CONCRETE FILLED GLASS FIBRE REINFORCED POLYMER (GFRP) SHELLS UNDER CONCENTRIC COMPRESSION

by

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A Thesis submitted in conformity with the requirements for the degree of Masters of Applied Science
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Concrete Filled Glass Fibre Reinforced Polymer (GFRP) Shells Under Concentric Compression

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ABSTRACT

An experimental investigation was conducted to study the behaviour of concrete filled Glass Fibre Reinforced Polymer (GFRP) shells under concentric compression. The main objective of this study was to assess the suitability of prefabricated GFRP shells as a stay-in-place formwork and confining material.

A total of seventeen columns of dimensions 355.6 x 1524 mm (14 x 60 in.) were tested. The variables tested were number of GFRP layers, orientation of fibres, and the amount of longitudinal and lateral steel. Concrete with a compressive strength of 30 MPa was used. Results showed a significant increase in strength, ductility, and energy absorption capacity of columns due to confinement provided by GFRP shells. Fibres in the longitudinal direction improved the load carrying capacity of the columns. It was concluded that GFRP shells have the potential to replace lateral steel for confinement purposes.
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CHAPTER 1

INTRODUCTION

1.1 BACKGROUND

Many observations have lead to the conclusion that column failures can result in total collapse of structures, particularly during severe earthquakes. Thus, strength and ductility of columns are of utmost importance in any structure. Several researchers have reported that confinement of concrete by suitable arrangement of transverse reinforcement results in a significant increase in both its strength and the ductility.

The idea of confining concrete columns using lateral or transverse steel was originally put forward by Considere. Subsequently an extensive experimental research was carried out by Richart, Brandtzaeg, and Brown to develop a mathematical expression for strength applied to both spirally reinforced and hydraulically confined columns. Later, Richart studied the effectiveness of the protective concrete shells in tied and spirally reinforced concrete columns. Roy and Sozen and Kent and Park, based on their research, suggested that rectilinear lateral reinforcement increases concrete ductility significantly but has little effect on concrete strength. Extensive experimental and analytical research carried out at the University of Toronto by Sheikh and Uzumeri showed that appropriately detailed rectilinearly confined concrete demonstrates large gains in strength and ductility due to confinement. An analytical model was proposed which was based on the determination of the effectively confined concrete inside the column core. The effectively confined concrete area was a function of the distribution of both longitudinal and lateral reinforcement. Mander, Priestley, and Park also performed tests and proposed a theoretical stress-strain model to predict the behaviour of confined concrete. The model allowed for the effect of various configurations of transverse steel as suggested by Sheikh and Uzumeri, cyclic loading, and strain rate. Ahmad and Shah, Fafitis and Shah, Saatcioglu and Razi also proposed models to predict the behaviour of confined concrete. It was observed that the confinement of
concrete increases its compressive strength and ductility. Further research focused on different concrete confining techniques. Circular spirals were found to confine concrete much more effectively than rectangular or square hoops. Factors, such as ratio of the volume of transverse steel to the volume of the concrete core, yield strength of the transverse steel, spacing of transverse steel, and minimum required diameter of transverse steel were also studied.

The ACI Code (ACI 318-99) provides equations for the volumetric ratio of spiral reinforcement ($\rho_s$) based on the requirement that the increase in the strength of the core concrete due to confinement should offset the loss in the strength due to spalling of the shell concrete. These equations were derived on the basis of strength enhancement of concrete due to confinement as observed by Richart et al.\cite{2,3,12}

In recent years, retrofitting and repair of concrete columns by wrapping and bonding fibre reinforced polymer (FRP) sheets or straps around the column or by FRP jackets has become popular. Advancements in the applications of FRP materials have accelerated the research on FRPs as external reinforcing material in columns.\cite{13}

1.2 PROBLEM

With the advancement in the field of FRP materials and their successful experimental application as a retrofitting and repair material, engineers need design guidelines and more information regarding the behaviour of concrete columns reinforced externally with different types of Fibre Polymers. The relationship between the behaviour of concrete confined with FRP and that confined with steel has to be determined. The suitability of applying the models originally developed for transverse steel reinforcement to FRP reinforcement need to be investigated further.

Fibre Reinforced Polymers (FRPs) are yet to be used frequently in new construction of concrete columns. Since confining concrete columns using FRP is relatively new, theoretical and experimental work in this area is still limited.\cite{14,15} Prefabricated FRP shells can be used to confine concrete columns. The FRP shells will also act as a permanent formwork and protect the encased concrete against harsh environmental effects including salt attack. This thesis addresses the issues of column behaviour as affected by the FRP shells.
1.3 OBJECTIVE AND SCOPE OF RESEARCH

This research is aimed at studying the behaviour of large-scale circular concrete columns reinforced with prefabricated FRP shells and subjected to concentric monotonic axial compression. Effects of various factors, such as amount (number of layers) and orientation of FRP confinement and presence of FRP reinforcement in the longitudinal direction, on the strength and ductility of the columns are investigated. The research also includes a comparative study of concrete columns confined by both lateral steel and FRP.

A total of seventeen columns were designed, constructed, tested, and analysed. All the specimens were of the same dimensions, 355.6 mm (14 in.) in diameter and 1524 mm (60 in.) in height. Eleven of the seventeen columns contained glass FRP shells while six columns did not have any FRP shells. A similar parallel program investigated the behaviour of columns with carbon FRP shells.

1.4 ORGANIZATION

Chapter 2 explains the behaviour of confined concrete. The mechanism and benefits of confinement of concrete are discussed. Chapter 3 discusses the properties of different types of FRPs and their applications.

An extensive literature review of relevant research regarding confinement of concrete columns is presented in Chapter 4. Chapter 5 discusses the experimental program. Analysis and discussion of the test results are presented in Chapter 6. Conclusions are reported in Chapter 7 along with recommendations for future research. An appendix containing plots/graphs demonstrating the behaviour of specimens as obtained from the tests is also provided at the end.
CHAPTER 2

CONCRETE CONFINEMENT

2.1 GENERAL

In this chapter a comparison between behaviour of unconfined and confined concrete is presented. The mechanism of confinement in reinforced concrete columns and various factors affecting the behaviour of confined concrete are also discussed.

2.2 BEHAVIOUR OF UNCONFINED CONCRETE IN COMPRESSION

"While the compressive stress-strain responses of the constituents of concrete i.e. the aggregate and the cement paste are linear, the stress-strain response of concrete is non-linear"[16], as shown in Figure 2.1

![Figure 2.1 Stress-Strain Responses of Concrete and its Constituent Materials][16]

"The interaction between the cement paste and the aggregate causes the non-linearity of the concrete stress-stain response. At relatively low stress levels, the development and propagation of micro-cracks at the aggregate-paste interfaces soften the concrete, resulting in a somewhat parabolic stress-strain curve".[16]
The response of concrete in uniaxial compression is usually determined by loading cylinders of concrete with a height to diameter ratio of 2. These cylinders, 150 mm x 300 mm, are loaded so that the maximum stress ($f_c'$) is reached in 2 to 3 minutes. Figure 2.2 shows typical stress-strain curves obtained from concrete cylinders loaded in uniaxial compression.

![Figure 2.2 Typical Compressive Stress-Strain Curves](image)

Figure 2.2 demonstrates that with the increase in concrete strength, the ductility decreases, whereas initial stiffness and linearity of the curve increases. Once the maximum stress ($f_c'$) is reached at a strain $\varepsilon_0$, concrete cannot support this high level of stress with increasing deformation. For concrete strengths less than about 6000 psi (41 MPa), the stress-strain relationship can be reasonably described by a simple parabola.
2.3 CONFINEMENT OF CONCRETE

Previous research has demonstrated that confinement of concrete can considerably improve its stress-strain characteristics at high strains. Considere\textsuperscript{[1]} in 1903 showed that confinement of axially loaded columns increases the strength and ductility of the columns by a considerable amount. Richart, Brandtzaeg, and Brown\textsuperscript{[2]} reported that lateral confining pressure greatly enhances the strength and stiffness of concrete cylinders and dramatically increases the strain at which the peak stress is reached. The lateral confining pressures reduces the tendency for internal cracking and volume increase just prior to cracking, thus increasing ductility and strength of the confined concrete. The stress-strain curves obtained show improved peak compressive stress and ductility. Figure 2.3 shows the effect of hydraulic confining pressure on stress-strain response.\textsuperscript{[2]}

\begin{figure}[h]
\centering
\includegraphics[width=\textwidth]{figure23.png}
\caption{Figure 2.3 Effect of Lateral Confining Pressure on Stress-Strain Response\textsuperscript{[2]}}
\end{figure}
Confinement considerably increases the energy absorption capacity of concrete. Thus in seismic regions, appropriately detailed transverse reinforcement is provided to confine the concrete and hence increase the ductility of columns and beams.\[6, 7, 17\]

### 2.4 MECHANISM OF CONFINEMENT

"In practice, columns are confined by lateral reinforcement, commonly in the form of closely spaced steel spirals or hoops. At low levels of stress in the concrete the lateral reinforcement is hardly stressed, thus the concrete exhibits unconfined behaviour. When stresses approach the uniaxial strength, the progressive internal cracking cause high lateral strains. The concrete bears out against the lateral reinforcement, which then applies a confining reaction to the concrete and hence the concrete exhibits confined behaviour."\[18\]

Circular spirals, because of their shape, are in axial hoop tension and provide continuous confining pressure around the circumference. However, square and rectangular hoops can apply confining pressure only at the corners of the ties, thus causing a portion of the core concrete to remain unconfined.\[7\]

---

**Figure 2.4** Confinement by Transverse Reinforcement (A) Rectilinear Ties (B) Spirals or Circular Hoops\[18\]
2.5 FACTORS AFFECTING CONFINEMENT

Following are some of the variables that affect the confinement of concrete and thus its stress-strain curve:\(^{[7,17]}\)

1. The configuration of transverse steel.
2. The ratio of volume of transverse steel to the volume of concrete core.
3. The yield strength of transverse steel.
4. The ratio of spacing of transverse steel to the dimensions of concrete core.
5. The ratio of diameter of transverse bar to the unsupported length of transverse bars in the case of rectangular stirrups or hoops, since a stiffer bar leads to more effective confinement. In the case of a circular spiral this variable has no significance; given its shape, the spiral will be in axial tension and will apply a uniform radial pressure to concrete.
6. The amount and size of longitudinal steel.
7. The strength of concrete.
8. The rate of loading.

2.6 ACI CODE (ACI 318-99)\(^{[19]}\) CONFINEMENT REQUIREMENTS

As discussed earlier, the confinement of concrete by transverse steel increases the strength of concrete due to confining pressure applied by the lateral reinforcement. The concrete cover outside the transverse steel, however, is not confined and will crush and spall off as soon as the concrete reaches its limiting strain, after which the transverse steel is effective in confining concrete and prevents the expansion of the concrete core.

The ACI code expressions for the amount of steel for confinement are based on the requirement that the increase in the strength of the core concrete due to confinement should be equal to the loss in the strength due to spalling of the shell concrete, thus keeping the axial load carrying capacity of the columns equal before and after spalling of cover.

The ACI code gives the following equations for spiral reinforcement:

\[
\rho_s = 0.45\left(\frac{A_g}{A_c} - 1\right)\frac{f'_c}{f_y} \quad (2.1)
\]

\[
\rho_s \geq 0.12\frac{f'_c}{f_y} \quad (2.2)
\]
where
\( \rho_s = \) volumetric ratio of spiral steel to concrete core measured from outside of spirals
\( A_g = \) gross area of column cross section
\( A_c = \) column core area measured from outside of lateral steel
\( f_c' = \) compressive strength of unconfined concrete
\( f_y = \) specified yield strength of spiral reinforcement but not more than 410 MPa (60000 psi)

2.7 CANADIAN CODE CONFINEMENT REQUIREMENTS\(^{[20]}\)

According to the Canadian code (A23.3-94), the required volumetric ratio of spiral steel (\( \rho_s \)) for the non-seismic design of column is identical to Equation (2.1) used by the ACI 318-99 code, with the exception that yield strength of spiral (\( f_y \)) is not to be taken more than 500 MPa and the concrete strength (\( f_c' \)) is not to be more than 80 MPa. The required volumetric ratio of spiral steel (\( \rho_s \)) for the seismic design of column is also identical to Equation (2.2) used by ACI 318-99 code, with the exception that concrete strength (\( f_c' \)) is not to exceed 55 MPa.

2.8 SUMMARY

Chapter 2 discusses the behaviour of unconfined concrete and that of confined concrete. The chapter explains the mechanism of concrete confinement and also describes the benefits of confinement.
CHAPTER 3

FIBRE REINFORCED POLYMERS

3.1 GENERAL

A brief description of different types of commonly used fibre reinforced polymers (FRPs) and their properties is presented in this chapter. Factors affecting properties of FRP and applications of FRP including its use in confining concrete are also reviewed.

3.2 FIBRE REINFORCED POLYMERS

Composite materials obtained by reinforcing polymer matrices using fibrous materials like glass or carbon are known as Fibre Reinforced Polymers (FRPs), or Advanced Composite Materials. The reinforcing fibre provides the composite with its structural properties such as high modulus of elasticity and high ultimate strength; whereas the matrix binds the fibres together, protects them from damage, and distributes the stresses among them. The most common matrices are resinous materials such as vinyl esters, polyesters, and epoxies. [21]

3.3 PROPERTIES OF FRPs

The most common FRPs in civil engineering applications are glass fibre reinforced polymers (GFRP), carbon fibre reinforced polymers (CFRP), and aramid fibre reinforced polymers (AFRP). The fibres and matrix are combined in such a manner that the resulting composite material shows properties that are superior to those of its individual constituents. These properties mainly depend on the fibre volume, mechanical properties of constituents, and the procedure used to fabricate the composite. Properties of commonly used matrices are presented in Table 3.1. [22, 23]

The fibres are characterised by very high length to diameter ratios. When embedded, the fibres will improve the stiffness and strength characteristics of the polymer. A summary of typical fibre properties is presented in Table 3.2 [21]
### Table 3.1 Typical Matrix Properties \(^{[21, 22, 23]}\)

<table>
<thead>
<tr>
<th>Material</th>
<th>Density (kg/m(^3))</th>
<th>Modulus of Elasticity in Tension (E(_t)) (MPa)</th>
<th>Strength in Tension (f(_t)) (MPa)</th>
<th>Strength in Compression (f(_c)) (MPa)</th>
<th>Poisson's Ratio (v)</th>
<th>Co-efficient of Thermal Expansion ((\alpha)) (10(^{-6}/^{\circ})C)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Polyester</td>
<td>1200-1400</td>
<td>2500-4000</td>
<td>45-90</td>
<td>100-250</td>
<td>0.37-0.40</td>
<td>100-120</td>
</tr>
<tr>
<td>Epoxy</td>
<td>1100-1350</td>
<td>3000-5500</td>
<td>40-100</td>
<td>100-250</td>
<td>0.38-0.40</td>
<td>45-65</td>
</tr>
<tr>
<td>P.V.C</td>
<td>1400</td>
<td>2800</td>
<td>58</td>
<td>-</td>
<td>-</td>
<td>50</td>
</tr>
<tr>
<td>Nylon</td>
<td>1140</td>
<td>2800</td>
<td>70</td>
<td>-</td>
<td>-</td>
<td>100</td>
</tr>
<tr>
<td>Polyethylene</td>
<td>960</td>
<td>1200</td>
<td>32</td>
<td>-</td>
<td>-</td>
<td>120</td>
</tr>
</tbody>
</table>

### Table 3.2 Typical Fibre Properties \(^{[21]}\)

<table>
<thead>
<tr>
<th>Material</th>
<th>Density (kg/m(^3))</th>
<th>Modulus of Elasticity (E) (MPa)</th>
<th>Strength in Tension (f(_t)) (MPa)</th>
<th>Strain in Tension ((\varepsilon_t)) (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>E-Glass</td>
<td>2500</td>
<td>70000</td>
<td>1500-2500</td>
<td>1.8-3.0</td>
</tr>
<tr>
<td>S-Glass</td>
<td>2500</td>
<td>86000</td>
<td>4800</td>
<td>-</td>
</tr>
<tr>
<td>High-Modulus Carbon</td>
<td>1950</td>
<td>380000</td>
<td>2000</td>
<td>0.5</td>
</tr>
<tr>
<td>High-Strength Carbon</td>
<td>1720</td>
<td>240000</td>
<td>2800</td>
<td>1.0</td>
</tr>
<tr>
<td>Carbon</td>
<td>1400</td>
<td>190000</td>
<td>1700</td>
<td>-</td>
</tr>
<tr>
<td>Boron</td>
<td>2570</td>
<td>400000</td>
<td>3400</td>
<td>-</td>
</tr>
<tr>
<td>Graphite</td>
<td>1400</td>
<td>250000</td>
<td>1700</td>
<td>-</td>
</tr>
<tr>
<td>Kevlar49</td>
<td>1450</td>
<td>120000</td>
<td>2700-3500</td>
<td>2.0-2.7</td>
</tr>
<tr>
<td>Kevlar</td>
<td>1450</td>
<td>60000-130000</td>
<td>2900</td>
<td>-</td>
</tr>
</tbody>
</table>
Factors such as properties of constituents, procedure of fabrication, fibre orientation within the matrix, and strength of the fibre matrix bond affect the final properties of the composite material \[^{24}\]. All these factors can be controlled to generate a wide range of physical and mechanical properties for the composite material.

Typical mechanical properties of GFRP (Glass Fibre Reinforced Polymers) and CFRP (Carbon Fibre Reinforced Polymers) are given in Table 3.3\[^{21, 22}\].

Table 3.3 Typical Mechanical Properties of GFRP and CFRP \[^{21, 22}\]

<table>
<thead>
<tr>
<th>Material</th>
<th>Fibre content % by weight</th>
<th>Density kg/m(^3)</th>
<th>Modulus of elasticity in tension (E_t) MPa</th>
<th>Strength in tension (f_t) MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unidirectional GFRP/Polyester</td>
<td>(50-80)</td>
<td>1600-2000</td>
<td>20000-50000</td>
<td>400-1250</td>
</tr>
<tr>
<td>laminate</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>GFRP/Polyester Randomly Oriented</td>
<td>(25-25)</td>
<td>1400-1600</td>
<td>6000-11000</td>
<td>60-180</td>
</tr>
<tr>
<td>Hand lay-up</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>GFRP/polyester Material Metal Dye</td>
<td>(25-50)</td>
<td>1400-1600</td>
<td>6000-12000</td>
<td>60-200</td>
</tr>
<tr>
<td>GFRP/Polyester Woven Roving</td>
<td>(45-62)</td>
<td>1500-1800</td>
<td>1200-2400</td>
<td>300-350</td>
</tr>
<tr>
<td>Hand Lay-ups</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sheet Moulding Compound, Unidirectional Laminate</td>
<td>(20-25)</td>
<td>1750-1900</td>
<td>9000-13000</td>
<td>60-100</td>
</tr>
<tr>
<td>Carbon/Epoxy</td>
<td>(70)</td>
<td>1930</td>
<td>120000</td>
<td>800</td>
</tr>
<tr>
<td>Aramid/Epoxy</td>
<td>(50-80)</td>
<td></td>
<td>70000-80000</td>
<td>1000-1400</td>
</tr>
</tbody>
</table>

12
"The main features of the composite materials are their high fracture energy, ease of fabrication, and potential for low cost. The low cost is particularly true for the glass-reinforced polymers, which involve low material cost as well as low capital equipment cost, compared to metal processing. The advantages of the composites over the conventional bulk material are as follows:

- They can be made with high strength and high specific strength (ratio of strength to specific weight).
- They can be made with high stiffness and high specific stiffness (ratio of stiffness to specific weight).
- Density is generally low.
- Strength can be high at elevated temperature.
- Impact and thermal shock resistance are good.
- Fatigue strength is good, often better than the metals.
- Oxidation and corrosion resistance are particularly good.
- Thermal expansion is low and can be controlled.
- Stress-rupture life is better relative to many metals.
- Predetermined properties can be produced to meet individual needs.
- Fabrication of large components can often be carried out at lower costs than for metals."

FRPs are most commonly found as laminates, which are manufactured by unifying a number of thin layers of fibres and matrix into a desired thickness. Orientation and amount of fibres affect the properties of laminates. Laminates may be available in unidirectional, two-dimensional or three-dimensional arrangements of the fibres. The properties in any direction will be proportional to the amount of fibre by volume in that direction.

The coefficient of thermal expansion of concrete is $10 \times 10^{-6}/\degree C$ [26] and that of GFRP is approximately $9.9 \times 10^{-6}/\degree C$ whereas that of CFRP is very close to zero. [27] Hence, GFRPs bonded to concrete and when exposed to temperature fluctuations are not expected to cause any problems of differential thermal deformations. However, problems may arise with CFRP. A manufacturer of CFRP recommends the use of fibre anchors oriented in a radial pattern around an epoxy-grouted hole [28] to provide the required
an anchorage to account for differential deformations between the CFRP and the concrete when exposed to temperature fluctuations.

3.4 APPLICATIONS OF FRP

Development of composites can be considered as one of the biggest advances in material technology in the 20th century. It has found its application in many fields e.g. medicine, communication engineering and other industries. FRPs are also being introduced in the construction industry. Significant research is being conducted in exploring the various uses of FRP in the field of construction. Two sophisticated structures, the dome structure erected in Benghazi in 1968 and the roof at Dubai Airport built in 1972, confirm the attractiveness of composites as a building material. [29] Composites are also being effectively used to manufacture pipes. “Standards for Fibreglass Pressure pipe” developed by American Water Works Association and ASTM methods for testing indicate the effectiveness and importance of composites. [30]

Considerable progress in application of FRPs to bridge engineering has been achieved in Germany (1986) where GFRP strands (Polyestal) were used to post-tension concrete beams in a two span highway bridge. [27] One of the most promising applications of FRP in structural engineering appears to be repair and rehabilitation of different members such as beams and columns.

3.5 CONFINEMENT OF CONCRETE COLUMNS USING FRP

Use of FRPs as external reinforcement for concrete structures, such as columns, has gained popularity in Europe, Japan and North America. [13] Concrete columns have already been successfully retrofitted using FRP jackets. [31] However, FRPs have yet to be used in new construction involving confinement of concrete columns. Research has shown that FRP tubes have the potential to replace the conventional steel to confine concrete columns. [15] The confining action of the tube is created through the passive restraint to transverse dilation of concrete under uniaxial compression. The confinement due to FRP tube puts the concrete under triaxial compression, a stress state that increases the compressive strength of confined concrete by suppression of crack initiation in the core. Prefabricated FRP tubes can be used as permanent formwork to confine columns
and to act as a protective jacket against harsh environmental effects. Thus the potential benefits of using FRPs to confine concrete are quite attractive.

3.6 SUMMARY

In this chapter properties of various FRPs and their applications are discussed with an emphasis on confinement of concrete columns using FRPs.
CHAPTER 4

LITERATURE REVIEW

4.1 GENERAL

At the beginning of the twentieth century, engineers observed that concrete columns with longitudinal reinforcement develop longitudinal cracks and excessive lateral deformation under large compressive loads. This observation led Considere\(^1\), in 1903, to suggest the use of transverse reinforcement in order to slow down the lateral deformation. He carried out an experimental program and found that circumferential hoops, when placed at an appropriate spacing, increase the strength and ductility of the concrete columns considerably.

In 1928-29, Richart et al.\(^3\) carried out a series of tests employing hydraulic pressure for confinement of circular concrete columns. These columns were thus subjected to triaxial compressive stresses. It was observed that the increase in strength was directly proportional to the amount of confining pressure.

Later in 1930 and 1933, ACI directed an extensive research and developed expressions for compressive strength of columns reinforced with both longitudinal and lateral steel. Since then, many researchers have carried out research in this area and a few have proposed models to predict the behaviour of confined concrete columns. Extensive research has also been done on steel-jacketed columns and concrete filled steel tubes.

With the progress in the field of advanced composite materials, several studies have been carried out on the confinement of concrete columns with FRPs\(^{14, 33, 34, 35}\). It was observed that FRP-confined concrete columns exhibit considerable increase in compressive strength and ductility over the conventional confining methods.

The work done by numerous researchers to study the behaviour of circular confined concrete columns is reviewed in this chapter.
4.2 PREVIOUS RESEARCH

4.2.1 CONSIDERE, A. (1903)\textsuperscript{[1]}

Considere was one of the first researchers to study the behaviour of laterally confined concrete. He made an attempt to slow down the lateral expansion of concrete in columns with the use of transverse reinforcing steel.

In order to study the effectiveness of lateral reinforcement for slowing down the lateral deformation of circular concrete columns subjected to axial compression, Considere performed a series of tests on plain and spirally reinforced concrete specimens.

Six groups of test specimens with a diameter of 152 mm (6 in.) and heights ranging between 508 mm and 1295 mm (20 in. and 51 in.) were constructed. Longitudinal reinforcement was used in some of the test groups and consisted of eight 6.4 mm (0.25 in.) or 8.9 mm (0.35 in.) bars. Lateral reinforcement consisted of spirals or hoops with bar diameters between 4.3 mm and 6.4 mm (0.17 in. and 0.25 in.) and spacing between 15 mm and 30 mm (0.59 in. and 1.18 in.). Plain concrete strength ranged between 4.0 MPa and 46.5 MPa.

It was observed that the concrete specimens without reinforcement or with only longitudinal reinforcement show relatively brittle failure without any warning of the approaching collapse. On the other hand, specimens with lateral confining steel exhibited ductile failure.

Considere concluded from the tests that lateral reinforcement improves the maximum compressive strength of the specimens. Effectiveness of the lateral reinforcement was more pronounced for the specimens with smaller pitch. Confined concrete sustained excessive axial deformation prior to collapse thus indicating marked improvement in ductility.

4.2.2 RICHART, BRANDTZAEG, and BROWN (1929)\textsuperscript{[3]}

Tests were conducted to study the behaviour of plain and spirally reinforced concrete columns under uniaxial and triaxial compression. The relationship between the lateral pressure developed by the reinforcement and the axial stress at various stages of loading was one of the main objectives of the study.
All test columns had a diameter of 254 mm (10 in.) and a height of 1016 mm (40 in.). Concrete strength ranged between 16.7 MPa and 19.9 MPa. The volumetric ratio of spiral reinforcement used was as high as 4.4%. The specimens had no longitudinal reinforcement.

It was observed that the columns sustained the load steadily until the peak load for the corresponding plain concrete columns was reached. After this point, the load increased at a relatively slow rate until the column’s maximum load was obtained. Due to technical difficulties in recording the descending branch of the load-deformation curve of the specimens tested, the tests were terminated at the maximum load but it was anticipated that this maximum load recorded was not the failure load of the columns. It was noticed that large lateral deformations were required to bring the spiral reinforcement to bear against the expanding concrete core. This stage of loading at which the spiral reinforcement became effective was termed as “Spiral Stage”. The concrete inside the spiral at the maximum load was considered to be in a disintegrated granular mass state. From the test results, the following equation was deduced for the maximum column strength:

\[
f_1 = f'_c + 4.1 \, f_2
\]

where
\( f_1 = \) maximum column strength
\( f'_c = \) compressive strength of plain concrete
\( f_2 = \) lateral stress

Using the analogy of a thin-walled cylinder subjected to radial tension the following relation was defined:

\[
f_2 = \rho_s \, f_s / 2
\]

where
\( f_s = \) spiral stress
\( \rho_s = \) volumetric ratio of spiral reinforcement to core measured from outside of spirals.

These two relationships lead to the following equation:

\[
f_1 = f'_c + 2.05 \rho_s f_s
\]
4.2.3 ACI COMMITTEE 105 (1930-1933)

The ACI committee 105 directed an extensive research on the usefulness of the strength added by the lateral reinforcement in determining the working loads of columns. The research was summarised in a series of progress reports of the committee at five stages of the investigation. [36-42]

In 1931, the second progress report [37] of the committee was published which discussed the significance of spiral reinforcement in providing lateral support and keeping the concrete from splitting. The increase in concrete strength provided by the reinforcement was greatly emphasised instead of the increase in ultimate strain at failure. This was due to technical difficulties in performing properly controlled deformation tests at that time. However the “toughness” term was used to describe the strain at maximum load and spiral reinforcement effect was considered accordingly. The amount of lateral reinforcement required was still an unresolved issue before the committee. The third progress report (1931) concluded that time dependent deformations in reinforced concrete columns were independent of the spiral reinforcement.

In the fourth progress report of University of Illinois [39], efforts were made to determine the relationship between column concrete strength and cylinder concrete strength, the yield point of columns and the effectiveness ratio of spiral steel. The ratio of column concrete strength to cylinder concrete strength was observed to be 0.86, which became the basis for the co-efficient of 0.85. The yield point was defined as the load at which longitudinal steel reaches its yield point and the concrete develops its ultimate strength. This was also the maximum load for tied columns. For the spiral columns, it was realised that the lateral expansion of concrete produced stress in spiral steel and thus the confining pressure which increased the load carrying capacity of the concrete core.

The effectiveness factor, \( k \), defined as the ratio of contribution of the spiral steel to the contribution of the longitudinal steel of the same volume in carrying the axial load, was found to have an average value of 2.4 for air stored columns and 1.66 for wet stored columns.

The yield point of a column was given by:

\[
P = 0.85 f_c' (A_g - A_{st}) + A_{st} f_y
\]  

(4.4)
where
\[ A_g = \text{gross cross-section area of the column} \]
\[ A_{st} = \text{total area of longitudinal steel} \]
\[ f_y = \text{yield strength of longitudinal steel} \]

It was observed that at yield point the laterally bulging concrete induced stresses in the spiral reinforcement, which increased the ultimate load bearing capacity due to the confinement effect. The ultimate strength of laterally confined columns was given by:
\[
P_{ult} = 0.85f'_{c}(A_c-A_{st}) + f_yA_{st} + k\rho_sf_s'A_c \tag{4.5}
\]
where
\[ A_c = \text{core concrete area} \]
\[ A_{st} = \text{total area of longitudinal steel} \]
\[ \rho_s = \text{volumetric ratio of spiral steel to core measured from outside of spirals} \]
\[ f_s' = \text{useful stress limit of the spiral steel (assumed as the stress at a tensile strain of 0.005)} \]
\[ f_y = \text{yield strength of longitudinal steel} \]
\[ k = \text{spiral effectiveness factor with an average value of 2.4} \]

In the “Recommended Design Formulas” section of the Committee’s final report, a maximum spiral pitch of 76 mm (3 in.) was recommended to ensure a reasonable uniform confining pressure along the column height.

4.2.4 RICHART (1946)  
Richart conducted tests on 108 plain, tied, and spirally reinforced concrete columns to study the effectiveness of the protective concrete shells. The columns were 178 mm, 203 mm, or 229 mm (7 in., 8 in., or 9 in.) round or square and 1143 mm (45 in.) long. Both the ties and the spirals were circular with 152 mm (6 in.) outside diameter. Three grades of concrete having average compressive strengths of 19.9 MPa, 33.8 MPa, and 43.3 MPa were used. Four 12.7 mm (0.5 in.) hard grade plain steel bars were used as longitudinal reinforcement in each of these columns. Different sizes of lateral steel reinforcing wire were used at a pitch of 25.4 mm (1 in.).
Three designs of spirals were used to reinforce the columns. Design A (spiral strength equivalent to that of concrete shell), complying very closely with the ACI Building Regulations, Section 1103\(^{[43]}\); Design B (spiral stronger than concrete shell), with roughly 40% more spiral than design A, and Design C (spiral weaker than concrete shell), with roughly 40% less spiral than design A.

Richart observed that nearly all of the columns of design A and design C failed when the protective shell began to spall. While with further compression and shortening some of these columns developed a second “maximum” load due to the action of the spiral; this load never exceeded the load at first spalling. On the other hand, all of the columns of design B developed considerable additional load after the shell failure.

From the results and analysis of the spirally reinforced columns, Richart stated the yield load at spalling and the ultimate load as given in Equations (4.6) and (4.7) respectively.

\[
P = C (A_g - A_{st}) f_c' + A_{st} f_y \quad (4.6)
\]

\[
P_{ult} = C (A_g - A_{st}) f_c' + A_{st} f_y + k \rho' f_s' A_c \quad (4.7)
\]

where

- \( C \) = experimental factor
- \( k \) = spiral effectiveness factor
- \( A_g \) = gross area of the column
- \( A_{st} \) = total area of longitudinal steel in column
- \( A_c \) = column core area measured from outside to outside of lateral steel
- \( f_c' \) = concrete compressive strength as measured from standard cylinder
- \( f_y \) = yield strength of steel
- \( f_s' \) = useful limit stress in spiral
- \( \rho' \) = percent spiral by volume of core

The values of \( k \), spiral effectiveness, ranged between 1.34 and 2.24 with an average value of 1.8. The values of \( C \) ranged from 0.75 to 0.94 with an average value of 0.83 for spirally reinforced columns, which was considered to be in good agreement with the value of 0.85 stated by ACI Committee 105. A surprising and somewhat disturbing observation was that the value of \( C \) for identically designed tied columns was about 0.75
instead of the expected value of 0.85. It was noted that there was no consistent effect of class of concrete, shell thickness or design of spiral on the average value of $C$.

The results, combined with the sudden type of failure characteristics of tied columns, furnished a good argument for requiring a higher factor of safety for the design of tied columns.

The following conclusions were made from the study:

1. The shell concrete of spirally reinforced columns can be counted on for full effectiveness as a load-carrying element, if the concrete is properly placed and compacted.
2. The effectiveness factor, $k$, for the spiral reinforcement was 1.8 for columns in which the strength produced by the spirals was greater than that contributed by the cover concrete.
3. The results of the studies of shell effectiveness would seem to support the present ACI design methods in which the gross area of spirally reinforced columns is employed. Spiral columns designed on this basis have two very desirable physical characteristics, the relatively high stiffness right up to the maximum load and a slow manner of failure, marked by the spalling of the shell, at the maximum load.
4. The tied columns show a little less effectiveness of the cover concrete as compared to the spirally reinforced columns.

4.2.5 HUANG, T. (1964)[44]

Huang in 1964 presented a discussion on the ACI Building Code (318-63) formula for the minimum amount of spiral reinforcement. He gave an explanation for the rationale behind the spiral column formula. Huang's derivation is as follows:

Let

$D_c =$ diameter of the core

$A_{s'} =$ cross-sectional area of spiral reinforcement

$s =$ pitch of spiral

$S_3 =$ lateral pressure in concrete

$\Delta S_1 =$ increment in compressive strength of concrete due to lateral pressure
k = beneficiary factor taken to be approximately 4.0 based on the data from triaxial tests on concrete

$\rho_s = \text{ratio of volume of spiral reinforcement to total volume of core (out to out of spirals) of column.}$

$A_g = \text{gross area of column}$

$A_c = \text{core area of column}$

$f'_c = \text{compressive strength of concrete}$

$f_y = \text{yield strength of spiral reinforcement}$

Then:

$$\rho_s = \frac{A_g \pi D_c}{(A_g/A_c)}$$

$$= 4A_g/(sD_c)$$

$$S_3 = 2A_g f_y/(sD_c)$$

$$= \rho_s f_y/2$$

The load carrying capacity of the spalling concrete cover is:

$$0.85 f'_c (A_g - A_c)$$

Equalising the capacity of the cover to the additional capacity of the core:

$$(\Delta S_1)A_c = 0.85 f'_c (A_g - A_c)$$

$$\Delta S_1 = 0.85 f'_c [(A_g/A_c) - 1]$$

then

$$\Delta S_1 = kS_3$$

$$= kp_s f_y/2$$

and

$$\rho_s = 2\Delta S_1/(kf_y)$$

$$= (1.70/k)[(A_g/A_c) - 1]f'_c/f_y$$

substituting $k = 4.0$

$$\rho_s = 0.425[(A_g/A_c) - 1])f'_c/f_y \quad (4.8)$$
which is almost the same as the ACI code formula given as:

\[ \rho_s = 0.45[(A_p/A_e) - 1](f_{c'}/f_y) \]  \hspace{1cm} (4.9)

4.2.6 IYENGAR S. R, DESYA P., REDDY K. N. (1970)[45]

Iyengar et al. performed axial compression tests on specimens in which the variables were strength of concrete, size and shape of test specimens, diameter and type of spiral wire. The test specimens consisted of 150 x 300 mm (6 x 12 in.) and 100 x 200 mm (4 x 8 in.) cylinders with circular spiral steel and 150 x 150 x 300 mm (6 x 6 x 12 in.) and 100 x 100 x 200 mm (4 x 4 x 8 in.) prisms with square spiral steel. Concrete cylinder strength \((f_c')\) of specimens ranged between 17.3 MPa and 37.9 MPa. Two types of steel, 5 mm high-tensile steel and 6.5 mm mild steel, with yield strengths of 627.6 MPa and 318.7 MPa respectively, were used for the spiral wire. Spacing of the spirals ranged between 30 mm and 150 mm (1.2 in. and 6 in.). No concrete cover was provided, as the external dimensions of the spirals were kept almost equal to the dimensions of the test specimen. None of the specimens were provided with longitudinal steel.

It was observed that confinement increased both the strength and the deformation capacity of concrete in compression. The increase in strain capacity was found to be considerably higher as compared to increase in strength. Peak strength and the corresponding axial strain were found to increase with increasing lateral steel yield strength and volumetric ratio of lateral steel. Circular spiral steel was found to provide more effective confinement than an equivalent amount of square spiral steel. The less effective performance of square spirals was attributed to bending along the straight lengths of the spirals. Relative gains in the confined specimen peak strength and corresponding strain were found to decrease as the cylinder strength \((f_c')\) of the concrete increased.

It was concluded that the steel binders generated a confining effect similar to the influence of hydrostatic pressure on the strength of cylinders.
4.2.7 KURT C. E. (1978)[33]

Kurt studied the structural behaviour of concrete columns confined with commercially available plastic pipes. Two types of tubes were used; polyvinyl chloride (PVC), and acrylonitrile butadene styrene (ABS). Tests were conducted on short and long columns having a wide range of slenderness ratios.

Pipe diameters ranged from 38 mm to 100 mm (1½ in. to 4 in.) and lengths varied from 200 mm to 1450 mm (8 in. to 57 in.). Three specimens were made for each slenderness ratio.

The 28-day concrete strength was 20.6 MPa. The specimens were tested under an axial compressive load at a constant cross-head movement rate of 5.1 mm/min (0.2 in./min). Table 4.1 gives the properties of pipes used. Specimen properties and average ultimate column loads are given in Table 4.2.

Table 4.1 Properties of Pipe Materials

<table>
<thead>
<tr>
<th>Pipe material</th>
<th>Ultimate tensile strength MPa (psi)</th>
<th>Modulus of elasticity MPa (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>PVC</td>
<td>40.9 (5930)</td>
<td>2760 (400)</td>
</tr>
<tr>
<td>ABS</td>
<td>29.5 (4275)</td>
<td>1510 (219)</td>
</tr>
</tbody>
</table>

Specimens were grouped into three types: Type (A) were the unconfined specimens; Type (B) were the specimens with pipes slightly shorter in length than the concrete columns so that only the concrete would be axially loaded; Type (C) were the specimens with pipes equal in length to that of concrete columns, thus both the pipes and the concrete were axially loaded.
<table>
<thead>
<tr>
<th>Material</th>
<th>Specimen number/specimen type</th>
<th>Pipe thickness mm (in.)</th>
<th>Pipe diameter mm (in.)</th>
<th>Length mm (in.)</th>
<th>Average ultimate column load, kN (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>PVC</td>
<td>1(A)</td>
<td>N/A</td>
<td>76 (3)</td>
<td>200 (8)</td>
<td>97 (21.8)</td>
</tr>
<tr>
<td></td>
<td>2(B)</td>
<td>4.8 (0.19)</td>
<td>76 (3)</td>
<td>200 (8)</td>
<td>171 (38.5)</td>
</tr>
<tr>
<td></td>
<td>3(C)</td>
<td>4.8 (0.19)</td>
<td>76 (3)</td>
<td>200 (8)</td>
<td>182 (40.9)</td>
</tr>
<tr>
<td></td>
<td>4(C)</td>
<td>4.8 (0.19)</td>
<td>76 (3)</td>
<td>455 (18)</td>
<td>172.6 (38.8)</td>
</tr>
<tr>
<td></td>
<td>5(C)</td>
<td>4.8 (0.19)</td>
<td>76 (3)</td>
<td>1470 (58)</td>
<td>102.3 (23.0)</td>
</tr>
<tr>
<td></td>
<td>6(C)&lt;sup&gt;a&lt;/sup&gt;</td>
<td>6.4 (0.25)</td>
<td>100 (4)</td>
<td>200 (8)</td>
<td>315 (70.8)</td>
</tr>
<tr>
<td></td>
<td>7(C)&lt;sup&gt;a&lt;/sup&gt;</td>
<td>6.4 (0.25)</td>
<td>100 (4)</td>
<td>455 (18)</td>
<td>308 (69.3)</td>
</tr>
<tr>
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<td>8(A)</td>
<td>N/A</td>
<td>76 (3)</td>
<td>200 (8)</td>
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<tr>
<td></td>
<td>9(B)</td>
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<td>76 (3)</td>
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<td>148.6 (33.4)</td>
</tr>
<tr>
<td></td>
<td>10(C)</td>
<td>5.8 (0.23)</td>
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<td>200 (8)</td>
<td>162.4 (36.5)</td>
</tr>
<tr>
<td></td>
<td>11(C)</td>
<td>5.8 (0.23)</td>
<td>76 (3)</td>
<td>455 (18)</td>
<td>155.3 (34.9)</td>
</tr>
<tr>
<td></td>
<td>12(C)</td>
<td>5.8 (0.23)</td>
<td>76 (3)</td>
<td>610 (24)</td>
<td>151.3 (34.0)</td>
</tr>
<tr>
<td></td>
<td>13(C)&lt;sup&gt;a&lt;/sup&gt;</td>
<td>3.8 (0.15)</td>
<td>38 (1.5)</td>
<td>200 (8)</td>
<td>49 (11.0)</td>
</tr>
<tr>
<td></td>
<td>14(C)</td>
<td>3.8 (0.15)</td>
<td>38 (1.5)</td>
<td>455 (18)</td>
<td>41.4 (9.3)</td>
</tr>
<tr>
<td></td>
<td>15(C)</td>
<td>3.8 (0.15)</td>
<td>38 (1.5)</td>
<td>585 (23)</td>
<td>36.5 (8.2)</td>
</tr>
<tr>
<td></td>
<td>16(C)&lt;sup&gt;a&lt;/sup&gt;</td>
<td>4.0 (0.16)</td>
<td>50 (2)</td>
<td>200 (8)</td>
<td>79.7 (17.9)</td>
</tr>
<tr>
<td></td>
<td>17(C)&lt;sup&gt;a&lt;/sup&gt;</td>
<td>6.4 (0.25)</td>
<td>100 (4)</td>
<td>200 (8)</td>
<td>250.5 (56.3)</td>
</tr>
<tr>
<td></td>
<td>18(C)&lt;sup&gt;a&lt;/sup&gt;</td>
<td>6.4 (0.25)</td>
<td>100 (4)</td>
<td>455 (18)</td>
<td>245.6 (55.2)</td>
</tr>
</tbody>
</table>

<sup>a</sup> Only one specimen tested

It was observed that the ultimate strength of the concrete columns increased due to the confinement provided by plastic pipes (PVC and ABS). The strength of short columns increased 3.3 times the burst pressure of tubes, actual values ranged from 2.93 to 3.68. An increase in ductility was also observed. The increase in strength and ductility of
intermediate columns was less than that in short columns. No conclusions were drawn for long columns.

4.2.8 FARDIS M. N. and KHALILI H. (1981)[34]

Fardis and Khalili studied the mechanical behaviour of FRP-encased concrete in compression. The study was limited to concentrically loaded short circular columns. They performed compression tests on several 76 x 152 mm (3 x 6 in.) and 100 x 200 mm (4 x 8 in.) concrete cylinders, encapsulated by four different types of FRPs. The average value of unconfined compressive strength for 76 x 152 mm (3 x 6 in.) cylinders was 34.5 MPa and for the 100 x 200 mm (4 x 8 in.) was 31 MPa. Four different types of FRP, based on weight per unit area, were used as given below:

(1) A 0.339 kg/m² (10 oz/sq. yd) fibreglass cloth with same density of fibres in both directions
(2) A 0.8136 kg/m² (24 oz/sq. yd) unbalanced woven roving.
(3) A 0.4407 kg/m² (13 oz/sq. yd) unbalanced woven roving.
(4) A 0.5085 kg/m² (15 oz/sq. yd) unbalanced woven roving.

The specimen were cast in removable moulds and wrapped with FRP after curing. The wrapping technique used for the FRP can be seen elsewhere[34]. The FRP casing then stayed permanently on the column, thus confining the concrete.

It was observed that failure of specimens occurred when the lateral strain of confined concrete reaches the failure strain of the FRP in the circumferential direction. It was stated that the fracture of the FRP and concrete crushing occurred essentially simultaneously. The authors concluded that the FRP-encased concrete cylinders tested in concentric compression exhibit highly increased strength and ductility. The use of FRPs in concrete confinement was found to be very promising.

Figure 4.1 shows the axial stress-strain plots of FRP-encased 100 x 200 mm (4 x 8 in.) specimen.
**Figure 4.1** Axial Stress-Strain Plots of FRP Encased 100 x 200 mm (4 x 8 in.) Concrete Cylinders, FRP Type = 0.4407 kg/m² (13 oz/sq. yd) Unbalanced Woven Roving

4.2.9 AHMAD and SHAH (1982)\(^9\)

Ahmad and Shah studied the stress-strain curves of concrete confined with spiral reinforcement. The influence of compressive strength, aggregate type, and spacing and yield strength of hoop reinforcement was investigated. A model was proposed to predict the stress-strain relationship of confined concrete based on the properties of the confining reinforcement and constitutive relationship of plain concrete.

A total of ninety-six cylinder specimens were tested. Fifteen of the specimens were 75 x 300 mm (3 x 12 in.) cylinders while the remaining eighty-one specimens were 75 x 150 mm (3 x 6 in.) cylinders. Concrete compressive strengths (\(f'_c\)) ranged from 20.7 MPa to 65.5 MPa. The specimens were confined by steel wires with diameters ranging from 1.6 mm to 2.4 mm (1/16 in. to 3/32 in.) and with yield strengths between 413 MPa and 1433 MPa. The spirals were fabricated to have a pitch of 12 mm, 25 mm, and 38 mm (½ in., 1 in., and 1½ in.). Longitudinal reinforcement was not provided in the specimens.

Ahmad and Shah expressed the effectiveness of confinement as:

\[
\begin{align*}
    f_{oc} &= f_o + k_1 (f_r)_p \\
    \varepsilon_{oc} &= \varepsilon_o + k_2 (f_r)_p
\end{align*}
\]

where

\(f_{oc} = \text{peak stress of confined concrete}\)
The peak stress of unconfined concrete, $f_o$, equals the strain corresponding to peak stress of confined concrete, $\varepsilon_{oc}$, equals the strain corresponding to peak stress of unconfined concrete, $\varepsilon_o$, equals the average confining pressure at the peak due to the spirals, $(f_r)_p$, equals the constants $k_1$ and $k_2$.  

It was observed that the effect of confinement was negligible when spacing of spirals exceeded the distance equal to 1.25 times the diameter of the confined concrete ($d_{cc}$). Based on the above observation along with the usual equilibrium consideration, and assuming that the spiral steel yields at the peak of the stress-strain curve, the following equation was developed for calculating the value of $(f_r)_p$:

$$ (f_r)_p = \rho_s f_y / 2 (1 - \sqrt{(S_{sp}/1.25d_{cc})}) $$

where

- $\rho_s = 2\pi d_{sp}/(d_{cc}S_{sp})$ is the ratio of volume of spiral reinforcement to volume of confined concrete core
- $d_{sp}$ is the diameter of the spiral wire
- $d_{cc}$ is the diameter of confined concrete core
- $S_{sp}$ is the pitch of the spiral
- $f_y$ is the yield strength of the spiral wire

The following conclusions were drawn:

1. As compressive strength of unconfined concrete increases, effectiveness of the spirals at the peak decreases.
2. With the increase in compressive strength of unconfined concrete specimens, the slope of the descending region of the stress-strain curve becomes steeper for both the unconfined and confined specimens. However, the change in compressive strength does not affect the relative improvement in slope of identically confined specimens.
3. The confining reinforcement is less effective for lightweight concrete than for normal weight concrete of comparable strength and confinement.
4. The effects of using higher strength or lightweight concrete are different at the peak as compared to that on the descending region of the stress-strain curve.
5. For normal weight concrete, the following equations were developed for the values of $k_1$, $k_2$, $\theta$, and $\varepsilon_0$.

$$k_1 = 6.61(f_r)^{0.04}/f_o$$  \hspace{1cm} (4.13) \\
$$k_2 = 0.047(f_r)^{0.12}/f_o^{1.2}$$  \hspace{1cm} (4.14) \\
$$\theta = 6.6128 + 2.9137 (f_o) - 44.2315 (f_r)$$  \hspace{1cm} (4.15) \\
$$\varepsilon_0 = 0.001648 + 0.000114 f_o$$  \hspace{1cm} (4.16)

where $\theta$ = average value of the slope of the descending part between strain at peak and twice the strain at peak.

The rest of the parameters used have already been defined earlier.

An algorithm was also presented to generate stress-strain curves for a given spirally reinforced specimen. Theoretical curves were compared with experimental data from the investigation and were found to be in good agreement. It was finally stated that it is possible to accurately predict the complete stress-strain curve of confined concrete from the triaxial stress-strain curves of plain concrete and the tensile stress-strain curve of the confining reinforcement. Furthermore, the theoretical model showed that steel stresses in the spirals at the peak of the confined concrete's stress-strain curve were smaller with higher concrete compressive strength and were not influenced by the yield strength of the spiral wire for the same compressive strength.

4.2.10 FAFTTIS and SHAH (1985)[10]

Fafitis and Shah studied the behaviour of confined concrete and proposed a relationship for the stress-strain behaviour of reinforced circular and square concrete columns subjected to axial and lateral loading.

To predict the complete stress-strain curve of the confined core and the unconfined cover concrete, the following expressions were proposed:

for the ascending part

$$f = f_o[1-(1-\varepsilon/\varepsilon_o)^4]$$  \hspace{1cm} (4.17)

and for the descending part

$$f = f_o \exp[-k(\varepsilon - \varepsilon_o)^{1.15}]$$  \hspace{1cm} (4.18)

where
\[ f = \text{stress} \]
\[ \varepsilon = \text{strain} \]
\[ f_0 = \text{peak stress} \]
\[ \varepsilon_0 = \text{strain corresponding to peak stress} \]
\[ A = \text{parameter which determines the shape of the curve in the ascending part} \]
\[ k = \text{parameter which determines the shape of the curve in the descending part} \]

A and k were given by:

\[ A = E_c \varepsilon_0 / f_0 = \text{secant modulus at the peak} \quad (4.19) \]
\[ k = 0.17 f_c' \exp(-0.01f_c) \quad (4.20) \]
\[ f_r = 2A_s f_y / d \quad \text{(for circular core)} \quad (4.21) \]

where
\[ E_c = \text{tangent modulus of elasticity of plain concrete} \]
\[ f_c' = \text{confinement index} \]
\[ d = \text{diameter of the core} \]
\[ s = \text{spacing of the spiral hoops} \]
\[ A_s = \text{cross-sectional area of the spiral hoops} \]
\[ f_c' = \text{unconfined concrete strength} \]

The value of \( k = 0 \) corresponds to a horizontal descending part (perfectly plastic) while the value of \( k = \infty \) corresponds to a vertical descending part (perfectly brittle).

The following expressions for evaluating \( f_0 \) and \( \varepsilon_0 \) were determined from the statistical analysis of experimental data on 76 x 152 mm (3 x 6 in.) concrete cylinders reinforced with spirals at 13 mm, 25.4 mm, and 38 mm (½ in., 1 in., or 1½ in.):

\[ f_0 = f_c' + (1.15 + 3048/f_c')f_r \quad (4.22) \]
\[ \varepsilon_0 = 1.027 \times 10^{-7} f_c' + 0.0296 f_r / f_c' + 0.00195 \quad (4.23) \]

For verification of the proposed model, it was applied to the specimens of experiments performed earlier at the University of Canterbury (Priestly, Park and Poutangoroa, 1981[46] and Ghee, Priestley and Park, 1981[47]). The circular confined core of the columns had a diameter of 559 mm (22 in.) outside the spiral and a cover of 20 mm (0.8 in.). All four units had the same amount (2.8% by volume) of longitudinal reinforcement. The properties of the four units tested are given in Table 4.3.
Table 4.3 Properties of Specimens

<table>
<thead>
<tr>
<th>Unit</th>
<th>$f_c'$ (MPa)</th>
<th>Axial load (kN)</th>
<th>Lateral steel</th>
<th>Lateral pressure</th>
<th>$f_yh$ (MPa)</th>
<th>$f_r$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>28.4</td>
<td>97</td>
<td>10</td>
<td>75</td>
<td>300</td>
<td>1.12</td>
</tr>
<tr>
<td>3</td>
<td>26.6</td>
<td>217</td>
<td>10</td>
<td>50</td>
<td>300</td>
<td>1.68</td>
</tr>
<tr>
<td>4</td>
<td>32.9</td>
<td>191</td>
<td>10</td>
<td>70</td>
<td>423</td>
<td>2.38</td>
</tr>
<tr>
<td>5</td>
<td>32.5</td>
<td>171</td>
<td>16</td>
<td>55</td>
<td>280</td>
<td>3.65</td>
</tr>
</tbody>
</table>

The reported values of compressive and yield strengths of the spirals along with the cross-sectional properties were used to calculate the stress-strain curves of the core and cover for each unit. Similar studies were conducted for square columns as well. The authors concluded the proposed model to be satisfactory in predicting the ultimate loads and the behaviour of confined concrete columns.

4.2.11 MANDER, PRIESTLEY, and PARK (1988)\textsuperscript{[8]}

Experimental studies were carried out by Mander, Priestly, and Park to study the behaviour of confined concrete members and for comparison to the theoretical stress-strain model developed by Mander et al.\textsuperscript{[48]} in a companion paper. The model allowed for the effect of various configurations of transverse confining reinforcement, cyclic loading, and strain rate.

Thirty-one nearly full-size reinforced concrete columns of circular, square, or rectangular wall cross-section, and containing various arrangements of reinforcement, were loaded concentrically with axial compressive strain rates up to 0.0167/s.

The cylinders were of 500 mm (19.7 in.) diameter and 1500 mm (59.1 in.) height. Concrete strength of 28 MPa and slump of 75 mm was used. Grade 275 steel was used for longitudinal reinforcement, except for one column (column number 12) in which grade 380 steel was used. For spiral joints a lap of 200 mm and fillet weld of 150 mm was also used. The cylinders were loaded concentrically.

Table 4.4 and Figure 4.2 shows the details of the columns. The symbols D and R stand for Deformed bar and Round (plain) bar, respectively, and the following number is
the bar diameter in millimetres. Thus R12-52 means 12 mm diameter round at 52 mm pitch, whereas 12-D16 means 12-16 mm diameter-deformed bars.

Figure 4.2 Details of the Test Specimens
Table 4.4 Properties of Spirally Reinforced Circular Columns

<table>
<thead>
<tr>
<th>Test series</th>
<th>Unit</th>
<th>Vertical steel</th>
<th>Lateral steel</th>
<th>Longitudinal steel ratio$^a$</th>
<th>Transverse steel ratio</th>
<th>Material strength$^b$ (MPa)</th>
<th>Core diameter $d_0$ (mm)</th>
<th>Confinement effectiveness$^c$</th>
<th>Lateral confining pressure (MPa)</th>
<th>Testing strain rate ($s^{-1}$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pilot</td>
<td>a</td>
<td>12-D16</td>
<td>R12-52</td>
<td>0.0123</td>
<td>0.016</td>
<td>$f' = 28$</td>
<td>31</td>
<td>438</td>
<td>0.97</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td>b</td>
<td>12-D16</td>
<td>R12-52</td>
<td>0.0123</td>
<td>0.016</td>
<td>$f' = 31$</td>
<td>31</td>
<td>438</td>
<td>0.97</td>
<td>3.3</td>
</tr>
<tr>
<td></td>
<td>c</td>
<td>12-D16</td>
<td>R12-52</td>
<td>0.0123</td>
<td>0.016</td>
<td>$f' = 33$</td>
<td>33</td>
<td>438</td>
<td>0.97</td>
<td>3.3</td>
</tr>
<tr>
<td>1 Cyl 1</td>
<td>1</td>
<td>12-D16</td>
<td>R12-41</td>
<td>0.0123</td>
<td>0.016</td>
<td>$f' = 28$</td>
<td>28</td>
<td>438</td>
<td>0.983</td>
<td>4.18</td>
</tr>
<tr>
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<td>2</td>
<td>12-D16</td>
<td>R12-69</td>
<td>0.0123</td>
<td>0.016</td>
<td>$f' = 28$</td>
<td>28</td>
<td>438</td>
<td>0.95</td>
<td>2.42</td>
</tr>
<tr>
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<td>3</td>
<td>12-D16</td>
<td>R12-103</td>
<td>0.0123</td>
<td>0.016</td>
<td>$f' = 28$</td>
<td>28</td>
<td>438</td>
<td>0.911</td>
<td>1.55</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>12-D16</td>
<td>R10-119</td>
<td>0.0123</td>
<td>0.0159</td>
<td>$f' = 28$</td>
<td>28</td>
<td>440</td>
<td>0.89</td>
<td>0.85</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>12-D16</td>
<td>R10-36</td>
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<td>0.0159</td>
<td>$f' = 28$</td>
<td>28</td>
<td>440</td>
<td>0.986</td>
<td>3.14</td>
</tr>
<tr>
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<td>R16-93</td>
<td>0.0123</td>
<td>0.0163</td>
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<td>28</td>
<td>434</td>
<td>0.926</td>
<td>2.84</td>
</tr>
<tr>
<td>2 Cyl 2</td>
<td>7</td>
<td>8-D28</td>
<td>R12-52</td>
<td>0.0251</td>
<td>0.0327</td>
<td>$f' = 31$</td>
<td>31</td>
<td>438</td>
<td>0.987</td>
<td>3.35</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>11-D24</td>
<td>R12-52</td>
<td>0.0253</td>
<td>0.033</td>
<td>$f' = 31$</td>
<td>31</td>
<td>438</td>
<td>0.987</td>
<td>3.35</td>
</tr>
<tr>
<td></td>
<td>9</td>
<td>16-D20</td>
<td>R12-52</td>
<td>0.0256</td>
<td>0.0334</td>
<td>$f' = 31$</td>
<td>31</td>
<td>438</td>
<td>0.987</td>
<td>3.35</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>24-D16</td>
<td>R12-52</td>
<td>0.0246</td>
<td>0.032</td>
<td>$f' = 31$</td>
<td>31</td>
<td>438</td>
<td>0.986</td>
<td>3.34</td>
</tr>
<tr>
<td></td>
<td>11</td>
<td>36-D16</td>
<td>R12-52</td>
<td>0.0396</td>
<td>0.048</td>
<td>$f' = 31$</td>
<td>31</td>
<td>438</td>
<td>1.002$^d$</td>
<td>3.4</td>
</tr>
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<td>12</td>
<td>24-DH16</td>
<td>R12-52</td>
<td>0.0246</td>
<td>0.032</td>
<td>$f' = 31$</td>
<td>31</td>
<td>438</td>
<td>0.986</td>
<td>3.34</td>
</tr>
</tbody>
</table>

$^a$Based on gross section area

$^b$Based on core area.

$^c$At time of testing of units.

$^d$From companion paper by Mander et al.$^{[48]}$

$^e$k$_e$ may exceed 1.0 by definition when $\rho_{ce}$ is high

$^f$Dynamic cyclic loading

A pilot series of three columns (a, b, and c) was followed by two series of six confined columns, each with a companion unreinforced column (CYL 1 and CYL 2), thus enabling the stress-strain curve of unconfined concrete to be assessed from tests on unreinforced units of the same size as the confined units so as to avoid scale effects.

Series 1 had columns with identical longitudinal steel arrangements but different amount and sizes of transverse spiral reinforcement, resulting in volumetric ratios of confining reinforcement ($\rho_s$) between 0.006 and 0.025. Series 2 column units had identical transverse reinforcement, but different amounts and sizes of longitudinal reinforcement.
Table 4.5 compares the theoretical behaviour predicted by the stress-strain model described in the companion paper by Mander et al.\textsuperscript{[48]} and the experimental behaviour measured in the tests for the circular columns. It was obvious that the most important parameter affecting the shape of the stress-strain curve of confined concrete was the quantity of confining reinforcement. As the volumetric ratio of confining reinforcement increased, the strength developed increased, the slope of the falling branch decreased, and the longitudinal strain at which hoop fracture occurred increased.

<table>
<thead>
<tr>
<th>Unit</th>
<th>Plain concrete data</th>
<th>Confined strength</th>
<th>Confined strain</th>
<th>Strain at hoop fracture</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>£c' £c'' £c∞</td>
<td>£c</td>
<td>Exp</td>
<td>Theo</td>
</tr>
<tr>
<td></td>
<td>MPa</td>
<td>MPa</td>
<td>GPa</td>
<td>MPa</td>
</tr>
<tr>
<td>a</td>
<td>30</td>
<td>24</td>
<td>0.002</td>
<td>24</td>
</tr>
<tr>
<td>b</td>
<td>31</td>
<td>30</td>
<td>0.0015</td>
<td>31</td>
</tr>
<tr>
<td>c</td>
<td>33</td>
<td>32</td>
<td>0.0015</td>
<td>32</td>
</tr>
<tr>
<td>Cyl 1</td>
<td>28</td>
<td>29</td>
<td>0.0015</td>
<td>26</td>
</tr>
<tr>
<td>L</td>
<td>28</td>
<td>29</td>
<td>0.0015</td>
<td>26</td>
</tr>
<tr>
<td>2</td>
<td>28</td>
<td>29</td>
<td>0.0015</td>
<td>26</td>
</tr>
<tr>
<td>3</td>
<td>28</td>
<td>29</td>
<td>0.0015</td>
<td>26</td>
</tr>
<tr>
<td>4</td>
<td>28</td>
<td>29</td>
<td>0.0015</td>
<td>26</td>
</tr>
<tr>
<td>5</td>
<td>28</td>
<td>29</td>
<td>0.0015</td>
<td>26</td>
</tr>
<tr>
<td>6</td>
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<td>12</td>
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<td>32</td>
<td>0.0014</td>
<td>28</td>
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</tbody>
</table>

\textsuperscript{a}Theoretical values computed from equations given in companion paper by Mander et al.\textsuperscript{[48]}

\textsuperscript{b}Average =1.017

\textsuperscript{c}Average =0.987

\textsuperscript{d}Average = 1.095

It was also concluded that the influence of the configuration of transverse reinforcement could be predicted through the confinement effectiveness coefficient $k_c$. 

35
that ranged between 0.4 to 0.7 for rectangular sections and 0.89 to 1.0 for the circular sections.

The theoretical model for circular concrete column confined by transverse reinforcement and subjected to uniaxial compression loading developed by the author in a companion paper\cite{48} gave the following expressions:

\[ f'_{cc} = f'_{co} \left( -1.254 + 2.254 \sqrt{1 + 7.94f'/f_{co}'} - 2f'/f_{co}' \right) \]  \hspace{1cm} (4.24)

\[ \varepsilon_{cc} = \varepsilon_{co} \left( 1 + 5(f'_{cc}' / f_{co}' - 1) \right) \]  \hspace{1cm} (4.25)

\[ f'_l = k_c \rho_s f_{yh}/2 \]  \hspace{1cm} (4.26)

\[ k_c = (1 - s'/2d_s)/(1 - \rho_{cc}) \]  \hspace{1cm} (4.27)

where

\[ f'_{cc} = \] maximum concrete stress
\[ f'_{co} = \] unconfined concrete compressive strength
\[ f'_l = \] effective lateral confining pressure
\[ \varepsilon_{cc} = \] strain corresponding to \( f'_{cc} \)
\[ \varepsilon_{co} = \] strain corresponding to \( f'_{co} \)
\[ k_c = \] confinement effectiveness coefficient
\[ s' = \] clear vertical spacing between spiral or hoop bars
\[ d_s = \] diameter of spiral between bar centres
\[ \rho_{cc} = \] ratio of area of longitudinal steel to area of core of section
\[ \rho_s = \] ratio of the volume of transverse confining steel to the volume of confined concrete core
\[ f_{yh} = \] yield strength of the transverse reinforcement

It was concluded that the analytical stress-strain model proposed by the author in the companion paper was found to give good predictions of experimental behaviour of the columns with different configurations.

### 4.2.12 SAATCIOGLU and RAZVI (1992)\cite{11}

Saatcioglu and Razvi in 1992 presented an analytical model to construct a stress-strain relationship for confined concrete. It was based on calculations of lateral confining
pressure generated by circular or rectilinear reinforcement, the resulting improvements in strength, and ductility of confined concrete.

They used the following expression to express triaxial strength of concrete in terms of uniaxial strength and lateral pressure:

\[ f_{cc} = f_{co} + k_1 f_l \]  \hspace{1cm} (4.28)

where

- \( f_{cc} \) = confined strength of concrete
- \( f_{co} \) = unconfined strength of concrete
- \( f_l \) = lateral pressure
- \( k_1 \) = a function of the Poisson’s ratio which may vary with loading due to material non-linearity.

The variation of coefficient \( k_1 \) with lateral pressure \( f_l \) was obtained from experimental data. Figure 4.3 shows experimental data obtained from specimens subjected to different levels of hydrostatic pressure (Richart et al.[2] 1928). It was observed that at higher values of lateral pressure, \( k_1 \) decreased, approaching a constant value in the high-pressure range. Expression for \( k_1 \) obtained from regression analysis of test data was given as:

\[ k_1 = 6.7 (f_l)^{-0.17} \]  \hspace{1cm} (4.29)

where

- \( f_l \) = uniform confining pressure in MPa.

\[ \text{Figure 4.3 Variation of Co-Efficient } k_1 \text{ with Lateral Pressure} \]
It was stated that a constant value of 4.1 for $k_1$, as taken by Richart et al. 1928\cite{2}, produced a good correlation with spirally reinforced test cylinders. For circular sections, the lateral pressure was found from statics as shown in Figure 4.4.

![Figure 4.4 Lateral Pressure in Circular Columns](image)

These equations were used to predict confined concrete strengths of fifteen circular columns (tested earlier by Mander et al.\cite{48}) reinforced with spiral and longitudinal steel. Both slow and fast rates of concentric loading were applied to the columns. The results are shown in Table 4.6. On comparison a good agreement between experimental and analytical values can be observed. Similarly, equations were developed for square and rectangular sections.
Table 4.6 Strength Enhancements in Circular Columns

<table>
<thead>
<tr>
<th>Column Label</th>
<th>$f_t$ (MPa)</th>
<th>$k_t$</th>
<th>$f_{co}'$ (MPa)</th>
<th>$f_{cc}'$ exp (MPa)</th>
<th>$f_{cc}'$ ana (MPa)</th>
<th>$f_{cc}'$ana/$f_{cc}'$exp (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>a</td>
<td>3.08</td>
<td>5.53</td>
<td>24.0</td>
<td>38.0</td>
<td>41.0</td>
<td>1.08</td>
</tr>
<tr>
<td>b</td>
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<td>5.45</td>
<td>30.0</td>
<td>48.0</td>
<td>48.4</td>
<td>1.01</td>
</tr>
<tr>
<td>c</td>
<td>3.38</td>
<td>5.45</td>
<td>32.0</td>
<td>47.0</td>
<td>50.4</td>
<td>1.07</td>
</tr>
<tr>
<td>d</td>
<td>4.28</td>
<td>5.23</td>
<td>29.0</td>
<td>51.0</td>
<td>51.4</td>
<td>1.01</td>
</tr>
<tr>
<td>2</td>
<td>2.54</td>
<td>5.72</td>
<td>29.0</td>
<td>46.0</td>
<td>43.5</td>
<td>0.95</td>
</tr>
<tr>
<td>3</td>
<td>1.70</td>
<td>6.12</td>
<td>29.0</td>
<td>40.0</td>
<td>39.4</td>
<td>0.99</td>
</tr>
<tr>
<td>4</td>
<td>0.96</td>
<td>6.75</td>
<td>29.0</td>
<td>36.0</td>
<td>35.5</td>
<td>0.99</td>
</tr>
<tr>
<td>5</td>
<td>3.17</td>
<td>5.51</td>
<td>29.0</td>
<td>47.0</td>
<td>46.5</td>
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<tr>
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<td>1.00</td>
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<td>5.45</td>
<td>32.0</td>
<td>52.0</td>
<td>50.4</td>
<td>0.97</td>
</tr>
<tr>
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<td>3.38</td>
<td>5.45</td>
<td>30.0</td>
<td>49.0</td>
<td>48.4</td>
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<tr>
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<td>52.0</td>
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<td>48.4</td>
<td>0.97</td>
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<td>32.0</td>
<td>52.0</td>
<td>50.4</td>
<td>0.97</td>
</tr>
</tbody>
</table>

Column "a" was tested under slow strain rate, and all others under high strain rate of 0.0133/s.

It was reported that the strain at peak stress is dependent on the effectiveness of confinement. The following expressions were presented:

$$\varepsilon_t = \varepsilon_{01} \left( 1 + 5K \right) \quad (4.30)$$

$$K = k_t f_{le}/f_{co}' \quad (4.31)$$

where

$\varepsilon_t =$ strain corresponding to peak stress of confined concrete

$\varepsilon_{01} =$ strain corresponding to peak stress of unconfined concrete

$f_{le} =$ overall equivalent lateral pressure

$f_{co}' =$ unconfined concrete strength
\[ k_1 = \text{a function of the Poisson's ratio which may vary with loading due to material non-linearity.} \]

The stress-strain curve proposed for confined concrete was parabolic for the ascending branch and linear for descending branch up to 20% of the peak strength. For the ascending portion of the stress-strain curve the following equation was suggested:

\[
f_{c} = f_{cc}' \left[ 2(\varepsilon_{c}/\varepsilon_{1}) - (\varepsilon_{c}/\varepsilon_{1})^2 \right]^{1/(1+2K)} \leq f_{cc}'
\]

where

\[ f_{cc}' = \text{confined concrete strength} \]
\[ \varepsilon_{1} = \text{strain corresponding to peak stress of confined concrete} \]
\[ \varepsilon_{1} = \text{strain corresponding to } f_{c} \]
\[ K \text{ as defined earlier by equation (4.31)} \]

The analytical relationship was compared with a large volume of experimental data, covering a wide range of confinement parameters and different geometry of sections and reinforcements. The comparisons indicated good correlation between the analytical and experimental results.

4.2.13 SHEIKH AND TOLKUCU (1993)\[49\]

The objective of the study by Sheikh and Toklucu was to investigate the effects of different variables, such as amount and type of confinement, lateral steel spacing, and specimen size on the behaviour of circular columns. They also examined the relationship between lateral pressure on concrete and concrete strength enhancement, and the variation of spiral steel stress and confinement effectiveness co-efficient (k) with respect to the amount of spiral steel.

The experimental program consisted of twenty-seven specimens, divided into three sets, based on dimensions, with nine specimens in each set. First set had specimens of the dimensions 356 x 1425 mm (14 x 56 in.), second set consisted of columns of size 254 x 1016 mm (10 x 40 in.), and the third set had specimens with the dimensions 203 x 812 mm (8 x 32 in.). These columns were reinforced with spirals or hoops and longitudinal steel. Five deformed 15M, 20M, and 25M longitudinal bars were used in 203
mm, 254 mm, and 356 mm diameter columns respectively. Specified 28-day compressive strength of concrete was 35 MPa. One specimen of each size was laterally reinforced with circular hoops while all the others were spirally reinforced. The columns were tested under monotonic concentric compression. Failure of the specimens was forced to occur in the central test region by reducing the spacing of lateral steel outside the test region and also by providing steel collars in the end regions.

Electric strain gauges were used to measure strains in the steel, while LVDTs (Linear Variable Differential Transducers) were used to measure axial deformation of the central region of the columns.

It was concluded that the strength and ductility of confined concrete increases with an increase in the amount of lateral steel, the increase in ductility being more pronounced than the increase in strength. A reduction in $S/D_c$ ratio (the ratio between spiral or hoop spacing ‘$S$’ and the core diameter ‘$D_c$’) resulted in improvement of ductility of the columns. Closely spaced spirals showed better ductility than the widely spaced ones. The increase in concrete strength due to confinement was observed to be between 2.1 and 4.0 times the lateral pressure. Table 4.7 shows some details of the specimens and selected results.
### Table 4.7 Specimen Details and Selected Results

<table>
<thead>
<tr>
<th>Spec No.</th>
<th>Spiral provided</th>
<th>Spiral steel ratio $\rho_s$</th>
<th>Peak strength enhanc. $f_{\text{c}}/0.85f_{\text{c}}$</th>
<th>Strain at $f_{\text{c}}$, max $\varepsilon_{\text{c}, \text{max}}$</th>
<th>Spiral stress $\sigma$ (MPa)</th>
<th>Spiral strength $f_{\text{rr}}$ (MPa)</th>
<th>Lateral pressure $\sigma_2$ (MPa)</th>
<th>Strain at 0.85$f_{\text{c, \text{max}}}$ $\varepsilon_{\text{c}, \text{max}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>10M @ 56mm</td>
<td>0.0230</td>
<td>1.70</td>
<td>0.0133</td>
<td>574</td>
<td>452</td>
<td>6.89</td>
<td>452</td>
</tr>
<tr>
<td>2</td>
<td>10M @ 76mm</td>
<td>0.0169</td>
<td>1.59</td>
<td>0.0179</td>
<td>536</td>
<td>452</td>
<td>4.72</td>
<td>0.0374</td>
</tr>
<tr>
<td>3</td>
<td>10M @ 112mm</td>
<td>0.0115</td>
<td>1.36</td>
<td>0.0036</td>
<td>452</td>
<td>452</td>
<td>2.71</td>
<td>0.0018</td>
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<td>0.0030</td>
<td>77</td>
<td>452</td>
<td>0.31</td>
<td>0.0057</td>
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<td>607</td>
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<td>452</td>
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<td>0.0227</td>
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<td>580</td>
<td>607</td>
<td>3.54</td>
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<td>607</td>
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<td>593</td>
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<td>593</td>
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<td>0.0145</td>
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<tr>
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<td>607</td>
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<td>5.61</td>
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<td>629</td>
<td>5.61</td>
<td>-</td>
</tr>
<tr>
<td>26</td>
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<td>0.0034</td>
<td>400</td>
<td>605</td>
<td>2.52</td>
<td>0.0156</td>
</tr>
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<td>27</td>
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<td>400</td>
<td>605</td>
<td>2.52</td>
<td>0.0156</td>
</tr>
</tbody>
</table>

* Prefix H refers to hoop confined specimen while all remaining are spirally confined

### 4.2.14 NANNI, NORRIS, and BRADFORD (1993)\[^4\]

An experimental and analytical study of concrete strengthened with FRP confinement was conducted. Three groups of concrete specimens were used [Group A (150 x 300 mm), Group B (150 x 600 mm), and Group C (150 x 1525 mm)]. Groups A and B were used for uniaxial compression tests. Group C specimens were used for cyclic...
Flexural test with and without axial compression. 28-day compressive strength of concrete used was 35.6 MPa. A deformed wire of size D8 (8.1 mm diameter) and yield strength of 516 MPa was used as longitudinal reinforcement. Welded wire mesh (2 mm wire diameter and 25 x 25 mm wire spacing) was used as transverse ties for some column-type specimens.

Braided Aramid fibre (Kelvar 49, Grade 6000 Denvier) was used for first type of lateral external reinforcement. The epoxy resin was Dow Chemical DER 330 combined with hardener Ancamide 506 by Pacific Anchor Chemical. Three tape sizes K24, K48, and K64 with corresponding fibre area of 10.8 mm², 21.7 mm², and 28.9 mm² were used. The nominal cross-section areas after impregnation and flattening were 19 mm², 39 mm², and 52 mm². For lateral reinforcement, a pretensioning force of 0.2 kN was used which was sufficient to ensure acceptable strength and stiffness of the tape. The procedure adopted to attain the pretensioning force can be seen in detail elsewhere\[14]. The nominal load capacity of the three tape sizes pretensioned at 0.2 kN was 21 kN, 42 kN, and 56 kN.

Twelve concrete specimens of 600 mm length were laterally confined by filament winding. In this case, the FRP shell was made of E-Glass strands impregnated with a polyester resin. The strand was helically wound around the specimen mounted on a rotating mandrel. The winding angle was 65°. The thicknesses of FRP shells used were 0.6 mm, 1.2 mm, 2.4 mm, and 3.6 mm.

It was observed that a considerable amount of FRP, provided as a confining material, significantly improved the behaviour of the specimens. It was concluded that the compressive strength and the ductility of concrete increased by the presence of FRP confinement.

4.2.15 SAADATMANESH, EHSANI, and LI (1994)\[35]

Saadatmanesh, Ehsani, and Li studied the gain in strength and ductility of concrete columns externally confined by means of high-strength fibre composite straps. They conducted a parametric study to examine the effects of various design parameters such as concrete compressive strength, thickness and spacing of straps, and types of straps.
Figure 4.5 Confinement Details and Confining Action of Composite Strap
An approach similar to the one used by Sheikh [17] and Mander, Priestly, and Park [8] was adopted to determine the effective lateral confining pressure, which is given by:

\[ f_i' = f_i k_e \]  \hspace{1cm} (4.33)

\[ k_e = A_e / A_{cc} \]  \hspace{1cm} (4.34)

where

- \( f_i \) = lateral pressure from transverse reinforcement, in this case the composite strap
- \( k_e \) = confinement effectiveness coefficient
- \( A_e \) = area of effectively confined concrete
- \( A_{cc} \) = effective area of concrete enclosed by composite belts given by:

\[ A_{cc} = A_e (1 - \rho_{cc}) \]  \hspace{1cm} (4.35)

\( \rho_{cc} \) = ratio of area of longitudinal reinforcement to gross area of concrete
- \( A_c \) = area of concrete enclosed by composite strap

From the equilibrium of forces as shown in Figure 4.5:

\[ f_i = 1/2 \rho_{st} f_{us} \]  \hspace{1cm} (4.36)

where

- \( \rho_{st} \) = ratio of volume of composite belt to the volume of confined concrete core
- \( f_{us} \) = tensile strength of composite belt

Assuming that an arching action in the form of a second-degree parabola with an initial slope of 45° occurs in the clear area between successive belts, then:

\[ A_e = (\pi/4)d_s^2(1-s'/2d_s)^2 \]  \hspace{1cm} (4.37)

where

- \( s' \) = clear vertical spacing between belts
- \( d_s \) = diameter of column.

Thus:

\[ k_e = (1-s'/2d_s)^2/(1-\rho_{cc}) \]  \hspace{1cm} (4.38)

A similar approach was used for rectangular cross-sections. Two types of composite belts, E-glass fibre reinforced and carbon fibre reinforced, were used in the study. The modulus of elasticity and tensile strength of glass and carbon fibre belts were 48.2 GPa (7000 ksi), 1103 MPa (160 ksi); and 172 GPa (25000 ksi), 2862 MPa (415 ksi), respectively. Figure 4.6 (a) and (b) show the stress-strain curves of unconfined concrete.
and concrete confined with 5 mm, 10 mm, and 15 mm thick E-glass and carbon fibre belts, respectively. These were obtained from models discussed above for a 1524 mm diameter circular column. Figure 4.7 shows the cross-section of the circular column. Unconfined concrete strength of 34.4 MPa and Grade 60 steel rebars were used as longitudinal reinforcement. The column was fully confined with 10 mm thick composite belts. It was observed that there was a significant increase in the strength and ductility of the columns as compared to the unconfined columns. The carbon fibre belt resulted in higher strength and ductility gain as compared with values for the E-glass belt. It was concluded that the strengthening method by external reinforcement using FRPs could prove useful to increase the strength and ductility of concrete columns.

![Stress-Strain Models](image)

Figure 4.6 Stress-Strain Models of Unconfined and Confined Concrete for Circular Column
4.2.16 MIRMIRAN A. and SHAHWY M. (1997)[50]

Mirmiran and Shahawy in 1997 studied the behaviour of concrete columns confined by fibre composites. They showed that the existing confinement models for steel generally overestimate the strength of FRP confined concrete columns, and thus result in unsafe design for FRP reinforced concrete columns. Therefore, the models developed for confinement of concrete by steel may not be accurately applicable to FRP confined concrete columns.

A total of thirty 152.5 x 305 mm (6 x 12 in.) cylinders were tested. Twenty-four of which were concrete filled FRP tubes while six were plain concrete specimens. Table 4.8 shows the details of the test program.
Table 4.8 Test Program and Properties of Test Specimens

<table>
<thead>
<tr>
<th>Batch number</th>
<th>Average concrete strength (MPa)</th>
<th>Total number of specimens</th>
<th>Mix proportions (by weight)</th>
<th>FRP tube thickness (mm)</th>
<th>Number of layers</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>C</td>
<td>S</td>
<td>G</td>
</tr>
<tr>
<td>A</td>
<td>30.9</td>
<td>11 (9F, 2P)</td>
<td>1.0</td>
<td>1.8</td>
<td>3.52</td>
</tr>
<tr>
<td>B</td>
<td>29.6</td>
<td>11 (9F, 2P)</td>
<td>1.0</td>
<td>1.55</td>
<td>3.12</td>
</tr>
<tr>
<td>C</td>
<td>32.0</td>
<td>8 (6F, 2P)</td>
<td>1.0</td>
<td>1.66</td>
<td>3.30</td>
</tr>
</tbody>
</table>

Note: C = cement, F = FRP-encased concrete, P = Plain (unconfined) concrete, S = sand, G = gravel, W = water.

Glass fibre with single strand tensile strength of 1420 MPa and elongation of 2.65% was used. Hoop strength of FRP tubes was 524 MPa, 579 MPa, and 641 MPa for 6, 10, and 14-layered tubes, respectively. Modulus of elasticity was 37233 MPa, 40336 MPa, and 40749 MPa for 6, 10, and 14-layered tubes, respectively. Interaction between the jacket and concrete in axial direction was prevented by cutting a 5 mm (3/16 in.) thick groove through the thickness of the tube on both sides of the specimen. The groove was at about 19 mm (3/4 in.) from the end surface and ran through the entire perimeter of the jacket. Various arrangements of strain gauges were used to record all possible strain readings.

A significant increase in strength and ductility of concrete was observed for FRP reinforced columns. The researchers applied different existing confining models for concrete confined by steel on the FRP-confined specimens and compared the results with the experimental observations. They concluded that the available confining models produce acceptable results for steel-encased concrete, but overestimate the strength of FRP-confined concrete. And thus they emphasised the need of developing new models specifically for FRP-confined concrete columns.


Saafi, Toutanji, and Li performed tests to investigate the performance of concrete columns confined with prefabricated carbon and glass FRP composite tubes which also acted as formwork.
A total of thirty concrete cylinders were tested. Eighteen of the specimens were confined with FRP while twelve were plain concrete columns. The specimens, 152.4 mm in diameter and 435 mm in length, were short columns with a length to diameter ratio of 2.85. Table 4.9 gives the properties of FRP tubes used. Specimens encased by GFRP of thickness 0.8 mm, 1.6 mm, and 2.4 mm were designated as GE₁, GE₂, and GE₃ respectively, whereas those encased by CFRP of thickness 0.11 mm, 0.23 mm, and 0.55 mm were designated as C₁, C₂, and C₃, respectively.

**Table 4.9 Mechanical and Physical Properties of Composites**

<table>
<thead>
<tr>
<th>Spec.</th>
<th>Number of specimens</th>
<th>Thickness, t (mm)</th>
<th>Hoop strength (MPa)</th>
<th>Modulus of elasticity (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>GE₁</td>
<td>3</td>
<td>0.8</td>
<td>450</td>
<td>32</td>
</tr>
<tr>
<td>GE₂</td>
<td>3</td>
<td>1.6</td>
<td>505</td>
<td>34</td>
</tr>
<tr>
<td>GE₃</td>
<td>3</td>
<td>2.4</td>
<td>560</td>
<td>36</td>
</tr>
<tr>
<td>C₁</td>
<td>3</td>
<td>0.11</td>
<td>3300</td>
<td>367</td>
</tr>
<tr>
<td>C₂</td>
<td>3</td>
<td>0.23</td>
<td>3550</td>
<td>390</td>
</tr>
<tr>
<td>C₃</td>
<td>3</td>
<td>0.55</td>
<td>3700</td>
<td>415</td>
</tr>
</tbody>
</table>

The average value of 28-day compressive strength of concrete used was 38 MPa and modulus of elasticity was 30000 MPa.

The specimens were tested under concentric compression using a 300 kip testing machine. Lateral and longitudinal strains were measured by strain gauges installed on the outer surface at mid height of the specimens. Two LVDTs were attached to steel block that transferred the load from the machine to concrete. Table 4.10 summarises the results of the tests.
It was observed that FRP-confined concrete columns show higher strength, ductility, and energy absorption capacity as compared to unconfined concrete. In comparison, the CFRP confined columns showed greater increase in strength while the GFRP confined column showed a greater increase in strain.

The researchers also developed a stress-strain model for concrete filled FRP tubes. The model predicted results comparable with the experimental results. But the proposed model was limited to circular short columns, axial loading, and fibre orientation of $0^\circ$ and $15^\circ$ with respect to the perpendicular axis of the column tubes. The following equations were developed for ultimate compressive strength and the corresponding axial strain:

\[
 f_{cc}' = f_c' [1 + 2.2(2t f_{com}/df_c')^{0.84}] \quad (4.39)
\]

\[
 \varepsilon_{cc} = \varepsilon_{co}[1 + (537\varepsilon_{com} + 2.6)(f_{cc}'/f_c' - 1)] \quad (4.40)
\]

where

- $f_{cc}'$ = maximum strength of confined concrete
- $\varepsilon_{cc}$ = strain corresponding to $f_{cc}'$
- $f_c'$ = unconfined concrete strength
- $f_{com}$ = tensile strength of the composite material
- $t$ = thickness of composite material

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Ultimate strength (MPa)</th>
<th>Ultimate axial strain (%)</th>
<th>Increase in strength (%)</th>
<th>Increase in axial strain (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plain</td>
<td>35</td>
<td>0.25</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>GE₁</td>
<td>52.8</td>
<td>1.9</td>
<td>51</td>
<td>660</td>
</tr>
<tr>
<td>GE₂</td>
<td>66</td>
<td>2.47</td>
<td>89</td>
<td>888</td>
</tr>
<tr>
<td>GE₃</td>
<td>83</td>
<td>3</td>
<td>137</td>
<td>1100</td>
</tr>
<tr>
<td>C₁</td>
<td>55</td>
<td>1</td>
<td>57</td>
<td>300</td>
</tr>
<tr>
<td>C₂</td>
<td>68</td>
<td>1.6</td>
<td>94</td>
<td>540</td>
</tr>
<tr>
<td>C₃</td>
<td>97</td>
<td>2.22</td>
<td>177</td>
<td>788</td>
</tr>
</tbody>
</table>

Table 4.10 Experimental Results
\[ d = \text{diameter of concrete core} \]
\[ \varepsilon_{co} = \text{strain at failure of unconfined concrete} \]
\[ \varepsilon_{com} = \text{ultimate strain of the FRP tube} \]

It was concluded that the FRP tube confinement increases both the strength and ductility of the concrete columns. The rate of increase depended on the thickness and properties of FRP. It was also observed that the stress-strain response of FRP confined columns is bilinear. The first slope of the response depended on concrete core, while the second depended on FRP. The point of change in slope was at stress levels slightly higher than the unconfined strength of the concrete core. A comparison was made between the experimental results and the analytical results obtained from various available models, and it was observed that available models generally overestimated the strength of concrete confined by FRP tubes.

4.3 SUMMARY

This chapter reviews the experimental and analytical work of various researchers on confinement of circular columns under axial loading. The confinement was provided by lateral steel or composite materials including glass and carbon composites.
CHAPTER 5

EXPERIMENTAL PROGRAM

5.1 GENERAL

The main objective of this experimental program was to categorically assess the suitability of GFRP as a potential confining material. For this purpose an integrated framework was followed which incorporated characteristics of materials and configurations of specimens quantitatively.

Details of the experimental phase of this particular study are given in this chapter. The properties of materials used, characteristics of specimens, construction phase, instrumentation of test specimens, test setup, and testing procedure adopted during the course of the study are presented.

5.2 MATERIAL PROPERTIES

5.2.1 CONCRETE

Ready mix concrete with a 28-day nominal compressive strength of 30 MPa was used for construction of the specimens. Nineteen 150 x 300 mm (6 x 12 in.) concrete cylinders were cast and tested at various ages after casting. Concrete strength versus age relationship and typical concrete stress versus strain curves were obtained from the standard cylinder tests and are shown in Figure 5.1 and Figure 5.2, respectively. The 28-day concrete strength measured from the cylinder tests ranged between 29.0 MPa and 30.3 MPa with an average value of 29.8 MPa. Longitudinal compressive strain at peak stress varied between 0.0018 and 0.0019.
Figure 5.1 Average Concrete Strength versus Age

Figure 5.2 Typical Stress-Strain Curves for Concrete used in the Experimental Program
5.2.2 REINFORCING STEEL

Properties of the longitudinal steel and the lateral steel used for test specimens (see Section 5.3 for specimen details) are provided in Tables 5.1 which include values of reinforcing bar diameter, cross-sectional area, yield stress, yield strain, modulus of elasticity, and ultimate stress. 20M bars were used as longitudinal steel whereas #3 steel was used for lateral reinforcement. Typical stress-strain curves for the longitudinal bars and lateral steel are shown in Figure 5.3.

<table>
<thead>
<tr>
<th>Bar type</th>
<th>Bar diameter (mm)</th>
<th>Bar area (mm$^2$)</th>
<th>Yield stress $f_y$ (MPa)</th>
<th>Yield strain $\varepsilon_y$</th>
<th>Modulus of elasticity $E$</th>
<th>Ultimate stress $f_u$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>20M</td>
<td>19.5</td>
<td>300</td>
<td>402</td>
<td>0.002102</td>
<td>192000</td>
<td>588</td>
</tr>
<tr>
<td>#3</td>
<td>9.5</td>
<td>71</td>
<td>500</td>
<td>0.00255</td>
<td>196700</td>
<td>781</td>
</tr>
</tbody>
</table>

**Figure 5.3 Stress-Strain Curves for Steel**
5.2.3 GLASS FIBRE REINFORCED POLYMERS (GFRP)

Prefabricated shells made of GFRP from TYFO™ S Fibrewrap System were used as confining material for specimens. TYFO™ S Fibrewrap System consists of high strength fibres embedded in a specially designed epoxy matrix. The GFRP shells, with varying volume and configuration/orientation of fibres, also acted as formwork for the columns. The thickness of one layer of GFRP shell was about 1 mm. The diameter of the shells was the same as that of the column specimens i.e. 355.6 mm.

Eight coupons made of the same material used for the GFRP shells were tested for tensile strength. Table 5.2 gives some details of the coupon tests results whereas a typical force per unit width versus strain curve for the GFRP material used is shown in Figure 5.4.

<table>
<thead>
<tr>
<th>Table 5.2 Selected Details of the GFRP Coupon Test Results</th>
</tr>
</thead>
<tbody>
<tr>
<td>No. of GFRP coupons</td>
</tr>
<tr>
<td>---------------------</td>
</tr>
<tr>
<td>8</td>
</tr>
</tbody>
</table>
5.3 TEST SPECIMENS

A total of seventeen circular columns of dimensions 355.6 x 1524 mm (14 x 60 in.) were constructed and tested under monotonic concentric axial compression. All the columns differed in configuration from each other in terms of number of GFRP layers in the shells, direction/orientation of the GFRP layers and the amount of longitudinal steel or lateral steel. Table 5.3 provides the details of the specimens.

The alphanumeric characters in the names of the specimens denote a code giving information about their configuration. The first letter “G” represents the presence of glass fibres, omitted when there are no fibres. The digit that comes just after the letter “G” represents the number of layers of glass fibres in the longitudinal direction, the second digit represents the number of layers of GFRP in the lateral direction. If the fibres are oriented at 45°, then it is indicated right after “G” followed by the number of layers of glass fibres. The letter “L” after the numbers indicates the presence of longitudinal steel. “0” replaces “L” for specimens without longitudinal steel. The next letter “S” represents
transverse steel while “0” if transverse steel is not provided. This is followed by the spacing in “mm” of the lateral steel. Finally, the last number indicates the specimen number. Hence G01-LS320-10 refers to no glass fibre in longitudinal direction, 1 layer of glass fibre in lateral direction, longitudinal reinforcement with lateral steel at a spacing of 320 mm, and the specimen number 10. Specimens 1 and 2 were plain concrete specimens without any GFRP shell. These specimens were reinforced with only one longitudinal bar in the centre for the purpose of handling the specimen safely after testing. In the designation of these two specimens, the number ‘1’ denoting only one longitudinal bar in the specimen replaces the alphabet ‘L’ for longitudinal steel. Hence specimen 1 is designated as 00-10-1, which refers to no glass fibres, one longitudinal bar, no lateral steel, and specimen number 1.

**Table 5.3 Specimen Details**

<table>
<thead>
<tr>
<th>Sr No.</th>
<th>Specimen designation</th>
<th>Lateral steel</th>
<th>Longitudinal steel</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Size (US#)</td>
<td>Spacing (mm)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>ρs %</td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>00-10-1</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>2</td>
<td>00-10-2</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>3</td>
<td>00-LS320-3</td>
<td>#3</td>
<td>320</td>
</tr>
<tr>
<td>4</td>
<td>00-LS320-4</td>
<td>#3</td>
<td>320</td>
</tr>
<tr>
<td>5</td>
<td>00-LS75-5</td>
<td>#3</td>
<td>75</td>
</tr>
<tr>
<td>6</td>
<td>00-LS75-6</td>
<td>#3</td>
<td>75</td>
</tr>
<tr>
<td>7</td>
<td>G01-00-7</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>8</td>
<td>G11-00-8</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>9</td>
<td>G01-L0-9</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>10</td>
<td>G01-LS320-10</td>
<td>#3</td>
<td>320</td>
</tr>
<tr>
<td>11</td>
<td>G02-00-11</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>12</td>
<td>G22-00-12</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>13</td>
<td>G02-L0-13</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>14</td>
<td>G02-LS320-14</td>
<td>#3</td>
<td>320</td>
</tr>
<tr>
<td>15</td>
<td>G02-LS75-15</td>
<td>#3</td>
<td>75</td>
</tr>
<tr>
<td>16</td>
<td>G45°2-00-16</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>17</td>
<td>G45°2-L0-17</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

ρs = volumetric ratio of lateral steel to concrete core

ρl = ratio of area of longitudinal steel to that of cross-section

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5.4 CONSTRUCTION OF THE SPECIMENS

Ten reinforcement cages with different configuration were constructed. Four cages consisted of longitudinal bars and hoops at 320 mm spacing while three cages were constructed with longitudinal bars and spirals at 75 mm spacing. The remaining three cages had longitudinal bars and one hoop at both ends of the bars to hold them together. These three cages were constructed for specimens required to have only longitudinal steel.

Six cylindrical sonotubes and eleven prefabricated GFRP tubes were used to construct the columns. The base of the formwork was made of two plywood sheets of 19 mm (3/4 in.) thickness, fastened together and laid out on a level floor. Circular cuts were made on the upper sheet to secure the GFRP tubes and the sonotubes. The bottom of the form was coated with thin layer of oil to avoid possible bonding between concrete and plywood.

The reinforcing cages were setup vertically at the centre of the circular cuts in the base of the formwork and rested on plastic chairs. Epoxy paste was used to stand the cages on the chairs. GFRP tubes and sonotubes were put around the cages. Eight holes were drilled at both ends of the test region of the tubes and 6 mm (1/4 in.) diameter threaded rods were inserted into the tubes. These threaded rods were used to install LVDT mounts. The circumference of the tubes at the bottom was sealed using silicone to prevent leakage. A wooden bracing system was provided to support the tubes and also to keep them upright and vertical during casting.

Before casting, the slump of the ready mix concrete was measured in order to determine its workability and was found to be 150 mm. The forms were filled with concrete in three layers. Each layer was compacted using an internal rod vibrator. After casting, specimens were covered with wet burlap and plastic sheet for a three-day curing period, after which the columns without FRP shells were stripped and all the columns were left for air curing. Nineteen cylinders were also cast in three layers. Each layer was given twenty-five strokes of tampering rod for compaction. The cylinders were also covered with wet burlap and plastic for three days, after which they were stripped and left with the column specimens for air curing. Clear concrete cover in the columns was 20
mm to the lateral steel and 30 mm to the longitudinal steel. Figure 5.8 through Figure 5.12 show photographs taken at various stages during the construction of the specimens.

A vinyl sheet was applied inside the shells during their construction, which provided a barrier between the GFRP and concrete. The purpose of the vinyl sheet was to:

1. Protect the GFRP from chemically reacting with alkalis from concrete.
2. Avoid formation of bond between GFRP and concrete.

The specimens were so designed to force the column failure to occur in a central test region of 620 mm (24.5 in.) height. To achieve this forced failure in the central region, extra layers of GFRP were provided in the end regions (above and below the test region). The following procedure was used to wrap extra layers of GFRP on the end regions:

1. Required size of GFRP was cut and laid on a plastic sheet.
2. TYFO S epoxy resin was prepared and applied on both sides of the GFRP. (The TYFO S epoxy is a two-part resin, TYFO™ A and TYFO™ B and is mixed together in ratios of 100:42, respectively. They are mixed thoroughly for 5 minutes using powered mixing blade at a speed of 450-600 rpm. The ready-to-use liquid has a pale yellow and very viscous appearance, and a slight ammonia odour. It has a working time of up to four hours at 38°C.)
3. The epoxy resin was also applied on the column or shell surface where GFRP wrapping was required.
4. The GFRP was rolled and then wrapped slowly over the required region as shown in Figure 5.13.
5. The wrapped columns were cured for a minimum of one week before testing. The extra layers of GFRP helped in preventing premature failure in the end regions and provided further insurance for failure within the test region.

5.5 INSTRUMENTATION

Instrumentation was provided to observe longitudinal column deformations, longitudinal reinforcing bar strains, longitudinal GFRP strains, lateral reinforcement strains, lateral GFRP strains, and total column axial force. The longitudinal column
deformations of the specimens were recorded using four Linear Variable Differential Transducers (LVDTs), each installed on North, South, East, and West sides of the test regions. These LVDTs were mounted vertically between threaded rods that were installed in the specimen prior to casting. The gauge lengths of the LVDTs ranged between 610 mm and 620 mm.

In order to measure strains in the steel, electric strain gauges (FLA-5-11) were installed on the surface of the steel. Figure 5.5 shows the general arrangement of strain gauges on the reinforcement. One strain gauge per longitudinal bar was installed at the centre of the bar, which was also the centre of the test region.

![Figure 5.5 General Arrangement of Strain Gauges on Reinforcement](image)

The specimens with spirals at 75 mm pitch had four gauges at 90° from each other on the periphery of the spiral in the test region. Whereas the specimens containing hoops at 320 mm had eight strain gauges, four on each of the two hoops in the test region. These strain gauges were again at 90° from each other on the periphery. These strain gauges were used to measure tensile strain in the lateral reinforcement. The properties of the electric strain gauges used on the reinforcement are given in Table 5.4.

The general procedure used to attach these strain gauges on the bars was as follows:
1. The ribs of the deformed bar were removed using a power tool. A coarse sanded belt was used to remove the ribs followed by the use of a fine sanded belt to make the surface smooth.

2. The surface was further prepared with sand paper. Coarse sand paper was used in one direction (normal to the direction in which power tool was used), then medium sand paper was used in the orthogonal direction to that of the coarse paper, finally a fine sand paper was used in the same direction as that of the coarse sand paper.

3. An acid (HCl) was used to clean the prepared surface and a base (NaOH) was applied to neutralise the acid afterwards. Finally an emery paper was used to finish the surface.

4. "CN glue" was used as an adhesive to attach the strain gauges to the bars.

5. "M-coat-A" was applied to the surfaces of the strain gauges to waterproof them.

6. The strain gauges were covered with wax and then covered with self-adhesive aluminium foil.

Four strain gauges (YL-60) were also attached externally on the surface of the GFRP shells. Two of these strain gauges, at 180° to each other on the periphery, were in longitudinal direction to measure longitudinal strains. The remaining two, also at 180° to each other on the periphery, were in the lateral direction to measure lateral strains. The properties of the electric strain gauges used on the GFRP are also given in Table 5.4. The general arrangement of strain gauges on the surface of GFRP shells is shown in Figure 5.6.

The general procedure followed in attaching the surface strain gauges was as follows:

1. A thick PS paste was smoothly spread over the surface of GFRP to cover the ridges.

2. The paste was covered with masking tape and was left overnight for curing.

3. The masking tape was removed.

4. An RP paste was used as an adhesive to attach the strain gauges.
5. After the RP paste had dried, "M-coat-A" was applied on the surface of strain gauge to waterproof it.

6. The wires of the strain gauges were then soldered to a terminal. A 5 m long wire was soldered on the other side of the terminal for its connection to the data acquisition system.

Table 5.4 Properties of Electric Strain Gauges

<table>
<thead>
<tr>
<th>Gauge type</th>
<th>Gauge length (mm)</th>
<th>Gauge factor (G.F)</th>
<th>Coefficient of thermal expansion</th>
<th>Temperature coefficient of G.F</th>
</tr>
</thead>
<tbody>
<tr>
<td>FLA-5-11 (on steel)</td>
<td>5</td>
<td>2.13 (± 1%)</td>
<td>$11.8 \times 10^{-6}/^\circ C$</td>
<td>$+0.1 \pm 0.05 %/10^\circ C$</td>
</tr>
<tr>
<td>YL-60 (on GFRP)</td>
<td>60</td>
<td>2.06 (± 1%)</td>
<td>$11.8 \times 10^{-6}/^\circ C$</td>
<td>$+0.05 \pm 0.05 %/10^\circ C$</td>
</tr>
</tbody>
</table>

Figure 5.6 General Arrangement of Surface Strain Gauges and LVDT
5.6 TESTING

The specimens were tested under monotonically increasing axial compression. Every possible effort was made to centre and align the specimens in order to ensure concentric loading. Plaster of Paris was used at the top and bottom of specimens in thin layers to keep the specimens essentially vertical.

A plate (343 mm diameter and 25.4 mm thick) with a diameter slightly less than that of the column (355.6 mm diameter) was placed on the top of the specimen during compressive loading. This arrangement specifically ensured concrete compression under applied loading and avoided having the GFRP under direct compression. Figure 5.7 shows the detailed diagram of the test setup while Figure 5.14 (a), (b), and (c) show the photographs of the actual experimental arrangement.

![Figure 5.7 Test Setup](image)
A Hewlett Packard high-speed Data Acquisition Unit capable of reading one hundred sets of readings per second was used to record measurements from all of the strain gauges and LVDTs.

The specimens were properly placed in the testing machine and all the strain gauges and LVDTs were connected to the Data Acquisition System. While keeping them in the elastic range, the specimens were loaded up to 25% of the estimated ultimate capacity, and the LVDTs were monitored for concentric load application. If concentric load application was not observed, adjustments were made by unloading the specimen and placing shims where required and again checking for concentric load application. This process was repeated until the axial deformation readings from LVDTs suggested concentric application of the load. All the specimens were tested up to failure.

For specimens without GFRP shells, failure was ascertained by crushing of the concrete core along with buckling of longitudinal steel or rupture of lateral steel, whereas for specimens with GFRP shells, failure was ascertained by rupture of the GFRP in addition to the above mentioned features.

5.7 SUMMARY

The material properties of the concrete, longitudinal steel, lateral steel and GFRP used in the experimental program are reported in this chapter. Design and construction details of the specimens, instrumentation details, and the test setup are also presented.
Figure 5.8 Steel Cage under Construction
Figure 5.9 Different Steel Cages

Figure 5.10 Steel Cage inside the GFRP Shell
Figure 5.11 Placing the Steel Cage
Figure 5.12 (a) Wooden Bracing Holding the Specimens

Figure 5.12 (b) Wooden Bracing Holding the Specimens
Figure 5.13 Wrapping the GFRP Sheets

Figure 5.14 (a) Test Setup
Figure 5.14 (b) Test Setup

Figure 5.14 (c) Test Setup
Figure 5.14 (d) Test Setup
CHAPTER 6

RESULTS AND DISCUSSION

6.1 GENERAL

The results of the columns tested in the experimental program are reported and discussed in this chapter. Stress-strain responses of confined concrete in the specimens are plotted and used to study characteristics such as strength enhancement, ductility, and energy absorption properties. The effect of number of layers of GFRP shells on the stress-strain response is also studied.

6.2 TEST OBSERVATIONS

The testing procedure adopted for all seventeen specimens essentially involved inspecting and monitoring the effect of compressive axial load in order to characterise the failure tendencies and behaviours of different specimens in question. It is important to mention here that all the specimens, in a physical sense, failed within the test region i.e. within the LVDT gauge length. A pictorial sketch is presented as Figure 6.1 to distinctly earmark the test regions and extensively damaged zones.

Details of specimen behaviour under increasing loads as observed during the actual test runs are given below with special reference to their composition in terms of confinement provided.

For the six control specimens without any GFRP shell, first signs of distress appeared as vertical cracks within the test region at their mid heights. For the plain concrete columns (specimens 1 and 2) the failure was a sudden burst of the test region immediately after the development of vertical cracks around the entire circumference of the specimens. Specimens 3 and 4 with hoops at 320 mm spacing had very low confinement. These specimens failed spontaneously as the cover spalled off in addition to an abrupt outburst of the core and buckling of longitudinal steel.
Vertical cracks appeared to develop in a symmetrical fashion around the entire central region of specimens 5 and 6 which were adequately confined with spirals at 75 mm pitch. With increasing load the cracks appeared to increase in size and number until pieces of the cover concrete started to spall off. Due to adequate confinement, the core concrete did not experience the same lateral deformation as encountered by the cover. As a result, under incremental loading the core expanded laterally outward against the lateral and longitudinal steel. Meanwhile the longitudinal steel began to buckle and further pushed against the lateral steel. Finally the failure of the specimens was accompanied by the crushing of the core concrete. Furthermore, spirals were found to rupture at one or more location in both specimens.

All the specimens with GFRP shells failed more or less in a similar fashion, except specimens 16 and 17 with fibres inclined at 45°. The failure was marked by the disruption of the GFRP shell in the test region. The rupture of the GFRP shell was found to start at one location and then with a sudden thrust, moved around in the circumferential direction very rapidly accompanied by irregular exploding sounds. The specimens with GFRP shells exhibited considerable ductile behaviour, which was also accompanied by sufficient warning before failure. The failure of specimens with one layer of GFRP was slightly abrupt as compared to those with two layers of GFRP.

Specimen 13, with two layers of GFRP shell, did not perform well in the test series. Although every effort was made to achieve concentric load application, it was observed to be under eccentric loading. This specimen experienced extensive bending about the East-West axis. The South side was under compression while the North side was under tension. East and West sides were found to be somewhat damaged.

Specimen 16 and 17 with fibres inclined at 45° to the horizontal, behaved in a very ductile manner. They did not burst out or ripped off like other columns, rather these specimens settled down softly, which is representative of their ductile behaviour. These samples were found to undergo a rupture plane inclined at an angle of about 45°. Figure 6.2a through Figure 6.2q shows all the specimens after testing.
Figure 6.1 Extensively Damaged Regions Shown in Shaded Portion (cont. next page)
Figure 6.1 Extensively Damaged Regions Shown in Shaded Portion
Figure 6.2a Specimen 1 after Testing
Figure 6.2b Specimen 2 after Testing
Figure 6.2c  Specimen 3 after Testing
Figure 6.2d Specimen 4 after Testing
Figure 6.2e Specimen 5 after Testing
Figure 6.2f Specimen 6 after Testing
Figure 6.2g Specimen 7 after Testing
Figure 6.2h Specimen 8 after Testing
Figure 6.2i Specimen 9 after Testing
Figure 6.2j Specimen 10 after Testing
Figure 6.2k Specimen 11 after Testing
Figure 6.21 Specimen 12 after Testing
Figure 6.2m  Specimen 13 after Testing
Figure 6.2n Specimen 14 after Testing
Figure 6.20 Specimen 15 after Testing
Figure 6.2p Specimen 16 after Testing
Figure 6.2q Specimen 17 after Testing
6.3 INTERPRETATION OF RESULTS

The data for each specimen obtained during the test was analysed and used to plot various curves for investigating their behaviour. Stress-strain curves of the confined concrete in the specimens were significant in defining performance characteristics. For the specimens with longitudinal steel, it was necessary to separate the load carried by concrete and that carried by longitudinal reinforcement.

6.3.1 CONCRETE CONTRIBUTION

For specimens without longitudinal steel, the stresses at any point could be obtained from the total applied load. For specimens with longitudinal steel, the portion of the total applied load carried by concrete was obtained by subtracting the amount of load carried by longitudinal steel \( P_{\text{steel}} \) from the total applied load \( P_{\text{total}} \) as given in Equation 6.1 and also shown in Figure 6.3.

\[
P_{\text{conc}} = P_{\text{total}} - P_{\text{steel}}
\]

![Figure 6.3 Calculation of Load Carried by Concrete](image)
The load carried by longitudinal fibres was not considered for specimens with glass fibres in the longitudinal direction. The average axial column strains obtained from the LVDT readings were used on the stress-strain curve of steel to obtain the load carried by the longitudinal steel (P_{steel}). It was assumed that there was no slippage of the longitudinal reinforcement in the concrete. This was supported by the observation that the plots of total applied load versus the longitudinal steel strains were in good agreement with the plots of total applied load versus strains from LVDT readings. These plots are given in Appendix A.

It was also assumed that the buckling of longitudinal steel started when the strains from the LVDT readings differed significantly from the longitudinal steel strains. However, the axial strains after the peak load were not available from the strain gauges on the longitudinal bars, and therefore it was not possible to determine axial strains at which buckling of the bars occurred. Hence the effect of buckling was ignored.

6.3.2 CONFINED CONCRETE STRESS-STRAIN CURVE

For plain concrete specimens 1 and 2 the concrete stress was calculated by dividing the total load (P_{total}) by the gross area of concrete. Specimens 3 and 4 with lateral steel at 320 mm spacing had a very low confinement, thus the core and the cover were assumed to behave alike until the failure of the specimens. These specimens failed as the cover spalled off. For these specimens, load carried by concrete (P_{conc}) was divided by gross area of concrete to obtain stresses at all levels of loading.

Specimens 5 and 6 were effectively confined with spirals at 75 mm spacing, thus the core and the cover exhibited different mechanical properties. The confined concrete behaviour in these columns was determined following the procedure suggested by Sheikh and Uzumeri[7]. In the initial stages of loading, when the axial strains and the Poisson’s ratio effect of concrete were small, the behaviour of the core and the cover was similar. With an increase in the axial strain and the Poisson’s ratio effect of concrete, the cover started to spall off. Whereas due to the effect of lateral reinforcement, the core did not experience the same lateral deformation and remained intact. Thus at this stage the core concrete started to behave differently than the cover concrete. While it was the gross
concrete area representing the column behaviour before cover started to spall off, it was only the core concrete area representing the column behaviour after the concrete cover became ineffective in resisting the applied load. Thus, there were two stress-strain curves for each of these specimens as represented in Figure 6.4. The lower curve shows the stresses which were calculated by dividing the load carried by concrete \( P_{\text{conc}} \) by its gross area, whereas the upper curve shows the stresses which were calculated by dividing the load carried by concrete \( P_{\text{conc}} \) by the core concrete area. A smooth transition took place from the lower curve to the upper curve between the strain values at which cover started to spall off and at which it became completely ineffective. It was suggested by Sheikh and Uzumeri\(^7\) that the cover starts spalling off at a strain value \( (\varepsilon_0) \) corresponding to the maximum plain concrete stress and becomes completely ineffective at a strain value \( (\varepsilon_{50u}) \) which corresponds to 50% of the peak stress of plain concrete on the descending branch of its stress-strain curve. Curves of specimens 5 and 6 show that spalling of concrete cover started at strain values ranging between 0.0015 and 0.0016. The smooth transition curves terminated at a strain ranging from 0.0040 to 0.0045.

**Figure 6.4** Concrete Contribution Curves with Respect to \( A_{\text{core conc}} \) and \( A_{\text{gross conc}} \).
For all the other specimens encased in GFRP shells the stresses were calculated by dividing the load carried by concrete ($P_{concrete}$) by its effectively confined gross area. These calculated values of stresses and the average axial strains obtained from the LVDT readings were used to plot the confined concrete stress strain curves.

![Confined Concrete Stress-Strain Curve](image)

**Figure 6.5** Typical Confined Concrete Stress-Strain Curve

The confined concrete stress-strain curve for each specimen was used to calculate the following parameters:

1. Peak confined concrete stress ($f_{ccmax}$)
2. Axial strain at peak concrete stress ($\varepsilon_c$)
3. Initial tangent modulus of elasticity of concrete ($E_t$)
4. Axial strain ($\varepsilon_1$) corresponding to $f_{ccmax}$ on the initial tangent modulus, as shown in Figure 6.5.
5. Axial strain ($\varepsilon_2$) at 80% of $f_{ccmax}$ on the descending branch, as shown in Figure 6.5.
6.4 ANALYSIS OF RESULTS

The stress-strain curves of all the specimens were analysed to study various properties such as peak load enhancement, ductility, and energy absorption characteristics.

6.4.1 SPECIMEN STRESS-STRAIN RESPONSE

The stress-strain plots are presented in Figure 6.6a through Figure 6.6q. The procedure used to construct these curves is outlined in Section 6.3.

Reference to Figure 6.6a through Figure 6.6q, the behaviour of all the specimens under consideration was typically linear during the initial stages of loading. However, the number of layers of GFRP shells affected the concrete peak stress values and behaviour after the peak.

The stress-strain response of all the test specimens are summarised below:

- All the specimens showed initial linear ascending portion of the stress-strain curve. The average value of initial tangent of modulus of elasticity for concrete was found to be 29300 MPa.
- The peak stress value and the axial strain at peak stress increased with an increase in confinement.
- Specimens that were moderately confined with one layer of GFRP shell (7, 8, 9) displayed moderate ductility with a slow degradation of the descending branch of the stress-strain curve.
- Specimens that were well confined (10, 11, 12, 13, 14, and 15) showed two slopes of the ascending branch before peak stress. The first slope was almost the same as that of unconfined concrete specimens. However, the second slope was less steep and started near the peak stress of unconfined concrete i.e. 30 MPa. This behaviour indicated that at initial stages of loading the GFRP had almost no confining effect on the concrete. But after peak stress of unconfined concrete, the Poisson’s ratio increased rapidly and so did the lateral expansion, which lead to higher stresses in the GFRP. At this stage the concrete was subjected to triaxial stresses while the GFRP was under uniaxial tensile stress.
Specimens that were well confined with one or two layers of GFRP shell along with lateral steel performed well among the test series. They exhibited relatively higher peak stress and ductility. Moreover, they were also able to maintain strength at relatively high strains.
Figure 6.6a Behaviour of Confined Concrete for Specimen 1
Figure 6.6c Behaviour of Confined Concrete in Specimen 3
Figure 6.6d Behaviour of Confined Concrete in Specimen 4
Figure 6.6e Behaviour of Confined Concrete in Specimen 5

Specimen 5
00-LS75-5
No lateral GFRP
No longitudinal GFRP
Longitudinal steel (6-20M)
#3 spiral at 75mm pitch
Figure 6.6f Behaviour of Confined Concrete in Specimen 6
Figure 6.6g Behaviour of Confined Concrete in Specimen 7
Figure 6.6h Behaviour of Confined Concrete in Specimen 8

Specimen 8
G11-00-8
1 Layer of lateral GFRP
1 Layer of longitudinal GFRP
No longitudinal steel
No lateral steel

Results and Discussion
Figure 6.6j Behaviour of Confined Concrete in Specimen 10
Figure 6.6k Behaviour of Confined Concrete in Specimen 11
Figure 6.61 Behaviour of Confined Concrete in Specimen 12
Figure 6.6m Behaviour of Confined Concrete in Specimen 13
Figure 6.6n Behaviour of Confined Concrete in Specimen 14
Figure 6.60 Behaviour of Confined Concrete in Specimen 15

Specimen 15
G02-LS75-15
2 Layers of lateral GFRP
No longitudinal GFRP
Longitudinal steel (6-20M)
#3 Spiral at 75mm pitch
Results and Discussion

Figure 6.6p Behaviour of Confined Concrete in Specimen 16
Figure 6.6q Behaviour of Confined Concrete in Specimen 17
6.4.2 SPECIMEN STRENGTH

Peak strength enhancement of all the columns was calculated using two different expressions:

\[ K_1 = \text{Peak strength enhancement with respect to column concrete strength} \]
\[ = \frac{f_{c_{\text{max}}}}{0.85f_c'} \tag{6.1} \]
\[ K_2 = \text{Peak strength enhancement with respect to cylinder concrete strength} \]
\[ = \frac{f_{c_{\text{max}}}}{f_c'} \tag{6.2} \]

where
\[ f_{c_{\text{max}}} = \text{maximum concrete stress} \]
\[ f_c' = \text{unconfined concrete strength obtained from standard cylinder tests} \]

In Equation (6.1) the unconfined concrete strength in the columns was considered to be 85% of the concrete strength obtained from standard cylinder test. The factor of 85% was incorporated due to the difference in size and shape of the cylinders and columns as suggested by various researchers \cite{4,5,51,52}. Peak Strength Enhancement of all the specimens is tabulated in Table 6.1.

The confining pressure for specimens with lateral steel was calculated using the expression (4.26) given by Mander, Priestley, and Park\cite{3} as discussed in Chapter 4. The same equation could be used to calculate the confining pressure generated by GFRP shell. As specimens were confined with continuous GFRP and there was no vertical spacing (i.e. \( s' = 0 \)), the confinement effectiveness co-efficient, \( k_e = 1 \). So a simplified form of the same expression given in Equation (4.36) in Chapter 4, by Saadatmanesh, Ehsani, and Li\cite{35} was used. The tensile strength of GFRP in units of N/mm (535 N/mm) was used in Equation (4.36) instead of \( f_{y_{th}} \) of steel as used in Equation (4.26). These expressions were developed by considering the free body diagram of a circular cross section confined with the lateral steel or GFRP and applying the equations of equilibrium of forces.
Table 6.1 Peak Strength Enhancements

<table>
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<tr>
<th>Spec no</th>
<th>Name</th>
<th>( P_{\text{max}} )</th>
<th>( f_{\text{ccmax}} )</th>
<th>( \varepsilon' )</th>
<th>( \varepsilon'_c ) among all LVDTs</th>
<th>( k_e )</th>
<th>Conf. Pressure</th>
<th>( K_1 )</th>
<th>( K_2 )</th>
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6.4.3 SPECIMEN DUCTILITY

"The behaviour of reinforced concrete sections and members is not elasto-plastic, hence there is no universal definition for ductility. In evaluating the column performance ductility is defined using different parameters, which are assumed to provide a reasonable basis for consistent evaluation of section and member behaviour."[53]

Ductility factor of the specimens was calculated using the following relationship:

\[ \mu = \frac{\varepsilon_2}{\varepsilon_1} \] (6.3)

where

\( \varepsilon_1 \) = axial strain corresponding to the maximum confined concrete stress \( (f_{\text{ccmax}}) \) on the initial tangent (see Figure 6.5)

\( \varepsilon_2 \) = axial strain corresponding to a strength of 0.8\( f_{\text{ccmax}} \) on the descending portion of the specimen stress-strain curve (see Figure 6.5)

Table 6.2 shows the ductility factor of all the specimens.
Table 6.2 Ductility Factor of Specimens

<table>
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<th>Spec no</th>
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<th>ɛ₂ (mm/mm)</th>
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<td>G45°2-L0-17</td>
<td>8.4390</td>
<td>0.00123</td>
<td>0.04734</td>
<td>38.41</td>
</tr>
</tbody>
</table>

NA = not available

6.4.4 SPECIMEN ENERGY ABSORPTION CAPACITY

The energy absorption capacity is calculated as the area under the confined concrete stress-strain curve and is in units of MPa mm/mm. Energy absorption capacity was calculated from the stress-strain curves of all the specimens, which indicated the amount of energy absorbed by the specimens. Energy absorption capacity values were calculated for three different levels of stresses. The first was the area under the stress-strain curve up to 80% of \( f_{c_{\text{max}}} \) on the descending portion of the curve. The second was the area up to 50% of \( f_{c_{\text{max}}} \) on the descending portion of the curve and the third was up to 20% of \( f_{c_{\text{max}}} \) on the descending portion of the curve. These were denoted as \( \varepsilon_{80} \), \( \varepsilon_{50} \), and \( \varepsilon_{20} \), respectively and are represented in Figure 6.7.
Figure 6.7 Area under the Stress-Strain Curves up to Various Points

For Specimens 1 and 2, $e_{50}$ and $e_{20}$ could not be calculated because of the non-availability of data beyond 60% of $f_{ccmax}$ on the descending branch. Similarly $e_{20}$ for specimens 5, 9, 13, 15, and 16 could not be calculated due to non-availability of data beyond about 30% of $f_{ccmax}$ on the descending branch. Sufficient data was not available for Specimen 4 to calculate energy absorption capacity at required levels of stresses. Table 6.3 shows the values of the energy absorption capacity of all the specimens.

Table 6.3 Energy Absorption Capacity of the Specimens

<table>
<thead>
<tr>
<th>Spec no</th>
<th>Name</th>
<th>$e_{50}$</th>
<th>$e_{20}$</th>
<th>$e_{20}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>00-10-1</td>
<td>0.081</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>2</td>
<td>00-10-2</td>
<td>0.118</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>3</td>
<td>00-LS320-3</td>
<td>0.092</td>
<td>0.122</td>
<td>0.153</td>
</tr>
<tr>
<td>4</td>
<td>00-LS320-4</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>5</td>
<td>00-LS75-5</td>
<td>0.513</td>
<td>0.936</td>
<td>NA</td>
</tr>
<tr>
<td>6</td>
<td>00-LS75-6</td>
<td>0.584</td>
<td>0.900</td>
<td>1.120</td>
</tr>
<tr>
<td>7</td>
<td>G01-00-7</td>
<td>0.336</td>
<td>0.517</td>
<td>0.660</td>
</tr>
<tr>
<td>8</td>
<td>G11-00-8</td>
<td>0.390</td>
<td>0.500</td>
<td>0.830</td>
</tr>
<tr>
<td>9</td>
<td>G01-L0-9</td>
<td>0.350</td>
<td>0.490</td>
<td>NA</td>
</tr>
<tr>
<td>10</td>
<td>G01-LS320-10</td>
<td>0.580</td>
<td>0.680</td>
<td>0.800</td>
</tr>
<tr>
<td>11</td>
<td>G02-00-11</td>
<td>0.817</td>
<td>1.186</td>
<td>1.346</td>
</tr>
<tr>
<td>12</td>
<td>G22-00-12</td>
<td>1.320</td>
<td>1.528</td>
<td>1.716</td>
</tr>
<tr>
<td>13</td>
<td>G02-L0-13</td>
<td>1.160</td>
<td>1.560</td>
<td>NA</td>
</tr>
<tr>
<td>14</td>
<td>G02-LS320-14</td>
<td>1.070</td>
<td>1.570</td>
<td>1.690</td>
</tr>
<tr>
<td>15</td>
<td>G02-LS75-15</td>
<td>1.420</td>
<td>2.100</td>
<td>NA</td>
</tr>
<tr>
<td>16</td>
<td>G45º2-00-16</td>
<td>0.610</td>
<td>1.070</td>
<td>NA</td>
</tr>
<tr>
<td>17</td>
<td>G45º2-L0-17</td>
<td>1.600</td>
<td>1.830</td>
<td>1.970</td>
</tr>
</tbody>
</table>

NA = not available
6.4.5 SPECIMEN WORK INDEX

The energy absorption capacity as calculated earlier in Section 6.4.4 was normalised to obtain a dimensionless parameter termed as Work Index. The following expression was used to normalise the area under the stress-strain curve:

\[ \text{Work Index} = \frac{e}{f_{cmax} \times \varepsilon_1} \]  

(6.4)

where

e = energy absorption capacity
\varepsilon_1 = strain corresponding to the maximum confined concrete stress \( f_{cmax} \) on the initial tangent
\( f_{cmax} = \) peak concrete stress

Similar expressions have been used earlier in a cyclic form by Sheikh and Khoury\(^{53}\) and also by Ehsani and Wight\(^{54}\).

The work indices were denoted as \( W_{80}, W_{50}, \) and \( W_{20} \) corresponding to the energy absorption capacities \( e_{80}, e_{50}, \) and \( e_{20} \), respectively. Table 6.4 gives the work index values of all the specimens.

Table 6.4 Work Index of the Specimens

<table>
<thead>
<tr>
<th>Spec no</th>
<th>Name</th>
<th>( W_{80} )</th>
<th>( W_{50} )</th>
<th>( W_{20} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>00-10-1</td>
<td>2.52</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>2</td>
<td>00-10-2</td>
<td>3.27</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>3</td>
<td>00-LS320-3</td>
<td>4.02</td>
<td>5.35</td>
<td>6.70</td>
</tr>
<tr>
<td>4</td>
<td>00-LS320-4</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>5</td>
<td>00-LS75-5</td>
<td>7.75</td>
<td>14.15</td>
<td>NA</td>
</tr>
<tr>
<td>6</td>
<td>00-LS75-6</td>
<td>8.84</td>
<td>13.62</td>
<td>16.95</td>
</tr>
<tr>
<td>7</td>
<td>G01-00-7</td>
<td>8.14</td>
<td>12.53</td>
<td>16.00</td>
</tr>
<tr>
<td>8</td>
<td>G11-00-8</td>
<td>7.75</td>
<td>9.93</td>
<td>16.48</td>
</tr>
<tr>
<td>9</td>
<td>G01-L0-9</td>
<td>9.75</td>
<td>13.65</td>
<td>NA</td>
</tr>
<tr>
<td>10</td>
<td>G01-LS320-10</td>
<td>12.29</td>
<td>14.41</td>
<td>16.95</td>
</tr>
<tr>
<td>11</td>
<td>G02-00-11</td>
<td>11.92</td>
<td>17.30</td>
<td>19.63</td>
</tr>
<tr>
<td>12</td>
<td>G22-00-12</td>
<td>12.90</td>
<td>14.93</td>
<td>16.77</td>
</tr>
<tr>
<td>13</td>
<td>G02-L0-13</td>
<td>14.24</td>
<td>19.15</td>
<td>NA</td>
</tr>
<tr>
<td>14</td>
<td>G02-LS320-14</td>
<td>12.89</td>
<td>18.92</td>
<td>20.36</td>
</tr>
<tr>
<td>15</td>
<td>G02-LS75-15</td>
<td>14.27</td>
<td>21.10</td>
<td>NA</td>
</tr>
<tr>
<td>16</td>
<td>G45°2-00-16</td>
<td>12.64</td>
<td>22.18</td>
<td>NA</td>
</tr>
<tr>
<td>17</td>
<td>G45°2-L0-17</td>
<td>35.95</td>
<td>41.12</td>
<td>44.27</td>
</tr>
</tbody>
</table>

NA = not available
6.5 DISCUSSION ON RESULTS

The effect of number of layers of GFRP shells on the behaviour of confined concrete is studied in this section. Specimens can be divided into three main types:

1. Specimens with no longitudinal and lateral steel
2. Specimens with longitudinal steel and hoops at 320 mm spacing
3. Specimens with longitudinal steel and spirals at 75 mm pitch

Plots of axial stress versus average axial strain and plots of applied load versus average axial strain for these groups are presented in Figure 6.8a through Figure 6.8f showing the effect of number of layers of GFRP on the specimen behaviour. Further, Figure 6.9a through Figure 6.9i shows the effect of confining pressure on various variables. It can be observed from the figures that with increasing confining pressure, the performance characteristics improved considerably.

A comparison is also made between the behaviour of specimens confined with GFRP shells and those confined with steel in order to investigate the suitability of GFRP to replace steel for confinement.

6.5.1 EFFECT OF NUMBER OF LAYERS OF GFRP SHELLS ON SPECIMENS WITH NO LONGITUDINAL AND LATERAL STEEL

The effect of GFRP shell layers on behaviour of specimens without any longitudinal or lateral steel is evaluated by comparing the responses of specimens 1, 2, 7, 11, and 16. Table 6.5 shows the properties of these specimens.

<table>
<thead>
<tr>
<th>Name</th>
<th>$P_{max}$ (kN)</th>
<th>$f_{cc}$ (MPa)</th>
<th>Peak load enhancements</th>
<th>Ductility</th>
<th>Energy absorption capacity</th>
<th>Work index</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$K_1$</td>
<td>$K_2$</td>
<td>$\mu$</td>
<td>$e_{80}$</td>
<td>$e_{50}$</td>
<td>$e_{20}$</td>
</tr>
<tr>
<td>00-10-1</td>
<td>3125</td>
<td>30.70</td>
<td>1.200</td>
<td>1.023</td>
<td>3.19</td>
<td>0.081</td>
</tr>
<tr>
<td>00-10-2</td>
<td>3350</td>
<td>32.53</td>
<td>1.276</td>
<td>1.084</td>
<td>3.98</td>
<td>0.118</td>
</tr>
<tr>
<td>G01-00-7</td>
<td>3460</td>
<td>34.77</td>
<td>1.364</td>
<td>1.159</td>
<td>9.08</td>
<td>0.336</td>
</tr>
<tr>
<td>G02-00-11</td>
<td>4460</td>
<td>44.82</td>
<td>1.758</td>
<td>1.494</td>
<td>13.56</td>
<td>0.817</td>
</tr>
<tr>
<td>G45°2-00-16</td>
<td>3740</td>
<td>37.60</td>
<td>1.475</td>
<td>1.253</td>
<td>13.93</td>
<td>0.610</td>
</tr>
</tbody>
</table>

NA= not available
It is observed that an increase in the number of layers improved the behaviour of these specimens. The peak strength enhancements, $K_1$ and $K_2$, of these specimens increased by 10% when confined with one layer of GFRP shell and increased by 40% when confined with two layers of GFRP. The increase in peak strength enhancement due to confinement with two layers of GFRP shell was more pronounced as compared to that due to confinement by one layer of GFRP shell. For specimen 16 with fibres at 45°, both $K_1$ and $K_2$ increased by 20%. The ductility factors also increased with an increase in the number of layers of GFRP shells. It is evident that the confinement by two layers of GFRP shell tremendously increased the ductility factor. Similar observations can be made for energy absorption capacity and work indices on comparison. Plots of applied load versus axial strain for this group of specimens are presented in Figure 6.8a, showing the effect of number of layers of GFRP on the specimen behaviour.

### 6.5.2 EFFECT OF NUMBER OF LAYERS OF GFRP SHELLS ON SPECIMENS WITH LONGITUDINAL STEEL AND HOOPS AT 320 mm SPACING

The effect of GFRP confinement on specimens with hoops at 320 mm spacing can be examined by comparing specimens 3, 4, 10, and 14. Specimen 3 and 4 were confined only with lateral hoops at 320 mm and were not encased by GFRP shells, while specimens 10 and 14 were confined with GFRP shells along with the hoops at 320 mm spacing. Properties of these specimens are compared in Table 6.6.

**Table 6.6 Effect of Number of Layers of GFRP Shells on Specimens with Longitudinal Steel and Hoops at 320 mm Spacing**

<table>
<thead>
<tr>
<th>Name</th>
<th>$P_{\text{max}}$ (kN)</th>
<th>$f_{\text{cmax}}$ (MPa)</th>
<th>Peak load enhancements</th>
<th>Ductility</th>
<th>Energy absorption capacity</th>
<th>Work index</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>$K_1$ $K_2$ $\mu$ $e_{80}$ $e_{50}$ $e_{20}$</td>
<td>$W_{80}$ $W_{50}$ $W_{20}$</td>
<td>NA= not available</td>
<td></td>
</tr>
<tr>
<td>00-LS320-3</td>
<td>3130</td>
<td>25.86</td>
<td>1.014 0.862 4.92 0.092 0.122 0.153</td>
<td>4.02</td>
<td>5.35 6.70</td>
<td></td>
</tr>
<tr>
<td>00-LS320-4</td>
<td>3665</td>
<td>30.08</td>
<td>1.180 1.003 NA NA NA</td>
<td>NA</td>
<td>NA NA NA</td>
<td></td>
</tr>
<tr>
<td>G01-LS320-10</td>
<td>4360</td>
<td>37.19</td>
<td>1.458 1.240 14.30 0.580 0.680 0.800</td>
<td>12.29</td>
<td>14.41 16.95</td>
<td></td>
</tr>
<tr>
<td>G02-LS320-14</td>
<td>5570</td>
<td>49.31</td>
<td>1.934 1.644 15.49 1.070 1.570 1.690</td>
<td>12.89</td>
<td>18.92 20.36</td>
<td></td>
</tr>
</tbody>
</table>
The value of $K_2$ was less than 1 for specimen 3 indicating the concrete strength to be less than the unconfined cylinder strength of concrete. The peak strength enhancements increased with increase in number of layers of GFRP shells. Specimens 4 did not have data points beyond 0.95 $f_{c\text{cmax}}$ on the descending branch of the stress-strain curve and hence the ductility factor, energy absorption capacity, and work index could not be calculated for this specimen. It can be observed that the ductility factor increased from 4.92 to 14.30 when these specimens were confined with one layer of GFRP shell and increased from 4.92 to 15.49 when confined with two layers of GFRP shells. It can also be observed that the energy absorption capacity of specimen 14 is almost double than that of specimen 10. A tremendous increase in energy absorption capacity for these specimens is observed with increase in number of GFRP layers. Increase in work index values with increase in number of layers of GFRP shells is also evident. Plots of applied load versus axial strain for this group of specimens are presented in Figure 6.8b, showing the effect of number of layers of GFRP on the specimen behaviour.

### 6.5.3 EFFECT OF NUMBER OF LAYERS OF GFRP SHELLS ON SPECIMENS WITH LONGITUDINAL STEEL AND SPIRAL AT 75 mm PITCH

To evaluate the effect of confinement of GFRP shell on specimens with spirals at 75 mm pitch, specimens 5, 6, and 15 are considered. A comparison of the properties of these specimens is presented in Table 6.7.

**Table 6.7 Effect of Number of Layers of GFRP Shells on Specimens with Longitudinal Steel and Spiral at 75 mm Pitch**

<table>
<thead>
<tr>
<th>Name</th>
<th>$P_{\text{max}}$ (kN)</th>
<th>$f_{c\text{cmax}}$ (MPa)</th>
<th>$K_1$</th>
<th>$K_2$</th>
<th>$\mu$</th>
<th>$e_{80}$</th>
<th>$e_{50}$</th>
<th>$e_{20}$</th>
<th>$W_{80}$</th>
<th>$W_{50}$</th>
<th>$W_{20}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>00-LS75-5</td>
<td>4125</td>
<td>44.00</td>
<td>1.725</td>
<td>1.470</td>
<td>9.27</td>
<td>0.513</td>
<td>0.936</td>
<td>NA</td>
<td>7.75</td>
<td>14.15</td>
<td>NA</td>
</tr>
<tr>
<td>00-LS75-6</td>
<td>4030</td>
<td>44.00</td>
<td>1.725</td>
<td>1.470</td>
<td>10.26</td>
<td>0.584</td>
<td>0.900</td>
<td>1.12</td>
<td>8.84</td>
<td>13.62</td>
<td>16.95</td>
</tr>
<tr>
<td>G02-LS75-15</td>
<td>6020</td>
<td>54.00</td>
<td>2.118</td>
<td>1.800</td>
<td>16.49</td>
<td>1.420</td>
<td>2.100</td>
<td>NA</td>
<td>14.27</td>
<td>21.10</td>
<td>NA</td>
</tr>
</tbody>
</table>

NA= not available
Results and Discussion

It can be observed that when specimen with spirals at 75 mm pitch was encased with two layers of GFRP shells, all parameters improved considerably. Peak strength enhancements improved by 23%, while ductility factor increased by 65%. Energy absorption capacity appears to be affected more as compared to the other parameters. Plots of applied load versus axial strain for this group of specimens are presented in Figure 6.8c, showing the effect of number of layers of GFRP on the specimen behaviour.
Figure 6.8a Axial Stress versus Axial Strain for Specimens with No Longitudinal and Lateral Steel
Figure 6.8b Axial Stress versus Axial Strain for Specimens with Longitudinal Steel (6-20M bars) and Lateral Steel (#3 hoops @ 320 mm spacing)
Figure 6.8c Axial Stress versus Axial Strain for Specimens with Longitudinal Steel (6-20M bars) and Lateral Steel (#3 spiral @ 75 mm pitch)
Figure 6.8d Axial Load versus Axial Strain for Specimens with No Longitudinal and Lateral Steel
Figure 6.8e Axial Load versus Axial Strain for Specimens with Longitudinal Steel (6-20M bars) and Lateral Steel (#3 hoops @ 320 mm spacing)
Figure 6.8f Axial Load versus Axial Strain for Specimens with Longitudinal Steel (6-20M bars) and Lateral Steel (#3 spiral @ 75 mm pitch)
Figure 6.9a Effect of Confining Pressure of Peak Strength Enhancement (K1)

Figure 6.9b Effect of Confining Pressure of Peak Strength Enhancement (K2)
Results and Discussion

Figure 6.9c Effect of Confining Pressure on Ductility

Figure 6.9d Effect of Confining Pressure on Energy Absorption Capacity $e_{80}$
Figure 6.9e Effect of Confining Pressure on Energy Absorption Capacity $e_{50}$

Figure 6.9f Effect of Confining Pressure on Energy Absorption Capacity $e_{20}$
Results and Discussion

**Figure 6.9g** Effect of Confining pressure on Work Index $W_{80}$

**Figure 6.9h** Effect of Confining pressure on Work Index $W_{50}$
Figure 6.9i Effect of confining pressure on Work Index $W_{20}$
6.5.4 COMPARISON BETWEEN CONFINEMENT DUE TO GFRP SHELLS AND CONFINEMENT DUE TO LATERAL STEEL

A comparison is made between specimens 3, 4, 5, 6, 9, 13, and 17. These specimens are similar in every respect except that the specimens 3, 4, 5, and 6 are confined with lateral steel while specimens 9, 13, and 17 are confined with GFRP shells. The purpose of this comparison is to evaluate the potential of GFRP shells to replace the conventional steel for lateral reinforcement. Table 6.8 shows the properties of these specimens.

Table 6.8 Comparison between Confinement due to GFRP Shells and Confinement due to Steel

<table>
<thead>
<tr>
<th>Name</th>
<th>P&lt;sub&gt;max&lt;/sub&gt; (kN)</th>
<th>f&lt;sub&gt;emax&lt;/sub&gt; (MPa)</th>
<th>Peak load enhancements</th>
<th>Ductility</th>
<th>Energy absorption capacity</th>
<th>Work index</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>K&lt;sub&gt;1&lt;/sub&gt;</td>
<td>K&lt;sub&gt;2&lt;/sub&gt;</td>
<td>µ</td>
<td>e&lt;sub&gt;40&lt;/sub&gt;</td>
<td>e&lt;sub&gt;50&lt;/sub&gt;</td>
<td>e&lt;sub&gt;20&lt;/sub&gt;</td>
</tr>
<tr>
<td>00-LS320-3</td>
<td>3130</td>
<td>25.86</td>
<td>1.014</td>
<td>0.862</td>
<td>4.92</td>
<td>0.092</td>
</tr>
<tr>
<td>00-LS320-4</td>
<td>3665</td>
<td>30.08</td>
<td>1.180</td>
<td>1.003</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>00-LS75-5</td>
<td>4125</td>
<td>44.00</td>
<td>1.725</td>
<td>1.470</td>
<td>9.27</td>
<td>0.513</td>
</tr>
<tr>
<td>00-LS75-6</td>
<td>4030</td>
<td>44.00</td>
<td>1.725</td>
<td>1.470</td>
<td>10.26</td>
<td>0.584</td>
</tr>
<tr>
<td>G01-L0-9</td>
<td>3895</td>
<td>32.43</td>
<td>1.272</td>
<td>1.081</td>
<td>10.81</td>
<td>0.350</td>
</tr>
<tr>
<td>G02-L0-13</td>
<td>5500</td>
<td>48.85</td>
<td>1.916</td>
<td>1.628</td>
<td>10.87</td>
<td>1.160</td>
</tr>
<tr>
<td>G45°2-L0-17</td>
<td>4250</td>
<td>36.11</td>
<td>1.416</td>
<td>1.204</td>
<td>38.41</td>
<td>1.600</td>
</tr>
</tbody>
</table>

NA= not available

A comparison of specimens 3, 4, and 9 shows that one layer of GFRP shell can safely replace lateral steel at 320 mm spacing. The peak strength enhancement of specimen 9 with a GFRP shell is comparable to that of specimens 3 and 4 with hoops at 320 mm. It can be noted that the ductility factor, energy absorption capacity, and work index of specimen 9 are significantly higher than those of specimen 3. In fact they are approximately equal to those of specimens 5 and 6 with spirals at 75 mm pitch. It is evident that specimen 13 with two layers of GFRP shells has all the parameters higher than that of specimens 5 and 6 (even though specimen 13 did not perform well). Thus, a specimen confined with two layers of GFRP shells exhibits better performance than a
specimen confined with spirals at 75 mm spacing. It can be stated that two layers of GFRP shells have the potential to replace spirals at 75 mm spacing.

The behaviour of specimen 17 with fibres inclined at 45° showed lesser improvements in strength but higher improvements in ductility factors, energy absorption capacity, and work index.

6.5.5 EFFECT OF LONGITUDINAL FIBRES ON COLUMN BEHAVIOR

Effect of longitudinal fibres on the behaviour of GFRP-encased columns can be examined by considering behaviours of specimens 7 and 8, and specimens 11 and 12. Table 6.9 compares the behaviour of these specimens.

Table 6.9 Effect of Longitudinal Fibres on Column Behaviour

<table>
<thead>
<tr>
<th>Name</th>
<th>$P_{\text{max}}$ (kN)</th>
<th>$f_{\text{ccmax}}$ (MPa)</th>
<th>Peak load enhancements</th>
<th>Ductility</th>
<th>Energy absorption capacity</th>
<th>Work index</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>$K_1$ $K_2$ $\mu$ $e_{80}$ $e_{50}$ $e_{20}$</td>
<td></td>
<td>$W_{80}$ $W_{50}$ $W_{20}$</td>
<td></td>
</tr>
<tr>
<td>G01-00-7</td>
<td>3460</td>
<td>34.77</td>
<td>1.364 1.159 9.08 0.336 0.517 0.660</td>
<td>8.14</td>
<td>12.53 16.00</td>
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</tr>
<tr>
<td>G11-00-8</td>
<td>3820</td>
<td>38.41</td>
<td>1.506 1.280 8.57 0.390 0.500 0.830</td>
<td>7.75</td>
<td>9.93 16.48</td>
<td></td>
</tr>
<tr>
<td>G02-00-11</td>
<td>4460</td>
<td>44.82</td>
<td>1.758 1.494 13.56 0.817 1.186 1.346</td>
<td>11.92</td>
<td>17.30 19.63</td>
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</tr>
<tr>
<td>G22-00-12</td>
<td>5450</td>
<td>54.76</td>
<td>2.148 1.825 14.16 1.320 1.528 1.716</td>
<td>12.90</td>
<td>14.93 16.77</td>
<td></td>
</tr>
</tbody>
</table>

NA= not available

It can be observed that glass fibres in the longitudinal direction slightly improve the peak strength enhancements. For specimen 8 with one layer of GFRP shell, peak strength enhancement improved by 10%, while for specimen 12 with two layers of GFRP shell it improved by 22%. Comparing the ductility factors and work index for these specimens, it can be observed that the glass fibres in the longitudinal direction do not have any significant affect on these parameters. Energy absorption capacities for the specimens with two layers of GFRP shells show that the presence of longitudinal GFRP improves the total energy dissipation although the non-dimensionalized indices (W) may not reflect any improvement. It can be stated that the glass fibres in the longitudinal direction did not significantly improve the overall behaviour of the column specimens.
6.6 SPECIMENS CONFINED WITH FIBRES INCLINED AT 45°

Specimens 16 and 17 were confined with two layers of GFRP shells with fibres at 45°. Specimen 17 was reinforced with longitudinal steel while specimen 16 had no longitudinal reinforcement. The results of both the specimens are shown in Table 6.10.

<table>
<thead>
<tr>
<th>Name</th>
<th>P&lt;sub&gt;max&lt;/sub&gt; (kN)</th>
<th>f&lt;sub&gt;ecmax&lt;/sub&gt; (MPa)</th>
<th>Peak load enhancements</th>
<th>Ductility</th>
<th>Energy absorption capacity</th>
<th>Work index</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>K&lt;sub&gt;1&lt;/sub&gt;</td>
<td>K&lt;sub&gt;2&lt;/sub&gt;</td>
<td>μ</td>
<td>ε&lt;sub&gt;80&lt;/sub&gt;</td>
<td>ε&lt;sub&gt;50&lt;/sub&gt;</td>
<td>ε&lt;sub&gt;20&lt;/sub&gt;</td>
</tr>
<tr>
<td>G45°2-00-16</td>
<td>3740</td>
<td>37.60</td>
<td>1.475</td>
<td>1.253</td>
<td>13.93</td>
<td>0.610</td>
</tr>
<tr>
<td>G45°2-L0-17</td>
<td>4250</td>
<td>36.11</td>
<td>1.416</td>
<td>1.204</td>
<td>38.41</td>
<td>1.600</td>
</tr>
</tbody>
</table>

It is observed that the values of peak stress and peak strength enhancement for both the specimens are in good agreement whereas the values of ductility factor, energy absorption capacity, and work index are significantly different. Specimen 17 shows very high values of ductility, energy absorption capacity, and work index as compared to those of specimen 16. The data was reconsidered for any human or observational error to investigate the inconsistency in behaviour of these particular specimens. Finally it was decided to cut GFRP from the shells used to confine these columns, make coupons and test them.

One GFRP coupon of dimensions 127 x 432 mm (5 x 17 in.) was cut from each specimen shell. The length of the coupon was 432 mm (17 in.) with gauge length of 127 mm and grip length of 152.5 mm on each end. As the fibres were running at 45°, it was not possible to grip all the fibres at both the ends in the coupon. Therefore a large [127 x 127 mm (5 x 5 in.)] test region was taken in order to get a representative behaviour of the GFRP. The coupons had curvature equal to the curvature of the columns. Metallic plates [127 x 152.5 mm (5 x 6 in.)] used at the end were also curved to provide better grip for the GFRP as shown in Figure 6.10. It is worth mentioning here that the surfaces directly in contact with the GFRP were curved while the exterior surfaces were flat (surfaces not in contact with GFRP).
TYFO epoxy resin was roll-brushed on the grip portions. Then, the metal plates were placed on the grip portions. Extra GFRP sheets of the same dimension as the metal plates, impregnated with TYFO epoxy resin, were placed in between the coupon and the metal plates. These GFRP sheets placed in the grip portion helped in bonding the metal plates and the coupons. The metal plates at both ends of the coupons were held tightly in a vise. The assembly was cured for seven days at room temperature.

The coupons were tested using a MTS machine. The cross-section of the coupon in the test set up is shown in Figure 6.11. The strains were measured by an LVDT attached by a special arrangement, comprising of a small pulley and a wire attached to the LVDT (Figure 6.11). Furthermore, a strain gauge (YL-60) was also attached on the GFRP coupon. The force versus strain curves obtained from the test data are shown in Figure 6.12. It was observed that the failure of the GFRP coupons occurred along a rupture plane inclined at 45° to the horizontal as shown in Figure 6.13a and Figure 6.13b. The coupon from specimen 17 did not perform well during the test. This can also be justified from the failure plane shown in Figure 6.13b. The reasons can be attributed to improper grip or improper alignment in the testing machine.

It is apparent from the plots that GFRP with fibres inclined at 45° show high strains at rupture. Further the curves are parabolic unlike for the GFRP without inclined fibres. These observations establish the fact quite reasonably that GFRP-confined specimens with fibres at 45° will exhibit higher ductility as compared to those with fibres in the lateral direction. However, the phenomenon responsible for the inconsistency of
these two specimens could not be ascertained from the tests. More experimental research needs to be performed in order to investigate the behaviour of GFRP-confined specimens with fibres at 45°.

Figure 6.11 Cross-Section of the GFRP Coupon in the Test Setup
Figure 6.12 Force versus Strain Curves for GFRP Coupons with Fibres at 45°
Figure 6.13a Coupon from Specimen 16 after testing
Figure 6.13b Coupon from Specimen 17 after testing
6.7 BEHAVIOUR OF GFRP SHELL IN THE LATERAL DIRECTION

During the tests, the lateral strains in the GFRP shell were measured by two strain gauges in the lateral direction on the shells. Table 6.11 gives the values of lateral strain in GFRP at the peak axial load and also the maximum lateral strains measured in the GFRP. Plots between lateral strains and axial strains of the specimens are shown in Figure 6.14a through Figure 6.14j. The axial strains at any given concrete stress could be read from the stress-strain curve of any specimen, and then the lateral strain could be obtained corresponding to that axial strain from the plots of lateral strain versus axial strain. With the increase in load and axial strain, the lateral strain also increased.

It was observed that for specimens 10 and 17, the GFRP shell ruptured just under the lateral strain gauges at the peak load. For specimen 10, it ruptured under strain gauge S1, and for specimen 17 it ruptured only under lateral strain gauge S3. For all other specimens, the lateral strain gauges were reporting strain values below the rupture strain of GFRP shell at the peak load. However, it is important to note that rupture at the peak load did not occur at a location directly under the lateral strain gauges. Furthermore, in some specimens, the lateral strain readings obtained from the two strain gauges were giving significantly different values. This was due to the reason that in most of the specimens, gauges were not able to read the rupture as the rupture took place elsewhere.
Table 6.11 Lateral Strains in GFRP at Peak Axial Stresses.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Specimen</th>
</tr>
</thead>
<tbody>
<tr>
<td>7</td>
<td>G01-00-7</td>
</tr>
<tr>
<td>8</td>
<td>G11-00-8</td>
</tr>
<tr>
<td>9</td>
<td>G01-L0-9</td>
</tr>
<tr>
<td>10</td>
<td>G01-LS320-10</td>
</tr>
<tr>
<td>11</td>
<td>G02-00-11</td>
</tr>
<tr>
<td>12</td>
<td>G22-00-12</td>
</tr>
<tr>
<td>13</td>
<td>G02-L0-13</td>
</tr>
<tr>
<td>14</td>
<td>G02-LS320-14</td>
</tr>
<tr>
<td>15</td>
<td>G02-LS75-15</td>
</tr>
<tr>
<td>16</td>
<td>G45°2-00-16</td>
</tr>
<tr>
<td>17</td>
<td>G45°2-L0-17</td>
</tr>
</tbody>
</table>

- **ε'**
- **ε_in GFRP corresponding to peak axial stress**
- **Max. ε_in GFRP measured**

<table>
<thead>
<tr>
<th>Specimen</th>
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</tr>
</thead>
<tbody>
<tr>
<td>7</td>
<td>G01-00-7</td>
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<td>16</td>
<td>G45°2-00-16</td>
</tr>
<tr>
<td>17</td>
<td>G45°2-L0-17</td>
</tr>
</tbody>
</table>

NA= not available
Specimen 7
G01-00-7
1 Layer of lateral GFRP
No longitudinal GFRP
No longitudinal steel
No lateral steel

Figure 6.14a Average Axial Strain versus Average Transverse Strain
Figure 6.14b Average Axial Strain versus Average Transverse Strain
Figure 6.14c Average Axial Strain versus Average Transverse Strain
Figure 6.14d Average Axial Strain versus Average Transverse Strain
Results and Discussion

Figure 6.14e Average Axial Strain versus Average Transverse Strain

Specimen 11
G02-00-11
2 Layers of lateral GFRP
No longitudinal GFRP
No longitudinal steel
No lateral steel
Figure 6.14f Average Axial Strain versus Average Transverse Strain
Figure 6.14g Average Axial Strain versus Average Transverse Strain
Figure 6.14h Average Axial Strain versus Average Transverse Strain
Figure 6.14: Average Axial Strain versus Average Transverse Strain

2 Layers of lateral GFRP at 45 degrees
No longitudinal GFRP
No longitudinal steel
No lateral steel

Specimen 16
C45-2-00-16
Results and Discussion

Figure 6.14: Average Axial Strain versus Average Transverse Strain

Specimen 17
G452-L0-17
2 Layers of lateral GFRP at 45 degrees
No longitudinal GFRP
Longitudinal steel (6-20M)
No lateral steel
6.8 SUMMARY

Analyses of the test results are discussed in this chapter. A comparative study of various characteristics such as strength enhancement, ductility, work index, and energy absorption capacity is presented. Stress-strain curves of all the specimens are plotted. Effects of various factors including the numbers of layers of GFRP shells, presence of longitudinal and lateral steel, and presence of longitudinal glass fibres on the behaviour of columns are also reported.
CHAPTER 7
CONCLUSIONS AND RECOMMENDATIONS

7.1 GENERAL

With the advancement in the field of FRP materials and their successful experimental application as a retrofitting and repair material, detailed information regarding the behaviour of concrete columns reinforced externally with different types of Fibre Polymers is needed. The main objective of this study was to evaluate the behaviour of large-scale concrete columns confined with different number of layers of prefabricated GFRP shells subjected to monotonic concentric axial loading. Effects of various factors, such as amount and orientation of FRP confinement and presence of FRP reinforcement in the longitudinal direction on the strength and ductility of the columns were investigated. A total of seventeen columns were constructed and tested. Eleven of the seventeen columns contained GFRP shells while six columns did not have any FRP. All of the specimens had a diameter of 356 mm and an overall height of 1524 mm. The 28-day strength of concrete used in all the specimens was 30 MPa.

7.2 CONCLUSIONS

The study performed on concrete columns confined by prefabricated GFRP shells indicates that this method of confining concrete is effective in significantly improving the performance of concrete columns.

The following conclusions can be drawn from the results of this study:

1. There is a significant increase in compressive strength, axial strain at peak stress, ductility, energy absorption capacity, and work index of concrete columns owing to confinement provided by GFRP shells. The increase in peak strength enhancements ranged between 10% and 33% when confined with one layer of GFRP shell and ranged between 40% and 76% when confined with two layers of GFRP shells. The ductility factor increased by 155% to 190% when confined with one layer of GFRP and by 170% to 280% when confined
with two layers of GFRP shells. Similar trends of improvement are observed in energy absorption capacity and work index.

2. The stress-strain response of confined concrete improves considerably with the increase in the number of layers of GFRP.

3. Comparing the behaviour of specimens confined by lateral steel at 320 mm spacing and that of specimens confined by one layer of GFRP shell, it can be stated that one layer of GFRP shell can safely replace lateral steel at 320 mm spacing for confinement purposes.

4. Two layers of GFRP shell have a potential to replace spirals at 75 mm spacing for confinement. This is found by comparing the response of specimens confined with spirals at 75 mm pitch and those of specimens confined with two layers of GFRP shell.

5. Fibres in the longitudinal direction increased the load carrying capacity but did not contribute significantly towards improving the over-all behaviour of the specimens tested under compression.

6. Columns confined with both GFRP and lateral steel performed the best among all the specimens in the test series.

7. The effect of GFRP shells with fibres inclined at 45° on the behaviour of specimens was inconclusive.

8. The stress-strain response of well-confined specimens shows two slopes prior to peak stress. The first slope represents the unconfined concrete while the second slope represents the confining effect of the GFRP.

7.3 RECOMMENDATIONS

1. In this study only one specimen of each type/configuration was tested, therefore, additional tests are required for every specimen type/configuration to confirm the findings of this study. Further, the behaviour of GFRP confined columns under axial loading and bending needs to be investigated.

2. The number of lateral strain gauges used on the surface of GFRP shell need to be increased. A new approach should be developed in order to measure
average lateral strains of GFRP rather than taking the mean of two strain gauges on the shell, which provide local strain readings.

3. GFRP shells with fibres at 45° orientation need to be studied in a greater depth in order to understand its behaviour.

7.4 SUMMARY

This chapter presents the conclusions that can be drawn from the research conducted. A few recommendations for future study are also made.
LIST OF REFERENCES


5. Roy, H. E. H.; and Sozen, M. A.; “Ductility of Concrete”, Flexural Mechanics of Reinforced Concrete, SP-12, American Concrete Institute/American Society of Civil Engineers, Detroit, 1965, 213-224 pp.


43. American Concrete Institute; “Building Regulations for Reinforced Concrete (ACI 318-41)”, ACI, 1941.


APPENDIX A

SPECIMEN BEHAVIOUR DURING TESTING
Figure A.1.a Applied Load vs. Average Axial Strain (LVDT Readings)
Figure A.2.a  Applied Load vs. Average Axial Strain (LVDT Readings)
SPECIMEN 3

![Graph](image1)

**Figure A.3.a** Applied Load vs. Average Axial Strain (LVDT Readings)

![Graph](image2)

**Figure A.3.b** Applied Load vs. Average Axial Strain (Strain Gauges on Longitudinal Bars)
Figure A.3.c  Applied Load vs. Average Lateral Strain (Strain Gauges on Hoops)
Figure A.4.a  Applied Load vs. Average Axial Strain (LVDT Readings)

Figure A.4.b  Applied Load vs. Average Axial Strain (Strain Gauges on Longitudinal Bars)
Figure A.4.c  Applied Load vs. Average Lateral Strain (Strain Gauges on Hoops)
Figure A.5.a Applied Load vs. Average Axial Strain (LVDT Readings)

Figure A.5.b Applied Load vs. Average Axial Strain (Strain Gauges on Longitudinal Bars)
Figure A.5.c  Applied Load vs. Average Lateral Strain (Strain Gauges on Spiral)
SPECIMEN 6

Specimen 6
00-LS75-6
No lateral GFRP
No longitudinal GFRP
Longitudinal steel (6-20M)
#3 Spiral at 75mm pitch

Figure A.6.a Applied Load vs. Average Axial Strain (LVDT Readings)

Figure A.6.b Applied Load vs. Average Axial Strain (Strain Gauges on Longitudinal Bars)
Figure A.6.c  Applied Load vs. Average Lateral Strain (Strain Gauges on Spiral)
**SPECIMEN 7**

![Graph showing Applied Load vs. Average Axial Strain (LVDT Readings)](image)

**Figure A.7.a** Applied Load vs. Average Axial Strain (LVDT Readings)

![Graph showing Applied Load vs. Average Axial Strain (Strain Gauges on GFRP shell)](image)

**Figure A.7.b** Applied Load vs. Average Axial Strain (Strain Gauges on GFRP shell)
SPECIMEN 7 (Cont.)

Figure A.7.c Applied Load vs. Average Lateral Strain (Strain Gauges on GFRP shell)
SPECIMEN 8

Figure A.8.a Applied Load vs. Average Axial Strain (LVDT Readings)

Figure A.8.b Applied Load vs. Average Axial Strain (Strain Gauges on GFRP shell)
SPECIMEN 8 (Cont.)

Figure A.8.c Applied Load vs. Average Lateral Strain (Strain Gauges on Lateral steel)
SPECIMEN 9

Figure A.9.a Applied Load vs. Average Axial Strain (LVDT Readings)

Figure A.9.b Applied Load vs. Average Axial Strain (Strain Gauges on Longitudinal Bars)
Figure A.9.c Applied Load vs. Average Axial Strain (Strain Gauges on GFRP shell)

Figure A.9.d Applied Load vs. Average Lateral Strain (Strain Gauges on GFRP shell)
SPECIMEN 10

Figure A.10.a Applied Load vs. Average Axial Strain (LVDT Readings)

Figure A.10.b Applied Load vs. Average Axial Strain (Strain Gauges on Longitudinal Bars)
Figure A.10.c Applied Load vs. Average Axial Strain (Strain Gauges on GFRP shell)

Figure A.10.d Applied Load vs. Average Lateral Strain (Strain Gauges on Lateral steel)
Figure A.10.e Applied Load vs. Average Lateral Strain (Strain Gauges on GFRP shell)
SPECIMEN 11

Figure A.11.a Applied Load vs. Average Axial Strain (LVDT Readings)

Figure A.11.b Applied Load vs. Average Axial Strain (Strain Gauges on GFRP shell)
Figure A.11.c Applied Load vs. Average Lateral Strain (Strain Gauges on GFRP shell)
SPECIMEN 12

Figure A.12.a Applied Load vs. Average Axial Strain (LVDT Readings)

Figure A.12.b Applied Load vs. Average Axial Strain (Strain Gauges on GFRP shell)
Figure A.12.c Applied Load vs. Average Lateral Strain (Strain Gauges on GFRP shell)
Specimen 13

G02-L0-13
2 layers of lateral GFRP
No longitudinal GFRP
Longitudinal steel (6-20M)
No lateral steel

Figure A.13.a Applied Load vs. Average Axial Strain (LVDT Readings)

Figure A.13.b Applied Load vs. Average Axial Strain (Strain Gauges on Longitudinal Bars)
Figure A.13.c Applied Load vs. Average Axial Strain (Strain Gauges on GFRP shell)

Figure A.13.d Applied Load vs. Average Lateral Strain (Strain Gauges on GFRP shell)
SPECIMEN 14

Figure A.14.a Applied Load vs. Average Axial Strain (LVDT Readings)

Figure A.14.b Applied Load vs. Average Axial Strain (Strain Gauges on Longitudinal Bars)
SPECIMEN 14 (Cont.)

Figure A.14.c Applied Load vs. Average Axial Strain (Strain Gauges on GFRP shell)

Figure A.14.d Applied Load vs. Average Lateral Strain (Strain Gauges on Lateral steel)
Figure A.14.e Applied Load vs. Average Lateral Strain (Strain Gauges on GFRP shell)
SPECIMEN 15

Figure A.15.a Applied Load vs. Average Axial Strain (LVDT Readings)

Figure A.15.b Applied Load vs. Average Axial Strain (Strain Gauges on Longitudinal Bars)
Figure A.15.c Applied Load vs. Average Axial Strain (Gauges on GFRP shell)

Figure A.15.d Applied Load vs. Average Lateral Strain (Strain Gauges on Spiral)
SPECIMEN 15 (Cont.)

Figure A.15.e Applied Load vs. Average Lateral Strain (Strain Gauges on GFRP shell)
SPECIMEN 16

![Graph showing Specimen 16 with details on the graph.]

**Figure A.16.a** Applied Load vs. Average Axial Strain (LVDT Readings)

![Graph showing Applied Load vs. Average Axial Strain (Strain Gauges on GFRP shell).]

**Figure A.16.b** Applied Load vs. Average Axial Strain (Strain Gauges on GFRP shell)
Figure A.16.c Applied Load vs. Average Lateral Strain (Strain Gauges on GFRP shell)
SPECIMEN 17

Specimen 17
G452-L9-17
2 layers of lateral GFRP at 45 degrees
No longitudinal GFRP
Longitudinal steel (6-20M)
No lateral steel

Figure A.17.a Applied Load vs. Average Axial Strain (LVDT Readings)

Figure A.17.b Applied Load vs. Average Axial Strain (Strain Gauges on Longitudinal Bars)
Figure A.17.c Applied Load vs. Average Axial Strain (Strain Gauges on GFRP shell)

Figure A.17.d Applied Load vs. Average Lateral Strain (Strain Gauges on GFRP shell)