

Retrofit of RC square short columns

K. Galal^a, A. Arafa^b, A. Ghobarah^{b,*}

^aDepartment of Building, Civil & Environmental Engineering, Concordia University, Montreal, Canada H3G 1M8

^bDepartment of Civil Engineering, McMaster University, Hamilton, Canada L8S 4L7

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Abstract

There exist many reinforced concrete short columns that are vulnerable to brittle shear failure as repeatedly demonstrated during recent severe seismic events. Due to the large scale of the problem, it is important to investigate the behaviour of short columns and approaches to their retrofit. The objective of this research program is to evaluate the performance enhancement of short reinforced concrete columns with high and low transverse steel content when retrofitted using fibre-reinforced composites. Seven reinforced concrete short columns were designed and tested under lateral cyclic loading and constant axial load. Carbon or glass fibre-reinforced polymers were used to strengthen the short columns.

It is found that short columns, even ones designed according to current codes, suffered brittle shear failures. It is demonstrated experimentally that it is possible to strengthen the shear resistance of short columns such that a flexural ductile failure occurs by developing plastic hinges at both ends of the column. Anchoring of the fibre wraps to the columns was found to be effective in increasing the shear resistance and energy dissipation capacities of the columns.

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1. Introduction

One of the observed common reinforced concrete (RC) structural failures during recent earthquakes is column shear failure. The types of columns that are susceptible to shear failure are short columns with low shear span/depth ratio. In seismically active zones, it has been the practice to avoid the construction of short columns. However, there exist a large number of columns that are susceptible to brittle failure. These columns may have been originally designed as long columns and then partial supporting walls were later constructed, which created a short column. Short columns may also have been the result of recent design following current codes. Columns built prior to 1970 were designed on the basis of strength. However, often before the flexural strength of the short column was reached, abrupt non-ductile failure

occurs. Brittle shear failure reduces the energy dissipating capacity of the column. It is important to investigate practical and effective methods to rehabilitate these columns.

The use of fibre-reinforced composites in a wide variety of industrial applications has rapidly increased during the last decade. Advanced composite materials are being used to strengthen and enhance the performance of various structural elements, particularly RC columns with circular and elliptical cross sections.

The seismic rehabilitation of square or rectangular short columns using fibre-reinforced polymer (FRP) materials has not been extensively studied. Yoshimura et al. [1] tested eight 150 × 150 × 1000 mm RC columns strengthened externally using carbon fibre-reinforced polymer (CFRP) under constant gravity load and repeated lateral force. The specimens had no transverse ties and the shear span/depth ratio was 1.0. The small size and the non-standard design of the specimens make it difficult to arrive at reliable results.

Haroun et al. [2] tested seven rectangular RC columns with transverse steel reinforcement ratio $\rho_t = 0.1\%$ under

* Corresponding author. Tel.: +1 905 525 9140x24913; fax: +1 905 529 9688.

E-mail address: ghobara@mcmaster.ca (A. Ghobarah).

reversed cyclic load. One of the seven specimens was the control test, while the others were repaired using crack injection and addition of carbon fiber jacket, then retested. The shear span/depth ratio of the specimens was 2.0. The repaired specimens performed in a more ductile response compared to the as built one.

Ye et al. [3] tested seven square 200×200 mm RC short columns with shear span/depth ratios ranging from 1.5 to 3.0 and strengthened with CFRP under lateral cyclic loading. The transverse steel reinforcement ratio of the specimens, ρ_t was 0.18%. Two of the specimens were fully wrapped with continuous CFRP sheets along the column height, while four were wrapped with discontinuous CFRP wraps with different widths and spacing. The strengthened specimens had a more ductile behaviour compared to the unstrengthened ones.

The objective of this study is to evaluate the performance enhancement of short reinforced concrete columns with high and low transverse reinforcement content when strengthened using glass or carbon composite materials. The experimental program was designed to evaluate the viability of anchoring techniques [4] used to improve the confinement of square columns.

2. Test program

In an earlier study at McMaster University, three RC short columns (SC1, SC2, and SC3) were constructed and tested when subjected to cyclic lateral loading and constant axial load [5]. The cyclic load represented lateral seismic forces while the vertical load represented gravity load on the column. One unstrengthened column was designated as control test, while the other two were rehabilitated using three layers of CFRP. The unstrengthened column failed in shear while the rehabilitated ones exhibited ductile behaviour. An analytical model to predict the confining effect and the total shear resistance of rectangular RC columns wrapped with composite sheets with and without anchors was proposed. In the current study, the behaviour of four additional square RC short columns strengthened using fibre composites is described. The proposed rehabilitation schemes were initially designed to enhance the shear resistance of the short columns and prevent their brittle shear failure. It is desired that the failure of the rehabilitated specimens occur through the formation of ductile plastic hinges at the top and bottom ends of the column. The rehabilitation patterns and materials were selected to evaluate the effects of FRP anchoring, transverse steel content, number of FRP layers and FRP material on the seismic performance of RC square short columns.

The columns were tested in the fixed–fixed end configuration and were subjected to cyclic displacements applied to the top end while the bottom end was fixed. The horizontal actuator used to apply the horizontal displacements and shear forces was centered at the mid-height level of the tested columns. The load was transmitted

to the top of the specimen using a stiff L-shaped fixture. The column height/depth ratio was 3.0. The tested columns were subjected to double curvature with the point of contraflexure being maintained at mid-height. The shear span to depth ratio (a/d) for the tested columns was 1.5. The top and bottom ends of the column were fixed to the test set-up using a rigid reinforced concrete block.

2.1. Specimen details

The seven specimens had the same column overall dimensions. The dimensions of the column represented a 2/3-scale model of a prototype column. The specimens were divided into two groups. Group 1 was designed according to the current Canadian Reinforced Concrete Design Code [6], while Group 2 was designed according to pre-1970 code [7]. Group 1 includes specimens SC1, SC2, SC1R, SC2R and SC1U. Group 2 includes specimens SC3 and SC3R. The column cross-section, elevation and reinforcement details for the seven specimens are shown in Fig. 1. The longitudinal reinforcement ratio for all the columns is $\rho_g = 2.6\%$. Group 1 columns had #10 ties spaced at 65 mm. Group 2 columns had #10 ties spaced at 305 mm.

In Group 1, specimen SC1 was unstrengthened and was tested as the control specimen. Specimens SC2, SC1R, SC2R, and SC1U were strengthened using different systems before testing. A schematic representation of the strengthening schemes is shown in Fig. 1. The column SC2 was strengthened using three layers of CFRP. The fibre sheets were expected to provide confinement to the column. An attempt was made to reduce the potential bulging of the square sides of the column by using four clamping plates with rods through the column section. The rods were 12 mm diameter threaded rods and the clamping plates were 6 mm thick. The strengthening of specimen SC1R included using four layers of unidirectional glass fibre-reinforced polymer (GFRP) at the plastic hinge regions at the top and bottom ends of the column. The height of the hinge region of 150 mm was taken as half the section depth. Between the column end hinges two layers of bi-directional GFRP sheets were applied. The four layers of unidirectional GFRP and the two layers of bi-directional GFRP overlapped by 50 mm. Mechanical anchors similar to those used for specimen SC2 were used for SC1R. The amount of the GFRP at the plastic hinge location of SC1R was chosen such that it provides shear strength close to that provided by the CFRP of specimen SC2. This will enable the evaluation of the effect of FRP material on the response of RC short columns.

Specimen SC2R was retrofitted using two layers of unidirectional CFRP at the plastic hinge zones of 150 mm and two layers of bi-directional CFRP in the intermediate zone. No anchors of the FRP sheets into the concrete were provided. The amount of CFRP at the plastic hinge location of SC2R was chosen to be less than that of specimen SC1U in order to evaluate the effect of number of FRP layers on the behaviour of columns. In order to evaluate the effect of

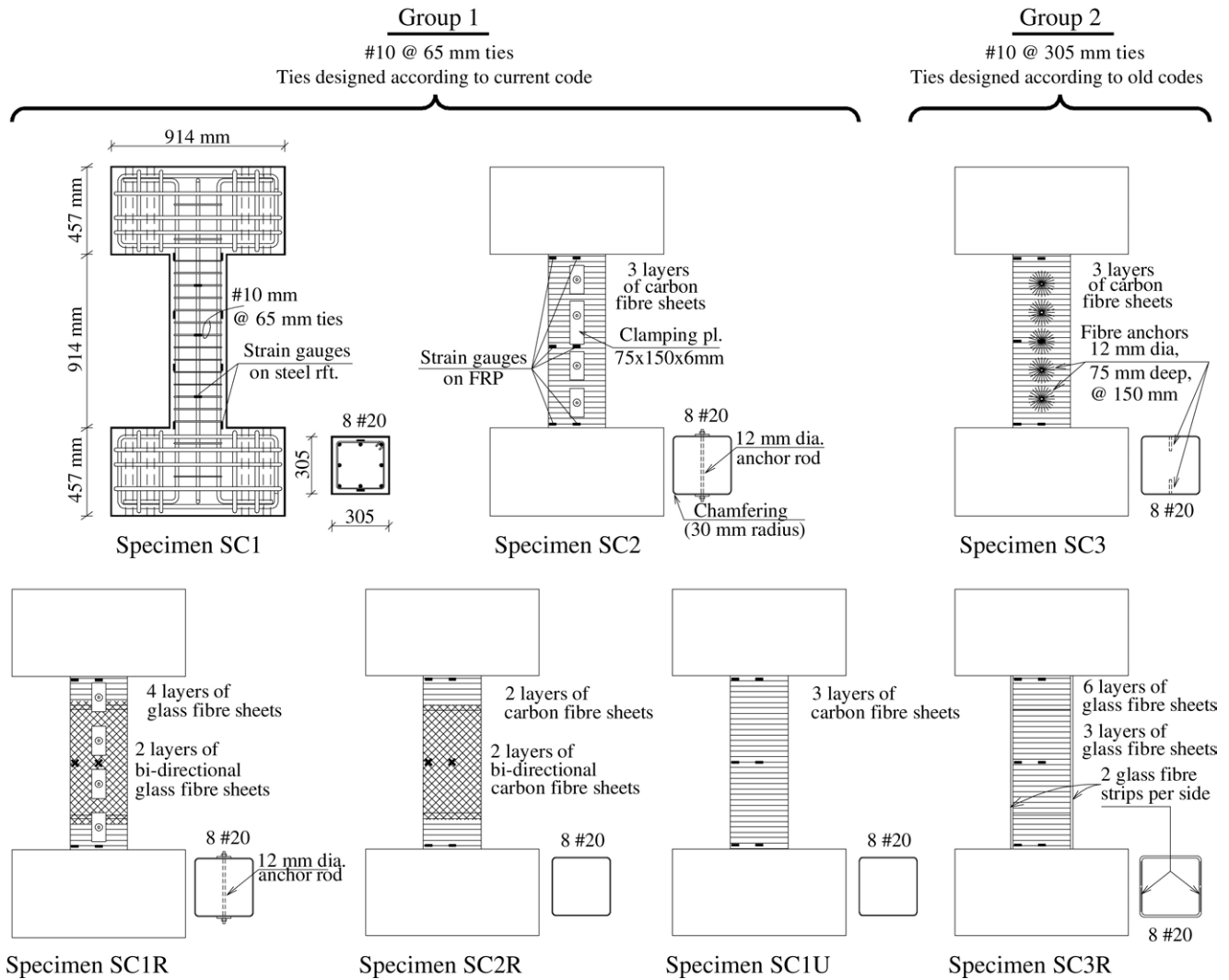


Fig. 1. Dimensions, details of reinforcement and FRP strengthening for the seven specimens.

anchoring, specimen SC1U was strengthened by three layers of CFRP, similar to specimen SC2 but without anchors.

The two columns of Group 2 (SC3 and SC3R) had a low transverse reinforcement ratio selected to satisfy the minimum transverse reinforcement requirement according to 1968 ACI design practice [7]. Without strengthening, the columns of this group would have experienced non-ductile shear failures due to their low transverse reinforcement ratio. This will be demonstrated analytically in Section 5 by predicting the lateral load capacity of an unstrengthened RC short column in Group 2. The shear failure of the unstrengthened column in Group 2 is expected since the control specimen SC1 with higher transverse steel ratio failed due to brittle shear. Column SC3 was strengthened using three layers of CFRP sheets. In this specimen, another technique was used to enhance confinement and reduce the bulging of the column sides. Five 75 mm deep carbon fibre anchors were used in each of the two sides of the specimen. Column SC3R was retrofitted using six and three layers of

Table 1(a)
Concrete properties

Specimen	f'_c (MPa)	f_t (MPa)
SC1, SC2, and SC3	39	3.2
SC1R, SC2R, and SC3R	34	2.8
SC1U	43	3.9

Table 1(b)
Reinforcing steel properties

Bar	Nominal bar area (mm ²)	Yield stress (MPa)	Ultimate strength (MPa)
#20	300	432	684
#10	100	421	662

unidirectional GFRP sheets at the plastic hinge zones and intermediate zone, respectively. The amount of the GFRP at the plastic hinge zone provides shear strength that is close to

Table 2

Properties of the composite materials used in rehabilitation as provided by the suppliers [8,9]

Property	Unidirectional GFRP (MBrace EG900)	Unidirectional CFRP (MBrace CF130)	Bi-directional $\pm 45^\circ$ GFRP (Tyfo BC)	Bi-directional $\pm 45^\circ$ CFRP (Tyfo BCC)	Tyfo UG Composite strip
Ultimate tensile strength (in primary fibre direction)	1700 MPa	3550 MPa	279 MPa	717 MPa	896 MPa
Tensile elongation	2.0%	1.5%	1.5%	1.1%	2.2%
Tensile modulus	71 GPa	235 GPa	19 GPa	65 GPa	41 GPa
Sheet thickness	0.353 mm	0.165	0.864 mm	0.864 mm	1.200 mm

that provided by the three CFRP layers of specimen SC3. Two vertical GFRP strips (150 mm wide each) were laid onto the loading and opposite sides of the column. The vertical GFRP strips were sandwiched between the lateral GFRP confining sheets. The vertical strips were thought to provide an out-of-plane stiffness to the flexible sheets that are subjected to bulging of concrete and buckling of vertical reinforcement bars between transverse ties. A schematic of the test columns SC3 and SC3R are shown in Fig. 1.

2.2. Materials

The nominal concrete strength f'_c was 25 MPa. Table 1(a) lists the 28 day concrete cylinder test results for the seven column specimens. In Table 1(a), f_t represents the concrete tensile strength obtained from a split cylinder test. Grade 400 steel bars were used for longitudinal and transverse reinforcement. Coupons of the deformed bars were tested to obtain yield and ultimate stresses as summarized in Table 1(b). The mechanical properties of the fibre composite materials as provided by the suppliers [8,9] are shown in Table 2.

3. Test set-up

The test set-up consisted of a reaction frame supporting the lateral hydraulic actuator and a vertical structure supporting the vertical hydraulic actuator. A pantograph system was installed to restrain the top fixed end of the test column against rotation. Fig. 2 shows the test set-up and details of the loading system. During the test, loads, deflections and strains were measured to monitor the behaviour of the column. Lateral deflections of the specimen and the relative rotations of the two end blocks of the column were measured using linear variable differential transformers (LVDTs). Strains in the longitudinal and transverse reinforcement as well as the fibre sheets were measured using strain gauges. The locations of the strain gauges are shown in Fig. 1. For each specimen, four strain gauges were placed on each of the four corner longitudinal bars and located at the bottom, 1/3 height, 2/3 height, and the top of each bar. Two strain gauges were placed on three ties for each specimen. The three ties were chosen to have consistent locations for the seven specimens, thus the position of the three ties of Group 2 specimens was used for

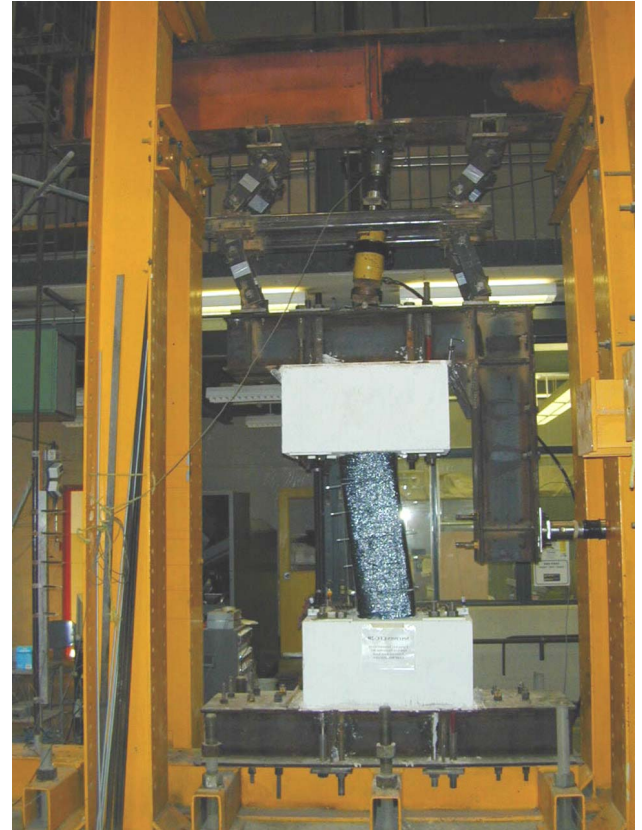


Fig. 2. Test set-up.

the seven specimens. Six strain gauges were placed on the outer surface of the FRP sheets; three gauges were at the centerline axis of the column while the other three were near the edge of the column and were located at the bottom, mid-height and top end of the column as shown in Fig. 1.

The test specimens were subjected to a 500 kN constant axial load. For all specimens, the lateral cyclic displacement level was selected as multiples of Δ_y , with two cycles at each displacement level. The yield deflection Δ_y for each specimen was defined as the displacement at the top of the column when first yielding of the longitudinal bars occurs. The measured yield displacements for all the test columns were fairly consistent. The measured axial load on the column tended to vary during the lateral loading of the specimen due to the elongation or shortening of

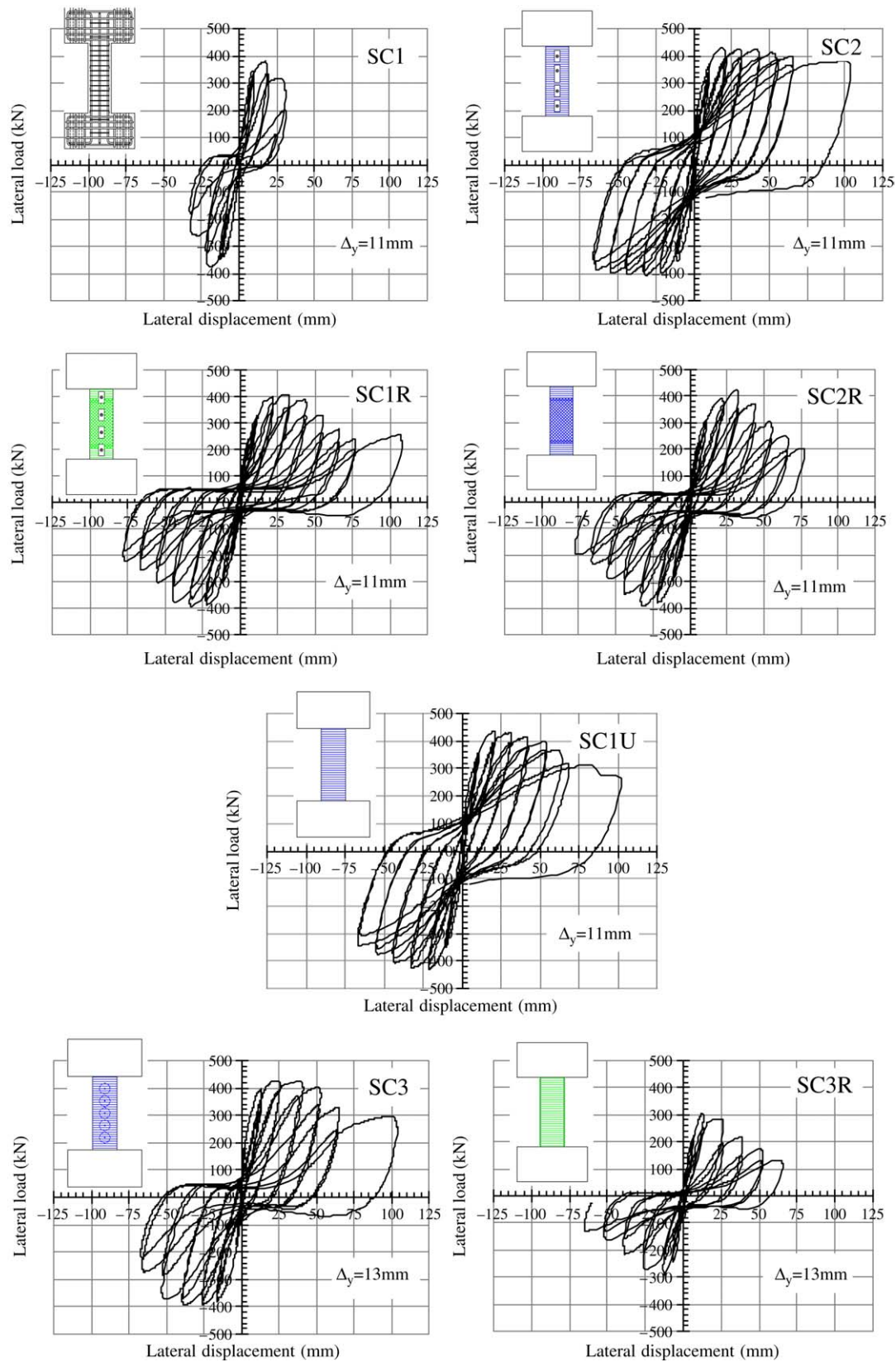


Fig. 3. Comparison between the lateral force–displacement relationships for the seven specimens.

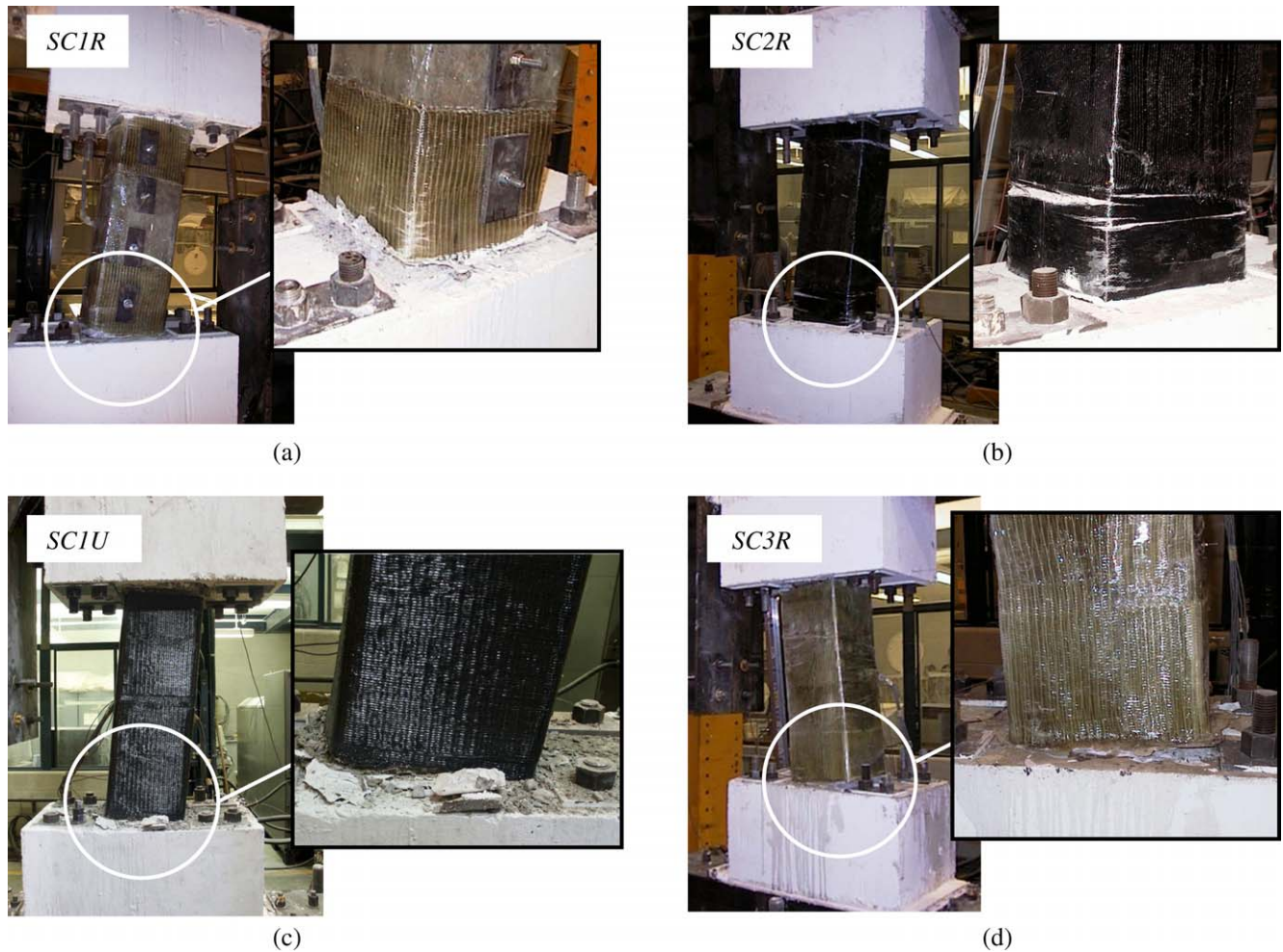


Fig. 4. Test specimens at maximum lateral displacement: (a) SC1R; (b) SC2R; (c) SC1U; and (d) SC3R.

the specimen's height. Although attempts were made to maintain the axial load constant, fluctuations in the order of $\pm 4\%$ of the required 500 kN load were measured. This level of variation may be tolerated since analysis has shown that vertical loads can vary by more than 4% during earthquakes [10].

4. Test results

4.1. Lateral deflection and crack-pattern

The lateral load–deflection relationships for the seven columns are plotted in Fig. 3. The deflection is that of the top block (close to the column top end). The lateral load is adjusted to include the contribution of the horizontal component of inclined axial load. Fig. 4 shows the crack pattern of specimens SC1R, SC2R, SC1U and SC3R at maximum lateral displacement. The detailed behaviour of specimens SC1R, SC2R, SC1U and SC3R is described in the following sections.

4.1.1. Specimen SC1R

For Group 1 specimens, an unstrengthened RC column (specimen SC1) designed using current code [6] failed in shear at a drift ratio of 3.5%. Fig. 3 shows that column SC1R reached a maximum lateral displacement of 105 mm, which corresponds to a drift ratio of 11.5% before its failure. The drift ratio at 70% of the specimen's maximum lateral load capacity was 7%. Fig. 4(a) shows the test column at maximum lateral displacement. After the test, the GFRP wrapping was removed from the top and bottom parts of the column. The concrete under the fibre sheets at the plastic hinge location near the two ends of the column had several shear and flexure cracks. Yet the crack widths were small and the concrete remained functional due to the confining effect of the GFRP. The strengthening scheme used was successful in preventing the column shear failure, improving the ductility and strength of the specimen and developing flexural plastic hinges at the top and bottom of the column as compared with the control specimen SC1.

4.1.2. Specimen SC2R

Fig. 3 shows that specimen SC2R reached a maximum lateral displacement of 75 mm, which corresponds to a drift ratio of 8% before its failure. The drift ratio at 70% of the specimen's maximum lateral load capacity is 6.5%. Fig. 4(b) shows the specimen at its maximum lateral displacement. The repeated loading cycles at each displacement ductility level up to $\mu = 3$ did not show significant loss of strength. At a displacement ductility factor of 4, strength deterioration started and this can be attributed to the formation of new cracks and widening of the old cracks. Horizontal cracks in the concrete and separation between the fibres were observed at a distance of 150 and 75 mm from the bottom and top ends of the column, respectively. A bulge of the fibre sheets was also observed approximately 75 mm above the column base in the compressed side of the column. Pinching of the hysteretic loops and debonding of the CFRP sheets in the plastic hinge zones were observed during the eleventh cycle at a displacement ductility level of 5. At a displacement ductility level of 6, pinching of the hysteretic loops increased due to development of shear cracks in the concrete under the CFRP. Debonding of the FRP jacket increased at a displacement ductility of 7. After this displacement limit, the test was halted as the specimen had lost approximately 50% of its ultimate lateral load resistance capacity. The CFRP sheets were removed from the top and bottom parts of the column. The concrete under the material at the plastic hinge location at the two ends of the column was severely crushed. The strengthening scheme used was successful in preventing the column shear failure, improving the ductility and strength of the specimen and developing flexural plastic hinges at the top and bottom ends of the column as compared with the control specimen SC1.

4.1.3. Specimen SC1U

Fig. 3 shows that specimen SC1U reached a maximum lateral displacement of 105 mm, which corresponds to a drift ratio of 11.5% before its failure. Fig. 4(c) shows specimen SC1U at maximum displacement. Unlike specimen SC2, SC1U showed signs of debonding between the CFRP wrap and the concrete column. This could be attributed to the absence of anchorage for the jacket. The debonding started at a displacement ductility of 4. The ductile behaviour of the specimen was due to the formation of plastic hinges at the column ends as a result of the increased shear capacity of the column.

4.1.4. Specimen SC3R

For Group 2 specimens, the analytical model described in Section 5 of this study predicts that an unstrengthened RC short column designed according to 1968 ACI code [7] and similar to specimen SC3 will exhibit a brittle shear failure with no ductility. This behaviour was also confirmed by the shear failure of column SC1 with more transverse reinforcement. On the other hand, specimen SC3 that is rehabilitated using three layers of anchored CFRP reached

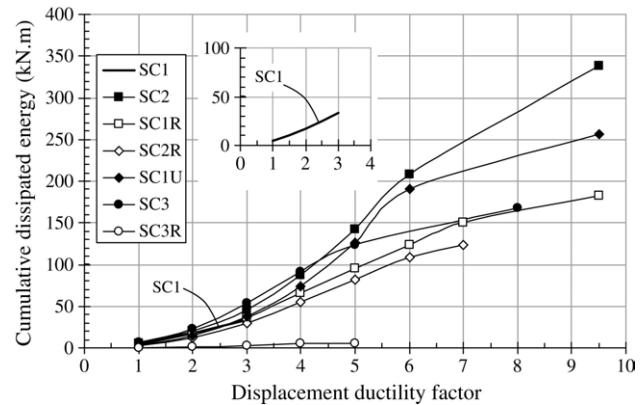


Fig. 5. Cumulative dissipated energy–displacement ductility factor relationship.

a drift ratio of 11.5%. Fig. 3 shows that a similar column SC3R retrofitted by six and two layers of GFRP sheets at plastic hinge and intermediate zones, respectively, did not reach its full shear strength and exhibited limited ductility. This can be attributed to the bulging of the vertical GFRP strips on the compression side of the column, which reverses during cyclic loading. In fact, this bulging weakened the bond between the GFRP jacket and the column due to its imposed outward pressure on the jacket. This action in turn reduced the effectiveness of the confinement. In addition, the absence of mechanical anchorage between the GFRP jacket and the column expedited the debonding between the jacket and the column. Fig. 4(d) shows the column at its maximum lateral displacement. At lateral displacement of ± 13 mm ($1.0\Delta_y$) the specimen reached its maximum positive and negative lateral load capacities of +304 kN and –291 kN, respectively. During this cycle, debonding of GFRP in the middle zone of the column occurred. At a displacement ductility factor of 3, considerable loss of strength and degradation of stiffness occurred and the hysteretic loops became increasingly pinched. The test was halted after the specimen reached lateral displacements of ± 65 mm as the specimen had lost more than 50% of its maximum lateral load carrying capacity. The column failed due to the formation of a wide diagonal shear crack at mid-height. This rehabilitation scheme failed to prevent the shear failure of the column because of the low modulus of elasticity property of the unidirectional GFRP and the absence of an anchoring system for the fibre. The GFRP wrapping was removed from the column. The concrete under the fibre jacket had several diagonal shear cracks forming an X-pattern at the middle of the column and zigzag shapes near the edges.

4.2. Energy dissipation capacity

For each specimen, the energy dissipated in each cycle was obtained by calculating the area enclosed by the corresponding load–displacement hysteretic loop. The cumulative energy dissipated was then calculated by summing the energy dissipated in consecutive cycles

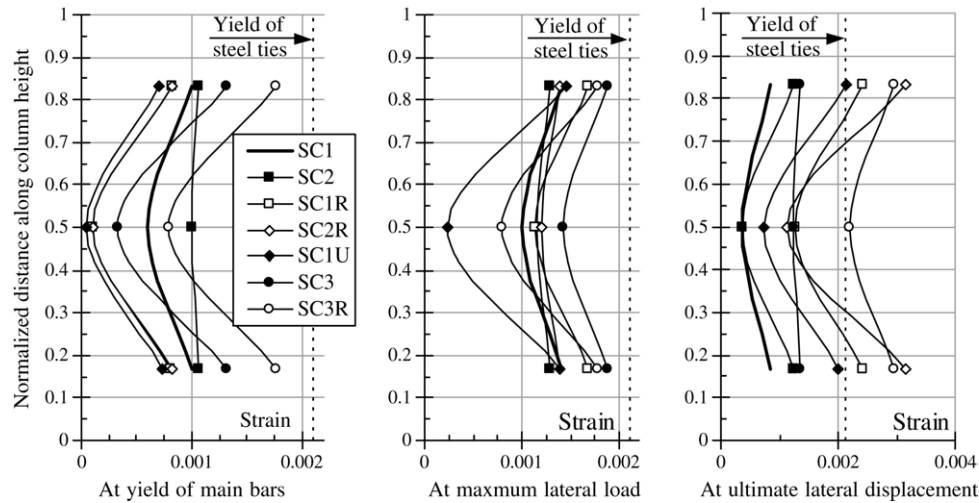


Fig. 6. Tie strain distribution along the column height.

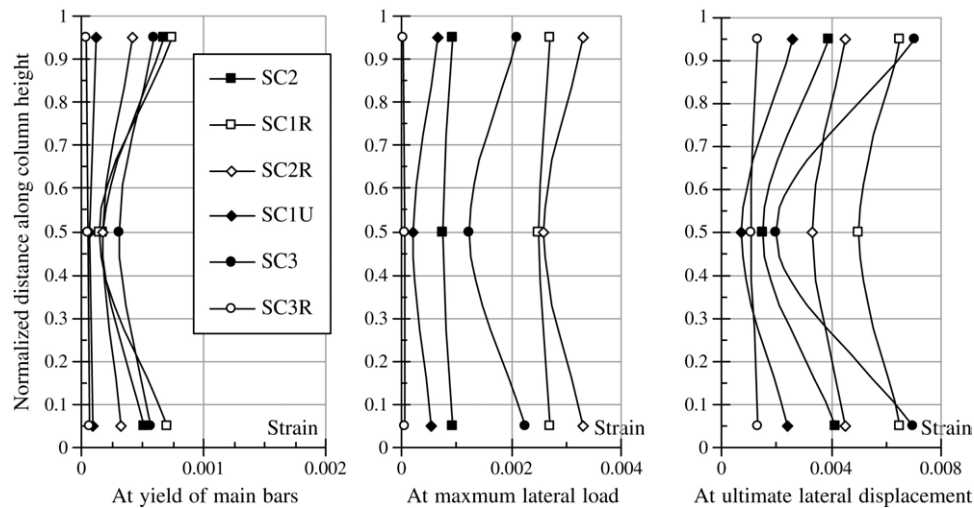


Fig. 7. FRP strain distribution along the column height.

throughout the test. Fig. 5 shows the cumulative dissipated energy versus the displacement ductility factor for the test columns. The figure shows that all the strengthened specimens except for SC3R with low reinforcement, far out performed the control column SC1. For clarity, the behaviour of column SC1 is plotted in a magnified zoom in inset in Fig. 5.

4.3. Strains in transverse reinforcement and fibre sheets

Fig. 6 shows the distribution of strain in the transverse reinforcement along the height of the column at the first yield of main reinforcement bars, maximum lateral load and ultimate lateral displacement for the seven column tests. The strains in the ties of the three strengthened specimens SC1R, SC2R, and SC3R at maximum drift were higher than the yield strain. On the other hand, the strain in the ties of the three rehabilitated specimens SC2, SC1U and SC3 did not

reach the yield strain throughout the loading history. This indicates that the rehabilitation schemes used for specimens SC2, SC1U and SC3 were successful in providing adequate confinement and shear strength to allow the ductile hinging at the column ends to take place before shear failure. Three layers of CFRP were applied in the strengthening of these columns.

Fig. 7 shows the distribution of strain in the FRP along the column height at the first yield of main reinforcement bars, maximum lateral load and ultimate lateral displacement for the six rehabilitated columns. High FRP strains were recorded in columns SC2, SC1R, SC2R, SC1U and SC3 at the ultimate lateral displacement. This is a strong indication that the rehabilitation schemes used for these columns were successful in improving their performance, while the GFRP jacketing of column SC3R with low reinforcement ratio failed to achieve its objectives.

Table 3
Parameters used to calculate the confinement effectiveness

Specimen	A_e (mm ²)	K_e	$2t_f$ (mm)	ε_{fe}	E_f (GPa)	A_v (mm ²)	s (mm)	ρ_{eff}	f'_l (MPa)	f'_{cc} (MPa)
SC1	41 850	0.642	–	–	–	200	65	0.0118	3.000	54
SC2	64 370	0.710	0.990	0.006	235	200	65	0.0246	7.340	75
SC1R	64 370	0.710	2.824	0.006	71	200	65	0.0224	6.686	67
SC2R	64 370	0.710	0.660	0.004	235	200	65	0.0166	4.940	60
SC1U	64 370	0.710	0.990	0.004	235	200	65	0.0199	5.950	74
SC3	20 735	0.229	0.990	0.006	235	200	305	0.0153	1.480	48
SC3R	20 735	0.229	4.236	0.004	71	200	305	0.0140	1.347	42
SC3n	8946	0.133	–	–	–	200	305	0.0025	0.141	40

Table 4
Shear strength contribution from different mechanisms and shear capacity envelope (in kN)

Specimen	V_c	V_p	V_s	V_f	$\max(V_s + V_f) = 0.66\sqrt{f'_{cc}bd}$	V_n	$V_{\mu=4}$	$V_{\mu=6}$	V_r
SC1	92	83	336	–	392	511	394	336	252
SC2	167	83	336	351	462	712	545	462	347
SC1R	158	83	336	303	437	678	517	437	328
SC2R	150	83	336	156	413	646	491	413	310
SC1U	167	83	336	234	459	709	542	459	345
SC3	43	83	71	351	370	496	412	370	278
SC3R	40	83	71	303	345	468	386	345	259
SC3n	17	83	71	–	330	171	–	–	–

5. Analytical modeling

Ghobarah and Galal [5] proposed a procedure to predict the combined flexure and shear response of short RC rectangular columns retrofitted using anchored and unanchored FRP jackets. The analytical predictions were compared to the experimental results for specimens SC1, SC2, and SC3 with good agreement. Appendix A includes a summary of the analytical model.

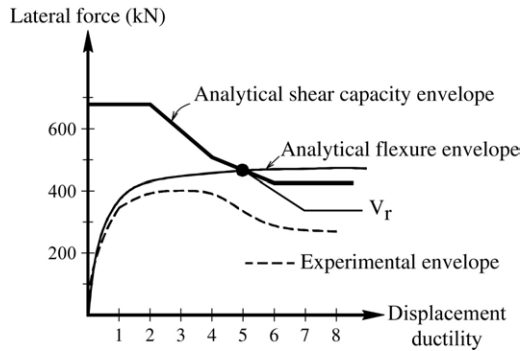
In the current study, the same analysis procedure is applied to specimens SC1R, SC2R, SC1U, and SC3R. Table 3 contains the parameters used to calculate the confinement effectiveness as provided by the steel ties and the FRP jacket. Table 4 lists the shear strength contribution from different mechanisms and the total shear capacity envelope at different displacement ductility levels. Fig. 8 shows the analytical flexure and shear envelopes and the experimental response for columns SC1R, SC2R, SC1U, and SC3R. The column's analytical backbone behaviour will follow the flexure envelope until it reaches the shear capacity envelope. Subsequently, the column loses its lateral force resistance capacity following a negative shear stiffness, until the residual shear capacity V_r is reached. Brittle or moderate-ductile behaviours are expected if the column's shear capacity is reached at displacement ductility levels less than 2 or between 2 and 6, respectively such as column SC3R. A ductile behaviour is expected if the column's shear capacity is higher than its flexural envelope at high displacement ductility levels such as columns SC1R, SC1U

and SC2R. Reasonable correlation between the experimental results and the analytical predictions were shown with the theoretical results overestimating the load as compared to the experimental measurements. The difference may be attributed to the interaction between the flexural and shear which was not included in the analysis.

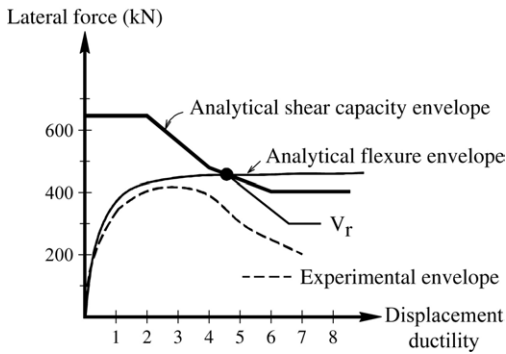
The analytical procedure was then used to predict the shear capacity of a RC short column, SC3n, similar to SC3 but with no FRP strengthening. The column SC3n represents an existing RC square short column with shear reinforcement that satisfies the minimum transverse reinforcement requirement according to the ACI 1968 [7] design practice code. Column SC3n can be considered as the control specimen for Group 2 columns, which was not tested. Tables 3 and 4 include pertinent data that is used to calculate the shear capacity of column SC3n. Table 4 shows that SC3n is expected to have a shear capacity V_n that is less than half the flexure capacity at yield of a similar but rehabilitated column SC3. Therefore, the unstrengthened column SC3n is expected to suffer a brittle shear failure before it reaches its flexural yield capacity. On the basis of the analytical predictions, column SC3n was not tested since the test was not expected to provide any new information.

6. Effect of anchoring

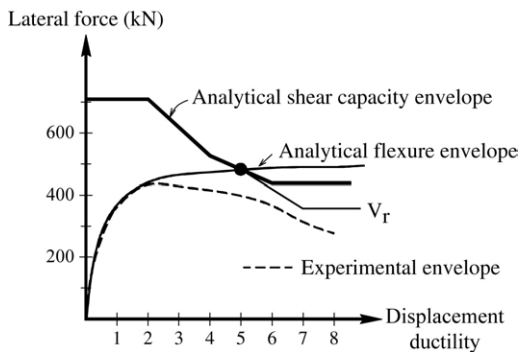
One of the recognized problems in the confinement of rectangular column with FRP sheets is debonding between the FRP and concrete. The debonding is accelerated by



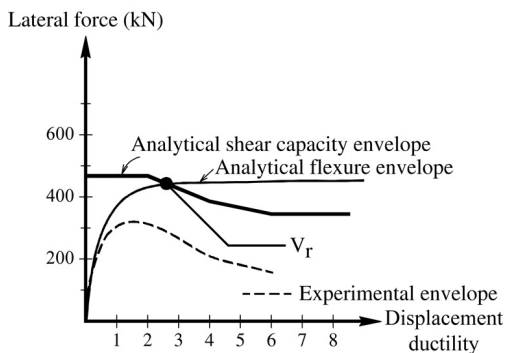
(a) Specimen SC1R.



(b) Specimen SC2R.



(c) Specimen SC1U.



(d) Specimen SC3R.

Fig. 8. Analytical and experimental response for specimens: (a) SC1R; (b) SC2R; (c) SC1U; and (d) SC3R.

concrete cracking under the FRP. Although confinement of the concrete at the column corners is effective, the column sides are not well confined and can bulge out due to concrete

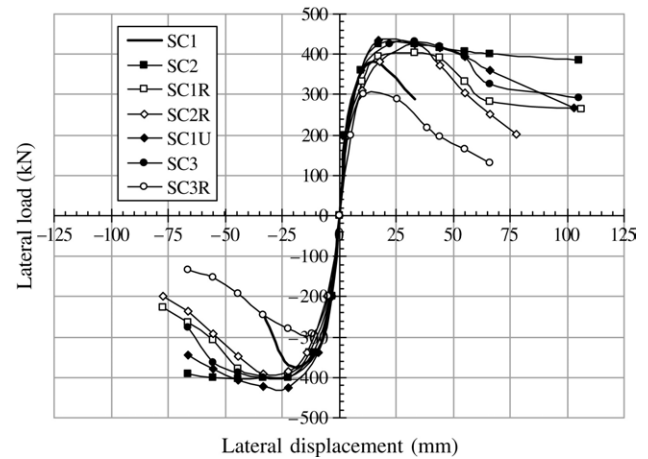


Fig. 9. Lateral force–displacement envelopes for the seven specimens.

cracking when the column is subjected to lateral loads. Debonding between the concrete and composite sheets occurred in the tested specimens SC2R, SC1U, and SC3R.

Providing anchors to improve the distribution of confinement pressure on rectangular columns, was successfully implemented by Ghobarah and Said [4]. Anchoring of the FRP to concrete may be achieved using tie rods or mechanical or fibre anchors. The improvement in the distribution of the confinement pressure on the column reduces the bulging of the column sides. Strains recorded in the anchor rods at maximum drift reached approximately half the yield strain in specimens SC2 [5].

Both specimens SC2 and SC1U had the same transverse steel content and were strengthened with three layers of CFRP. In column SC1U no anchors were used. Four anchor rods with clamping plates were used in specimen SC2 to provide anchorage between the column and the jacket. Fig. 9 shows a comparison between the lateral load–displacement envelopes for the seven test columns. Column SC2 showed a more ductile response without recognizable decrease in lateral force resistance, as compared to the response for column SC1U. In addition, comparing the dissipated energy capacity for the two specimens as shown in Fig. 5, indicates that the dissipated energy capacity of the anchored CFRP jacketed column is approximately 25% more than that of an identical column with an unanchored jacket.

A comparison between the tie strain and CFRP strain distribution along the column height for the two specimens SC2 and SC1U is shown in Figs. 6 and 7, respectively. From the figures it can be seen that anchoring the CFRP sheets to the concrete column as in the case of column SC2 decreased the strains in the transverse reinforcement and increased the strains in the CFRP as compared to the column SC1U, which was not provided with anchors. It is noted that the foregoing conclusions concerning the performance of anchored jackets are based on limited number of tests. However, these two tests confirm earlier conclusions on anchored jackets of beam–column joints. Anchoring FRP sheets to the concrete column may increase the cost of the rehabilitation scheme by

approximately 20%. However, the associated performance enhancement would make the additional cost worthwhile.

7. Effect of transverse steel content

Prior to 1970 not much attention was paid to ductile detailing of short columns. Once the strength of a short column is reached, abrupt non-ductile deterioration follows, which reduces the energy dissipating capacity of the column and results in brittle failure. Current codes attempt to provide for high lateral reinforcement content to increase confinement and increase the shear resistance of the column.

Specimens SC2 and SC3 were designed according to current and pre-1970 codes, respectively. Specimen SC2 has #10 bar ties at 65 mm spacing, while specimen SC3 has #10 bar ties at 305 mm spacing. Both specimens were strengthened using three layers of anchored CFRP jackets. Comparing the lateral force–displacement envelopes for specimens SC2 and SC3 shown in Fig. 9 indicated that at large lateral displacement (larger than displacement ductility of 3), the column with the higher transverse steel content will have higher lateral load capacity due to improved confinement. Comparing the dissipated energy capacity for specimens SC2 and SC3 shown in Fig. 5 indicates that increasing the transverse steel content increases the dissipated energy capacity again due to the ductile behaviour of the confined column.

Comparing the tie strain and CFRP strain distribution along the height of columns SC2 and SC3 at ultimate lateral displacement shown in Figs. 6 and 7 indicates that increasing the transverse steel content decreases the tie strains at the column mid-height, while it has negligible effect on the tie strain at the column ends. On the other hand, increasing the transverse steel content reduced the strains in the FRP jacket along the column height. The ties at the top and bottom ends of the column are effective in confining the flexural hinge location while the ties near the mid-height of the column resist the shear. In column SC3 with low lateral reinforcement content, the fibre material is relied upon for the confinement of the flexural hinge zones.

8. Effect of the number of FRP layers

Specimens SC2R and SC1U have the same transverse steel content. However, test columns SC2R and SC1U were rehabilitated using two and three layers of unidirectional CFRP layers, respectively. Comparing the lateral force–displacement envelopes for the two specimens as shown in Fig. 9 indicates that increasing the number of FRP layers increased the column's shear capacity, which resulted in a more ductile response. The cumulative dissipated energy capacity for specimen SC1U with the larger number of FRP layers is higher than that of SC2R as can be seen in Fig. 5. Comparing the tie strain and CFRP strain distribution along the height of both specimens at ultimate lateral displacement

shown in Figs. 6 and 7 indicates that increasing the number of layers of CFRP sheets consistently decreases the strains in both the transverse steel ties and fibre sheets.

The beneficial improvement in the behaviour due to the increase in the number of FRP layers of Group 1 specimens is applicable to Group 2 specimens that represent columns built prior to 1970. The analytical model described above shows that increasing the number of FRP layers increases the column's shear capacity by increasing its concrete V and FRP V_f shear mechanisms. This effect is beneficial until the failure mode is no longer brittle shear but is governed by the flexural capacity of the column.

9. Effect of FRP material

Specimens SC2 and SC1R have the same transverse steel content and are strengthened using FRP jackets that are anchored to the column by means of through anchor rods with clamping plates. Two different types of fibre material were used in strengthening the two specimens. The jacket for specimen SC2 included three layers of CFRP, while specimen SC1R was wrapped with four layers of GFRP. Although the two specimens were designed to have similar shear capacity envelopes, the experimental response for the two specimens was different.

Specimen SC2 strengthened using CFRP had higher lateral force capacity as compared to column SC1R that was strengthened using GFRP as shown in Fig. 9. Similar observation can be also made from examining Fig. 5 where strengthening RC short column SC2 with anchored CFRP has higher energy dissipating capacity as compared to the similar column SC1R strengthened with GFRP.

Comparing the tie strain and CFRP strain distribution along the height of both test columns at ultimate lateral displacement shown in Figs. 6 and 7 indicates that in the case of CFRP jackets the strains in the steel ties and the FRP along the column height decreased. This is due to the high modulus of elasticity of the carbon fibre material as compared to the glass fibre material. The poor confinement of the glass fibre jacket allowed the cracks in the concrete to widen with the consequence of debonding of the fibre and loss of shear resistance. The difference in behaviour between the two materials is consistent along the height of the column.

10. Conclusions

The results from an experimental study that included seven RC square short columns tested under cyclic lateral loads and constant axial load were discussed. The specimens were separated into two groups. Group 1 is designed according to current code CSA [1994] with high content of transverse reinforcement, while Group 2 was designed according to pre-1970 code [ACI 1968] with low content of transverse reinforcement. The analytical prediction of the

response of the as built column indicated that it will suffer a brittle shear failure with no ductility. Based on the results of the experimental program, the following conclusions are reached:

1. Anchoring of the FRP jacket to the concrete cross section of short column subjected to lateral cyclic displacements increases the column's shear force and energy dissipation capacities.
2. The anchors improved the confinement of the concrete with the resulting decrease in the strains in the transverse reinforcement and increase in the strains in the fibre jacket.
3. Tie strain at column ends in RC short columns strengthened with anchored FRP is not affected by transverse steel reinforcement ratio. However, the strain in the FRP jacket along the column height was found to decrease with increasing transverse steel content of the column. The transverse steel reinforcement ratio has negligible effect on the tie strain at the column ends and reduces the strains in the FRP jacket along the column height.
4. Increasing the number of FRP layers in short column strengthening decreases the strains in both the transverse steel ties and fibre materials.
5. Using anchored carbon fibre sheets rather than anchored glass fibre sheets for strengthening RC short columns increases both the shear force and the energy dissipating capacity. It also decreases the strains in the steel ties and the FRP along the column height.
6. From the test results, the most appropriate strengthening technique for short columns was found to be the use of anchored CFRP jackets over the height of the column and the additional confinement of the end flexural hinge zones using CFRP sheets with unidirectional hoop fibre orientation.

Appendix A. Summary of the analytical model

In this appendix, a summary of the analytical model is presented. Details of the model are available elsewhere [5].

The nominal shear capacity, V_n , of an RC column retrofitted with FRP composites is equal to the sum of the contributions of four mechanisms; concrete V_c , axial load V_p , transverse steel V_s , and FRP V_f :

$$i.e. V_n = V_c + V_p + V_s + V_f. \quad (1)$$

Fig. 10 shows the shear strength envelope of an axially and laterally loaded short column with respect to its displacement ductility. The model assumes that the column shear capacity decreases bi-linearly with the increase of the lateral displacement ductility, μ , after reaching $\mu = 2$ such that:

$$V_{\mu=4} = 1/3(V_c + V_p) + V_s + V_f \quad \text{at } \mu = 4 \quad (2a)$$

$$V_{\mu=6} = V_s + V_f \quad \text{at } \mu = 6. \quad (2b)$$

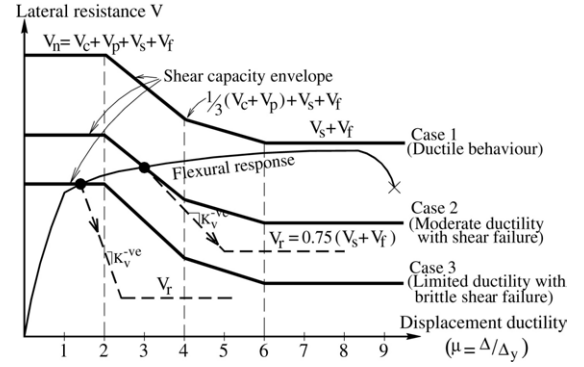


Fig. 10. Combined flexural and shear response for columns.

Combining the shear strength envelope with the flexural capacity envelope of the column results in one of three responses; (1) Ductile behaviour; (2) Moderate ductility with shear failure; and (3) Limited ductility with brittle shear failure. Fig. 10 shows the combined flexure and shear response of the axially and laterally loaded RC column. The column is assumed to undergo degradation in the shear capacity from V_c to V_r (residual shear strength) for the cases of moderate and limited ductilities. The residual shear strength V_r is estimated from experimental observations to be $0.75(V_s + V_f)$. The degradation in strength occurs through a displacement equivalent to once and twice the yield displacement for limited and moderate ductilities, respectively.

In calculating the shear strength of the FRP mechanism V_f , the model accounts for the simultaneous confinement effect of the FRP jacket on increasing of the concrete strength, which in turn increases the shear strength contribution of the concrete mechanism V_c . The contributions of the four mechanisms that add up to the nominal shear strength of a rectangular RC short column retrofitted with FRP are given by the following equations:

$$V_c = 0.3\sqrt{f'_{cc}}A_e \quad (3)$$

$$V_p = k_p \frac{Pt/2}{H} \quad (4)$$

$$V_s = \frac{A_v f_{yv} d}{s} \quad (5)$$

$$V_f = 0.95(2t_f)(\varepsilon_{fe} E_f) d_f \quad (6)$$

where

- f'_{cc} is the confined compressive strength of concrete,
- A_e is the area of effectively confined concrete core,
- k_p = 1 for columns in double curvature and 0.5 for columns in single curvature,
- P is the axial load on the column,
- t is the total depth of the column cross section,
- H is the column height,
- d is the column section depth to the tensile steel,
- A_v, f_{yv}, s are the total cross sectional area, yielding strength and spacing of transverse reinforcement,

$2t_f$ is the total transverse design thickness of FRP sheets (i.e. for two opposite sides),
 ε_{fe} is the design strain for FRP; $\varepsilon_{fe} = 0.004$ for unanchored FRP sheets and $\varepsilon_{fe} = 0.006$ for anchored FRP sheets,
 E_f is the Young's modulus of the FRP composite material, and
 d_f is the depth of the FRP in the direction of load.

The ACI [11] limits the total shear strength where more than one type of shear reinforcement is used to:

$$V_s + V_f \leq 0.66\sqrt{f'_{cc}}bd \quad (7)$$

where b is the width of the column.

To calculate the confined concrete compressive strength, f'_{cc} , at the design strain ε_{fe} of the FRP, the Mander model [12] was applied at constant lateral confining pressure using an effective transformed confinement content, ρ_{eff} , for steel ties and FRP, provided that ε_{fe} is larger than the yield strain of the steel ties.

$$\frac{f'_{cc}}{f'_{co}} = 2.254\sqrt{1 + 7.94\frac{f'_l}{f'_{co}}} - 2\frac{f'_l}{f'_{co}} - 1.254 \quad (8)$$

where

f'_{co} is the unconfined concrete compressive strength, and
 f'_l is the effective lateral confining pressure.

$$f'_l = K_e \rho_{eff} f_{yv} \quad (9)$$

where K_e is the confinement effectiveness coefficient

$$K_e = \frac{A_e/b_c d_c}{\left(1 - \frac{A_v}{b_c d_c}\right)} \quad (10)$$

with

$$A_e = \left(b_c d_c - \sum_{i=1}^n \frac{(w'_i)^2}{6}\right) \left(1 - \frac{s_x}{2b_c}\right) \left(1 - \frac{s_y}{2d_c}\right). \quad (11)$$

Such that b_c and d_c are: (a) in the case of no FRP: the core dimensions to centerlines of perimeter ties in x and y directions, respectively; and (b) in the case of FRP jacket: the width b and the total depth t of the column, respectively.

w'_i is the i th clear distance between adjacent longitudinal bars, and
 s_x, s_y are the least of transverse hoop reinforcement and FRP anchor spacing in x and y directions, respectively.
 ρ_{eff} is the effective transformed confinement content; transforming the FRP sheets at strain ε_{fe} into an equivalent steel content having the same steel yield strength and is given by:

$$\rho_{eff} = \frac{A_v}{s d_c} + \frac{2t_f}{d_c} \cdot \frac{\varepsilon_{fe} E_f}{f_{yv}}. \quad (12)$$

References

- [1] Yoshimura K, Kikuchi K, Kuroki M, Ozawa K, Masuda Y. Experimental study on seismic behavior of R/C short columns strengthened by carbon fiber sheets. In: Composite and hybrid structures: Proceedings of the sixth ASCS international conference on steel-concrete composite structures. 2000. p. 927–34.
- [2] Haroun MA, Mosallam AS, Feng MQ, Elsanadedy HM. Composite jackets for the seismic retrofit and repair of bridge columns. In: Seventh U.S. national conference on earthquake engineering. 2002. p. 10.
- [3] Ye L, Yue Q, Zhao S, Li Q. Shear strength of reinforced concrete columns strengthened with carbon-fiber-reinforced plastic sheet. Journal of Structural Engineering 2002;128(12):1527–34.
- [4] Ghobarah A, Said A. Shear strengthening of beam-column joints. Engineering Structures 2002;24(7):881–8.
- [5] Ghobarah A, Galal K. Seismic rehabilitation of short rectangular RC columns. Journal of Earthquake Engineering 2004;8(1):45–68.
- [6] CSA. Design of concrete structures. Standard A23.3. Rexdale (Ontario, Canada): Canadian Standards Association; 1994.
- [7] ACI. Manual of concrete practice. Part 2, Committee 318-1R-68. Detroit (USA): American Concrete Institute; 1968.
- [8] Master Builders Inc. Product technical specification. August 2001. <http://www.mbrace.com>.
- [9] Fyfe. Product technical specification. August 2002. <http://www.fyfeco.com>.
- [10] Ghobarah A, Elnashai AS. Contribution of vertical ground motion to damage of RC buildings. In: Eleventh European conference on earthquake engineering. Rotterdam: Balkema; 1998. p. 10.
- [11] ACI 440.2R-02. Guide for the design and construction of externally bonded FRP systems for strengthening concrete structures. Committee 440 Report. Detroit (USA): American Concrete Institute; 2002.
- [12] Mander JB, Priestley MJN, Park R. Theoretical stress-strain model for confined concrete. Journal of Structural Engineering, ASCE 1988; 114(8):1804–26.