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An investigation into the behavior and strength of reinforced concrete columns strengthened with ferrocement jackets

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Abstract

This study presents behavior and strength of reinforced concrete (RC) columns strengthened with ferrocement jackets. A total of six identical reference columns were prepared and tested after being strengthened with circular or square ferrocement jackets. Other than the ratio of axial load, parameters studied include the jacketing schemes, and the number of layers of wire mesh. Unless failure occurred at an earlier stage of loading, the columns were tested under cyclic lateral forces and constant axial load. Test results show that by providing external confinement over the entire length of the RC columns, the ductility is enhanced tremendously. Also, test results of this investigation revealed that the design method, proposed earlier by the authors, is very effective.

Keywords: Ductility; Ferrocement jacket; RC column; Shear strength; Strengthening

1. Introduction

Brittle shear failure in RC columns, especially constructed before 1980s, has been identified as one of the main causes of structures' failure observed in the recent major earthquakes [1–3]. Typical damage was attributed to the large spacing of tie in the columns, and the use of 90° hooks, even in conjunction with close tie spacing. Many other columns, although not damaged by the earthquakes and which have been properly designed and constructed in accordance with earlier building standards, had to be strengthened due to more stringent existing Building Code requirement. Such columns typically have problems such as insufficient ductility due to improper transverse confinement, and insufficient shear strength. Also, the hoops in existing columns were typically lap-spliced with a relatively small length without bending the tails of the hoops into the core concrete as required in the most modern seismic provisions. Such details cannot provide sufficient anchorage for hoops, when cover concrete spalls off. However, researches conducted in the past have shown that the compressive strength of core concrete, ultimate concrete compression strain and ductility of the strengthened column increased significantly if proper external confinement by mean of jacketing was provided [4–7]. Therefore, retrofitting techniques usually involve methods for increasing the confining forces either in the potential plastic hinge regions or over the entire column.

Recent research works by the authors [8–10] have shown that ferrocement jacket could be used as an alternative and effective technique to strengthen RC column with inadequate shear strength. The authors [9] investigated the use of circular ferrocement jackets for strengthening RC columns with inadequate shear strength. Only one parameter, the number of layers of wire mesh, was studied in this investigation. It was found that by providing circular ferrocement jacket that contained three layers of wire mesh, the brittle shear failure that occurs on the control specimen can be prevented, and the strengthened column shows extremely well in strength and ductility performance. It was also found that circular ferrocement jackets containing six layers of wire mesh can be used for repair damaged RC columns that have failed in shear [10].

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The availability of its materials in most developing countries, and no skilled labor required and it being suitable for both pre-fabrication and self-help construction could lead ferrocement to become one of the most inexpensive and attractive alternative techniques for strengthening and rehabilitation of existing and damage concrete columns. It should be noted, however, that ferrocement is not aimed as a replacement for other strengthening methods such as steel jacket or composite base materials jacket – sometimes it may coexist with others or be used in areas where steel or composite base material is expensive or not suitable.

By considering that the jacket acts as a series of independent spiral reinforcements of thickness $= t_{fj}$ at a spacing $= s$, a simple method to determine shear strength enhancement provided by a circular or square ferrocement jacket was proposed by the authors [9]. In this method, the additional strength imparted by the membrane shear flow in the jacket is conservatively ignored, and the contribution to shear strength from the ferrocement jacket is based on the analogy of circular or square columns having transverse reinforcement as hoops or spirals, and a 45° inclined crack is assumed. Then, the number of layers of wire mesh required to strengthen a column weak in shear is given as:

for circular jacket:

$$n = \frac{0.81 g_w V_j}{d_w^2 f_{yj} D'}, \tag{1}$$

for square jacket:

$$n = \frac{0.785 g_w V_j}{d_w^2 f_{yj} D'} \tag{2}$$

in which, g_w is the grid size of wire mesh, V_j the shear force to be carried by the jacket, d_w the diameter of wire mesh, f_{yj} the allowable stress of wire mesh for ferrocement jacket, and D' the core diameter of the jacket, from center to center of wire mesh reinforcement in the jacket. Details of design recommendation are available elsewhere [9]. Note that, to avoid premature yielding of the ferrocement jacket, which could lead to rapid loss of ductility, it is also recommended that the yield stress of wire mesh in the jacket, f_{yj}, for use in Eqs. (1) and (2) be limited to $\leqslant 0.4 f_y$, where f_y is the yield strength of wire mesh.

In the design method for strengthening suggested here, the nominal shear capacity of an RC column with inadequate shear strength is given by

$$V_{su} = V_c + V_s + V_j, \tag{3}$$

where V_{su} is the ultimate shear strength; V_c is the nominal shear strength provided by concrete; V_s is the nominal shear strength provided by transverse reinforcement, and V_j is the nominal shear strength provided by ferrocement jacket. Here, the reduction factor is not included. It is left to the design engineers to follow their national Building Code.

The present study, which focuses on improving the seismic performance of RC columns that are susceptible to shear type failure, is primarily aimed at complementing the investigation reported earlier [8–10] and at developing an alternative strengthening method by using ferrocement. Parameters studied include ratio of axial load, jacketing scheme, and number of layers of wire mesh employed in the jacket.

2. Experimental investigation

2.1. Test program and materials

A total of six identical reference RC columns, based on about 1:3–1:8 scale were constructed. Table 1 shows the details of the test program and materials properties. Three columns, designated as CJ-AL10-6L, CJ-AL15-6L, and CJ-AL20-6L were tested under different axial loads after being strengthened with circular ferrocement jackets containing six layers of wire mesh. Specimen CJ-AL15-6/3L, strengthened with reduced number of layers of wire mesh for the center portion, was tested to investigate the behavior and strength of the important practical aspect of strengthening RC column with ferrocement. Two reference columns, SJ-AL15-4L and SJ-AL15-6L, were strengthened with square ferrocement jackets, with four and six layers of wire mesh, respectively, before tested to their failure to study the effects of different shapes of jacketing on lateral load–displacement response.

Each of the reference columns was reinforced with 12 deformed D-6 bars distributed evenly around the perimeter of the column cross-section. Smooth R-2 (diameter = 2 mm) bars were used as transverse reinforcement spaced at 50 mm. Details of the reference and strengthened columns are shown in Fig. 1.

To ensure that the reference columns will fail in shear, the ratio of nominal shear strength to shear force required to develop the theoretical nominal flexural strength calculated according to AIJ 1994 [11] was designed to be less than 1.

The same woven wire mesh comes in 900 mm wide roll of 2.5 mm square opening and 0.45 mm wire diameter was used as reinforcement for jacket throughout the test. Tension tests were conducted on three representative mesh samples by following the procedure suggested by ACI Committee 549 [12]. The average yield strength based on 0.2% permanent strain was found to be 267 MPa.

Ordinary portland cement and natural sand passing through JIS sieve No. 2.5 (2.38 mm) were used in the ratio of 1:3.75 by weight for the concrete of the original columns. The water–cement ratio used was 0.65. To

Table 1
Test program and materials properties

Column specimen	Compressive strength (MPa)		Yield strength of reinforcement (MPa)		Axial load N (kN)	Wire mesh			Variable studied
	Mortar, f'_c	Slurry cement, f'_{cc}	Main, f_y	Transverse, f_y		Yield strain, ε_y (%)[a]	Yield strength, f_y (MPa)[a]	No. of layers, n	
CJ-AL10-6L	33.7	30.2	374	697	48	0.53	267	6	Ratio of axial load
CJ-AL15-6L	30.4	31.7			68				
CJ-AL20-6L	34.3	33.7			88				
CJ-AL15-6/3L	31.6	34.2			68			6 and 3	Jacketing scheme and number of layers of wire mesh
SJ-AL15-4L	33.4	30.1						4	
SJ-AL15-6L	32.9	30.2						6	

CJ, circular jacket; SJ, square jacket; AL, ratio of axial load; L, layer.
[a] 0.2% off set method.

improve workability, a superplasticizer was added at 0.05% by weight of cement. The target compressive strength of mortar was 30 MPa. The columns were cast in a horizontal position. A number of 100×200 mm^2 cylinders were cast for each batch of concrete to determine their compressive strength and the test result is summarized in Table 1.

2.2. Strengthening procedure

The required width and length of wire mesh was cut and properly wrapped around the entire column. At several places, the first and the second layers of the wire mesh were tied together with the same diameter of steel wire. An overlap of 100 mm was provided in the lateral direction for the wire mesh. A clear cover of 3 mm on the outer face of the jacket was provided by bonding 5 mm square and of 3 mm thick steel plates at several places.

Cement slurry mixed in the proportions in accordance with the manufactures recommendation was used as infill mortar and for ferrocement jacket. It was injected under pressure through a hole provided in the steel mold. A number of 50×100 mm^2 cylinders were cast to determine their compressive strength. Test result on the compressive strength of the slurry paste is shown in Table 1. Fifteen millimeter gaps were provided between ends of the jacket and the adjacent column stubs to prevent the strengthening jacket from carrying any direct axial stress.

2.3. Testing procedure and instrumentation

The test setup was designed to subject the test column to cyclic lateral forces, with the axial load remaining constant. The bottom stub of the columns was securely tied to the reaction floor beam of the frame using bolts, while the top stub was free to slide without inducing any rotation. The axial load was applied using the dead weight coming from a number of steel plates (see Fig. 2(a)). This loading system displaced the tested columns in a double bending, Fig. 2(b), a condition similar to that of an actual case in a moment-resisting frame. To ensure that the top and bottom stubs remain consistently parallel during testing, a parallel keeping system, consisting of eight units of hydraulic jack, was employed. A detailed explanation of the hydraulic jacks arrangement for parallel keeping is available elsewhere [13].

As shown in Fig. 2(c), three full cycles of lateral load were applied to the specimens before loading monotonically to failure. The displacements in both push and pull directions for the first, second, and third cycles were 20, 30, and 40 mm, respectively, followed by monotonic loading to failure in the push direction. The lateral load was applied by a 500 mm stroke, manually operated,

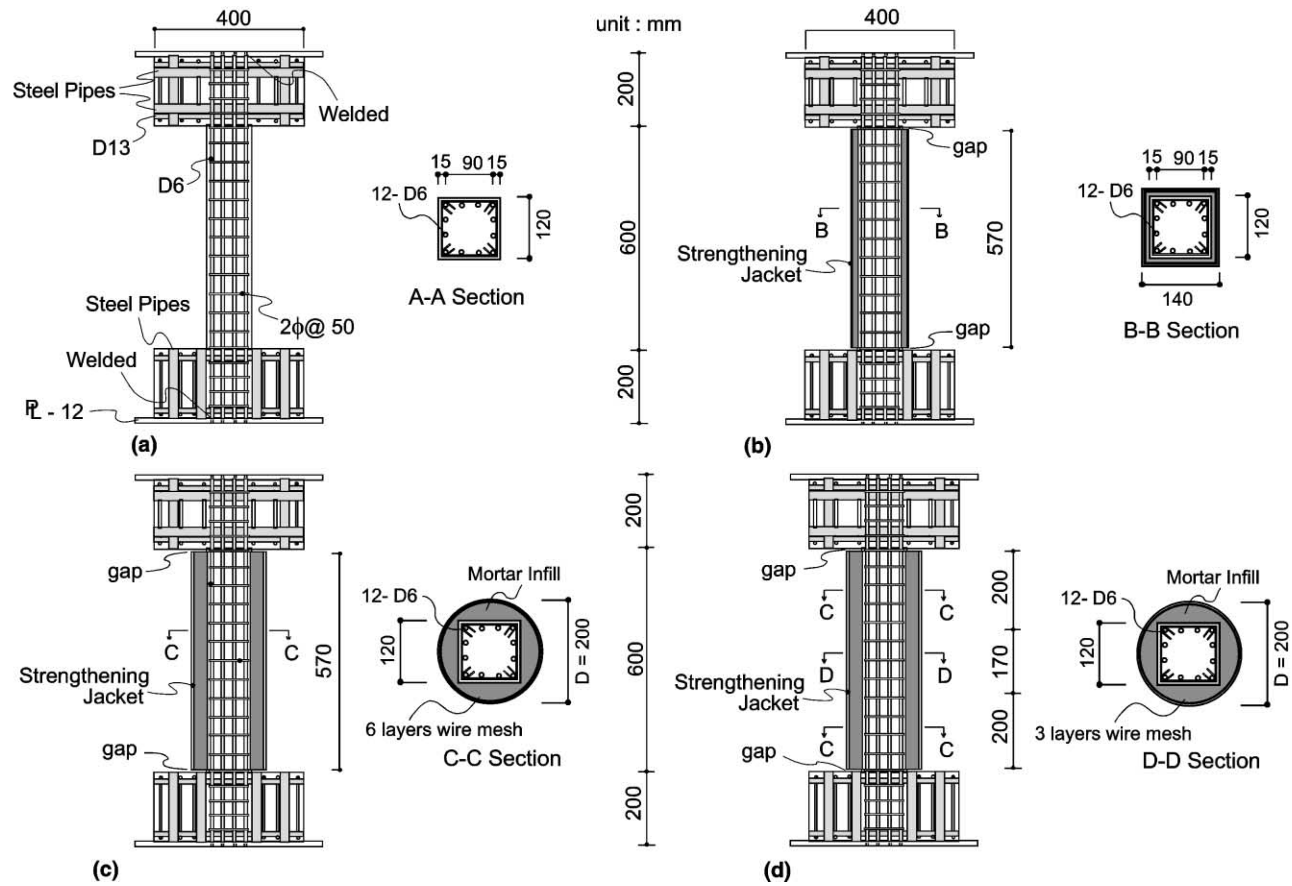

Fig. 1. Column details: (a) Reference column; (b) column with square jacket; (c) column with circular jacket and (d) column CJ-AL 15-6/3L.

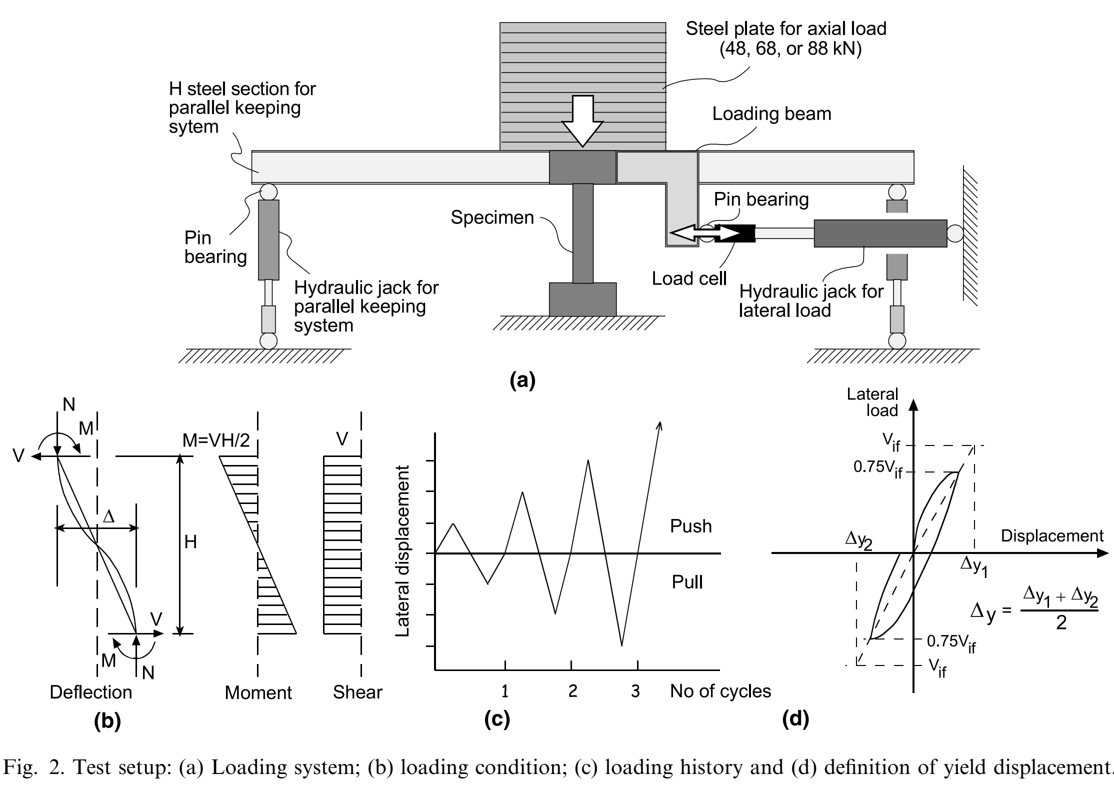

Fig. 2. Test setup: (a) Loading system; (b) loading condition; (c) loading history and (d) definition of yield displacement.

hydraulic jack with a capacity of 200 kN connected to the L-shaped loading arm at mid-height of the column.

Displacements of column specimens in the vertical, horizontal and diagonal directions were measured and recorded by a displacement measuring system consisting of three units of wire type transducers of 1000 mm measuring capacity, and three and two units of LVDTs of 100 and 25 mm strokes, respectively. This measurement system is able to take into account any rotation of the top stub that may occur during testing, which is necessary to determine the actual lateral displacement. Here, the yield displacement Δ_y for strengthened columns was found from the stiffness at a lateral displacement of $0.75V_{if}$, extrapolated linearly to V_{if}. This definition of the yield displacement Δ_y is illustrated in Fig. 2(d). Nominal displacement ductility, μ_n, is defined as Δ_n/Δ_y.

3. Test results and discussion

3.1. General observation

In general, first crack of the specimens starts at almost the same load of about 19 kN. It was observed that up to formation of the first crack, the lateral load–displacement response is linear (Fig. 3). Then, as lateral displacement was increased, additional flexural and shear cracks were formed at the gaps and within end parts of the jackets. However, these cracks did not have any significant effect on the load–displacement response of the columns until the completion of the third cycle of loading, corresponding to a drift ratio of 6.7%, except for the column CJ-AL20-6L. Beyond this lateral displacement, the slope of load–displacement response decreased gradually probably due to yielding of jacket reinforcement and penetration of diagonal cracks into concrete compression zone at both gaps. Note that, dashed lines in these figures indicate the ideal flexural capacity V_{if} of the reference column section calculated in accordance with additional theorem where the stress–strain of both steel reinforcement and concrete is assumed in rigid and perfectly plastic relation. In this theorem, the yield strength f_y of steel reinforcement and the cylinder compressive strength f'_c of concrete are used and the tensile strength f_t of concrete is assumed to be zero in determining the interaction diagram of the column section. Detailed explanations of this theorem are available elsewhere [14,15].

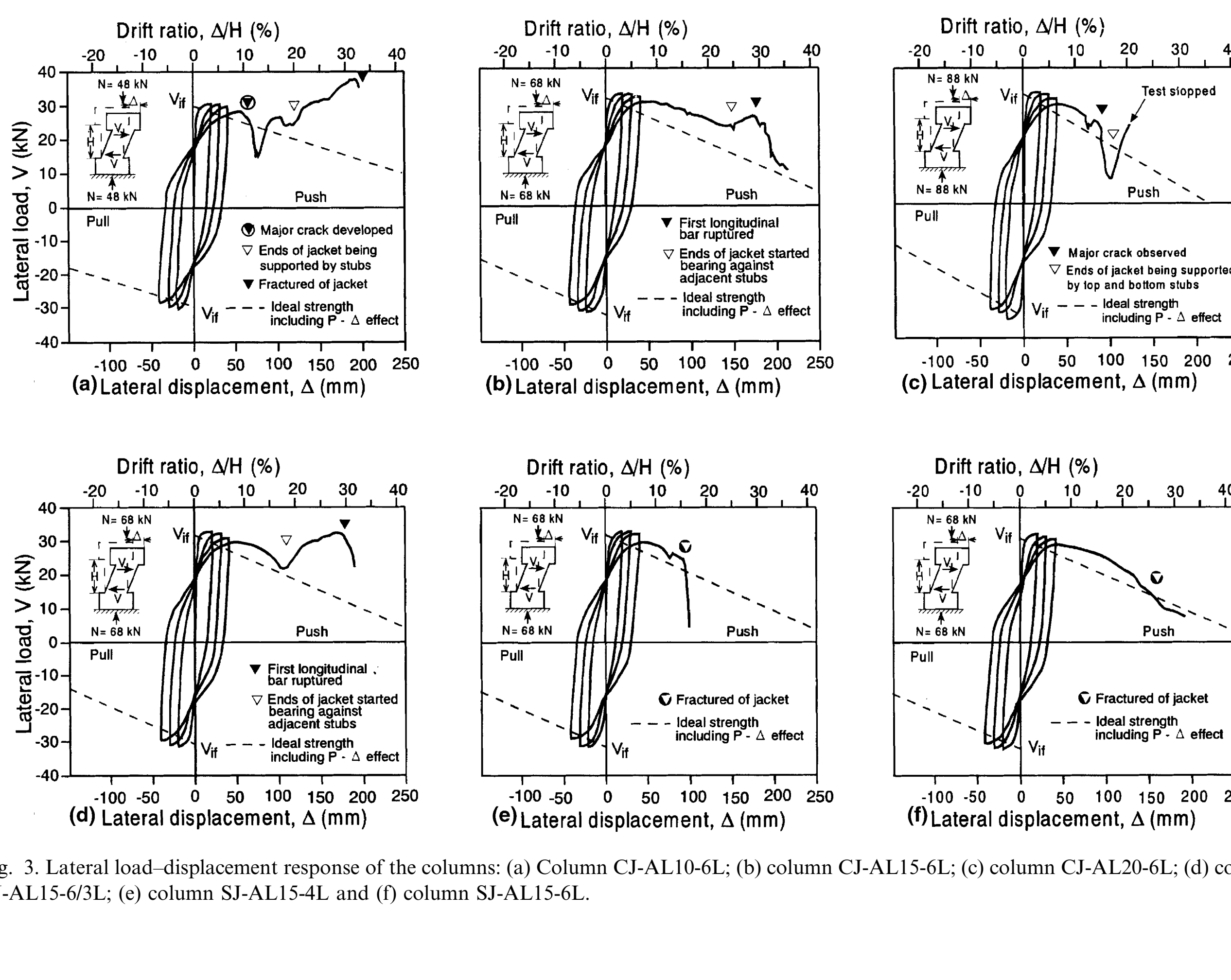

Fig. 3. Lateral load–displacement response of the columns: (a) Column CJ-AL10-6L; (b) column CJ-AL15-6L; (c) column CJ-AL20-6L; (d) column CJ-AL15-6/3L; (e) column SJ-AL15-4L and (f) column SJ-AL15-6L.

Regardless of the different properties of their jackets, it was found that the strengthened columns exhibited superb ductility performance. As can be observed from Fig. 3 that in general, up to a drift ratio of about 10%, the strengthened columns exhibited extremely stable lateral load–displacement response. Gradual lateral strength degradation of the columns was probably due to the P–Δ effect and crushing of concrete within the gaps. It was also found that as gaps of 15 mm were provided between strengthening jackets and the adjacent stubs, the strengthened columns showed no increase in flexural capacity. This may be preferable to avoid overloading the footing or the other adjacent structural elements in actual structures.

3.2. Effect of ratio of axial load

Lateral load–displacement response of the strengthened columns CJ-AL10-6L, CJ-AL15-6L, and CJ-AL20-6L is shown in Figs. 3(a)–(c), respectively. From these figures it can be seen that, column CJ-AL20-6L showed higher initial stiffness than the other two specimens. However, the difference was insignificant. Also, column CJ-AL20-6L with ratio of axial load of about 18% showed higher flexural capacity than columns tested under lower axial load. Except for column CJ-AL20-6L with flexural strength degraded gradually with increasing lateral displacement, basically, up to the completion of the three cyclic loading, the specimens tested under different ratios of axial load showed stable lateral load–displacement response. This stable and ductile response continued up to a drift ratio of about 10%. Up to this displacement, the damages to the columns CJ-AL10-6L, CJ-AL15-6L, and CJ-AL20-6L were concentrated mainly within the gaps, and only fine vertical cracks parallel to the column axis were observed on the surface of the jacket, predominantly within the upper and lower portions. It was observed that, as the lateral displacement was increased, the number and the width of vertical cracks increased. However, no significant strength degradation was observed.

Contrary to the columns CJ-AL20-6L, and CJ-AL15-6L, which maintained their strength degradation gradually, column CJ-AL10-6L lost its lateral strength rapidly at a drift ratio of about 10% due to widening of one of the vertical cracks at the upper end of its jacket formed during earlier stage of loading. At a drift ratio of about 15%, however, column CJ-AL10-6L regains its flexural strength and reached about 85% of its maximum strength achieved at earlier stage of loading before dropping again to about 75% of its maximum strength at a drift ratio of about 18%. Then, as lateral displacement was increased, the lateral load recovered (see Fig. 3(a)) and reached a new peak at a drift ratio of 32% due to the contribution of the jacket bearing against the top and bottom stubs.

At a drift ratio of about 29%, the longitudinal bars of strengthened columns CJ-AL15-6L ruptured in the gaps between the top and bottom stubs and the ends of jacket. Similar results were also observed for the two RC columns loaded to complete collapse, as reported by the second author elsewhere [16], where the fracture of the first longitudinal bar was noted at a column drift ratio of about 23%, and 28%, respectively. This finding indicated that if size of the gaps was designed properly, sufficient enough to prevent the strengthening jacket from bearing any direct axial stress, and depends on the design drift ratio, column height, and dimension of the jacket, the premature fracture of the longitudinal bar could be prevented. Note that, due to safety reasons, testing of column CJ-AL20-6L had to be stopped at a drift ratio of about 20%. Testing of column CJ-AL10-6L was stopped due to fracturing of its jacket at a drift ratio of about 33%. Contrary to the jacket of column CJ-AL15-6L, which was still in good condition after the conclusion of the test, jackets of columns CJ-AL10-6L and CJ-AL20-6L fractured during testing. Fig. 4 shows typical failure types of the column specimens after the tests.

3.3. Behavior of column strengthened with less layer of wire mesh at the center portion

The lateral load–displacement response of column CJ-AL15-6/3L, which was strengthened with six layers of wire mesh at plastic hinge regions, and three layers within center portion, is shown in Fig. 3(d). It can be seen from this figure that up to a drift ratio of about 10%, specimen CJ-AL15-6/3L exhibited extremely stable and ductile response. Up to this lateral displacement, the lateral load–displacement response of this column is comparable to that of column CJ-AL15-6L. A drop of about 10% of its flexural strength from the maximum capacity recorded during earlier stage of loading was probably due to the damages within its gaps.

As the lateral displacement was increased beyond a drift ratio of 10%, a major vertical crack, within tension zone of the top end of the jacket was observed. Then, at a drift ratio of about 15%, column CJ-AL15-6/3L lost its strength rapidly due to penetration and widening of this crack. At a drift ratio of about 18%, both ends of the jacket in compression zone touched the top and bottom stubs and the jacket began to bear axial stresses and the lateral strength recovered (see Fig. 3(d)) and reached almost the same maximum load of about 32.7 kN as measured in the first cycle of loading. This load recovery was due to contribution of the jacket bearing against the top and bottom stubs.

At a drift ratio of about 29%, the strengthening jacket of column CJ-AL15-6/3L suddenly fractured followed by a rapid strength loss. The fracture extended deep

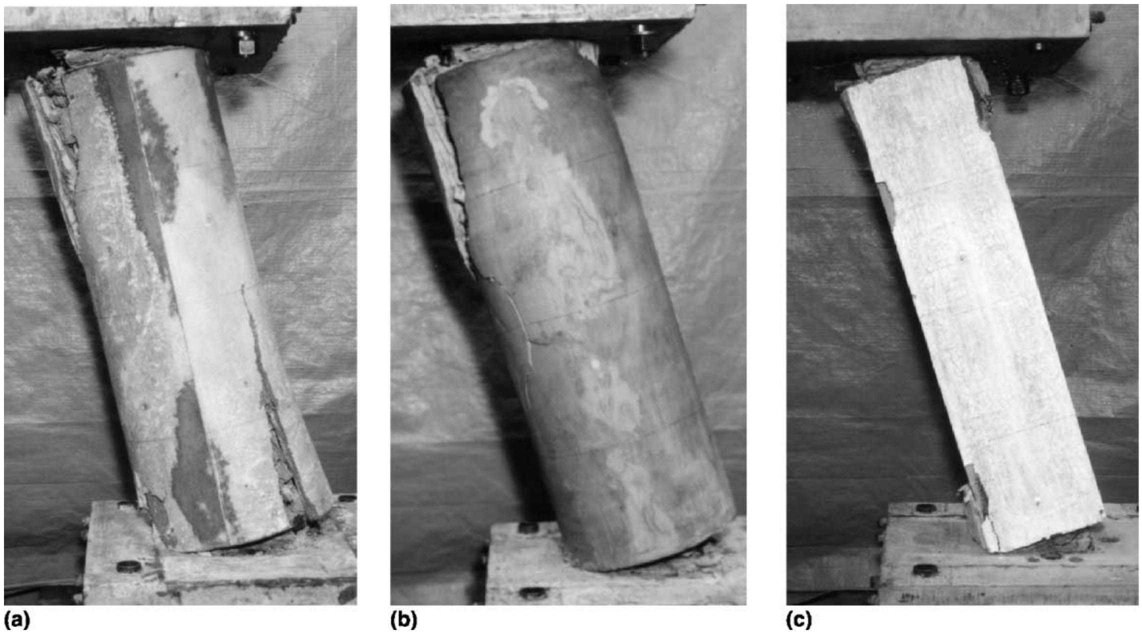

Fig. 4. Typical failure types of jackets after the tests. (a) column CJ-AL10-6L; (b) column CJ-AL15-6/3L; (c) column SJ-AL15-6L.

toward the center part of column height, see Fig. 4(b), where less layer of wire mesh (three layers) was employed. Note that, although the lateral displacement imposed to this column was more than that of column CJ-AL15-6L, no rupture of longitudinal reinforcement was observed. This could be due to yielding of the jacket and transverse reinforcement at earlier stage that resulted in local buckling of the longitudinal reinforcement in compression zone, hence, at the same time releasing the tensile strain of the reinforcement in tension zone. The result of this column indicates that lesser reinforcement of the jacket might be provided beyond the plastic hinge regions without affecting the ductility performance of the strengthened column significantly.

3.4. Response of columns strengthened with square jacket

Although tests on rectangular columns retrofitted with rectangular composite-materials jackets have been shown to provide a reasonable enhancement of ductility capacity, square or rectangular jacket is not recommended for retrofitting RC column for a number of reasons because of increased probability of longitudinal bar buckling [17]. For example, when axial load ratio is $\geqslant 15\%$. With ratio of axial load of about 15%, see Figs. 3(e) and (f), test results of this investigation revealed that with square ferrocement jackets, the strengthened columns SJ-AL15-4L and SJ-AL15-6L showed very high ductility and stable response. Up to the completion of the third cyclic loading, which is corresponding to a drift ratio of 6.67%, the lateral load–displacement response of these specimens was comparable to the columns strengthened with circular ferrocement jackets containing the same number of wire meshes [9]. However, column specimens strengthened with square jackets tested in this investigation showed early strength degradation, especially after large lateral displacements were applied.

From Figs. 3(e) and (f) it can be seen that, initially, both columns SJ-AL15-4L and SJ-AL15-6L showed similar response and reached almost the same maximum strength of about 32 kN (see also Table 3). It was observed during the last monotonic loading in the push direction that a major vertical crack formed on the top end of the jacket of column SJ-AL15-4L at a drift ratio of about 12%. Since then, this column experienced dramatic strength degradation at a drift ratio of about 16% due to yielding of the jacket and transverse reinforcement that resulted in widening of cracks formed at earlier stage of loading. At a drift of about 16%, the jacket of column SJ-AL15-4L fracture was followed by rapid loss of strength. Meanwhile, although its lateral strength kept on decreasing, as lateral displacement was increased, no significant physical damage could be found on the surface of the jacket of column SJ-AL15-6L until a drift ratio of about 20%. Up to this lateral displacement, the damage was concentrated mainly within the gaps, and within upper and lower parts of the jacket.

The jackets of columns SJ-AL15-4L and SJ-AL15-6L fractured within plastic hinge regions before testing of these specimens were terminated. Fig. 4(c) shows view of column SJ-AL15-6L after the test. Test results of this investigation revealed that with adequate wire mesh reinforcement employed in their jackets, the strength and ductility of columns weak in shear could be enhanced with square ferrocement jackets as well.

3.5. Envelopes of lateral load–displacement response

Lateral load–displacement response envelopes of the columns are shown in Fig. 5. Except for column CJ-AL20-6L, from this figure it can be concluded that up to a drift ratio of about 10%, the response of the strengthened columns was extremely excellent. Up to this displacement, the average of the strength degradation of the strengthened columns was about 10%. This figure also clearly shows that regardless of jackets' properties, initially the stiffness of the strengthened columns was identical.

The effect of the ratio of axial load on lateral load–displacement response of the strengthened columns can clearly be seen from Fig. 5(a). From this figure, it can be seen that the higher the ratio of axial load imposed on the column the faster the strength degradation. With the ratio of axial load of about 18%, which is the highest among three specimens tested in this group, the lateral strength of column CJ-AL20-6L degraded rapidly. On the other hand, the response of columns CJ-AL10-6L and CJ-AL15-6L was very stable. Within the scope of this investigation, the effect of axial load on shear strength is not immediately apparent. Therefore, further study is needed to investigate the effect of ratio of axial load on the nominal shear capacity of the column to be included in the formula given in Eq. (3).

The envelopes of the lateral load–displacement response shown in Fig. 5(b) clearly demonstrate that square ferrocement jacket with four layers of wire mesh and circular ferrocement jacket reinforced with a reduced number of layers of wire mesh can significantly improve the seismic performance of square R/C columns with inadequate shear strength.

3.6. Design examples

Table 2 shows design examples of a square column (reference column of specimen SJ-AL15-4L) strengthened with square ferrocement jacket. From this table it can be seen that by assuming $f_{yj} = 0.4f_y$, five layers of wire mesh are needed to reinforce the jacket. However, as discussed earlier, with four layers of wire mesh provided in its jacket, column SJ-AL15-4L showed superb seismic performance. Note that, ultimate shear strength V_{su} for calculating nominal shear strength provided by ferrocement jackets, V_j, is based on theoretical shear strength V_{if}. As can be seen from Fig. 3(e), up to a drift

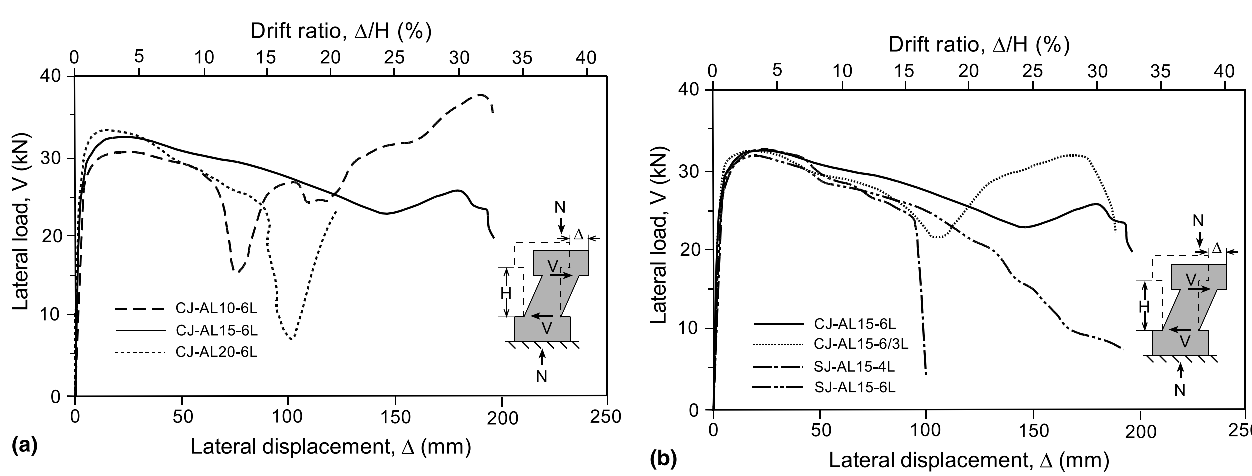

Fig. 5. Lateral load–displacement envelopes.

Table 2
Design example

Shape of jacket	Shear strength properties of reference column of specimen SJ-AL15-4L (kN)				D' (mm)	Wire mesh properties			Number of layers, n		
	V_{su}	V_c[a]	V_s[a]	V_j		g_w (mm)	d_w (mm)	f_y (MPa)	$f_{yj} = 1.0f_y$	$f_{yj} = 0.6f_y$	$f_{yj} = 0.4f_y$
Square	32.10	10.07	16.07	5.96	125	2.50	0.45	267	1.73 (two layers)	2.88 (three layers)	4.33 (five layers)

[a] Based on AIJ stuctural design guidelines.

Table 3
Summary of the test results

Column specimen	Calculated		Measured						V_{II}/V_{max}	V_{III}/V_{max}	Displacement ductility, μ [drift ratio, (%)]				Failure mode
	$0.75V_{if}$ (kN)	f_{yj}	V_{max} (kN)	V_{II}[a] (kN)	V_{III}[a] (kN)	Δ_y (mm)	Δ_{max}[b] (mm)	Δ_u[c] (mm)			at V_{max}	at V_{II}	at V_{III}	at V_u	
CJ-AL10-6L	22.22	$0.13f_y$	30.8	29.1	28.8	5.0	17.2	49.0	0.94	0.94	3.44 [2.87]	6.00 [5.00]	8.00 [6.67]	9.80 [8.17]	D
CJ-AL15-6L	23.55	$0.21f_y$	32.3	30.9	29.4	3.33	18.4	110.0	0.96	0.91	5.53 [3.07]	9.01 [5.00]	12.01 [6.67]	33.03 [18.33]	D
CJ-AL20-6L	25.53	$0.26f_y$	34.4	30.8	30.0	5.2	17.2	63.1	0.90	0.87	3.31 [2.87]	5.78 [5.00]	7.71 [6.67]	12.16 [10.52]	D
CJ-AL15-6/3L	23.69	0.21 and 0.41 fy	32.5	31.4	30.0	5.1	17.6	84.2	0.97	0.92	3.49 [2.93]	5.94 [5.00]	7.92 [6.67]	16.67 [14.03]	D
SJ-AL15-4L	23.99	$0.44f_y$	32.5	31.0	29.1	5.0	17.8	72.7	0.95	0.90	3.57 [2.97]	6.01 [5.00]	8.02 [6.67]	14.57 [12.12]	D
SJ-AL15-6L	23.91	$0.29f_y$	32.1	30.5	29.8	4.8	19.8	93.8	0.95	0.93	4.13 [3.30]	6.26 [5.00]	8.35 [6.67]	19.58 [15.63]	D

[a] Last data recorded at each cycle.
[b] At maximum lateral load.
[c] When the load carrying capacity dropped to 80% of its maximum strength; V_{max} = maximum lateral load; Δ = lateral displacement; D = Ductile flexural failure.

ratio of 15%, the lateral load–displacement response of column SJ-AL15-4L is comparable to the experimental result of the column strengthened with circular ferrocement jacket containing four layers of wire mesh [9]. This finding indicated that by limiting the allowable yield stress of the wire mesh used as reinforcement in the ferrocement jacket, a significantly ductile and stable performance of a strengthened column could be achieved even with square shape of jacket.

The lateral load–displacement response of the strengthened columns indicates that the proposed design method for calculating the number of layers needed to strengthen an RC column weak in shear for enhanced seismic performance gives good conservative result and is confirmed experimentally.

3.7. Overall comparison

Overall comparison of the tested columns is summarized in Table 3. It can be seen from this table that by adopting the yield strength of wire mesh for jacketing, f_{yj}, of about $0.4f_y$, displacement ductility μ achieved by strengthened column with square jacket (SJ-AL15-4L) after completion of the second cycle of lateral load was > 6 with flexural strength drop of only about 5%. To achieve much higher ductility, the allowable yield strength of wire mesh should be limited to $< 0.3f_y$. However, other than having extremely higher ductility the strengthened column will not suffer brittle failure after displacement ductility > 8, adopting $f_{yj} < 0.3f_y$ is probably not necessary because not only the usage of wire mesh is uneconomic but also penetration of infill mortar into layers of wire mesh becomes more difficult. Instead, by ensuring that the number of layers of wire mesh within the plastic hinge or end parts of the jacket is sufficient to prevent yielding to take place, and by providing sufficient reinforcement within center portion of the jacket, reasonable displacement ductility of the strengthened column can be achieved.

After careful examination of the specimens after testing, three types of failure modes of the strengthened columns have been identified, jacket fractured within plastic hinge region, jacket ruptured due to bearing axial loading after its ends come in contact with the top and bottom stubs, and failure due to fracture of longitudinal reinforcements within the gaps. Here, on the basis of the experimental results, some of which have been reported elsewhere [9], the columns were classified into three categories, as follows.

Ductile flexural. Column specimens that achieved nominal ductility $\mu \geqslant 6$ without any indication of shear failure.

Moderately ductile with shear failure. Column specimens that achieved ductility of $4 \leqslant \mu < 6$ before exhibiting shear failure.

Brittle shear failure. Columns that exhibited shear failure at $\mu < 4$.

It can be seen from Table 3 that all of the strengthened columns tested in this investigation failed in ductile flexural mode with nominal ductility > 6 and shear strength dropped only by about 10% of its maximum strength, and no indication of shear failure was observed.

4. Conclusions

This study summarizes the experimental research program on the use of circular and square ferrocement jackets for strengthening square reinforced concrete columns with inadequate shear resistance. Other than ratio of axial load, the effects of jacketing schemes were investigated in this study. Based on the results of six model columns prepared and tested in this investigation, the following conclusions can be drawn.

1. None of the strengthened columns developed additional flexural strength, which may be desirable to avoid overloading of the footings or beams.
2. Regardless of the ratio of axial load and strengthening scheme, all strengthened columns showed extremely stable and ductile response.
3. Within the scope of this investigation, the effect of ratio of axial load on lateral load–displacement response is inconclusive. The jackets of both strengthened columns with ratio of axial load of about 18% and about 10% fractured during testing.
4. Square ferrocement jacket can be used to strengthen RC column with inadequate shear strength to enhance its ductility.
5. Based on the test results of this investigation, less number of layers of wire mesh within the center portion of the circular ferrocement jacket could be adopted in strengthening shear failure type RC column.
6. Although it seemed that the jacket caused the formation of plastic hinge at the gaps, this investigation revealed that if the size is properly designed, premature rupture of the longitudinal bar could be avoided.
7. Shear strength enhancement provided by a circular and square ferrocement jacket can be conservatively estimated by considering the jacket acting as a series of independent spiral reinforcement with the amount of hoop reinforcement equal to the volume fraction of wire mesh. It is found necessary to limit the yield stress of the wire mesh in jacket, f_{yj} to $\leqslant 0.4 f_y$ to avoid premature yielding of the wire mesh which could lead to fracture of jacket within the plastic region.

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