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BEHAVIOUR OF GFRP COLUMNS WITH GFRP CIRCULAR AND RECTILINEAR CONFINING BEHAVIOUR

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Abstract

Glass Fibre-Reinforced Polymer (GFRP) bars are becoming a feasible alternative to steel bars to produce corrosion-free reinforced concrete structures. In an effort to assess the effectiveness of GFRP spirals and rectilinear ties as internal reinforcement in columns, an extensive research program is underway at the University of Toronto. In the experimental part of the program, fifteen 356 mm diameter full-scale circular columns and sixteen 305 mm x 305 mm cross-section square columns were constructed and tested under constant axial loading and cyclic lateral displacement excursions simulating earthquake load. The test parameters included column shape, types of longitudinal reinforcement (steel and GFRP bars), lateral reinforcement ratio, spacing and configuration of lateral reinforcement, and level of axial load. This paper compares the experimental results from a select group of the circular and the square concrete columns in which steel longitudinal reinforcement was used along with GFRP lateral reinforcement. Results are presented in the form of moment vs. curvature response and shear vs. lateral deflection behaviour. A number of ductility parameters related to curvature, displacement, energy dissipation and drift capacity were used to compare the seismic performance of the circular and square specimens. The drift capacity of the columns at failure was in the range of 2.5% to 3.1% for the columns presented here which satisfy the limitations of North American building codes with circular columns performing better than square columns. Strength degradation before the final collapse at large displacement was minimal for both square and circular columns primarily due to the continuing confinement provided by GFRP lateral reinforcement to the concrete core. Based on the data from the tests, it can be concluded that internal GFRP lateral reinforcement in many cases can provide better confinement and better performance of columns than provided by steel.

Keywords: GFRP; Columns; Seismic, Confinement; Reinforced Concrete

1. Introduction

Reinforced concrete structures with conventional steel reinforcement have a short service life in even moderately aggressive environments, such as bridges and highways with exposure to de-icing salts, due to corrosion of the steel reinforcement. In addition to loss of productivity, corrosion also costs billions of dollars in repair and rehabilitation every year. Corrosion of lateral steel in columns causes spalling of concrete cover which results in a drop in load-carrying capacity, ductility and energy dissipation capacity of columns. Furthermore, the steel longitudinal bars may also get exposed to corrosion that can eventually cause structural collapse. The replacement of steel with a non-corroding material like glass fiber reinforced polymer (GFRP) bars is a feasible and cost effective solution that can alleviate this problem.

Fig. 1 - Highway bridge with corrosion damage

Information on the seismic performance of columns containing GFRP as internal longitudinal and lateral reinforcement is almost non-existent. The results from a few studies in which columns with GFRP longitudinal bars were tested concluded that the specimens displayed softer response and lower energy capacity in comparison with steel-reinforced columns for both circular and square sections [1, 2]. The lateral GFRP reinforcement was found to be quite effective. Additionally, based on the results of the tests performed on columns under concentric load, it was found that replacing longitudinal steel bars with GFRP bars irrespective of the type of ties (steel or GFRP) reduced the capacity by 13% [3]. In order to optimize the benefits of GFRP, maximize the advantages and minimize the disadvantages of GFRP bars, it was decided to investigate the performance of hybrid columns with steel longitudinal reinforcement and GFRP lateral reinforcement. It was theorized that the hybrid columns with GFRP spirals and rectilinear ties will prevent cover deterioration, since these bars do not corrode, and the steel longitudinal reinforcement will ensure a stiffer member response. In this study, the performance of circular and square columns hybrid columns subjected to seismic loading was investigated. The effectiveness of confinement provided by GFRP spirals in circular columns versus GFRP rectilinear ties in square columns was also investigated.

2. Experimental Program

A total of 31 columns with GFRP lateral reinforcement have been constructed and tested at the Structures Laboratories at the University of Toronto in this research program. Of the total 31 specimens, four circular and four square columns reinforced longitudinally with steel reinforcement and laterally with GFRP reinforcement were directly comparable to evaluate the configuration of lateral reinforcement. All the eight columns were tested in a similar manner under constant axial load and cyclic quasi-static lateral displacement excursions. The circular columns had a diameter of 356 mm and the cross-section of the square columns was 305 mm x 305 mm. All the columns were 1470 mm long and the corresponding gross cross-sectional areas of the circular and square columns were comparable at 99538 mm^2 and 93025 mm^2, respectively. All eight columns were cast integrally with a 485 × 700 × 800 mm stub which represented a discontinuity like a beam column joint or a footing adjacent to the section of maximum moment.

The circular columns were reinforced longitudinally with six 25M steel bars [4], while the square columns were reinforced with eight 20M steel bars. The circular columns were confined with GFRP spirals and the square columns were confined by GFRP rectilinear ties that were made by the same manufacturer and had similar properties. For the GFRP rectilinear ties, one of the more commonly used tie configuration consisting of an internal diamond shape tie in addition to the external peripheral ties was selected. The schematics of the circular and square column specimens and the corresponding cross-sectional details can be seen in Figure 2.

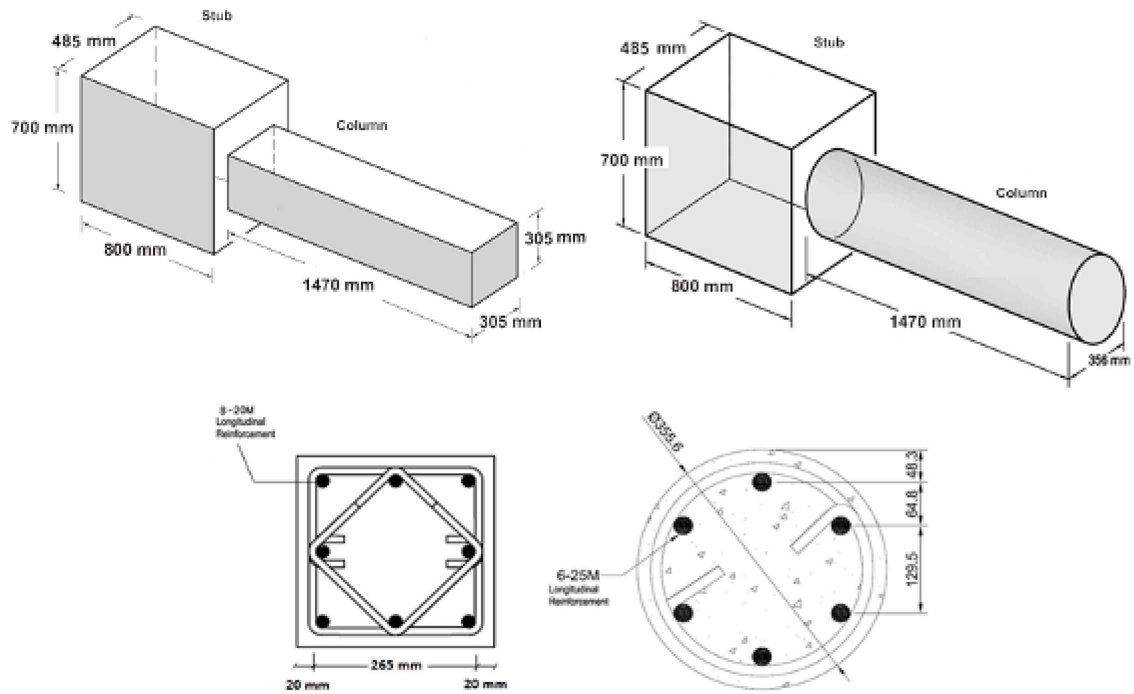


Fig. 2 - Schematics and cross-sections of square and circular specimens

Due to space limitation, only select results from two square and two circular columns are discussed in this paper to highlight the effectiveness of GFRP rectilinear ties and GFRP circular stirrups as confinement; and to compare their behaviour with respect to each other. Other variables that can be studied include the level of axial load and the amount and spacing of transverse reinforcement. The square and circular columns compared had either similar spacing, reinforcement ratio or both spacing and reinforcement ratio. All the other parameters were kept the same in both groups to ensure a proper comparison. Table 1 gives the details of the four specimens.

Table 2 and Table 3 provide relevant information regarding the properties of the reinforcement materials used in the aforementioned four specimens.

All the specimens were tested at the University of Toronto in the Column Testing Frame (CTF); the specimens were placed in the CTF in a horizontal position and subjected to simultaneous constant axial load and cyclic quasi-static lateral excursions simulating earthquake loading. Figure 3 shows the CTF test set-up with a fully instrumented specimen. The part of the column away from the stub was wrapped with carbon fiber reinforced polymer sheet in order to provide additional confinement to ensure that failure occurs within the instrumented potential plastic hinge region close to the intersection of column and stub. Specimens were tested with extensive instrumentation including LVDTs and strain gauges to ensure that all the required test data, specifically the deflections along the specimen length, the axial and lateral loads, and strains in concrete, steel and GFRP reinforcement, were appropriately recoded to gain a thorough understanding of the column behaviour.

Table 1 - Specimen Details

Column Shape	Specimen Name (No.)	f'_c (MPa)	Axial load level, P/P$_o$	Load (kN)	Longitudinal Steel Reinforcement No. – Size	ρ_s (%)	Lateral GFRP Reinforcement Size @ Spacing (mm)	Ratio, ρ_{fh} (%)
Square	TA-P56-S-11 (11)	44	0.56	2334	8 - 20M	2.58	12 @ 90	3
Circular	P-55-LS-16-90 (3)	41	0.55	2450	6 - 25M	2.96	16 @ 90	2.92
Square	TA-P28-S-10 (10)	44	0.28	1167	8 - 20M	2.58	12@160	1.7
Circular	P-28-LS-12-160 (6)	40	0.28	1243	6 - 25M	2.96	12@160	0.94

Table 2 - Mechanical Properties of Reinforcing Steels

Bar size	Area A_s (mm^2)	Elastic Region: Yield strength, f_y (MPa)	Elastic Region: Yield strain, ε_y (mm/mm)	Elastic Region: Elastic modulus, E_s (MPa)	Start of Strain hardening, ε_{sh}	Strain Hardening Region: Ultimate strength, f_u (MPa)	Strain Hardening Region: Strain at strength f_u, ε_u (mm/mm)
20M	300	421	0.0023	191000	0.017	584	0.203
25M	500	463	0.0025	194000	0.0086	645	0.14

Table 3 - Mechanical properties of GFRP bars in tension

GFRP Lateral Reinforcement	Nominal diameter d_N (mm^2)	Actual diameter, d_A (mm)	Elastic modulus, E_f (MPa)	Ultimate strength, f_u (MPa)	Ultimate strain, ε_{uf}
Ties	12	12.25	54400	841	0.0154
	16	15.75	51700	802	0.0155
Spirals	12	12.25	58500	1050	0.0179

Fig. 3 - Test set-up and instrumentation of a typical specimen

Even though the length of the columns was 1470 mm, the actual shear span of each column was 1840 mm, measured from the column-stub interface to the contraflexure point which is the centerline of the hinge at the column end. This resulted in a shear span to depth ratio of about 6.0 for the square columns and 5.17 mm for the circular columns; the depth was taken as the outer dimension of the columns.

In order to study the effect of axial load on the performance of the specimens, axial load of $0.28P_o$ was applied to two of the specimens and $0.56P_o$ was applied to the other two specimens. P_o is the nominal axial load capacity of column and was calculated to be 4168 kN for the square columns and 4329 kN for the circular columns in accordance with CSA S806-12 standard. The axial load was kept constant at the required level throughout the duration of the test.

Lateral load was applied at the stub approximately 150 mm away from the stub-column interface, so that the most critically loaded region of the column was adjacent to the stub and subjected to combined flexure, shear, and axial loading. In the first cycle, a peak displacement of 0.75Δy was applied to the specimen. This was followed by two cycles each to peak displacements of Δy, 2Δy, 3Δy, and so on till specimen failure. The specimen was considered failed when it was unable to maintain the originally applied axial load due to the damage to the concrete, the rupture or failure of GFRP ties or spirals, buckling of steel longitudinal bars or a combination of all these. Figure 4 provides the transverse displacement history of a typical specimen. The Δy is the theoretical displacement corresponding to the column lateral load capacity on a straight line joining the origin and the point corresponding to 65% the column capacity on the ascending part of the load-deflection curve.

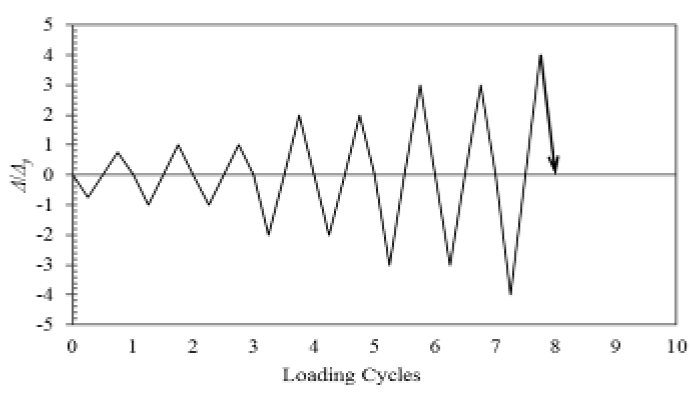


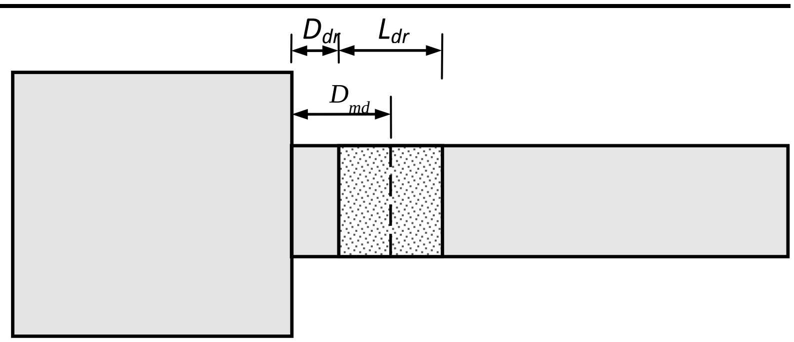
Fig. 4 - Typical Transverse Displacement History

3. Results and Discussion

The first visible signs of distress in all the specimens was the propagation of small cracks, 0.1 to 0.3 mm in width, on the tension face of the columns. These flexural cracks initiated during the second cycle under lower axial load on the columns and during fourth cycle under higher axial load. During the next few cycles, the crack widths slowly increased to about 1 mm and more cracks initiated at a spacing ranging between 100 and 150 mm.

The initiation of spalling of concrete cover for circular columns was observed in the 6th lateral load cycle and major spalling occurred during the 7th cycle. However, in case of square column specimens major spalling occurred in the 6th cycle. Spalling of concrete cover was more rapid and more extensive for square columns than for circular columns irrespective of the level of axial load. The length of the damaged region was found to be larger for square columns than circular columns as well. Table 4 gives the length of the most damaged region and displacements corresponding to the stages at which the cover spalled off for each of the four column specimens. Figure 5 shows a column during testing and typical failure.

Table 4 - Observations of Specimen Damage



Column Shape	Sp. No.	Specimen Name	L_{dr} (mm)	D_{dr} (mm)	D_{md} (mm)	*Small cracks propagating (displ.)*	*Cover spalling both top and bottom (displ.)*
Square	11	TA-P56-S-11	420	90	180	Cycle 4, 6.8 mm	Cycle 6, 10.2 mm
Circular	3	P-55-LS-16-90	320	60	180	Cycle 4, 7 mm	Cycle 7, 10.5 mm
Square	10	TA-P28-S-10	330	70	155	Cycle 2, 4.5 mm	Cycle 6, 13.5 mm
Circular	6	P-28-LS-12-160	250	30	180	Cycle 2, 4.65 mm	Cycle 7, 14 mm

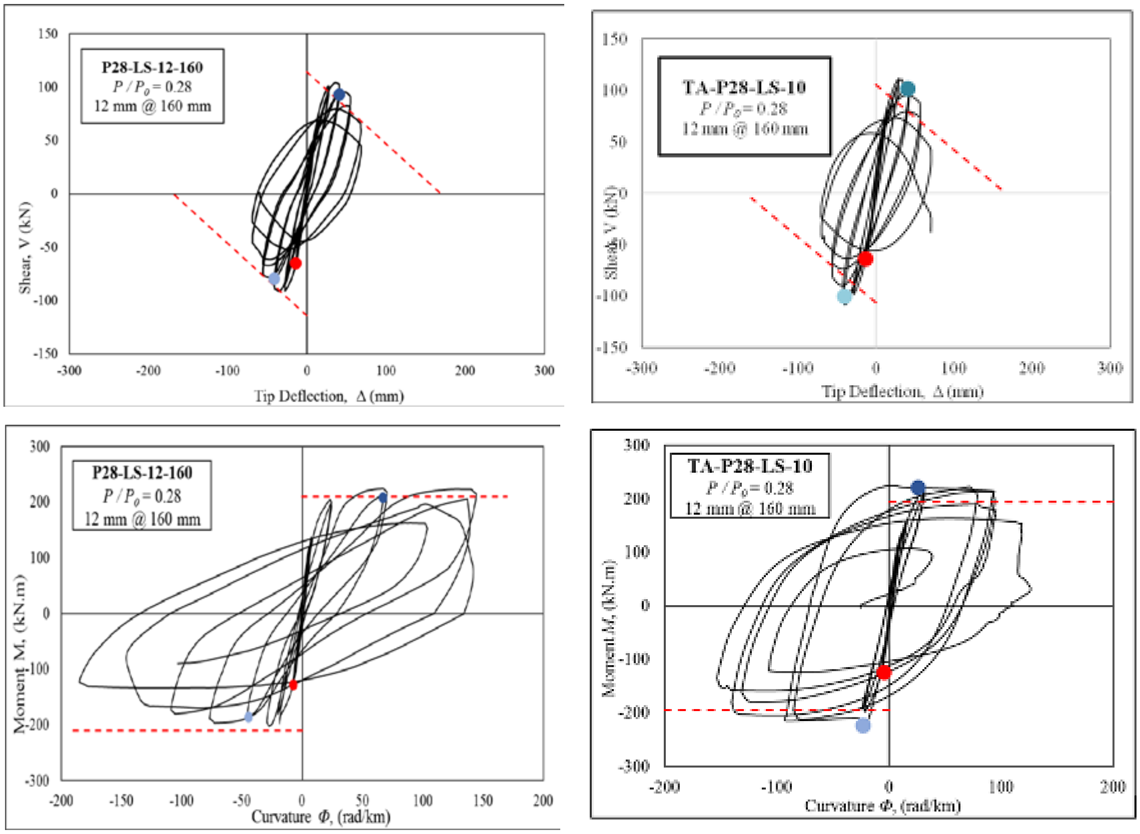
Definitions
L_{dr} = Length of most damaged region;
D_{dr} = Distance from stub to the end of the most damaged region;
D_{md} = Distance from stub to the most damaged section;

Figure 5. A typical column during testing and at failure

The spacing of the lateral reinforcement had a significant effect on the final failure mode of the columns. For Specimen 10 and 6, the square and circular column specimens with lateral reinforcement spacing of 160 mm, the longitudinal steel bars buckled prior to rupture of GFRP spirals or ties. This concept of premature buckling under cyclic loading in square and rectangular columns of longitudinal steel bars has been observed previously [5] and can be prevented if the ratio between tie or spiral spacing and longitudinal bar diameter, s/d_b, is no more than 6. When the spacing was decreased to 90 mm in circular columns (Specimen 3), the confinement was much more efficient. The column provided effective confinement to the core concrete till the rupture of the GFRP spiral. After that, a combination of buckling of the longitudinal bars in compression accompanied by the crushing of the concrete core in the most damaged zone led to the specimen failure. This was because there was no redundancy after the rupture of GFRP spirals and confinement provided to the core concrete vanished as soon as the spiral ruptured. However, in square columns when the spacing was 90 mm (Specimen 11), the failure was much more prolonged occurring slowly over a few cycles. This was likely because there were two ties at each level and partial confinement was still provided by the undamaged ties after one tie failed. Also, rupture of one tie does not result in complete loss of confinement along the length of the column. The final failure of the ties in many cases was due to the unhooking of peripheral ties which slowly occurred over a few cycles.

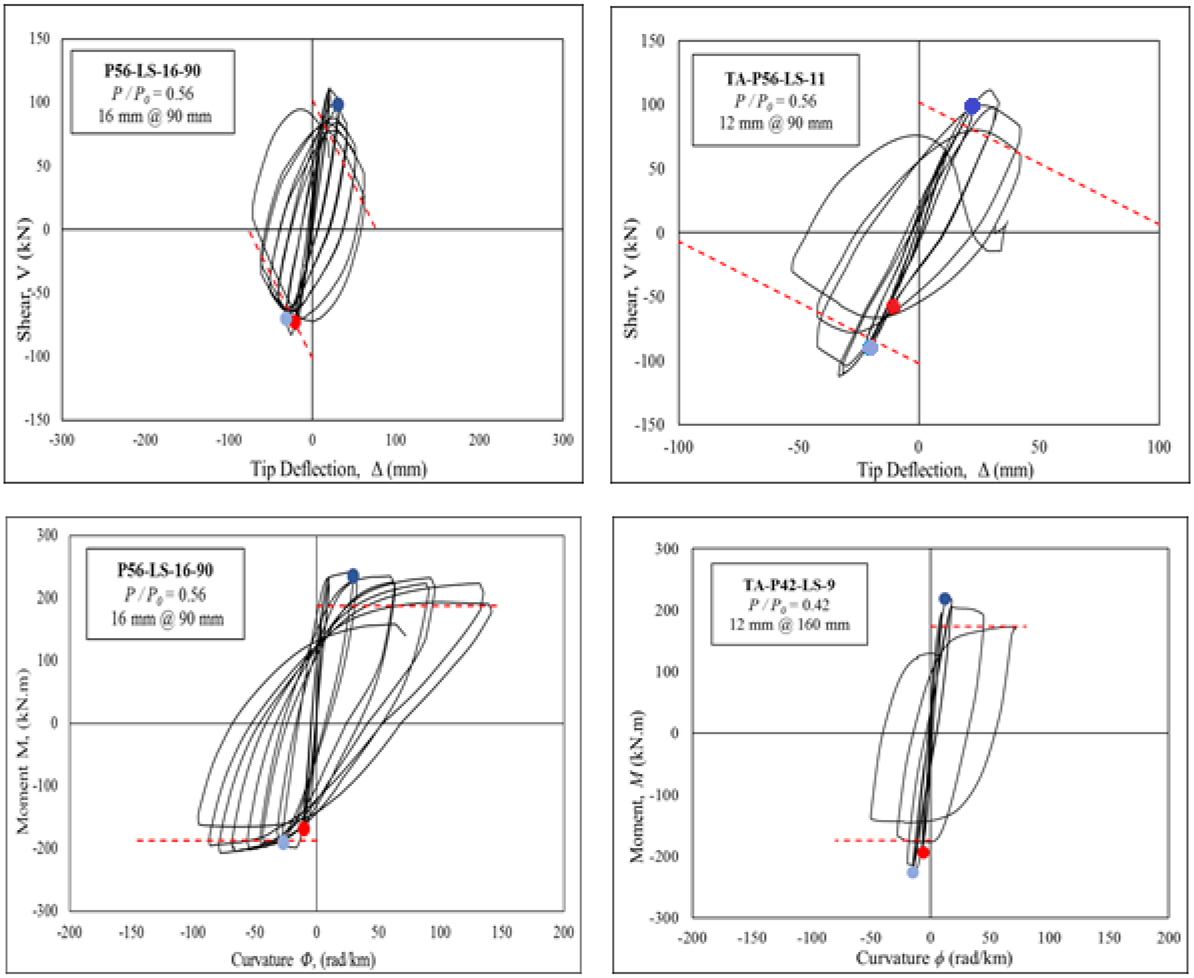
To compare the effectiveness of using GFRP rectilinear ties and GFRP circular spirals for confinement, the results in terms of shear versus tip deflection and moment versus curvature from Specimen 10 (square column) and Specimen 6 (circular column) are shown in Figure 6 and the results from Specimen 11 (square column) and Specimen 3 (circular column) are shown in Figure 7. The red dashed lines in the M-phi diagrams represent the moment capacity of the unconfined column section for which the concrete ultimate strain was taken as 0.0035 as per the CSA A23.3-14 requirements and all the resistance reduction factors were taken as unity. The red dashed lines in the V-delta diagrams represent the nominal shear capacity V_n with the decreasing slope caused by secondary effects.

The columns compared in Figure 6 were subjected to 28% of their axial load capacity that remained constant throughout the test and had the same lateral reinforcement spacing. Results show that both columns behaved in a very similar manner with almost equal ductility parameters. The amount of lateral reinforcement in circular column was a little more than half that of the square column. Almost similar behaviour of both columns indicates the higher efficiency of confinement provided by GFRP circular spiral compared with rectilinear lateral confinement. The two columns compared in Figure 7 were subjected to 56% of their axial load capacity and had almost equal amount of transverse reinforcement and equal spacing The results show that the performance of the circular column was significantly better than that of the square column at the same reinforcement ratio which again confirms the relatively low efficiency of the rectilinear lateral confinement.



(a) (b)

Fig. 6 - Shear vs. Deflection and Moment vs. Curvature relations for columns under lower axial load level (a) Circular Column (Specimen No. 3); and (b) Square Column (Specimen No. 11)



(a) (b)

Fig. 7 - Shear vs. Deflection and Moment vs. Curvature relations of columns under higher axial load level: (a) Circular Column (Specimen No. 11); and (b) Square Column (Specimen No. 3)

Table 5 shows the enhancement of flexural and shear strength of the two specimens. Table 6 provides a summary of the ductility parameter including the displacement ductility factor (μ_Δ), curvature ductility factor (μ_Φ) and the drift ratio (δ) achieved by each specimen. The ductility parameters were determined following the procedure suggested by Sheikh and Khoury [7].

Table 5 – Comparison of flexural strength enhancement in specimens

Shape	No.	Specimen	Failure Mode: Last Cycle	Failure Mode: Max. Disp (mm)	Axial load level P/P_o	V_{max} (kN)	V_n (kN)	$\frac{V_{max}}{V_n}$	M_{max} (kNm)	M_n (kNm)	$\frac{M_{max}}{M_n}$
Square	11	TA-P56-S-11	10	-17	0.56	108	73	1.47	236	172	1.37
Circular	3	P-55-LS-16-90	14	24.5	0.55	97	80	1.21	226	187	1.21
Square	10	TA-P28-S-10	11	-22.5	0.28	110	88	1.25	225	194	1.16
Circular	6	P-28-LS-12-160	12	-28	0.28	98	96	1.02	210	210	1.00

Table 6 – Comparison of ductility factors in specimens

Shape	Specimen	Axial load P/P_o	Displacement ductility factor			Curvature ductility factor			δ (%)
			Δ_y (mm)	Δ_u (mm)	μ_Δ	ϕ_y (rad/km)	ϕ_u (rad/km)	μ_ϕ	
Square	TA-P56-S-11	0.56	14.4	45.1	3.1	4.92	70	14.21	2.5
Circular	P-55-LS-16-90	0.55	13.5	42.8	3.2	5.5	114.2	20.8	2.5
Square	TA-P28-S-10	0.28	13.5	48.5	3.6	6.9	95	12.6	2.9
Circular	P-28-LS-12-160	0.28	18.4	56.8	3.1	12.8	141.6	11.1	3.1

The results in Figures 6 and 7, and from Tables 5 and 6 show that the square and circular columns reinforced longitudinally with steel and laterally with GFRP were able to undergo several load cycles before failure and achieved high levels of deformability at both high and low axial loads. At a low axial load of 0.28Po, when the lateral reinforcement ratio of the square column (Sp 10) was almost double that of the circular column (Sp 6) yet the spacing of the lateral reinforcement was the same, the circular column still underwent one more full displacement cycle before failure. When the axial load was increased to 0.56Po and the spacing was decreased to 90 mm in both columns keeping the reinforcement ratio the same, the behaviour of the circular column (Sp 3) was much better in comparison with the square column (Sp 11). The circular column underwent four more cycles than the square column. However, the flexural enhancement was found to be higher in the square columns; one reason for that was the relatively higher concrete compressive strength.

The drift capacity of the two circular columns compared at failure was 2.5% and 3.1%, and the drift capacity of the equivalent rectangular columns at failure was 2.5% and 2.9%, both of which satisfy the limitations of North American building codes. Despite the fact that the square and circular column at a spacing of 160 mm were under designed as per CSA S806-12 [6] standard, they were both able to achieve a drift capacity of more than 2.5 %, the minimum requirement according to some of the seismic code provisions. The square and circular columns that were subjected to a high axial load of $0.56P_o$, were also able to achieve the minimum drift requirement of 2.5 %. The magnitudes of the ductility parameters presented in Table 5 show that GFRP reinforced columns can be very ductile. Moreover, as the spiral spacing is decreased from 160 mm to 90 mm, despite a significant increase in load, all ductility parameters obtained were satisfactory. It should be noted that columns are less ductile under higher axial loads.

Strength degradation before failure for both square and circular columns was found to be insignificant as a result of the well-confined concrete core. This effective confinement until the end is due to the linear elastic behaviour of GFRP up to an approximate strain of 0.02. The concrete core was confined more effectively and longitudinal bars more effectively supported than they would be by steel spirals since the steel stiffness drops significantly after yield beyond a strain of 0.002.

The preliminary results from this research show that GFRP spirals and GFRP rectilinear ties can be used as primary lateral reinforcement for shear and confinement in concrete columns designed for seismic resistance and in many cases can provide better confinement and better performance of columns than provided by steel.

4. Concluding Remarks

This study investigated the application of corrosion-resistant GFRP ties in square columns and GFRP spirals in circular concrete columns under constant axial load and cyclic lateral displacement excursions simulating earthquake forces.

- Results from this research show that GFRP spirals and rectilinear ties can be used as primary lateral reinforcement for shear and confinement in concrete columns designed for seismic resistance. The GFRP transverse reinforcement performed very well even when the spacing was about 60% of the core dimension.
- It was observed that for circular columns, there was no redundancy after the rupture of GFRP spiral and confinement provided to the core concrete vanished as soon as the spirals ruptured. The loss of confinement in the square columns was found to be not quite as sudden. In most square columns, the failure was more prolonged due to the fact that there were two ties at each level and it took several cycles for the ties to unhook and they did not give way suddenly.
- The behaviour of square columns confined by GFRP ties was found to be very similar to circular columns confined by GFRP spirals in terms of shear versus tip deflection and moment versus curvature when the amount of tie reinforcement ratio was almost twice the spiral reinforcement but the spacing of the lateral reinforcement was the same.
- The strength degradation before failure for both square and circular columns was found to be insignificant due to the well-confined concrete core. The drift capacity of the circular columns at failure was 2.5% and 3.1%, and the drift capacity of the square columns at failure was 2.5% and 2.9%, both of which satisfy the limitations of North American building codes.

5. Acknowledgement

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6. References

[1] Tavassoli, A, Liu, J, Sheikh, SA (2014): Glass Fiber-Reinforced Polymer-Reinforced Circular Columns under Simulated Seismic Loads. *ACI Structural Journal*, V. 112, No. 1, Jan-Feb, 2015, 103-114.

[2] Ali, M and El-Salakawy, E (2015): Seismic Perfomance of GFRP-Reinfoced Concrete Rectangular Columns. *ASCE Journal of Composite Construction*, V.10, No. 1, Oct, 2015.

[3] Alsayed, SH; Al-Salloum, YA; Almusallam, TH and Amjad, M A. Concrete Columns Reinforced by GFRP Rods. *Fourth International Symposium on Fiber-Reinforced Polymer Reinforcement for Reinforced Concrete Structures*, pp 188.

[4] Tavassoli, A (2015): Glass Fiber-Reinforced Polymer-Reinforced Circular Columns under Simulated Seismic Loads. MastersThesis. University of Toronto.

[5] Bayrak, O and Sheikh, SA (2001): Plastic Hinge Analysis. *Journal of Structural Engineering*, ASCE, 127(9), pp. 1092-1100.

[6] Canadian Standards Association, (2012): Design and Construction of Building Structures with Fibre-Reinforced Polymers. Standards Council of Canada, Mar. 2012, Mississauga, ON, Canada.

[7] Sheikh, SA. and Khoury, S.S., (1997), "A Performance-Based Approach for the Design of Confining Steel in Tied Columns", *ACI Structural Journal*, 94(4), pp.421-431.