STRUCTURAL BEHAVIOR OF RC COLUMNS TRANSVERSELY REINFORCED WITH FRP STRIPS

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ABSTRACT

This paper presented the structural capacity of reinforced concrete columns transversely reinforced with FRP strips compared with the conventional carbon steel stirrups. Results were obtained through non-linear Finite Element (FE) simulation. Inadequate and improper design of transverse reinforcement includes the used of 90° hooks and large spacing between the stirrups may lead to shear failure or even collapse of the whole building. Stringent requirements have been outlined for transverse reinforcement in seismic design codes to maintain structural integrity and provide sufficient ductility to the structure which includes the usage of 135° end hook and close spacing between stirrups in the potential area of plastic hinge is among the requirements given. However, in most construction, the requirements are not adhered. Based on literature reviews, FRP composites demonstrate excellent behavior in enhancing the overall load capacity and ductility of RC columns due to its high-strength-and stiffness-to-weight-ratio. FRP sheets has been mainly used as wrapping of RC elements for repair and retrofitting purposes, while FRP bars were used to substitute the conventional steel bars. However, limited study has been conducted in the application of FRP strips as transverse reinforcement in RC columns subjected to cyclic loading. Results from the numerical analyses shows that the RC columns transversely reinforced with FRP strips has 12.5% and 36.8% higher in the ultimate load capacity and lateral displacement, respectively compared to the one reinforced with carbon steel. It also has higher ductility ratio and energy dissipation capacity.

Key words: Structural, RC Columns, Transver, Reinforced and FRP Strips
1. INTRODUCTION

Majority of construction of structures and infrastructures worldwide uses reinforced concrete (RC). For many years carbon steel has been used for making the reinforcement bars, hoops and ties. However, one main problem with steel carbon reinforcement is its corrosion in humid and harsh environmental condition [1-5]. Another main issue with confinement in RC elements is the improper design of transverse reinforcement, which lead to structural failure when the structure is subjected to horizontal loads such as earthquake. Large spacing between stirrups may lead to shear failure as been observed in many earthquakes worldwide. Improper design of stirrups resulted to extensive of concrete crushing followed by buckling of rebars when the structure is subjected to earthquake [6-10]. The used of 90° hooks of stirrups tend to open up during the severe loading, which lead to little or no confinement of concrete. Due to their important effect to the ultimate capacity and ductility of RC elements like columns, stringent requirements were outlined in design codes regarding the specification of stirrups in order to maintain structural integrity under earthquake.

In many countries including Malaysia, confinement in RC structural elements is among the main concern for both academicians and industry practitioners [14]. Current seismic design guidelines [11-13] has stringent requirement details for transverse reinforcement which involve more complicated design with close spacing between the stirrups, crossties and 135° end hooks. Such detailing techniques require more time consuming even for skilled labours [15].

In order to enhance the shear capacity and ductility of existing RC columns under seismic loading, researchers have proposed seismic retrofitting method using FRP laminates as external confinement [16-18]. The high tensile strength of FRP composites enhances the ultimate load carrying capacity as well as ductility of RC columns by delaying the shear failure until a certain displacement [19]. Furthermore, the application of FRP bars also has become an alternative construction material to replace the conventional carbon steel reinforcement due to its advantages such as high corrosion resistance, high strength-to-weight ratio and lightweight [20-23]. However, limited research on the application and high cost of FRP composites as construction material prevents its wider use compared with other traditional materials such as steel and concrete. Therefore, a simplest way to introduce a new construction material with optimal solution can be obtained by combining the use of FRP composites with traditional structural materials to create innovative structural forms that are cost-effective and of high-performance [24 - 26].

Unlike carbon reinforcements, FRP bars are hard to be bent and to be used as hoops or ties. Additionally, so far, many studies have investigated the effect of externally bonded FRP sheets on the existing columns. Therefore, this study investigates the behavior of concrete columns with Carbon Fiber Reinforced Polymers, CFRP strips wrapped internally on the perimeter of longitudinal steel reinforcement. It is worthy to mention that, CFRP sheets have superior properties with elastic linearly behavior without yielding in tension compared to the mild steel used as stirrups in conventional design. Using FRP strips as spiral-shaped may avoid the open up of 90° hooks during an earthquake which can enhance shear strength and confinement in the columns, as well as prevent the buckling of longitudinal steel reinforcements. Besides, FRP sheets with 10 times higher tensile strength and 60 times lighter
than the carbon mild-steel reinforcements that usually used as stirrup will decrease overall weight of structures and seismic force imposed to it. This will contribute to economic design and reduce construction cost. These benefits together with the high resistance of FRP against corrosion and harsh environment show the significance of this new approach and its potential in increasing structural sustainability and safety at competitive. In order to solve the problem with the conventional steel stirrups, this study has proposed Carbon Fiber Reinforced Polymers, CFRP strips as new transverse reinforcement to improve the shear resistance and ductility of RC columns instead of using as strengthening material as per reported in literatures [16, 17].

In this paper, numerical analyses are conducted to determine the ultimate load capacity and structural behaviour of this new design. Due to importance of modelling technique in simulating properties and behaviour of RC column, detail attention has been given in the development of FE model under a constant axial load and cyclic lateral loads and validation were made based on laboratory results [19, 27].

2. FINITE ELEMENT MODELLING

Numerical analysis was carried out using ABAQUS software [29]. Three-dimensional FE model with nonlinear material properties is used to develop the RC column. Fig.1 shows the geometrical properties of RC column using conventional steel stirrups [27]. The concrete was modelled using 8-node 3-D solid elements while steel bars are modelled using 2-node truss element which is embedded in the concrete column has been done by other researcher [38]. A discrete shell element was placed on top of the column as load cell to transfer the gravity load to the bottom of column. Meanwhile, FRP strips is modelled using 4-node of shell with the membrane element as previously used by Kim et al. [19]. Furthermore, the surface interaction of FRP-steel is modelled using “contact” option with cohesion properties that is available in ABAQUS [29].

A constant axial load of 100 kN is applied on top of the shell plate placed on the column. The loading protocol for cyclic load is adopted according to FEMA 461 [30] as used in the laboratory works [27]. Every cycle repeated twice and increase in amplitude for each cycle is 40% from the previous cycle.

The non-linear constitutive model for concrete was modelled using “concrete damage plasticity” (CDP), which is design for monotonic and cyclic behaviour of concrete with two failure mechanism; tensile cracking and compressive crushing. This type of modelling
technique considers the degradation of the elastic stiffness induced by plastic straining both in tension and compression. It also considers the stiffness recovery effects under cyclic loading associated with the opening and closing of cracks in the concrete. The CDP model is based on five plasticity parameters and uniaxial stress-strain curves of the concrete in tension and compression. The plastic damage parameters used is summarized in Table 1 which are suggested based on [29, 31].

### Table 1 Plastic damage parameters for CDP in ABAQUS [29].

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Explanation</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\psi$</td>
<td>31</td>
<td>Dilation angle</td>
</tr>
<tr>
<td>$e$</td>
<td>0.1</td>
<td>Flow potential eccentricity</td>
</tr>
<tr>
<td>$\sigma_0/\sigma_c$</td>
<td>1.16</td>
<td>The ratio of initial equibiaxial compressive yield stress to initial uniaxial compressive yield stress</td>
</tr>
<tr>
<td>$K_c$</td>
<td>0.6667</td>
<td>The coefficient determining the shape of the deviatoric cross-section</td>
</tr>
<tr>
<td>$\mu$</td>
<td>0.0001</td>
<td>Viscosity parameter</td>
</tr>
</tbody>
</table>

The CDP model uses the yield condition proposed by Lubliner et al. [32] and further modified by Lee and Fenves [33] for different evolution of strength under tension and compression. The yield function can be defined as:

$$ F = \frac{1}{1-\alpha} (q - 3ap + \beta(e^{pl})(\sigma_{max}) - \gamma(-\sigma_{max})) - \sigma_c(e^{pl}) = 0 \tag{1} $$

$$ \alpha = \frac{(\sigma_{b0}/\sigma_c)^{-1}}{2(\sigma_{b0}/\sigma_c)} \tag{2} $$

$$ \gamma = \frac{3(1-K_c)}{2K_c} \tag{3} $$

$$ \beta(e^{pl}) = \frac{\sigma_c(e^{pl})}{\sigma_t(e^{pl})} (1 - \alpha) - (1 + \alpha) \tag{4} $$

where $\alpha$ and $\gamma$ are dimensionless material constants, determined by $\sigma_{b0}/\sigma_c$ and $K_c$. $q$ is the Mises equivalent effective stress, $p$ is the effective hydrostatic pressure, $\sigma_{max}$ is the maximum principle effective stress and $\sigma_c$ is the effective compressive cohesion stress, $\sigma_t$ is the effective tensile cohesion stress [29].

The flow potential $G$ chosen for this model is based on Druker-Prager hyperbolic function:

$$ G = \sqrt{(e\sigma_{t0}tan\psi)^2 + q^2} - p \tan \psi \tag{5} $$

where $\psi$ is the dilation angle, $\sigma_{t0}$ is the uniaxial tensile stress at failure, $e$ is a parameter of flow potential eccentricity.

There are plenty stress-strain relationships of the concrete has been proposed [34-36]. Normally, nonlinear material properties are illustrated by stress-strain curves. In this study, the results of Elastic modulus ($E_c$) test for concrete samples is used to model the nonlinear stress-strain curves of concrete. The Mander’s model [34] has been used to defined the stress-strain curve of confined concrete. The stress-strain curve of the concrete material is illustrated in Fig. 2. For opening and closing of cracks in the concrete, 0.9 for opened crack and 0.0 closed crack has been used as per reference [27].
Structural Behavior of RC Columns Transversely Reinforced with FRP Strips

Figure 2 Stress-strain curves of (a) concrete and (b) steel [27].

The longitudinal and steel stirrups are modelled with a bilinear elastic-plastic model. In this study, the results obtained from tensile test of steel reinforcements are used to define the stress-strain curve of reinforcement bars for nonlinear material properties. The stress-strain curve of the steel reinforcement is illustrated in Fig. 2.

In this study, carbon FRP has been selected as the confinement material. The FRP strips are treated as orthotropic elastic material with linear-fracture behaviour using “lamina” option. The properties of FRP composites are nearly the same in any direction perpendicular to the fibers. This modelling technique follows the approach previously used by Kim et al. [19]. The input parameters were assigned in the FE model according to the data provided by manufacturer, which are summarized in Table 2.

<table>
<thead>
<tr>
<th>Material</th>
<th>Elastic Modulus (MPa)</th>
<th>Poisson’s ratio</th>
<th>Tensile strength (MPa)</th>
<th>Shear modulus (MPa)</th>
<th>Tensile elongation (%)</th>
<th>Thickness (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CFRP</td>
<td>74700</td>
<td>48500</td>
<td>0.22</td>
<td>933</td>
<td>2900</td>
<td>2900</td>
</tr>
</tbody>
</table>

The bond between FRP-steel is a critical part under cyclic loading. In order to model such behaviour, surface-to-surface contact elements are adopted with cohesive behaviour of contact property. This type of interaction option is capable to model small-sliding contact between two deformable bodies or between a deformable body and a rigid body in two or three dimensions. Then, all the reinforcements are embedded in the concrete so that they will deform together with the surrounding concrete.

In order to avoid convergence problem during the analysis, the model is meshed into different numbers of element; ranges between 2704 to 6456 elements for mesh size 30mm, 35mm and 40mm, respectively. Effects of the element size is verified by comparing the ultimate load capacity obtained of the FE models and the most accurate result with the literature is chosen. Fig. 3 shows the comparison of load-displacement curves using different mesh size. Based on the comparison, the 40mm mesh size is selected with ultimate load capacity and corresponding displacement error by 4.3% and 20%, respectively; ultimate load capacity of 16.7kN with corresponding displacement 60.9mm obtained from FE model for a constant 100kN axial load and 100mm of lateral displacement applied as compared to 16kN and 76mm displacement from laboratory work [27]. The accuracy can be improved with adequate information provided from laboratory test.
Figure 3 Comparison of load-displacement curves obtained from laboratory works and FEM.

Furthermore, to simulate according to the real test, the nodes of the bottom surface of the footing are totally constrained as fixed boundary conditions. A constant axial load is modelled as concentrated force and loaded on top of discrete shell plate. Meanwhile, the horizontal displacement controlled cyclic load is imposed based on the insert amplitude on top of the column in x-direction as shown in Fig. 4, which is the same model that has been used in the reference [27].

Figure 4 Meshed FE model; (a) concrete and (b) reinforcements.

a. Validation of Finite Element Modelling

Validation of FE modelling technique and material model shows the accuracy of numerical model in simulating the behaviour of RC columns under a constant axial load and horizontal cyclic loadings. The validated numerical models are used to investigate the structural behaviour of different factors on the cyclic performance of RC columns.

Fig. 5 shows the hysteresis and backbone curves obtained from FE modeling. The initial stiffness obtain from FE model is 0.55 kN/mm, which is correlates well with the experimental result (i.e. 0.55 kN/mm). Note that the lateral stiffness of the column starts to reduce before reaching its ultimate load. Such degradation in the stiffness mainly due to the growth of cracks in the column as reported in the literature [27]. Ultimate lateral load capacity of the column obtained from the FE model is equal to 16.64 kN with corresponding displacement of 73.5 mm. This correlate closely with values obtained from experimental works [27], 16 kN
and 76 mm for the ultimate lateral load capacity and corresponding displacement respectively. This is equal to 4% and 3.3% of error respectively.

![Graphs showing hysteresis and backbone curves](image)

**Figure 5** Results obtained from FE model; (a) hysteresis curve and (b) backbone curve.

Tensile cracks distribution is shown in Fig. 6. Based on the result, it can be concluded that the trend exhibit from the FE model are comparable with the experimental works. Damage in concrete mostly concentrated at the bottom of the column and perpendicular to the edge of the column, similarly occurred in the tested specimen. The column experienced brittle failure mode, where the spalling and crushing in concrete occurred without yielding of reinforcement as shown in Fig. 6(a).

![Cracking and failure mode](image)

**Figure 6** Cracking and failure mode of the RC column; (a) Experimental works [27] (b) FE model.

Fig. 7 shows the hysteresis and the backbone curves of the RC column externally bonded with FRP strips [19] compared to the one obtained in this study using similar material model of the FRP. As be observed, the initial stiffness of the column is well estimated by the FE modelling. The ultimate lateral load carrying capacity of the FE model is 168 kN, which is slightly overestimated the ultimate strength from the experimental result (155 kN) with 8.3%. Meanwhile, the displacement exhibit by the FE model is 60 mm, in which 6.2% underestimated the result of the referred literature [19]. The unmatched results may be due to some data which are not provided in the journal such as the stress-strain curve of concrete and steel bars. This can be improved with adequate data of tested specimen that can be obtained from laboratory works.
b. Results of Finite Element Modelling

The results of FE models are presented based on two different types of column; column transversely reinforced with FRP strips and conventional RC column using carbon steel stirrup. Spaces between FRP strips are 200 mm similar with the carbon steel stirrups. Fig. 9 presents the backbone curves for both columns, while Table 3 summarized the extracted data obtained from the backbone curves includes the initial stiffness and ductility ratio. Based on Fig. 9, the ultimate load capacity of equivalent design column (FRP) is 18kN corresponding to 57mm displacement, which increases the ultimate load capacity and displacement by 12.5% and 36.8%, respectively compared with conventional RC column (Carbon Steel). The initial stiffness, $K_e$ and ductility ratio, $\mu$ of FRP column are 0.55 kN/mm and 2.72, respectively which increase 0.74% of it ductility ratio compared with Carbon Steel-200 column. Like Carbon Steel-200, the initial stiffness is similar correlates well with the result from previous study of FRP-RC column by Maranan et al. [20], where FRP spirals increase the overall shear resistance of RC member, except for stiffness. The lateral stiffness
starts to reduce at displacement 28.87 mm due to the growth of cracks in the column. Unlike column with carbon steel, column with FRP can maintain its ultimate lateral load at displacement 57 mm until it reaches lateral displacement of 78 mm. This confirms the beneficial effect of FRP strips in delaying the shear failure by allowing the column to fully utilized the flexural capacity provided by longitudinal steel reinforcements [19, 39].

**Figure 9** FE models load-displacement curves

**Table 3** Effective stiffness and ductility ratio of FE models

<table>
<thead>
<tr>
<th>FE models</th>
<th>$K_e$ (kN/mm)</th>
<th>$\Delta_y$ (mm)</th>
<th>$\Delta_u$ (mm)</th>
<th>$\mu = \Delta_u/\Delta_y$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Carbon Steel-200</td>
<td>0.55</td>
<td>27.21</td>
<td>73.55</td>
<td>2.70</td>
</tr>
<tr>
<td>FRP-200</td>
<td>0.55</td>
<td>28.87</td>
<td>78.62</td>
<td>2.72</td>
</tr>
</tbody>
</table>

Fig. 10 shows the hysteresis curves and cumulative energy dissipations of both the analysed columns. It can be observed that the confinement method has significant effects on the hysteresis curves of the columns since it protects the core concrete in the columns. For column that transversely reinforced with FRP strips, the energy dissipation is 1569kN.mm at 4.5 drift ratio, which is 9% higher than the one with carbon steel, respectively. The elastic-linearly characteristic of FRP strips without yielding in tension allows it high tensile strength to well confined the core concrete as well as prevent the buckling of steel reinforcement. Due to that reasons, even with low-ductile design of RC column, FRP strips can enhance the energy dissipation of column compared with RC column with carbon steel.

**Figure 10** Result of FE models; (a) Hysteresis curves and (b) cumulative energy dissipation
Fig. 11 shows the tensile stress distribution on concrete surface of column with carbon steel and column with FRP. The FRP drastically reduces the tensile cracks in the concrete compared with FE steel model. It can be seen through the reduction of the height of the cracks area in column transversely reinforced with FRP strips. Little or no excessive cracks in concrete indicates the ductile behaviour of column confined with FRP strips compared with the conventional stirrups. This indicates that a good confinement may enhance the failure mode of RC column from brittle failure to ductile failure.

3. CONCLUSIONS
This paper presented the investigation on structural behaviour of RC columns transversely reinforced with FRP strips as compared to the conventional carbon steel. Results were obtained from non-linear numerical analyses based on validated finite element models. Results from FE modelling indicated that RC columns with FRP strips has higher ductility ratio. Usage of FRP strips also helps in delaying the shear failure by allowing the column to fully utilize the flexural capacity provided by longitudinal steel reinforcements. RC columns transversely reinforced with FRP strips has 12.5% and 36.8% higher in the ultimate load capacity and lateral displacement, respectively compared to the one reinforced with carbon steel. It also has higher ductility ratio and energy dissipation capacity. This new design shows little or no excessive cracks in concrete.

ACKNOWLEDGEMENT
The authors would like to acknowledge supports from Universiti Teknologi Malaysia and financial support from the Ministry of Higher Education of Malaysia through the RUG Vot. No. 17H80 and 19H36, and Science Fund Vot. No. 4S125.

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