortar cover of BM were about 100 times the size of those of HEBM. As a result, the third stage of the *P-Δ* curve was shorter than that of HWBM columns.

Like other researches (Mirmiran and Shahawy 1997), response of CFRP-strengthened concrete was bilinear with no flat branch because CF had no plastic behavior. Failure in all strengthened specimens occurred owing to rupture of FRP, followed by instantaneous explosive breaking of the concrete cylinder between the lower and upper clamp (see Figure 3). The sudden fracture of CFRP with a small elongation of less than 1% at mid-height of the specimens was responsible for the brittle failure of the strengthened columns.

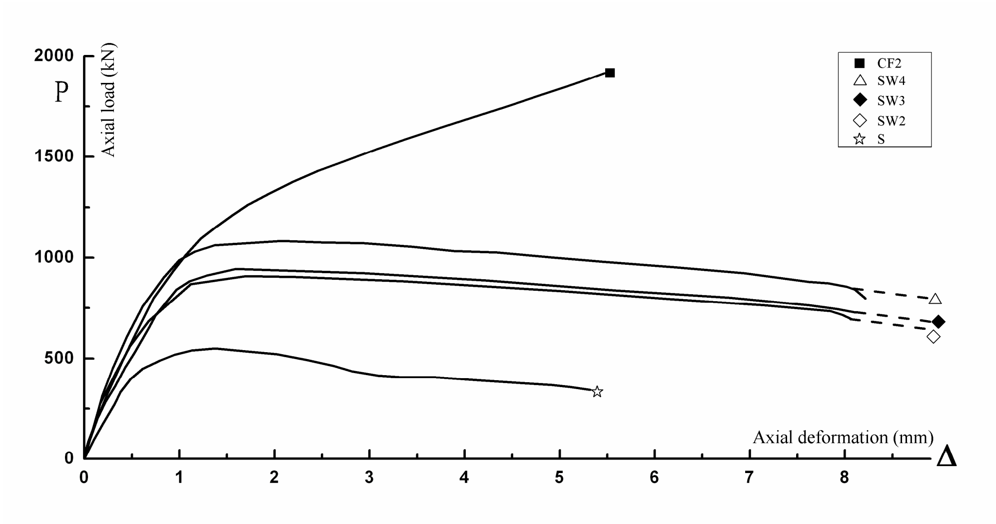
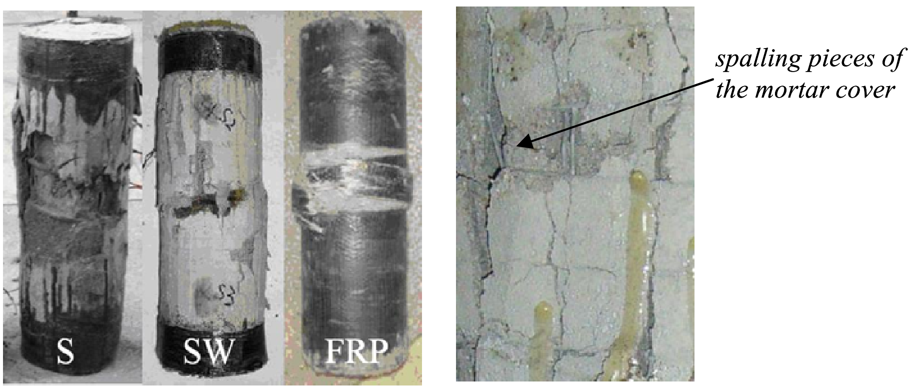


Figure 2: Relationships between load and longitudinal deformation (P-Δ) of typical specimens



a. Failure modes of BM, HWBM and FRP columns *b. Crack pattern and spalling of HWBM column*

Figure3: Typical failure modes of specimens

Ultimate Load-Bearing Capacity The tested ultimate load P_u^e and main displacement values based on the average of three tested specimens for each group are presented in Table 2. For a more rational discussion, the core concrete nominal strength $f_3^e=P_u^e/A_{core}$ (A_{core} is the area of the core concrete calculated according to the diameter of column before strengthening) was used to value the effects of different strengthening systems. It can be seen that the f_3^e values of BM and HWBM specimens were 80% and over 260% than that of control specimens P1. The f_3^e value of CF specimens was about 3.9 times as high as that of P2 specimens.

Table 2: Main experimental results

Specimen type	Specimen group [a]	Tested values Ultimate load P_u^e (kN) /Coefficient of variation	Nominal compressive strength of strengthened core concrete $f_3^{e\,b}$(MPa)	Axial displacement Δ_y /Δ_r [c] (mm)	Displacement ductility $\beta_\Delta=\Delta_r/\Delta_y$
P	P1	234.6 / 6.73%	27.0	-	-
	P2	489.3 / 3.43%	27.7	-	-
BM	BM	503.0 / 4.73%	57.6	2.6/5.3	3.4
HWBM	HWBM2	848.8 / 8.28%	97.0	6.9/8.0	7.0
	HWBM 3	880.2 / 7.31%	100.7	7.2/8.1	7.3
	HWBM 4	992.7/ 7.74%	113.6	7.6/8.2	7.8
CFRP	CFRP	1910.0 / 3.24%	108.1	-/-	-

[a] Three specimens for each group.
[b] $f_3^e=P_u^e/A_{core}$. Where A_{core} is the area of the core concrete calculated according to the diameter of column before strengthening (ref. Table 1).
[c] Δ_y and Δ_r are the axial displacements corresponding to yield, 0.85 ultimate load, respectively.

Ductility Displacement ductility $\beta_\Delta=\Delta_r/\Delta_y$ was used to evaluate the ductility of the HWBM and SW specimens (Table 2). Where, Δ_y and Δ_r are the axial displacements corresponding to yield and 0.85 ultimate load (on the descending branch), respectively (Fig, 2). It can be seen that the β_Δ values were 3.4 and larger than 7 for BM and HWBM specimens, respectively (Table 2). The crack spaces of HWBM were nearly equal to the mesh opening size (about 11 mm, see Figure 3), and were much narrower than those of BM (about 80-100 mm), mainly because the transverse wires form preferential locations for cracks. The developing process of the multiple cracks in the HWBM columns acted as an energy dissipation system, bringing the ductility to a level much higher than that obtained by the BM columns. Because CF was a brittle material with a small elongation of less than 1%, the CF strengthened columns developed a small ductility. It should also be noted that the ductility evaluating method mentioned above is not applicable for CFRP strengthened columns. Because CFRP is a brittle material, the elastic energy of CFRP will be accumulated even in the nonlinear stage (refer Figure2).The all accumulated elastic energy of CFRP should be deducted when evaluating the ductility of CFRP strengthened columns.

Conclusions

The steel bar-wire mesh-mortar can be efficiently used for enhancing both the ultimate load carrying capacity and the ductility of existed concrete columns. The axial displacement ductility ratio β_Δ of the steel bar-wire mesh-mortar strengthening specimens was about 7 in this study.

Strengthening concrete columns with carbon fiber reinforced polymer resulted in a great increase of the ultimate load carrying capacity, the ductility of these specimens, however, was obviously lower than that of steel bar-wire mesh-mortar strengthening specimens.

Acknowledgements

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References

[1] ACI Committee 440 (2002). "Guide for the Design and Construction of Externally Bonded FRP System for the Strengthening Concrete Structures." American Concrete Institute, Farmington Hills, Mich.

[2] ACI Committee 549 (1999). "Guide for the Design, Construction and Repair of Ferrocement." American Concrete Institute, Re-approved.

[3] Kondraivendhan, B, and Pradhan, B (2009). "Effect of ferrocement confinement on behavior of concrete."*Journal Construction and Building Materials*, 23, 1218-1222.

[4] Kubaisy, AI M, and Jumatt, M Z (2005). "Crack Control of Reinforced Concrete Members Using Ferrocement Tension Zone Layer." *Journal of Ferrocement*, 35, 490-499.

[5] Mirmiran, A, and Shahawy, M (1997). "Behavior of Concrete Columns Confined by Fiber Composites." *Journal of Structural Engineering*, 123, 583-590.

[6] Porter, M L, and Harries, K (2007). "Future Directions for Research in FRP Composites in Concrete Construction." *Journal of Composites for Construction*, ASCE, 11, 252-257.

[7] Shang, S P, Jiang, L M, Zhang, and M X (2005). "Experimental investigation into the strengthening of eccentric compression RC columns using composite mortar laminate reinforced with mesh reinforcements." *Journal of Building Structures*, 26, 742-749 (in Chinese) .

[8] Takiguchi, T (2001). "Shear Strengthening of Reinforced Concrete Columns Using Ferrocement Jackets." *ACI Structual Journal*, 98, 696-704.

[9] Waliuddin A, M, and Rafeeqi, S F A (1994). "Study of the behavior of plain concrete strengthened with ferrocement." *Journal of Ferrocement*, 24, 139-15.