Brittle shear failure prevention of a non-ductile RC column using glass fiber reinforced polymer (GFRP)

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Abstract

The objective of the research is to improve the seismic behaviour of a non-ductile column failing in a brittle shear using glass fiber-reinforced polymer (GFRP). Two non-ductile columns with similar size, concrete and cross-sectional properties were tested under lateral cyclic loading. The first column (G1) possesses longitudinal reinforcement ratio of 2.76% with 0.25x0.35m in size and 1.1 m in height. The minimum requirement for the amount of stirrup was used in these columns. For the second column (G2), the column was wrapped by 3-layer GFRP 500mm from the base. When column G1 was subjected to lateral cyclic loading, the column failed in shear at the lateral drift of 3.5%. For column G2, the GFRP could increase column shear capacity and also confine the concrete in the plastic hinge region. Consequently, the column maximum displacement was dramatically increased up to 12% drift and the column eventually failed in flexure mode associated with longitudinal reinforcement yielding, compression bar buckling and crushing of concrete inside the GFRP. It is evident that the GFRP could effectively improve the unfavourable column shear failure of non-ductile columns by increasing column shear strength and concrete ductility in the plastic hinge region.

1. Introduction

Typical existing reinforced concrete columns in area of low to moderate seismicity, especially in developing countries are not designed for earthquake. Consequently, these columns are vulnerable to damage or even collapse in the event of a strong earthquake. These columns are known to be non-ductile with generally poor detailing. These
columns, particularly those with a low shear span-depth ratio, are susceptible to brittle failure associated with shear. For such columns shear mode of failure often prevails with small drift capacity [1-4]. Under moderate to high axial load ratios, longitudinal bars tend to buckle, with the consequence of abrupt shear failure as reported by Sezen and Moehle [3] and Wibowo et al. [4]. Wibowo et al. [4] reported that for columns with light axial load, shear failure would be triggered due to apparent strength degradation after development of the flexural strength whereas columns with high axial load would suffer abrupt shear compression failure. To prevent brittle shear failure, there are many available methods that could be used to strengthen short columns such as using reinforced concrete jacketing, steel jacketing [5] or using carbon fiber reinforced polymer (CFRP) [6]. Although the use of reinforced concrete jacketing is an effective method to enhance the column shear strength, the column size has to be considerably increased at least 200 – 250 mm due to considerable thickness of concrete cover is required when pouring concrete surrounding the existing column. The size of column may slightly increase by 40 – 50 mm when using steel jacketing technique but such material is very heavy and installation required a good quality welding to assure the effectiveness of this method. The use of CFRP is also well known to be an effective shear strengthening and not increasing column size after strengthening but the cost of retrofitting using CFRP is extremely high and many people could not afford to use this strengthening technique. The use of lower cost fiber such as glass fiber is an option but research on using GFRP is still limited [7-8]. Although the durability of glass fiber is less than carbon fiber (about 15-20% according to ACI440.2R-08) the cost of glass fiber is 4 to 5 time lower than carbon fiber based on similar tensile strength. Therefore, the objective of the research is to investigate the effectiveness of glass fiber reinforced polymer (GFRP) in improving the seismic behavior of a non-ductile column failing in a brittle shear failure mode. The non-ductile short column (column G1) that has been tested by Rodsin et al. [9] in 2010 was served as a controlled specimen. The short columns with identical cross-sectional property but one column is strengthen using GFRP will be tested under cyclic loading. The test emphasizes on investigation the effectiveness of GFRP in preventing the brittle shear failure in a non-ductile short column.

2. Test program

Two reinforced concrete column specimens were examined in this study. Column G1 is the control specimen and another is wrapped by a 3-layer of GFRP 500 mm from the base (G2). Both of them have the same dimension – 0.25 m wide, 0.35 m deep and 1.55 m high as shown in Fig. 1. The lateral load was applied at 1.10 m distance measured from the base, Therefore the column shear span for this test is 1.10 m. The property of GFRP used in this study is shown in Table 1.

| Product     | Fabric           | Warp          | Tensile Strength | Fiber Stiffness | Elongation at Break | Thickness |
|-------------|------------------|---------------|------------------|-----------------|--------------------|-----------|
| NIBICHI-CFU92 | Uni-directional | E-Glass Fiber | > 2300 MPa       | > 76 GPa        | > 3.1%             | 0.350 mm  |

The columns were reinforced longitudinally with twelve 16-mm diameter deformed bars having yield strength of 507 MPa and transversely with 6 mm diameter round bars having yield strength of 396 MPa as seen in Fig. 1(a). The ties of 6 mm round bar spaced at 0.20 m intervals were used. The longitudinal reinforcement ratio was 2.76% by using 12-Φ16mm deformed bars aligned as shown in Fig. 1(a). The compressive strength of both columns is in the same order. The column property is shown in Table 2.

The experimental setup was shown in Fig. 2. A hinge connector was placed on the top of column. It was connected to an axial load jack which was used to constantly create a vertical load of 400 kN on the column. The actuator was lateral displacement controlled to push and pull the column to an assigned drift ratio (the ratio of the drift to column height where a lateral is applied). Five pairs of displacement transducers were installed to monitor the deformation of the column. Also, a series of strain gauges were attached to reinforcing steels either longitudinal or transverse steels in the plastic hinge region to monitor their responses during the loading test. In addition, to monitor strain transferred to GFRP, strain gauges were attached along the high of GFRP. However, the result from all sensors will not be discussed in this paper due to the page limit but will be reported in the future paper. To perform the test, the lateral displacement was applied cyclically and the load resistance was recorded. Then, the corresponding load-displacement curves were plotted to demonstrate hysteresis loops.
Table 2. Property of column specimens

| Specimen | Concrete Strength (MPa) | Long. Reinf. | Percent of Long. Reinf. | Long. Reinf. Yield Str. (MPa) | Trans. Reinf. | Trans. Reinf. Yield Str. (MPa) | GFRP Wrap |
|----------|-------------------------|--------------|-------------------------|------------------------------|---------------|-----------------------------|----------|
| G1       | 28.5                    | 12-DB16      | 2.76 %                  | 507.0                        | 2-RB6@0.2 m.  | 396.0                       | N.A.     |
| G2       | 26.5                    | 12-DB16      | 2.76 %                  | 507.0                        | 2-RB6@0.2 m.  | 396.0                       | 3-layer  |

Fig. 1. Column specimen (a) detailing of test specimens; (b) specimen wrapped by GFRP (dimension in cm.).

Fig. 2. Installation of a test specimen in test frame.

3-layer Glass Fiber Reinforced Polymer (GFRP)
3. Test results and discussion

3.1 Crack pattern and failure mode

The crack pattern of the column G1 is shown in Fig. 3. At a very low drift the crack was dominated by flexure as observed by the cracks perpendicular to the column axis as shown in Fig. 3(a). When the column subjected to a larger drift, a number of crack increases and the cracks have enlarged and inclined (about 45 degree) when they propagate closer to the center of the column as shown in Fig. 3(b). The inclined cracks may imply that shear force has dominated the column behavior. At a drift of 2%, the dominant crack line was observed and clearly observed at around 2.5% drift as shown in Fig. 3(c). This dominant crack is the largest inclined shear crack compared to others. When the dominant crack formed, the lateral force has significantly dropped and the cyclic behavior of the column was controlled by this crack. The significant rate of lateral strength deterioration is due to the reduction of force transferred along the shear crack. The cyclic load applied to the column caused this crack to open up, close and slip back and forth. The failure shear plane cut through the concrete core at roughly 45º in between the ties. Thus, practically no shear resistance was provided by the transverse reinforcement at the failed section. Eventually, the concrete surrounding the crack has been crushing and spalling followed by bar buckling. Consequently, the column could not be able to sustain the gravity load and collapse at a very low drift of 3.5% as shown in Fig. 3(d). The failure is triggered by shear followed by concrete spalling and bar buckling. This abrupt failure type is not a favorable type of failure when the columns are subjected to earthquake load.

For the column G2, the GFRP was wrapped 3 rounds 500 mm from the base to prevent concrete spalling and to increase column shear strength. The required amount of GFRP was calculated in accordance with ACI 440R-08 but the reduction factors for design were neglected and the strain limit of GFRP was used up to 0.004 resulting in the stress level in GFRP about 300 MPa based on the datasheet in Table 1. From the calculation, the use of GFRP could enhance the column shear strength about 90 kN. The column shear strength contributed from concrete and stirrup is 290 kN. After strengthening the shear strength has increased up to 380 kN. However, in reality, the column shear strength at the stage near failure is much lower than 290 kN since the concrete has been crushing and could not much contribute much shear strength. For shear strength contributed by GFRP, the strain limit of GFRP 0.004 recommended by the code is overly conservative. Based on datasheet in Table 1, the strain limit could increase very high up to 0.03 and consequently, the shear strength contributed by GFRP might be very high up to 724.5 kN.

Fig. 3. Crack pattern of the column G1 at a drift of (a) 0.5%; (b) 1.0%; (c) 2.5% (d) 3.5% (failure)
When the column G2 was subjected to lateral cyclic load at 1% drift as shown in Fig. 4, the crack could be only observe above the GFRP. The crack is very small and perpendicular to the column axis showing that the column behavior is dominated by flexure. When the larger drift applied, at 4% drift, the existing cracks have extended and inclined as shown in Fig. 4(b). The cracks seemed to stop progressing and were not propagated when the larger drift applied. Even a very high drift up to 8%, no further significant damage was observed particularly in the plastic hinge area which was strengthen by GFRP as shown in Fig. 4(c). It was evident that the GFRP could stop the shear crack by dramatically increase shear strength and effectively confined concrete in the plastic hinge region. After 8% drift, the GFRP near the column base was slightly swollen. At 12% drift, the GFRP near the base was evidently swollen due to crushing of the concrete at the base and at this drift the lateral load resistant of the column has been significantly dropped. However, the GFRP could still be able to hold the crushed concrete inside and preventing the progressive buckling of longitudinal steel in compression. Even a very large drift, the column G2 could still carry gravity load and no evidence of failure could be observed after the test. The test was terminated due to the actuator reached the stroke limit. The failure mode of the column G2 is clearly considered to be flexure mode.

3.2 Cyclic behavior of column G1 and G2

The relationship between lateral load and displacement relationship of the columns G1 and G2 was shown in Fig. 5. After strengthening, the column G2 could sustain lateral force slightly larger than that in G1. In column G1, the lateral resistance has abruptly dropped soon after the peak load has been reached. The significant loss of lateral strength is due to opening of the dominant shear crack that could not properly transferred shear force along the open crack. The column shear failure associated with shear crack slippage and loss of axial load carrying capacity eventually occurs at 3.5% drift. In contrast, the column G2 could sustain lateral load after the peak lateral force and the hysteresis loop of the column was very stable compared to that of G1. The lateral resistance gradually reduced when the drift increased and even at a very high drift (12% drift) the column still has a considerable amount of lateral resistance force.

The summary of the test results was shown in Table 3. The lateral load resistance of the column G1 and G2 was 162.8 kN and 180.9 kN respectively. The increase in flexural strength was due to confining effect provided by GFRP. When the concrete is confined, the compressive strength as well as compressive strain at failure could be increased depending on amount of confining force. In this study, the compressive strength is expected to be increased by 20 – 30% but the compressive strain at failure may increase more than 10 times compared to the unconfined concrete. The increase in compressive strength in the plastic hinge could slightly increase flexural strength of the column as shown by the marginal increase of the lateral load resistance. In contrast, the confining
force provided by GFRP could significantly increase the concrete ductility and result in the increase in the maximum drift of the column before collapse. The maximum drift was significantly increased from 3.5% to 12% when the column was strengthening by GFRP. It is evident that the GFRP could effectively improve the unfavourable column shear failure of non-ductile columns by increasing column shear strength and concrete ductility in the plastic hinge region.

![Fig. 5. Lateral load and displacement relationship of the columns G1 and G2.](image)

| Column Specimen | Maximum lateral load (kN) | Displacement at maximum load (mm) | Lateral load at yield (kN) | Displacement at 80% of max. load (mm) | Displacement at collapse (mm) | Column ductility |
|-----------------|--------------------------|----------------------------------|---------------------------|-------------------------------------|-------------------------------|-----------------|
| G1              | 162.8                    | 13.6                             | 122.1                     | 7.0                                 | 35.0 (3.5%)                  | 2.7             |
| G2              | 180.9                    | 33.2                             | 135.7                     | 20.5                                | 132.2 (12%)                  | 3.9             |

4. Conclusion

The use of GFRP to strengthen the shear critical column is one of the effective and low-cost strengthening methods. The non-ductile column (G1) failed in abrupt shear failure mode and could not carry gravitational load at a very low drift of 3.5%. After strengthened by GFRP, the column G2 could sustain the lateral drift up to 12% without gravity load being compromised. The column G2 was eventually failing in flexure mode associated with longitudinal reinforcement yielding and crushing of concrete inside the GFRP. The use of GFRP could increase column shear capacity and also confine the concrete in the plastic hinge region. It is evident that the GFRP could effectively improve the unfavorable column shear failure of non-ductile columns by increasing column shear strength and concrete ductility in the plastic hinge region.
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