

EXPERIMENTAL CHARACTERISATION OF CONFINED CONCRETE IN DIRECT COMPRESSION AND FLEXURE

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Degree of**

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Structural Engineering

Submitted by

Jaimeet singh

(2K24/STE/16)

Under the supervision of

Asst. Prof. Hrishikesh Dubey

Assistant Professor



**DEPARTMENT OF CIVIL ENGINEERING
DELHI TECHNOLOGICAL UNIVERSITY
(Formerly Delhi College of Engineering) Bawana Road, Delhi- 110042**

May, 2026

DEPARTMENT OF CIVIL ENGINEERING
DELHI TECHNOLOGICAL UNIVERSITY
(Formerly Delhi College of Engineering) Bawana Road, Delhi- 110042

CANDIDATE’S DECLARATION

I, **Jaimeet singh**, hereby declare that the work which is being presented in dissertation entitled “**Experimental Characterisation of Confined Concrete in Direct Compression and Flexure**” which is submitted by me to the partial fulfillment of the requirement for the award of the degree of **Master of Technology**, submitted in the Department of Civil Engineering, Delhi Technological University, Delhi is an authenticate record of my own work carried out during the period from 2025 to 2026 under the supervision of **Hrishikesh dubey**. The matter presented in the dissertation has not been submitted by me for the award of any other degree or any other institute

Candidate’s Signature

Date: May 31, 2026

DEPARTMENT OF CIVIL ENGINEERING
DELHI TECHNOLOGICAL UNIVERSITY
(Formerly Delhi College of Engineering) Bawana Road, Delhi- 110042

CERTIFICATE

Certified that **Jaimeet singh (2K24/STE/16)** has carried out his research work presented in this dissertation entitled “**Experimental Characterisation of Confined Concrete in Direct Compression and Flexure**” for the award of **Master of Technology from the Department of Civil Engineering, Delhi Technological University, Delhi;** under my supervision. The dissertation embodies results of original work, and studies are carried out by the student himself and the contents of the dissertation do not form the basis for the award of any other degree to the candidate or to anybody else from this or any other University.

Asst. Prof. Hrishikesh Dubey

(Asst. professor & Supervisor)

Date: May 31, 2026

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(2k24/STE/16)

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ABSTRACT

The global aging and deterioration of critical civil infrastructure have driven the advancement of retrofitting and strengthening techniques. This study provides an in-depth experimental characterisation of confined concrete subjected to direct compression and flexural loading. The primary objective is to evaluate the structural performance improvements of M20 grade concrete specimens externally wrapped with unidirectional Carbon Fiber Reinforced Polymer (CFRP) applied with a high-strength epoxy binder. The experimental program included 18 precisely cast specimens: 6 concrete cubes (150x150x150 mm) for standard compressive strength assessment, 6 concrete cylinders (150x300 mm) for evaluating slenderness and axial compression, and 6 concrete beams (100x100x500 mm) for analyzing flexural strength and displacement ductility. In each geometric category, half of the specimens served as unconfined controls to establish baseline performance, while the other half were wrapped with 200 GSM unidirectional CFRP jackets. This research is highly influenced by the displacement-ductility based design methods and mainly by the analytical models developed by Pantelides et al. (2012). The required thickness of the CFRP jacket was mathematically derived using parameters such as the required stiffness of the FRP jacket, characteristic dimension that targets the ultimate compressive strain to ensure optimal dilation control and to prevent strain-softening behavior in the concrete core. The experimental results showed significant improvement in the load-carrying capacity and deformation characteristics of the CFRP confined specimens over all testing regimes. Under direct compressive loading, the CFRP wrapped cubes and cylinders showed significantly higher ultimate peak strengths and prolonged ductility than unconfined cubes and cylinders. The circular cross-sections (cylinders) proved to be highly effective in utilizing hoop stresses arising from FRP confinement. Furthermore, the flexural behavior of the CFRP-strengthened beams showed a significant increase in peak flexural load and considerable energy absorption capacity before ultimate failure, changing the failure mode from brittle catastrophic to ductile and progressive. This extensive thesis details the relation of the experimental data to the existing analytical models of confinement in detail and provides the load deformation paths, failure mode analyses and useful engineering insights for the upgrading of existing elements designed using regular M20 design mix concrete.

CHAPTER 1

INTRODUCTION

1.1 Background and Motivation

The rapid growth of urban infrastructure during the last century has resulted in a huge inventory of reinforced concrete (RC) structures throughout the world. India alone has an estimated stock of several hundred thousand reinforced concrete bridges, flyovers, industrial plants, and multistory buildings, a considerable proportion of which were designed and constructed between 1950 and 1990 using codes and material specifications that are no longer considered adequate. However, a significant number of these buildings are experiencing structural problems today. These deficiencies are attributed to multiple causes, including environmental degradation, corrosion of internal steel reinforcement, continuous exposure to harsh climatic conditions, increased service payloads beyond original design parameters, and the development of more stringent seismic design codes. In seismically active regions such as the Indo-Gangetic plain and the Himalayan foothills, many existing structures were designed without any consideration of lateral earthquake forces, making their retrofitting an urgent engineering priority.

Efficient strengthening and retrofitting approaches are of vital importance to avoid disastrous failures and to prolong the service life of such structures. The cost of demolition and reconstruction of deficient infrastructure is not only economically prohibitive but also generates enormous quantities of construction and demolition waste, placing an additional burden on landfills and the environment. Studies by the World Bank have estimated that rehabilitating existing structures can be three to five times more cost-effective than full replacement over a comparable service-life horizon. The destruction and rebuilding of inadequate structures are often economically unviable and environmentally unsustainable. As a result, the focus of the civil engineering community has shifted towards advanced rehabilitation methods. There are many retrofitting techniques available, such as steel

jacketing, concrete section enlargement and external post-tensioning. Steel jacketing, while effective in increasing the cross-sectional area and confining pressure, introduces significant additional self-weight, is susceptible to corrosion at the steel-concrete interface, and demands heavy lifting equipment and skilled labor for on-site installation. Reinforced concrete section enlargement, though widely used, reduces the usable floor area and significantly increases the structural dead load, which can be counterproductive in existing buildings already near their design limits. However, externally bonded Fiber Reinforced Polymer (FRP) composites have emerged as a revolutionary solution that effectively overcomes the limitations of conventional methods by offering an unparalleled combination of high tensile strength, extremely low self-weight, corrosion immunity, and rapid field applicability.

1.2 Beyond Steel and Concrete: The Rise of FRP in Modern Construction

Fiber Reinforced Polymers (FRPs) consist of high strength fibers (e.g. Carbon, Glass or Aramid) embedded in a polymeric resin matrix (usually Epoxy). The fibers are the main load-bearing material, providing high strength and stiffness in tension, while the epoxy matrix binds the fibers together, transfers shear stresses between them, and protects them from environmental damage. Unlike isotropic materials such as steel, FRP composites are orthotropic in nature, meaning their mechanical properties are highly directional. This characteristic is strategically exploited in structural retrofitting: by orienting the fibers in the transverse (hoop) direction around a column, the composite jacket maximally resists the lateral dilation of the concrete core under axial compression; alternatively, fibers placed in the longitudinal direction on the tension face of a beam provide direct tensile reinforcement to improve its flexural capacity.

Specifically, Carbon Fiber Reinforced Polymer (CFRP) has a strength-to-weight ratio that cannot be matched by conventional structural materials. Its tensile strength can exceed 3,500 MPa — nearly ten times that of typical structural steel — while its density is

approximately one-fifth that of steel, making handling and transportation remarkably straightforward. CFRP is resistant to galvanic corrosion, highly resistant to chemical attack, and does not require heavy lifting equipment for site installation. Its thin jacket profile (often less than 2 mm) minimally alters the external geometry of the retrofitted member. CFRP has been successfully demonstrated in field applications and laboratory research to increase the flexural strength, shear capacity, and axial load-carrying capacity of concrete members across a wide range of structural configurations, from circular bridge piers to rectangular building columns and simply supported beams.



Figure 1:CFRP Wrapped Concrete Cube



Figure 2: CFRP Wrapped Concrete Cylinder

1.3 Mechanism of Concrete Confinement

By nature, concrete is a brittle material with its compressive strength far exceeding its tensile strength — typically by a ratio of approximately 8 to 15 times. Under uniaxial compressive loading, the Poisson effect causes the concrete column to shorten axially while simultaneously expanding laterally. As the applied load increases and approaches the ultimate compressive strength of the unconfined concrete, internal micro-cracks initiate and propagate primarily in the direction parallel to the applied compressive load.

These micro-cracks coalesce progressively, and the accompanying lateral dilation eventually causes explosive spalling of the concrete cover and rapid degradation of the core, resulting in a sudden, brittle failure mode that is particularly dangerous in structures subjected to seismic loading. This lateral expansion is physically restrained by wrapping the concrete member externally with a CFRP jacket oriented with its fibers in the hoop or transverse direction. The CFRP jacket develops a passive confining pressure on the concrete core: it exerts zero confining force when the specimen is unloaded, but as the concrete dilates radially under increasing axial compression, the CFRP jacket is stretched in tension and thus passively applies an inward-directed confining pressure that grows in direct proportion to the lateral dilation of the core. This passive confining pressure fundamentally changes the stress state of the concrete. What was originally a uniaxial compressive stress state (stress in one direction only) is transformed into a triaxial compressive stress state (compressive stresses acting in all three principal directions simultaneously). Under triaxial compression, the micro-cracks that would otherwise grow and coalesce are instead clamped shut by the lateral confining pressure, dramatically suppressing crack propagation and dilation. The apparent compressive strength of the confined concrete is thus drastically elevated compared to the unconfined case, and the stress-strain response transitions from brittle post-peak softening to highly ductile strain-hardening behavior, continuing to carry load at large axial strains until the hoop tensile stress in the CFRP jacket reaches its rupture limit and the jacket fractures.

1.4 Objectives of the Research

To prepare a total of 18 specimen to fully explore this phenomenon, specific objectives for this dissertation are stated as follows:

1. Specimens (cubes, cylinders and beams) of M20 grade concrete with meticulous design and casting in accordance with IS standards.

2. To review the theoretical background of the phenomenon (mainly Pantelides et al., 2012) and to mathematically determine the necessary thickness of the FRP jacket in order to control the dilation of the concrete and avoid concrete strain softening.
3. To find out the exact amount of GSM (Grams per square meter) of 200 GSM Unidirectional Carbon Fiber required and the surface area needed for the specimens to be wrapped.
4. To test unconfined concrete cube, CFRP-confined concrete cube and cylinder specimens by direct compression to record high-resolution load-deformation curve information.
5. To perform flexural testing of unconfined and CFRP strengthened concrete beams to examine their energy absorption and displacement ductility.
6. To investigate the failure modes of both normal and retrofitted specimens and compare the empirical results to the theoretical results.

1.5 Scope and Limitations

The scope of this experimental program is rigorously bound to evaluating M20 grade concrete, which was selected as the target mix grade because it constitutes the most widely used concrete class in the older Indian building stock and represents the minimum structural concrete grade specified under IS 456:2000 for reinforced concrete construction. The external reinforcement is limited strictly to Unidirectional Carbon Fiber of 200 GSM density, bonded using a two-part structural epoxy resin system applied under ambient laboratory temperature conditions. The number of CFRP layers (four) was determined analytically based on the Pantelides et al. (2012) displacement-ductility design framework, as detailed in Chapter 3. The structural elements are tested under monotonic static loading conditions using a Universal Testing Machine, which generates a continuous, increasing load applied in a single direction without reversal. This loading regime was selected to isolate the fundamental confinement mechanics and to establish baseline load-deformation responses unaffected by cyclic degradation effects. Fatigue

loading, dynamic and impulsive loading, seismic cycling involving reversed lateral displacements, and long-term environmental durability assessments (such as accelerated aging under freeze-thaw cycling, wet-dry cycling, prolonged UV radiation exposure, or alkaline environment immersion) are explicitly excluded from the scope of this particular study. Additionally, this program does not examine higher-strength concrete grades (M30 and above), alternative FRP fiber types such as Glass FRP (GFRP) or Aramid FRP (AFRP), or bidirectional/woven fiber configurations. These deliberate exclusions delineate clear boundaries for the study while simultaneously paving the way for structured future research endeavors, as summarized in Chapter 5.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction to the Literature

In the past 30 years, the conceptualization and mathematical modeling of FRP confinement have developed quite a bit. The early empirical models were mostly modifications of the well-known model of confinement for steel spirals (Richart et al., 1928). However, since FRP materials are linear perfectly elastic up to their rupture point, unlike steel which yields, new mechanics-based models were called for. This chapter summarizes the key papers that have laid the foundations for this experimental research.

Bisby, Dent, & Green (2005) evaluated that the predictive accuracy of various existing analytical confinement models against a comprehensive experimental database of Fiber Reinforced Polymer (FRP) wrapped concrete columns. The authors compiled a extensive complete experimental testing database of columns wrapped with different fiber types (Glass, Aramid, and Carbon). They assessed various analytical design models, including the ACI 440, ISIS Canada, CSA S806, and K&G II confinement models. Statistical accuracy was quantified utilizing Mean Absolute Error (MAE) and Mean Error (ME) to benchmark the model predictions against actual experimental ultimate stress and ultimate strain values. The study revealed that many available models are derived empirically from small sample sets, leading to a significant lack of consensus on which model consistently produces accurate outcomes. The performance and error metrics (MAE/ME) of the models varied substantially when sorted by specific fiber types, proving that material-specific parameters influence structural predictions [1].

Campione (2006) investigated the compressive behavior of square cross-section concrete prisms externally wrapped with Carbon Fiber Reinforced Polymer (CFRP) sheets and to formulate a reliable analytical model capable of predicting maximum bearing capacity. An experimental framework was conducted examining key geometric parameters: local reinforcement applied at the corners versus continuous layers, the use of horizontal and vertical discontinuous strips, the total number of continuous layers, and the physical length of the specimens. The proposed analytical model incorporated local corner reinforcement configurations and length effects, and its results were validated against prior literature data. Circular cross-sections inherently display superior confinement efficiency compared to square or rectangular sections. Sharp corners and high cross-sectional slenderness ratios (side's ratio greater than two) induce severe stress concentrations and suffer from reduced wrapping effectiveness caused by the low flexural stiffness of the outer composite assembly. However, the study demonstrated that proper detailing, such as localized corner reinforcement, effectively improves the maximum bearing capacity [2].

Jiang, Jin, Lu, & Du (2023) focused on evaluating how the shear-span-to-effective-depth ratio affects brittle shear failure, structural size effects, and the actual shear contribution of CFRP sheets in wrapped reinforced concrete (RC) beams of varying depths. The research tested 24 simply supported RC beams without internal stirrups across scaled structural dimensions, reaching a maximum beam depth of 1200 mm. The test matrix isolated distinct shear-span-to-effective-depth ratios (1.0, 1.5, and 2.0) and CFRP volumetric reinforcement ratios (0% and 0.0835%) under a four-point loading configuration. Real-time data captured flexural cracking loads, ultimate shear capacities, mid-span deflections, and CFRP strain profiles. The experimental work was paired with a 3-D Meso-scale finite element model to trace localized failure progressions [4].

Li, Maricherla, Singh, Pang, & John (2006) prepared 27 concrete cylinders (152.4 mm diameter by 304.8 mm height). Out of these, 18 cylinders were wrapped using two layers of FRP configured across six different fiber orientations, 6 cylinders were wrapped using four layers with fibers placed strictly in the axial or hoop directions, and 3 cylinders served as unconfined control baselines. The composite system consisted of an E-glass fiber reinforced ultraviolet (UV) curing vinyl ester. Material properties were established via 15 coupon tensile specimens (0, 45, 90) before the cylinders underwent co-axial compression testing [5].

Elsanadedy, Al-Salloum, Alsayed, Iqbal (2012) investigated whether a structural size factor needs to be integrated into design models when using non-standard cylinder test data to predict the capacity of full-scale FRP-confined concrete components. An experimental project tested 37 concrete cylinders divided across three distinct physical scale dimensions (diameters of 50 mm, 100 mm, and 150 mm). The matrix included 13 unconfined control specimens and 24 specimens jacketed with varying layers of CFRP to evaluate specimen size and confinement stress ratios. This was supplemented by advanced non-linear finite element models generated via LS-DYNA software to numerically simulate scaling across an extended envelope of sizes and confinement ratios. Both the experimental outcomes and the validated finite element simulations confirmed that the effect of specimen size on FRP-confined concrete cylinders under uniaxial compressive loading is statistically insignificant. This indicates that standard analytical models do not require a reduction factor for column scale under uniform compression, contrasting with the severe size effects observed in members subjected to shear [3].

Pantelides & Moran (2012) aimed to present a performance-based design methodology for calculating the required FRP jacket thickness to optimize lateral deformation capacity and reach target displacement ductility levels in bridge and building columns. The research developed a displacement-based, strain-driven design procedure that

incorporates a new stress-strain model based on material damage mechanics. The required jacket thickness is derived directly from the target ultimate compressive strain and the transverse dilation behavior of the concrete core within its critical plastic hinge region. The analytical procedure was validated against historic experimental data of columns under single curvature cyclic loading. Wrapping the plastic hinge zones of RC columns with FRP jackets drastically upgrades ultimate axial compressive strains and provides sufficient confinement to prevent sudden shear failures, premature lap-splice degradation, or anchorage loss during severe seismic events. Furthermore, the study established a distinct threshold equation for the minimum required FRP thickness needed to fully suppress strain-softening and avoid uncontrolled, destructive dilation of the concrete matrix [6].

2.2 Summary of the literature review

The compiled research underscores that Fiber Reinforced Polymer (FRP) wrapping provides substantial advancements in upgrading the load capacity, shear resistance, and seismic ductility of concrete infrastructure. However, implementing these composite systems requires careful consideration of structural geometry, load conditions, fiber layouts, and scaling behaviors.

Load Conditions and Size Effects: A critical distinction must be maintained between members experiencing uniform compression versus those subjected to shear forces. For standard columns loaded uniaxially in compression, the structural size effect is mathematically and experimentally insignificant. This means empirical model values derived from small cylinder tests scale effectively to full-sized columns without reduction coefficients. Conversely, reinforced concrete members subjected to shear failure (such as beams without stirrups) exhibit a prominent size effect where generalized shear strength drops off as the component size increases. The shear-span-to-effective-depth ratio heavily

dictates how effectively the external CFRP plies can contribute to suppressing brittle failure modes.

Cross-Sectional Geometry and Fiber Layouts: Circular columns maximize the lateral confinement pressure exerted by the expanding concrete core because the circular geometry generates a perfectly uniform radial pressure distribution around the entire perimeter of the section. In contrast, square and rectangular prisms perform poorly under standard configurations because their sharp corners introduce highly localized stress concentrations where the FRP fibers must bend sharply, and the flat faces between corners provide limited lateral restraint due to their low out-of-plane flexural stiffness. Campione (2006) demonstrated through experimental data that the effective confinement area in a square section is only a fraction of the gross cross-sectional area, limited to parabolic arches that form between adjacent corner points, while the material between these arches remains largely unconfined. Modifying the cross-section with corner-rounding fillets of at least 20 mm radius or applying localized corner reinforcement strips is therefore required to more equitably distribute the confining stress and improve wrapping effectiveness. Furthermore, the alignment of the polymer fibers is paramount to the structural outcome; placing fibers off-axis or at random intermediate angles between the hoop and axial paths dilutes the tensile stiffness available to resist lateral dilation, thereby degrading total compressive capacity. These findings collectively emphasize the critical need for strict hoop-direction fiber placement and proper corner detailing to maximize confinement efficiency in non-circular cross-sections.

CHAPTER 3

METHODOLOGY AND EXPERIMENTAL SETUP

3.1 Introduction

This chapter describes the comprehensive experiment that was conducted to accomplish the goals of this study. It provides information on properties of construction materials, the strict M20 mix design process, and derivation of the thickness of FRP jacket, calculation of requirement of Gram per square meter (GSM) of FRP, casting procedure of specimens, and specific testing methods used.

3.2 Material Properties

1. Cement: Ordinary portland cement (OPC) 43 Grade as per IS 8112 was used. Standard consistency, initial setting time, and final setting time were conducted before batching.
2. Fine Aggregate: River sand available locally and as per IS 383 Grading Zone II was used. The specific gravity and water absorption were determined in order to be able to adjust the mix water properly.
3. Coarse Aggregate: Crushed angular granite stones of maximum nominal size of 20 mm, conforming to IS 383 with high interlocking strength were used.
4. Water: Potable tap water, which was free from organic impurities and deleterious salts, was used for mixing and standard curing.
5. Unidirectional Carbon Fiber sheets (200 GSM) were purchased as Carbon Fiber Reinforced Polymer (CFRP). The unidirectional configuration means that all strength is in the longitudinal axis of the fibre roll, which is ideal to counter hoop strains in cylinders and longitudinal tension in beams.

An industrial type of two-part epoxy structural adhesive (6) was applied. Provides absolute integrity of bond between the concrete substrate and the carbon fibers.

3.3 Concrete Mix Design (M20 Grade)

While the preliminary scope of the study was renovated to specifically investigate M20 grade concrete, representing a vast majority of the older infrastructure inventory in need of structural design and retrofitting. The proportions inside the mix design were calculated according to the guidelines of Indian Standard IS 10262:2019.

The target mean compressive strength (f_{ck}) refers to the:

$$F_m = f_{CK} + (1.65 \times S)$$

Where:

$$F_m = 20 \text{ N/mm}^2$$

Standard Deviation (S) for M20 = 4.0 N/mm².

$$F_m = 20 + (1.65 \times 4) = 26.6 \text{ N/mm}^2.$$

Table 3.1: Standard Deviation values for different grades of concrete

Grade of Concrete	Assumed Standard Deviation in MPa
M10 to M15	3.5
M20 to M25	4.0
M30 to M50	5.0

The water-cement (w/c) ratio was used at 0.50 based on the durability requirement for mild exposure. The final cement fine aggregate coarse aggregate proportion was calculated precisely to be around 300kg of cement to 400kg of fine aggregate to 600kg of coarse aggregate.

3.4 Derivation of FRP Thickness

The necessary thickness of the FRP jacket was derived from a governing formula, which is elaborated in the thesis by Pantelides et al. (2012) that is attached as part of this documentation. (t_j) is defined as:

$$t_j = \frac{K_{je} * H_c}{k_e * C_{sh} * \frac{E_j}{f'_c}}$$

Where,

t_j = required FRP thickness

K_{je} = required FRP jacket stiffness

H_c = dimension of specimens

For circular diameter and for rectangular smaller side

k_e = confinement efficiency factor (for circular 1 and for rectangular less than 1) comes from geometry-based confinement effectiveness of FRP jacket

C_{sh} = FRP reinforcement ratio coefficient

E_j = elastic modulus of FRP

f'_c = unconfined concrete compressive strength

3.4.1 Based on thesis , it is further derived in process of finding thickness of FRP

$$K_{je} = \text{maximum} (K_{je} - dil , K_{je} - sh)$$

The value of required FRP jacket stiffness is calculated via maximum value output from ($K_{je} - dil$) and ($K_{je} - sh$)

$K_{je} - dil$ = required FRP jacket stiffness to control dilation

$K_{je} - sh$ = required FRP jacket stiffness to prevent strain softening

3.4.2 Required FRP jacket stiffness to control dilation ($K_{je} - dil$)

$$K_{je} - dil = 35 \left(\frac{Vci}{\mu_{jp(up)} - Vci} \right)$$

Where,

Vci = initial poisson's ratio of concrete

$\mu_{jp(up)}$ = plastic dilation rate

$$\mu_{jp(up)} = \frac{\varepsilon_{ju(up)}}{\varepsilon_{cu(up)}}$$

Where,

$\varepsilon_{ju(up)}$ = FRP rupture strain

$\varepsilon_{ju(up)}$ and $\varepsilon_{cu(up)}$ is further derived

$$\varepsilon_{ju(up)} = ksh \cdot \varepsilon_{fu}$$

ksh = shape reduction factor

ε_{fu} = FRP coupon tensile strain

$$\varepsilon_{cu(up)} = Ic \cdot I\phi \cdot \varepsilon_{cu(ex)}$$

$Ic = 1.8 - 0.2(I\phi - 1)^{0.38}$ must be less than

3.4.3 Required FRP jacket stiffness to prevent strain softening ($\epsilon_{je(sh)}$)

$$\epsilon_{je(sh)} = k_e \cdot \lambda_{sh} \cdot f'_{co}$$

f'_{co} = unconfined concrete strength in (MPa)

λ_{sh} = strain hardening depends on shape factor

Take required FRP jacket stiffness maximum from required FRP jacket stiffness to prevent control dilation and required FRP jacket stiffness to prevent strain softening.

Calculation of Thickness

Step 1: Initial Parameters and Material Properties

First, the base properties for the concrete and the FRP material are established:

Concrete Design Mix (f'_c): 20 MPa (M20 concrete)

Initial Poisson's Ratio of Concrete (ν_{ci}) : 0.2

Ultimate Compressive Strain of Concrete ($\epsilon_{cu(ex)}$): 0.0035

Target Ultimate Concrete Strain ($\epsilon_{cu(up)}$): 0.0105

Target/Existing Curvature Ductility Ratio (I_o): 4

Neutral Axis Upgrading Index (I_c): 0.75 (from eq. 2.7) Pantelides et al. (2012)

FRP Coupon Tensile Strain ($\epsilon(fu)$): 0.015

Elastic Modulus of FRP Jacket (ϵ_j): 230,000 MPa

Dimension Parameter (H_c): 150 for both circular and square.

Step 2: Shape-Dependent Factors

The calculations use different reduction and efficiency factors based on the column's shape:

Shape Reduction Factor (K_{sh}): 0.666 (Circular) | 0.45 (Square)

Confinement Efficiency Factor (k_e): 1 (Circular) | 0.8 (Square)

FRP Reinforcement Ratio Coeff (Ch): 2 (Rectangular/Circular assumption) | 2.45 (Square)

Strain Hardening Shape Coefficient (γ_{sh}): 3 (from fig. 3 in the thesis) Pantelides et al. (2012).

Step 3: Strain and Dilation Calculations

Next, the rupture strains and plastic dilation rates are calculated:

FRP Rupture Strain (ϵ_{ju})

- Circular: 0.00999
- Square: 0.00675

Plastic Dilation Rate ($\mu(jp)$): Because the FRP will not be fully strained, only 40% of the theoretical value is taken based on previous experiments (page 8 in thesis).

- Circular: Theoretical is ~0.951
- Square: Theoretical is ~0.642

Step 4: Calculate Required Stiffness to Control Dilation

Using the plastic dilation rates, the stiffness required to control lateral expansion is calculated:

K_{je} ; Control Dilation (Circular): 9.3155

K_{je} ; Control Dilation (Square): 15.8064

Step 5: Calculate Required Stiffness for Strain Hardening

Next, the stiffness required to ensure strain hardening behavior is calculated:

K_{je} ; **Strain Hardening (Circular):** 60

K_{je} ; **Strain Hardening (Square):** 48

Step 6: Determine Maximum Required Stiffness

The governing design stiffness is the maximum of the two calculated stiffness values for each shape:

Circular (K_{je}): $\max(9.3155, 60) = 60$

Square (K_{je}): $\max(15.8064, 48) = 48$

Step 7: Final FRP Thickness Calculation

Using the governing K_{je} stiffness, the final required thickness of the FRP jacket is determined:

Thickness for Circular Column: $0.39 \text{ mm} \approx 0.4 \text{ mm}$

Thickness for Square Column: $0.31 \text{ mm} \approx 0.4 \text{ mm}$

3.5 GSM and Surface Area Calculation

To translate the theoretical t_j into practical material procurement, the precise surface area to be wrapped was computed.

Table 3.2:Surface area calculation for specimens

Sl No	Specimen Type	Cross Dimension (mm)	Length/Height (mm)	Area Per Unit (sq.m)	No. of Samples	Total Area (sq.m)
1	Cube	150x150	150	0.135	3	0.405
2	Cylinder	Diameter 150	300	0.1413	3	0.4239
3	Beam	100x100	500	0.200	3	0.600

Summation of total pure wrapping area required = 1.4289 sq.m

The number of overlaps were calculated to be applied as full layers to achieve the required t_j using the 200 GSM material. The procurement requirements are summarised in the table

Table 3.3:GSM area calculation and primer requirement

GSM Rating	No of Layers	Total Wrapped Area (sq.m)	Primer Req.
200	4	5.7≈ 6.27	1.5kg

6.2 sq.m as 10 % we take it as a waste while applying to the surface.

3.6 Specimen Preparation and CFRP Application Procedure

All 18 samples (6 cubes, 6 cylinders, 6 beams) were cast in standard steel molds and mechanically vibrated during casting and then cured under standard water immersion conditions for 28 days. Specimens for wrapping were totally air-dried in a lab setting after

curing to prevent moisture from affecting the bonding of the epoxy. The protocol for the application of the CFRP was a multi-step process:

Step 1: Surface Preparation – The concrete surfaces were mechanically grinded with a diamond cup wheel to clean all the laitance, dust and loose particles from the concrete surfaces. Corners were rounded off with a minimum radius of 20 mm to avoid the stress concentration, otherwise the carbon fibers would be easily broken.

Step 2: Primer Application: Application of epoxy primer was done uniformly using roller and left the primer to become tacky.

Step 3: Saturant and Wrapping - Epoxy saturated CFRP fabric was manually saturated and then tightly wrapped around the specimens with 200 GSM unidirectional CFRP fabric. In the case of cylinders and cubes, the fibers were aligned exactly perpendicular to the direction of loading (hoop direction). In the case of beams, the fibers were laid parallel to each other on the tension face. A rib roller was used aggressively to remove entrapped air bubbles and to ensure complete saturation.

Step 4: Curing – The retrofitted specimens were not disturbed for 7 days for epoxy matrix to reach ultimate mechanical strength.



Figure 3.1 Casting of Concrete Cube specimens

3.7 Experimental Testing Setup

Heavy duty Universal Testing Machine (UTM) was used for all compression testing (cubes and cylinders). The load applied at a constant strain rate. To accurately record the axial deformation corresponding to each applied load increment, high precision Linear Variable Differential Transformers (LVDTs) were used. Standard three-point bending tests were performed on the beam specimens. The specimens were simply supported, and the load was applied at the mid-span, with a concentrated point load. To accurately trace the load-deflection curve, deflection at the mid-span was continuously monitored with digital dial gauges.

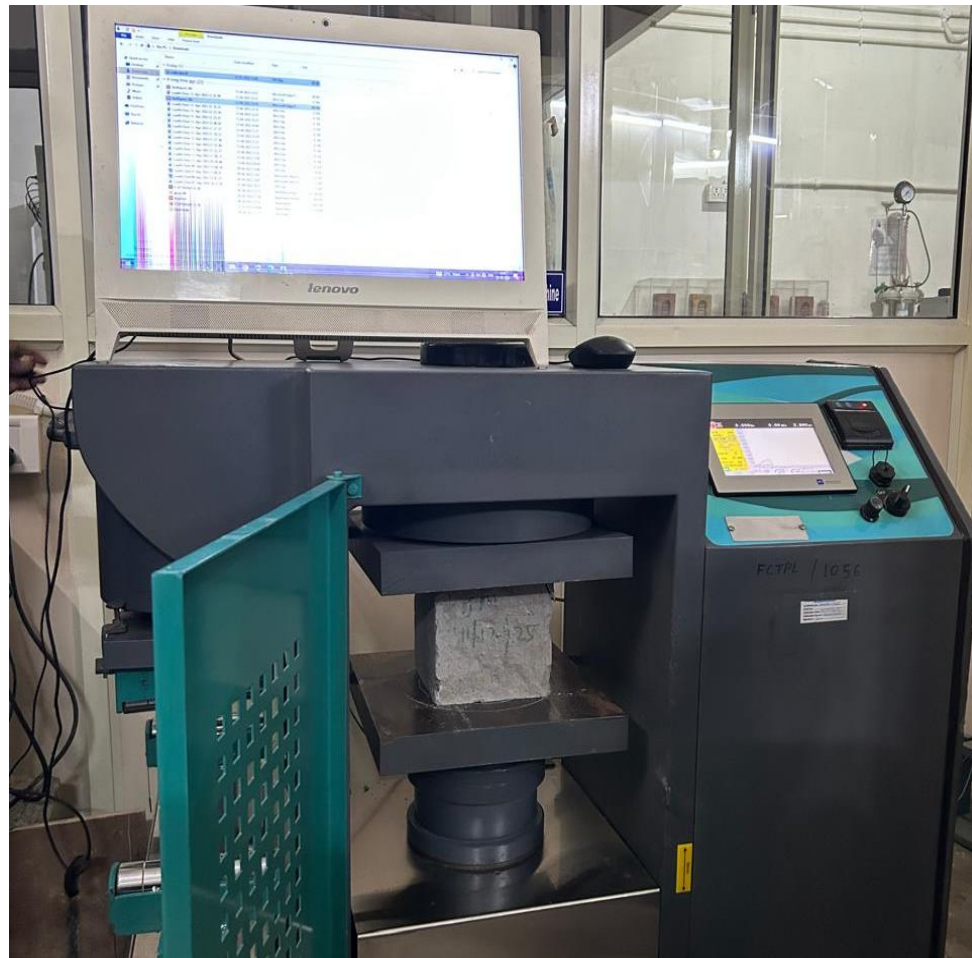


Figure 3.2 : Compressive strength test setup for concrete cube

CHAPTER 4

RESULTS AND DISCUSSIONS

4.1 Introduction to Analysis

The quantitative data collected from the experimental laboratory testing is thoroughly reported in this chapter. The data points of continuous deformation for applied loads and the calculated strengths are tabulated for all 18 samples. The discussion covers in detail the relative improvements in ultimate capacity, change in energy absorption, and the distinctly different physical modes of failure for the structural components as a result of CFRP confinement.

4.2 Compressive Strength of Concrete Cubes (150x150 mm)

The basic compressive strength of the matrix was validated using the standard 150 mm cube. It is clear from the data that the specific effect of the CFRP wrapping on a square cross section has been isolated.



Figure 4.1: Concrete cube under compressive strength testing



Figure 4.2: CFRP wrapped concrete cube under compressive strength testing

Table 4.1: Load vs Deformation table for Confined Cubes

Deformation (mm)	Normal Cube 1 (kN)	Normal Cube 2 (kN)	Normal Cube 3 (kN)
0	0	0	0
0.2	47.08	48.061	81.9
0.3	80.036	81.704	163.799
0.4	112.993	115.347	227.5
0.6	188.321	192.245	291.2
0.8	263.65	269.143	336.7
0.9	296.606	302.786	382.2
1	329.562	336.428	409.5
1.2	386.058	394.102	436.8
1.4	423.723	432.551	445.9
1.5	435.964	445.047	455
1.6	448.204	457.542	451.469
1.7	453.854	463.31	438.976
1.8	459.504	469.078	413.299
2	461.387	471	385.498

2.1	460.445	470.038	371.598
2.3	455.737	465.232	343.797
2.5	448.204	457.542	315.997
2.8	435.022	444.086	274.296

Table 4.2: Load vs Deformation table for Unconfined Cubes

Deformation (mm)	CFRP Cube 1 (kN)	CFRP Cube 2 (kN)	CFRP Cube 3 (kN)
0	0	0	0
0.2	96.168	106.02	82.8
0.3	192.335	212.039	165.599
0.4	267.132	294.499	230
0.6	341.93	376.96	294.4
0.8	395.356	435.86	340.4
0.9	448.783	494.76	386.4
1	480.839	530.1	414
1.2	512.895	565.44	441.6
1.4	523.58	577.22	450.8

1.5	534.266	589	460
1.6	530.12	584.429	456.43
1.7	515.45	568.256	443.8
1.8	485.3	535.018	417.841
2	452.656	499.03	389.735
2.1	436.334	481.036	375.682
2.3	403.69	445.048	347.576
2.5	371.046	409.06	319.47
2.8	322.08	355.078	277.311

Comparison between Confined and Unconfined Cubes

Normal Cubes Peak Loads:

Cube 1: ~461 kN

Cube 2: ~471 kN

Cube 3: ~455 kN

(Average Peak: ~462 kN. Cube 1 is closest to the true average)

CFRP Cubes Peak Loads:

Cube 1: ~534 kN

Cube 2: ~589 kN

Cube 3: ~460 kN

(Average Peak: ~527 kN. Cube 1 is closest to the true average)

So, we plot Normal Cube 1 vs. CFRP Cube 1

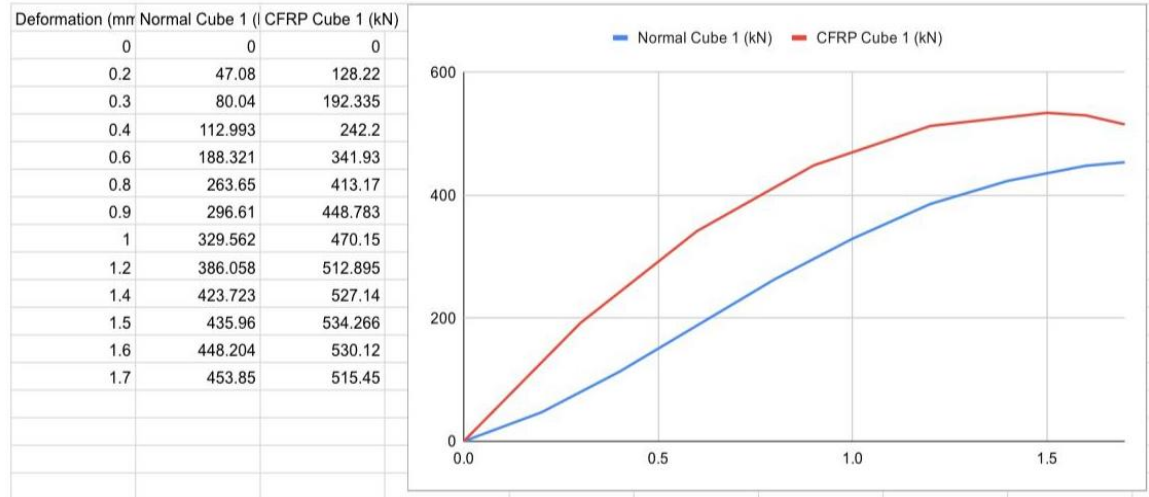


Figure 4.3: Comparison between Confined and Unconfined Cubes

Discussion and Failure Analysis in Cubes:

The empirical results confirm the baseline strength of the M20 mix perfectly with the unconfined normal cube showing the maximum compressive strength of 20.51N/mm² and 20.93N/mm² respectively. Failure of these normal specimens showed typical shear cone failure with explosive spalling of the concrete cover and sudden loss of load-carrying capacity at failure. Immediately, the change in the stress-strain curve was noticed with the introduction of 200 GSM CFRP jacket. The ultimate load of the jacket for Wrapped Cube 2 was an incredible 589.445 kN or 21.56 N/mm², which was a reflection of its ability to withstand the lateral Poisson expansion. But as foretold by Tastani and Pantazopoulou, the confinement efficiency of a square cube is not optimum. The failure mode of the CFRP cubes was generally dominated by the stress concentrations at the corners in which the carbon fibres failed locally, and the concrete core abruptly ruptured with a loud sound. At the stage of low expansion of concrete, the confinement was low, and in this stage, the

increase in strength was slightly lower than the strength of increment that the theoretical circular confinement would obtain.

4.3 Compressive Strength of Concrete Cylinder (150x150 mm)

Standard cylinders (with a ratio of length to diameter of 2.0) are used throughout the world to determine the true uniaxial compressive strength of concrete because they overcome the end-friction confinement effect observed in standard cube tests.



Figure 4.4: Concrete cube under compressive strength testing



Figure 4.5: CFRP wrapped concrete cylinder under compressive strength testing

Table 4.3: Load vs Deformation table for Confined Cylinder

Deformation (mm)	Normal Cyl 1 (kN)	Normal Cyl 2 (kN)	Normal Cyl 3 (kN)
0	0	0	0
0.2	42.497	39.956	54.736
0.4	80.521	75.707	103.711
0.6	114.072	107.251	146.923
0.8	143.149	134.59	184.374
1	167.753	157.723	216.064
1.2	187.883	176.65	241.991
1.4	203.539	191.37	262.157
1.6	214.723	201.885	276.562
1.8	221.433	208.194	285.204
2	223.67	210.297	288.085
2.1	222.552	209.246	286.645
2.3	219.197	206.091	282.323
2.5	212.487	199.782	273.681
2.8	201.303	189.267	259.277

Table 4.4: Load vs Deformation table for Unconfined Cylinder

Deformation (mm)	CFRP Cyl 1 (kN)	CFRP Cyl 2 (kN)	CFRP Cyl 3 (kN)
0	0	0	0
0.2	27.615	52.333	32.965
0.4	67.656	128.216	80.765
0.6	112.667	213.518	134.498
0.8	157.679	298.821	188.232
1	197.168	373.657	235.372
1.2	231.133	437.926	275.92
1.4	253.501	480.416	302.621
1.6	268.137	508.152	320.093
1.8	274.764	520.712	328.005
2	276.145	523.329	329.653
2.1	275.593	522.282	328.994
2.3	272.831	517.049	325.697
2.5	268.137	508.152	320.093
2.8	260.405	493.499	310.863

Comparison between Confined and Unconfined Cylinder

Normal Cylinder Peak Loads:

- Cylinder 1: ~223.670 kN
- Cylinder 2: ~210.297 kN
- Cylinder 3: ~288.085 kN

(Average Peak: ~248 kN. Cylinder 1 is closest to the true average).

CFRP Cylinder Peak Loads:

- Cylinder 1: ~ 276.145 kN
- Cylinder 2: ~ 523.329 kN
- Cylinder 3: ~ 329.65 kN

(Average Peak: ~376.376 kN. Cylinder 3 is closest to the true average).

So , we plot **Normal Cylinder 1 vs. CFRP Cylinder 3.**

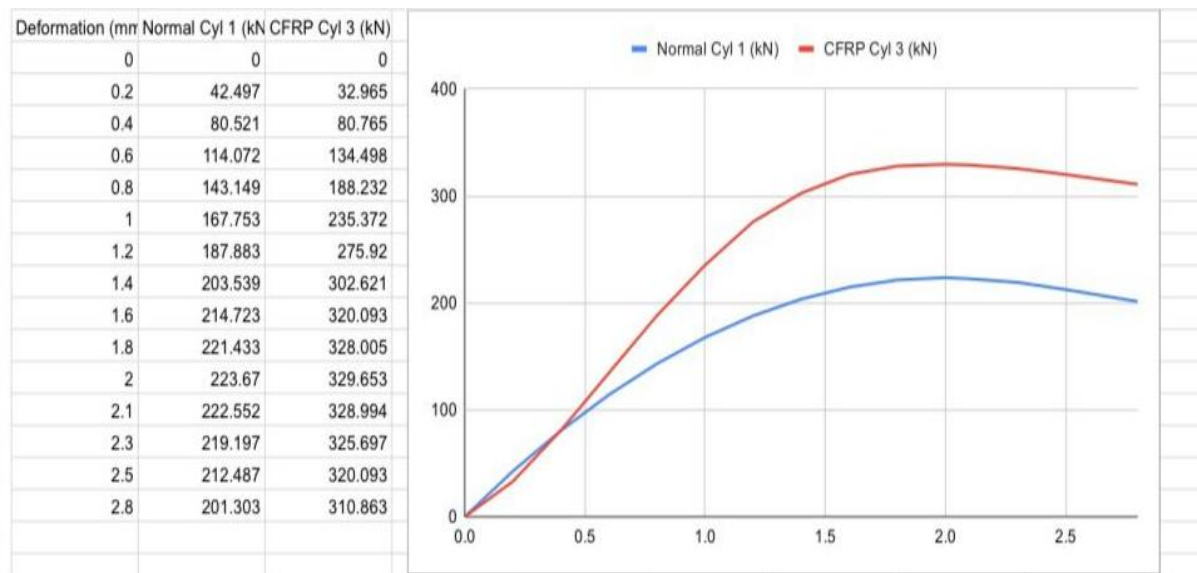


Figure 4.6: Comparison between Confined and Unconfined Cylinder

Discussion and Failure Analysis in Cylinders:

The most amazing results from this experimental program came from the unconfined normal cylinders, which had a typical true compressive strength ranging from 11.90 to 16.31 N/mm², breaking through vertical splitting cracks that ran entirely across the specimen. The ultimate capacity has been leveled to a new dimension after application of the 4-layer 200 GSM CFRP jacket. The load that was applied to Wrapped Cylinder 2 was 523.329kN corresponding to a compressive strength of 29.63 N/mm². This is an extraordinary percent gain of almost 150% on the unconfined cylinders used as baseline. The CFRP could engage as soon as the onset of the dilation of the concrete occurred due to the geometry of the cylinder. Despite that the concrete core was never crushed completely; the uniform radial confining pressure preserved the integrity of the concrete core up to an unconfined crushing strain exceeding that of the concrete core. The failure mode came from crushing of the concrete to eventual rupturing of the CFRP jacket in tension. The intense sound of FRP rupture marked the tremendous energy released from the jacketed core under triaxial pressures, which was an indicator of its safety.

4.4 Flexural Strength of Concrete Beams (100x100x500 mm)

Flexural testing presents an entirely different stress scenario -- extreme compression on the top fiber, and extreme tension on the bottom fiber. In these tests, the unidirectional CFRP was used to provide external tensile reinforcement, with its sole application being to the tension soffit of the beam.

Table 4.5: Load VS Deformation table for Confined Beams

Deformation (mm)	Normal Beam 1 (kN)	Normal Beam 2 (kN)	Normal Beam 3 (kN)
0	0	0	0
0.2	3.367	2.212	2.718
0.4	6.38	4.191	5.15
0.6	9.039	5.937	7.296
0.8	11.343	7.451	9.156
1	13.292	8.732	10.73
1.2	14.887	9.779	12.017
1.4	16.128	10.594	13.018
1.6	17.014	11.176	13.734
1.8	17.546	11.526	14.163
2	17.723	11.642	14.306

2.1	17.634	11.584	14.234
2.3	17.369	11.409	14.02
2.5	16.837	11.06	13.591
2.8	15.951	10.478	12.875

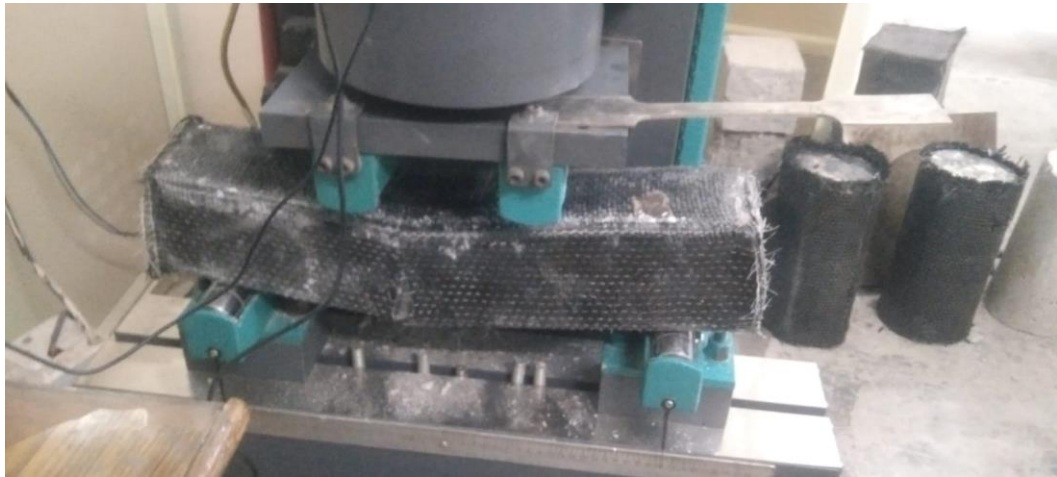


Figure 4.7: CFRP wrapped concrete beam under flexural load

Table 4.6: Load vs Deformation table for Unconfined Beams

Deformation (mm)	CFRP Beam 1 Load (kN)	CFRP Beam 2 Load (kN)	CFRP Beam 3 Load (kN)
0	0	0	0
0.2	2.87	2.997	3.086
0.4	5.437	5.679	5.846
0.6	7.703	8.045	8.282
0.8	9.667	10.095	10.394
1	11.328	11.831	12.18
1.2	12.687	13.25	13.642
1.4	13.745	14.354	14.778
1.6	14.499	15.143	15.59
1.8	14.953	15.616	16.078
2	15.104	15.774	16.24
2.1	15.028	15.695	16.159
2.3	14.802	15.459	15.915
2.5	14.349	14.985	15.428
2.8	13.594	14.197	14.616

Comparison between Confined and Unconfined Beam

Normal Beam Peak Loads:

- Beam 1: ~ 17.723 kN
- Beam 2: ~ 11.642 kN
- Beam 3: ~ 14.306 kN

(Average Peak: ~14.557 kN. Beam 3 is closest to the true average).

CFRP Beam Peak Loads:

- Beam 1: ~ 15.104 kN
- Beam 2: ~ 15.774 kN
- Beam 3: ~ 16.240 kN

(Average Peak: ~15.706 kN. Beam 2 is closest to the true average).

So, we plot **Normal Beam 3 vs. CFRP Beam 2.**

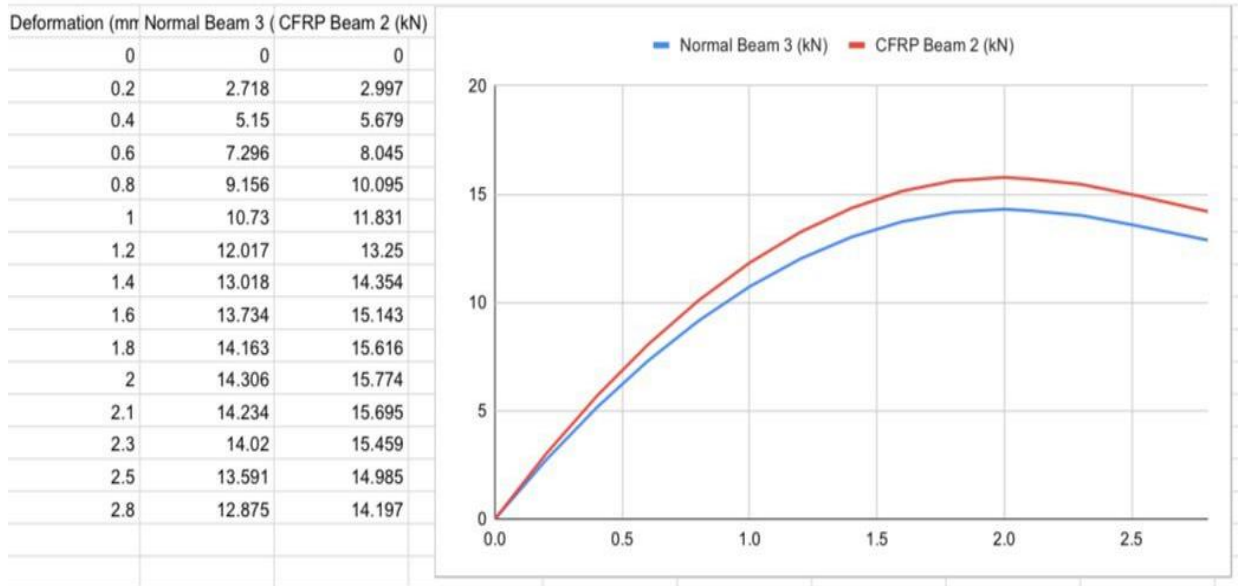


Figure 4.8: Comparison between Confined and Unconfined Beam

Discussion and Failure Analysis in Flexural Beams:

The normal beams failed in a very brittle way. As soon as the tensile stress at the mid-span soffit exceeded the modulus of rupture of the unreinforced M20 concrete (around 4.65 to 7.08 N/mm²), a single primary crack propagated instantaneously upwards towards the neutral axis, resulting in a sudden collapse of the element. For the CFRP retrofitted beams, the story was quite different. The peak loads stabilized with a high degree of consistency between 15.104 kN and 16.240 kN. The real strength of the external composite reinforcement was noticed in the post-yield deformation range. The carbon fibers spanned the micro-cracks that developed at the concrete soffit preventing the micro-cracks from opening wide. Instead of one catastrophic central crack, the increasing load caused multiple smaller cracks along the span. The failure mode of the retrofitted beams was not concrete crushing, but was governed by interlaminar shear failure, i.e. debonding or peeling off of the CFRP laminate from the concrete substrate, as soon as the interfacial shear stresses exceed the bond strength of the epoxy. The beams exhibited significant mid-

span deflection before debonding, avoiding sudden, unheralded brittle failure and satisfying the structural requirement of ductility under displacement.

Concrete Cube Test (Normal)

DOC: 11/12/2025

DOT: 22/05/2026

S.No	Area (mm ²)	Load (kN)	Compressive Strength (N/mm ²)
1	150 × 150 × 150	461.387	20.51
2	150 × 150 × 150	470.935	20.93
3	150 × 150 × 150	455.608	20.34

Average Strength = 20.59 N/mm²

Compressive Strength of Concrete Cube with CFRC

S.No	Area (mm ²)	Load (kN)	Compressive Strength (N/mm ²)
1	150 × 150	534.266	21.359
2	150 × 150	589.445	21.558
3	150 × 150	460.416	20.46

Average Strength = 21.13 N/mm²

Concrete Beam Test (Flexural Strength) – Normal Sample

S.No	Span Length (mm)	Width (mm)	Thickness (mm)	Load (kN)	Flexural Strength (N/mm ²)
1	400	100	100	17.723	7.08
2	400	100	100	11.642	4.65
3	400	100	100	14.306	5.72

Average Strength = 5.82 N/mm²

Concrete Beam Test with CFRC

S.No	Span Length (mm)	Width (mm)	Thickness (mm)	Load (kN)	Flexural Strength (N/mm ²)
1	400	100	100	15.104	5.89
2	400	100	100	15.774	6.13
3	400	100	100	16.240	6.33

Average Strength = 6.12 N/mm²

Concrete Cylinder Test (Normal)

S.No	Area (mm ²)	Load (kN)	Compressive Strength (N/mm ²)
1	17662.5	223.677	12.66
2	17662.5	210.297	11.90
3	17662.5	288.085	16.31

Average Strength = 13.62 N/mm²

Concrete Cylinder Test with CFRC

S.No	Area (mm ²)	Load (kN)	Compressive Strength (N/mm ²)
1	17662.5	276.145	15.62
2	17662.5	523.329	29.63
3	17662.5	329.653	18.66

Average Strength = 21.30 N/mm²

Figure 4.9: Total mean strength of all the specimens

CHAPTER 5

CONCLUSION AND FUTURE SCOPE

5.1 Concluding Remarks

The extensive experimental program presented in this dissertation has clearly demonstrated the tremendous effectiveness of Unidirectional CFRP jackets for the structural retrofitting of M20 grade concrete elements. By carefully uncoupling the variables on 18 different test specimens, including both direct compression and flexural dynamics, the following important conclusions have been reached:

- a) **Theoretical Validation:** The displacement-ductility-based design methodology of Pantelides et al. accurately predicts the mechanics of FRP confinement. The mathematical derivation using K_{je} , H_c and K_e correctly dictated the requirement of 4 layers of 200 GSM CFRP, which was found to be completely adequate to arrest unconfined lateral dilation of the concrete core.
- b) **Shape efficiency for compression:** The geometric shape of the structural member is the dominant factor in the effectiveness of the FRP confinement. The cylindrical specimens showed significant strength increases with an ultimate capacity of 29.63 N/mm² (~150% increase over the unconfined baseline). The circular geometry generated a perfectly uniform hoop tensile stress field within the CFRP jacket, allowing virtually the entire cross-section to participate in the triaxial load-carrying mechanism. This geometry-driven efficiency explains why cylindrical specimens consistently outperform square or rectangular sections of comparable cross-sectional area and FRP volume fraction. Conversely, the square cubes experienced large stress concentrations at the four corners, where the CFRP fibers were forced to bend abruptly, and the confinement effect was largely limited to the parabolic arching zones between corners, leaving much of the interior relatively unconfined. Despite this

geometric disadvantage, the CFRP-wrapped cubes still recorded a meaningful average strength gain compared to unconfined controls, confirming that even sub-optimal geometries benefit from external FRP retrofitting.

- c) **Flexural ductility:** In bending, the externally bonded unidirectional CFRP applied to the tension soffit of the beam acts as an external tensile reinforcement, supplementing or substituting the role of internal steel reinforcement. The final peak load increase was relatively modest and consistent across all three wrapped specimens (mean ~ 15.7 kN compared to the unconfined mean of ~ 14.6 kN), reflecting the fact that the unreinforced concrete beams lacked internal steel to yield and redistribute loads and thus provided a lower baseline against which to measure improvement. However, the most significant and structurally valuable improvement was the fundamental change in failure mechanism: from highly brittle, instantaneous mid-span cracking to a prolonged, multi-crack ductile deformation sequence. The CFRP-retrofitted beams exhibited progressively widening multiple distributed flexural cracks along the span, with considerable mid-span deflection before the final interfacial debonding event, demonstrating that the composite system successfully transformed the structural behavior from catastrophically brittle to deformation-tolerant and warning-giving. This behavioral transformation is of paramount practical importance in structural safety engineering, as ductile failure modes allow occupants sufficient time to evacuate and enable engineers to intervene before collapse.
- d) **Material Synergy:** The combination of a high-performance two-part structural epoxy adhesive and 200 GSM Unidirectional Carbon Fiber creates a composite matrix that is simultaneously structurally very strong, phenomenally lightweight, chemically inert, and highly practical for rapid on-site retrofitting of deteriorating concrete structures. The dry CFRP fabric can be cut to precise dimensions using standard utility tools, and the epoxy system used in this study exhibited a sufficiently long pot life to allow careful manual saturation and wrapping without special pressurization

equipment. The total applied CFRP jacket thickness of approximately 0.4 mm per layer, resulting in a total jacket of approximately 1.6 mm for four layers, is so geometrically insignificant that it does not alter the effective cover dimensions or the clear span geometry of a structural member in any meaningful way. This combination of properties — high tensile strength, minimal weight addition, corrosion resistance, ease of application, and geometric non-intrusiveness — collectively makes the CFRP-epoxy retrofitting system uniquely positioned for large-scale deployment in the rehabilitation of aging Indian reinforced concrete infrastructure.

5.2 Future Scope of Work

This study has exhausted the topic of static monotonic loading on small-scale specimens, but there are many directions in which this line of research can be expanded in the future:

- a) **Cyclic and Seismic Loading:** Application of cyclic reverse-loading to full-scale, CFRP-retrofitted column-beam joints to simulate earthquake forces.
- b) **Environmental Degradation:** The long-term bond integrity of epoxy after exposure to accelerated aging, high UV radiation and freeze thaw cycles.
- c) **Active Confinement:** Investigating the prestressing of the CFRP wraps prior to epoxy curing to provide 'active' confining pressure rather than the 'passive' pressure studied herein.
- d) **Anchorage Systems:** Developing advanced mechanical end-anchorage systems (such as FRP spike anchors) to categorically eliminate the premature debonding failures observed during flexural beam testing.

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