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Strength and ductility of biaxially loaded high strength RC short square columns wrapped with GFRP jackets

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Abstract. The present study is an experimental investigation into the behaviour of high strength concrete square short columns subjected to biaxial bending moments and strengthened by GFRP laminates. The main objectives of the study are: to evaluate the improvement in the structural performance of HSC short square columns subjected to small biaxial eccentricity when strengthened by externally applied FRP laminates, and to investigate the optimum arrangement and amount of FRP laminates to achieve potential enhancement in structural performance especially ductility. The parameters considered in this study are: number of FRP layers and arrangement of wraps. The load eccentricity is kept corresponding to e/t = 0.125 in two perpendicular directions to the columns principal axes, and the wraps are applied in single or double layers (partial or full wrapping). In the present work, test results of five full scale concrete columns are presented and discussed. The study has shown that FRP wraps can be used successfully to enhance the ductility of HSC columns subjected to biaxial bending by 300%.

Key words: RC columns; biaxial bending; strengthening; rehabilitation; advanced composite materials; fiber reinforced plastics (FRP).

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1. Introduction

Biaxially loaded RC columns present a complex case of stress-strain relationship. Bresler (1960), suggested the load contour procedure as a signified design method for the biaxial loading problem. Gurfinkl (1985), pointed out that, the ultimate moment will be developed at higher strains (than the 0.003 value specified for axial compression). El-Mihilmy (1992) emphasized that the ultimate compressive strain is higher than the value specified in The Egyptian Code of Concrete Structures. He also, concluded that the ultimate capacity of biaxially loaded columns with big eccentricity slightly increases with the presence of compression reinforcement. Hsu *et al.* (1995) showed that the decrease of lateral tie spacing and also the addition of steel fibers do not affect the ultimate load capacity, but they significantly reduce the crack length and plastic hinge length, and also improve the ductility of HSC columns. Torkey and Shaaban (2001) concluded that increasing lateral confinement led to an increase in the ultimate capacity of 22%. They also suggested that tie configuration was more effective than the tie reinforcement ratio on the ductility of the studied HSC columns. They also found that increasing vertical steel percentage led to an improvement of ductility of the tested columns.

Most recently, the Advanced Composite Materials (ACM) were introduced to civil applications. Their favorable weight to strength ratios, their corrosion resistance and their compact sizes are very appealing to structural engineers especially in the strengthening applications. Early researches on this subject focused on concrete cylinders. Demers *et al.* (1994), Miyauchi *et al.*(1997), Watanabe *et al.* (1997), Miramiran and Shahawy (1997), Samaan *et al.* (1998), Rochette and Labossierre (2000), and Fam and Rizkallah (2001) are examples of the studies on the effect of confining of circular concrete cylinders using ACM. The confinement took the shapes of tubes, or wraps. The studies agree upon the following conclusions:

1-Compressive capacity increases up to 300% over the unconfined specimens with the increase in number of confining layers.

2-Ductility increases up to 700% over the unconfined specimens with the increase in number of confining layers.

3-Perfect bond between concrete and ACM is essential to reach the mentioned ultimate capacities and ductilities.

Confining square and rectangular concrete sections with ACM material has been the subject of the studies by Demers *et al.* (1994), Hosotani *et al.* (1997), Rochette and Labossierre (2000), and Suter and Pinzelli (2001). All researches agreed that the confinement of square concrete prisms by wrapping is less efficient than that of circular cylinders. They also pointed out that rounding the concrete prism corners is essential to prevent premature fiber rupture at the corners. Depending on the fiber type, number of layers, concrete grade, and corner rounding radius; the gain in ultimate strength and ductility vary considerably.

Intensive research on the axial load behaviour of full scale or scale model reinforced concrete columns strengthened by FRP wrapping has been carried out during the last decade. Among the pertinent studies on the behaviour of FRP wrapped reinforced concrete columns are; Neale and Labossiere (1997), Matthys *et al.* (1999), Wang and Restrepo (2001), Mahfouz *et al.* (2001), and El-Afandy *et al.* (2002). The mentioned studies show that FRP wrapping is more efficient for circular sections than it is for rectangular sections. They agree on the importance of rounding the square column corners. Economic considerations limit the amount of FRP wrapping used for the strengthening and so one or two layers of FRP wraps were considered in most studies. Ultimate load gain of up to 70% is recorded for circular sections. Considerable ductility improvements are cited for both circular and rectangular sections.

Very few studies are available on the behaviour of FRP wrapped and eccentrically loaded RC columns. Chaallal and Shahawy (2000), conducted an experimental investigation into the behaviour of normal strength rectangular reinforced concrete beam-columns. They found that a strength gain of up to 70% can be achieved using CFRP wrapping. They also reported compressive strain increase of up to 160%. Bencardino *et al.* (2002), concluded that CFRP wrapping is feasible strengthening mechanism for normal strength square RC piers subjected to eccentric axial forces. Ismail and Ghoneim (2002), proposed an analytical model for the confinement effect of FRP wraps on normal strength RC sections subjected to eccentric axial forces.

2. Scope and objectives

The research work reported in this paper has been conducted to achieve the following objectives: evaluating the improvement in the structural performance of HSC short square columns subjected to small biaxial eccentricity when strengthened by externally applied GFRP laminates, and investigating the optimum arrangement and amount of GFRP laminates to achieve potential enhancement in structural performance. The prime interests of the authors were the enhancement of the strength and ductility of the strengthened columns.

An extensive experimental program has been conducted to achieve the target objectives. The experimental program consists of testing five full scale high-strength concrete short square columns strengthened with GFRP laminates under the effect of axial load accompanied with biaxial bending moments. The key variables were: number of layers and arrangement of FRP wraps.

3. Testing program

This section illustrates the test specimen preparation and instrumentation. It also describes the test set up and testing procedures.

3.1 Materials

Local Egyptian concrete constituent materials have been used to manufacture the test specimens including type (I) cement. The proposed mix was designed to develop characteristic cube strength of 80 MPa. 10% of the cement was replaced by Silica fume. Mix proportions are given in Table 1. The specimens' main reinforcement is high strength deformed steel bars (Grade 360) while the steel hoops are mild smooth bars (Grade 240).

Table 1 Design of the concrete mix (Characteristic strength: 80 MPa)

Mix constituents	Cement	Silica fume	Crushed dolomite	Sand	Water	Super plasticizer
Mix proportions (kg/m^3)	475	47.50	1224	612	146.30	18.30

Wrapping material is “SIKA Wrap Hex-430G (VP)” which is E-Glass fabric of 0.17 mm fiber thickness and 2250 N/mm^2 fiber tensile strength. Fiber tensile modulus of elasticity is 70000 MPa. Sikadur-330 adhesive was used to bond external FRP laminates to concrete surface. The flexural modulus, tensile strength, and adhesion strength of resin to concrete are 3800, 30, and 4 MPa, respectively, as reported by SIKA Egypt.

3.2 Specimens details

The column specimens had square cross section of 200×200 mm, 1050 mm clear height and 1850 mm overall height including corbel heads, while the top and bottom corbels were $400 \times 400 \times 400$ mm dimensions. Those corbel heads were introduced to prevent premature failure. Fig. 1 shows the typical concrete dimensions of the column specimen. The longitudinal reinforcement of the column consisted of four steel rebars of 16 mm nominal diameter, and 360 MPa nominal yield strength, arranged symmetrically in the cross section. Steel reinforcement percentage was 2% which is a typical practical value for eccentrically loaded columns. The transverse reinforcement of columns comprised 8 mm nominal diameter peripheral hoops with 240 MPa nominal yield stress. The hoops were spaced 190 mm with a volumetric ratio of 0.424%. To avoid

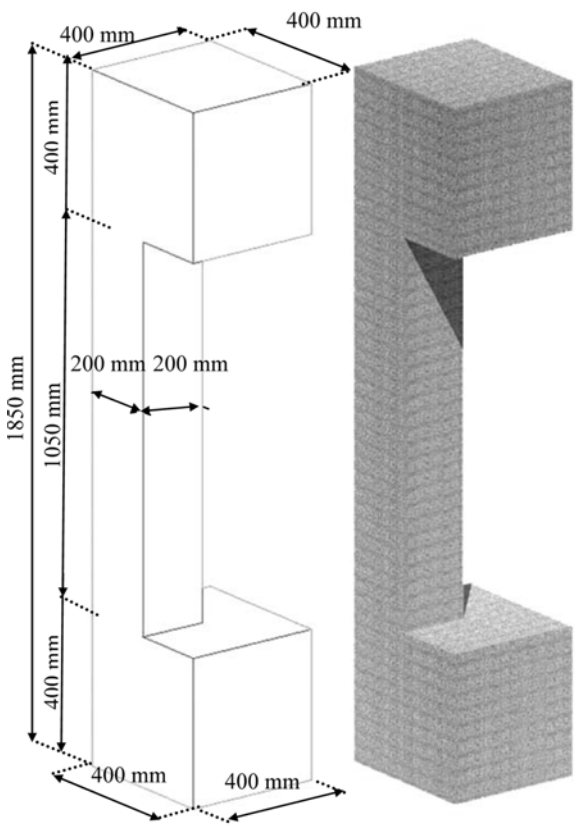

Fig. 1 Concrete dimensions of columns

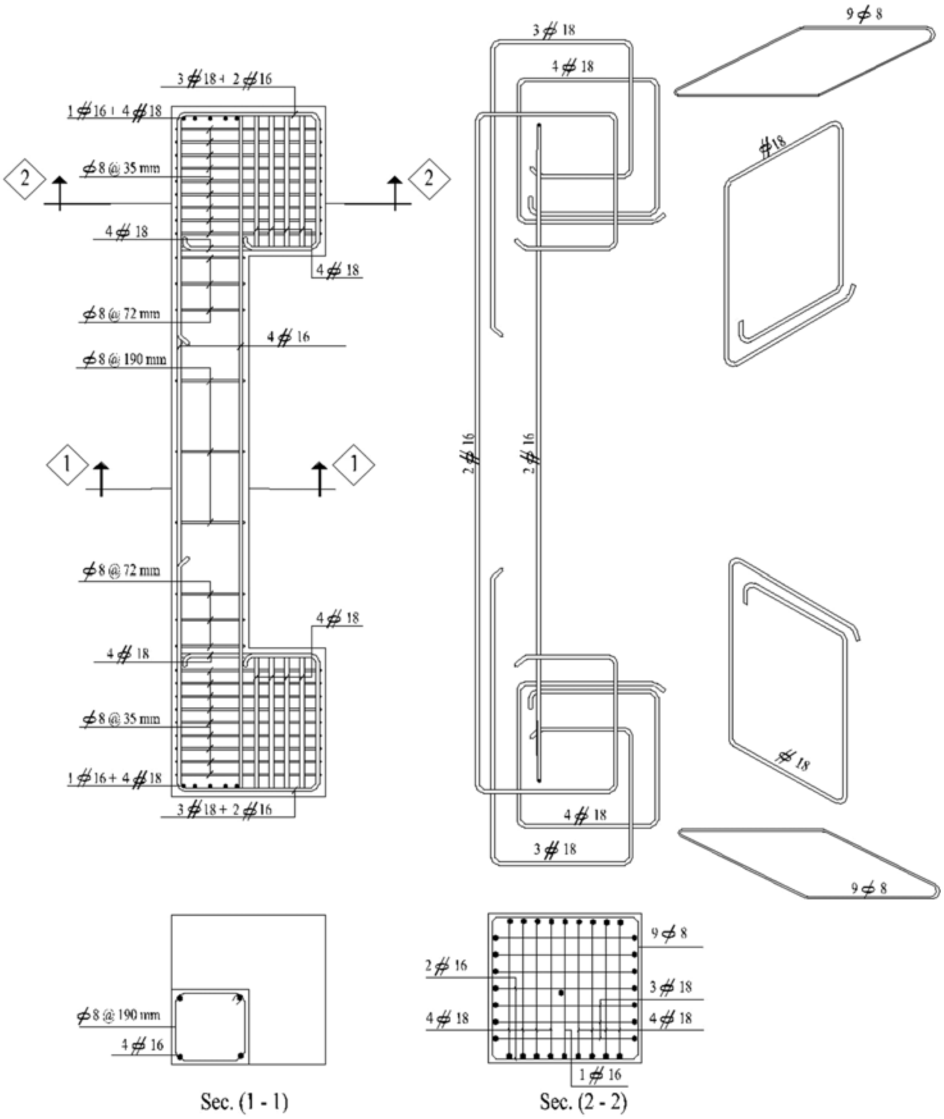

Fig. 2 Reinforcement details

the premature failure, the corbels were heavily reinforced with 18 mm nominal diameter deformed rebars in the shape of vertical reinforcement in both directions of bending moments. At 35 mm spacing, transverse ties of 8 mm diameter were arranged to sustain splitting forces. Another precaution to avoid premature failure, was concentrating the transverse reinforcement of the column at its ends in order to give enough confinement to overcome stress concentration at those ends. Fig. 2 shows detailed dimensions and reinforcement of the test specimens.

FRP laminates were arranged in the transverse direction of the column either in continuous or stripped manner using one layer and two layers of FRP wraps. In this case, FRP transverse wraps acted as an external transverse confining reinforcement for the column through the confinement effect of unidirectional fibers embedded in FRP laminates. In the case of partial wrapping of

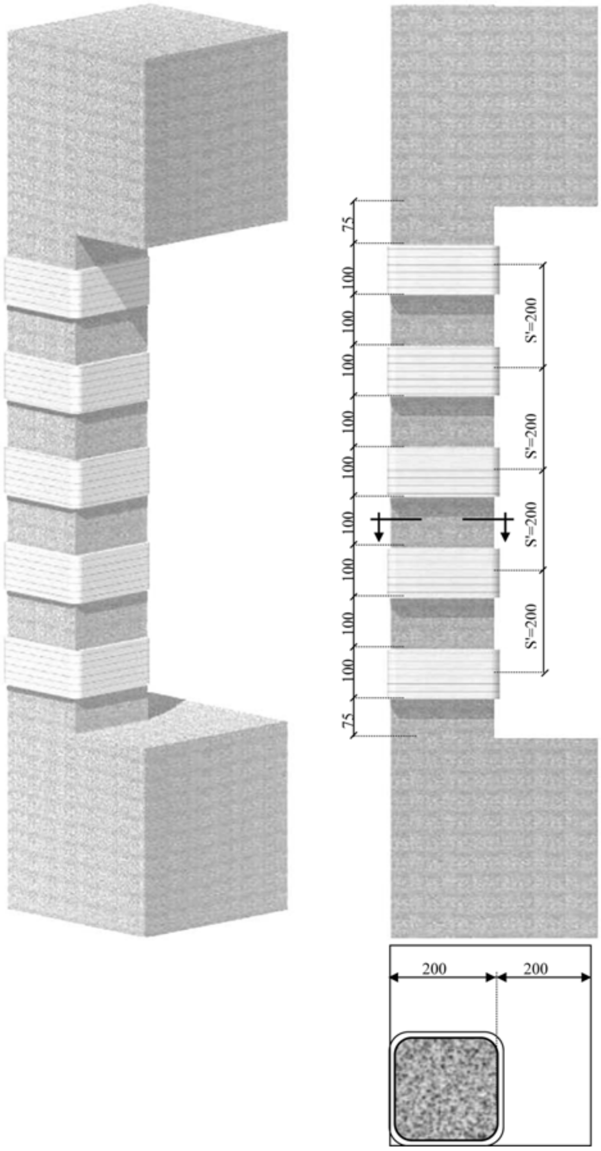

Fig. 3 Dimensions and arrangement of partial wraps

specimens, the FRP wraps were spaced 200 mm center to center. Fig. 3 is an artistic rendering of the illustrated wrapping schemes.

The test specimens were cast and cured for 28 days before applying the FRP layers. To attain utmost benefit from the strengthening layers, the column corners were cast round, with 25 mm radius, instead of sharp edged as specified by FIB 2002 to reduce stress concentrations to the minimum possible. Fig. 4 shows two specimens after strengthening with FRP wraps. Table 2 summarizes the general description of test specimens. For details of the specimen preparations refer to Hassan (2004).

(a) Specimen strengthened by partial FRP wrapping

(b) Specimen strengthened by full FRP wrapping

Fig. 4 Short column specimens strengthened by GFRP wrapping

Table 2 Test specimen general description

No.	Designation	No. of FRP layers	Wrapping scheme	e_x/t	e_y/t
1	BS-00C	N/A	N/A		
2	BS-G1P	1	Partial wrapping		
3	BS-G2P	2		0.125	0.125
4	BS-G1F	1	Full Wrapping		
5	BS-G2F	2			

3.3 Specimen instrumentation

Three Linear Variable Displacement Transducers, LVDTs, of 50 mm stroke, were used to measure; the extreme concrete compressive strain, the extreme tensile longitudinal strain, and the longitudinal concrete strain near the expected neutral axis position of the column's cross section. Fig. 5 shows the distribution of the LVDT's over the column specimen. Each specimen was instrumented with five electrical resistance strain gauges located at the expected locations of extreme strains. One strain gauge is installed to measure the longitudinal strain of each steel rebar at mid height of specimen. One strain gauge is attached to the mid height transverse section tie-which was the expected failure section-at the compression zone apex of the tie. Fig. 6 shows the strain gauge distribution over the column specimen.

Specimens wrapped with FRP laminates were instrumented with five additional strain gauges installed around the mid height section to measure strains in fiber orientation direction as shown in Fig. 7.

Mid height column deflection (measured on the tension side of the column section) was measured during test using an independently supported LVDT. The measured deflection induced additional

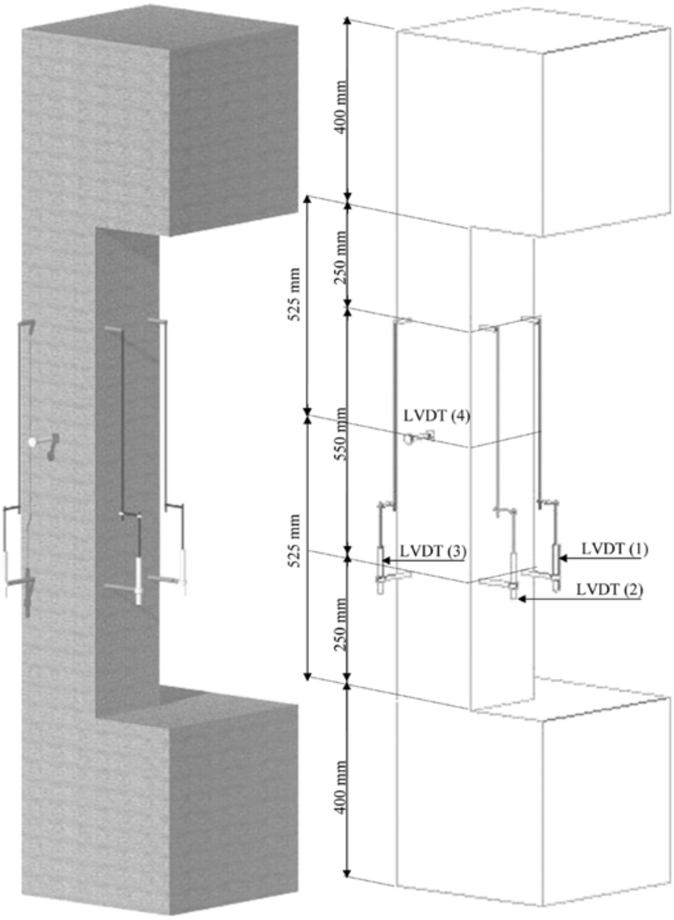

Fig. 5 Instrumentation layout for concrete strains and lateral deformation of the test columns

secondary bending moments; therefore the actual bending moments are calculated taking secondary effects into consideration. The applied load was recorded through the pressure sensor of the testing machine.

3.4 Test procedure

The specimens were tested up to failure using AMSLER compression testing machine of 5000 kN capacity. The testing machine allows a maximum specimen height of seven meters.

The load eccentricity was achieved using bearing plates of 22500 mm^2 bearing area, which is a relatively small area, to simulate a concentrated eccentric column load. The specimen was placed between two of these special bearing plates. In addition, the bearing plates were provided with semispherical masses to allow for specimen rotation. Thus, the aforementioned considerations assured pin-ended specimen conditions. The test specimen was placed between the machine heads. The axis of applied load was centered with machine axis, and the specimen itself was displaced to achieve the specific required eccentricity as shown in Fig. 8. Specimens were loaded using a “load

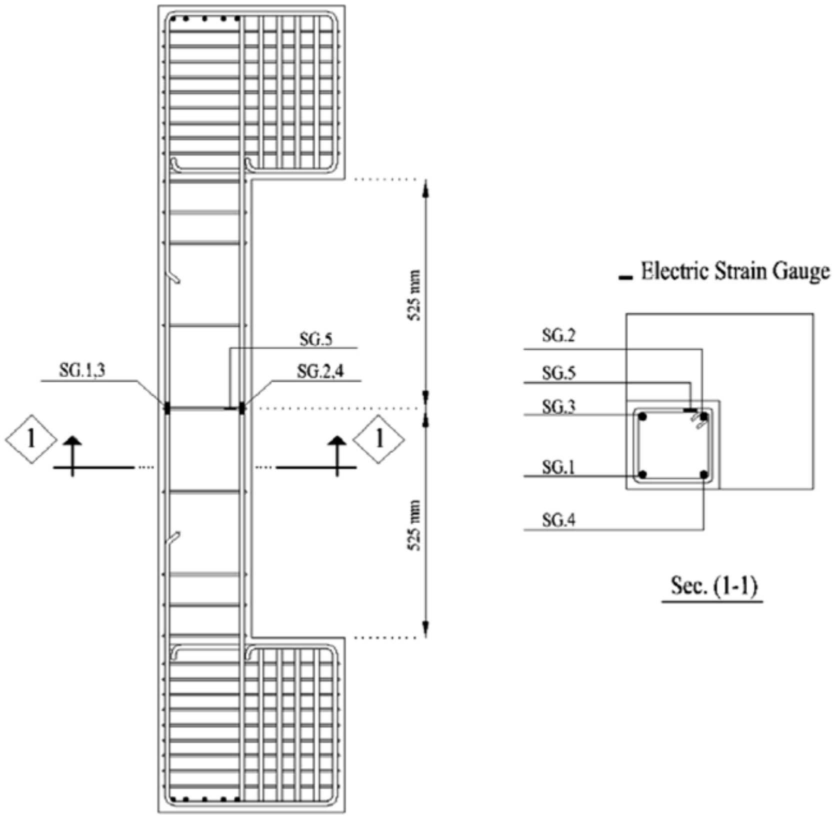

Fig. 6 Strain gauge distribution over the reinforcement

control" system. The load was incrementally applied with an initial rate of 100 kN/min. Before failure (at a load of 70% of the ultimate load), the rate was reduced to one half of the aforementioned loading rates. At every stage of loading, cracks were observed and marked, if any. In addition, the deflections and strains are recorded automatically using a data acquisition system.

4. Mechanical behavior

The analyses and comparisons of test results comprise longitudinal compressive and tensile strains of concrete as well as steel, strain of steel tie and FRP wraps, mid height deflections, and specimens' energy and ductility.

4.1 Ultimate load and stiffness

The Load-Longitudinal concrete compressive strain relationships recorded at the extreme compression apex for the tested specimens are shown in Fig. 9.

Each load-strain curve can be divided into three distinct zones between the origin and the ultimate load. It is clearly noticed from the figure that concrete longitudinal compressive strains of the five

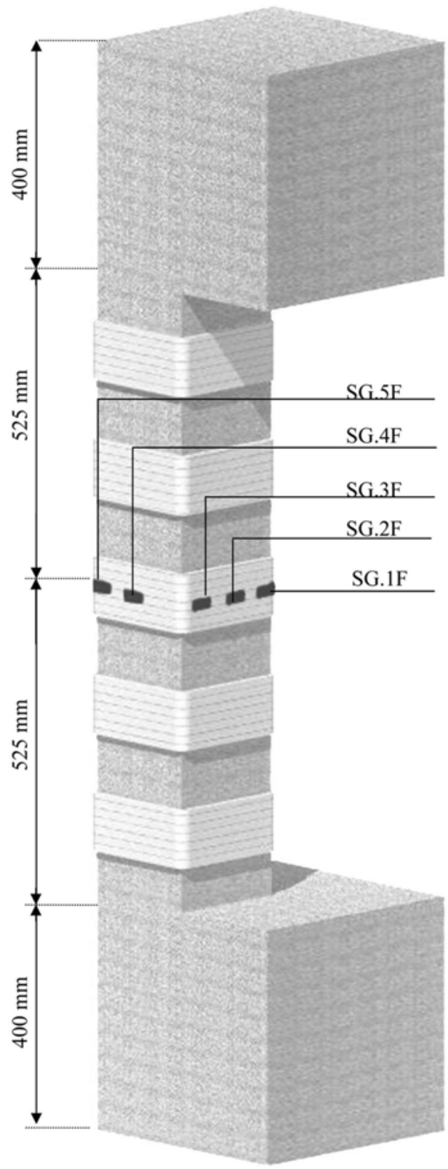

Fig. 7 Arrangement of strain gauges on GFRP wraps

specimens have the same values with the same gradient up to 800 kN, which is 40%-50% of the specimens' ultimate load. This emphasizes that the FRP jacket has not increased the stiffness of the strengthened specimens in the first stage of loading. This is attributed to the insignificant lateral expansion of concrete at this stage of loading. From 800 kN load up to 1400 kN the second stage of loading is recognized. In this "transition" stage, the slopes of the curves start to vary. This is because of the stiffness degradation occurring due to microcracking of concrete and shifting of aggregates. Thus, the FRP wrap in strengthened specimens starts to exert lateral passive pressure on concrete to counteract stiffness degradation. This means that stiffness degradation rate decreases

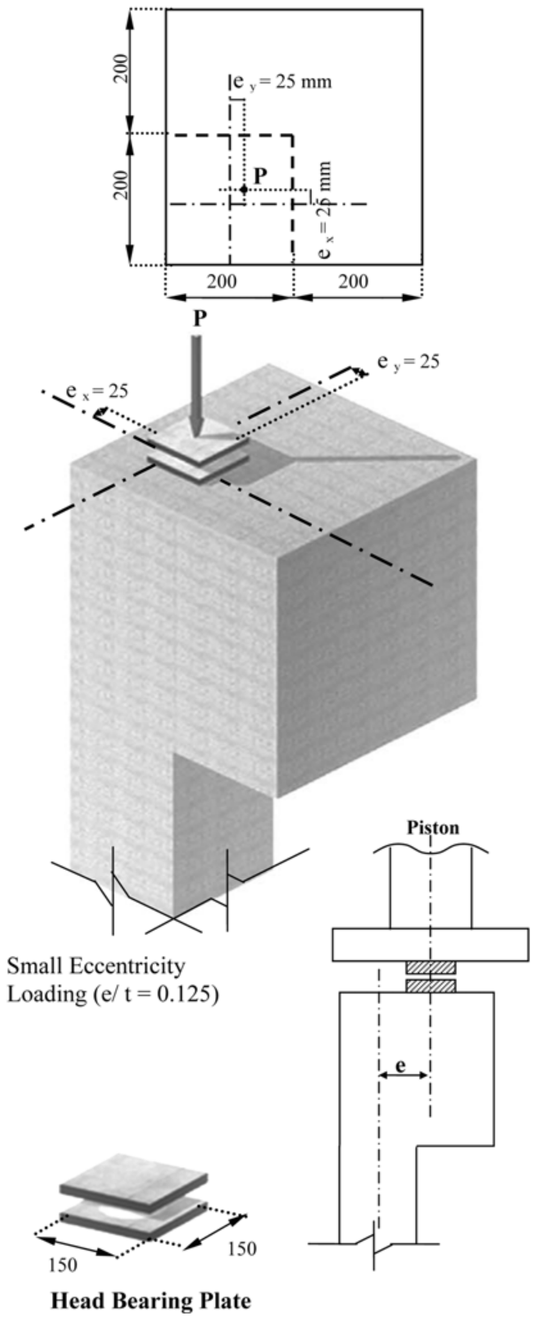

Fig. 8 Biaxial eccentric load application

when strengthening the column with one or two (full or partial) FRP wraps. Finally, a third region was recognized, in which the concrete cracking is significant and FRP wraps are strained to hold it. At this stage, specimens' stiffness was generally stabilized around a constant value. The above behavior agrees with that reported by Miramiran and Shahawy (1997), for circular columns, and

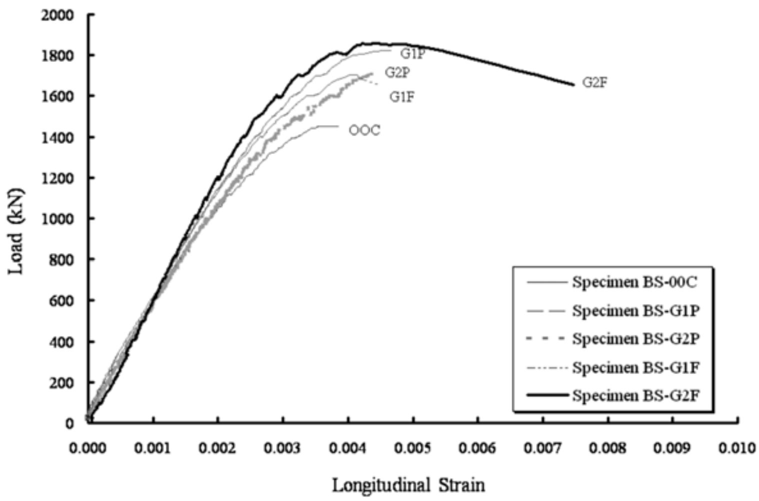

Fig. 9 Load-longitudinal max. concrete compressive strain

Table 3 Ultimate load and ultimate moment of test specimens

No.	Designation	P_u (kN)	% P_u/P_c	M_u (kN·m)	% M_u/M_c
1	BS-00C	1452	100	43.84	100
2	BS-G1P	1823	126	45.58	104
3	BS-G2P	1707	118	51.93	118
4	BS-G1F	1705	117	54.24	124
5	BS-G2F	1855	128	61.92	141

Miyauchi *et al.* (1997), for cylinders strengthened using FRP wraps.

Table 3 summarizes the test results of all specimens. Comparing the values recorded in Table 3, it is evident that the ultimate loads of strengthened specimens are higher than that of control specimen. The increase in ultimate load ranges from 18% to 26%. The moment enhancement ranges from 19% to 41%. It is noteworthy to point out that, the secondary moment due to P-Δ effect increased by 1.5 and 2 times for specimens BS-G1F and BS-G2F compared to the control specimen, which indicates unfavorable probable conversion of the short column into a slender column. This result agrees with ECFRP (2004) recommendations for axial columns. Fig. 10 shows the failure section of specimen BS-G2F.

It is clear from the ultimate load values, given in Table 3, that increasing number of FRP wrapping layers in partially wrapped specimens reduces enhancement of strength in biaxially loaded HSC columns. This behaviour may be explained by the observed failure sequence of the different specimens. Failure of specimen BS-G2P, that had two FRP layers, was initiated at the concrete between two successive mid height wraps by vertical cracking at the compression zone followed by rupture of FRP wraps at the extreme compression apex. Fig. 11 shows specimen BS-G2P at failure. On the other hand, the failure of specimen BS-G1P, that had one FRP wrap, was initiated by rupture of FRP wrap at the extreme compression apex without vertical concrete cracks between mid height wraps. This is explained by the higher stiffness of double FRP wraps compared to single wrap,

Fig. 10 Failure of specimen BS-G2F

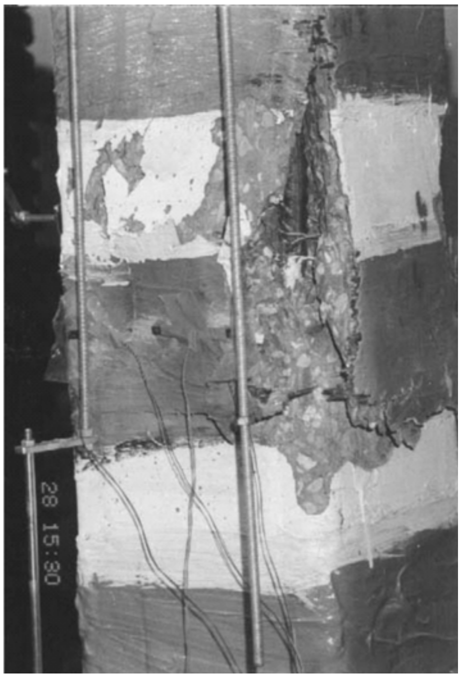

Fig. 11 Failure of specimen BS-G2P

which was enough to resist the confined concrete expansion in specimen BS-G2P mid height section and hence the failure was initiated by concrete expansion at the weak section between two successive wraps and then extended to the FRP wraps. Moreover, the stress concentration at the extreme compression apex of FRP mid height wrap accelerated the failure and did not allow the tensile strength of FRP wrap to be fully utilized. This result is thought to be a characteristic of biaxially loaded specimens since stress concentration at the extreme compression apex is more pronounced for biaxially loaded columns than it is for axially or uniaxially loaded columns. This behaviour of FRP wrapped biaxial HSC columns is different from that of partially wrapped rectangular NSC columns whose strengths are not significantly enhanced by FRP wrapping (El-Afandy 2002). There is another test related reason for the observed results; the cube compressive strength of specimen BS-G2P is lower than that of specimen BS-G1P by about 9%, which helped to have concrete compression failure between mid height wraps of specimen BS-G2P. Thus, in conclusion, increasing number of layers of FRP transverse wraps has a minor effect in enhancing strength of column due to stress concentration at corners which restrict the full utilization of FRP wrap strength.

4.2 Column ductility

The maximum longitudinal compressive concrete strains of the tested specimens are given in Table 4. On the ascending part of the load-strain curves it is noticeable that; at any specific load level, the longitudinal concrete compressive strain of control specimen was always higher than those of strengthened specimens. This indicates the delay in the compression strain progress the strengthened specimens had acquired which, led finally to ultimate load and moment enhancement.

Table 4 Ductility of different test specimens

No.	Designation	ε_{cc}	ε_{cu}	$\varepsilon_{cu}/\varepsilon_{cc}$	Area (A) kN·m/m	A_1 kN·m/m	A_2 kN·m/m	Ductility factor (A_2/A_1)
1	BS-00C	0.39	0.39	1.00	3.86	2.01	1.85	0.92
2	BS-G1P	0.47	0.61	1.30	7.96	2.57	5.39	2.10
3	BS-G2P	0.34	0.43	1.26	6.47	2.76	3.71	1.34
4	BS-G1F	0.41	0.45	1.10	4.97	2.01	2.93	1.46
5	BS-G2F	0.48	0.75	1.56	10.56	2.57	7.99	3.11

A = total area under load-concrete compressive strain

It is noticeable, in turn, that drastic increase in the ultimate longitudinal compressive concrete strain is achieved by using FRP wrapping. The measured ultimate strains for the wrapped specimens are as high as 200% of the control specimen. The ratio of the failure (ultimate) longitudinal compressive concrete strain to that recorded at the maximum load ($\varepsilon_{cu}/\varepsilon_{cc}$) is a measure of the column's ductility. The ratio of ($\varepsilon_{cu}/\varepsilon_{cc}$) for control specimen was 1.00. On the other hand, this ratio ranges between 1.25 and 1.56 for wrapped specimens as shown in Table 4. This reflects the improvement in ductility gained after strengthening. Table 4 gives the area under the load-concrete compressive strain curve. Another expression of the ductility is given by the ductility factor which is defined as the ratio (A_1/A_2). The first area, A_1 is measured from zero load up to the maximum elastic load. The second area, A_2 is calculated from the elastic load down to 80% of the maximum load on the descending branch of the curve. The ductility factor provides an indication of the inelastic capacity of the columns (El-Afandy 2002).

Ductility factors of all tested specimens are given in Table 4. Fully wrapped specimen with two FRP layers has 300% ductility enhancement over the control specimen. Based on the results shown in Table 4, it is concluded that increasing number of full wrapping layers results in significant ductility enhancement for short square biaxially loaded HSC columns.

4.3 Strain of tensile apex steel

The Load-Longitudinal Rebar Tensile strain relationship is shown in Fig. 12. The figure shows that the strengthened specimens had achieved slightly higher loads at any certain tensile strain level compared to control specimen during most of loading stages. This may be explained by the fact that the confined cross sectional area of strengthened specimens is larger than that of control specimen due to the confinement action of transverse FRP wrapping. This caused the longitudinal compression strain to increase and the neutral axis to move towards the extreme tension apex, which in turn caused the longitudinal rebar tensile strain in strengthened specimens to be smaller than that of control specimen. The only exception is Specimen BS-G2F which demonstrated significantly higher loads at any strain value compared with all other specimens. Compression strains of specimen BS-G2F are 227% of that of specimen BS-G1F and that is the reason for the very low tensile strain value recorded for specimen BS-G2F. At the maximum load of the control specimen, the recorded tensile steel strains of the strengthened specimens BS-G1P and BS-G2P were smaller than that of the control specimen by about 29% and 14%, respectively due to the same aforementioned reason.

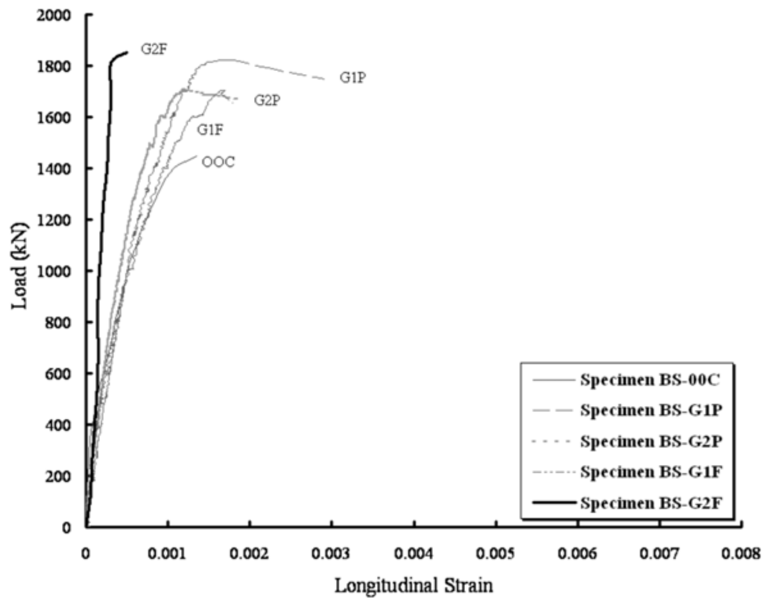

Fig. 12 Load-longitudinal maximum steel tensile strain

4.4 Strain of steel hoops

Fig. 13 shows the effect of number of FRP transverse layers on the strain of mid column height transverse tie at the extreme compression apex. The figure shows that the transverse tie strains of control specimen are always larger than those of strengthened specimen at any load level. It is also apparent that the transverse tie strain of specimen BS-G1P is larger than that of specimen BS-G2P for any load level. This is attributed to the contribution of FRP wraps in confining the concrete

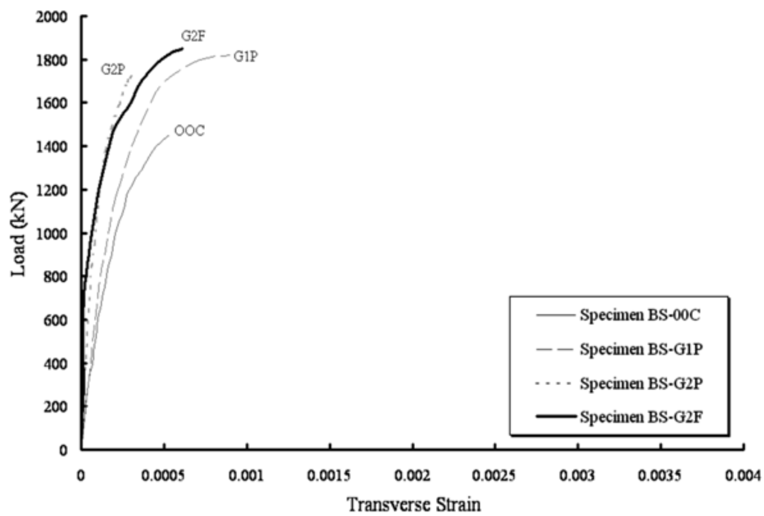

Fig. 13 Load-maximum tie tensile strain

core. Thus, the larger the number of FRP layers, the higher the confinement level and the lower the transverse tie strains. However, when comparing the two specimens BS-G2P and BS-G2F, it is noticed that the transverse strains of strengthened specimens were of the same values up to failure. This may be attributed to the presence of transverse tie at mid column height in both specimens centered with the mid column height FRP wrap.

4.5 Strain of FRP laminates

Fig. 14 shows the relationship between load and transverse strain measured on FRP laminate at specimens mid height extreme compression apex. It is concluded from Fig. 14 that, specimens with two FRP wraps experience significantly higher FRP strains (up to 250% over specimens with single wraps). This is explained, as mentioned in the previous section, by the higher confinement produced by the double FRP wraps. This higher confinement allowed the confined concrete to produce higher active confined pressure on FRP wraps which, in turn, introduced higher passive pressure on concrete with larger transverse strains.

It is worthy to say that the load-transverse strain relationship for FRP laminate is linear only up to a low load level (20% of ultimate load), then it shows a perfectly nonlinear profile in spite of the bilinear behavior of individual fibers and FRP wrap in direct tension test. This is due to the fact that the transverse expansion of concrete is of non-linear nature, which leads the perfectly bonded FRP, to behave non-linearly. This result agrees with that reported by Hosotani *et al*. (1997).

Fig. 14 shows that transverse strain values for specimens BS-G2P and BS-G2F are almost the same up to 1200 kN load. Then, transverse FRP strain of specimen BS-G2P exceeded that of specimen BS-G2F up to failure. This may be explained by the vertical cracks observed between the mid height successive wraps in specimen BS-G2P which weakened the plastic hinge region and added more stresses and more confining effort to the mid height wrap. Thus, higher transverse strains were encountered by the mid height wrap of specimen BS-G2P accelerating its failure by rupture at corner of compression zone. Failure FRP transverse strains of specimens BS-G2P and

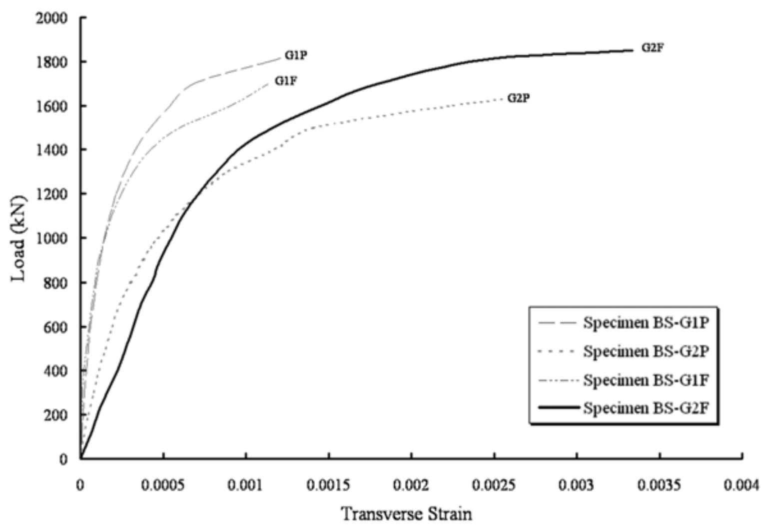

Fig. 14 Load-maximum GFRP tensile strain

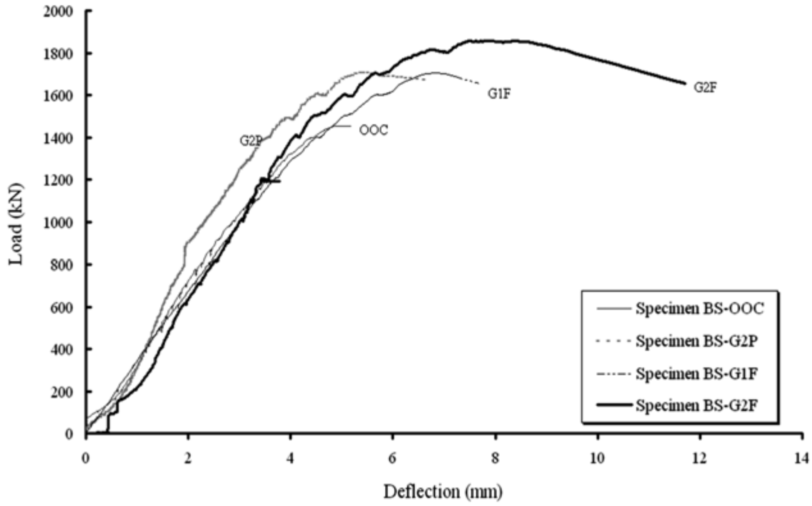

Fig. 15 Load-maximum lateral column deflection

BS-G2F were 0.32% and 0.34%, which seems to be a threshold value of FRP strain in specimens strengthened with two layers of FRP under biaxial bending.

4.6 Mid-height lateral deflection

Fig. 15 shows the load-mid column height lateral deflection relationship. The load-deflection relationship of specimen BS-G1P was not recorded because of LVDT failure during the test. The figure shows that the behavior and lateral deflection values of specimens were almost the same up to failure, with slight reduction in lateral deflection of strengthened specimens up to 1400 kN. Specimens with full wrapping experience larger lateral deformations in the late loading stages due to the eminent cracking in the confined concrete.

5. Conclusions

Based on the results of the work presented in the previous sections and within the range of investigated parameters, the main conclusions of this investigation are summarized hereafter:

Ductility Enhancement of biaxially loaded short square HSC columns is achieved by the use of two full GFRP wraps. Increasing transverse FRP wrap width up to full wrapping is more efficient in increasing column ductility than increasing the number of partial transverse wraps. Full wrapping with two GFRP layers is the most appropriate method to enhance column ductility as its ductility enhancement reached 300%.

When seeking Strength Enhancement of biaxially loaded short square HSC columns:

1. Partial wrapping with single layer of GFRP laminates is more efficient to enhance load and flexural capacities compared with double layer GFRP, as their strength enhancement ratios were 26% and 18%, respectively. This is due to the higher stiffness of double FRP wraps, which restricted concrete dilation at the strengthened sections and allowed the lateral expansion of

concrete to be concentrated at the unconfined section between the mid height successive wraps.

2. Increasing the width of transverse FRP laminates up to full wrapping does not result in strength enhancement in specimens wrapped with one FRP layer. On the other hand, increasing their width in specimens wrapped with two FRP layers significantly enhances strength of the column up to 28% over unwrapped specimens. Thus, strength enhancement is proportional to the number of FRP layers in fully wrapped specimens, while it is inversely proportional to the number of FRP layers in partially wrapped specimens.

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