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EXPERIMENTAL STUDY ON REINFORCED CONCRETE COLUMN STRENGTHENED WITH FERROCEMENT JACKET

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SUMMARY

Investigation by many researchers have indicated that by providing external confinement at plastic region or over the entire reinforced columns, the strength and ductility can be enhanced. In this paper, a strengthening method using circular ferrocement jacket to improve the confinement of a substandard column was investigated and compared with control specimens and different strengthening methods. Five 1:6 scale model square columns were constructed and have been tested under constant axial load while simultaneously being subjected to cyclic lateral load. The loading system used in this experiment displaced the tested columns in a double bending. Two columns were tested as control specimens, one column was strengthened with circular ferrocement jacket and were compared with those of other two identical square RC columns strengthened circularly with steel plate and carbon fiber. The control specimens suffered shear failure and significant degradation of strength during testing whereas the strengthened columns showed a ductile flexural response and higher strength. The test results indicate that circular ferrocement jacket can be an effective alternative material to strengthen reinforced concrete column with in adequate shear resistance.

INTRODUCTION

A number of column strengthening techniques, such as steel jacketing, use of composite materials jackets, and jacketing with additional reinforced concrete, have been investigated by Masukawa et al. [7], Priestley et al. [9], Rodriquez and Park [10] and Saadatmanesh et al. [11], and have been used in practice as reported by Yashinky [17]. Although strengthening by these material have been widely used in practice, investigation on possibility to employ other type of material, such as ferrocement, is necessary as an alternative method to improve the retrofitting process for the vast number of existing, structurally deficient RC column throughout the world.

Defined as a thin wall reinforced concrete and made of cement mortar and layers of fine wire mesh closely bound together to create a stiff structural form [1], ferrocement has a great potential to be used as a strengthening jacket material for substandard reinforced concrete columns. Several researchers, Andrew and Sharma [3], Basanbul et al. [5] and Lub and Wainroi [6], have investigated on ferrocement as a strengthening material for RC beams. Meanwhile, Arya [4] and Singh [12] have studied on ferrocement as a repair and strengthening materials for low rises housing. However, data on application and the behavior of ferrocement as a strengthening material for RC column are not available.

In this paper, a technique by using ferrocement jacket for seismic strengthening of reinforced concrete column was investigated and compared with different strengthening method. Three methods of strengthening were studied, including steel jacket, carbon fiber sheet, and ferrocement jacket. This research work is part of a research program aimed at developing methods for strengthening existing reinforced concrete columns by ferrocement jacket to enhance their seismic resistance.

EXPERIMENTAL STUDY

Test Program and Materials

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A total of five square RC columns based on 1:6 scale model to represent the as-built columns were constructed. Two columns named as CS-1 and CS-2, were tested as control specimens. Three columns, indicated as SCSP,

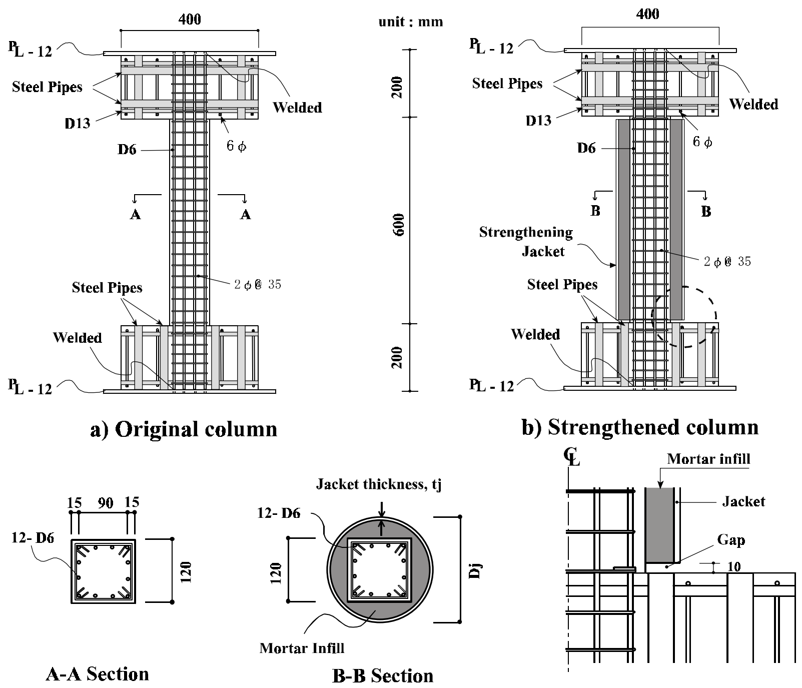


Figure 1: Column Details

SCCF, and SCFC, were strengthened by steel jacket, carbon fiber sheet, and ferrocement jacket, respectively.

The cross sections of the origin column were 120 mm square and had a height of 600 mm. As shown in Figure 1, the as-built columns were reinforced with 12 deformed D-6 (diameter = 6.35 mm) distributed evenly around the perimeter of the column cross section. Round R-2 (diameter = 2 mm) hoops spaced at 35 mm intervals were used as transverse reinforcement. The ratio of nominal shear strength of the as-built columns calculated based on AIJ code approach [2] to shear force required to develop the theoretical nominal flexural strength was designed to be unity. Details of the test columns are shown in Table 1.

Table 1: Details of Test Columns Design

	Specification	Specimen				
		CS-1	CS-2	SCSP	SCCF	SCFC
Column section	Column height, H	600 mm	600 mm	600 mm	600 mm	600 mm
	Shear span, L	300 mm	300 mm	300 mm	300 mm	300 mm
	Column depth, D	120 mm	120 mm	120 mm	120 mm	120 mm
	Column width, B	120 mm	120 mm	120 mm	120 mm	120 mm
	Concrete cover, cc	10 mm	10 mm	10 mm	10 mm	10 mm
	Concrete strength, f'_c	43.8 MPa	49.4 MPa	50.8 MPa	50.5 MPa	48.4 MPa
	Axial load, P	62 kN	62 kN	62 kN	62 kN	62 kN
	Ratio of axial load, $P/f'_c Ag$	9.80%	8.68%	8.40%	8.45%	8.87%
Longitudinal reinforcement	Bar diameter, d_b	6.35 mm	6.35 mm	6.35 mm	6.35 mm	6.35 mm
	Bar area, A_s	31.67 mm^2	31.67 mm^2	31.67 mm^2	31.67 mm^2	31.67 mm^2
	Yield strain, ε_y	0.19%	0.19%	0.19%	0.19%	0.19%
	Yield strength, f_y	418 MPa	418 MPa	418 MPa	418 MPa	418 MPa
Transverse reinforcement	Bar diameter, d_b	2 mm	2 mm	2 mm	2 mm	2 mm
	Bar area, A_{st}	3.14 mm^2	3.14 mm^2	3.14 mm^2	3.14 mm^2	3.14 mm^2
	Yield strain, ε_y^*	0.38%	0.38%	0.38%	0.38%	0.38%
	Yield strength, f_y^*	697 MPa	697 MPa	697 MPa	697 MPa	697 MPa
	Spasing, s	35 mm	35 mm	35 mm	35 mm	35 mm

* **0.2% permanent strain; Ag = gross sectional area**

Ordinary portland cement and natural sand passing through JIS sieve designed No. 2.5 (2.38 mm) were used in the ratio of 1: 2.5 by weight for concrete. The water-cement ratio used was 0.50. To improve workability, 0.05 % of cement weight of superplasticizer were added. The columns were cast in the horizontal position. To minimize differences in concrete compressive strength among specimens, columns CS-1 and CS-2, and origin columns of specimen SCSP, SCCF, and SCFC, were cast with the same batch of mortar on the same day.

A number of 100 x 200-mm cylinders were cast for each batch of concrete to determine their compressive strength. About 4 hr after casting, the specimens were covered with damp burlap to prevent moisture loss. The specimens were then stripped of the molds 7 days after casting and then air-cured in the laboratory before testing. Similar treatment was employed after infill mortar for strengthened specimens were cast through four steel pipes sleeve made in the bottom stub. The strength of materials used for the test columns is shown in Table 1.

Strengthening Procedure

Dimension of the strengthening jackets diameter were constructed so that distance between center of axis of the transverse tensile force developed in the jacket is equal, and the load resistance of jacket materials in the direction considered for every strengthened columns are the same. However, as shown in Table 2, due to the tensile strength of carbon fiber sheet is very high, it load resistance is much higher.

Table 2 - Designs and Properties of Strengthening Jackets

	Specification	Specimen		
		SCSP	SCCF	SCFC
Jacket section properties	Height of jacket, Hj	580 mm	580 mm	580 mm
	Jacket diameter, Dj	188 mm	188 mm	200 mm
	Jacket thickness, tj	0.8 mm	0.2 mm	15 mm
	Infill mortar strength, f'_c	42.5 MPa	40.4 MPa	46.3 MPa
	Ratio of axial load, P/f'_c Ag	4.8 %	4.9 %	4.2 %
Jacket material properties	Yield strain, ε_{jy}	0.23%*	-	0.53%*
	Yield strength, f_{jy}	228 MPa*	-	273 MPa*#
	Ultimate strength, f_{ju}	317 MPa	2740 MPa	363 MPa#
	Ultimate strain, ε_{ju}	2.17 %	1.16 %	1.50 %
	Load resistance	104 kN	311 kN	109 kN

* 0.2% permanent strain; # wire mesh; Ag = gross sectional area

Smaller maximum size of sand was used for the infill mortar so that it can penetrate easily in to layers of wire mesh. The same batch of mortar was used as the infill mortar for columns SCSP, SCCF, and SCFC and was cast in vertical position on the same day in an unloaded condition. Ten millimeter gaps were provided between end jackets and the adjacent column stub to avoid additional strength or stiffness from strengthening jacket. To investigate the properties of the strengthening materials, tensile test was conducted on three identical specimens for each material, and the results were summarized in Table 2.

Ferrocement jacket

Woven wire mesh, comes in 900 mm wide roll of 2.5-mm square opening and 0.45-mm wire diameter was used. The required width of 580 mm and length for 11-layers of wire mesh was cut and properly wrapped by two people around the entire column. One person held the first end of the wire mesh in position while the second person wrapped the rest of it around the column. At several places, the first and the second layer of the wire mesh were tie together with the same diameter of steel wire. Similarly, this process was repeated when the third layer, and the fourth layer has been wrapped around the column. One hundred mm overlapping of wire mesh was provided in lateral direction. A clear cover of 3-mm on outer face of jacket was provided by bonding 5 x 5 mm square of 3-mm thick steel plates at several places. It needs about 3 hr starting from cutting the wire mesh from its roll until the steel mold was ready to be set.

Infill mortar was made with water-cement ratio = 0.55 and cement sand ratio of 1:2.5. Natural sand passing through JIS sieve designed No. 1.2 (1.19 mm) was used. In order to improve workability, 0.05 % of cement weight of superplasticizer were added.

The infill mortar and mortar for ferrocement jacket were cast at the same time. Even though special care was applied when fresh infill mortar with slump of 180 mm was cast, and properly vibrated by two units of hand vibrators, some defects were observed on the surface of concrete. It was observed that only about 150 mm of jacket height was penetrated properly by mortar. Meanwhile within almost 90 % of the rest part of jacket, mortar penetrated up to the outer layer of wire mesh, and a number of layers of wire mesh was not fully covered by mortar on the rest part of jacket. This 10 % part was concentrated mostly at the corner of the square section of original column. Therefore, repair work by epoxy resin was executed to fill-up the 3-mm cover and part of the jacket that did not filled-out by mortar.

Steel plate jacket

Column SCSP was strengthened with steel jacket fabricated from 0.8 mm thick steel plate. Two half shells of steel plate are positioned over the area to be strengthened and are connected using 5 mm high tensile strength bolt up the vertical seem to provide a continuous tube around the column. Bolt were arranged in double shear at spacing of 15 mm. Gap between the steel jacket and the concrete column was filled with mortar.

Carbon fiber sheet

Prior to the application of the epoxy coat to the bare column of specimen SCCF, the concrete surface was cleaned from dust. The carbon fiber sheet which is available in 300 mm wide roll was then wrapped directly on the fresh epoxy coat. One hundred mm overlapping was provided in lateral direction and no overlapping for vertical direction. Any air-trapped under neat the carbon fiber sheet was forced out by hand operated pressure roller. About 30 minutes later, the second epoxy coating was applied on the surface of carbon fiber sheet. These works also carried out by two people.

Testing Procedure and Instrumentation

All specimens were tested under cyclic lateral load while simultaneously being subjected to constant axial load of 62 kN. Unless failure occurred at an earlier stage, two full cycle lateral load followed by monotonically lateral load in push direction until the column specimens could not maintain the axial load were applied to every specimen. The applied cyclic loading was displacement-controlled.

The top and bottom stubs of the column specimen were post tensioned to 13 pieces of steel plates acting as a constant axial load and the reaction floor beam, respectively. This loading system, as shown in Figure 2, displaced the tested columns in a double bending condition similar to the actual case in a moment-resisting frame. To assure that the top and bottom stubs were consistently parallel during testing, a parallel keeping systems which consist of 8 units identical oil jacks with 300 mm stroke were used and connected with high pressure flexible hoses. Configuration of oil jacks of the loading arrangement I is shown in Figure 3. For loading arrangement I, the parallelism of the top and bottom stubs of columns while in testing was relied upon self adjustment of the oil jacks. Mean while, in loading arrangement II, a pump was used to adjust any rotation of the top stub during testing. Specimens CS-1, CS-2, and SCSP were tested using loading arrangement I whereas columns SCCF and SCFC were tested using loading arrangement II. Detail explanation of the loading system is

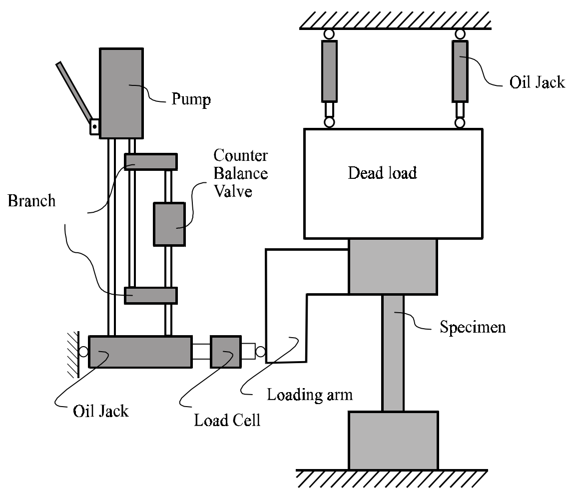


Fig. 2 Loading System

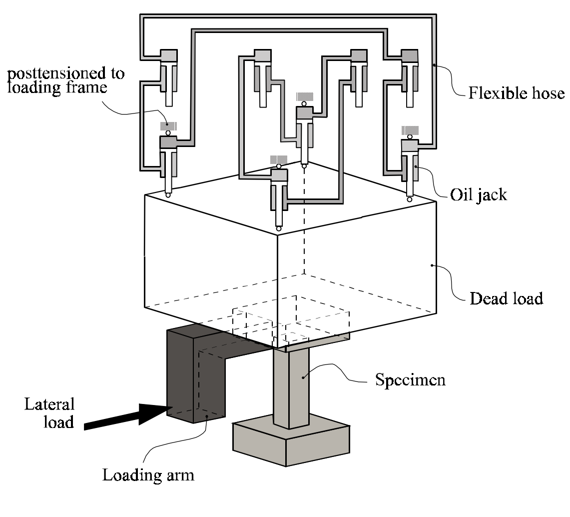


Fig. 3 Oil Jack Arrangement I

given in Takiguchi [14].

The lateral load was applied by a 500 mm stroke hydraulic jack with capacity of 200 kN connected to the L-shape loading arm at the mid height of the column. The applied lateral load was monitored and recorded using a calibrated load cell. The ratio of applied axial load to the column axial load capacity varies from 8.5 % to 9.8 %

for origin columns. This ratio was 4.8 %, 4.9 %, and 4.2 % (see Table 2) for column SCSP, SCCF, and column SCFC, respectively.

Displacements of column in vertical, horizontal and diagonal direction were measured and recorded by a displacement measuring system. This system consist of 3 units wire transducers of 1000 mm measuring capacity, and 6 units and 2 units LVDTs of 100 mm 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 when the real lateral displacement is determined.

TEST RESULTS AND DISCUSSION

As shown in Figure 4a, the parallel keeping system of loading arrangement II used to load column SCFC was able to control the top stub consistently parallel to the bottom stub even after an extremely large lateral

displacement was applied.

a) Column SCFC under testing b) Column CS-2 c) Column SCFC

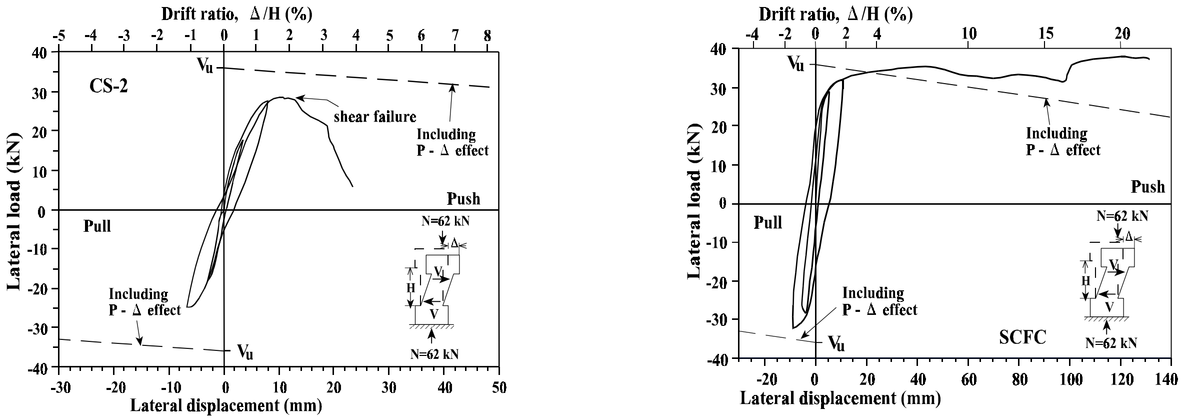
Figure 4: Specimen Under Testing and Damage of Strengthened Specimens After Testing

Both control specimens develop similar cracks pattern and achieved almost similar shear strength. Flexural cracks perpendicular to the column axis developed first in the regions close to the top and bottom end of the columns. As the displacement increased, incline cracks were started to develop. Figure 4b exhibits shear failure of column CS-1 with inclined cracking, spalling of cover concrete, and rupture of two out of four exposed transverse reinforcements. For SCSP, SCCF, and SCFC columns, first cracks were observed at the boundary of the top and bottom stubs and column. At later stages of loading, cracks also observed within 10-mm gaps. No crack was observed within strengthening jacket of SCFC specimen.

The control specimens developed shear failure at drift ratio less than 2.5 % and failed by disintegration of core concrete resulting from lack of concrete confinement and yielding of transverse reinforcement. On the other hand, strengthened specimen exhibited ductile flexural deformation. However, due to safety reason, testing of both specimens SCSP and ASCF has to be stopped even before their shear strength did not drop to 85 % of maximum shear strength achieved yet.

The damages to the strengthened specimens were concentrated at the boundary of the column sections and the top and bottom stubs, and within the gaps as shown in Figure 4c. The employment of circular jackets prevented the strengthened columns SCSP, SCCF, and SCFC from brittle shear failure as happened to the control specimens. Although, in case of column SCFC, very large lateral displacement was applied and the concrete within the gaps was fracture, no physical damage was observed on strengthening jackets throughout the test. It is also observed that the strengthening part of column SCSP and SCCF was also still in good condition after the completion of the test.

Figure 5 show examples of the experimental lateral load-displacement relationship of the tested column specimens. Also shown in these figures is the theoretical lateral load Vu, calculated based on additional theorem. This theoretical load is plotted as dashed lines that reduce with increase in displacement due to the P-Δ effect. Details explanation of this theorem are available in Takiguchi et al. [13, 15]. Both control specimens CS-1 and CS-2 could not develop their flexural strength. The maximum measured strength were about 83 % of the theoretical strength calculated including the P-Δ effect. The CS-1 specimen exhibiting brittle shear failure before two full cycle lateral load was completed at drift ratio about 1.7 %. Similarly, as shown in Figure 5a, even though a limited ductile response was achieved, CS-2 column failed in sudden shear failure at drift ratio about



a) column CS-2

b) column SCFC

Figure 5: Typical Lateral Load-Displacement Relationship of Specimens

2.3 % followed by rapid degradation of lateral resistance.

All strengthened columns exhibiting higher flexural capacity than control specimens. Compared with the control specimens, which had nearly identical shear strength, the maximum shear strength of strengthened columns was enhanced by 6 %, 12 %, and 28 % for column SCSP, SCCF, and column SCFC, respectively. Results of this experiment shows that, compared with column SCSP and column SCCF, the maximum shear strength of columns SCFC was higher about 20 % and 14 %, respectively. The comparison also indicates that the maximum lateral displacement of strengthened specimens compare to the average maximum lateral displacement of both

Table 3: Summary of Test Results

Column	V_u [a] (kN)	$V_{u-\Delta_{max}}$ [b] (kN)	K_i [c] (kN/mm)	V_{exp} [d] (kN)	Δ_{max} [e] (mm)	$\frac{V_{exp}}{V_u}$	$\frac{V_{exp}}{V_{u-\Delta_{max}}}$	θ [f]	Drift ratio [g] (%)	ED2 [h] (%)
CS - 1	35.4	34.5	4.88	28.9	8.3	0.82	0.84	32	1.4	-
CS - 2	35.9	34.7	4.26	28.3	11.5	0.79	0.82	34	1.9	47
SCSP	35.9	34.7	7.41	30.4	17.0	0.85	0.88	-	2.8	60
SCCF	36.0	33.3	7.14	32.1	25.8	0.89	0.96	-	4.3	58
SCFC	35.8	31.3	6.67	36.5	42.7	1.02	1.17	-	7.1	78

[a] Shear at theoretical flexural strength.
[b] Theoretical strength including P-Δ effect.
[c] Initial stiffness at first cycle when lateral load is 20 kN.
[d] Maximum measured shear strength.
[e] Lateral displacement at maximum shear force.
[f] Measured angle of cracking to column axis at maximum lateral load.
[g] At maximum shear force.
[h] Ratio of energy dissipation to elastic perfectly plastic idealization loop of second

control specimens was almost fourfold for column SCFC. Detail of the test results are summarized in Table 3.

As it can be seen from Figure 5b, the SCFC specimen exhibited extremely stable and ductile response. After the maximum strength was achieved, the lateral load resistance was gradually decreased. At drift ratio about 15.5 %,

the lateral load increased and reached a new highest lateral load. This enhancement was due to additional strength from longitudinal jacket that was being supported on the top and bottom stubs.

It was indicated from SCFC column that concrete within the gaps was fracture and this fracture extended about 10 mm into infill mortar within strengthening jacket and adjacent stub as well. Nevertheless, no single longitudinal bar was fracture in SCFC column despite of large lateral displacement with drift ratio of more than 22 % was imposed. This result shows similar behavior with two RC columns loaded to their complete collapse as reported by the first author elsewhere [16] where the first longitudinal bar fracture was observed when the columns drift ratio was about 23 % and 28 %.

A direct comparison of the lateral load-displacement peak envelopes of the tested columns is provided in Figure 6. From this figure it can be seen that both control specimen exhibited very poor behavior. However ductility, strength, stiffness, and energy dissipation were improved significantly when a shear failure type of the origin column were strengthened with circular jackets. Although the gaps of 10 mm were provided between strengthening jacket and the adjacent stubs to prevent an increase in flexural capacity, the strengthened columns showed significant increase in flexural capacity. One of the reasons of flexural capacity enhancement of strengthened columns is probably due to increase in core concrete compressive strength due to confinement provided by steel jacket, carbon fiber sheet, and ferrocement jacket.

As summarized in Table 3, compared to average initial stiffness of both control specimens, the initial stiffness of

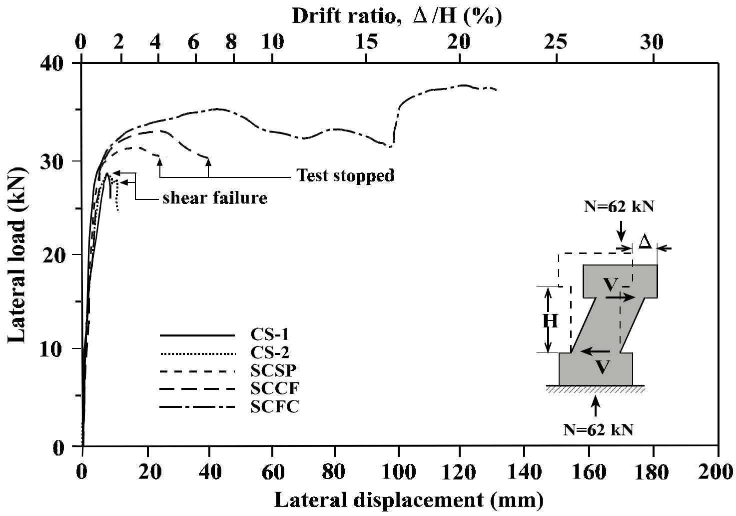


Figure 6: Lateral Load-Displacement Envelopes

columns SCFC was improved about 45 %. Although the enhancement of stiffness and strength was not always desirable since the strengthened column would attract more earthquake force, nevertheless as pointed out by Priestley et al. [9], the non-strengthened columns would experienced less shear force if selective strengthening columns is adopted in the overall strengthening design. Note that the enhancement in flexural capacity and the consequent increase in shear demand were not critical since strengthened jacket provided higher shear strength.

For comparison purpose, energy dissipation of second cycle of each specimen was calculated by taking the area under second loop of the lateral load-displacement curves. The ratio of energy dissipation to area enclosed by the elastic perfectly plastic idealization of the second lateral load displacement loop, ED2, of the strengthened columns are higher than specimen CS-2 (see Table 3). This indicate that the strengthened columns show better energy dissipation. Note that, the initial elastic slope of the elastic perfectly plastic idealization was calculated based on theoretical moment-curvature relationship of the section [8], and it was assumed that the unloading path follows the initial elastic slope.

CONCLUSIONS

Five columns have been prepared and tested in the investigation reported in this paper. Based on the work performed starting from preparation of the strengthening material until the completion of the test of specimens the following conclusions can be drawn.

1] Test results show that the two origin column specimens suffered shear failure and significant degradation of strength at a relatively low lateral displacement. Both columns were unable to develop their flexural strength.

2] By providing external circular confinement using carbon fiber sheet and ferrocement jacket to the origin columns, the stiffness, strength, energy dissipation, and ductility are improved significantly and the mode of failure changed from brittle shear failure to ductile flexural failure.

3] Although it seemed that the effect of the jacket is to concentrate plasticity at the gaps, as indicated from this research if they were designed properly the earlier fracture of the longitudinal bar can be prevented.

4] The results of this investigation indicated that strengthening of a square reinforced concrete column with circular ferrocement jacket was considered to successful.

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