EXPERIMENTAL AND ANALYTICAL MODELLING OF GFRP RETROFITTED COLUMNS UNDER BENDING AND AXIAL LOAD
CHAPTER 5

EXPERIMENTAL AND ANALYTICAL MODELLING OF GFRP RETROFITTED COLUMNS UNDER BENDING AND AXIAL LOAD

5.1 General

Sophisticated tools are now available for modelling of concrete and reinforced concrete structures. It is thought that Finite Element (FE) analysis can give more insight into the problem in question and provide more reasonable results as compared with the traditional methods. ANSYS was used in this study to analyse the load-deflection behaviour of the test specimens. In order to investigate the behaviour of confined concrete with GFRP jackets, confined concrete cylinder under axial compression, GFRP confined square RC column specimens which are subjected to bending and axial load and specimens subjected to sustained axial loading along with reversed cyclic loading were tested in the laboratory to study their behaviour. The experimental results were compared with that of model loaded under similar conditions.

5.2 Experimental Program

5.2.1 Introduction

In this study, six numbers of RC square column specimens scaled to one quarter size of a five storied building were cast and tested with different layers of GFRP. The specimens were subjected to bending moment and axial load. Two types of reinforcement detailing for lateral ties were considered in this study.

5.2.2 Materials

Materials used for casting the specimens such as concrete and steel and strengthening materials for retrofitting the specimens were already discussed in the section 4.2 of chapter 4.
5.2.3 Design of RC Columns

All the column specimens to be tested were designed as per IS 456 - 2000. The details of test specimens are given in Table 5.1.

5.2.4 Test Specimens

The control and retrofitted specimens were tested under bending and axial load shown in Fig.5.1.
Table 5.1 Details of Test Specimens

<table>
<thead>
<tr>
<th>Specimen Description</th>
<th>$f'_c$ MPa</th>
<th>Main bars dia. mm</th>
<th>Lateral tie bars dia. mm</th>
<th>Lateral ties spacing mm</th>
<th>Height of specimen mm</th>
<th>Level of axial load %</th>
<th>No. of GFRP Layers</th>
</tr>
</thead>
<tbody>
<tr>
<td>C</td>
<td>28</td>
<td>4#10</td>
<td>8</td>
<td>150</td>
<td>900</td>
<td>30</td>
<td>0</td>
</tr>
<tr>
<td>CD</td>
<td>29</td>
<td>4#10</td>
<td>8</td>
<td>75</td>
<td>900</td>
<td>30</td>
<td>0</td>
</tr>
<tr>
<td>S1</td>
<td>26</td>
<td>4#10</td>
<td>8</td>
<td>150</td>
<td>900</td>
<td>30</td>
<td>1</td>
</tr>
<tr>
<td>SD1</td>
<td>31</td>
<td>4#10</td>
<td>8</td>
<td>75</td>
<td>900</td>
<td>30</td>
<td>1</td>
</tr>
<tr>
<td>S2</td>
<td>28.5</td>
<td>4#10</td>
<td>8</td>
<td>150</td>
<td>900</td>
<td>30</td>
<td>2</td>
</tr>
<tr>
<td>SD2</td>
<td>28.5</td>
<td>4#10</td>
<td>8</td>
<td>75</td>
<td>900</td>
<td>30</td>
<td>2</td>
</tr>
</tbody>
</table>

5.3 Test set up

Column footings were fixed to test floor of the loading frame by means of four number of anchor bolts of 20mm diameter. In this test, axial loading was applied using a hydraulic push jack of 500 kN and the lateral load was applied using a push pull hydraulic jack of 600 kN at the top of the specimen. A proving ring of 1000 kN capacity was provided to measure the applied axial load and was placed on the top of the column. An electronic load cell of capacity 500 kN was used to measure the applied lateral load and was monitored by load indicator. The column specimens were adjusted so that the centerlines of both the axial and lateral loading coincided with the respective column faces. Test set up is shown in Fig.5.2. Two number of 50mm range linear variable differential transducers (LVDTs) were used to measure the deflection at the top and bottom of the column specimens. LVDTs were connected to electronic monitors to monitor the deflections.
5.4 Test Observations

All the specimens were tested under sustained axial load level of 0.30F₀. The test observations for each specimen are discussed below:

(a) Specimen C: The first specimen tested was a control specimen C with reinforcement detailing for lateral ties spacing of 150mm c/c. No fabric was applied. When the lateral load reached 14 kN, first crack appeared at the base of the column. As load increased, the cracks were widened and propagated further around the initial crack. Therefore, plastic hinge developed at the bottom of the column. The maximum lateral load measured was 24 kN. Most damaged zone of the specimen was 80mm above the column-footing interface. P-Δ curve of this specimen is given in Fig. 5.3. The maximum deflection measured was 20mm.

(b) Specimen CD: The second column tested was a control specimen CD with lateral ties spacing of 75mm c/c. When the lateral load reached 15 kN, first crack appeared at the base of the column. As load increased, cracks widened and propagated around the initial crack. The maximum lateral load
measured was 27 kN. Most damaged zone of the column was 85mm above the column-footing junction. The maximum deflection measured was 21mm. P-Δ curve of this specimen is also given in Fig. 5.3.

(c) Specimen S1: The specimen S1 was cast with lateral ties spacing of 150mm c/c and was wrapped with one layer of GFRP around all four sides of the column for the full height of the specimen. Since the column was completely wrapped, the initial crack could not be seen from outside. However, the continuous sound of crushing of concrete was heard after 24 kN. Further loading initiated rupture of GFRP. The maximum lateral load measured was 28 kN and the maximum deflection measured was 18mm. Most damaged zone of the specimen was 100mm above the column footing interface. Hence the plastic hinge shifted upwards when compared to control specimen C. P-Δ curve of this specimen is given in Fig. 5.4.

(d) Specimen S2: The specimen S2, which was wrapped with two layers of GFRP around all four sides of the column. Rupture of fabric initiated at 29 kN. The maximum lateral load measured was 30 kN and the maximum deflection measured was 25mm. Most damaged zone of this specimen was 145 mm above the column-footing junction. P-Δ curve of this specimen is also given in Fig 5.4. The comparison of P-Δ curves of Specimens C, S1 and S2 is given in Fig 5.5.

(e) Specimen SD1: The specimen SD1 was cast with lateral ties spaced at 75mm c/c and was wrapped with one layer of GFRP around all four sides of the column. Since the column was completely wrapped, the initial crack could not be seen from outside. However, the continuous sound of crushing of concrete was heard after the load reached 25 kN and rupture of GFRP initiated at 29 kN. The maximum lateral load measured was 30 kN and the maximum deflection measured was 26mm. Most damaged zone of the specimen was 120mm above the column-footing junction. P-Δ curve of this specimen is given in Fig 5.7.

(f) Specimen SD2: The specimen SD2 was tested with lateral ties spaced at 75mm c/c and wrapped with two layers of GFRP around all four sides of the column. Rupture of fabric was initiated at 30 kN. The maximum lateral load
measured was 32 kN and the maximum deflection measured was 27 mm. Most damaged zone of the specimen was 150 mm above the column-footing junction. P-Δ curve of this specimen is also given in Fig. 5.6. Comparison of deflection for the specimens CD, SD1 and SD2 is given in Fig. 5.7.

Fig. 5.3 P-Δ Curves for Specimens C and CD

Fig. 5.4 P-Δ Curves for Specimens S1 and S2
Fig. 5.5 Comparison of P-\(\Delta\) Curves for Specimens C, S1 and S2

Fig. 5.6 P-\(\Delta\) Curves for S2 and SD2
Fig. 5.7 P-Δ Curves for Specimens
CD, SD1 and SD2

Fig. 5.8 compares the deflection of all the test specimens.

Fig.5.8 Comparison of P-Δ Curves for all the Test Specimens
5.5 Comparison of Test Results

Table 5.2 provides summary of test results.

### Table 5.2 Summary of Test Results

<table>
<thead>
<tr>
<th>Specimen</th>
<th>No. of GFRP Layers</th>
<th>Max. lateral load kN</th>
<th>Max. Lateral Deflection (mm)</th>
<th>Max. BM kNm</th>
<th>% Improvement in lateral load capacity</th>
</tr>
</thead>
<tbody>
<tr>
<td>C</td>
<td>0</td>
<td>24</td>
<td>20</td>
<td>22.56</td>
<td>-</td>
</tr>
<tr>
<td>S1</td>
<td>1</td>
<td>28</td>
<td>18</td>
<td>24.27</td>
<td>16.67</td>
</tr>
<tr>
<td>S2</td>
<td>2</td>
<td>30</td>
<td>25</td>
<td>25.6</td>
<td>25</td>
</tr>
<tr>
<td>CD</td>
<td>0</td>
<td>27</td>
<td>21</td>
<td>27.9</td>
<td>-</td>
</tr>
<tr>
<td>SD1</td>
<td>1</td>
<td>30</td>
<td>26</td>
<td>27.9</td>
<td>11</td>
</tr>
<tr>
<td>SD2</td>
<td>2</td>
<td>32</td>
<td>27</td>
<td>29.8</td>
<td>18.5</td>
</tr>
</tbody>
</table>

Fig. 5.9 compares percentage improvement of the lateral load capacity of retrofitted test specimens with respect to their control specimens.

5.6 Analytical Modelling

5.6.1 Finite Element Modelling

5.6.1.1 Element Type

An eight-noded solid element 65, was used to model the concrete. The solid element has three degrees of freedom at each node-translation in the
nodal x, y and z directions. Shell 41 element was used to model GFRP composites. Link 8 element was used to model the steel reinforcement. The details of the elements, material properties and element connectivity are given in Appendix B.

5.6.2 Modelling of Confined Concrete Cylinder

One quarter of the two layers GFRP confined cylinder was modelled in uniaxial compression. Fig. 5.10 shows the finite element discretisation of a one-quarter model of the GFRP confined concrete cylinder. The user inputs are the core diameter and the number of divisions in the radial and angular directions as well as the number of depth-wise slices. As for the boundary conditions, two planes of symmetry exist such as XY and YZ. All the nodes on each plane of symmetry are fixed only in the direction normal to that plane, and are free to move within that plane. Loading was applied to the model and the stress distributions obtained from the analysis is given in Fig. 5.11. Table 5.3 compares predicted strength of model with the experimental strength of two layers GFRP confined cylinder.

![Fig.5.10 One Quarter Model of Confined Concrete Cylinder](image_url)
Table 5.3 Comparison of Predicted versus Experimental Strength for Confined Concrete (Two Layer Confinement)

<table>
<thead>
<tr>
<th>Predicted strength (MPa)</th>
<th>Experimental strength (MPa)</th>
<th>Experimental Predicted</th>
</tr>
</thead>
<tbody>
<tr>
<td>34.415</td>
<td>37.68</td>
<td>1.094</td>
</tr>
</tbody>
</table>

5.6.3 Modelling of Retrofitted RC Square Concrete Column Specimens

Square concrete RC column was modelled using the element types mentioned in the Appendix B 1.1. The size of the model is full scale. On the assumption that the column-footing junction is rigid and there wouldn’t be any displacement and hence the footing part of the column was avoided. The finite element model of the column and that of reinforcement is shown in Figs. 5.12 and 5.13.
Fig. 5.12 Finite Element Model of the Column

Fig. 5.13 Reinforcement Elements in the Model
5.6.3.1 Loading and Boundary Conditions

Boundary conditions were applied to ensure that the model acts in the same way as the test specimen. In order to simplify the model, the footing part of the column was neglected with a provision of perfect rigidity at all the nodes at the column base. Axial load on the model was applied equally on all the top most nodes of the elements at the top of the column. Lateral loads were applied on a line of nodes at a height corresponding to horizontal jack in the test set up. Typical loading and boundary conditions of the model are shown in Fig. 5.14.

![Finite Element Modeling of Column](image)

**Fig. 5.14 Loading and Boundary Conditions of the Model**

5.7 Load Versus Deflection (P-Δ) and Strain Plots for Retrofitted RC Columns Under Bending and Axial load

For ANSYS, deflections were measured at the same location as they were measured in experimental set up. Figs. 5.15 to 5.23 show the deflection plots and strain distribution from the finite element analyses for various specimens.
Fig. 5.15 Deflection of Specimen S1

Fig. 5.16 Strain Distribution in Specimen S1
Fig. 5.17 Deflection of Specimen S2

Fig. 5.18 Strain Distribution of Specimen S2
Fig. 5.19 Deflection of Specimen SD1

Fig. 5.20 Strain Distribution of Specimen SD1
Fig. 5.21 Deflection of Specimen SD2

Fig. 5.22 Strain Distribution of Specimen SD2
5.8 Predicted Versus Experimental P-Δ Curve

To validate, the predicted P-Δ curves from ANSYS were compared with those of experiment. Figs. 5.24 to 5.29 compare these two curves for all the test specimens.
Fig. 5.25 Comparison of P-\(\Delta\) Curves of Specimen CD

Fig. 5.26 Comparison of P-\(\Delta\) Curves of Specimen S1
Fig. 5.27 Comparison of P-Δ Curves of Specimen S2

Fig. 5.28 Comparison of P-Δ Curves of Specimen SD1
Fig. 5.29 Comparison of P-Δ Curves of Specimen SD2

Control specimen C

Fig. 5.24 compares the predicted and experimental load versus deflection for specimen C. The finite element plot and the experimental curve are having the same stiffness in the initial stage. The maximum load measured in the experiment is 24 kN while in FE model it was 27 kN. The difference in deflection corresponding to 24 kN between both FE model and experimental one is about 26.25%.

Specimen S1

Fig. 5.26 compares predicted and experimental load versus deflection for specimen S1. The finite element model and the experimental curves are having the same stiffness in the initial stage. The maximum load measured in experiment is 28 kN while in FE model it was 32 kN. However, the deflection corresponding to 28 kN the deflection of both FE model and experimental one coincides.

Specimen S2

Fig. 5.27 compares P-Δ curves of the experimental and the predicted one. Both these curves are similar in the initial linear range. After cracking, the
experimental plot yields more deflection than the FE model. The maximum load carried by the specimen in experiment is 30 kN whereas that of the FE model is 32 kN. The variation in deflection between the FE model and the experimental one corresponding to 30 kN is 41.3%.

**Control specimen CD**

Fig. 5.25 compares experimental and predicted load versus deflection plots for specimen CD. The finite element model and the actual one are having the same stiffness in the initial stage. The maximum load of the test specimen is 27 kN while in FE model it was 33 kN. The difference in deflection corresponding to 27 kN between the FE model and experimental one is about 18.68%.

**Specimen SD1**

Fig. 5.28 indicates that the experimental and predicted P-Δ curves coincide well with each other till the maximum load. The FE model plot is stiffer at the initial linear part and after cracking it becomes less stiff than the actual one. The maximum loads of both the plots are same and it is equal to 30 kN. A variation of 25% can be seen between the experimental and FE model deflections at the peak load.

**Specimen SD2**

Fig. 5.29 indicates that the P-Δ curves of predicted and experimental are fairly similar to each other. The FE model plot is stiffer than the actual one. The maximum load of experimental one is 32 kN. Even though the deflections of both plots at a load 32 kN were exactly same, the deflection of the test specimen increased at higher loads. A variation of 23% in the values of deflection between the experimental and FE model was observed.

### 5.9 Study of Crack Patterns

In ANSYS, a cracking sign represented by a circle appeared when the principal tensile stress exceeded the ultimate tensile strength of the concrete. Fig. 5.30 shows crack propagation in FE modelling of specimen subjected to bending and axial load. The principal tensile stresses occurred mostly in the y direction (longitudinally) at the bottom of the specimen model. It was noted that when the principal stresses exceed the ultimate tensile strength of the
concrete, circles with cracking signs appeared perpendicular to the principal stresses in the y direction. Since all the models had identical boundary and loading conditions, crack patterns for all of them were similar except the range of loadings.

![Fig. 5.30 Crack propagation in FE modeling](image)

5.10. Load Versus Deflection Plots for Specimens under Sustained Axial Loading and Lateral Reversed Cyclic Loading

Specimens retrofitted with GFRP jacketing and tested under 35% axial loading and simultaneous reversed cyclic loading were considered to validate the experimental load versus deflection with that of ANSYS model subjected to similar loading condition. All the modelling details were similar to that one used for the specimens subjected to bending and axial load. Only loading pattern was required to be changed. The loading history is given in Fig.5.31. Loads were incremented in positive and negative directions alternatively after each load steps and results were monitored till the solution was not converging. Results were plotted with the help of time history post processor.
available in ANSYS software. The results obtained from finite element modelling were compared with that of experimental work of specimen C1, S1 and S4 discussed in Chapter 4. Figs. 5.32 to 5.34 compare the load versus deflection of the experimental with that of modeled specimens. They show a fairly good agreement. The P-Δ curves of strengthened specimens have more area and number of cycles and it explains the enhancement in ductility and energy absorption to dissipate the energy before they fail.

![Fig. 5.31 Loading History](image1)

![Fig. 5.32 Comparison of P-Δ Curves for Model and Experimental for Control Specimen C1](image2)
Figs. 5.35 to 5.37 show the superimposed $P-\Delta$ curves for the experimental and modeled specimens.
Fig. 5.35 Superimposed P-Δ Curves – Experimental versus Model for Specimen C1

Fig. 5.36 Superimposed P-Δ Curves – Experimental versus Model for Specimen S1

Fig. 5.37 Superimposed P-Δ curves – Experimental versus Model for Specimen S4
5.11 Results and Discussions

In this study to investigate the behaviour of confined concrete with GFRP jackets, confined concrete cylinder under axial compression, retrofitted square RC column specimens which were subjected to bending and axial loading and specimens subjected to sustained axial loading along with reversed cyclic loading were modelled with ANSYS. To validate the analytical modelling, specimens were cast and tested in the laboratory.

In the case of concrete cylinder confined by two layers of GFRP under axial compression, the ultimate confined strength of concrete obtained from FE model and experiment were compared and it is given in Table 5.4. From the table it is shown that results from FE model differ marginally from that of the experimental value.

Table 5.4 Comparison of Predicted and Experimental Ultimate Confined Compressive Strength

<table>
<thead>
<tr>
<th>Description</th>
<th>Predicted MPa</th>
<th>Experimental MPa</th>
<th>Experimental Predicted</th>
</tr>
</thead>
<tbody>
<tr>
<td>GFRP confined concrete cylinder</td>
<td>34.415</td>
<td>37.68</td>
<td>1.094</td>
</tr>
<tr>
<td>(two layers)</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

In the case of retrofitted GFRP confined concrete columns subjected to bending moment and axial load, comparison of predicted and experimental results are given in Table 5.5.

Table 5.5 Comparison of Predicted and Experimental Maximum Deflections

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Predicted deflection (mm)</th>
<th>Experimental deflection (mm)</th>
<th>Experimental Predicted</th>
</tr>
</thead>
<tbody>
<tr>
<td>C</td>
<td>17</td>
<td>20</td>
<td>1.17</td>
</tr>
<tr>
<td>S1</td>
<td>19.93</td>
<td>18</td>
<td>0.903</td>
</tr>
<tr>
<td>S2</td>
<td>14.759</td>
<td>25</td>
<td>1.69</td>
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<tr>
<td>CD</td>
<td>19.5</td>
<td>21</td>
<td>1.076</td>
</tr>
<tr>
<td>SD1</td>
<td>27.69</td>
<td>26</td>
<td>0.93</td>
</tr>
<tr>
<td>SD2</td>
<td>28.773</td>
<td>27</td>
<td>0.94</td>
</tr>
</tbody>
</table>
Table 5.5 indicates that the predicted and experimental maximum deflection values closely agree with each other.

In the case GFRP confined concrete columns subjected to sustained axial load and lateral reversed cyclic loading, comparison of predicted and experimental results are given in the Table 5.6.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Number of loading cycles</th>
<th>Maximum deflection (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Predicted</td>
<td>Experimental</td>
</tr>
<tr>
<td>C1</td>
<td>6</td>
<td>7</td>
</tr>
<tr>
<td>S1</td>
<td>12</td>
<td>10</td>
</tr>
<tr>
<td>S4</td>
<td>13</td>
<td>14</td>
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</table>

5.12 Conclusions
1. The value of ultimate compressive strength of GFRP confined concrete cylinder predicted by FE modeling differs by 9.5% with respect to experimental value. The value obtained from the experiment was on the higher side.
2. Finite Element modelling using ANSYS 8.0 is presented to obtain the load deflection curve for retrofitted GFRP confined concrete columns subjected to bending moment and axial load. The finite element modelling results do have a good agreement with the experimental curves of test specimens.
3. The crack patterns at the final loads from the finite element models correspond well with the observed failure modes of the experimental columns.
4. Finite Element modelling for GFRP confined concrete columns under sustained axial loading and static reversed cyclic loading also had reasonable agreement with experimental results.