

Experimental evaluation of maximum debonding strain in thick CFRP layers for flexural strengthening

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Abstract

Carbon fiber reinforced polymer (CFRP) is widely used for strengthening RC structures; however, the externally bonded reinforcement (EBR) method is often governed by premature debonding at the CFRP-concrete interface, preventing CFRP from reaching its full tensile capacity. Current design provisions limit the effective strain to control debonding, but most are based on test results conducted using thin CFRP layers, leaving uncertainties in their applicability to thick CFRP systems.

This study investigated the maximum debonding strain, load-carrying capacity, and failure behavior of RC T-beams strengthened with multiple layers of thick CFRP sheets (V-Wrap C400HM) using the epoxy resin V-Wrap 770. The experimental program included ten RC T-beams, which were divided into two groups: low-strength concrete (2.86 ksi) and normal strength concrete (4.44 ksi). Each group included a control beam, three beams strengthened with one to three thick CFRP layers, and one hybrid beam using an externally bonded reinforcement on grooves (EBROG) technique, where the soffit was pre-grooved and filled with epoxy putty before applying one thick CFRP layer. All specimen were tested under four-point bending, using displacement – controlled loading at a constant rate of 0.05 in/min.

Experimental results showed significant strength enhancement with CFRP, with the peak performance observed in the two-layer systems, which provided the best strength – ductility balance. Specimens with three CFRP layers exhibited premature debonding or cover delamination, indicating a practical limit for effective CFRP thickness without additional mechanical anchorage. The hybrid grooved-putty configuration improved bond performance when applied in the normal strength concrete series compared to a single-layer externally bonded

system. Among evaluated design guidelines, ACI 440.2R-23 provided most reliable prediction as compared to TR55, CNR-DT 200, fib-14, and JSCE.

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Dedication

To my father and mother

Thank you for your unconditional love, your sacrifices, and your faith in me.

You gave me the opportunity to study abroad and chase a future that once felt far away.

This achievement is as much yours as it is mine. I love you both more than words can say.

Chapter 1 – Introduction

1.1 General

Reinforced concrete (RC) structures are extensively used in buildings, bridges, and infrastructure due to their strength, durability, and cost efficiency. However, many existing RC members experience deterioration or insufficient capacity over time because of aging, reinforcement corrosion, increased service loads, changes in design standards, or construction deficiencies. Therefore, strengthening and rehabilitation are essential to restore or enhance their structural performance.

Fiber-reinforced polymer (FRP) composites have become a widely accepted solution for strengthening RC structures. Their advantages include high tensile strength-to-weight ratio, resistance to corrosion, and ease of installation. Among the available FRP types, carbon fiber-reinforced polymer (CFRP) is commonly used for flexural strengthening of RC beams by bonding CFRP laminates or sheets to the tensile face using epoxy adhesives.

Despite its effectiveness, the externally bonded reinforcement (EBR) technique is often limited by premature debonding at the CFRP – concrete interface before the CFRP reaches its ultimate strength. As a result, the actual strain developed in CFRP at failure is significantly below its tensile capacity. Existing design guidelines attempt to limit this effective strain to prevent debonding, but most provisions are derived from tests using thin CFRP layers, making their applicability uncertain for thick CFRP applications involving multiple layers.

To address this gap, this experimental program investigates the maximum debonding strain developed in thick CFRP layers bonded to RC T-beams. A total of ten beams were tested, which were divided into two series, including low-strength and normal-strength concrete. Each

series consisted of one un-strengthened control beam and three beams strengthened with one, two, and three layers of externally bonded CFRP sheets, respectively. Additionally, one hybrid specimen was included in each concrete strength series to examine the influence of a modified bond interface on CFRP performance. In this configuration, two longitudinal NSM grooves along the soffit and filled with epoxy putty prior to installing a single CFRP layer (EBROG). This specimen was designed to determine whether surface modification alone can improve bond behavior and increase strength compared to the conventional single-layer CFRP strengthening configuration. All beams were tested under four-point bending with displacement-controlled loading to characterize flexural capacity, load-deflection behavior, strain distribution, and failure mechanisms, with particular emphasis on CFRP debonding strain behavior. The experimental results were complemented by numerical analysis, and the maximum CFRP debonding strain was compared against prediction models from various design guidelines to evaluate their conservativeness for thick CFRP strengthening systems.

1.2 Problem Statement

Premature debonding remains the most critical limitation in the application of externally bonded CFRP for flexural strengthening. Existing design guidelines provide formulations to limit the effective FRP strain to prevent debonding. However, these expressions are primarily calibrated using data from specimens strengthened with a single or thin CFRP layer. When two or more layers are used to enhance the reinforcement ratio and improve flexural capacity, the bond stress distribution along the CFRP–concrete interface changes, potentially altering the effective strain that can be developed before debonding occurs.

There is currently limited experimental evidence quantifying the relationship between CFRP thickness and maximum debonding strain, particularly for thick CFRP laminations bonded

to RC T-beams. Furthermore, the applicability of existing strain-limit provisions to thick CFRP layers remains uncertain. As a result, engineers may use conservative strain limits in design, which may underestimate the strengthening contribution of CFRP and lead to uneconomical or overdesigned systems. Conversely, overestimating the debonding strain may result in premature FRP failure and threaten structural safety. Therefore, there is a need for a systematic experimental investigation to characterize the debonding strain behavior of thick CFRP layers in flexural strengthening applications.

1.3 Research Objectives

The primary objective of this study is to evaluate the maximum debonding strain developed in thick CFRP layers bonded to RC T-beams under flexural loading. The specific objectives are:

1. To experimentally investigate the flexural behavior of RC T-beams strengthened with thick high-modulus CFRP sheet (V-Wrap C400HM) using one, two, and three bonded layers.
2. To quantify the maximum CFRP debonding strain and evaluate the influence of CFRP layer thickness and concrete compressive strength (low vs. normal strength series).
3. To evaluate the strengthening effectiveness of a grooved–putty hybrid configuration by comparing its flexural performance to that of a single-layer CFRP-strengthened beam.
4. To compare experimental results with numerical analysis predictions to validate strain development and load response.
5. To evaluate the applicability of existing debonding strain limits for thick CFRP applications.

1.4 Scope of Study

This study focuses on the flexural strengthening of RC T-beams using externally bonded unidirectional CFRP sheets applied to the tension face. The experimental program includes ten full-scale beams divided into two concrete-strength series: low-strength (approximately 3 ksi) and normal-strength (approximately 4.5 ksi). Each series consists of one control beam without strengthening, three beams strengthened with one, two, and three CFRP layers, and One hybrid beam, in which two longitudinal NSM grooves were cut along the soffit and fully filled with epoxy putty prior to the installation of a single externally bonded CFRP layer, creating a modified bond interface. All beams were tested under four-point bending with displacement-controlled loading and loaded until FRP failure to examine load–deflection behavior, strain distribution, and debonding failure. Numerical analysis is conducted in parallel to simulate flexural behavior and predict failure load and debonding strain for comparison with experimental findings.

1.5 Thesis Organization

This thesis consists of five chapters. Chapter 1 provides an overview of the research background, problem statement, objectives, and scope. Chapter 2 presents a comprehensive review of previous research on FRP strengthening techniques, bond behavior, debonding mechanisms, and strain-limit provisions. Chapter 3 describes the experimental program, including specimen design, material properties, strengthening configurations, and instrumentation setup. Chapter 4 presents and discusses the experimental results alongside numerical analysis and code comparisons. Finally, Chapter 5 summarizes the key conclusions of the study and provides recommendations for future research.

Chapter 2 – Literature Review

2.1 Background

Carbon Fiber Reinforced Polymer (CFRP) is a composite material that has gained widespread use in structural engineering, particularly for the strengthening and rehabilitation of reinforced concrete (RC) structures. Composed of high-strength carbon fibers embedded in a polymer matrix, typically epoxy, CFRP offers exceptional mechanical properties such as high tensile strength-to-weight ratio, corrosion resistance, and ease of installation. These advantages make CFRP an attractive alternative to traditional strengthening methods like steel plate bonding or section enlargement. Since the 1990s, CFRP has been increasingly applied in externally bonded reinforcement (EBR) systems for flexural and shear strengthening of beams, columns, slabs, and bridge elements. Its use is especially valuable in repair scenarios where minimal disruption, rapid application, and long-term durability are required. However, issues such as premature debonding between CFRP and concrete, long-term environmental effects, and variability in bond performance continue to be active areas of research, driving the development of improved materials, bonding techniques, and design guidelines.

In typical flexural strengthening application, the stresses are transferred to the CFRP through the bond between the two materials. Thus, the bond becomes an important factor that controls the level of strengthening achieved. In applications involving wrapping of columns, tensile stresses are produced in the CFRP by concrete bearing pressure against the FRP where the bond has relatively little importance. The CFRP provides confinement to the concrete, which increases the load carrying capacity and the ductility. For use of CFRP in bond sensitive

applications such as in flexural or shear strengthening of beams, the behavior of the bond between FRP and concrete must be well understood.

Today, CFRP continues to evolve, driven by advancements in material science and engineering design, offering sustainable alternatives to traditional construction materials and extending the lifespan of critical infrastructure.

2.2 Review Previous Studies on FRP Flexural Debonding

Teng, Smith, Yao, & Chen (2003) investigated intermediate crack (IC) debonding, a critical failure mode in reinforced concrete members strengthened with externally bonded FRP laminates. The purpose of the study was to understand the mechanism of intermediate crack (IC) debonding and to develop a rational model for predicting its onset. By analyzing test data and extending the Chen & Teng bond strength model, the authors proposed a predictive approach that showed reasonable agreement with experimental results, providing a practical design tool for assessing IC debonding in FRP-strengthened RC beams and slabs.

Chen & Teng (2003) developed a rational design model for the shear capacity of FRP strengthened RC beams failing due to FRP debonding. They reviewed existing models, highlighting their limitations, and proposed a new model incorporating an effective FRP strain approach. This model was validated against experimental data from the literature and showed improved accuracy in predicting shear strength. The study emphasized the importance of considering debonding failure in design and provided a practical design approach for predicting shear capacity in RC beams strengthened by FRP.

Cao et al. (2005) investigated RC beams shear-strengthened with complete FRP wraps, focusing on FRP debonding as a potential design limit state. Experimental results revealed that FRP strain distribution along critical shear cracks is nonuniform and influenced by the shear

span-to-effective depth ratio. The study applied Chen and Teng's bond strength and strain distribution models to estimate FRP strains at debonding. An empirical model was proposed to predict the FRP contribution to shear strength, accounting for strain distribution and critical crack angle, and predictions showed good agreement with experimental observations, providing a conservative yet practical design approach.

Laura Anania (2005) conducted experimental research on the structural response of RC beams strengthened with externally bonded CFRP laminates, with the objective of quantifying improvements in flexural behavior relative to un-strengthened control specimens. The research specifically assessed bearing capacity, flexural stiffness, and ductility under four-point bending. Experimental results showed that CFRP strengthening increased ultimate load capacity by more than 70%, nearly doubled flexural stiffness, and produced a threefold increase in ductility. These findings confirm the effectiveness of CFRP in significantly enhancing flexural performance without contributing additional self-weight to the structure.

Teng et al. (2006) examined debonding failures in reinforced concrete beams strengthened in flexure using near-surface mounted (NSM) CFRP strips. Through a comprehensive experimental program, they identified two dominant failure mechanisms: CFRP – concrete interface debonding and concrete cover separation. The study demonstrated that premature debonding is strongly influenced by bonded length, groove detailing, and concrete cover thickness. Results showed that adequate embedment length and improved anchorage detailing significantly delay debonding and increase load-carrying capacity. This work provided fundamental insights into bond mechanics for NSM strengthening and laid the groundwork for bond-based design provisions in later NSM design guidelines.

Ali-Ahmad et al. (2006) conducted an experimental study to investigate the debonding behavior between concrete and fiber-reinforced polymer (FRP) sheets. The purpose of the paper was to understand the fracture mechanics at the concrete - FRP interface and to characterize the interfacial shear response during debonding. Using detailed experimental measurements, including displacement and strain analysis, the authors developed a nonlinear cohesive law for the interface. The study concluded that debonding initiates at the interface and propagates progressively, with the fracture energy quantified, providing valuable insights for the design and performance prediction of FRP-strengthened concrete structures.

Leung (2006) investigated the mechanisms of FRP debonding from concrete substrates, challenging conventional assumptions that debonding initiates primarily at FRP plate ends. The study demonstrated that intermediate cracks often govern debonding initiation, and that end U-shaped FRP anchors may have limited effectiveness in preventing failure. These findings emphasized the importance of accurately characterizing stress distribution and crack development in FRP-strengthened RC members. The experiment highlighted the need to reconsider current design approaches and anchorage strategies to ensure reliable performance against debonding in externally bonded FRP systems.

Elsayed et al. (2007) investigated the interfacial behavior and debonding failures in FRP – strengthened reinforced concrete two-way slabs. The study employed finite-element analyses to simulate nonlinear load–deflection behavior and failure mechanisms, modeling the FRP - concrete interface with bond-slip relationships. The authors concluded that accurately representing the interface is essential for predicting ultimate capacities and failure modes. Their analyses demonstrated that increasing FRP width reduces interfacial stresses, while overlapping

laminates affect slips, highlighting the importance of proper bonding and anchorage in optimizing FRP strengthening configurations.

Yao and Teng (2007) conducted extensive four-point and three-point bending tests on simply-supported RC beams bonded with thin FRP or steel plates to investigate plate end debonding, particularly concrete cover separation failures. Results showed that flexural debonding is governed by interfacial stresses between the steel reinforcement and the “composite plate” of bonded FRP and concrete cover, with stiffer plates and thicker covers reducing debonding load. In shear-dominated cases, failure modes included cover separation, intermediate-crack-induced debonding, and critical diagonal crack debonding. The study highlighted that interfacial stress distribution and plate geometry critically control debonding behavior.

Iwashita et al. (2007) investigated the bonding and debonding behavior of FRP sheets on concrete and steel substrates under static and cyclic (fatigue) loading. The study identified multiple debonding modes and evaluated load-carrying capacity, interfacial fracture energy, and bond strength. Results indicated that specimens with higher aggregate shear strength exhibited greater interfacial fracture energy and higher debonding resistance. Fatigue loading reduced the FRP strain at debonding initiation compared to static tests, and effective bond length increased progressively before failure. The study emphasizes surface preparation and highlights the influence of cyclic loading on FRP-concrete interface performance.

Nigro et al. (2008) conducted an experimental study to examine the debonding behavior of FRP-concrete interfaces under cyclic loading. Single shear and double shear specimens were subjected to repeated loading to simulate service conditions and seismic actions. The results showed progressive degradation of bond capacity due to cyclic slip accumulation at the interface.

Compared to monotonic loading, cyclic actions significantly reduced the ultimate debonding load and stiffness, especially in specimens with shorter bonded lengths. The study emphasized the importance of considering fatigue-induced debonding mechanisms in FRP-strengthened members subjected to seismic or repeated service loading.

Oehlers et al. (2008) proposed an enhanced near-surface mounted (NSM) strengthening technique aimed at improving resistance to intermediate crack (IC) debonding in reinforced concrete beams. The study introduced a modified NSM configuration in which FRP plates were embedded deeper within the concrete cover to increase mechanical interlock and confine interfacial shear stresses. Experimental beam tests showed that deeper embedment delayed the onset of IC debonding and allowed greater mobilization of FRP tensile capacity before failure. The study concluded that embedment depth and anchorage detailing are critical parameters for mitigating premature debonding failures in NSM strengthening systems.

Lijuan Li (2008) and his co-workers examined the flexural performance of plain and fiber-reinforced concrete beams strengthened with CFRP, GFRP, and hybrid CFRP–GFRP sheets. The purpose was to assess the combined effects of polypropylene fibers (PFs), steel fibers (SFs), and FRP strengthening on crack resistance, stiffness, and load capacity. Results indicated that PFs improved cracking resistance, SFs enhanced strength and crack control, and PF–SF hybrids further improved stiffness and ductility. CFRP sheets provided superior strength and stiffness gains, GFRP sheets enhanced ductility, and combined CFRP–GFRP systems offered balanced improvements in strength, stiffness, and toughness.

Aram, Czaderski and Motavalli (2008) investigated debonding failure modes in flexurally FRP-strengthened reinforced concrete (RC) beams through experimental and analytical approaches. Their study identified several debonding mechanisms, including

intermediate crack (IC) debonding, plate-end debonding, and concrete cover separation. The study found that existing codes and guidelines produce widely varying predictions for midspan debonding, plate-end debonding, and concrete cover delamination, with some unconservative and others overly conservative. The authors recommended a combined FRP strain and bond shear stress limitation to improve prediction accuracy. Comprehensive bond models, like that of Neubauer and Rostasy, provided the best correlation with experimental data but are computationally complex, highlighting the need for practical and reliable design methods.

Pan et al. (2010) systematically investigated FRP debonding in RC beams under the presence of either a major crack or multiple secondary cracks. The study identified limitations in existing models and introduced an analytical model incorporating delayed softening due to secondary crack openings. Experimental validation on eight beams showed that multiple cracks increased the maximum FRP strain by about one-third compared to single-crack cases. The proposed model closely predicted strain and stress distributions, demonstrating strong potential for practical application in evaluating debonding failure.

Mostofinejad and Mahmoudabadi (2010) experimentally evaluated grooving as an alternative surface preparation technique to delay debonding of externally bonded FRP laminates in RC beams. Instead of conventional grinding, shallow grooves were introduced into the concrete surface prior to FRP installation to improve mechanical interlocking at the interface. Four-point bending tests demonstrated that grooving significantly increased debonding load and allowed greater utilization of FRP tensile capacity. The method also reduced premature peeling failures by enhancing interfacial shear transfer. Their findings highlight surface preparation as a critical parameter for improving bond performance in EB-FRP strengthening systems.

Chen et al. (2011) developed a nonlinear finite-element model to simulate intermediate crack (IC) debonding in FRP-plated reinforced concrete beams, a prevailing failure mode in flexural strengthening applications. The study incorporated nonlinear concrete behavior, bond–slip effects at the FRP – concrete interface, and tension stiffening to accurately capture the initiation and propagation of IC debonding. The numerical results demonstrated strong agreement with experimental data and highlighted the significance of interfacial fracture energy and shear stress transfer in predicting debonding failures. Their work provides a reliable analytical framework for understanding IC debonding mechanisms and improving FRP strengthening design.

Bilotta et al. (2011) conducted an experimental study to calibrate a capacity model for FRP-to-concrete interface debonding. The authors performed 34 single-shear tests and compiled an extensive database of 139 tests to analyze debonding behavior in sheets and plates. Their study showed that existing design codes tend to underestimate debonding loads for sheets and overestimate them for plates. A new calibration of the coefficient k_G was proposed, improving the prediction of debonding loads and enhancing the efficiency and safety of externally bonded FRP systems.

Kim and Shin (2011) investigated the flexural strengthening of RC beams using hybrid FRP sheets with different layering orders, including both preloaded and non-preloaded conditions. The study aimed to evaluate how hybrid FRP configuration and preloading influence strength, stiffness, and ductility under four-point bending. Results showed that hybrid FRPs effectively enhanced ultimate load and stiffness, with the best performance achieved when glass fiber was applied before carbon fiber. Strengthening was more effective in non-preloaded beams,

though epoxy crack repair mitigated preloading effects. Failure occurred before FRP rupture, indicating the need for improved strengthening design.

Wu et al. (2011) proposed an improved hybrid bonded FRP (IHB-FRP) technique to prevent debonding in externally bonded FRP (EB-FRP) systems. The method incorporates mechanical anchors to enhance interfacial bond strength. Experimental results demonstrated that the IHB-FRP system effectively prevented debonding, with the strengthened beams failing by FRP rupture even after seven plies of FRP strips. This approach significantly improved the shear capacity of the beams compared to traditional EB-FRP systems. The study provides a practical solution for enhancing the performance and reliability of FRP-strengthened concrete structures.

Oller and Mari (2013) investigated the long-term bond performance and debonding behavior of cracked reinforced concrete members strengthened with externally bonded FRP sheets. The study considered sustained loading conditions to simulate service environments and examined time-dependent effects such as creep in concrete and viscoelastic deformation in the adhesive layer. Results showed that sustained loading leads to progressive redistribution of bond stresses and gradual degradation of the FRP–concrete interface, which may trigger delayed debonding even at service load levels. The authors emphasized the importance of accounting for long-term bond-slip behavior in design and highlighted the need for durability-based bond models for FRP-strengthened structures.

Dong et al. (2013) researched on the flexural performance of RC beams strengthened with different layers of CFRP sheets, including both preloaded and non-preloaded specimens, under four-point bending. The study aimed to evaluate the effects of FRP layering, preloading, longitudinal reinforcement ratio, and concrete cover on cracking, load capacity, and ductility. Results showed that two-layer CFRP strengthening increased crack load by 23%, ultimate load

by 22%, and reduced mid-span deflection by 39%. Strengthening improved overall performance relative to the control beam, while preloading reduced both ultimate flexural strength and ductility, and thicker concrete covers did not enhance ultimate load.

Choi (2013) experimented the flexural strengthening of small RC beams with a half-depth saw cut at mid-span using different layers of CFRP or GFRP sheets under three-point bending. The study aimed to evaluate the influence of FRP type, thickness, layering order, and resin on debonding behavior. Results indicated that stiffer and thinner CFRP layers achieved higher debonding strengths than thicker, less stiff GFRP layers. Layering order was critical, with higher strengths observed when CFRP was applied before GFRP. Debonding was also significantly affected by the resin type, highlighting its role in the performance of hybrid FRP systems.

Hawileh et al. (2014) examined the flexural behavior of reinforced concrete beams strengthened with externally bonded hybrid FRP systems that combined carbon and glass fibers. The objective was to improve ductility and delay debonding while maintaining adequate strength enhancement. Experimental results showed that hybrid FRP configurations reduced the brittleness typically associated with carbon FRP systems by enabling a more gradual post-yield response. However, intermediate crack debonding remained the dominant failure mode, particularly in beams with insufficient bonded length or without proper anchorage. The study concluded that hybridization can enhance energy absorption capacity but does not eliminate the need for bond-dependent design considerations in FRP strengthening.

Yu et al. (2017) developed a three-dimensional finite element model to investigate debonding behavior at FRP-to-concrete interfaces under mixed-mode loading. The model utilized a bilinear cohesive zone approach to simulate fracture mechanisms at the interface and

incorporated traction–separation laws calibrated from single-lap shear and peeling tests. The simulation successfully reproduced progressive crack propagation and interfacial softening, demonstrating strong agreement with experimental results. The study highlighted the significant influence of mixed-mode fracture energy and interfacial stiffness on debonding capacity, offering improved numerical tools for predicting bond failure in FRP-strengthened RC structures.

Mostofinejad & Hosseini (2017) present a finite-element study to simulate the debonding of externally bonded FRP sheets from RC beams. Using a smeared-crack approach and cohesive elements in ABAQUS, they modelled four beam cases drawn from literature and validated against experiments. Their procedure captures FRP-concrete interface separation and tracks failure progression. Parametric analyses showed that increasing FRP sheet length and width enhances load capacity and mid-span displacement before debonding onset. The proposed modelling approach offers a practical tool for predicting debonding behaviour in FRP-strengthened RC beams.

Mohammadi et al. (2017) conducted both experimental and numerical investigations into the bond behaviour of FRP–concrete interfaces in the presence of intermediate-crack (IC) debonding. Their experimental setup used simple notched RC-beam specimens to induce IC debonding, while the numerical model combined a concrete damaged plasticity framework with an XFEM-based cohesive interface model to simulate the FRP – concrete interaction. They found that IC debonding initiates at the tip of a flexure-shear or diagonal crack and then propagates under shear stress at the interface, regardless of the moment – shear ratio at the crack location. The study advances understanding of bond-slip relationships under IC debonding conditions.

Chen and Cheng (2017) developed an analytical model to predict the flexural fatigue performance of RC beams strengthened with externally bonded FRP, emphasizing debonding

failure under cyclic loading. The model incorporated a trilinear bond - slip relation at the FRP-concrete interface and was validated against experimental results. They concluded that bond behavior progresses through bonded, softening, and debonding stages, and that higher concrete strength, larger rebar diameter, and greater FRP stiffness enhance fatigue life. The study offered a closed-form solution as a foundation for design-oriented applications.

Nerilli and Vairo (2018) conducted an experimental investigation to examine the debonding behavior of basalt fiber-reinforced polymer (BFRP) sheets bonded to concrete substrates. Through a series of single-lap shear tests, the study evaluated bond strength, interfacial slip mechanisms, and fracture characteristics. Results indicated that debonding initiated at the loaded end and propagated in a brittle manner along the adhesive-concrete interface. Increasing bonded length enhanced ultimate load capacity but showed diminishing returns beyond the effective bond length. The findings provide essential bond-slip parameters for modeling BFRP-concrete interfaces and advancing FRP strengthening design involving basalt composites.

Min et al. (2020) experimentally investigated the initiation and propagation of debonding along the FRP-concrete interface under fatigue loading. The study monitored strain profiles on CFRP plates and used visual observation to track debonding growth. A new predictive model was proposed, showing excellent agreement with test results, and indicated that fatigue load range and level strongly influenced debonding growth rate and bond life. Friction between the FRP and concrete also affected fatigue behavior. The research provides key insights for predicting and mitigating fatigue-induced debonding in FRP-strengthened RC members.

Li, Tan, and Fung (2020) experimentally investigated the dynamic debonding behavior of CFRP laminates bonded to concrete substrates. Using drop-weight impact tests, the study

examined the influence of loading rate on interfacial fracture and bond performance. Results showed that dynamic loading increased debonding strength and delayed crack initiation compared to quasi-static loading due to strain-rate effects in the adhesive and concrete. Despite the higher resistance, failure remained brittle and propagated along the FRP–concrete interface. The study highlights the importance of incorporating rate-dependent bond properties when designing FRP-strengthened RC structures exposed to impact or seismic loading conditions.

Zaki, Rasheed, Roukerd, and Raheem (2020) evaluated the flexural performance of reinforced concrete T-beams strengthened with externally bonded CFRP sheets and anchored using CFRP splay anchors. The study aimed to mitigate premature debonding by enhancing laminate anchorage near critical regions of stress transfer. Experimental four-point bending tests showed that the use of CFRP splay anchors significantly increased load capacity, delayed intermediate crack debonding, and enabled higher utilization of CFRP tensile strain. Anchored specimens also exhibited improved ductility and reduced interfacial slip compared to unanchored counterparts. The study confirmed that anchorage systems are effective in controlling debonding failures in CFRP-strengthened T-beams.

Mohammed A. Zaki and Hayder A. Rasheed (2021) illustrated the experimental results of debonding strain in sand-lightweight high strength RC beams strengthened with CFRP sheets under four-point bending. The study aimed to evaluate the effects of concrete type, CFRP layering, and bonding on debonding strain and structural performance. Results showed no significant difference in debonding strain between lightweight and normal-weight beams, though lightweight concrete exhibited slightly lower values. Thicker CFRP layers increased effectiveness, with four layers performing similarly to two layers. Experimental and numerical

results were in close agreement, supporting current design provisions with minor adjustments for lightweight concrete.

Al Shboul, Raheem, and Rasheed (2021) investigated the debonding behavior of all-lightweight reinforced concrete T-beams strengthened in flexure using externally bonded CFRP sheets. The study focused on the effect of lightweight aggregate concrete on interfacial bond performance and failure mechanisms. Experimental four-point bending tests revealed that intermediate crack debonding remained the predominant failure mode, similar to normal-weight concrete beams, but occurred at lower load levels due to reduced tensile strength and stiffness of the lightweight substrate. The authors also reported a shorter effective bond length and lower achievable CFRP strain at debonding. The findings highlight the need for bond-dependent design provisions specific to lightweight concrete members strengthened with CFRP

De Maio et al. (2022) investigated debonding mechanisms in FRP-strengthened RC members using a coupled cohesive/volumetric finite element modeling strategy. The approach combined a volumetric damage model for concrete with cohesive zone elements at the FRP–concrete interface to simulate both cracking and interfacial separation. Analytical results accurately reproduced common debonding failure modes, including intermediate crack (IC) debonding and end debonding. Parametric analyses revealed that interfacial fracture energy and adhesive layer thickness strongly influence debonding initiation and propagation. The proposed modeling framework provides an effective numerical tool for predicting complex debonding behavior in EB-FRP strengthening systems.

Min et al. (2023) proposed a nonlinear prediction model to simulate the progressive debonding of FRP–concrete interfaces under fatigue loading. The model integrates a cohesive zone approach with a fatigue damage evolution law to capture the degradation of interfacial bond

strength under repeated loading. Using a traction–separation relationship calibrated from experimental data, the model successfully reproduced stiffness reduction, slip accumulation, and fatigue-induced debonding propagation. Parametric analysis revealed that fatigue life is highly sensitive to interfacial fracture energy and load amplitude. This study provides a mechanistic framework for evaluating long-term bond performance in FRP-strengthened RC structures.

Hu et al. (2023) developed machine learning models to predict debonding failure in reinforced concrete beams strengthened with externally bonded FRP. A database of more than four hundred experimental tests was compiled, incorporating key parameters such as FRP stiffness, bonded length ratio, concrete compressive strength, adhesive layer thickness, and shear-span ratio. Among the models examined, the gradient boosting regression method provided the highest prediction accuracy with strong generalization capability. Parameter sensitivity analysis showed that effective bond length and FRP stiffness were the most influential factors affecting debonding resistance. The study demonstrated that data-driven methods can capture nonlinear trends in bond behavior more effectively than traditional empirical models.

Salahat, Rasheed, and Ashqar (2024) investigated the mechanics of concrete cover delamination in reinforced concrete beams strengthened with externally bonded CFRP. The study focused on explaining the delamination failure mechanism triggered by flexural – shear interaction and crack-induced stress concentrations at the termination of the CFRP laminate. Through a combination of experimental testing and mechanics-based modeling, the authors demonstrated that cover separation is governed by the tensile capacity of the concrete cover and dowel action from internal reinforcement. A simplified design oriented model was proposed to predict delamination onset based on interfacial shear stress and normal stress interaction. The

study emphasized that anchorage detailing and crack control are essential for mitigating premature delamination failures in CFRP-strengthened beams.

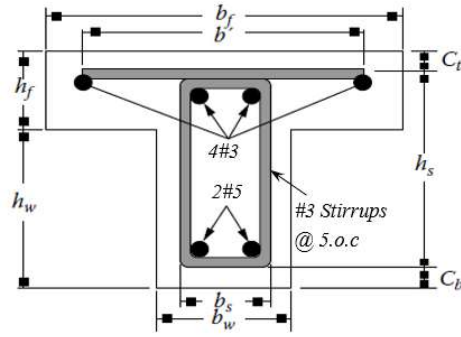
Chapter 3 – Test Specimens Preparation

3.1 General

The research involved designing and testing ten reinforced concrete (RC) T-beams divided into two series: low-strength concrete and normal-strength concrete, with five specimens in each series. Each series included one un-strengthened control beam and one beam incorporating NSM groove preparation, in which two longitudinal grooves were cut along the soffit and filled with epoxy prior to the application of externally bonded CFRP. The remaining three beams in each series were strengthened with externally bonded CFRP sheets applied to the tension face with varying numbers of layers. All beams were tested under four-point bending using displacement control to evaluate flexural behavior and identify CFRP failure modes. The results contributed to understanding the maximum debonding strain of thick CFRP layers in flexural strengthening applications.

3.2 Beam Geometry

The dimensions of the T-beams were 6 in x 12 in (width x depth), whereas the depth includes the thickness of the flange, as illustrated in Figure 3.1. The flange was 16 in wide and 4 in thick. For flexural reinforcement, all beams were equipped with 2#5 steel bars, while the compression steel included 4#3 bars, primarily used to support the shear reinforcement cage with #3 stirrups spaced at 5 in along the span length. All specimens have a span length of 16 ft with a clear span of 15.5 ft.



$$\begin{aligned}
 b_f &= 16 \text{ in.} \\
 b' &= 14 \text{ in.} \\
 C_t &= 1 \text{ in.} \\
 h_f &= 4 \text{ in.} \\
 h_w &= 8 \text{ in.} \\
 h_s &= 10 \text{ in.} \\
 C_b &= 1 \text{ in.} \\
 b_s &= 4 \text{ in.} \\
 b_w &= 6 \text{ in.}
 \end{aligned}$$

Figure 3.1: T-beam cross section details

3.3 Material Properties

3.3.1 Concrete

The concrete used for casting consisted of a mixture of Type I Portland cement, water, sand, gravel, and crushed stone. The ready-mixed concrete batches were designed for a nominal compressive strength of 3 ksi and 4.5 ksi for low-strength and normal-strength series, respectively. According to the batch tickets, each concrete load consisted of KDOT FAA aggregate (Kansas Department of Transportation Fine Aggregate for Air-Entrained concrete), ASTM C33 #5 coarse aggregate (standard graded stone meeting ASTM C33 size #5), Portland cement, an air-entraining admixture (AIR1), and water. The mix properties for each series were shown in Table 3.1 & 3.2.

Low-strength	Material	Batched/CYDS
	KDOTFAA	1992 lb
	ASTMC33#5	1264 lb
	CEMENT	432 lb
	AIR1	2.8 oz
	WATER	25.76 gal

Table 3.1 Mix design proportions for low-strength concrete

Normal-strength	Material	Batched/CYDS
	KDOTFAA	2004 lb
	ASTMC33#5	1272 lb
	CEMENT	490 lb
	WREDUCE1	21.6 oz
	AIR1	2.8 oz
	WATER	19.36 gal

Table 3.2 Mix design proportions for normal-strength concrete

For each concrete batch, six standard cylindrical specimens were prepared for compressive strength testing. The cylinders measured 4 in in diameter and 8 in in height and were tested in accordance with ASTM C39 (ASTM, 2011) using a compression testing machine in the Civil Engineering Laboratory at Kansas State University (Figures 3.2 a & b). After 28 days of moist curing, the average compressive strengths (f'_c) were 2.86 ksi and 4.44 ksi for the low-strength and normal-strength series, respectively. The detailed compressive strength results were presented in Tables 3.3 & 3.4.



(a)



(b)

Figure 3.2: Testing the cylinders: (a) before testing; (b) after testing

Low-Strength series	No	Ultimate Load (kip)	Max stress (ksi)
28-Day	1	28.53	2.27
	2	35.56	2.83
	3	43.91	3.49
	Average		2.86
Testing Day	4	36.07	2.87
	5	41.92	3.36
	6	37.64	3

Table 3.3: The test results of the cylinders for low-strength series

Normal-Strength series	No	Ultimate Load (kip)	Max stress (ksi)
28-Day	1	54.78	4.36
	2	55.98	4.46
	3	56.36	4.49
	Average		4.44
Testing Day	4	43.39	3.45
	5	45.53	3.62
	6	42.84	3.41

Table 3.4: The test results of the cylinders for normal-strength series

3.3.2 Steel Reinforcement

According the manufacturer, the longitudinal #5 reinforcing bars had a modulus of elasticity of 29000 ksi and a yield strength of 79.8 ksi, while the #3 bars used for compression reinforcement and stirrups exhibited a modulus of elasticity of 29000 ksi and a yield strength of 70.3 ksi.

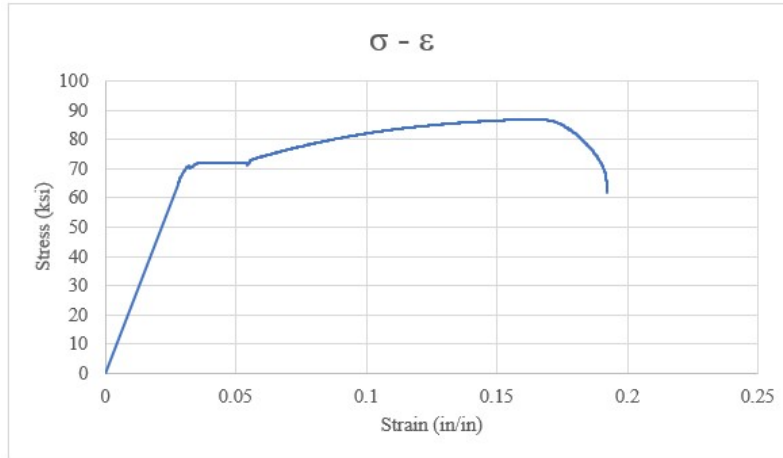


Figure 3.3 Typical stress – strain curve for #3 rebar per manufacturer mill certification

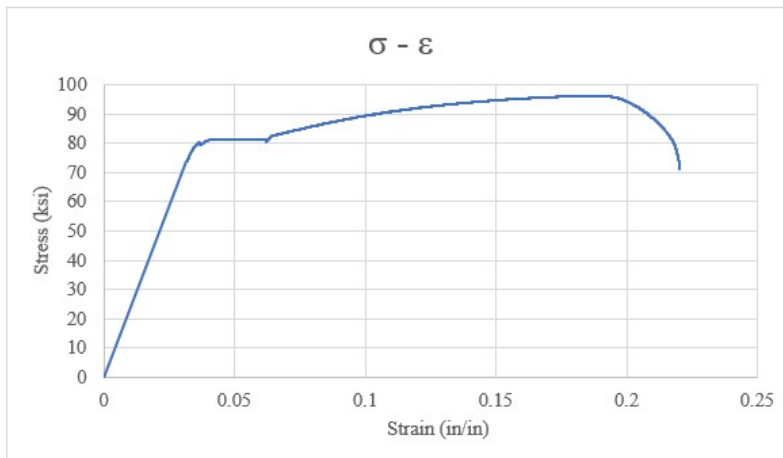


Figure 3.4 Typical stress – strain curve for #5 rebar per manufacturer mill certification

3.3.3 Carbon Fiber Reinforced Polymer (CFRP)

The externally bonded CFRP reinforcement consisted of V-Wrap C400HM unidirectional carbon fiber sheets supplied by Structural Technologies. As specified by the manufacturer, the CFRP fibers of C400HM sheets are oriented longitudinally along one direction of the beam axis only (0°). The corresponding mechanical and physical properties of the CFRP system are presented in Table 3.5.

Product	Thickness (in)	Tensile Strength (ksi)	Modulus of elasticity <i>E</i> (ksi)
V-Wrap C400HM	0.08	180	14240

Table 3.5: Laminate properties of a single CFRP sheet

3.3.4 Epoxy Adhesive

The epoxy adhesive V-Wrap 770 was used to saturate the carbon fiber and bond it to the concrete surface. V-Wrap 770 was a multi-purpose epoxy that serves both as a primer/tack coat and as a saturating resin for V-Wrap carbon and glass fiber systems. This epoxy consists of two parts A and B, which were mixed by weight in a 3:1 ratio (A:B) according to manufacture proportions to achieve a high strength composite bonding. The epoxy physical properties were shown in Table 3.6.

Physical Properties	MPa		Psi
Tensile Strength (ASTM D638)	60.7		8,800
Tensile Modulus (ASTM D638)	2760		400,000
Elongation at Break (ASTM D638)	0.044		0.044
Flexure Strength (ASTM D790)	95		13,780
Flexure Modulus (ASTM D790)	2620		380,000
Compressive Strength (ASTM D695)	85.8		12,450
Compressive Modulus (ASTM D695)	2670		387,000
Density	Packaging		
	Volume	Weight	Package
Part A 9.7 lbs/gal (1.16 kg/L)	2.8 gal	27.3 lbs	5 gal pail
Part B 7.9 lbs/gal (0.95 kg/L)	1.15 gal	9.1 lbs	5 gal pail

Table 3.6: Properties of V-Wrap™ 770 epoxy adhesive.

3.4 Specimens Design

A total of ten reinforced concrete beams were constructed following a consistent procedure. The process began with the preparation of wooden formwork to establish the desired beam geometry and provide support during casting. Steel reinforcement cages were then fabricated in accordance with the design specifications. Each cage was instrumented with two strain gauges positioned at midspan to measure strain responses during later testing. Once completed, the reinforcement cages were placed inside the formwork and secured to ensure proper alignment and concrete cover. Finally, concrete was poured into the formwork, compacted to eliminate air voids, and finished to produce uniform beam specimens suitable for structural testing.

3.4.1 Formwork and Steel Cage

The wooden formwork was constructed in the Civil Engineering wood shop at Kansas State University. Since the available plywood sheets in the shop measured only 48 in $\times$ 96 in, the beam forms had to be fabricated in two separate sections and subsequently joined to achieve the required overall length of 16 ft (Figure 3.6). Dimensional accuracy, alignment, and adequate stiffness of the assembled forms were ensured to prevent deformation or leakage during casting.



Figure 3.5: Wood formworks

The steel reinforcement cages were prepared in the steel fabrication shop at K-State Civil Infrastructure Systems Laboratory (CISL). Shear reinforcement stirrups were bent to the specified dimensions using a rebar bender to achieve uniformity and precision. The longitudinal reinforcement bars were then placed according to the design configuration and securely tied to the stirrups with steel rebar ties, ensuring proper spacing, alignment, and structural integrity of the cage.

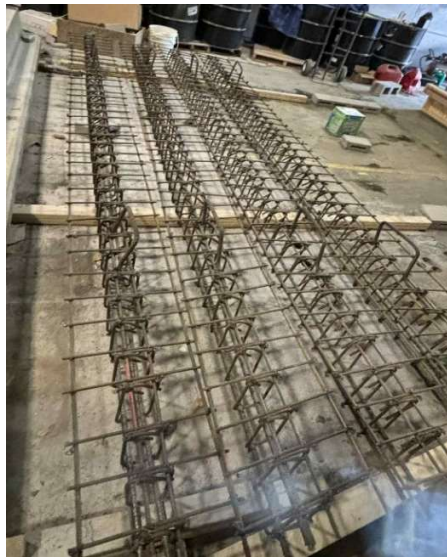


Figure 3.6: Steel cages

3.4.2 Casting

Concrete casting was carried out in two separate batches: one for the low-strength specimens and another for the normal-strength specimens. The low-strength beams were cast with ready-mix concrete having a target compressive strength of 3 ksi, while the normal-strength beams were cast with 4.5 ksi concrete. At the same time, six standard concrete cylinders were cast from each batch for quality control testing. All specimens were cured for at least 28 days. The beam curing was performed by covering with plastic sheets to minimize moisture loss, while

the cylinders were stored in a controlled moisture room until testing. The compressive strength of the cylinders was then determined at 28 days to establish the average concrete strength.

3.4.3 Installation Procedure for CFRP Strengthening

The flexural strengthening of the reinforced concrete beams using externally bonded (EB) CFRP sheets was performed in accordance with recommended practices in ACI PRC 440.2-23 and installation guidelines from the manufacture. Prior to installation, the FRP reinforcement was installed using V-WRAP C400 HM sheets with a width of 150 mm and a length of 4572 mm, and the concrete substrate was inspected to verify that it satisfied the minimum surface integrity and moisture requirements specified in ACI PRC 440.2-23. The general installation procedure was as follows:

Step 1: Surface Preparation – The concrete soffit was mechanically ground and cleaned to remove laitance, dust, and contaminants, producing a roughened bonding surface consistent.

- *For the specimens with NSM groove: two longitudinal grooves were saw-cut along the soffit before FRP installation.*

Step 2: Epoxy Application – A thin layer of structural epoxy adhesive was applied to fill surface voids and enhance adhesion at the FRP–concrete interface.

Step 3: Putty Application – An epoxy-based leveling putty was applied to eliminate surface irregularities and provide a uniform bonding surface for the CFRP laminate.

- *For the specimens with NSM groove: the epoxy putty was also used to completely fill the pre-cut grooves to restore surface continuity prior to laminate installation.*

Step 4: CFRP Saturation – The dry CFRP fabric was positioned on a flat working surface and saturated with the mixed resin. The resin was worked into the fibers using a roller to achieve full impregnation and eliminate dry spots.

Step 5: Application of Saturated CFRP Sheet – Saturated CFRP sheets were placed on the resin-coated surface and aligned longitudinally along the beam axis. Additional saturating resin was applied over the sheet for multiple layers.

Step 6: Impregnation and Rolling – A stiff rubber or aluminum roller was used to smooth the fabric and eliminate any trapped air bubbles, ensuring full contact with the substrate.

The resin-saturated composite must be allowed to cure completely. Curing can happen at room temperature or be accelerated with heat, depending on the specific resin used. All steps were illustrated in Figure 3.7.

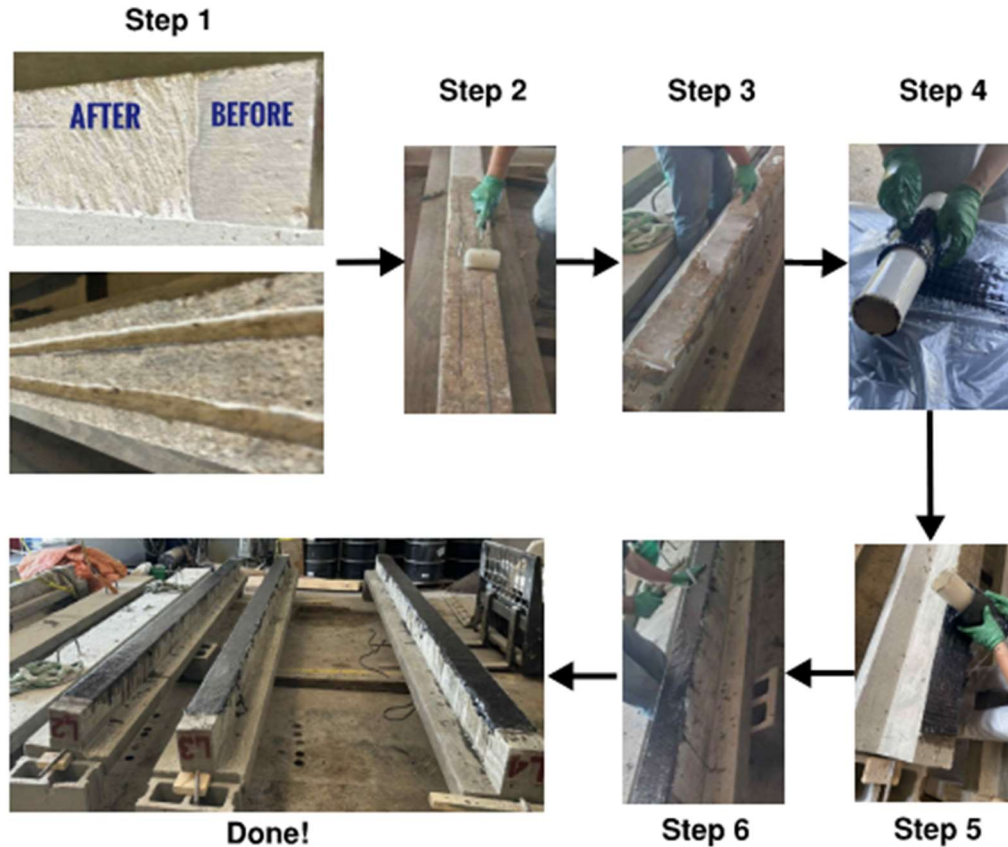


Figure 3.7: Procedure of FRP installation

3.5 Test Setup

The debonding tests were conducted in the structural testing laboratory at Kansas State University. All beams were tested under four-point bending using a 4 ft long steel spreader beam and a 50 kips hydraulic actuator, as shown in Figure 3.8. The beams were simply supported with a pinned support at one end and a roller support at the other, positioned 3 in from each edge to provide a clear span of 15.5 ft. The shear span on each side of the applied loads was 5.75 ft. Midspan deflection was recorded using two linear variable displacement transducers (LVDTs). Strain instrumentation consisted of two electrical resistance strain gages embedded on the longitudinal reinforcement at midspan prior to casting, two strain gages affixed to the top surface

of the concrete to capture compressive strains, and two strain gages bonded to the externally bonded CFRP sheets at the critical midspan section to monitor FRP tensile strains. Load application was displacement-controlled at a constant rate of 0.05 in/min until FRP debonding.

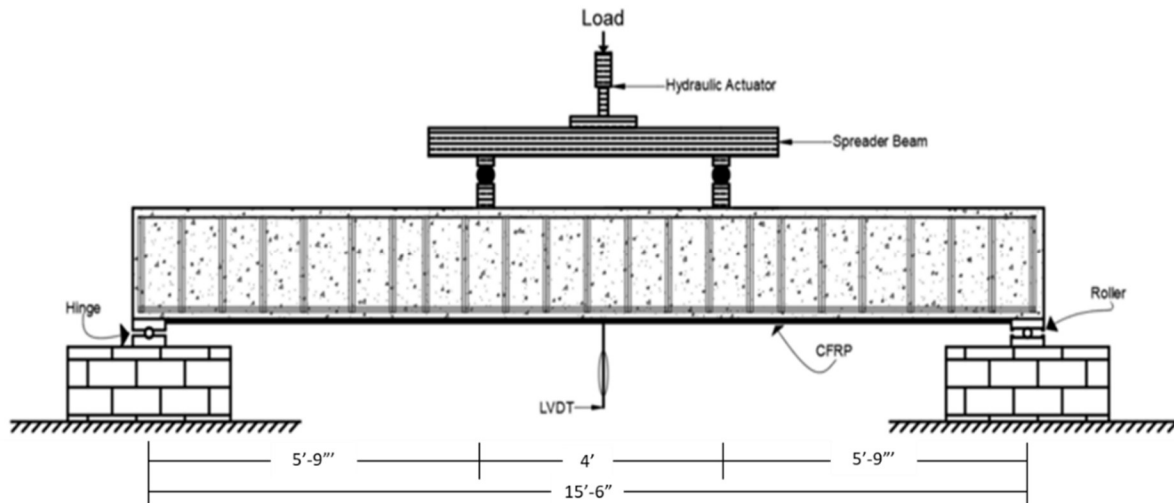


Figure 3.8: Experimental Setup

Chapter 4 – Testing

4.1 Numerical Analysis

4.1.1 Computer Program

The analytical model for FRP debonding strain was developed under the following constitutive assumptions. A linear strain distribution across the section depth was imposed in accordance with shallow beam theory. The compressive stress - strain response of concrete was represented by the Hognestad parabola, whereas the tensile behavior was considered linear elastic up to sudden brittle cracking. Reinforcing steel in both tension and compression was modeled as elastic - perfectly plastic. The FRP reinforcement in tension was assumed to follow a linear elastic response up to the governing failure mechanism, defined as either debonding, rupture, or crushing. These assumptions established the basis for quantifying the strain state at debonding and for evaluating the corresponding failure criteria. Debonding strain of FRP was calculated according ACI PRC-440.2-23.

An Excel-based analysis program was developed at Kansas State University to conduct nonlinear analysis of concrete beams reinforced with mild steel and strengthened with FRP. The program performs iterative analysis up to the governing limit state, defined by either FRP debonding strain, FRP rupture strain, or concrete crushing strain, whichever occurs first. A graphical interface is incorporated to display the beam cross-section, reinforcement layout, applied loads, and results of the flexural analysis, thereby providing both computational and visual assessment of the behavior's specimens.

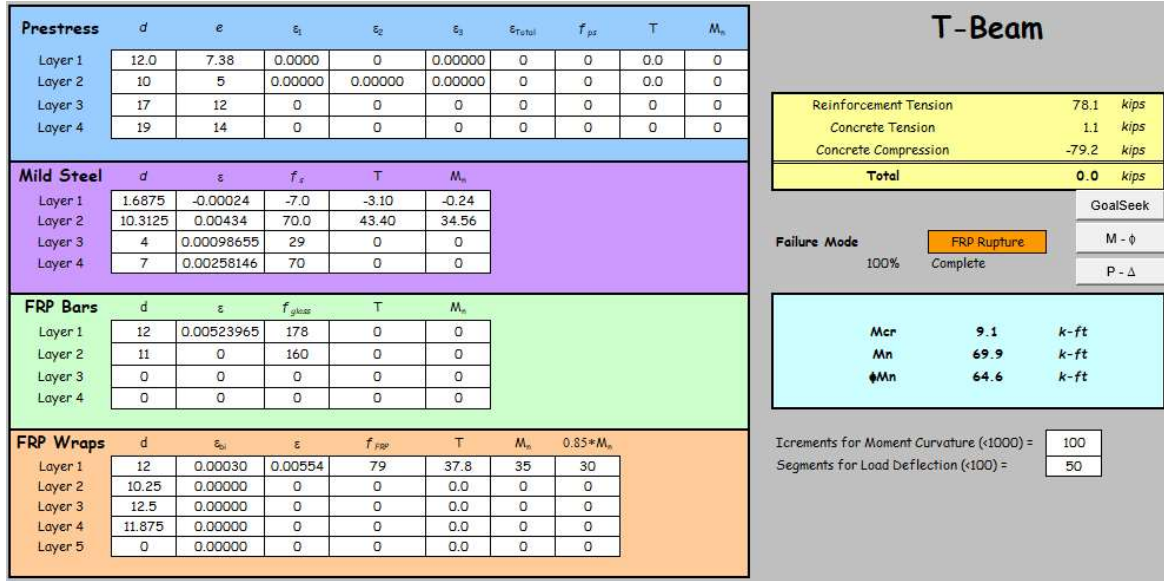


Figure 4.1: Output interface for the Excel analysis program

4.1.2 Debonding Strain Equations in Various Design Codes

This study compares general debonding strain equations proposed in major international design guidelines, including ACI PRC-440.2-23, TR 55-2012, CNR-DT 200, fib Bulletin 14-2001, and JSCE. Since the original formulations are expressed in SI units, all equations were rigorously converted into U.S. customary units to maintain consistency with the experimental program.

4.1.2.1 American Concrete Institute (ACI PRC-440.2-23)

The ACI PRC-440.2-23 guideline limits the design tensile strain in the FRP to prevent intermediate crack-induced debonding. The allowable effective strain in the FRP is given by:

$$\epsilon_{fd} = 0.41 \sqrt{\frac{f'_c}{n E_f t_f}} \leq 0.9 \epsilon_{fu} \quad (1)$$

Where:

f'_c : Concrete compressive strength (MPa)

n : Number layers of FRP

E_f : Modulus of elasticity of FRP (MPa)

t_f : Thickness of FRP (mm)

ε_{fu} : Design rupture strain of FRP

4.1.2.2 UK Concrete Society Technical Report 55 (TR 55-2012)

The TR55 guideline limits the FRP strain using a simplified equation:

$$\varepsilon_{fd} = \frac{\varepsilon_{fk}}{\gamma_{FRP,mE}} \leq 0.008 \quad (2)$$

Where:

ε_{fk} : Characteristic ultimate strain of FRP

$\gamma_{FRP,mE}$: Design partial safety factor for FRP strain

$$\gamma_{FRP,mE} = \gamma_{FRP,\varepsilon} \gamma_{FRP,m} \quad (3)$$

4.1.2.3 Italian Guidelines (CNR-DT 200)

The CNR-DT 200 R1/2013 guideline adopts a strain limit approach and introduces partial safety factors for material properties and interfacial bond behavior. The design debonding strain is calculated as:

$$f_{fd,2} = \frac{k_q}{\gamma_{f,d}} \sqrt{\frac{E_f}{nt_f} \cdot \frac{2k_b k_G}{FC} \sqrt{f_{cm} f_{ctm}}} \quad (4)$$

$$\varepsilon_{fd} = \frac{f_{fd,2}}{E_f} \geq \varepsilon_{sy} - \varepsilon_o \quad (5)$$

Where:

$f_{fd,2}$: The ultimate design strength for IC debonding (MPa)

k_q : Empirical coefficient

= 1 for concentrated loads

= 1.25 for distributed loads

k_b : Geometric (width) factor

$$k_b = \sqrt{\frac{2-b_f/b}{1+b_f/b}} \quad (6)$$

k_G : Fracture energy coefficient

= 0.023 mm for the average case

FC = 1: An appropriate confidence factor

f_{cm} : Concrete compressive strength (MPa)

f_{ctm} : Concrete tensile strength (MPa)

$\gamma_{f,d}$ = 1.2: A partial safety factor for the FRP material properties

E_f : Modulus of elasticity of FRP (MPa)

n: Number of FRP layers

ε_{sy} : Design strain of the existing reinforcement at the point of yielding

ε_o : Pre-existing strain before the application of the strengthening at the extreme tensile flange

4.1.2.4 International Federation for Structural Concrete (fib Bulletin 14-2001)

The fib Bulletin 14 provides more detailed approaches, including the calculation of the interfacial shear stress limit and the effective bond length. The design stress in FRP is limited based on the fracture energy of the bond interface:

$$\varepsilon_{fd} = \alpha c_1 k_c k_b \sqrt{\frac{f_{ct}}{n E_f t_f}}, \text{ if } l_b \geq l_{b,max} \quad (7a)$$

$$\varepsilon_{fd} = \alpha c_1 k_c k_b \sqrt{\frac{f_{ct}}{n E_f t_f}} \frac{l_b}{l_{b,max}} \left(2 - \frac{l_b}{l_{b,max}} \right), \text{ if } l_b < l_{b,max} \quad (7b)$$

$$l_{b,max} = \sqrt{\frac{n E_f t_f}{c_2 f_{ct}}} \quad (8)$$

Where:

α : Reduction factor accounting for the influence of inclined cracks on the bond strength ($\alpha=1$ for beams with sufficient internal and external shear reinforcement and for slabs, otherwise $\alpha=0.9$).

$c_1 = 0.64$ and $c_2 = 2$ for CFRP strips.

k_c : Factor accounting for the state of compaction of concrete (if FRP bonded to concrete faces with low compaction $k_c = 1$, e.g. faces not in contact with the form work during casting, $k_c = 0.67$)

l_b : FRP bond length.

k_b : Geometry factor.

$$k_b = 1.06 \sqrt{\frac{2 - \frac{b_f}{b}}{1 + \frac{b_f}{400}}} \quad (9)$$

4.1.2.5 Japan Society of Civil Engineers (JSCE)

To prevent IC debonding, the JSCE sets a limiting effective strain in the FRP material, calculated using the following equation:

$$\varepsilon_{fd} = \sqrt{\frac{2G_f}{nE_ft_f}} \quad (10)$$

Where:

G_f : The interfacial fracture energy between FRP and concrete and its value can be taken as 0.5N/mm.

n : Number layers of FPR

E_f : Modulus of elasticity of FRP (MPa)

t_f : Thickness of FRP (mm)

4.2 Experimental Program

The experimental program involved designing and testing ten reinforced concrete (RC) T-beams divided into two series: low-strength concrete and normal-strength concrete, with five specimens in each series. Each series included one un-strengthened control beam and one beam incorporating NSM groove preparation, in which two longitudinal grooves were cut along the soffit and filled with epoxy prior to the application of externally bonded CFRP. The remaining three beams in each series were strengthened with externally bonded CFRP sheets applied to the tension face with varying numbers of layers. All beams were tested under four-point bending using displacement control at a constant rate of 0.05 in/min to evaluate flexural behavior and

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4.2.1 Beam Geometry

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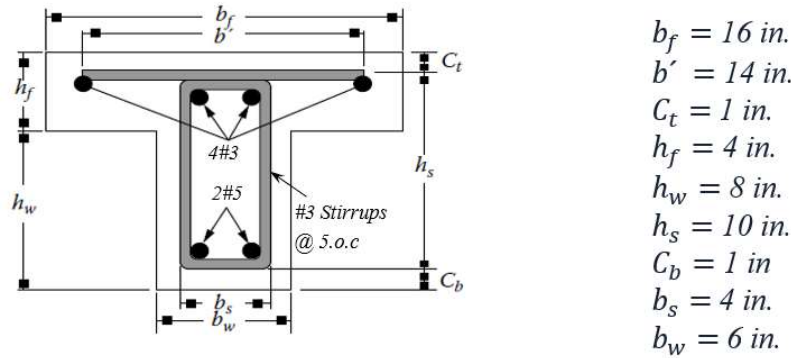


Figure 4.2: Beam Cross Section

4.2.2 FRP Layout

In this study, two series of reinforced concrete beams were strengthened using unidirectional CFRP sheets (V-Wrap C400HM) applied in a layered configuration. The low-strength series (L0LT, L1LT, L2LT, L3LT, L1LTG) and the normal-strength series (H0LT, H1LT, H2LT, H3LT, H1LTG). Control specimens L0LT and H0LT were left un-strengthened. Beams L1LT, L2LT, and L3LT in the low-strength series and H1LT, H2LT, and H3LT in the normal-

strength series were strengthened along the soffit using externally bonded unidirectional CFRP sheets measuring 6 in $\times$ 15 ft (width $\times$ length), applied in one, two, and three layers, respectively (Figure 4.3). The hybrid beams L1LTG and H1LTG were fabricated with two longitudinal grooves along the soffit, each with a cross-sectional dimension of 0.25 in $\times$ 0.75 in and a length of 15 ft. The grooves were filled with epoxy putty to restore surface continuity prior to the installation of a single externally bonded CFRP sheet measuring 6 in $\times$ 15 ft, as illustrated in Figure 4.4.

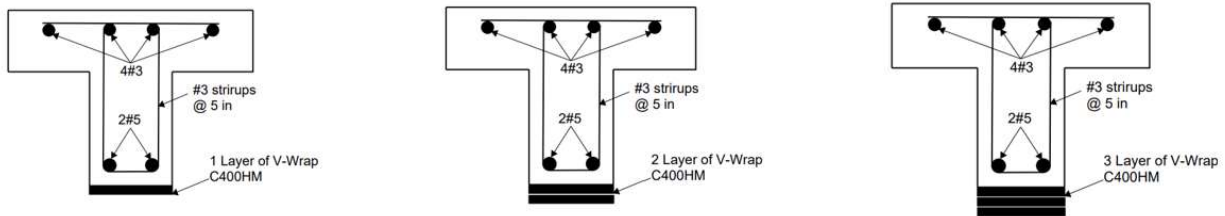


Figure 4.3: Externally bonded CFRP layout

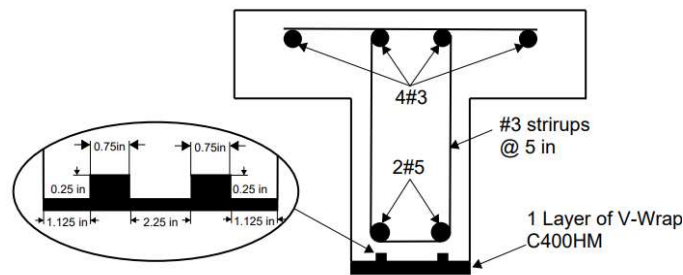


Figure 4.4: NSM groove with externally bonded CFRP layer

4.2.3 Low Strength Series

The geometry of the specimens in the low-strength series was illustrated in Figure 4.2. The concrete reached an average compressive strength of 2.86 ksi at 28 days. The longitudinal #5 reinforcing bars had a modulus of elasticity of 29000 ksi and a yield strength of 79.8 ksi, while

the #3 bars used for compression reinforcement and stirrups exhibited a modulus of elasticity of 29000 ksi and a yield strength of 70.3 ksi according to the manufacturer.

4.2.3.1 Control Specimen (L0LT)

The first specimen in the low-strength series, designated as L0LT, served as the control beam without any CFRP strengthening. The specimen was tested under four-point bending using displacement control at a loading rate of 0.05 in/min. The beam exhibited linear elastic behavior up to the onset of steel yielding, which occurred at an experimental load of approximately 12.01 kips, as identified from the change in slope of the load–strain response of the tensile steel reinforcement (Figure 4.8). The experimental ultimate load was 12.74 kips with a corresponding mid-span deflection of 3.38 in, and failure was governed by concrete crushing in the compression zone after steel yielding. The numerical analysis predicted ultimate load of 12.8 kips and a mid-span deflection of 3.03 in (Figure 4.6). The predicted failure load showed very close agreement with the experimental result. The failure mode of concrete crushing occurred immediately when the concrete compressive strain at failure approached the ultimate strain limit, with a recorded value of 0.0033 (Figure 4.7), confirming a flexural compression failure mode consistent with over-reinforced behavior. The final failure was characterized by crushing of the concrete in the compression zone near mid-span (Figure 4.5).



Figure 4.5: LOLT beam after testing

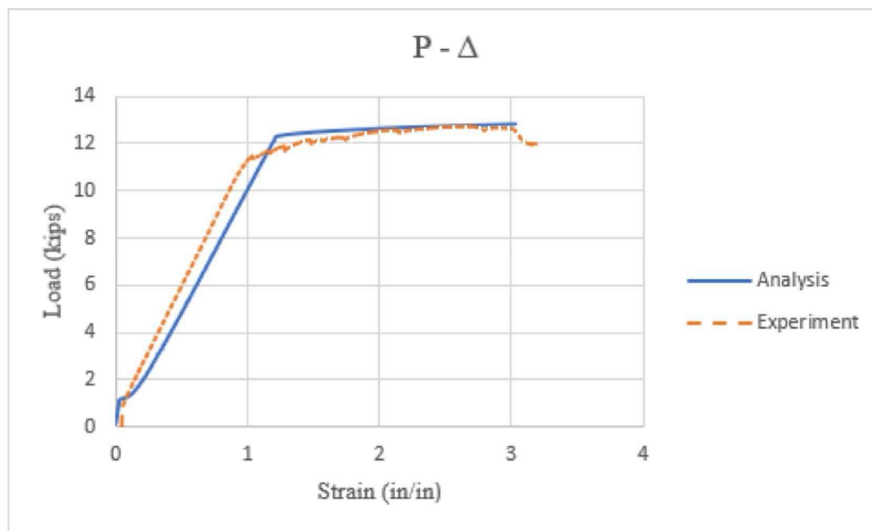


Figure 4.6: Load– displacement response (LOLT)

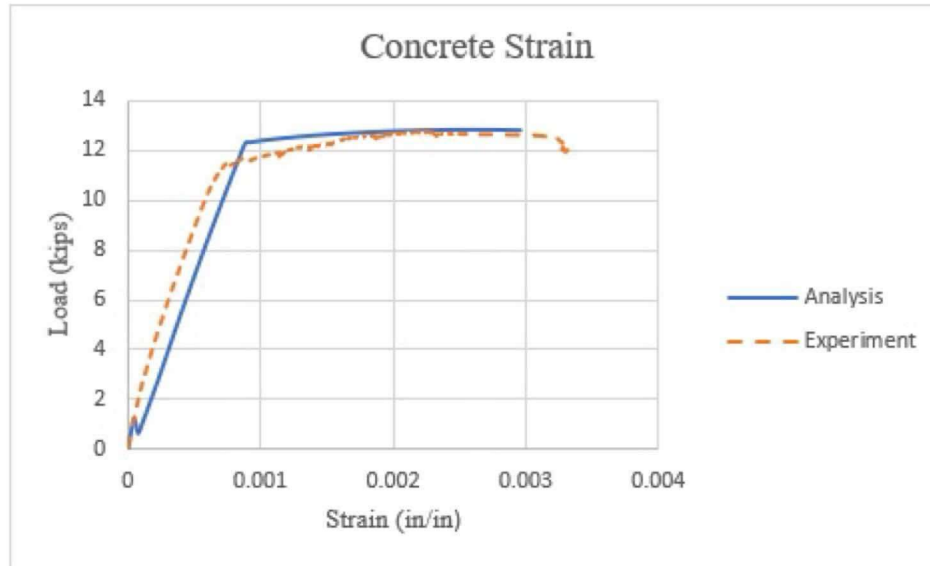


Figure 4.7: Load – concrete strain response (L0LT)

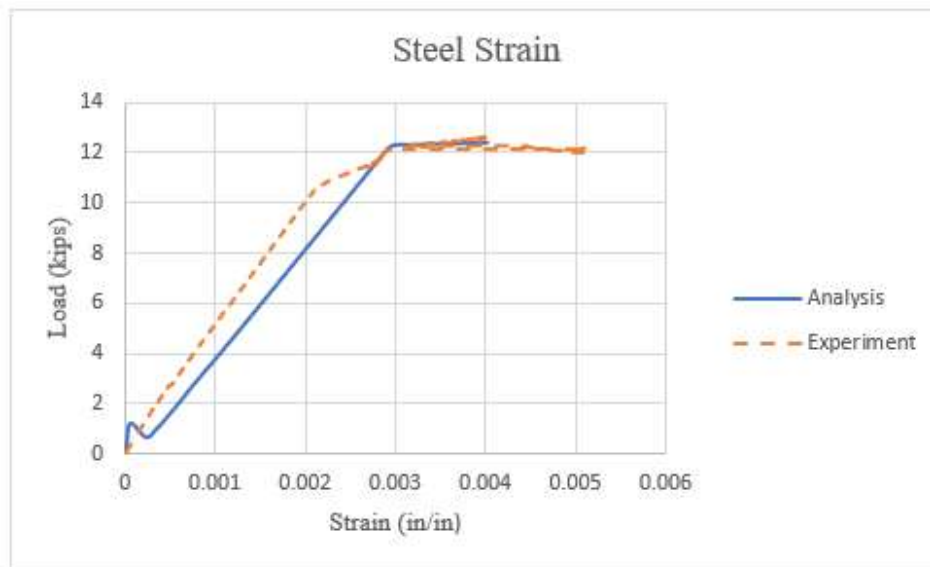


Figure 4.8: Load – steel strain response (L0LT)

4.2.3.2 Specimen with One Layer of CFRP (L1LT)

The L1LT specimen which was strengthened with a single layer of externally bonded CFRP sheet. Compared to the un-strengthened control beam L0LT, which failed at 12.74 kips,

the L1LT beam exhibited a significant increase in load capacity, reaching an experimental debonding load of 22.91 kips, representing an improvement of approximately 80%. The corresponding mid-span deflection at failure was 1.78 in, which is lower than that of L0LT (3.38 in), indicating increased stiffness due to CFRP strengthening. The numerical analysis predicted a slightly lower ultimate load of 21.2 kips and a corresponding mid-span deflection of 1.53 in. The maximum FRP strain recorded experimentally was 0.0042, showing excellent agreement with the analytical prediction of 0.0042. (Figure 4.13). The comparison between the theoretical and experimental results of load-strain in concrete and steel were shown in Figures 4.11 and 4.12 respectively.



Figure 4.9: L1LT beam after testing

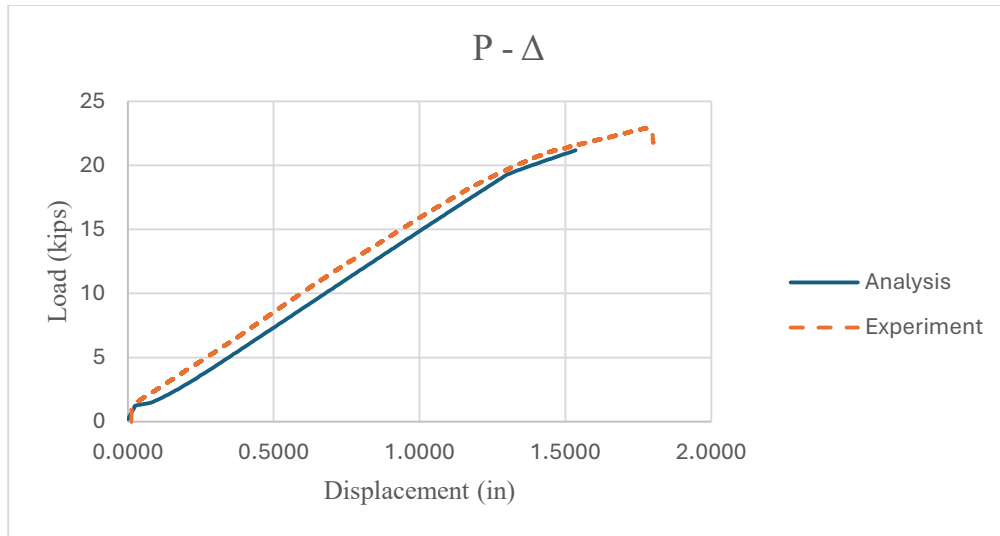


Figure 4.10: Load-displacement response (L1LT)

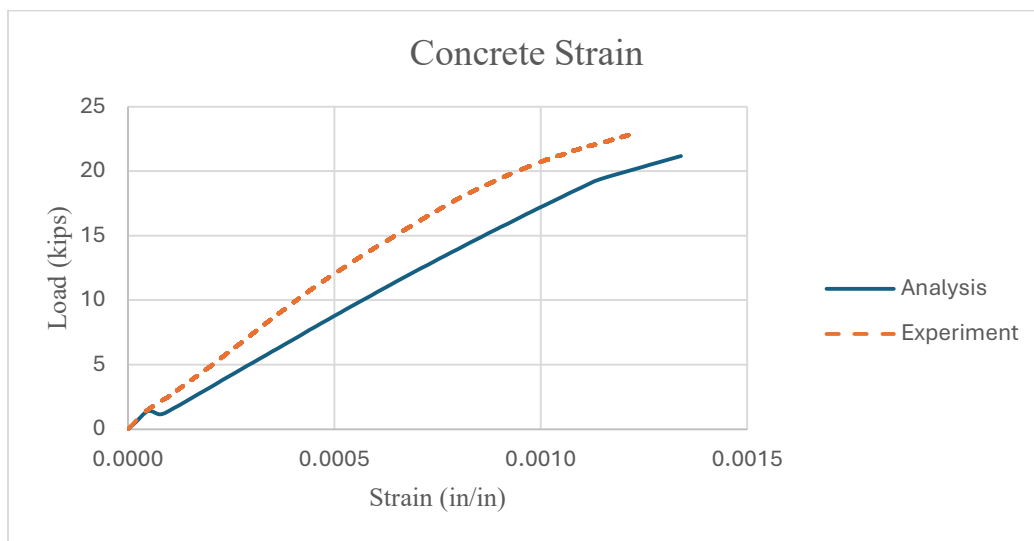


Figure 4.11: Load- concrete strain response (L1LT)

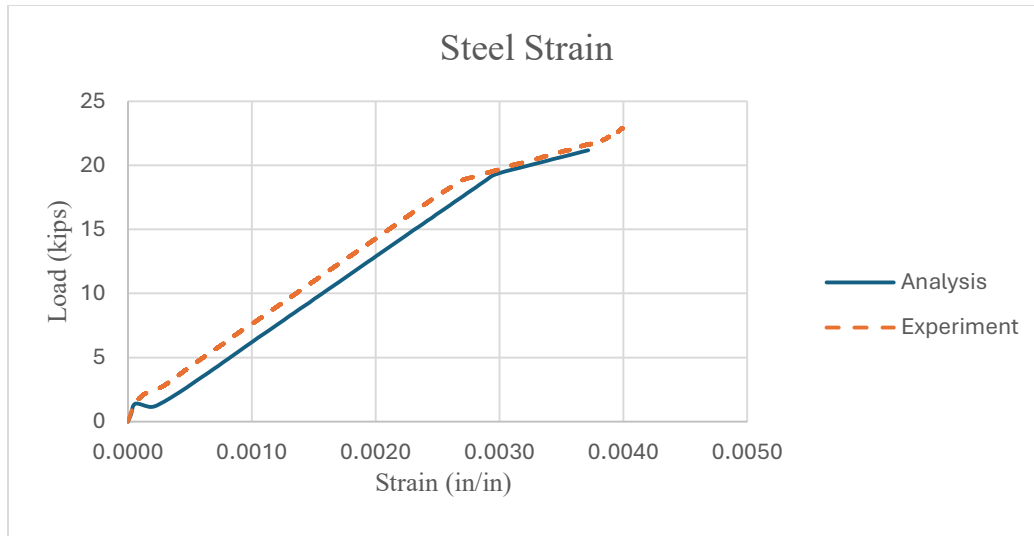


Figure 4.12: Load- steel strain response (L1LT)

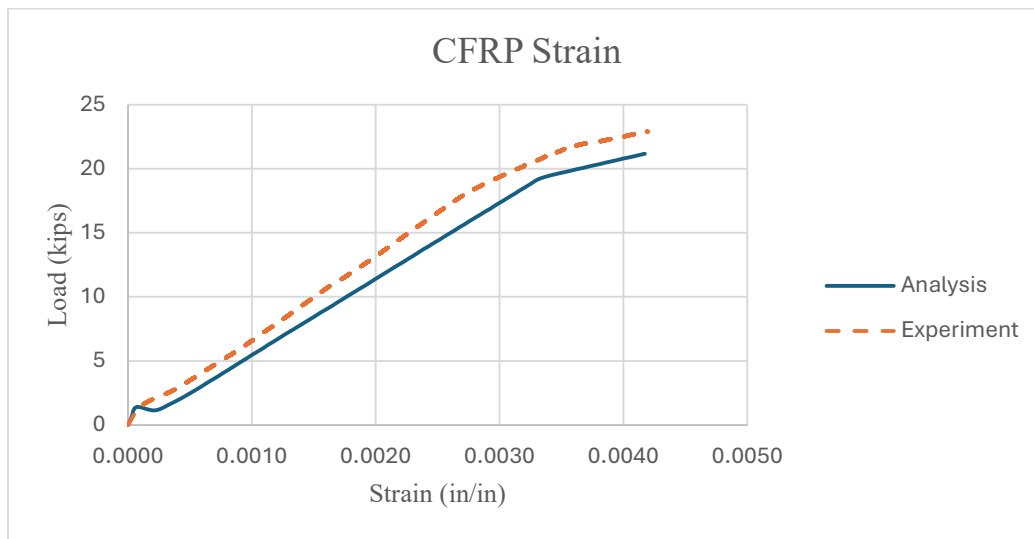


Figure 4.13: Load- CFRP strain response (L1LT)

4.2.3.3 Specimen with Two Layers of CFRP (L2LT)

The L2LT specimen which was strengthened with two layers of externally bonded CFRP sheets. The beam failed by FRP debonding under four-point bending. The experimental debonding load was 28.68 kips, which is approximately 125% higher than the ultimate load of L0LT (12.74 kips) and 25% higher than L1LT (22.91 kips), confirming the effectiveness of using

multiple CFRP layers in enhancing flexural strength. The corresponding mid-span deflection at failure was 1.53 in, lower than both L0LT (3.38 in) and L1LT (1.78 in), indicating increased stiffness due to higher CFRP reinforcement. The numerical analysis predicted a failure load of 23.1 kips and a deflection of 1.21 in, lower than the experimental capacity by approximately 21% (Figure 4.15). The maximum recorded FRP strain was 0.0039, while the analytical model predicted a lower strain of 0.003 (Figure 4.18). The comparison between the theoretical and experimental results of load-strain in concrete and steel were shown in Figures 4.16 and 4.17 respectively.



Figure 4.14: L2LT beam after testing

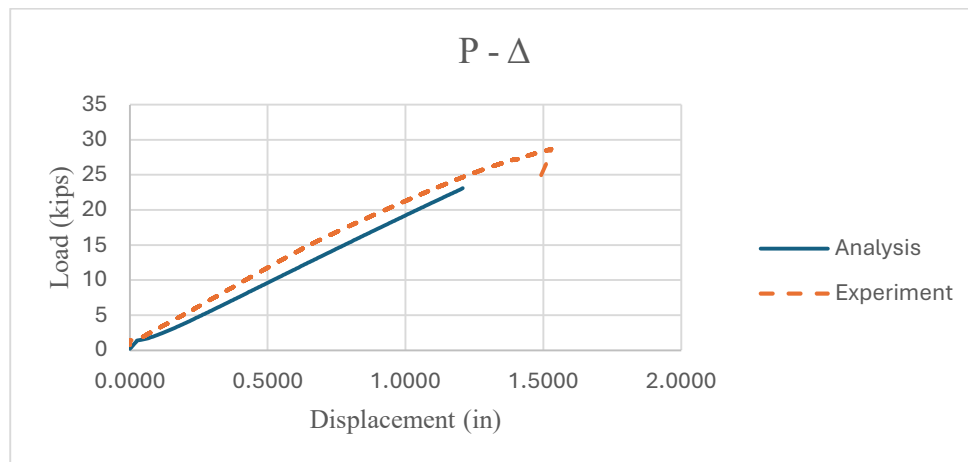


Figure 4.15: Load–displacement response (L2LT)

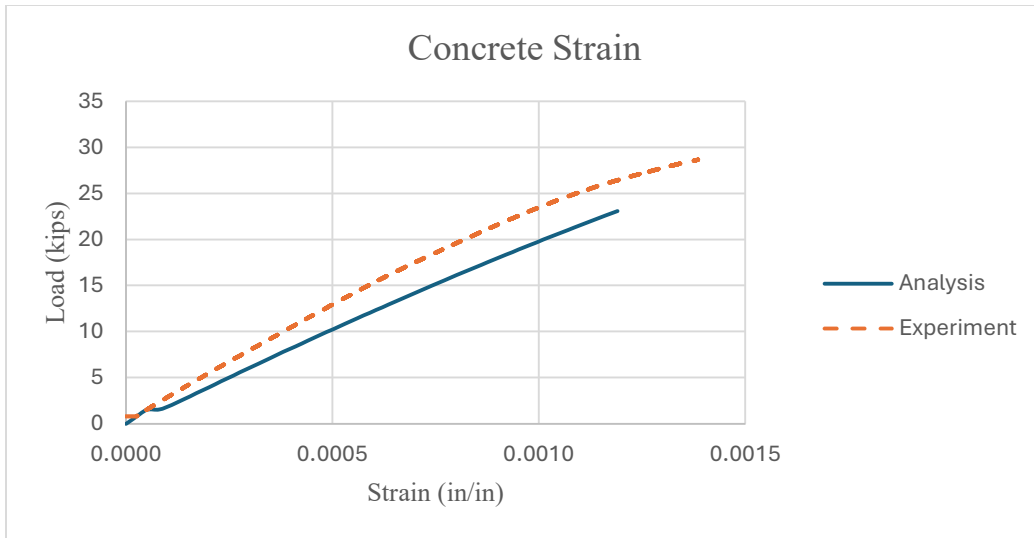


Figure 4.16: Load – concrete strain response (L2LT)

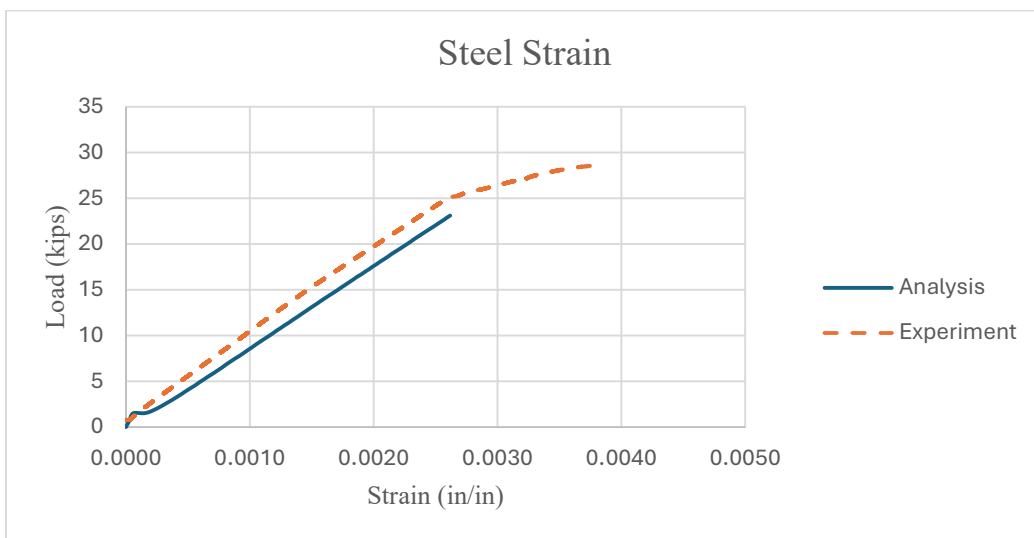


Figure 4.17: Load – steel strain response (L2LT)

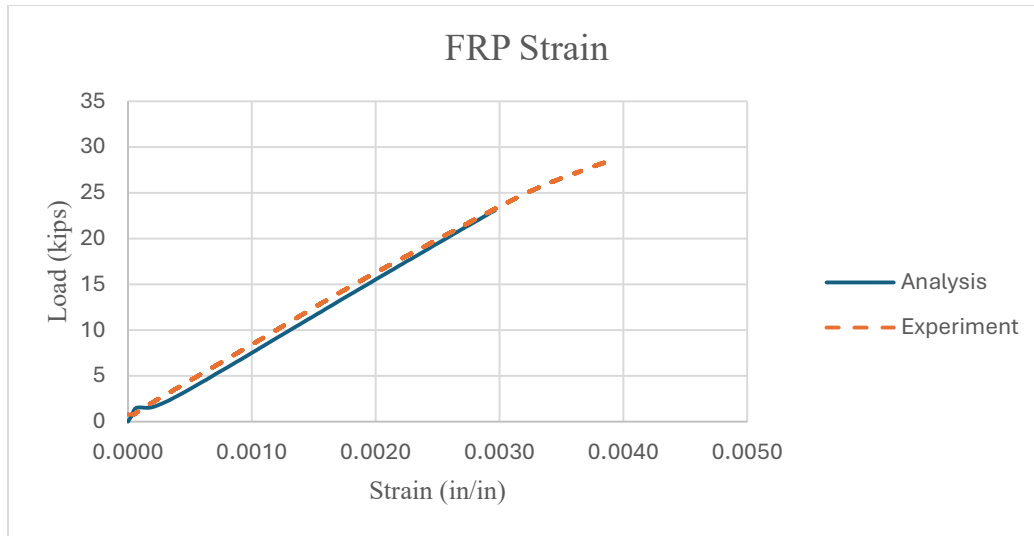


Figure 4.18: Load – FRP strain response (L2LT)

4.2.3.4 Specimen with Three Layers of CFRP (L3LT)

The L3LT specimen, strengthened with three layers of externally bonded CFRP sheets, was tested under four-point bending and failed by FRP delamination. The experimental debonding load was 27.3 kips, which is approximately 114% higher than the control beam L0LT (12.74 kips) but slightly lower than the capacity of L2LT (28.68 kips). The mid-span deflection at failure was 1.19 in, significantly lower than L0LT (3.38 in) and also lower than L1LT (1.78 in) and L2LT (1.53 in), reflecting increased stiffness due to higher CFRP reinforcement. The numerical analysis predicted a failure load of 23.32 kN with a corresponding deflection of 0.95 in, lower than the experimental capacity by approximately 15% (Figure 4.20). The maximum CFRP strain recorded during testing was 0.0035, whereas the analytical model predicted 0.0024 (Figure 4.23). The comparison between the theoretical and experimental results of load-strain in concrete and steel were shown in Figures 4.21 and 4.22 respectively.



Figure 4.19: L3LT beam after testing

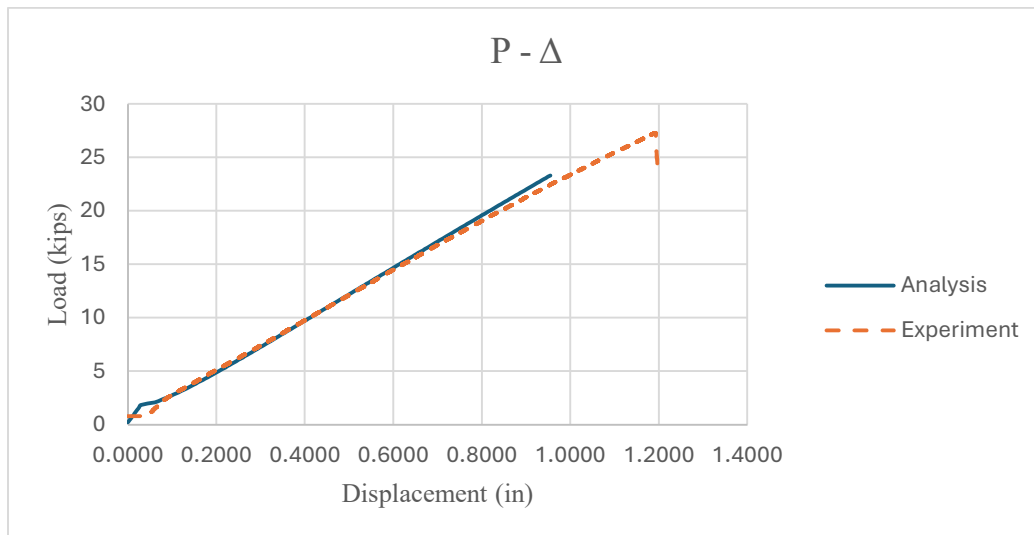


Figure 4.20: Load– displacement response (L3LT)

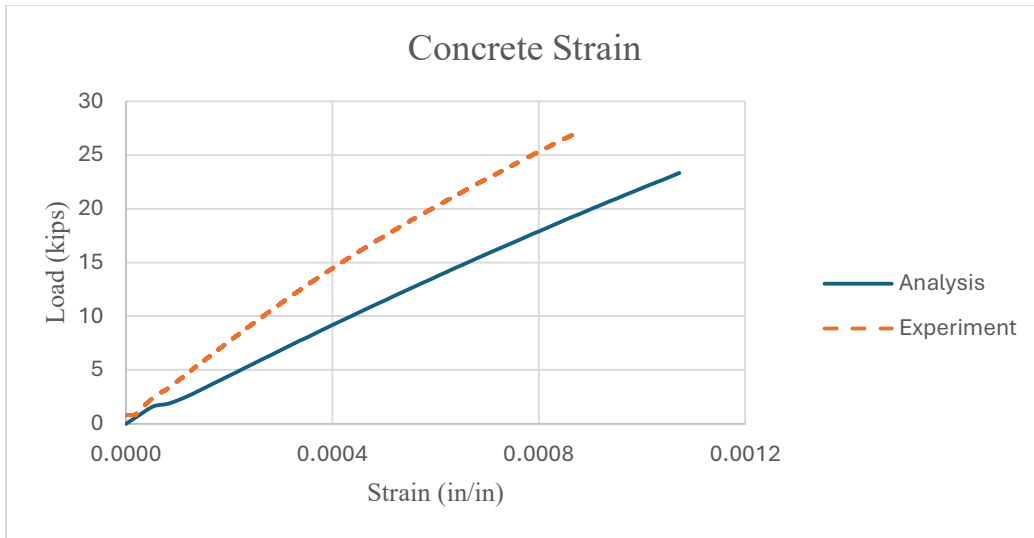


Figure 4.21: Load – concrete strain response (L3LT)

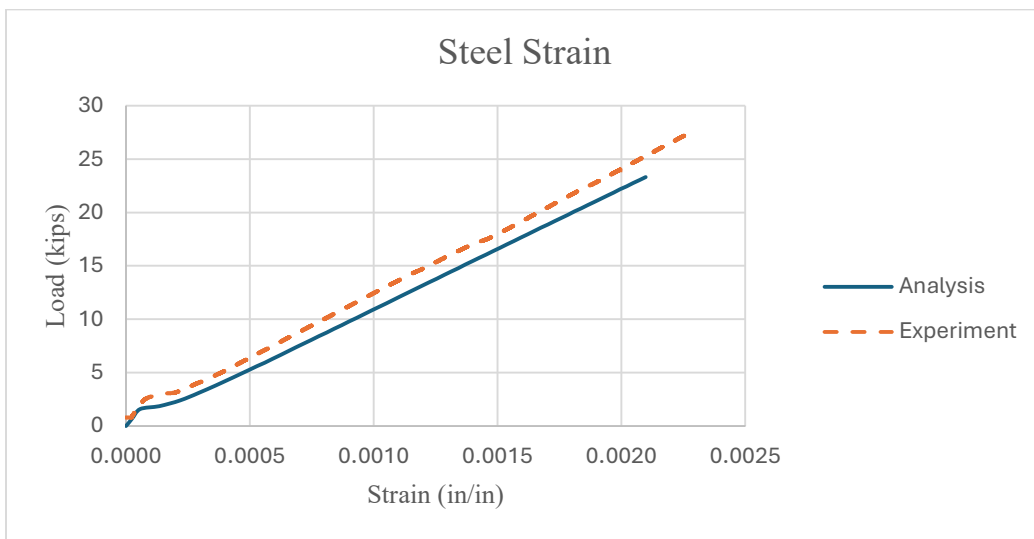


Figure 4.22: Load – steel strain response (L3LT)

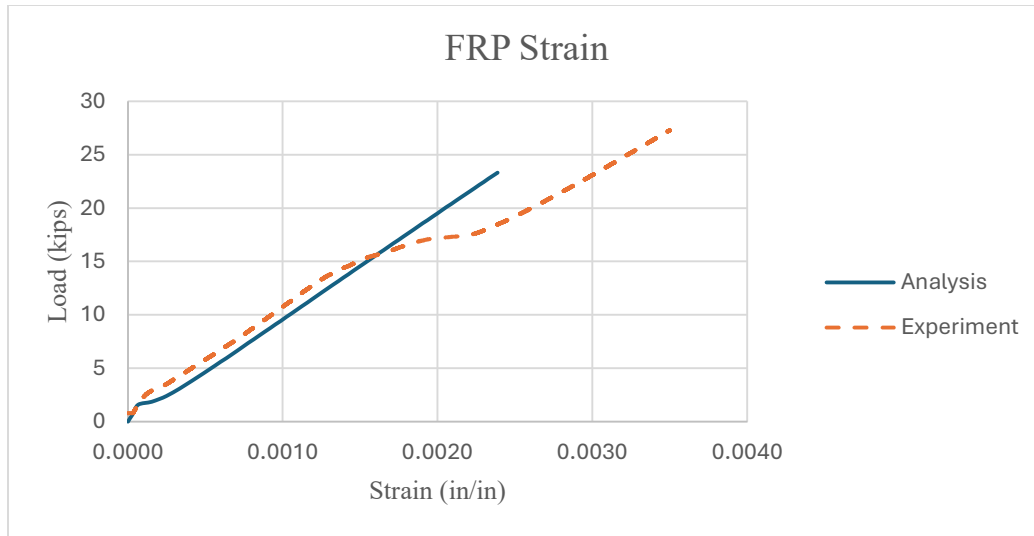


Figure 4.23: Load – FRP strain response (L3LT)

4.2.3.5 Specimen with One Layer of CFRP and Grooving (L1LTG)

The L1LTG specimen was strengthened using a hybrid configuration combining two near-surface mounted (NSM) CFRP strips (0.25 in $\times$ 0.75 in) with one layer of externally bonded CFRP (EB-CFRP) sheet on the soffit. The specimen was failed by FRP rupture rather than debonding. The experimental ultimate load was 21.36 kips, representing an increase of approximately 68% compared to the control beam L0LT (12.74 kips) but slightly lower than L1LT (22.91 kips). The mid-span deflection at failure was 1.7 in, which was lower than L0LT (3.38 in) and comparable to L1LT (1.78 in), suggesting similar stiffness improvement due to CFRP strengthening (Figure 4.25). The maximum recorded CFRP strain was 0.0037, which is lower than that of L1LT (0.0043), indicating early attainment of the rupture strain in the NSM strips due to their higher stress concentration (Figure 4.28). The comparison between the theoretical and experimental results of load-strain in concrete and steel were shown in Figures 4.26 and 4.27 respective.



Figure 4.24: L1LTG beam after testing

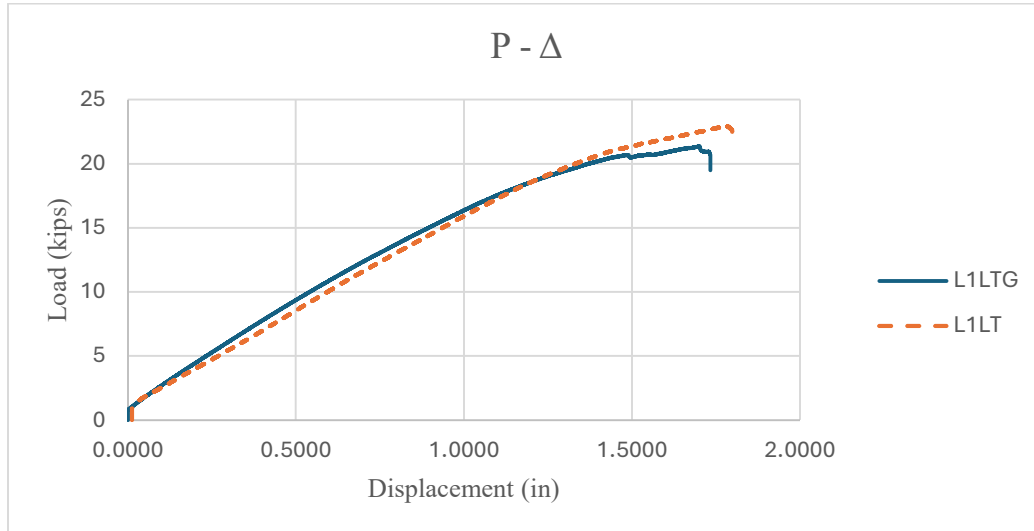


Figure 4.25: Load– displacement response

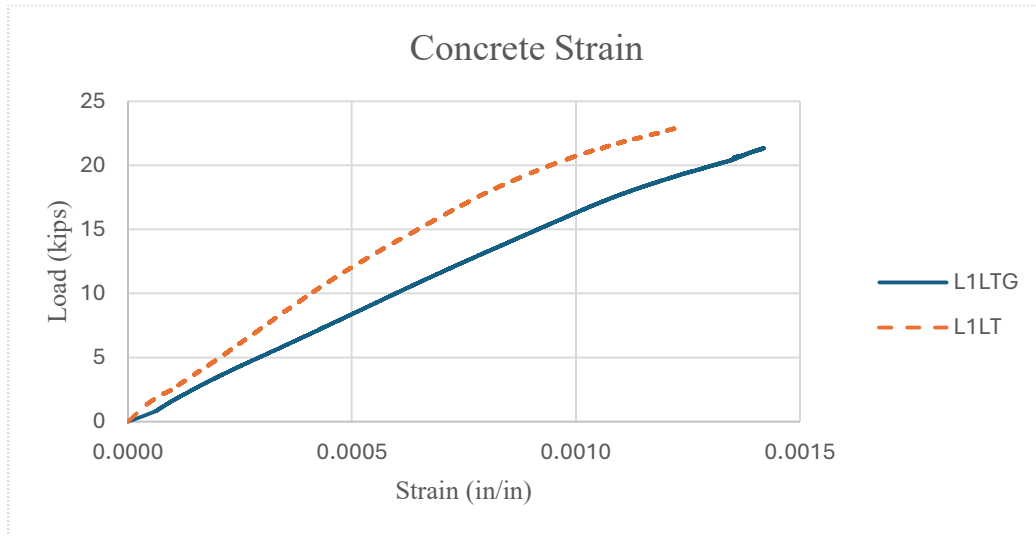


Figure 4.26: Load – concrete strain response

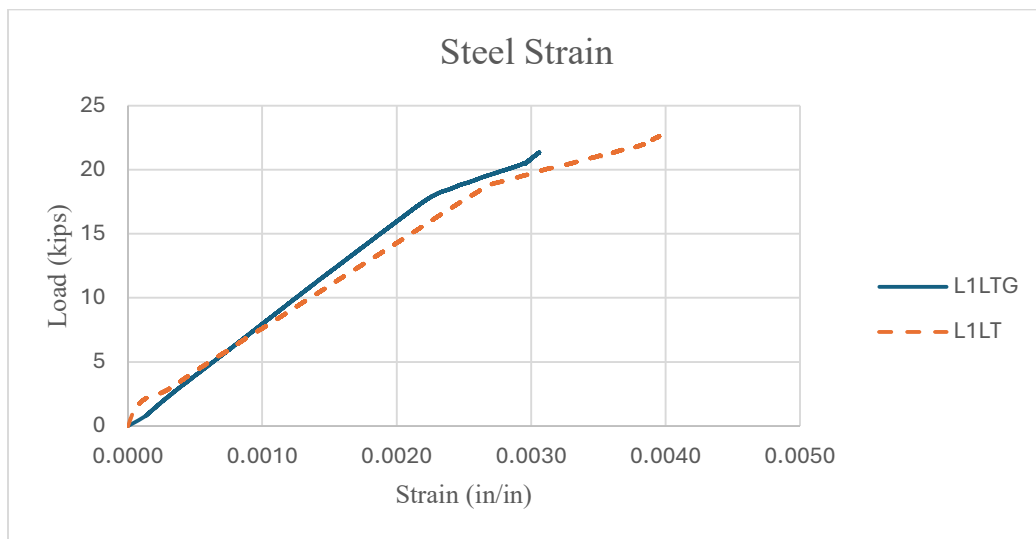


Figure 4.27: Load – steel strain response

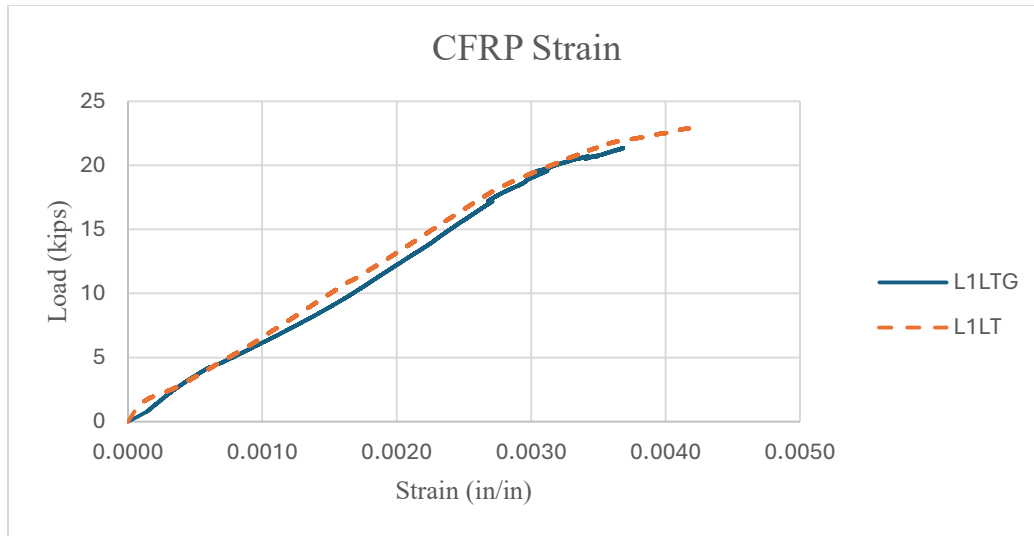


Figure 4.28: Load – FRP strain response

4.2.3.6 Load – Deflection Performance of Low-Strength Series

The load – displacement responses of the low-strength beam series are presented in Figure 4.29. The control beam L0LT exhibited the lowest stiffness and ultimate load, reaching 12.74 kips at a mid-span deflection of 3.38 in, followed by significant post-yield deformation. The CFRP – strengthened beams demonstrated substantially improved load capacity and reduced deflection due to increased flexural stiffness. L2LT showed the highest load-carrying capacity, reaching 28.68 kips, with corresponding deflection of 1.53 in at failure. However, increasing the CFRP layers further in L3LT did not result in additional strength gain, as the beam failed at 27.3 kips with a reduced displacement of 1.19 in. The hybrid-strengthened beam L1LTG, combining NSM strips with one EB-CFRP layer, failed by FRP rupture at 21.36 kips and 1.7 in deflection, exhibited improved performance over the control specimen. However, its enhancement was less pronounced than that of the single-layer EB-strengthened beam (L1LT), indicating that the hybrid configuration was not as effective in this low-strength concrete series.

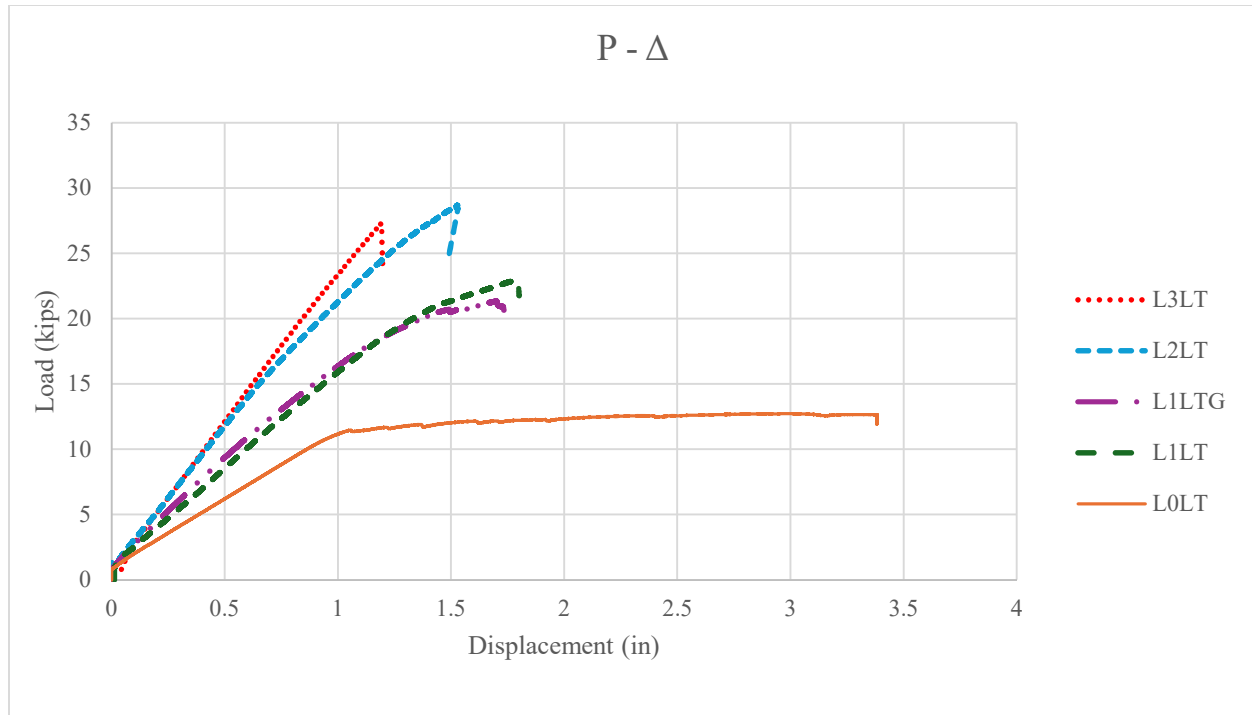


Figure 4.29: Load – Displacement response in low-strength series

4.2.4 Normal Strength Series

The geometry of the specimens in the normal-strength series was illustrated in Figure 4.2. The concrete reached an average compressive strength of 4.44 ksi at 28 days. The longitudinal #5 reinforcing bars had a modulus of elasticity of 29000 ksi and a yield strength of 79.8 ksi, while the #3 bars used for compression reinforcement and stirrups exhibited a modulus of elasticity of 29000 ksi and a yield strength of 70.3 ksi according the manufacturer.

4.2.4.1 Control Specimen (H0LT)

The H0LT specimen served as the control beam for the normal-strength concrete series. The beam exhibited an initial linear response up to the onset of yielding in the tensile reinforcement, which occurred at an experimental load of approximately 11.9 kips based on the steel strain measurements (Figure 4-33). The experimental ultimate load was 13.23 kips with a

corresponding mid-span deflection of 3.54 in, and failure was governed by concrete crushing in the compression zone. In comparison, the numerical analysis predicted a failure load of 13.41 kips and a mid-span deflection of 4.78 in. The concrete compressive strain at failure approached the ultimate strain limit, with a recorded value to 0.0018 (Figure 4.32). The failure mode was concrete crushing.

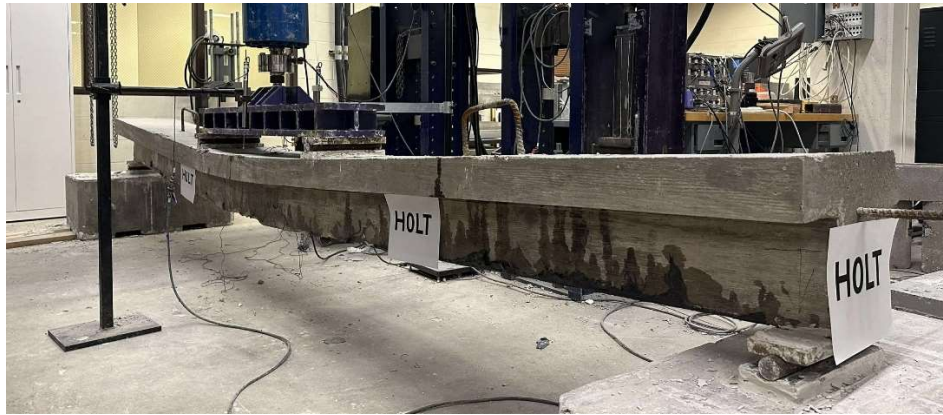


Figure 4.30: HOLT beam after testing

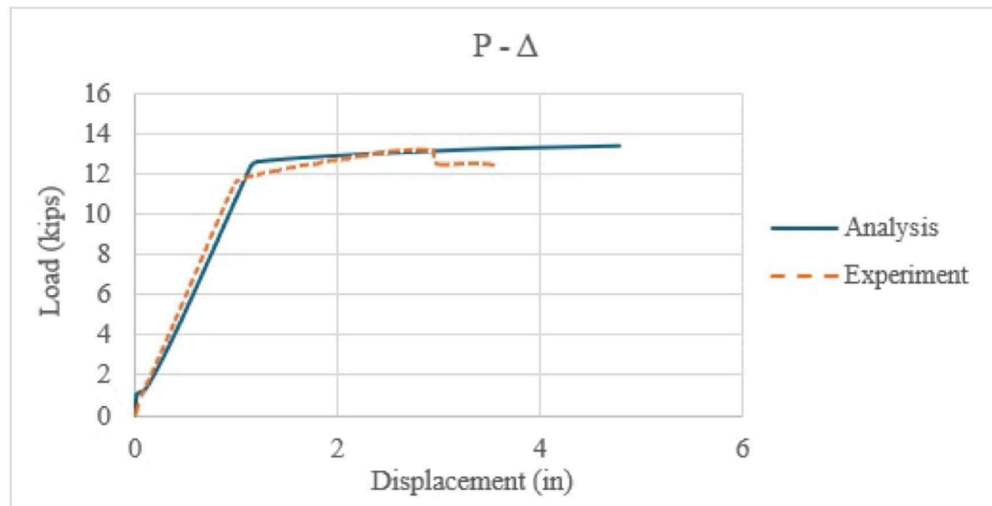


Figure 4.31: Load– displacement response (HOLT)

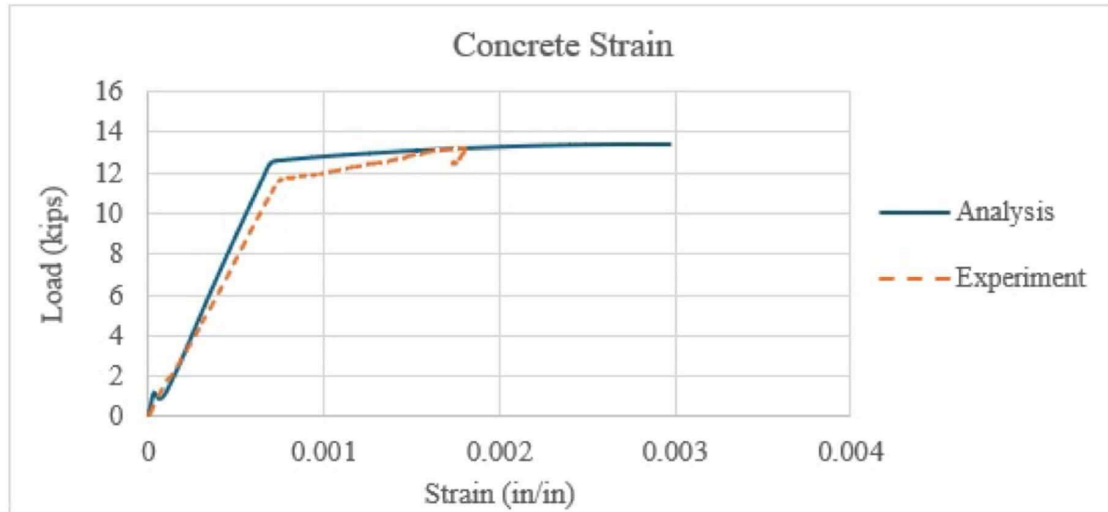


Figure 4.32: Load – concrete strain response (H0LT)

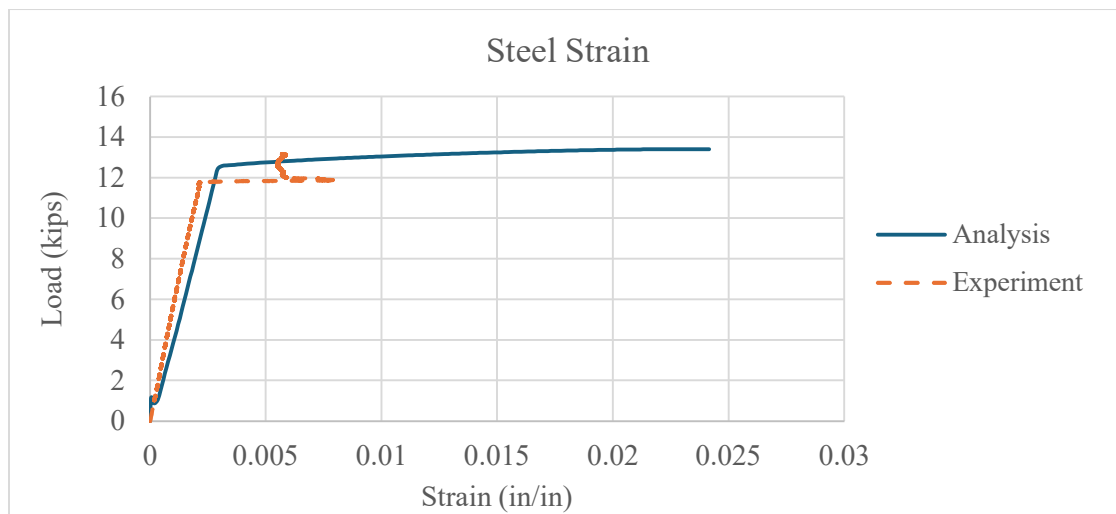


Figure 4.33: Load – steel strain response (H0LT)

4.2.4.2 Specimen with One Layer of CFRP (H1LT)

The H1LT specimen, strengthened with a single layer of externally bonded CFRP sheet. The failure mode of this specimen was FRP debonding. The beam was failed at load of 24.39 kips, the H1LT beam exhibited a significant improvement in flexural capacity, an increase of approximately 84%, compared to the control specimen (H0LT). The mid-span deflection at

failure was 1.92 in, lower than that of H0LT (4.78 in), indicating increased stiffness due to CFRP strengthening. The numerical analysis predicted a failure load of 24 kips and deflection of 1.75 in, with the predicted failure load showing very close agreement with the experimental result (Figure 4.35). The maximum recorded FRP strain was 0.0064, which was slightly higher than the analytical prediction of 0.0052 (Figure 4.38). The comparison between the theoretical and experimental results of load-strain in concrete and steel were shown in Figures 4.36 and 4.37 respectively.



Figure 4.34: H1LT beam after testing

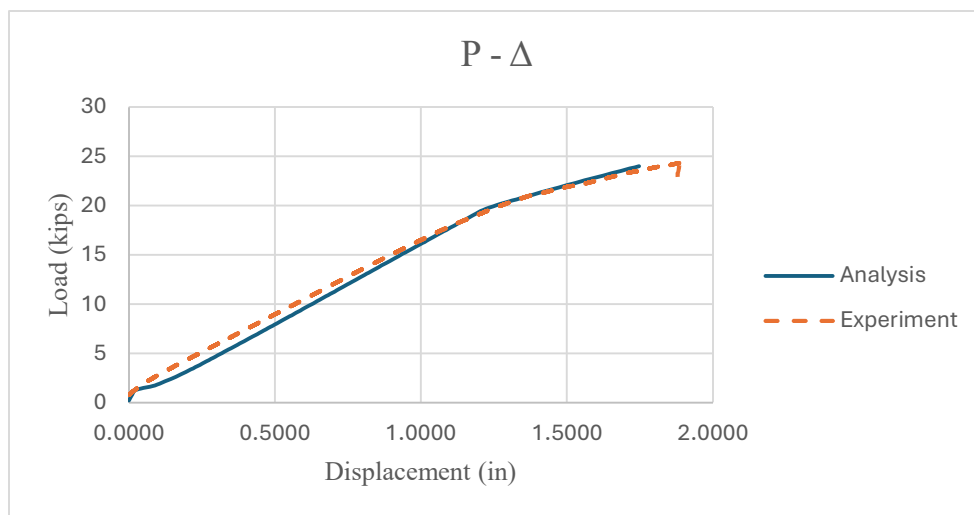


Figure 4.35: Load– displacement response (H1LT)

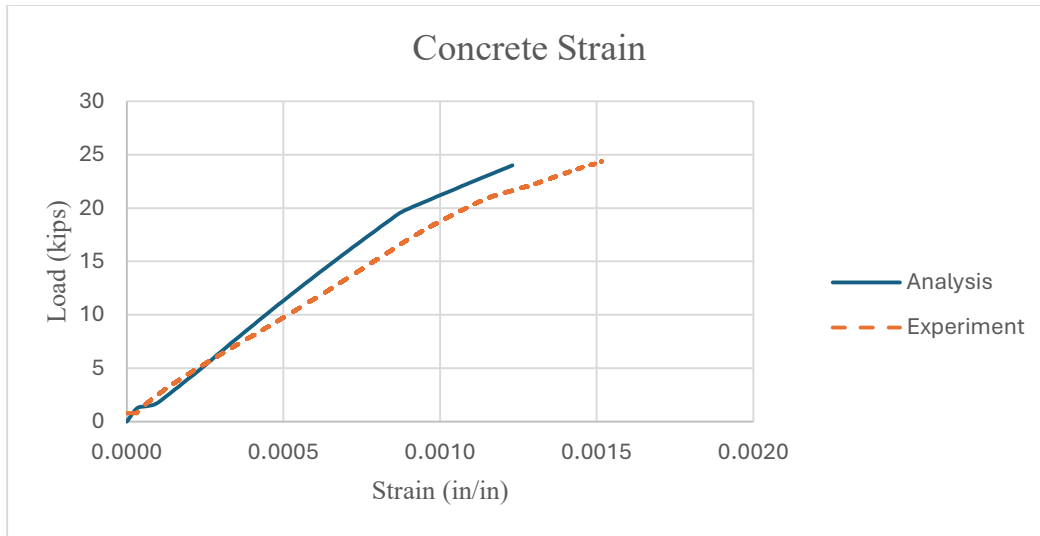


Figure 4.36: Load – concrete strain response (H1LT)

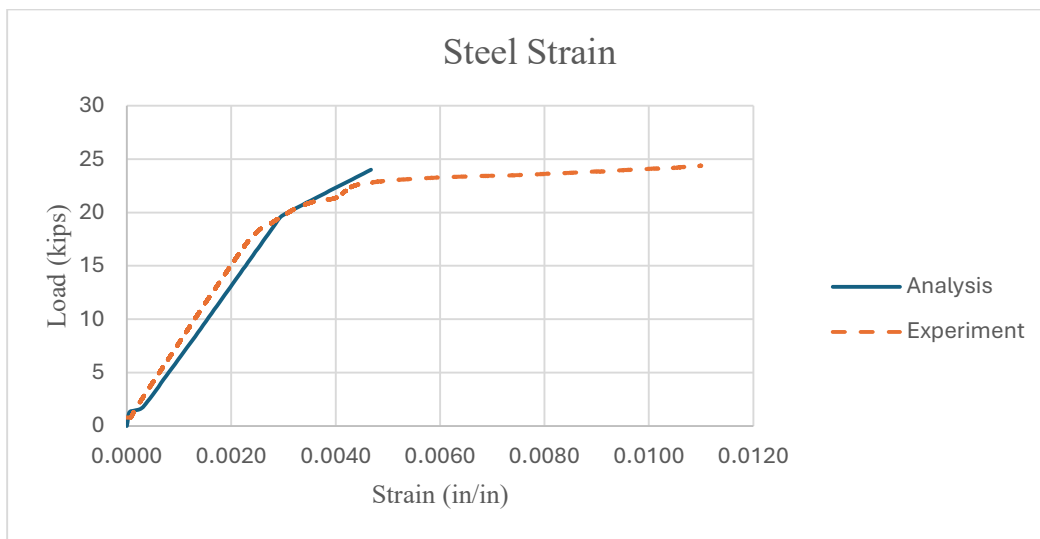


Figure 4.37: Load – steel strain response (H1LT)

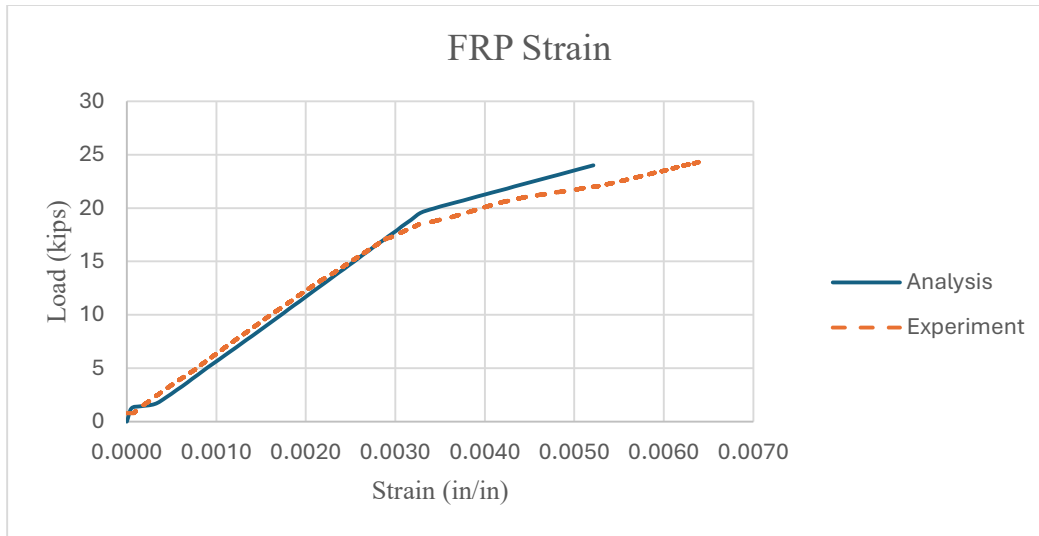


Figure 4.38: Load – FRP strain response (H1LT)

4.2.4.3 Specimen with Two Layer of CFRP (H2LT)

The H2LT specimen, strengthened with two layers of externally bonded CFRP sheets. The failure mode of this specimen was FRP debonding. The specimen reached an experimental failure load of 32.79 kips, which showed a significant improvement of 148% over H0LT (13.23 kips). The mid-span deflection at failure was 1.95 in, which is lower than H0LT (3.54 in) but slightly higher than H1LT (1.9 in), indicating increased stiffness due to CFRP strengthening. The numerical analysis predicted a failure load of 28.4 kips with a corresponding deflection of 1.38 in. The maximum CFRP strain recorded experimentally was 0.0039, which was very close to the analytical prediction of 0.0038 (Figure 4.41). The comparison between the theoretical and experimental results of load-strain in concrete and steel were shown in Figures 4.41 and 4.42 respectively.



Figure 4.39: H2LT beam after testing

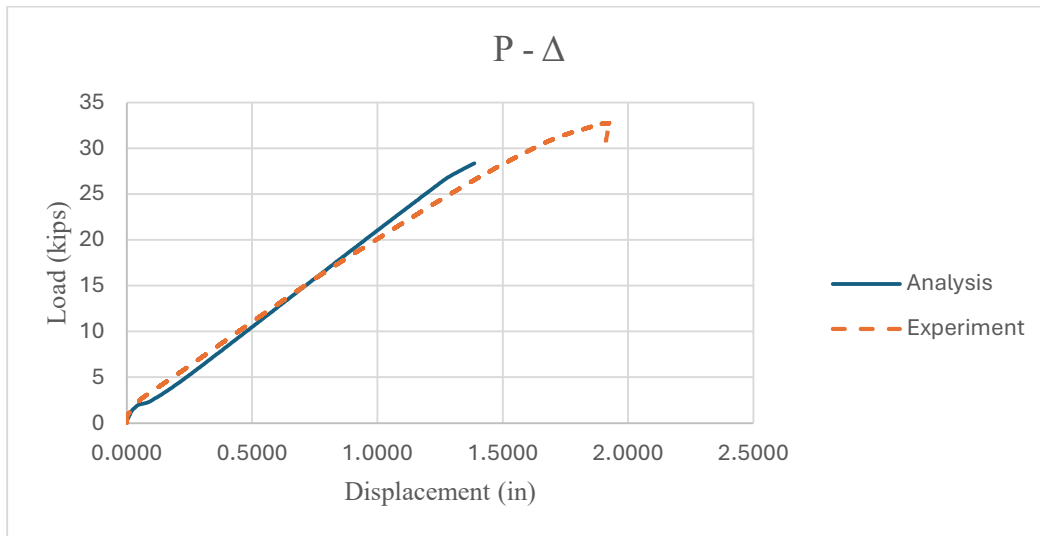


Figure 4.40: Load– displacement response (H2LT)

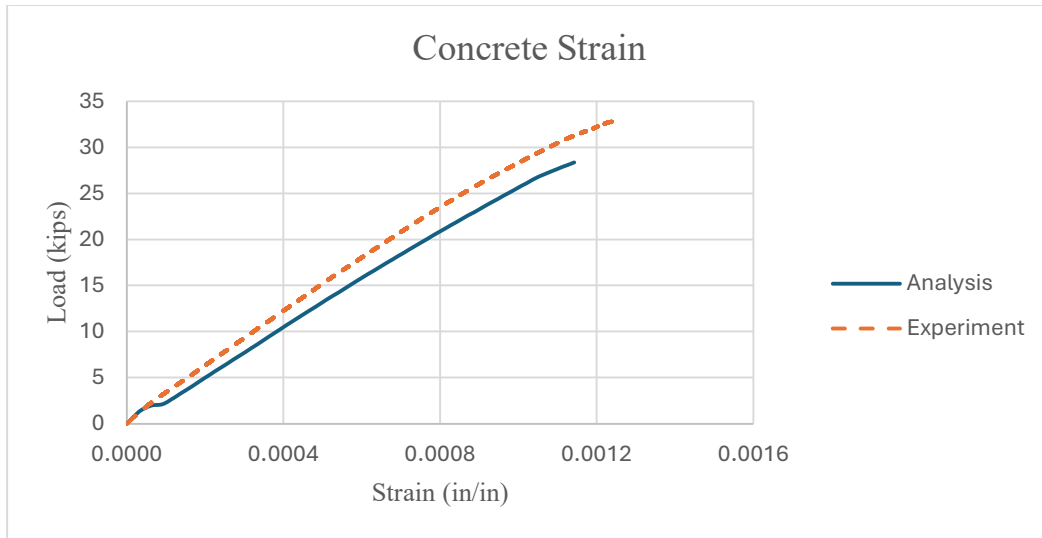


Figure 4.41: Load – concrete strain response (H2LT)

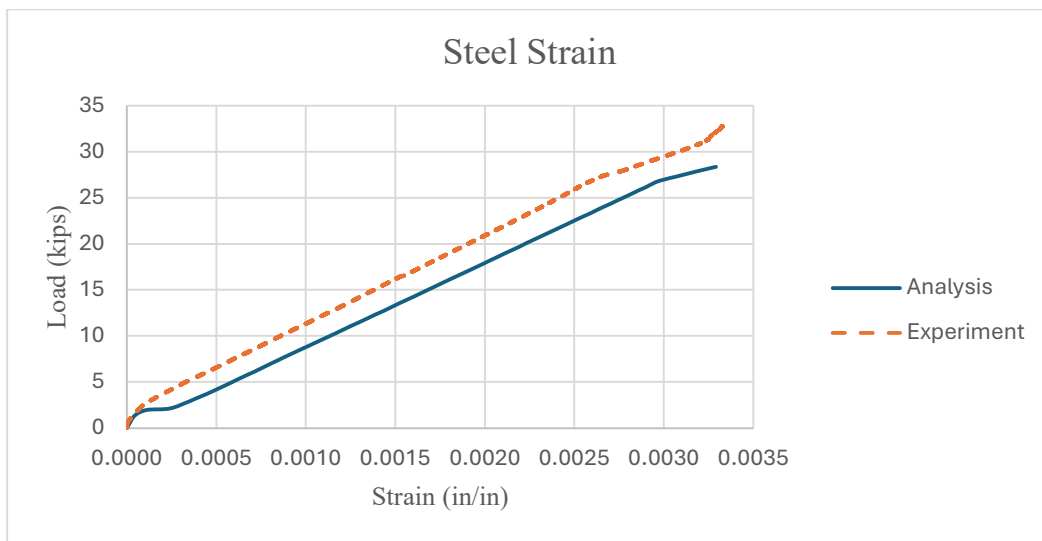


Figure 4.42: Load – steel strain response (H2LT)

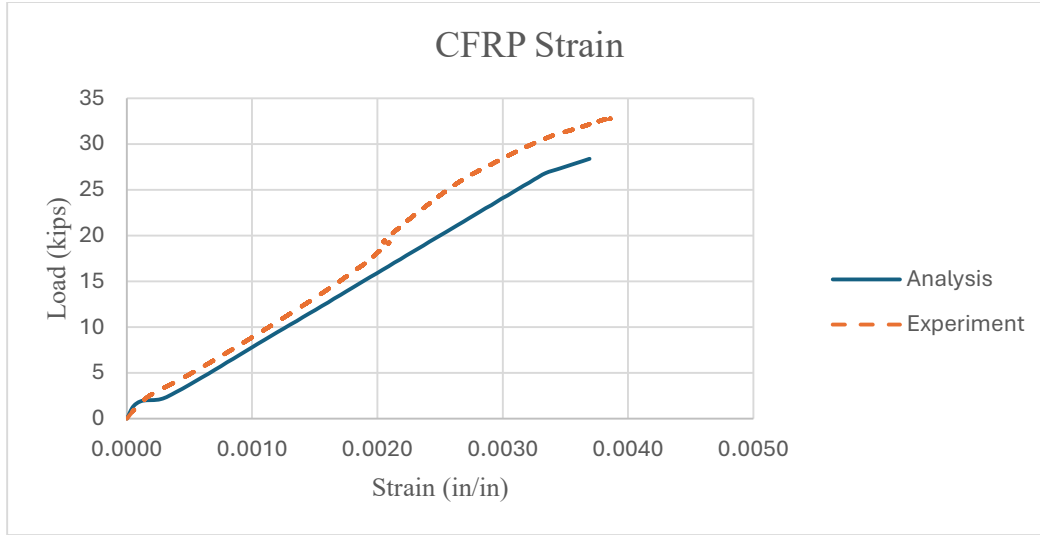


Figure 4.43: Load – FRP strain response (H2LT)

4.2.4.4 Specimen with Three Layer of CFRP (H3LT)

The H3LT specimen, strengthened with three layers of externally bonded CFRP sheets, was tested under four-point bending and failed by FRP delamination. The experimental ultimate load was 28.35 kips, representing an increase of approximately 114% compared to the control beam H0LT (13.23 kips). However, this capacity was lower than that of H2LT (32.79 kips), indicating that increasing CFRP layers beyond two did not enhance strength due to premature interfacial stresses leading to delamination. The mid-span deflection at failure was 1.26 in, significantly lower than H0LT (3.54 in) and also lower than H1LT (1.9 in) and H2LT (1.95 in), indicating increased stiffness but reduced ductility. The numerical analysis predicted a failure load slightly higher than the experimental result, estimating failure at 30.54 kips with a corresponding mid-span deflection of 1.2 in. The maximum recorded CFRP strain was 0.003, showing excellent agreement with the analytical prediction of 0.003. The results showed reasonable agreement between experimental and numerical strain development (Figure 4.48).

The comparison between the theoretical and experimental results of load-strain in concrete and steel were shown in Figures 4.46 and 4.47 respectively.



Figure 4.44: H3LT beam after testing

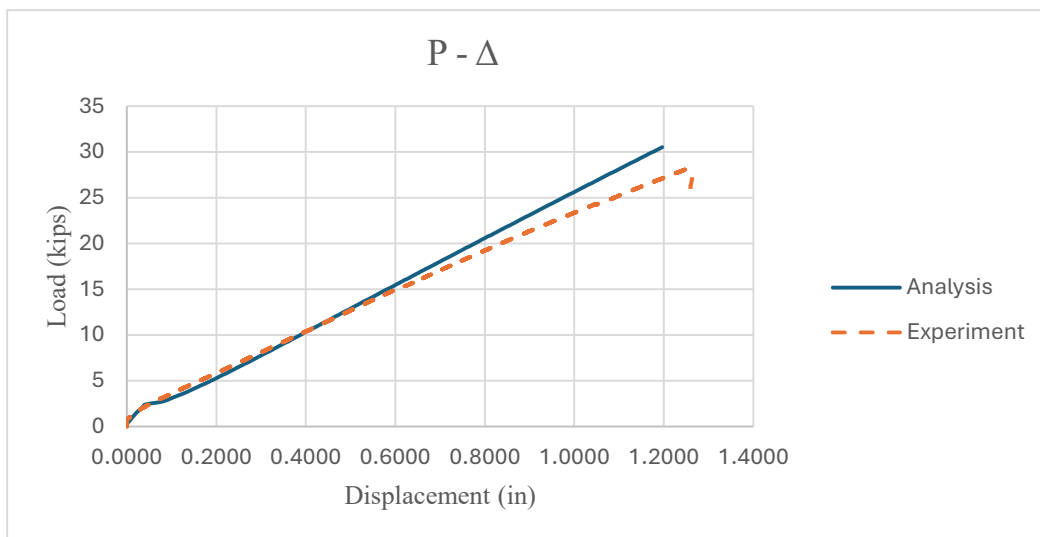


Figure 4.45: Load– displacement response (H3LT)

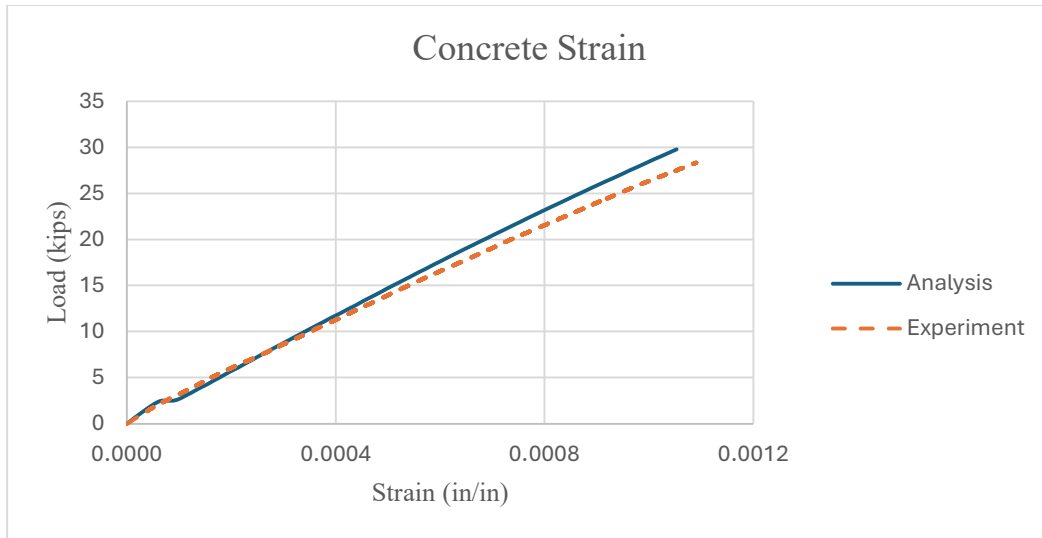


Figure 4.46: Load – concrete strain response (H3LT)

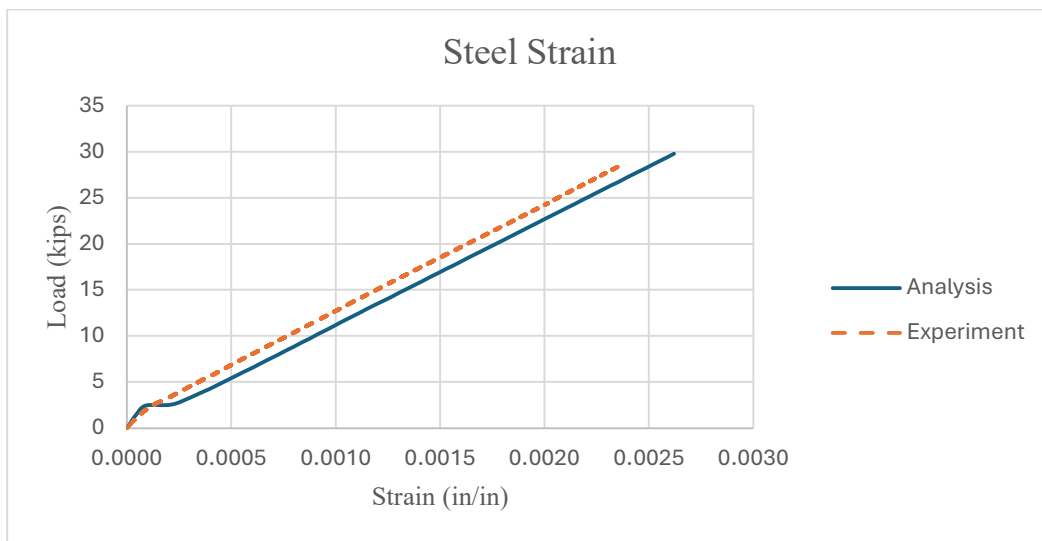


Figure 4.47: Load – steel strain response (H3LT)

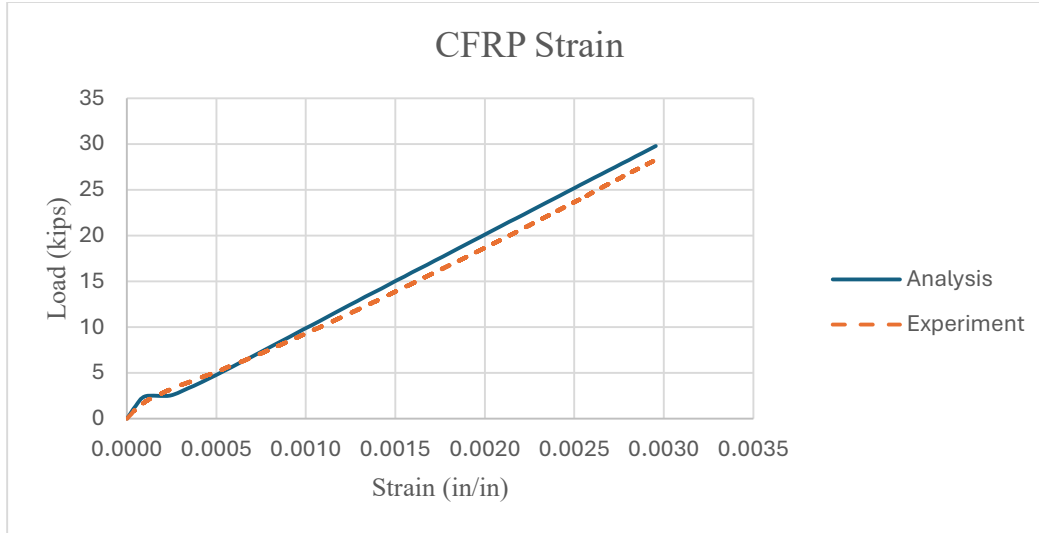


Figure 4.48: Load – FRP strain response (H3LT)

4.2.4.5 Specimen with One Layer of CFRP and Grooving (H1LTG)

The H1LTG specimen was strengthened using a hybrid configuration consisting of two near-surface mounted (NSM) CFRP strips (0.25 in $\times$ 0.75 in) in combination with one layer of externally bonded CFRP (EB-CFRP) on the soffit. The specimen was tested under four-point bending and failed by FRP debonding. The experimental ultimate load was 26.96 kips, representing an increase of approximately 104% compared to the control beam H0LT (13.23 kips) and slightly higher than H1LT (24.39 kips). The mid-span deflection at failure was 2.48 in, lower than H0LT (3.54 in) but greater than H1LT (1.9 in), indicating that the hybrid strengthening system improved deformation capacity and delayed stiffness degradation. The maximum recorded CFRP strain was 0.0051, which was lower than that of H1LT (0.0064) (Figure 4.53). The load – displacement responses of H1LTG and H1LT were very similar, as shown in Figure 4.50, although H1LTG exhibited slightly enhanced ductility due to the contribution of the NSM strips in redistributing tensile stresses prior to failure. The comparison

between the theoretical and experimental results of load-strain in concrete and steel were shown in Figures 4.51 and 4.52 respectively.



Figure 4.49: H1LT beam after testing

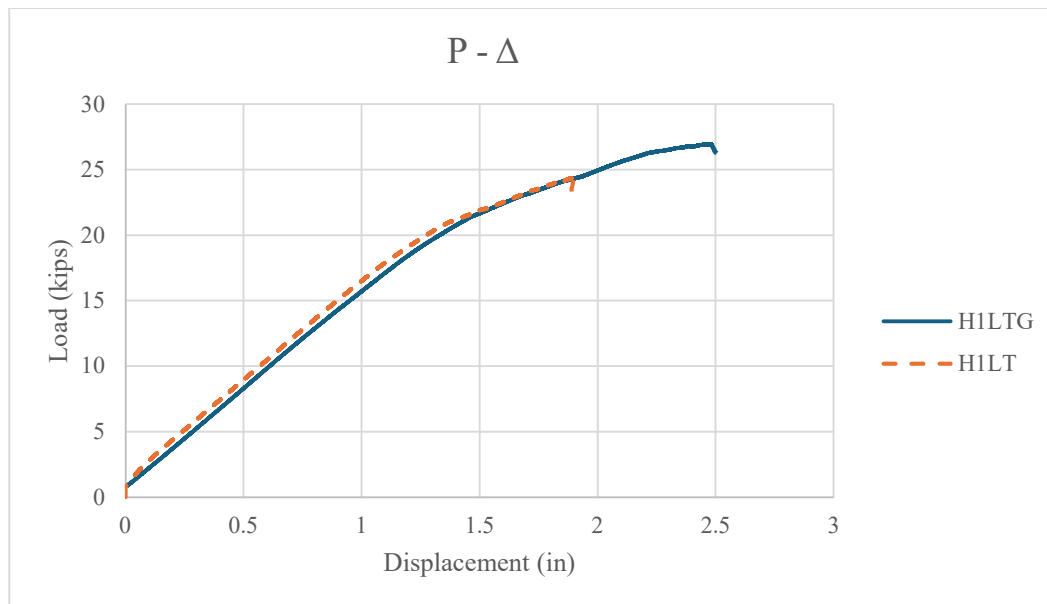


Figure 4.50: Load– displacement response

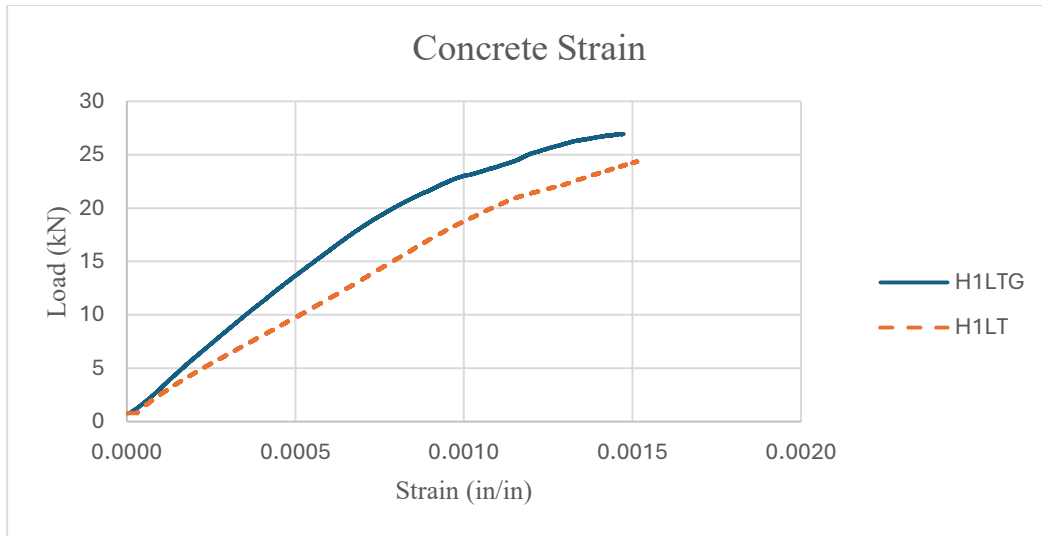


Figure 4.51: Load – concrete strain response

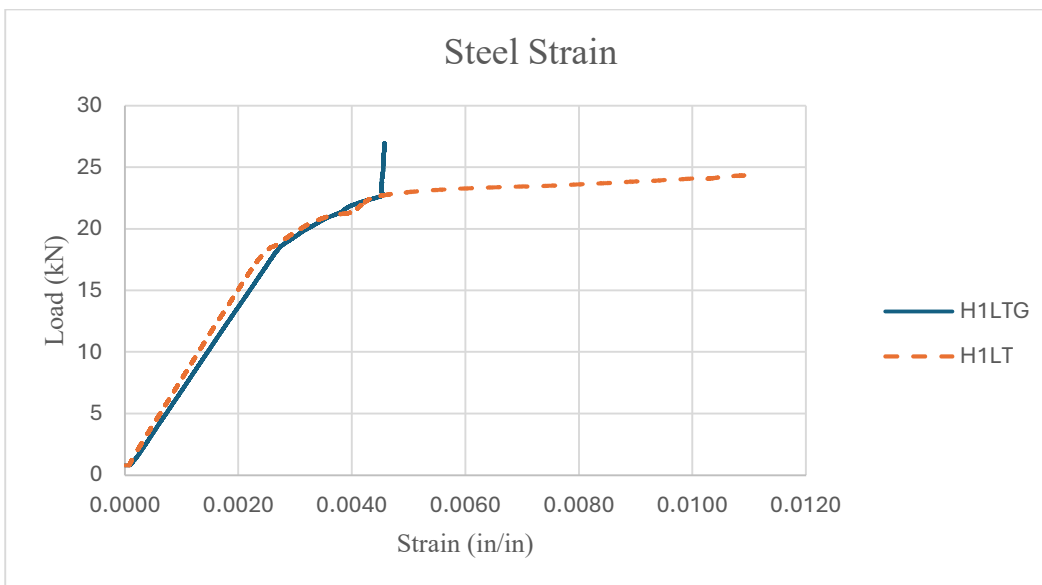


Figure 4.52: Load – steel strain response

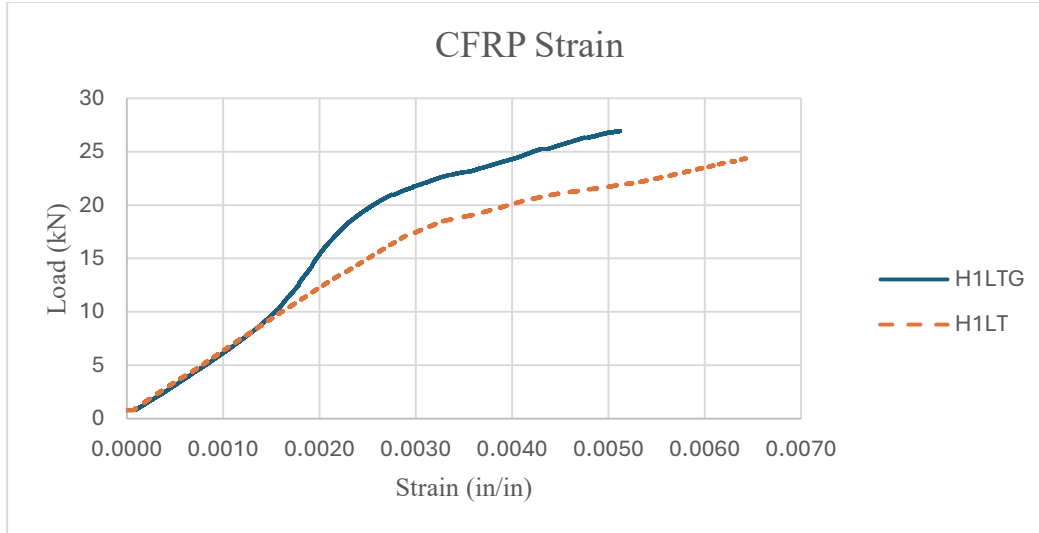


Figure 4.53: Load – FRP strain response

4.2.4.6 Load – Deflection Performance of Normal-Strength Series

The load–displacement responses of the normal-strength beam series are presented in Figure 4.54. The control specimen H0LT exhibited the lowest stiffness and ultimate load, reaching 13.23 kips at a relatively large deflection of 3.54 in before failing by concrete crushing. The CFRP-strengthened beams demonstrated a substantial increase in load capacity and stiffness. The flexural capacity increased progressively with CFRP strengthening, with H1LT and H2LT reaching ultimate loads of 24.39 kips and 32.79 kips, respectively, corresponding to increases of 84% and 148% over H0LT. While H3LT, strengthened with three CFRP layers, reached a lower ultimate load of 28.35 kips, likely due to premature delamination caused by interfacial stress concentration from additional CFRP layering. The hybrid-strengthened beam H1LTG achieved an ultimate load of 26.96 kips with deformation was 2.48 in, demonstrating higher load capacity and greater deformability than the single-layer EB specimen (H1LT). This indicates improved bond efficiency and a more effective utilization of the CFRP reinforcement before failure. As expected, CFRP strengthening reduced ductility as shown by decreasing deflection at failure with

added strengthening. As a result, the H2LT specimen showed the highest load capacity in the series. Overall, the flexural response of the normal-strength series confirms that CFRP strengthening significantly improved stiffness and load capacity, but bond performance controlled the ultimate behavior.

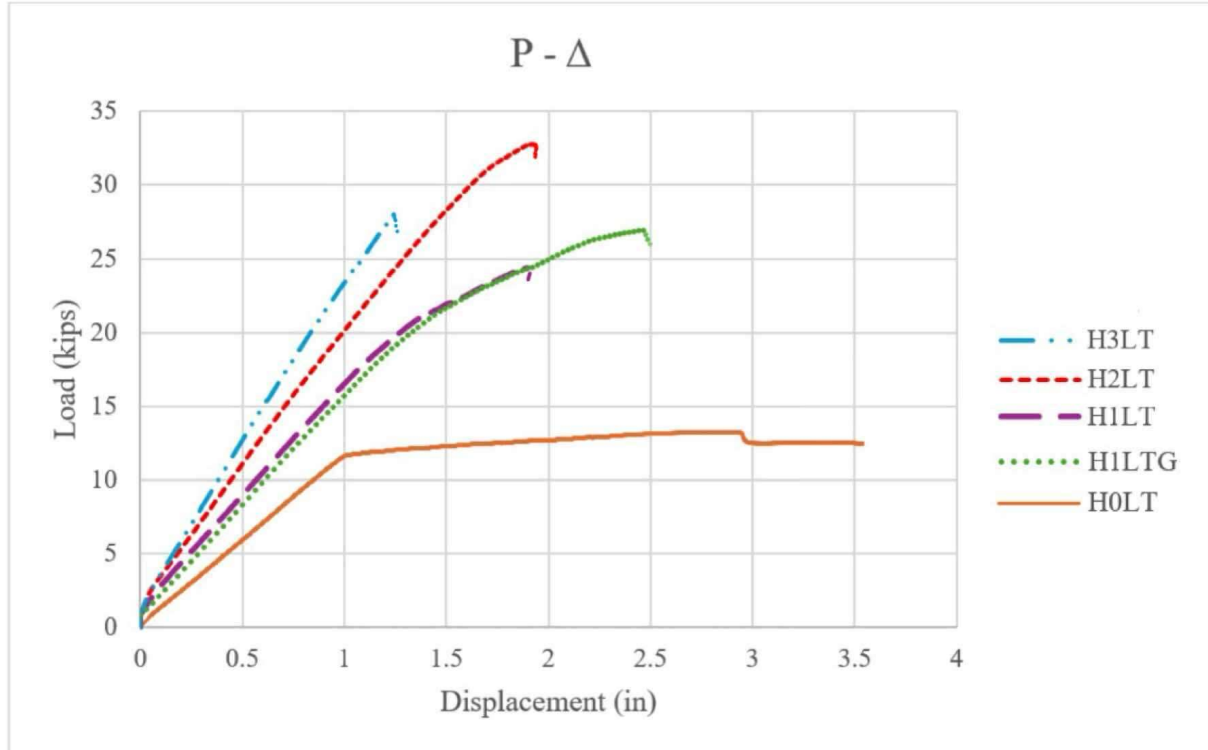


Figure 4.54: Load – Displacement response in normal-strength series

4.3 Comparison of Debonding Strain in Various Design Codes

In order to better interpret the experimental and analytical findings, it is essential to compare the predicted debonding strain limits against the provisions of established international design codes. Several guidelines have proposed different approaches for evaluating FRP debonding in reinforced concrete members, including ACI 440.2R-23, TR 55-2012 (UK), CNR-DT 200 (Italy), fib Bulletin 14-2001, and JSCE (Japan). Each of these codes provides specific

formulations and assumptions that reflect regional practices, safety factors, and experimental backgrounds. Those equations were summarized in Table 4.1

Design Guideline/Model	Strain Limit Causes IC debonding of FRP
ACI 440.2R-23	$\varepsilon_{fd} = 0.41 \sqrt{\frac{\varepsilon'_c}{n E_f t_f}} \leq 0.9 \varepsilon_{fu}$
TR 55-2012 (UK)	$\varepsilon_{fd} = 0.5 \frac{\varepsilon_{fk}}{Y_{FRP, mE}} \leq 0.008$ $Y_{FRP, mE} = Y_{FRP, s} Y_{FRP, m}$
CNR-DT 200 (Italy)	$f_{fd,2} = \frac{k_a}{\gamma_{fd}} \sqrt{\frac{E_f}{t_f} \cdot \frac{2k_b k_G}{FC} \sqrt{f_{cm} f_{ctm}}}$ $\varepsilon_{fd} = \frac{f_{fd,2}}{E_f} \geq \varepsilon_{sy} - \varepsilon_o$ $\varepsilon_{fd} = \alpha c_1 k_c k_b \sqrt{\frac{f_{ct}}{n E_f t_f}}, \text{ if } l_b \geq l_{b,max}$ $\varepsilon_{fd} = \alpha c_1 k_c k_b \sqrt{\frac{f_{ct}}{n E_f t_f}} \frac{l_b}{l_{b,max}} \left(2 - \frac{l_b}{l_{b,max}} \right), \text{ if } l_b \leq l_{b,max}$
fib Bulletin 14-2001	$l_{b,max} = 1.06 \sqrt{\frac{n E_f t_f}{c_2 f_{ct}}}$ $k_b = 1.06 \sqrt{\frac{2 - \frac{b_f}{b}}{1 + \frac{b_f}{400}}}$
JSCE (Japan)	$\varepsilon_{fd} = \sqrt{\frac{2G_f}{n E_f t_f}}$

Table 4.1: Equations of debonding strain in different codes

Table 4.2 compared the experimental FRP debonding strain (ϵ_{fd}) with predictions from five major international design guidelines: ACI 440.2R, TR55, CNR-DT 200, fib Bulletin 14, and JSCE. For the low-strength series, both ACI 440.2R provided most accurate predictions relative to the experimental values, with differences of 0–31%. In contrast, TR55, CNR-DT 200, fib-14, and JSCE significantly overestimated the debonding strain by 39–116%.

A similar pattern was observed in the normal-strength series, where ACI 440.2R again produced the most conservative estimates (0–19% difference), followed by TR55, which remained moderately conservative (–5% to –48%). Meanwhile, TR55, CNR-DT 200, fib Bulletin 14, and JSCE models significantly underestimated the experimental strains, in many cases by 40% to over 100%. Overall, these results indicate that ACI 440.2R provides the most reliable prediction for CFRP debonding strain in both low- and normal-strength concrete beams and therefore be adopted for analytical validation in this study.

	Beam	Debonding strain (ϵ_{fd})						Difference %				
		Experimental	ACI 440.2	TR 55	CNR -DT 200	fib-1	JSCE	ACI 440-2	TR 55	CNR -DT 200	fib-1	JSCE
Low-Strength Series	L0LT	-	-	-	-	-	-	-	-	-	-	-
	L1LT	0.0042	0.0042	0.0080	0.0024	0.0017	0.0022	0%	-90%	43%	59%	47%
	L2LT	0.0039	0.0030	0.0080	0.0017	0.0012	0.0016	24%	-90%	60%	71%	62%
	L3LT	0.0035	0.0024	0.0080	0.0014	0.0010	0.0013	31%	-90%	67%	76%	69%
	L1LTG	0.0037	0.0042	0.0080	0.0024	0.0017	0.0022	-14%	-116%	36%	53%	39%
Normal-Strength Series	H0LT	-	-	-	-	-	-	-	-	-	-	-
	H1LT	0.0064	0.0052	0.0080	0.0025	0.0020	0.0022	19%	-90%	41%	53%	47%
	H2LT	0.0039	0.0038	0.0080	0.0017	0.0012	0.0016	3%	-90%	59%	71%	62%
	H3LT	0.0030	0.0030	0.0080	0.0013	0.0010	0.0013	0%	-90%	70%	76%	69%
	H1LTG	0.0051	0.0052	0.0080	0.0025	0.0020	0.0022	-2%	-57%	52%	61%	56%

Table 4.2: Comparing debonding strain in different codes with experimental results

Table 4.3 compared the experimental ultimate loads with the corresponding numerical predictions for both the low-strength and normal-strength series. For the low-strength series, the analytical results showed close agreement with the experimental values, particularly for L0LT (-

0.5% difference) and L1LT (7% difference). The difference increased for L3LT (15%) and was highest for L2LT (21%). The L1LTG specimen showed a slight difference of 1%, where the analytical model predicted very close to the experiment.

Specimen		Load (kips)		Difference %
		Experiment	Numerical Analysis	
Low Strength	L0LT	12.74	12.80	-0.5%
	L1LT	22.91	21.20	7%
	L2LT	28.68	22.60	21%
	L3LT	27.30	23.32	15%
	L1LTG	21.36	21.20	1%
Normal Strength	H0LT	13.23	13.41	-1%
	H1LT	24.39	24.00	2%
	H2LT	32.79	28.40	13%
	H3LT	28.35	30.54	-8%
	H1LTG	26.96	24.00	11%

Table 4.3: Comparing failure load between experimental and numerical results.

For the normal-strength series, a similar trend was observed, with the closest match for H1LT (2%), followed by H0LT (-1%) and H2LT (13%). Larger deviations were observed for H3LT (-8%) and H1LTG (11%). Overall, the numerical analysis provided results that were in reasonable agreement with the experimental data across both concrete strength levels.

Chapter 5 – Conclusions and Recommendations

5.1 Summary

This study investigated the flexural behavior and debonding strain characteristics of reinforced concrete (RC) T-beams strengthened with thick carbon fiber reinforced polymer (CFRP) sheets (V-Wrap C400HM). A total of ten beams were tested under four-point bending with displacement-controlled loading until FRP failure at a constant rate 0.05 in/min. The specimens were divided into two series based on concrete compressive strength: low-strength (2.86 ksi) and normal-strength (4.44 ksi) concrete. Each series included one un-strengthened control beam, three beams strengthened with one, two, and three layers of externally bonded CFRP sheets, and one hybrid beam with two NSM grooves filled with epoxy putty prior to the installation of one CFRP layer. The research quantified the effect of CFRP thickness, and concrete strength on flexural strength, ductility, and debonding strain. Experimental findings were compared with a numerical model, and debonding strain predictions were evaluated against equations from existing design guidelines. A summary of the results is presented in Table 5.1.

Series	Specimen	Failure Load (kip)	Deflection (in)	Debonding Strain	Increase in Strength (%)	Failure Mode
Low Strength	L0LT	12.74	3.38	-	0%	D
	L1LT	22.91	1.78	0.0042	80%	A
	L2LT	28.68	1.53	0.0039	125%	A
	L3LT	27.30	1.19	0.0035	114%	C
	L1LTG	21.36	1.70	0.0037	68%	B
Normal Strength	H0LT	13.23	3.54	-	0%	D
	H1LT	24.39	1.90	0.0064	84%	A
	H2LT	32.79	1.95	0.0039	148%	A
	H3LT	28.35	1.26	0.0030	114%	C
	H1LTG	26.96	2.48	0.0051	104%	A
A: FRP debonding B: FRP rupture C: FRP delamination D: Concrete crushing						

Table 5.1: Summary of the experiment results for all specimens

5.2 Conclusions

For low-strength series, the results demonstrated that externally bonded thick CFRP sheets significantly enhanced flexural capacity compared to the control beam (L0LT). The beam strengthened with two CFRP layers (L2LT) demonstrated the most efficient strengthening performance, achieving a 125% increase in failure load (29.68 kips) over the un-strengthened beam. The three-layer specimen L3LT reached a failure load of 27.3 kips, showing improvement in flexural strength over both the control beam (L0LT) and the single-layer specimen (L1LT). However, its capacity remained lower than that of the two-layer configuration L2LT (29.68 kips), indicating that the incremental effectiveness of CFRP layering diminished beyond two layers. This reduction in efficiency is attributed to premature debonding and limited strain development caused by elevated interfacial stresses associated with increased CFRP thickness. The hybrid beam (L1LTG), which incorporated two longitudinal grooves filled with epoxy putty, achieved a 68% strength increase, only slightly lower than the single-layer beam (L1LT, +80%). This indicates that surface modification alone did not significantly enhance bond behavior in low-strength concrete, likely due to insufficient concrete substrate strength to resist interfacial stresses. Additionally, ductility decreased progressively with additional CFRP layers due to increased stiffness, although L1LTG showed slightly improved deformation capacity compared to L1LT, indicating improved bond stability from the grooved substrate.

A similar trend was observed in the normal-strength concrete series, where CFRP strengthening significantly increased load capacity. The two-layer CFRP beam (H2LT) again showed the highest strengthening efficiency, reaching a failure load of 32.79 kips (+148%), confirming that two layers of thick CFRP offered optimal performance for using thick sheet. Similar to the low-strength series, the three-layer specimen H3LT reached a failure load of 28.35

kips (+114%), which represented a significant improvement compared to the control beam, but its performance remained lower than that of the two-layer specimen H2LT (32.79 kips). This result again indicated that adding a third layer of thick CFRP sheet (V-Wrap C400HM) did not enhance flexural capacity and instead reduced strengthening efficiency. For thick CFRP sheets such as V-Wrap C400HM, the increased laminate thickness amplified interfacial shear stresses and induced significant shear transfer between CFRP layers. This condition promoted premature debonding and FRP delamination, which limited strain development and prevented full utilization of the additional CFRP material. In contrast to the low-strength series, the hybrid beam in the normal-strength group (H1LTG) demonstrated stronger performance compared to the single-layer specimen (H1LT), achieving a 104% increase in flexural capacity relative to the control beam, compared to 84% for H1LT. Moreover, H1LTG also achieve greater deformability than H0LT. This result showed that the modified bond interface created by the epoxy-filled grooves enhanced stress transfer and improved bond efficiency when combined with normal-strength concrete. The stiffer and stronger concrete substrate in this series provided better confinement and resistance to interfacial shear stresses, which delayed debonding initiation. This is reflected in the greater CFRP strain development observed in H1LT (0.0064) compared to its low-strength counterpart L1LT (0.0042), confirming that concrete compressive strength plays a significant role in bond behavior and CFRP strain utilization.

Overall, the experimental results demonstrate a critical relationship between the number of CFRP layers, failure mode, and structural efficiency. Beams strengthened with two thick layers of CFRP in both series failed by debonding from the concrete substrate and achieved a highest ultimate load capacity. Conversely, the addition of a third, thick layer resulted in a reduction of the ultimate load capacity due to a shift in the governing failure mechanism to

premature delamination within the internal plies of the composite material itself. This indicates that while strengthening generally enhances capacity, there is an optimum number of layers beyond which efficiency decreases and a more brittle, less predictable failure mode (delamination) occurs.

The comparative evaluation of FRP debonding strain prediction models demonstrates clear variability in their ability to represent the experimental behavior of CFRP-strengthened RC beams. The ACI PRC 440.2-23 provision exhibited the closest correlation to the measured debonding strains, with deviations generally within a moderate range for both low- and normal-strength concrete specimens. This consistency indicates that the ACI formulation captures the governing bond-failure mechanisms with a reasonable level of conservatism. Conversely, the TR55, CNR-DT 200, fib Bulletin 14, and JSCE models significantly underestimated the experimental strains, in many cases by 40% to over 100%, reflecting substantially more conservative assumptions regarding interfacial bond capacity and failure initiation. The results suggest that while all international guidelines aim to provide safe design limits, the non-ACI models may lead to overly restrictive strain predictions and sub-optimal utilization of CFRP reinforcement in flexural strengthening applications. Overall, the findings support the suitability of the ACI 440.2-23 approach for practical design, particularly for systems with proper surface preparation and anchorage provisions.

5.4 Recommendations

Based on the findings and limitations of this study, the following recommendations are proposed for future work:

1. Investigate anchorage techniques for thick CFRP systems

Since premature debonding limited strain utilization beyond two layers, future research should explore the use of mechanical anchors, such as fan anchors to improve bond performance and delay debonding failure.

2. Investigate additional hybrid interface configurations

While this study examined a grooved–putty configuration, future work may evaluate other hybrid bond enhancement techniques, including modified groove spacing, adhesive keying, or fiber-reinforced epoxy fillers.

3. Develop design recommendations for thick CFRP systems

Current design guidelines are calibrated using thin CFRP laminates and underestimate effective strain when thick CFRP is used. Additional data should be compiled to develop revised strain limit equations and design guidelines specifically for thick CFRP strengthening.

4. Increasing factor in FRP strain equation in ACI 440.2 for thick CFRP layers

The experimental results in this study consistently show higher FRP strains at failure than those predicted by the ACI 440.2R debonding expression. Therefore, it is recommended to increase the debonding strain factor in the ACI equation for cases involving thick CFRP laminates to better reflect actual strain utilization and avoid overly conservative designs.

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2.50	CYDS	25-A423JM	2500 PSI AE	2.50	2.50	\$167.00	\$417.50
3.50	ea	SLC	SHORT LOAD CHARGE	3.50	3.50	\$25.00	\$87.50
2.50	/y	WINTER	WINTER SERVICE	0.00		\$7.00	\$17.50
1.00	cy	FUELINV	FUEL SURCHARGE - PER LC	0.00		\$5.00	\$5.00

Water Withheld: 2.00 gl. per c/yd

SUBTOTAL		\$527.50
TAX		
TOTAL		\$527.50
GRAND TOTAL		\$527.50

PLANT	ARRIVE JOB	START UNLOAD	FINISH UNLOAD

PURCHASER OR HIS AUTHORIZED REPRESENTATIVE ACCEPTS RECEIPT OF THIS LOAD OF MATERIAL AND HAS INSTRUCTED THE DRIVER TO ADD WATER AS FOLLOWS: yds gal yds gal yds gal yds gal

CAUTION! Cement powder or freshly mixed concrete, grout or mortar may cause skin injury. Avoid contact with skin and wash exposed areas promptly with water. If any cement powder or mixture gets into the eyes, rinse immediately and repeatedly with water, and get prompt medical attention. Keep children away from all freshly mixed cement products. MSDS on reverse side.

ANY WATER ADDED TO MIX ON THE JOB WILL BE AT THE PURCHASER'S RISK

DELIVERY AGREEMENT: This purchase agreement is for delivery to the curb line. In the event Purchaser requests delivery beyond the curb line, the Purchaser hereby assumes liability for damages to sidewalks, driveways all other property, and personal injury. Purchaser hereby agrees to indemnify and hold the Seller harmless against all liability, loss or expense incurred as the result of such delivery. If ground conditions beyond the curb line are inadequate for heavy truck travel, and as a result of this condition the truck requires towing, towing will be at the customer's expense.

PRODUCT PERFORMANCE: All concrete sold on a basis of compressive strength, air-entrainment, slump, temperature, density and or yield must be sampled at the mixer truck discharge in accordance to ASTM C172 and tested according to the applicable ASTM testing standards. Compressive strength: ASTM C31 & ASTM C39; air content: ASTM C231 or ASTM C173; Slump: ASTM C143; Temperature: ASTM C1064; density and or yield: ASTM C138. Slump and air-entrainment variances in accordance to ASTM C94 are applicable. Unless identified in writing, no other condition of sale shall apply. Customer assumes all responsibility for the placement, consolidation, curing, and protection of all concrete after delivery to the curb line. The placement, consolidation, curing, and protection of all concrete can greatly affect its performance. The seller is not responsible for any product performance after placement is made by the customer.

ADDITIONAL INFORMATION: Ambient conditions may produce rapid evaporation of moisture from the concrete surface. As a result, plastic shrinkage cracking may occur. Precautions should be taken. Drying shrinkage is an inherent, unavoidable property of concrete and precautions should be taken to minimize random cracking. Control joints must be properly constructed with reference to location, spacing, depth and timing. Materials hereby sold become property of purchaser at point of origin. The purchaser shall in no event accept deliveries of materials not in accord with the agreement of the parties, but such materials shall be refused by the purchaser and returned to the seller with a written statement of the reason for the refusal thereof. No cancellation accepted after concrete has been batched.

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PRINTED NAME	TITLE	Material	Design Qty	Required	Batched	% Var	% Moisture	Actual	Wat	Trim
		KDOTFAA	1881 lb	4797 lb	4980 lb	> 3.82%	2.00% M	12	gl	
		ASTMC33#5	1260 lb	3156 lb	3160 lb	0.12%	0.20% M	1	gl	
		CEMENT	423.0 lb	1057.5 lb	1080.0 lb	> 2.13%				
		AIR1	2.20 oz #	5.50 oz	7.00 oz	27.27%				
		WATER	30.5 gal	59.2 gal	59.4 gal	0.49%		59.4	gl	-2.00 gl

Note: Added 5gal of water to mix on site.

us: Num Batches: 1

id: 9716 lb Design W/C: 0.601 Water/Cement: 0.589 T Design 76.2 gl Actual 71.9 gl To Add: 4.3 gl

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 ALL DRIVERS MUST WEAR A HARD HAT, SAFETY GLASSES,
 STEEL TOE BOOTS, AND SAFETY VEST ON EVERY JOB!

ORDER # 87 PO# CREDITCARD PROJECT CODE JOB# APPLICATION EXTFW PREVIOUS TRUCK SLUMP 4

QTY.	UNIT	PRODUCT	DESCRIPTION	ORDERED	DELIVERED	UNIT PRICE	AMOUNT
2.50	CYDS	25-A423JM	2500 PSI AE	2.50	2.50	\$167.00	\$417.50
2.50	/y	WINTER	WINTER SERVICE	0.00		\$7.00	\$17.50
1.00	ea	FUELINV	FUEL SURCHARGE - PER LC	0.00		\$5.00	\$5.00

Water Withheld: 2.00 gl. per c/yd

SUBTOTAL \$527.50
 TAX
 TOTAL \$527.50
 GRAND TOTAL \$527.50

RECEIVED IN GOOD CONDITION BY

PRINTED NAME TITLE CUSTOMER SIGNATURE 11/20/24 12:24
 NO WARRANTY IMPLIED OR EXPRESSED ON PRODUCT

Material	Design Qty	Required	Batched	% Var	% Moisture	Actual	Wat	Trim
KDOTFAA	1881 lb	4867 lb	5010 lb	2.94%	3.50% M	20	gl	
ASTMC33#5	1260 lb	3163 lb	3180 lb	0.55%	0.40% M	2	gl	
CEMENT	423.0 lb	1057.5 lb	1225.0 lb	> 15.84%				
WREDUCE1	21.15 oz	52.88 oz	54.00 oz	2.13%				
AIR1	2.50 oz #	6.25 oz	7.00 oz	12.00%				
WATER	30.5 gal	49.9 gal	50.4 gal	1.02%		50.4	gl	-2.00 gl

Retain 2 gal of water not added to the mix

Num Batches: 1
 Design W/C: 0.601 Water/Cement: 0.519 T Design 76.2 gl Actual 72.3 gl To Add: 3.9 gl
 Water in Truck: 0.0 gl Adjust Water: 0.0 gl / Load Trim Water: -2.0 gl / CYL

Appendix B – Steel Material Properties

B.1 Steel Material Properties for #3 Bar

NUCOR **Mill Certification**
07/11/2024

MTR#: 1714853-6
Lot #: 380004829320
3630 Fourth Street
Flowood, MS 39232 US
601 939-1623
Fax: 601 936-6202

Sold To: NUCOR REBAR FABRICATION MIDWEST
LLC
1342 S GRANDSTAFF DR
AUBURN, IN 46706 US

Ship To: NUCOR HARRIS REBAR MIDWEST LLC
500 REBAR RD
SEDALIA, MO 65301 US

Customer PO	PO130723A	Sales Order #	97014703 - 1.1
Product Group	Rebar	Product #	1048262
Grade	A615 Gr 60/AASHTO M31	Lot #	380004829320
Size	#3	Heat #	3800048293
BOL #	BOL-1760214	Load #	1714853
Description	Rebar #3/10mm A615 Gr 60/AASHTO M31 30' 0" [360"] 4001-8000 lbs	Customer Part #	
Production Date	06/01/2024	Qty Shipped LBS	20304
Product Country Of Origin	United States	Qty Shipped EA	1800
Original Item Description		Original Item Number	

I hereby certify that the material described herein has been manufactured in accordance with the specifications and standards listed above and that it satisfies those requirements.

Melt Country of Origin : United States
Roll Country of Origin : United States

Melting Date: 05/15/2024
Rolling Date: 06/01/2024

C (%)	Mn (%)	P (%)	S (%)	Si (%)	Ni (%)	Cr (%)	Mo (%)	Cu (%)	V (%)	Nb (%)
0.38	0.80	0.013	0.022	0.193	0.12	0.20	0.03	0.33	0.023	0.001

Mechanical

	Average Deformation Height (IN)	Bend Test
(1)	0.023	Pass

Tensile testing

	Yield (PSI)	Tensile (PSI)	Elongation in 8" (%)
(1)	70300	98500	12.5

Other Test Results
Weight Percent Variance (%) : -4.26
Tensile / Yield Ratio : 1.40

Comments:
Nucor Steel Jackson, Inc. is in compliance with and certified to: ISO 9001:2015, ISO 14001:2015, ABS Quality Assurance Program & AASHTO's NTPEP Reinforcing Steel and Wire (REBAR) Technical Committee Work Plan. All manufacturing processes of the steel materials in this product, including melting, have occurred within the United States.
Mercury not intentionally added at any point during manufacturing or testing of this material. No weld repair was performed.
Material and Mill Certification are in full compliance with the American Iron and Steel requirement as mandated in EPA's State Revolving Fund Programs and complies with the Buy American Act, (FTA) Buy America Requirements (49 CFR part 661), and (FHWA) Buy America Requirements (23 CFR part 635.410)

Nolan Guess
Nolan Guess, Quality Supervisor

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B.2 Steel Material Properties for #5 Bar



Mill Certification

09/26/2024

Sold To: NUCOR REBAR FABRICATION MIDWEST
LLC
1342 S GRANDSTAFF DR
AUBURN, IN 46706 US

Ship To: NUCOR HARRIS REBAR MIDWEST LLC
500 REBAR RD
SEDALIA, MO 65301 US

MTR# 1771246-2
Lot #: 970003423520
500 REBAR ROAD
SEDALIA, MO 65301 US
660-951-1700
Fax: 660 951-1698

Customer PO	PO131403A	Sales Order #	97015777 - 1.3
Product Group	Rebar	Product #	3019447
Grade	A615 Gr 60/AASHTO M31	Lot #	970003423520
Size	#5	Heat #	9700034235
BOL #	BOL-1822086	Load #	1771246
Description	Rebar #5/16mm A615 Gr 60/AASHTO M31 60' 0" [720"] 6001-10000 lbs	Customer Part #	
Production Date	09/19/2024	Qty Shipped LBS	24030
Product Country Of Origin	United States	Qty Shipped EA	384
Original Item Description		Original Item Number	

I hereby certify that the material described herein has been manufactured in accordance with the specifications and standards listed above and that it satisfies those requirements.

Melt Country of Origin : United States

Melting Date: 09/19/2024

Roll Country of Origin : United States

Rolling Date: 09/19/2024

C (%)	Mn (%)	P (%)	S (%)	Si (%)	Ni (%)	Cr (%)	Mo (%)	V (%)	Nb (%)
0.24	0.77	0.016	0.026	0.234	0.11	0.20	0.04	0.004	0.002

Mechanical

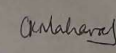
	Average Deformation Height (IN)	Bend Test
(1)	0.042	Pass

Tensile testing

	Yield (PSI)	Tensile (PSI)	Elongation in 8" (%)
(1)	79800	94300	13.1

Comments:

- All manufacturing processes of the steel materials in this product, including melting, casting and rolling were performed in the USA.
- Mercury, Radium, Hexavalent Chrome or Alpha source materials in any form have not been used in the production and testing of this material.
- Weld repair was not performed on this material.
- All spooled and straight rebar products produced by Nucor Steel Sedalia, LLC are compliant with the Buy American Act, and meet the requirements of Federal Highway Administration 23 U.S.C. § 313 – Buy America; 23 C.F.R. § 635.410 and the Buy America restrictions of the Federal Transit Administration (49 U.S.C. § 5323(j)). Material and Mill Certification are in full compliance with the American Iron and Steel requirement as mandated in EPA's State Revolving Fund Programs.


Candace Maharaj, Quality Supervisor

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