

UNIVERSITY OF BAHRAIN

College of Engineering



Department of Civil Engineering

**Behavior of Glass Fiber Reinforced Polymer (GFRP) Reinforced Concrete T-
Beam Under Shear**

A Thesis Submitted in Partial Fulfilment of the Requirements for the M.Sc Degree in Civil
Engineering

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Abstract

Corrosion is one of the significant disadvantages of steel reinforcement, especially when exposed to extreme environments. One of the alternative solutions is Glass Fiber Reinforced Polymer (GFRP), which is known for its high strength-to-weight ratio and good corrosion resistance. However, since its mechanical properties, such as modulus of elasticity, differ from the properties of normal steel, several studies were conducted to investigate the shear behavior of rectangular beams reinforced with GFRP. In contrast, the shear behavior of GFRP-RC T-beams has not yet been fully explored.

This study investigated the shear behavior of seven GFRP-reinforced T-beams without transverse reinforcement. The investigation was conducted in two parts; the first part was carried out experimentally in the University of Bahrain structural engineering lab, while the second part was a Finite Element Analysis (FEA) simulation. In the experimental part, seven T-beams were constructed and tested to failure. All the beams were reinforced with GFRP and had constant cross-section and length. Besides that, all the specimens had no shear or top reinforcement. The seven beams were divided into three groups; one parameter was varied in each group while all other parameters were kept constant. The test parameters were shear span-to-depth ratio (a/d), which included 2.0, 2.7, and 3.5; concrete compressive strength (f_c'), which included approximately 40, 50, and 60 MPa; and longitudinal reinforcement ratio (ρ), which were 0.7, 1.1, and 1.5%. The test results of this study are demonstrated in the form of mode of failure, ultimate strength, load-deflection behavior, and crack pattern. Furthermore, the experimental ultimate strengths were compared to the predicted shear capacity by The American Concrete Institute (ACI 440.1R-15), Japan Society of Civil Engineers (JSCE 1997), Canadian Guideline (CSA S806-12), and ISIS Canada Research Network (ISIS 2007). Additionally, the proposed models by Alam (2010) and Tureyen et al. (2006) were included in the comparison.

The outcomes of the experimental study showed that changing the three parameters has a significant effect on the ultimate load, crack pattern, and load-deflection behavior. At the same time, the mode of failure remained the same. In addition, it was found that the Canadian Guideline (CSA S806-12) and Alam's (2010) equations predict the shear strength of GFRP-RC T-beams with good accuracy and consistency.

On the other hand, the FEA was performed using Abaqus software. Simulated results are compared with experimental results to evaluate the software's accuracy in predicting the shear strength, load-deflection behavior, crack pattern, and failure mode. The modeled beams were simulated using the same mechanical properties used in the experimental study. A concrete damage plasticity model was used to model all seven beams. Two models were developed for each beam, each with a different tension-stiffening model, while all other properties remained unchanged. It was found that the model's predictions of the first cracking load and mode of failure fairly agreed with the experimental results. However, the models did not show accurate results regarding ultimate load and deflection behavior for all the beams. Finally, using two models for each beam showed that the tension-stiffening model significantly affects the model outcomes.

Publications

Dedication

In dedication to my Mother, Father, and Grandmother, may Allah have mercy on her for supporting me all these years and being beside me. I hope this achievement makes you all proud.

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List of Abbreviations:

A_{comp} = Computed flange area based on the concrete compression area using an elastic cracked section analysis considering the entire flange width

A_{req} = Required area obtained based on the reported shear strength at a distance away from the support

A_f : Area of FRP

E_c = Modulus of elasticity of concrete

E_f = FRP modulus of elasticity

E_o = Initial tangent modulus

E_s = Young modulus of steel

G_f = Fracture energy

K_c = Factor related to the minimum flexural stress at extreme compression fiber due to moment just above the cracking moment and concrete compressive strength

K_{exp} = Experimental K

M_d = Design bending moment

M_n = Nominal moment capacity of a beam

M_o = Decompression moment

N'_d = Design axial compressive force

P_{cr-exp} = Experimental first cracking load

P_{cr-M} = FEA first cracking load

V_c = Concrete shear strength

V_{calc} = Shear strength based on theoretical calculations

V_{exp} = Shear strength based on experimental results

V_u = Shear force due to ultimate load

a = Shear span

a' = Concrete compression zone

a/d = Shear span-to-depth-ratio

b : Beam width

b_f = Flange width of beam

b_w = Web width of beam

c = Neutral axis depth

COV = Coefficient of variation

d = Effective depth

d_a = Maximum aggregate size

d_c = Scalar degradation factors for compression

d_t = Scalar degradation factors for tension

f_c = Concrete compressive stress

f'_c = Concrete compressive strength

f'_{cd} = Design compressive strength of concrete

f_f = Tensile strength of FRP at concrete crushing

f_{fu} = Design tensile strength of FRP

f'_t = Concrete tensile strength

k = Stress decay factor

k_a = The effect of arch action

k_m = The effect of the interaction between the factored moment and the factored shear at a section on its shear strength

k_r = The effect of reinforcement rigidity

n_f = Ratio of the modulus of elasticity of FRP bars to the modulus of elasticity of concrete

n_c = Curve fitting factor

t = Flange thickness of beam

w_{cr} = Cracking width after a total release of stress

w_t = Cracking width

σ_t = Concrete tensile stress

σ_m = Minimum flexural stress at extreme compression fiber due to moment just above the cracking moment

ρ = Reinforcement ratio

ρ_b : Balanced FRP ratio

ρ_f : FRP reinforcement ratio

β_1 : Concrete factor f'_c : Concrete compressive strength

ε = Strain

ε_c = Concrete compressive strain

ε_{cu} : Concrete ultimate compressive strain

ε_{cr} = Concrete cracking strain

ε_o = Concrete compressive strain at f'_c

ε_c^{pl} = The compressive equivalent plastic strain

ε_t = The total strain of concrete under tension

ε_t^{pl} = The tensile equivalent plastic strain

γ_b = Strength reduction factor

λ = Factor related to concrete density

ϕ_c = Resistance factor related to concrete

φ = The dilation angle

ACI = American Concrete Institute

BFRP = Basalt Fiber Reinforced Polymer

CFRP = Carbon Fiber Reinforced Polymer

CSA = Canadian Standard Association

FEA = Finite Element Analysis

Fib = Fédération internationale du béton

FRP = Fiber Reinforced Polymer

GFRP = Glass Fiber Reinforced Polymer

ISIS = Intelligent Sensing of Innovative Structure

JSCE = Japan Society of Civil Engineers

Chapter One

Introduction

1.1 General:

Concrete is well known for its low tensile strength. Therefore, steel reinforcement is a significant material in construction because it is highly reliable in resisting tension forces due to its high tensile strength. Besides that, the strong bond achieved between concrete and steel, the acceptable stress variation due to thermal changes, and its high ductility make steel reinforcement favorable for construction projects. However, steel reinforcement is well known for its weakness in resisting corrosion. This weakness leads to property damage and shortens the structures' service life, especially projects subjected to challenging environments, causing high maintenance costs to governments and individual owners. One of the recent alternative solutions of steel bars is Fiber Reinforced Polymer (FRP), notably Glass Fiber Reinforced Polymer (GFRP). GFRP is a corrosion-free material, making it a viable option to replace steel in projects exposed to harsh environments. Besides that, GFRP bars have high tensile strength and are considered electrically non-conductive material. However, GFRP has a low modulus of elasticity and low ductility. All in all, GFRP has been spreading widely around the globe as a reinforcement for concrete structures and can be used as longitudinal and shear reinforcement.

1.2 Problem Statement:

The high tendency of steel corrosion has driven researchers to conduct several studies to find a proper alternative for steel. One of the solutions for preventing corrosion is using FRP. In spite of the fact that steel reinforcement replacement by FRP eliminates the issue, several studies reported that steel design models overestimate the shear strength of FRP beams (Nguyen-Minh and Rovňák 2011). The reason is that these models were developed based on beam specimens reinforced with steel, however, steel and GFRP have different mechanical properties, such as modulus of elasticity, tensile strength, transverse strength, ductility, and stress-strain relationship. Thus, several researchers have attempted to study the behavior of concrete beams reinforced with FRP as tensional reinforcement, such as Tureyen and Frosch (2003), who developed a design model that predicts the shear strength of concrete beams reinforced with FRP. However, the

majority of previous studies have focused on rectangular sections, resulting in a lack of knowledge of the shear behavior of FRP-reinforced T-beams. Therefore, there is insufficient information in the literature about the shear behavior of T-beams reinforced with FRP that enables researchers to develop and verify design models for T-beams. The purpose of this study is to investigate the shear behavior of simply supported T-beams reinforced with Glass Fiber Reinforced Polymer bars, as well as to assess the accuracy of the current design models for predicting the shear strength of these beams.

1.3 Research Objective:

The essential objectives of this research are to:

- 1) Study the shear behavior of simply supported T-beams reinforced with GFRP as longitudinal reinforcement.
- 2) Investigate the effect of compressive strength, tension reinforcement ratio, and span-to-effective depth ratio on the shear strength of T-beam reinforced with GFRP.
- 3) Determine the level of accuracy of current shear design models in predicting the shear capacity of T-beams reinforced with GFRP.
- 4) Monitor the effect of GFRP reinforcement on crack development and spacing.

1.4 Research Questions:

Q1: How do compressive strength, tension reinforcement ratio, and span-to-effective depth ratio affect the shear strength of the T-beam reinforced with GFRP?

Q2: What is the level of accuracy of current shear design models, such as the Canadian guideline (CSA S806-12), the American Concrete Institute (ACI 440.1R-15), ISIS Design Manual (ISIS 2007), and the Japanese guideline (JSCE, 1997), in predicting the shear capacity of T-beams reinforced with GFRP?

Q3: How does GFRP reinforcement affect crack development and spacing in T-beams?

1.5 Research Significance:

GFRP is worth investigating as it provides superior corrosion resistance and a reduction in thermal and electrical conductivity as compared to steel. However, GFRP reinforcement has not yet been fully explored. For example, most current GFRP shear design formulas are verified against rectangular beams only. Hence, this study is significant since it measures the accuracy of the shear design models for T-beams reinforced with GFRP and indicates whether the models provide balanced, overestimated, or unsafe results for the T-sections' shear strength. In addition, this study narrows the knowledge gap in the literature regarding the shear behavior of T-beams reinforced with GFRP bars. Further, the findings of this research provide the behavior of shear strength for different concrete strengths, span-to-depth ratio, and reinforcement ratios, which are valuable for developing and verifying new shear design models for FRP-reinforced T-beams.

1.6 Research Methodology:

This research uses experimental and finite element methods to collect quantitative data that describes the shear behavior of T-beams reinforced with GFRP as tension reinforcement. The empirical study was performed by testing seven different simply supported T-beams. The concrete specimens' shuttering and concreting were carried out in the Eastern Precast factory, and after that, the testing was executed in the University of Bahrain labs. Moreover, the finite element analysis was achieved by modeling and analyzing the beams using Abaqus software. All beams' cross-sectional dimensions and lengths were constant, and the test parameters were the GFRP reinforcement ratio, shear-to-span length ratio, and the compressive strength of concrete. Furthermore, all beams were designed to fail due to shear before flexural failure.

1.7 Thesis outline:

1.7.1 Chapter Two: Literature Review:

This chapter presents a summary of the previous studies that focused on investigating the shear behavior of beams reinforced with FRP bars. The literature includes several aspects of the

predecessor's methodology, such as specimen size, length, number, test parameters, and loading techniques. Then, it discusses the results obtained. In addition, this chapter highlights the areas where further studies for FRP are needed.

1.7.2 Chapter Three: Experiment Methodology:

This chapter determines the material and equipment used for the experiment, focusing on its specifications and properties. It then explains specimen design procedures based on the American Concrete Institute (ACI 440.1R-15). Furthermore, this chapter discusses the preparation of 7 T-beam specimens, the experiment steps, and the test setup.

1.7.3 Chapter Four: Experimental Results and Discussion:

This chapter presents the experimental results regarding shear strength, failure mode, crack pattern, and load-deflection behavior. Moreover, the chapter compares the experimental shear strength with several design codes.

1.7.4 Chapter Five: Finite Element Analysis:

This chapter demonstrates the procedures followed to define the materials (Concrete and GFRP), and to model the T-beams in the finite element analysis software Abaqus. Besides that, the chapter presents the FEA results and compares them with the experimental results.

1.7.5 Chapter Six: Conclusion and Recommendations:

The conclusion summarizes the research outcomes of shear behavior affected by reinforcement ratio, shear-span-depth ratio, and compressive strength. It also states this study's limitations and recommendations for future research

Chapter Two

Literature Review

2.1 Introduction:

In the event of flexural failure, if the reinforcement ratio is less than the balance ratio, and the tensile stress at failure exceeds the ultimate strength of the bars, the result will be FRP rupture. A higher reinforcement ratio than the balance ratio, on the other hand, will result in the cracks propagating toward the compression zone, causing a reduction in the compression area, which will result in concrete crushing failure (ACI 440.1R-15). Unlike flexural failure, however, shear failure behavior is more difficult to predict due to the higher complexity of the shear failure mechanism (Alam 2010) and the fact that it often occurs without warning. Moreover, it is also due to other factors affecting the behavior of the beam, such as concrete compressive strength, beam geometry, load magnitude, and location. Generally, FRP members are designed in a similar way as steel-reinforced members. However, it is important to consider the differences in mechanical properties between the two types of reinforcement since they affect the shear behavior (ACI 440.1R-15). In light of this, many researchers are investing their efforts in examining and studying the shear behavior of beams reinforced with FRP. This chapter presents some of the methods and outcomes of previous laboratory experiments and finite element studies performed on beams reinforced with FRP. Besides that, the chapter discusses and compares between the current available design codes for the shear strength of members reinforced with FRP.

2.2 Previous Experiments:

2.2.1 Lab Experiments of FRP Reinforced Beams:

2.2.1.1 Lab Experiments on Rectangular Sections:

Mudaqiq (2018) conducted an experiment to study the shear behavior of 5 high-strength concrete beams. The beams were divided into 4 groups: the first and the second groups were reinforced with Carbon and Glass Fiber Reinforced Polymer (CFRP & GFRP), the third group was reinforced with a hybrid of carbon fiber reinforced polymer and standard steel, while the fourth

group is reinforced with a combination of Glass Fiber Reinforced Polymer and normal steel. For all the groups, the compressive strength was 9000 psi (62 MPa), and no shear reinforcements were added. Besides that, all the beams had the same rectangular section with 12 inches of depth (304.8 mm), 6 inches of width (152.4 mm), and 7 feet long (2.13 m). Two strain gauges were attached to each steel bar to measure the strain of the beams during loading. Finally, the beams were subjected to four-point monotonic loads at a deflection control rate of 1 mm/min. The beams were tested with a shear span-to-depth ratio of 2 & 2.5. As a result, all beams failed due to the shear force as planned. The deflection of the beams reinforced with pure GFRP and CFRP was higher than that of hybrid beams. In addition, beams with pure GFRP had the maximum deflection among all the beams, and the hybrid GFRP beam deflected more than CFRP, but it had a higher maximum applied load (Table 2.1).

Table 2.1: Experimental vs. analytical results of Mudaqiq (2018)

Beam	Max. experimental loads (kip/kN)	ISIS Module 3 Predicted loads (kip/kN)	ACI-318 Predicted loads (kip/kN)	ISIS Module 3 Predicted deflection (in/mm)	Max. deflection (in/mm)
ICONT-MN-2	21.74 / 96.7	-		-	0.99 / 25.15
Cf_MN_2	15.09/ 67.11	10.42 /46.35	23.989/106.7	2.73/69.3	0.63 / 16
GF_MN_2	11.05/ 49.15	2.90/ 12.9	23.95/106.5	1.11/27.94	0.84 / 1.34
CF_ST_MN_2.5	21.15 / 94.09	21.63 / 96.2	24.57/109.28	1.13/28.7	0.32 / 8.13
CF_ST_MN_2	14.43/ 64.17	20.62 / 91.7	24.76/110.14	0.91/23.11	0.3 / 7.62
GF_ST_MN_2	19.15 / 85.2	20.30 / 90.3	24.72/109.97	1.2/30.48	0.38 / 9.65

The author followed the ISIS Module 3 and ACI-318 codes to predict the ultimate load and to compare them with the experimental loads. The results are shown in Table 2.1. The Table shows that the ISIS Module 3 did not predict all the test specimens accurately except for beam CF_ST_MN_2.5 and GF_ST_MN_2, while the shear strengths obtained by the ACI code are overestimated for the hybrid beams and underestimated for beams reinforced with FRP only. Furthermore, in all cases, the author found that beams with CFRP have lower strain and higher

load capacity than GFRP specimens. Finally, It was concluded that the ISIS Module 3 prediction for the beam deflection is poorly matched with the experimental results.

Hassan et al. (2022) studied the shear strength of 15 beams reinforced with GFRP as a stirrup and longitudinal reinforcement. All beams were simply supported rectangular beams with a length of 1550 mm. The concrete beams were tested by applying a four-point load until failure. This experiment focused on four parameters: shear reinforcement ratio, concrete compressive strength, and steel fiber content. The results showed that increasing the value of any of the three parameters increases the ultimate failure load. Besides that, the maximum crack width is measured during the experiment and calculated using ACI, CSA S806-12, and ISIS codes. It was noticed that ISIS agrees with the experimental values the most. However, the author recommends modifying the current equations to be more compatible with high-strength and FRP beams.

Nguyen-Minh and Rovňák (2011) conducted an experiment on 12 GFRP beams using a four-point bending loading scheme to study the shear resistance. All specimens' shear span to effective depth ratio (a/d) was less than 2.5. The 12 beams had a rectangular section with 150 mm width, 300 mm depth, and a span length of 2600 mm. The beams were classified into two groups based on concrete compressive strength equal to 36.4 MPa and 46.3 MPa. After that, each group was divided into two subgroups based on the bottom reinforcement type (steel and GFRP). All the bottom reinforcements had 16 mm diameter bars, and each type had beams with reinforcement ratios of 1.01%, 1.52%, and 2.02%. On the other hand, all top reinforcements were two steel bars of 10 mm diameter, and 6 mm diameter were used for stirrups. The beams were tested with a 15 kN/m loading rate. It was observed that flexural cracks in GFRP beams are longer than those that appeared in steel beams, which supported the conclusion that the neutral axis of GFRP beams is significantly less than the corresponding steel beams. In addition, it was reported that increasing the compressive strength of GFRP members by 27% led to an increment in the ultimate shear capacity from 9.5 to 13.2% and a reduction in the beam deflection. In all cases, the ultimate shear capacity of steel-reinforced members was higher than the equivalent GFRP-reinforced members, and the deflection was less. Moreover, the shear capacity gain and deflection reduction rate were found to be approximately similar due to increasing the GFRP and steel reinforcement ratio.

Cholostiakow et al. (2018) investigated experimentally the effect of depth on the shear strength of rectangular beams reinforced with sand-coated GFRP. The experiment was conducted

on eight simply supported beams. The specimens were divided into beams with shear reinforcement and beams without shear reinforcement. In addition, the beam depth range differed from 260 mm to 460 mm, but the longitudinal reinforcement ratio and the shear span-to-depth ratio were fixed, and they were determined to have shear failure before bending failure. The results reported that all the beams had a diagonal tension mode of shear failure. Besides that, it was also found that the first crack's vertical location and the crack spacing are related to the beam depth; increasing the beam depth lowers the location of the first crack point and increases the crack spacing. The author confirmed that the shear strength of FRP beams without shear reinforcement is affected by beam depth. However, beams with shear reinforcement showed no substantial effect due to size variation. In addition, The CSA (2012), fib (2007), and Hoult et al. (2008) models had approximately the same shear strength results as the experiment results. However, some predictions were unsafe. On the other hand, ACI predictions were uneconomically conservative.

Tureyen & Frosch (2003) supported the argument stating that the traditional formulas for shear strength of concrete, which were derived based on specimens reinforced with steel, are unsuitable for FRP specimens, because the modulus of elasticity of FRP is approximately 25 percent of the value of steel, also due the variance in the properties between FRP and steel. Besides that, multiple test results showed that beams reinforced with steel fail at higher loads than FRP beams. However, based on several investigations, the authors stated that beam members with equivalent stiffness had almost the same shear failure load and crack depth. Hence, the flexural reinforcement's stiffness is essential for the beams' shear strength because raising the member's stiffness will increase the cracked section's neutral axis's depth, causing higher shear resistance. The shear strength of rectangular reinforced concrete beams without shear reinforcement can be calculated considering stresses in the compression zone of a cracked reinforced concrete section. Tureyen & Frosch (2003) have developed a design model that calculates the shear strength of steel and FRP-reinforced beams based on the compression stresses above the neutral axis of a cracked reinforced concrete section. The model was examined against 370 experimental shear strengths from 26 previous studies. 326 specimens were steel-reinforced beams, while 44 were FRP-reinforced beams. The specimens were chosen according to the following conditions:

- a/d more than 2.7.
- Failure is due to shear and not flexural.

- No stirrup reinforcement.
- Sufficient anchorage for the longitudinal reinforcement

Equation 2.1 was developed to predict the shear strength of reinforced rectangular concrete beams based on the hypothesis that when the principal tensile stresses overcome the concrete's tensile strength in the compression zone of a cracked section, shear failure occurs in reinforced concrete beams without stirrups.

$$V_c = \frac{2}{3} b_w c \sqrt{f_t^2 + f_t \frac{\sigma_m}{2}} \quad (2.1)$$

Where,

V_c = concrete shear strength

b_w = beam width

c = neutral axis depth. (calculated using elastic cracked section analysis)

f_t = concrete tensile strength

σ_m = minimum flexural stress at extreme compression fiber due to moment just above the cracking moment.

To assess the accuracy of the proposed formula, a model ratio was calculated by dividing the shear strengths obtained from previous studies by their equivalent estimated shear strength using Equation 2.1, as Equation 2.2 shows.

$$Model\ ratio = \frac{V_{exp}}{V_{calc}} \quad (2.2)$$

The minimum and maximum model ratios were observed to be 0.88 and 1.72, respectively, with an average of 1.18 and standard deviations of 0.15. Besides that, the variation of the model ratio was analyzed against concrete compressive strength, longitudinal reinforcement ratio, and span-effective depth ratio (a/d). It was observed that specimens with a lower reinforcement ratio were more conservative and had a higher model ratio. On the other hand, the results maintained

approximately the same level of accuracy for various concrete strengths and effective depth. However, the study required more test results due to a need for specimens with a higher span effective depth ratio (a/d). All in all, these results showed that the formula presents a conservative shear strength. Furthermore, to simplify the formula for the design, the tensile strength of concrete in Equation 2.1 was substituted by $0.6\sqrt{f'_c}$, resulting in Equation 2.3.

$$V_c = \left(\sqrt{16 + \frac{4\sigma_m}{3\sqrt{f'_c}}} \right) \sqrt{f'_t} b_w c \quad (2.3)$$

Considering what is between the parenthesis as K_c , Equation 2.4 is obtained.

$$V_c = K_c \sqrt{f'_c} b_w c \quad (2.4)$$

Where,

$$K = \sqrt{16 + \frac{4\sigma_m}{3\sqrt{f'_c}}} \quad (2.5)$$

By applying experimental results to Equation 2.6, the average K is found to be 6.79. However, K is taken to be 5 for conservative estimation, resulting in Equation 2.7, which is more straightforward and more suitable for beam design than Equation 2.3; however, it provides more conservative estimates. Therefore, ACI Committee 440 adopted Equation 2.7.

$$K_{exp} = \frac{V_c}{\sqrt{f'_c} b_w c} \quad (2.6)$$

$$V_c = 5\sqrt{f'_c}b_w c \quad (2.7)$$

2.2.1.2 Lab Experiments on T-sections:

Tureyen et al. (2006) dedicated further studies to investigate the applicability of Equation 2.7 on flanged sections reinforced with steel. The investigation was carried out on 154 T-beams from 32 different studies. All the specimens that were selected had to be failed due to shear failure. Tureyen et al. (2006) compared the shear strength of the previous test results with the results of Equation. By substituting b_w as full flange length, it was found that most of the calculated shear strengths are significantly more than the test results. Therefore, the author concluded that using Equation with b_w as full flange width is unsafe. On the other hand, substituting b_w as web width only provides safe but uneconomical results. Consequently, it was concluded that the proper area size, which provides conservative and economical results, lies between the area of full flange width and web width. The author stated that this area could be determined by multiplying the full flange compression area by a required-computed area ratio (A_{req}/A_{comp}) as follows:

$$\text{Proper area} = b_f c (A_{req}/A_{comp}) \quad (2.8)$$

Where,

$$A_{comp} = c b_f$$

$$A_{req} = \frac{V_{test}}{5\sqrt{f'_c}}$$

Substituting Equation 2.8 in Equation 2.7 provides Equation 2.9.

$$V_c = 5\sqrt{f'_c}b_f c (A_{req}/A_{comp}) \quad (2.9)$$

After that, drawing A_{req}/A_{comp} vs b_f/b_w and then finding the best-fit curve led to the following relation.

$$\frac{A_{req}}{A_{comp}} = \frac{1}{\sqrt{b_f/b_w}} = \sqrt{b_w/b_f} \quad (2.10)$$

In addition, substituting Equation 2.10 in Equation 2.9 provided the final shear strength formula for T-sections as follows.

$$V_c = 5\sqrt{f'_c}(b_f c \sqrt{b_w/b_f}) \text{ when } c < t \quad (2.11)$$

$$V_c = 5\sqrt{f'_c}(b_f t \sqrt{b_w/b_f} + b_w(c - t)) \text{ when } c > t \quad (2.12)$$

Where,

t = flange depth

By comparing the final expression with the available experimental results, the authors observed that decreasing the span effective ratio (a/d) leads to more conservative shear strength, and increasing effective depth decreases the V_{exp}/V_{calc} ratios. However, some observations of low values of V_{exp}/V_{calc} appear at small effective depths. In addition, Equations 2.11 and 2.12 have provided conservative results for 150 T-beams out of 154 specimens. The average V_{exp}/V_{calc} ratio equals 1.32, which is similar to the average of the rectangular beams. Besides that, the standard deviation and the coefficient of variation were calculated to be 0.32, and 0.175, respectively. These results indicated that the final equations provide a fair level of accuracy and are suitable for both sections. However, it must be noted that the two equations are examined against 154 T-beams reinforced with steel only, and FRP members have not yet been tested against them.

Zhang et al. (2022) have published a paper that discusses the shear behavior of T-shaped concrete beams reinforced with FRP as a stirrup reinforcement. In this paper, the authors have conducted both an experimental study and a theoretical analysis. The experimental study was carried out on 18 simply supported concrete T-beams, as shown in Table 2.2, which were divided into different groups. Three beams were treated as control beams reinforced with ordinary longitudinal steel but without stirrup reinforcement. The other 15 beam specimens were also

reinforced with ordinary steel as longitudinal reinforcement but with FRP as stirrup reinforcement. All beams had the same total length of 2.1 m and 1.8 m clear distance between supports. The beams had an 80 mm & 400 mm flange depth and width, respectively, and 210 mm and 120 mm web depth and width, respectively. Besides that, the beams were tested against various variables, such as:

- Concrete strength.
- Longitudinal reinforcement ratio.
- FRP surface characteristics, type, and diameter.
- FRP stirrups spacing.
- Shear span ratio.

Table 2.2: Zhang et al. (2022) beam details.

Piece No.	Concrete strength	Reinf. ratio	FRP type	FRP Surface characteristics	FRP diameter	FRP spacing	Shear span ratio
BZS20C20	C20	0.021	-	-	-	-	2.39
BZS18C30	C30	0.017	-	-	-	-	2.39
BZS18C20	C20	0.017	-	-	-	-	2.39
L- G8D150S20C20U	C20	0.021	GFRP	Thread	8	150	2.39
L- G8D150S18C30U	C30	0.017	GFRP	Thread	8	150	2.39
L- G8D150S18C20U	C20	0.017	GFRP	Thread	8	150	2.39
G- G8D150S20C20U	C20	0.021	GFRP	Light circle	8	150	2.39
G- G8D150S18C30U	C30	0.017	GFRP	Light circle	8	150	2.39

Piece No.	Concrete strength	Reinf. ratio	FRP type	FRP Surface characteristics	FRP diameter	FRP spacing	Shear span ratio
G- G8D150S18C20U	C20	0.017	GFRP	Light circle	8	150	2.39
L- C8D150S18C20U	C20	0.017	GFRP	Thread	8	150	2.39
L- G8D150S18C20U	C20	0.017	GFRP	Thread	8	150	2.39
L-G8D150S18C20	C20	0.017	GFRP	Thread	8	150	2.39
L- G6D150S18C20U	C20	0.017	GFRP	Thread	6	150	2.39
L- G10D150S18C20U	C20	0.017	GFRP	Thread	10	150	2.39
L-G8D100S8C20U	C20	0.017	GFRP	Thread	8	100	2.39
L- G8D200S18C20U	C20	0.017	GFRP	Thread	8	200	2.39
L- G8D150S18C20U	C20	0.017	GFRP	Thread	8	150	2.79
L- G8D150S18C20U	C20	0.017	GFRP	Thread	8	150	1.99

After the load was applied, the failure mode among the beams was a combination of shear and bending failure. The experimenter discovered that compressive strength negatively affected the ultimate load of beams with surface-embedded threaded GFRP and plain GFRP stirrups. The test observation showed that the more compressive strength there is, the lower the ultimate bearing capacity. Therefore, the author concluded that the higher the concrete grade, the higher the brittleness. Moreover, it has also been found that the higher the longitudinal reinforcement ratio, the less the ultimate load; the author has justified this conclusion by two reasons: the first one is that the web size of the specimens was small, and the second reason is that the distance between

the longitudinal reinforcement was also small. In addition, the author argued that the embedded smooth GFRP stirrups resist higher ultimate load compared with the embedded threaded GFRP stirrups. Furthermore, the effects of GFRP diameter as shear reinforcement have also been studied; the study was accomplished by comparing the ultimate strength of three beams reinforced with GFRP stirrups of different diameters, 6 mm, 8 mm, and 10 mm. The results were unexpected because they showed that the beam with 8 mm diameter GFRP stirrups had the maximum bearing capacity; the researcher claimed that the reason behind this is that the bond in the groove is reduced because the diameter of 10 mm stirrups is too large. Furthermore, the author studied the effects of the spacing of the stirrups; this was done using 8 mm diameter bars for three different stirrups spacing, 100 mm, 150 mm, and 200 mm. The 150 mm spacing was optimal because it had the maximum bearing capacity among the three spacings. Finally, the author reported that the shear span ratio and the ultimate load capacity are inversely proportional.

2.2.2 Studies Based on Finite Element Analysis:

Alam (2010) conducted several FEA using Abaqus software and compared them to his experimental results. In the beginning, the researcher calibrated and verified the concrete's behavior in Abaqus; this was achieved by comparing a calculated stress-strain relation of both tension and compression with the Abaqus response results of a model. The concrete behavior was obtained based on Figure 2.1 and Equation 2.13, respectively, and Abaqus results were determined from a concrete cube model with 50x50x50 mm dimensions. As a result, the author found that both outcomes matched well.

$$\frac{f_c}{f'_c} = \frac{n_c \left(\frac{\epsilon_c}{\epsilon_o} \right)}{n_c - 1 + \left(\frac{\epsilon_c}{\epsilon_o} \right)^{nk}} \quad (2.13)$$

Where,

f_c = concrete compressive stress.

ϵ_c = concrete strain.

ϵ_o = concrete compressive strain at f'_c .

n_c = curve fitting factor = $0.8 + \frac{f'_c}{17}$.

k = stress decay factor, taken as 1.0 for $\frac{\varepsilon_c}{\varepsilon_o} < 1$ and as a value more than 1.0 if $\frac{\varepsilon_c}{\varepsilon_o} > 1$.

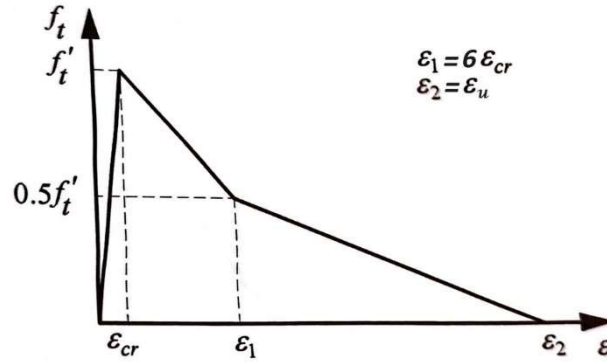


Figure 2.1: Concrete tension stiffening (Alam 2010)

Besides that, the mesh size was examined on one beam to find the proper mesh size. Three sizes were studied: 50x50x50 mm, 70x83x70 mm, and 100x125x87; the 3 mesh sizes were categorized as fine, medium, and coarse mesh. The medium mesh outcomes were found to be the closest to the experimental results regarding load-deflection behavior, but the fine mesh was found to be better in stress distribution. Based on Bažnat and Oh's (1983) recommendation, the mesh size should not be less than three times the maximum aggregate size. Therefore, the author decided to use fine mesh rather than medium mesh since it is the nearest option to Bažnat and Oh's (1983) recommendation. For the dilation angle, the researcher examined three values, 20, 30, and 40 degrees, using fine mesh; the load-deflection results did not show any significant differences between the results of the three angles, and the maximum difference between maximum load did not exceed 10%. Therefore, the 30-degree dilation angle was selected. After the successful verification of the FE software, several comparisons between the author's FE models and the experimental results were conducted to investigate the behavior of the simply supported beams with different parameters. The parameters investigated were reinforcement type, the span-to-effective depth ratio (a/d), beam depth, reinforcement ratio, and concrete strength. The models were loaded with 4-point displacement-controlled loading. Normal steel, GFRP, and CFRP were examined for comparison based on reinforcement type. GFRP and CFRP reinforcement had three span-to-effective depth ratios of 2, 2.7, and 3.57, while the normal steel was studied using one span-to-effective depth ratio equal to 2.5. The characteristics of load-deflection behavior obtained from the models of the three types had a fair match with the experimental results. Besides that,

FEA predicted the ultimate load and deflection of the GFRP beams fairly, but for CFRP reinforced beams with span-to-effective depth ratios of 1.5 and 2.5, the software overestimated the load and the deflection, while for the normal steel, they were underestimated. The author justified the variation for two reasons: the first reason is due to the higher axial stiffness of the CFRP, and the second reason is that the same tension stiffening was used for both reinforcements.

Additionally, for the comparison based on depth, the author found that both methods provide similar maximum deflection. Yet, the model prediction for the failure load of a beam reinforced with GFRP and with a depth of 800 mm was less than the experimental results by 13%, and for all the other beams, the model prediction was higher by 12%. On the other hand, for beams with CFRP, the model prediction for the failure load was higher than the experiment loads by 40 and 80%, which is significantly higher than the results obtained for GFRP. The author attributed the significant overestimation to the high stiffness of CFRP compared with GFRP. Furthermore, the comparison between the two methods for different reinforcement ratios was performed on GFRP-reinforced beams only, and it was found that the model underestimated the ultimate load and deflection of beams with a reinforcement ratio lower than the balanced ratio and overestimated the beams with a reinforcement ratio higher than the balanced value, while the beam with balanced reinforcement ratio had the closest value to the experiment results. Finally, for the comparison based on the concrete strength, the model slightly underestimated the load and the deflection of the beams reinforced with GFRP and with the high-strength concrete.

Salh (2014) used concrete damage plasticity with crack band theory to model a beam reinforced with normal steel and basalt fiber reinforced polymer (BFRP) by Abaqus software. The aim of the model was to compare Abaqus's results with the author's experimental results. The beam dimensions were 150x200x2000 mm, and the clear distance between supports was equal to 1800 mm. In addition, four loading points were applied to the beam, and the location of the load was the same as the location applied in the experiment. The beam was modeled in 2D and 3D; for the 2D model, half of the beam was modeled, while for the 3D model, only a quarter of the beam was modeled; this was done to the symmetry and to reduce the running time of the software. Furthermore, the concrete was defined in the software based on concrete damage plasticity, concrete tension stiffening, concrete compression hardening, and concrete tension and compression damage. However, since there were no loading and unloading cycles in this study,

the concrete tension and compression damage were optional parameters. The Poisson's ratio used to be 0.22 for concrete, while the author obtained the stress-strain relation based on the Saenz (1964) equation as follows:

$$f_c = \frac{E_c \varepsilon}{1 + \left[\left(\frac{E_c \varepsilon_p}{f'_c} \right) - 2 \right] \left(\frac{\varepsilon}{\varepsilon_o} \right) + \left(\frac{\varepsilon}{\varepsilon_o} \right)^2} \quad (2.14)$$

Where,

f_c = compressive stress

ε = compressive strain

f'_c = maximum compressive stress

ε_o = strain corresponding to maximum compressive stress

E_c = modulus of elasticity of concrete

E_c was taken according to ACI310 equal to $4730\sqrt{f'_c}$, while f'_c was obtained from cylinder test and ε_o was set to be equal to 0.002. On the other hand, the uniaxial tensional softening behavior was derived following the fracture energy approach based on the Hordijk (1991) curve as follows:

$$\frac{\sigma_t}{f'_t} = \left[1 + \left(c_1 \frac{w_t}{w_{cr}} \right)^3 \right] e^{\left(-c_2 \frac{w_t}{w_{cr}} \right)} - \frac{w_t}{w_{cr}} (1 + c_1^3) e^{(-c_2)} \quad (2.15)$$

Where,

w_{cr} = cracking width after a total release of stress, and it is equal to $5.14 \frac{G_f}{f_t}$

w_t = cracking width.

σ_t = concrete tensile stress

f'_t = concrete tensile strength

G_f = the fracture energy required to create a stress-free crack over a unit area

$$G_f = (0.0469d_a^2 - 0.5d_a + 26) \left(\frac{f_c'}{10} \right)^{0.7}$$

d_a = maximum aggregate size

Moreover, the embedded method was followed for modeling the reinforcement, and the author assumed a perfect bond between reinforcement and concrete. The ultimate stress and strain of the BFRP were used equal to 1350 MPa 0.031, respectively. To find optimum Abaqus results, 10, 20, and 40 mm mesh sizes were used for the reinforcement.

As a result, for the 2D model reinforced with steel as longitudinal reinforcement, a fair agreement was achieved between the model and the experimental results in load-deflection behavior, prediction of the first crack, and ultimate load. On the contrary, the author found that the load-deflection behavior of the 3D model reinforced with steel as longitudinal reinforcement, varied from the experimental behavior. Besides that, FEA underestimated the beam stiffness. Nevertheless, the ultimate load and its corresponding deflection of both methods were close (Figure 2.2). On the other hand, the author found that both models failed to converge after the elastic stage when BFRP was used as longitudinal reinforcement, as shown in Figure 2.3.

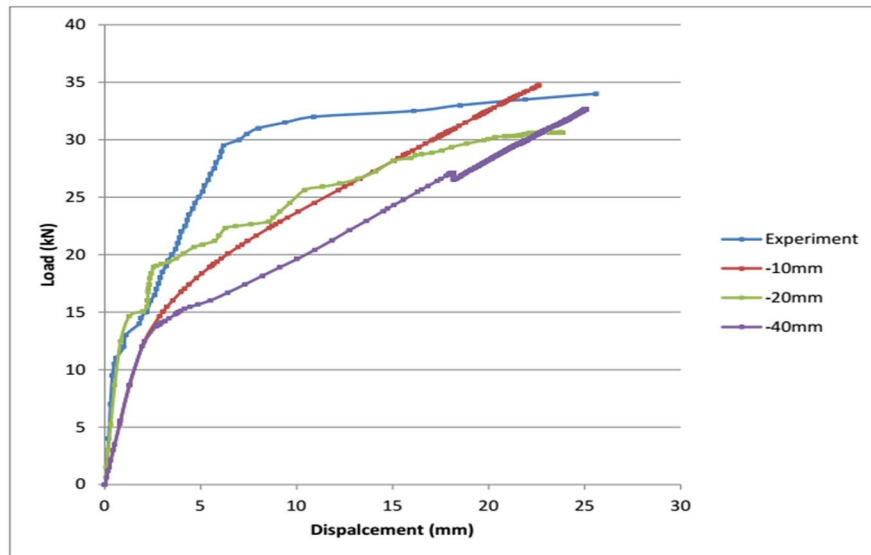


Figure 2.2: Load-deflection behavior of 3D model reinforced with steel (Salh 2014)

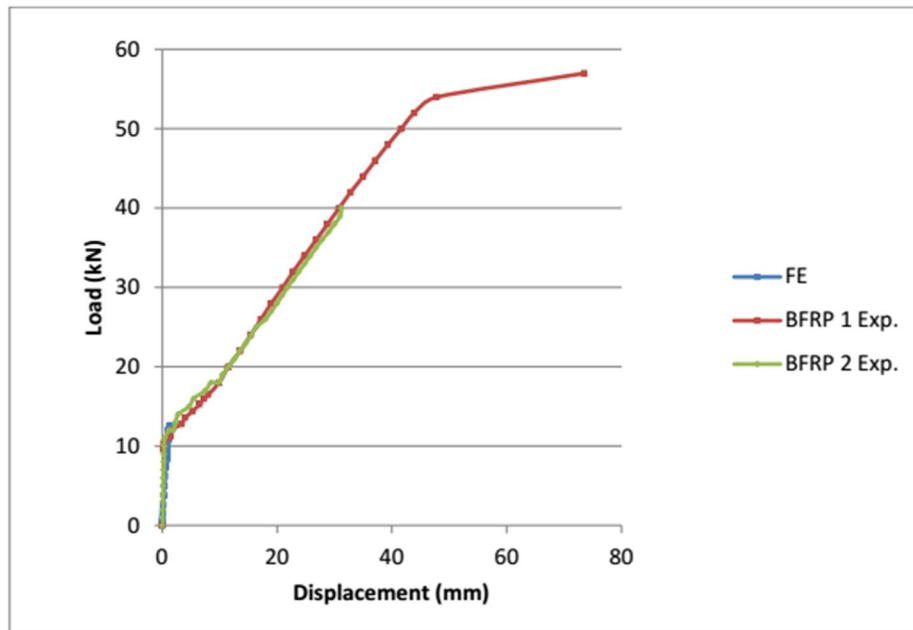


Figure 2.3: Load-deflection behavior of 2D model reinforced with BFRP (Salh 2014)

Suzan et al. (2017) have modeled 29 concrete beams using the nonlinear finite element software Ansys. The results of three modeled beams were compared with the available experimental tests to verify the software outcomes. Two types of beams were tested. The first type was a simply supported rectangular beam with a span length of 1750 mm, depth of 200 mm, and width of 120 mm. The beams were divided into four groups based on top and bottom reinforcement. On the other hand, the second type was a 6 m continuous rectangular beam with a 2.8 m span, 300 mm depth, and 200 mm width. The continuous beams were also divided into four groups with the same reinforcement arrangement as the simple beam. However, GFRP reinforcement is used for the continuous beam instead of CFRP. Based on the finite element results, the authors concluded that due to the low value of the elastic modulus of GFRP, a higher percentage of GFRP reinforcement in hybrid beams reduces the stiffness of the beam. CFRP hybrid beams also encountered the same relation with stiffness but with a slighter effect. The researcher also found that using pure steel as top reinforcement with hybrid GFRP and steel as tension reinforcement has improved the ultimate load capacity by 5 to 16 percent, depending on the GFRP/steel ratio. Besides that, before the concrete crashed, all steel reinforcements yielded; however, the GFRP and CFRP reinforcements reached a low value of their ultimate stress and did

not yield. In addition, GFRP and CFRP bars did not experience any rupture because of their high value of strain, which caused the beams with only FRP as top and bottom reinforcement to fail due to concrete crushing, which is undesirable. Therefore, the authors recommended using FRP in combination with steel as bottom reinforcement only.

2.3 Codes Provision for Beams Reinforced with FRP:

2.3.1 The American Concrete Institute (ACI 440.1R-15):

FRP-reinforced members exhibit two types of flexural failure: FRP ruptures and concrete crushing, also known as tension and compression-controlled behaviors. Tension-controlled behavior is when the FRP ruptures before the concrete crashes, while compression-controlled behavior is the opposite, concrete crushing before the FRP ruptures.

ACI Committee 440 states that although FRP rupture failure occurs suddenly, the high elongation FRP experience before failure generates limited warnings that can be visualized in the shape of large cracks and deflection. However, due to the lower ductility of FRP compared with regular steel, these warnings will always be more inadequate than the warnings steel provides. Consequently, the code reports that using safety factors higher than the values used with regular steel is necessary to compensate for FRP members' low ductility, and to allow the design to be used for both compression- and tension-controlled failures.

Similar to the normal steel reinforcement, the nominal moment capacity of FRP-reinforced members is calculated based on strain compatibility and internal force equilibrium. However, FRP-reinforced members' flexural strength varies based on the controlled behavior (concrete crushing or FRP rupture). The controlling behavior could be predicted based on reinforcement and balanced reinforcement ratios. The balanced reinforcement ratio is the ratio when both failures (FRP ruptures and concrete crashes) happen at the same time. Figure 2.4 shows the stress and strain distribution for the balanced reinforcement section.

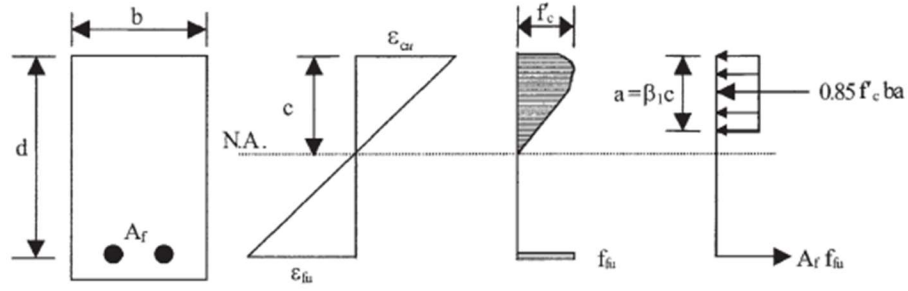


Figure 2.4: Stress and strain distribution for the balanced reinforcement section (ACI 440.1R-15)

In addition, the reinforcement and balanced reinforcement ratios could be calculated using Equations 2.16 and 2.17, respectively.

$$\rho_f = \frac{A_f}{bd} \quad (2.16)$$

Where,

ρ_f : FRP reinforcement ratio

A_f : area of FRP

b : beam width

d : effective depth

$$\rho_b = 0.85\beta_1 \frac{f'_c}{f_{fu}} \frac{E_f \epsilon_{cu}}{E_f \epsilon_{cu} + f_{fu}} \quad (2.17)$$

Where,

ρ_b : Balanced FRP ratio

β_1 : Concrete factor as follows:

$$\text{if } f'_c \leq 28 \text{ MPa} \quad \beta_1 = 0.85$$

$$\text{if } 28 < f'_c < 56 \quad \beta_1 = 0.85 - 0.05 \frac{f'_c - 28}{7}$$

$$\text{if } f_c' > 56 \quad \beta_1 = 0.65$$

f_c' : Concrete compressive strength

E_f = FRP modulus of elasticity

ϵ_{cu} : concrete ultimate strain

f_{fu} : Design tensile strength of FRP

Moreover, if the reinforcement ratio is higher than the balanced ratio, the tensile stress due to the applied load will not reach the ultimate stress of the bars, and the flexural failure will be due to concrete crushing, and the stress and strain distribution will be as Figure 2.5 demonstrates. Besides that, the nominal moment capacity will be found using Equation 2.18.

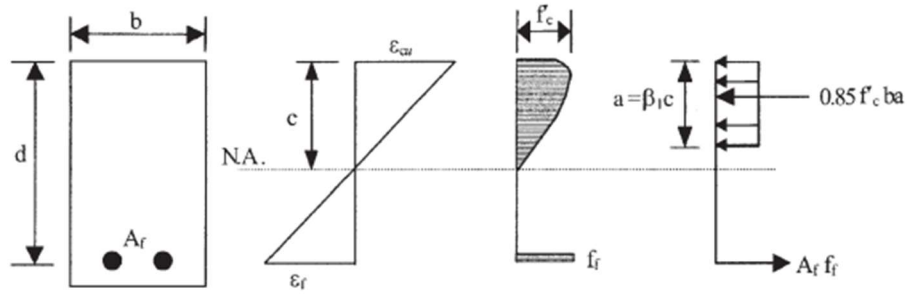


Figure 2.5: Stress and strain distribution for the over-reinforced section (ACI 440.1R-15)

$$M_n = A_f f_f \left(d - \frac{\beta_1 c}{2} \right) \quad (2.18)$$

Where,

$$a' = \frac{A_f f_f}{0.85 f_c' b}$$

$$f_f = E_f \epsilon_{cu} \frac{\beta_1 d - a'}{a'} = \left(\sqrt{\frac{(E_f \epsilon_{cu})^2}{4} + \frac{0.85 \beta_1 f_c'}{\rho_f}} E_f \epsilon_{cu} - 0.5 E_f \epsilon_{cu} \right) \leq f_{fu}$$

On the other hand, if the reinforcement ratio is lower than the balanced ratio, the tensile strength will be reduced and the maximum stress due to the applied load will reach the ultimate tensile strength. Therefore, the controlling limit state will be the FRP rupture, and the stress and

strain distribution will be as Figure 2.6 shows and the nominal moment capacity will be computed using Equation 2.19.

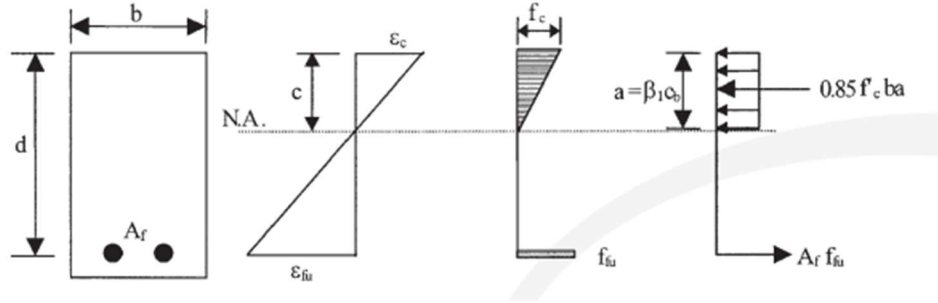


Figure 2.6: Stress and strain distribution for the under-reinforced section (concrete stress may be nonlinear) (ACI 440.1R-15)

$$M_n = A_f f_{fu} \left(d - \frac{\beta_1 c}{2} \right) \quad (2.19)$$

Where,

$$c = \left(\frac{\varepsilon_{cu}}{\varepsilon_{cu} - \varepsilon_{fu}} \right) d$$

It is worth to mention that ACI 440 recommends that the above equations must be applied with consideration the following assumptions:

- The plan section remains plain after loading.
- Maximum compressive strain equal to 0.003.
- The tensile strength of concrete is not taken into account.
- The stress-strain relation is linearly elastic until the ultimate strength.
- No bond failure between the concrete and FRP reinforcement.

Furthermore, the ACI 440 standard recommends using the following expression to determine the shear capacity of beams reinforced with FRP.

$$V_c = \frac{2}{5} \sqrt{f'_c} b_w (k_v d) \quad (2.20)$$

Where,

$$k_v = \sqrt{2\rho_f n_f + (\rho_f n_f)^2} - (\rho_f n_f)$$

$$n_f = \frac{E_f}{E_c}$$

Finally, it should be noted that experimental results have not yet confirmed the applicability of the above equations on T-beams reinforced with FRP.

2.3.2 Japan Society of Civil Engineers (JSCE 1997):

Japan Society of Civil Engineers (JSCE) has developed an equation to estimate the shear capacity (V_c) as expressed in Equation 2.21. The equation is based on the steel reinforcement model, taking into consideration the axial rigidity represented by the modulus ratio. In addition, the JSCE model considers the axial force of pre-stressed members, concrete compressive strength, effective depth, reinforcement ratio, and beam width. However, the span-to-effective depth ratio is ignored. Besides that, it can be noted that the formula considers the contribution of the longitudinal reinforcement and modulus ratios to a certain amount, and, in case of the absence of the longitudinal reinforcement, the model eliminates the shear strength (V_c).

$$V_c = \beta_d \beta_\rho \beta_n f_{vcd} b w \frac{d}{\gamma_b} \quad (2.21)$$

Where,

$$f_{vcd} = 0.2 \sqrt[3]{f'_{cd}} \leq 0.72 \text{ N/mm}^2$$

$$\beta_d = \sqrt[4]{\frac{1000}{d}} \leq 1.5$$

$$\beta_\rho = \sqrt[3]{100 \rho_w \frac{E_f}{E_s}} \leq 1.5$$

$$\beta_n = 1 + \frac{M_o}{M_d} \leq 2 \quad \text{if } N'_d \geq 0$$

$$\beta_n = 1 + \frac{2M_o}{M_d} \geq 0 \quad \text{if } N'_d < 0$$

f'_{cd} = design compressive strength of concrete.

E_f = Young modulus of longitudinal reinforcement

E_s (200000 MPa)

N'_d = design axial compressive force

M_d = design bending moment

M_o = decompression moment

γ_b = strength reduction factor (code recommendation $\gamma_b = 1.3$)

b_w = minimum web width

2.3.3 ISIS Design Manual (ISIS 2007):

The ISIS Design Manual (ISIS 2007), recommends Equation 2.22 for the estimation of the shear resistance of beams reinforced with FRP as longitudinal reinforcement without shear reinforcement.

$$V_c = 0.2\lambda\phi_c\sqrt{f'_c}b_wd\sqrt{\frac{E_f}{E_s}} \quad (2.22)$$

Where,

λ = factor related to concrete density

ϕ_c = resistance factor related to concrete

b_w = minimum effective web width within depth d

d = effective depth of the cross-section

E_f = modulus of elasticity of longitudinal FRP reinforcement

E_s = modulus of elasticity of steel (200x10³ MPa)

It can be noted that the ISIS Design Manual equation does not take into account either the effect of the span-to-effective depth ratio or the effect of the reinforcement ratio. Nevertheless, the standard states that in case of sections with an effective depth of more than 300 mm and the minimum transverse reinforcement is not provided, the concrete resistance V_c should be calculated as follows:

$$V_c = \left(\frac{260}{1000 + d} \right) \lambda \phi_c \sqrt{f'_c} b_w d \sqrt{\frac{E_f}{E_s}} \quad (2.23)$$

Where,

$$\sqrt{\frac{E_f}{E_s}} \leq 1$$

2.3.4 Canadian Guideline (CSA S806-12):

For section with effective depth less than 300 mm and without axial force, the Canadian Standard association standard (CSA S806-12) has presented an equation for shear strength as follows:

$$V_c = 0.5 \lambda \phi_c k_m k_r f'_c{}^{1/3} b_w d_r \quad (2.24)$$

Where

ϕ_c = resistance factor for concrete.

λ = factor to consider the concrete density.

$$k_m = \sqrt{\frac{V_1 d}{M_1}} \leq 1$$

$$k_r = 1 + (E_F \rho_{fw})^{1/3}$$

f'_c = compressive strength of concrete, not taken more than 60 MPa

b_w = minimum web width

d_r = effective depth, taken as maximum of 0.9d or 0.72

The code recommends that minimum shear strength V_c should not exceed $0.22\phi_c \sqrt{f'_c} b_w d_r$. Nor it should be lower than $0.11\phi_c \sqrt{f'_c} b_w d_r$. besides that, similar to the Italian code, the Canadian standard takes the effect of span-to-effective depth ratio into consideration by multiplying V_c of sections with $a < 2.5d$ by k_a which is equal to:

$$k_a = \frac{2.5}{M_1/Vd}, \text{ } k_a \text{ must not be less than 1 and greater than 2.5}$$

On the other hand, for such cases where the effective depth d is greater than 300 mm, V_c must be multiplied by k_a which is equal to $\frac{750}{450+d} \leq 1$.

2.3.5 Codes summary:

Table 2.3 shows a comparison between the parameters of the discussed standards:

Table 2.3: Shear strength parameters vs. standards

Code/parameter	ACI 440.1R-15	JSCE 1997	ISIS 2007	CSA S806-12
Beam width	✓	✓	✓	✓
Effective depth (d)	✓	✓	✓	✓
Span-to-effective-depth-ratio (a/d)	✗	✗	✗	✓
Concrete compressive strength	✓	✓	✓	✓
Reinforcement ratio (ρ)	✓	✓	✗	✓
Tensile reinforcement strength	✗	✗	✓	✗
Axial force	✗	✓	✗	✗
Flange width	✗	✗	✗	✗

Chapter Three

Experimental Methodology

3.1 Introduction:

As mentioned in the literature, Tureyen & Frosch (2003) developed an equation that predicts fairly the shear strength of rectangular beams reinforced with FRP and steel. After that, the equation was modified to be suitable for T-beams (Tureyen et al. 2006), however, their examination of the equation against T-beams did not include specimens reinforced with FRP. Therefore, one of the objectives of this study is to measure the shear strength of T-beams reinforced with GFRP as tension reinforcement without stirrups and to compare the results with various code equations and proposed design models. In addition, the study aims to explore the crack pattern and the deflection of the T-beams reinforced with GFRP. This is carried out by performing an experimental study verified by finite element analysis using Abaqus software. The quantitative results of both methods were obtained by controlling three independent variables: concrete strength, reinforcement ratio, and span-to-depth ratio. This chapter presents the beam design methods, sampling criteria, and preparation steps, as well as the material, machinery, and tools used to perform the lab test.

3.2 Section and Independent Variables Determination:

In this study, all the specimens were structurally analyzed as simply supported beams. The T-beams' design started with the size estimation and was based on factors such as hydraulic machine capacity and ACI code recommendations. The capacity of the hydraulic machine available in the University of Bahrain labs is 200 kN. Still, to perform the test with certainty that the hydraulic machine will be able to break the specimens, 150 kN is considered as the ultimate applied load, so the maximum applied shear force (V_u) that would be applied is 75 kN. The specimen's length was selected to be 1 m at the beginning. However, the 1 m length beam narrows the ranges of a/d and causes the arch actions, so the next integer was appointed as beam length, which is 2 m. In addition, the beam's cross-sectional dimensions were set to be constant, as Figure 3.1 shows. It can be noticed that the beam depth was selected to be 300 mm, which is more than the minimum depth recommended by ACI 318. Moreover, the code does not recommend the flange

width ' b_f ' to exceed $L/4$. Therefore, the b_f is assumed to be 300 mm. Besides that, the flange depth ' h_f ' was taken equal to 75 mm, representing 25% of the total depth. Finally, the number of bars is selected to be 2 to maintain proper bar spacing and concrete cover.

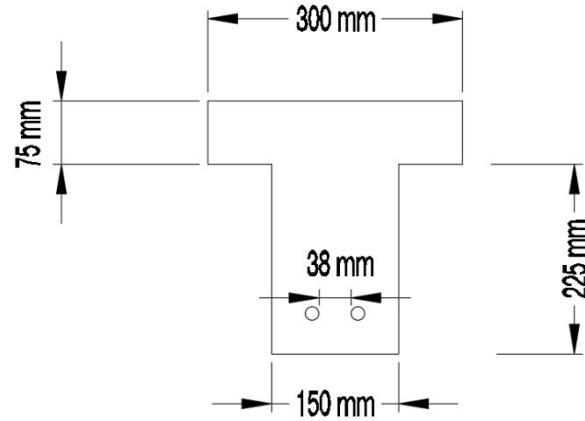


Figure 3.1: Typical specimen section.

The beam section in Figure 3.1 was set as the control section, and then six beams were integrated based on the three independent variables. The first variable is the compressive strength of concrete. Even though this study follows the ACI code, the compressive strength is calculated in terms of f_{cu} first, then 80% of the values were considered as f_c' as the ACI-318 code states, because the concrete strengths were planned to be obtained by a cube test rather than a cylinder test. Besides that, it was initially mapped out to use 35, 40, and 50 MPa as f_c' . However, the 28-day cube test showed that concrete design mixes provided higher compressive strength than expected, as shown in Table 3.1.

Table 3.1: Expected vs. actual compressive strengths

Expected compressive strength, f_{cu} (MPa)		Average actual compressive strength after 28 days, f_{cu} (MPa)	
f_{cu}	$f_c' = 0.8f_{cu}$	f_{cu}	$f_c' = 0.8f_{cu}$
44	35	53.3	42.6
50	40	63.4	50.7
62.5	50	76.1	60.9

In addition, the second variable is the reinforcement ratio. All the specimens were reinforced with a single layer of two tensile GFRP bars, and reinforcement ratio manipulation was done by controlling the diameter size. Three bar diameters were used: 13, 16, and 19 mm. These bars were selected based on the available sizes that the supplier (Industrial Control Solutions) provides and comply with the hydraulic machine capacity. The selected bar sizes provided reinforcement ratios of 0.007, 0.011, and 0.015, which are 1.4, 2.5, and 2.8 times the balanced reinforcement ratio based on the ACI-440 standard

The third variable is the span-to-depth ratio (a/d). This ratio represents the distance between the support and the applied point load (a) divided by the effective depth of the beam (d). The a/d ratios are selected to be 2, 2.7, and 3.57. These values were chosen to maintain the load within the shear span distance, to avoid arch action, and to comply with the specimen criteria of Tureyen & Frosch (2003) ($a/d > 2.7$).

3.3 T-beams Sampling:

Based on the number of independent variables, the T-beams were divided into three groups, each investigated against one independent variable only, as the following tables demonstrate. It should be noted that beams with a span-to-depth ratio (a/d) of 2 and 2.7 were tested under four-point loading conditions, whereas the beam with an a/d ratio of 3.57 was tested under three-point loading.

Table 3.2: Group one (a/d)

Independent variable	Beam No.	Beam Name	a/d	f_c' (MPa)	Bar size (mm)/ ρ (%)
a/d	1	GT-2-50-16	2	50.9	16/1.1
	2	GT-2.7-50-16	2.7	50.6	16/1.1
	3	GT-3.5-50-16	3.57	51.7	16/1.1

Table 3.3: Group two (f_c')

Independent variable	Beam No.	Beam Name	a/d	f_c' (MPa)	Bar size (mm)/ ρ (%)
f_c'	3	GT-3.5-50-16	3.57	51.7	16/1.1
	4	GT-3.5-60-16	3.57	60.9	16/1.1
	5	GT-3.5-40-16	3.57	42.6	16/1.1

Table 3.4: Group three (ρ)

Independent variable	Beam No.	Beam Name	a/d	f_c' (MPa)	Bar size (mm)/ ρ (%)
ρ	3	GT-3.5-50-16	3.57	51.7	16/1.1
	6	GT-3.5-50-19	3.59	50.3	19/1.5
	7	GT-3.5-50-13	3.55	50.1	13/0.7

Since each independent variable had three different values, three specimens were included in each group. Therefore, the number of specimens required for the experiment was seven beams. From the tables above, it can be noticed that beam number three contributes to all the groups, so it is treated as a control beam.

3.4 T-beams Naming:

Based on the number of independent variables, the T-beams were divided into three groups, each investigated against one independent variable only, as the following tables demonstrate. It should be noted that beams with a span-to-depth ratio (a/d) of 2 and 2.7 were tested under four-point loading conditions, whereas the beam with an a/d ratio of 3.57 was tested under three-point loading.

Table 3.2, 3.3, and 3.4 show the beam's names. The naming was set to state the reinforcement type, beam section, span-to-effective-depth ratio, compressive strength, and bar size, as the following example:

B1 is named GT-2-50-16.

Where,

G = Glass Fiber Reinforced Polymer

T = T-beam

2 = span-to-effective-depth-ratio (a/d)

50 = the targeted compressive strength in MPa (f_c')

16 = GFRP bar size in mm

3.5 Flexural Vs. Shear Capacity:

This study aims to investigate the shear strength of GFRP-reinforced T-beams, and to ensure the occurrence of shear failure before flexural failure, the specimens' shear and flexural capacity were determined based on the ACI 440 standard, as discussed in the literature review. Appendix A demonstrates the calculation of the flexural and shear capacity for specimen GT-2-50-16, while the calculation of the remaining beams was done using an Excel spreadsheet. As a result, all the beams have been found to have flexural strength higher than shear strength, as shown in Table 3.5.

Table 3.5: Flexural vs. shear ultimate strength based on ACI-440

Beam ID	Ultimate load based on flexural strength (kN)	Ultimate load based on shear strength (kN)
GT-2-50-16	225.91	33.80
GT-2.7-50-16	166.97	33.73
GT-3.5-50-16	127.03	33.93
GT-3.5-60-16	136.05	35.46
GT-3.5-40-16	119.31	32.19
GT-3.5-50-19	157.15	43.12
GT-3.5-50-13	107.68	28.28

3.6 Materials:

3.6.1 Concrete:

The concrete consisted of ordinary Portland cement, marine sand, 10 and 20 mm aggregate from Ras Al Khaimah, water, and admixture. The concrete design mix for each targeted compressive strength can be found in Table 3.6. Additionally, the mix was prepared in the eastern precast factory using a slump test of 125-220mm, and the beams were concretized daily. Therefore, six standard cube tests 150x150mm were prepared for each beam, divided into three pairs. Each pair was tested after 28 days, and the concrete compressive strengths are presented in Tables 3.2, 3.3, and 3.4

Table 3.6: Concrete Design mix

Targeted strength (MPa)	40	50	60
OPC (kg/m ³)	310	360	410
Marine sand (kg/m ³)	680	680	690
10 mm RAK (kg/m ³)	460	460	420
20 mm RAK (kg/m ³)	820	820	840
Water (kg/m ³)	148	136	131
XPC 39 (L/m ³)	3	2.4	-
RF 763 (L/m ³)	-	-	3

3.6.2 GFRP:

The high-tensile GFRP bars with sand coating (Figure 3.2) were imported from Industrial Control Solution Company in the Kingdom of Saudi Arabia. All the bars were straight with no bends at the ends, and their length was set to be the length of the beam minus 50 mm for concrete cover, resulting in 1.9 meters of bar. For each beam, two identical bars of 13 mm, 16 mm, and 19 mm in size were used as longitudinal tensile reinforcement, without any shear reinforcement. Based on the manufacturer's laboratory tests, Table 3.7 demonstrates the mechanical properties of each bar size.



Figure 3.2: Sand coated GFRP

Table 3.7: Mechanical properties of GFRP bars

Bar size	13 mm	16 mm	19 mm
Nominal Dia. (mm)	12.7	15.9	19.1
Nominal CSA (mm ²)	129	199	284
Ultimate tensile load (kN)	117	193	294
Ultimate tensile stress (MPa)	886	960	1038
Ultimate tensile strain (%)	1.95	2.2	1.91
Modulus of elasticity (GPa)	45	44	54
Transverse shear strength (MPa)	131	131	131
Unit weight (kg/m)	0.25	0.41	0.6

3.7 Instruments:

The load was applied using a hydraulic machine of 200 kN capacity. The bending test machine is designed to provide monotonic loads through two pressing points, but in this experiment, only one pressing point was used, and for beams with four-point loading, a steel spreader beam was used. The loads were measured by taking the readings from its built-up manometer by the lab specialists. A grease was applied continuously to the machine rail to ease the proper placement of the beam. A 50 mm dial gauge was used to measure the beam deflection with an accuracy of 0.01 mm. The dial gauge was placed at the mid-top of the beam for all the beams to avoid any damage after beam failure. In addition, the dial gauge readings were recorded by video camera throughout the loading process.

3.8 Beams Construction:

The beams were constructed in the Eastern precast factory, which adheres to the highest construction standards. The process began with building the T-shaped formwork using wooden sheets, as shown in Figure 3.3. The length of the formwork was two meters, and the dimensions of the sections are shown in Figure 3.1. A coating of oil was applied to the internal surface of the formwork to prevent water absorption and facilitate the removal of the formwork after the hardening of the concrete. Additionally, two millimeters of silicon were applied along the edges to

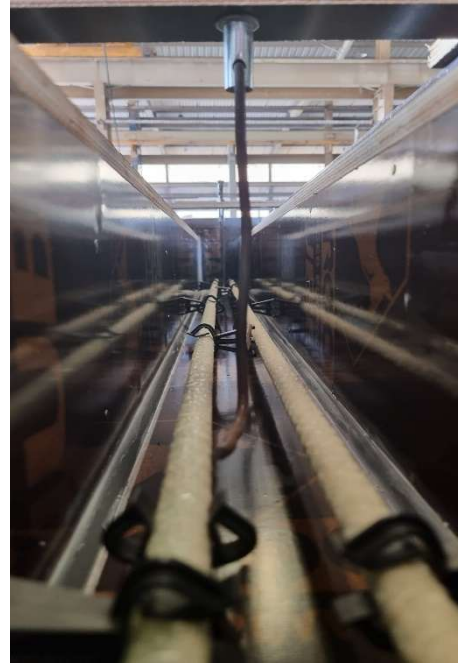
create a slight chamfer. Following the preparation of the wooden formwork and inspection of its dimensions, the GFRP bars were connected to a 40 mm plastic cover. Then, they were longitudinally laid out at the bottom of the beam. Besides that, a lifting thread with a diameter of 12 mm was placed 400 mm away from each edge, and no shear reinforcements were included (Figure 3.4). Meanwhile, the concrete was mixed using an automatic concrete mixer shown in Figure 3.5. The concrete was mixed according to the mix design shown in Table 3.6. Afterward, the planned beam of the day was cast in layers and with the proper use of vibrators (Figure 3.6). Furthermore, since each beam was cast on a different day, six cube tests were prepared for each beam to be tested after three, seven, and 28 days, and the average results are presented in Table 3.1. In addition, each beam was covered with hessian fabric and cured for seven days, as shown in Figure 3.7. After the curing was completed and the seven-day cube test results were found satisfactory, the beams were transported to the University of Bahrain campus, taking into consideration the highest levels of care and safety (Figure 3.8); therefore, no cracks occurred during the beam's transportation. Finally, the beams were painted white for crack marking, and a two-ton hand crane was used for beam mobilization, as seen in Figure 3.9.



Figure 3.3: T-beam formwork



(a)



(b)

Figure 3.4: Lifting thread: (a) positioning process, (b) cross-section view



Figure 3.5: Automatic concrete mixer used for concrete mixing



Figure 3.6: Beams Concreting process: (a) concrete batching, (b) concrete pouring using a bucket, (c) surface leveling and vibration, (d) completed beam after casting



Figure 3.7: Beams curing



Figure 3.8: Beams Transportation



Figure 3.9: Two-ton hand crane

3.9 Test Set Up:

The test was held in the University of Bahrain structural lab. The simply supported beams were loaded by a 200 kN hydraulic machine. Two loading methods were followed in this experiment based on the span-to-to-depth ratio (a/d); four-point loading was applied for specimens with a span-to-to-depth ratio (a/d) of 2 and 2.7, while three-point loading was applied for specimens with span-to-to-depth ratio (a/d) approximately equal to 3.5. The point loads used for the four-point loading were cylindrical steel bars welded to steel plate to prevent any movements as shown in Figure 3.10. In addition, a U shape beam is used as a spreader beam to divide the loads into two points for beams with a/d of 2.7, while a rectangular steel section was used for a/d of 2, and for beams with a/d of 3.5, the point load was applied to the beam trough a rectangular steel plate (Figure 3.11-3.16). Besides that, both ends of the beams were roller supports positioned 100 mm from the beam edge, so the effective beam length is considered 1800 mm. During the loading, the cracks were monitored and marked on the painted side of the beam every 5 kN increment. At the same time, the deflection was measured using a dial gauge placed at the top of the beam. The readings were taken based on the same loading increment. Video cameras were used to record the dial gauge readings and the cracks appearing during the test, however, a technical error occurred with the dial gauge camera during the testing of beam GT-3.5-60-16, resulting in a loss of the deflection readings of that beam. Nevertheless, the deflection at the ultimate load was measured. On the other hand, no strains were measured in this experiment.



Figure 3.10: Cylindrical steel bars welded to steel plates



Figure 3.11: Test set up of specimen with a/d of 2 (Lab photograph)

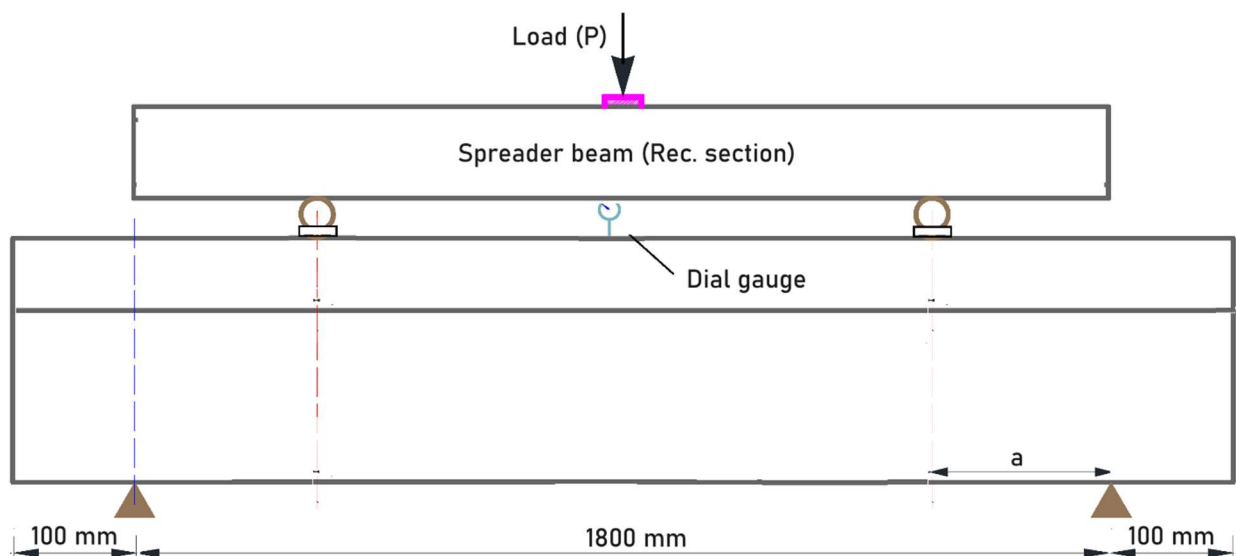


Figure 3.12: Test set up of specimen with a/d of 2

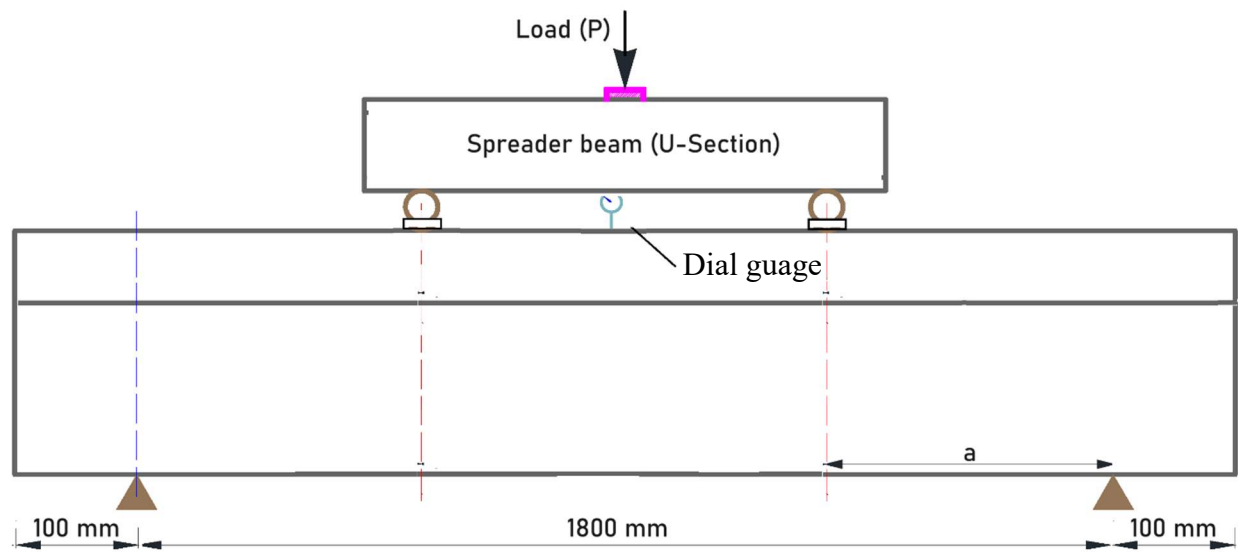


Figure 3.13: Test set up of specimen with a/d of 2.7

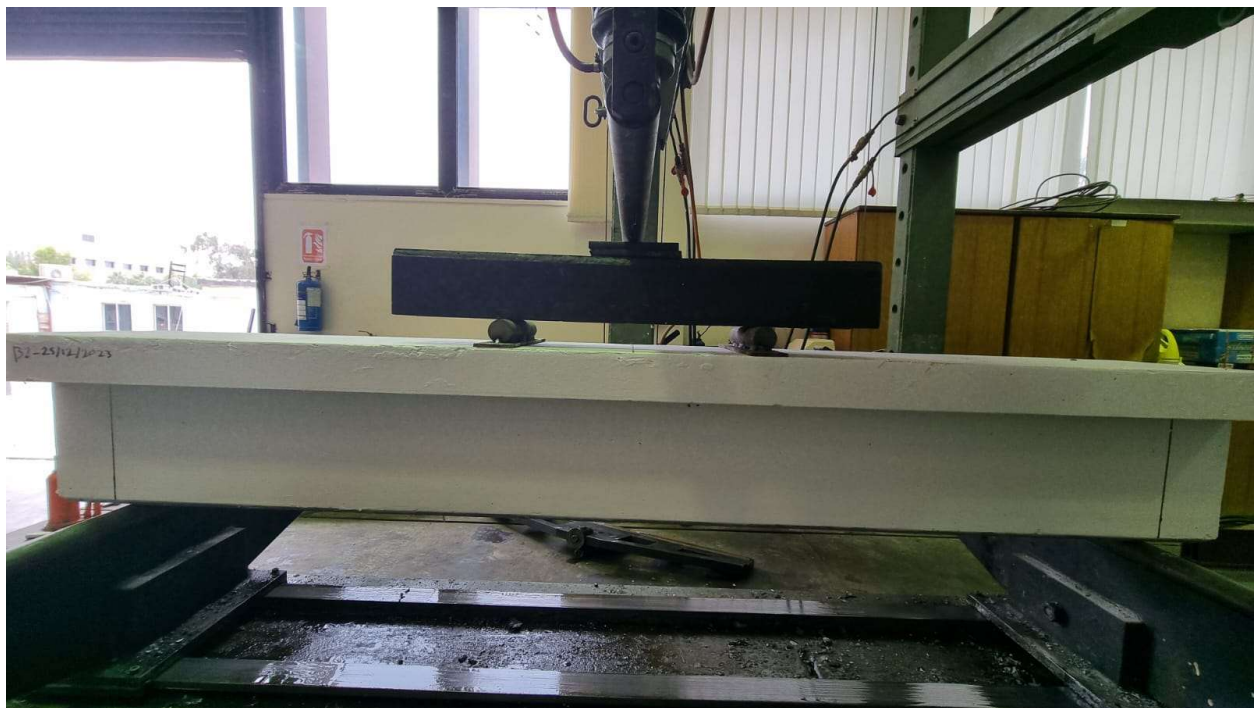


Figure 3.14: Test set up of specimen with a/d of 2.7 (Lab photograph)

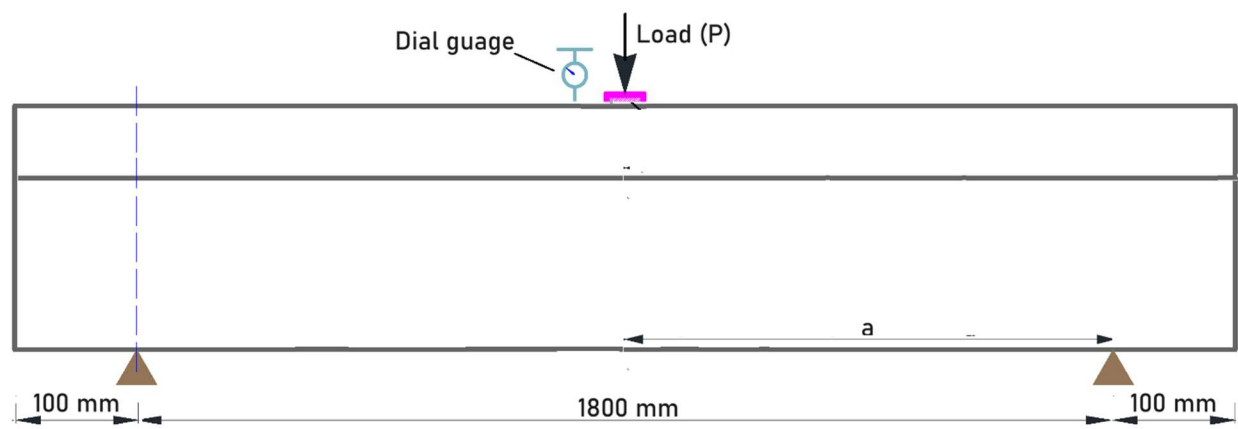


Figure 3.15: Test set up of specimen with a/d of 3.5

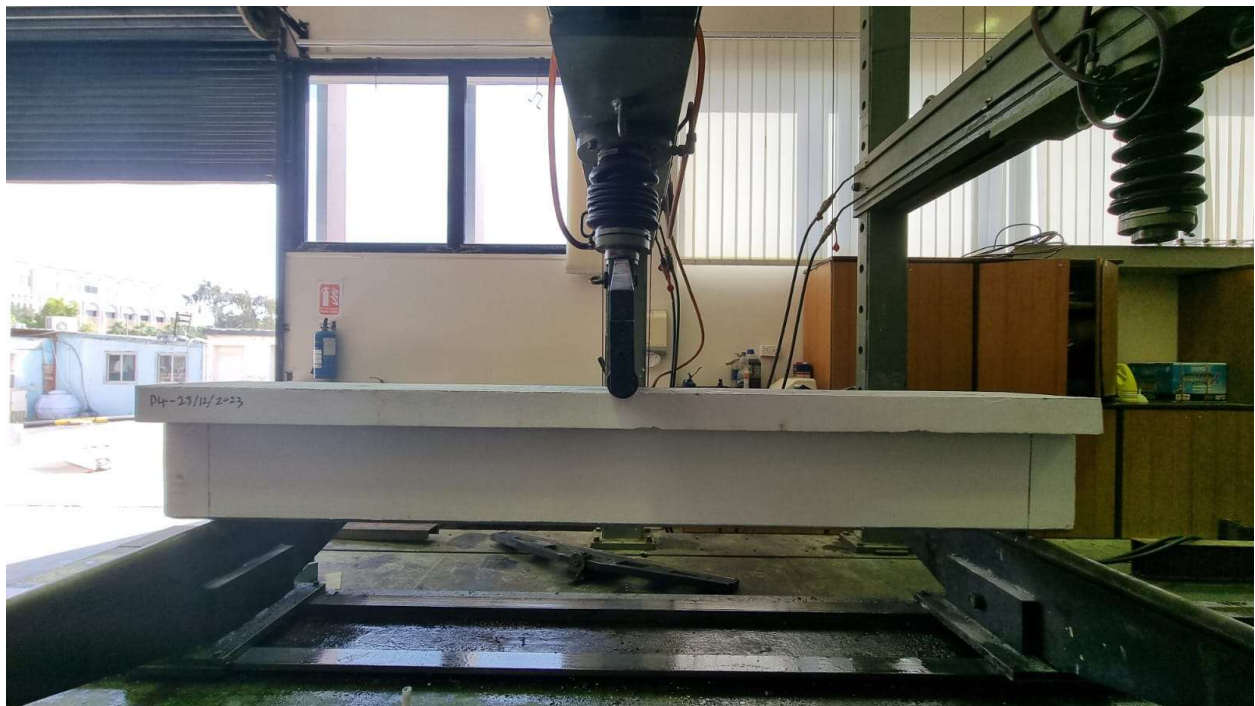


Figure 3.16: Test set up of specimen with a/d of 3.5 (Lab photograph)

Chapter Four

Experimental Results and Discussion

4.1 Introduction:

This chapter discusses the results of the tests conducted on the seven simply supported T-beams reinforced with GFRP without shear reinforcement. The seven beams were divided into three groups according to three variables: longitudinal reinforcement ratio, compressive strength of concrete, and shear span-to-effective depth ratio. Meanwhile, the beam's cross-sectional dimensions, length, and reinforcement type remained constant. The aim of the experiment is to investigate the shear strength of GFRP-reinforced T-beams and to compare it with the available design guides and models. In addition, this chapter discusses the effect of the three parameters on the mode of failure, crack pattern, and load-deflection behavior.

4.2 Cracking Behaviour:

Recording the crack patterns was carried out by marking the cracks every 5 kN load increment on the painted side of the beam; this method helped develop Figures 4.1, 4.3, and 4.4. The figures show cracks as they appeared in the actual test, and the bold line presents the path of the failure crack. In addition, the figure presents the crack's length, location, direction, slope, spacing, and the load causing each crack.

It is generally observed that the first crack has formed at the maximum flexural moment zone in all beams. As the load increased, more flexural cracks appeared at the mid-span, while diagonal cracks began to appear at the shear span area, due to shear stresses. With further loading, both flexural and diagonal cracks continued to propagate. It was noticed that some flexural cracks merged with the diagonal cracks and then followed the same path so as to approach the loading point. Among all the beams, the longest flexural crack has reached the flange except for GT-2.7-50-16, in which stopped at the bottom of the flange. In most specimens, the flexural cracks reached the flange at around 45-50% of the failure load. On the other hand, the diagonal cracks, which also known as shear cracks, have reached the flange, reflecting shear failure mode for all the beams. During the failure stage, in some beams, such as GT-2.7-50-16 and GT-3.5-60-16, the cracks that

caused the shear failure developed rapidly and appeared just seconds before they failed. In some beams, however, the failure resulted from cracks formed during the initial loading. Additionally, some particular beams, like GT-2.7-50-16 and GT-3.5-50-16, suffered sudden and rapid failures, as illustrated in Appendix B. Therefore, loading the remaining beams was applied cautiously to avoid catastrophic accidents. Generally, the failure cracks between the loading point and the support have approximately a 45-degree angle at the neutral axis. However, at some beam samples, the angle of the diagonal shear cracks varied from 45°, which could be due to the bond between the concrete and the aggregate being less than the aggregate strength. Moreover, as the cracks progressed toward failure, it was observed that the failure cracks extended in two directions simultaneously. In one direction, it extended horizontally back towards the support at the bottom of the beam in the form of shear splitting cracks. At the same time, it propagated towards the flange at the top of the beam, and once the flange is crossed, the direction is changed towards the loading point.

4.2.1 First Cracking Load:

It is well known that cracks occur once the tensile stress due to the applied loads exceeds the tensile strength of concrete. In this study, the cracking moment is predicted according to the transformed section method of the uncracked concrete stage using the following equation:

$$M_{crp} = \frac{f_r I_{NA}}{\bar{y}} \quad (4.1)$$

Where,

$$f_r = 0.6\sqrt{f_c'}$$

$$\bar{y} = \frac{\sum A_i y_i}{\sum A}$$

$$I_{NA} = \frac{b_f h_f^3}{12} + b_f h_f d_1^2 + \frac{b_w h_w^3}{12} + b_w h_w d_2^2 + (n - 1) A_s d_3^2$$

d_1, d_2 and d_3 are the distance between $\bar{y}$ and area's center.

The experimental cracking moment was found on the basis of the observed cracking load, taking into account the self-load of the beam with the concrete density of 23.6 kN/m³. It was observed that the first cracks were all vertical cracks, indicating that even without the shear

reinforcement, they were all caused by flexural moments rather than shear stresses. In beams with a/d of 3.5 and 2.7, the first crack developed at the center of the beam. However, with decreasing a/d to 2.5, the first crack of beam GT-2-50-16 was located below the loading point, which is at a distance from the specimen's center.

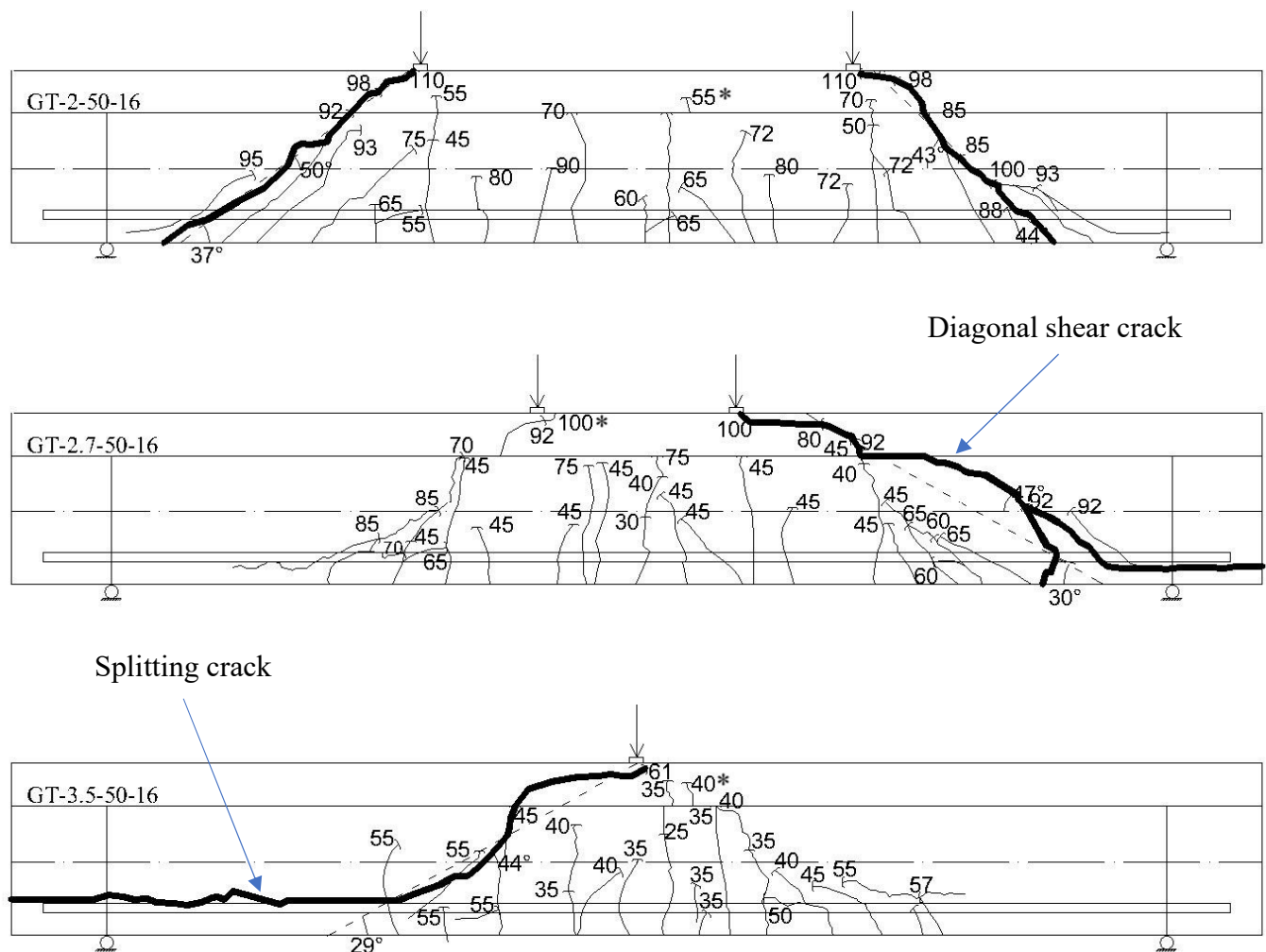
Table 4.1: Experimental and predicted first cracking loads

Beam ID	P_{cr-eq} (kN)	P_{cr-exp} (kN)	M_{cr-eq} (kN.m)	M_{cr-ex} (kN.m)
GT-2-50-16	43.2	45	11.42	11.88
GT-2.7-50-16	31.88	30	11.38	10.74
GT-3.5-50-16	24.4	25	11.5	11.79
GT-3.5-40-16	22.05	24	10.46	11.34
GT-3.5-60-16	26.47	27	12.45	12.69
GT-3.5-50-13	23.9	25	11.3	11.79
GT-3.5-50-19	24.27	25	11.46	11.79

Table 4.1 shows that the experimental cracking results are consistent with the predicted cracking loads and moments. An increase in the span-to-effective-depth ratio (a/d) has reduced the applied load, but the cracking moment has remained unchanged. Additionally, increasing the compressive strength from 40 to 50 MPa (25%) increased the cracking moment by 4 %, while an increment from 50 to 60 MPa (20%) caused an increase in the cracking moment by 7.6%. On the other hand, increasing the reinforcement ratio had no significant effect on the cracking moment, indicating that the first cracking moment is mainly influenced by the tensile stress in the concrete rather than the amount of longitudinal reinforcement.

4.2.2 Crack Patterns:

Figure 4.1 illustrates the crack pattern of group one, which has a different span-to-effective depth ratio. As mentioned earlier, the bold line represents the path of the failure crack, the inclined dashed line indicates the average slope of the failure crack, and the horizontal dash-dotted line presents the neutral axis. The figure shows that when the span-to-effective depth ratio (a/d) is increased, the number of cracks is slightly decreased. Since most of the cracks propagate toward the loading points, moving the loading points closer together reduced the cracks spacing.



*All values are in kN

Figure 4.1: Crack patterns for beams with different shear span-to-depth ratios (a/d)

In theory, all shear cracks occur vertically at the bottom of the beam. This is because, at the bottom, the flexural stresses are most significant while the shear stresses are minimal. However, with the propagation of shear cracks towards the neutral axis, the shear stress increases, as shown

in Figure 4.2, causing the cracks to incline. Despite this, it appears that the failure cracks at the beam bottom are ending horizontally rather than vertically, a phenomenon that can be attributed to the fact that some parts in the concrete section are unable to resist the shear stresses, causing the shear to be transferred to the longitudinal reinforcement. When the beam is subjected to vertical loading, the support reaction force creates stress trajectories that redirect inclined shear cracks to propagate horizontally toward the support. This phenomenon is known as shear splitting. The shear splitting crack is a horizontal crack near the support, caused by high shear forces and diagonal compression struts that generate transverse tensile stresses. When these stresses exceed the concrete's tensile capacity, the beam splits along the reinforcement level. This crack indicates a shear-dominated failure, not flexure, and typically forms in the anchorage or support zone (Suchorzewski et al. 2017).

Furthermore, it is noticeable that with increasing a/d , the angle of the failure crack decreases along with its siblings. Moreover, the failure cracks of beams GT-2.7-50-16 and GT-3.5-50-16 have approximately similar shape, which agrees with Tureyen & Frosch's (2003) test results, except in beam GT-2-50-16, which did not exhibit a shear splitting crack due to arch actions. Additionally, although the load location in beams GT-2-50-16 and GT-2.7-50-16 is different, the crack failure of both beams intersects with the GFRP approximately at the same point. Nevertheless, further investigations are required to confirm this point.

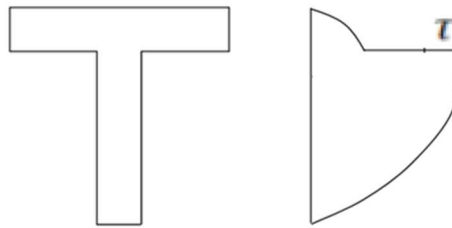
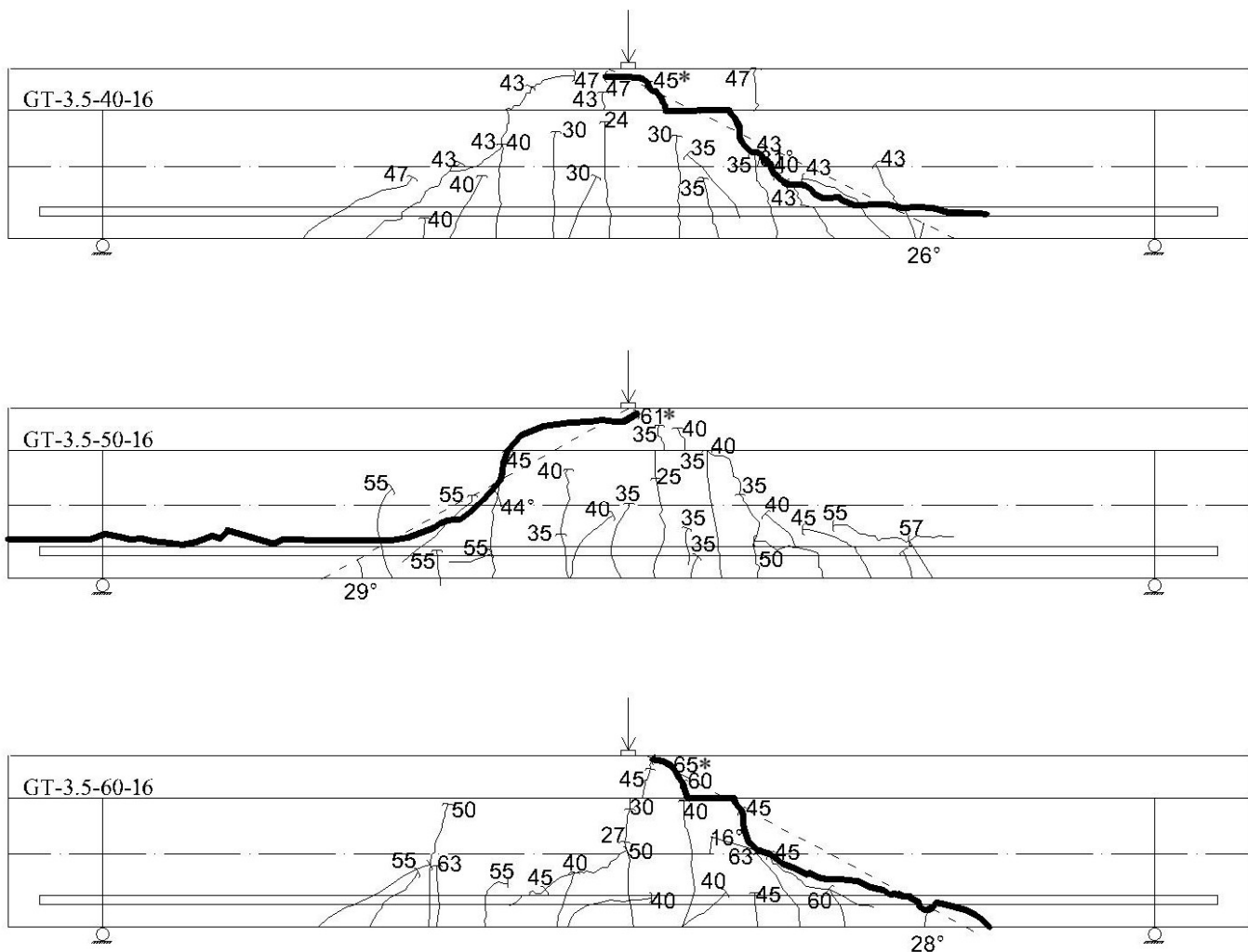


Figure 4.2: Shear stress distribution for T-beam sections

The effect of compressive strength on the crack pattern is presented in Figure 4.3. As can be seen, the compressive strength increased up to 50%, but the crack spacing and the number of cracks did not vary significantly. Both beams, GT-3.5-40-16 and GT-3.5-50-16, exhibited horizontal failure cracks. However, the horizontal crack of GT-3.5-40-16 did not extend further because the loading was stopped to prevent the sudden rapid failure that occurred in beams GT-2.7-50-16 and GT-3.5-50-16. In contrast, the beam with 60 MPa compressive strength showed no indication of a horizontal crack propagating toward the support. Additionally, the angle of the failure crack did not change significantly, causing the shear failure cracks of the three beams to interfere with the GFRP reinforcement nearly at the same distance from the loading point.



*all values are in kN

Figure 4.3: Crack patterns for beams with different concrete compressive strengths (f'_c)

Figure 4.4 shows the effect of the reinforcement ratio on the crack pattern. During the test of beam GT-3.5-50-13, the first crack was observed at 25 kN, and then it lengthened and grew in size rather than new cracks propagating as the load increased. After 40 kN of loading, the flexural crack almost reached the top of the flange, and there was no further possibility of propagation, so other cracks, along with the diagonal cracks, started to appear. After that, at 57 kN, the diagonal crack size became superior over flexural cracks, indicating a shear failure. In other words, due to the low reinforcement ratio, the crack behavior of the beam below 40 kN loading was similar to the behavior of plain concrete in the flexural strength test, however, due to the presence of 13 mm diameter GFRP longitudinal reinforcement, the flexural failure was prevented. Consequently, the number of cracks and spacing in beam GT-3.5-50-13 was less than in beam GT-3.5-50-16. In addition, the concentration of the cracks below the loading point in beam GT-3.5-50-13 indicates that the strain in the reinforcement is also concentrated at the mid-span. As can be seen, some of the cracks that appeared in beam GT-3.5-50-13 did not propagate upward, but instead merged with existing cracks. Alam (2010) also observed this behavior and attributed it to a possible higher tension stiffening effect caused by the low reinforcement ratio, bond degradation adjacent to the cracks, and the limitation in redistributing the internal forces. On the other hand, the increase in reinforcement ratio from 1.1 to 1.5% significantly reduced the number of cracks and increased crack spacing, indicating that the strain in the longitudinal reinforcement is approximately evenly distributed. This observation is in good agreement with the crack pattern of rectangular beams reinforced with GFRP and without shear reinforcement obtained by Menam et al. (2021). Finally, all three beams exhibited horizontal failure cracks, and the average angle failure crack varied between the ratios, with beam GT-3.5-50-13 having the steepest crack failure slope, followed by beam GT-3.5-50-19.

4.3 Load Deflection Behavior:

As discussed in Chapter 3, the deflection was measured at the mid-top of the beam using a 50 mm dial gauge with 0.01 mm accuracy. Besides that, the deflection measurements were taken every 5 kN increment. In some beams, the gauge was removed before the failure to avoid it being damaged. In general, all the beams exhibited approximately the same linear behavior before cracking with low deflection values, indicating that different a/d , f_c' , and ρ do not affect the deflection behavior noticeably before cracking. After cracking, the load-deflection behavior varied linearly based on the variable of each group. Figure 4.5 illustrates the load-deflection behavior of group 1 beams with different a/d values. The results show that increasing a/d made the deflection more sensitive to the applied load, resulting in steeper slopes. The beam with an a/d ratio of 2 (GT-2-50-16) exhibited the least deflection under increasing load, indicating higher stiffness compared to the other specimens. It was followed by the beam with an a/d ratio of 2.7 (GT-2.7-50-16), while the beam with $a/d = 3.5$ (GT-3.5-50-16) showed the greatest midspan deflection under the same loading conditions. At the failure load of the beam with $a/d = 3.5$, the deflection was found to decrease by 34% and 66% when the a/d ratio was reduced to 2.7 and 2, respectively. Furthermore, decreasing the a/d value from 2.7 to 2 resulted in a 38% reduction in deflection at 75 kN, which corresponds to the failure load of the specimen with a/d of 2.7. This trend confirms that a lower a/d ratio enhances beam stiffness and shear resistance, resulting in reduced deflections at comparable load levels.

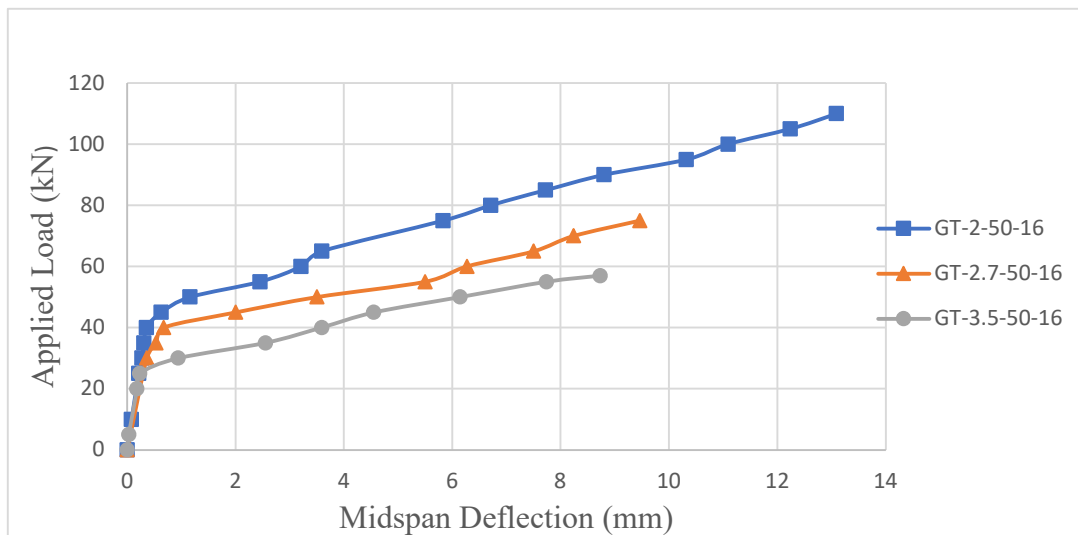


Figure 4.5: Experimental load-deflection behavior for beams with different shear span-to-depth ratios (a/d)

For the second group with different concrete compressive strengths, Figure 4.6 shows that increasing the compressive strength decreases the deflection values after cracking, indicating an increase in the stiffness of the beam. Moreover, under the maximum load of the beam with a compressive strength of 40 MPa, the midspan deflection was 41% greater than that of the beam with a compressive strength of 50 MPa.

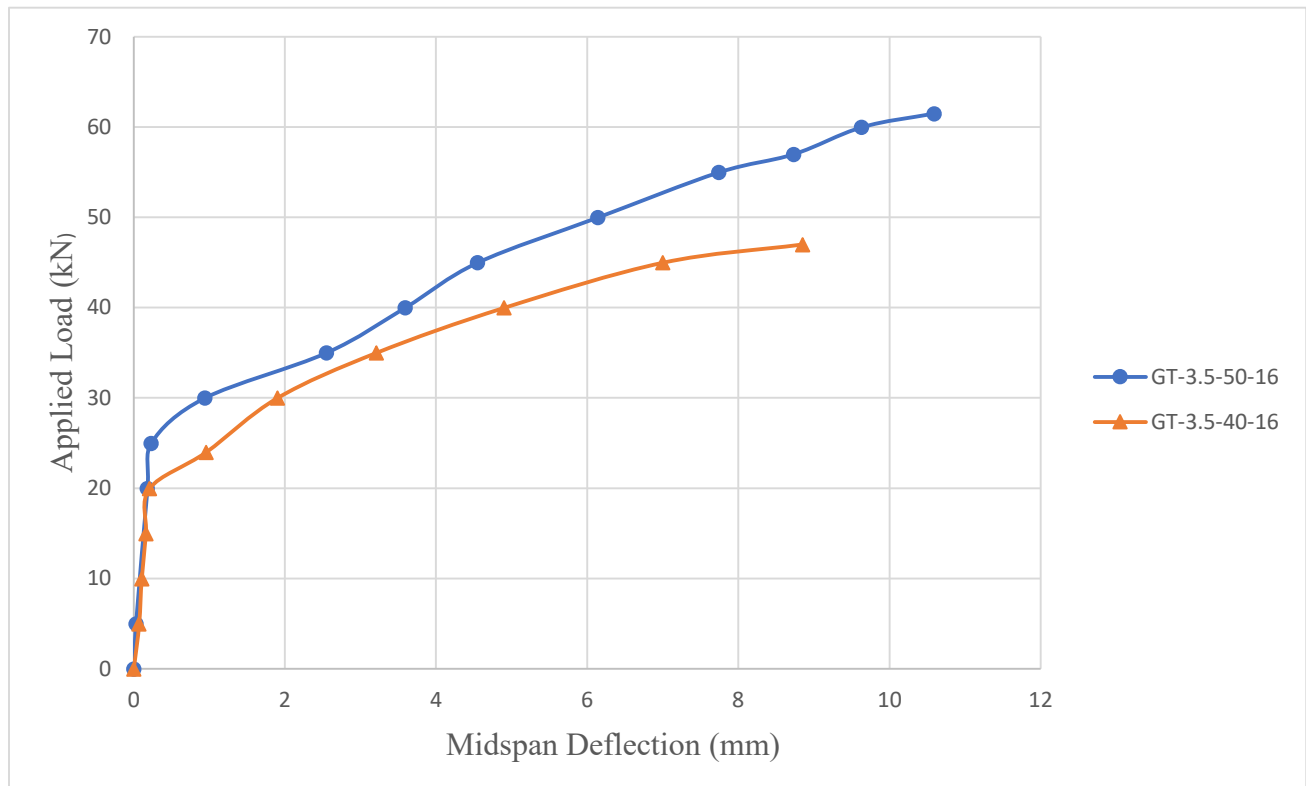


Figure 4.6: Experimental load-deflection behaviour for beams with different concrete compressive strengths (f'_c)

In addition, the effect of reinforcement ratio on the load–deflection behavior is illustrated in Figure 4.7. The figure clearly shows that increasing the reinforcement ratio from 0.007 to 0.011 results in a significant reduction in deflection. At a load of 57 kN, the deflection decreased by approximately 30% when increasing the reinforcement ratio from 0.007 to 0.011, and by about 60% when increased to 0.015. Furthermore, at a load level of 60 kN, increasing the reinforcement ratio from 0.011 to 0.015 resulted in a 45% reduction in deflection. These findings highlight the

strong correlation between higher GFRP reinforcement ratios and improved structural stiffness, demonstrating that greater longitudinal reinforcement ratio effectively limits deflection

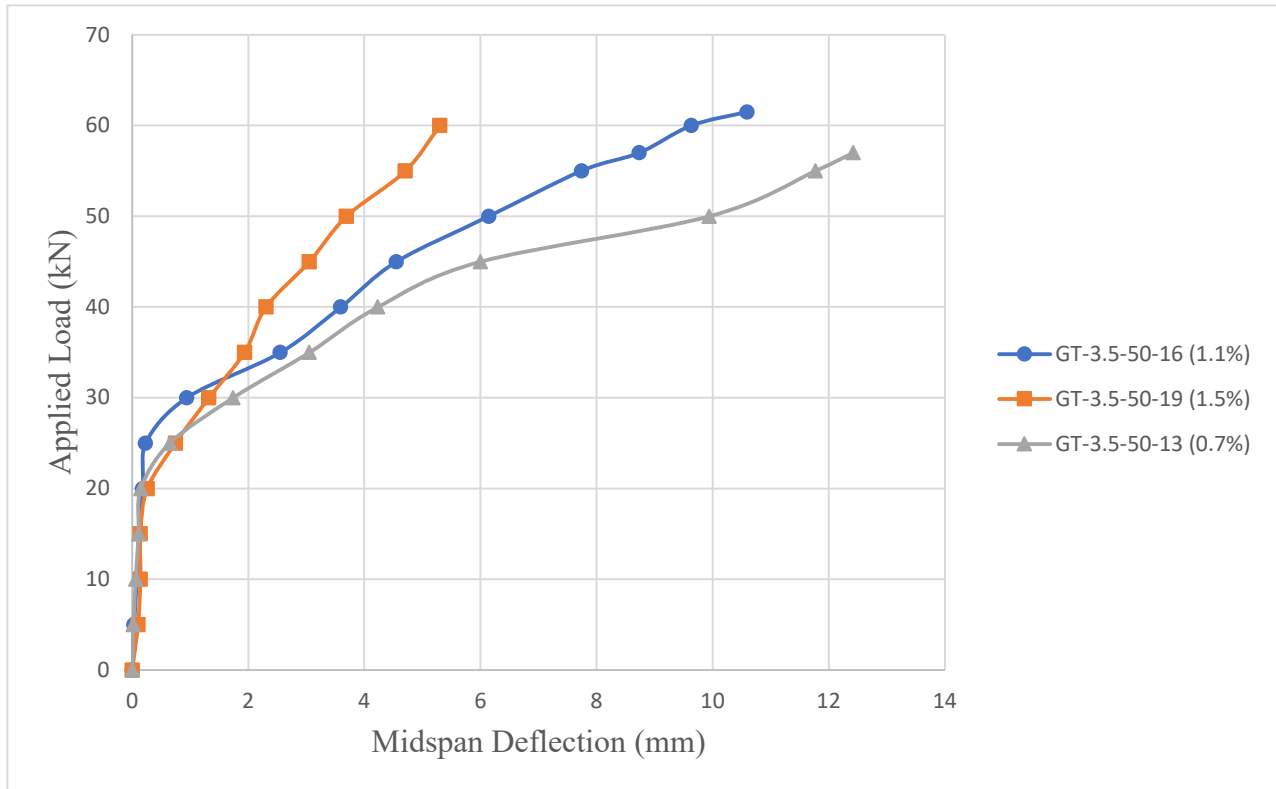


Figure 4.7: Experimental load-deflection behavior for beams with different reinforcement ratios (ρ)

4.4 Shear Capacity:

Table 4.2 presents the experimental shear capacity results for Group One, which includes beams with varying span-to-effective-depth (a/d) ratios. In this study, the shear capacity is defined as half of the maximum applied load at failure. The test results indicate that increasing the a/d ratio from 2.0 to 2.7 (35% increase) led to a 9.09% reduction in shear capacity. A further increase to an a/d of 3.57 (78.5% increase) resulted in a 44.1% reduction. As illustrated in Figure 4.8, the influence of the a/d ratio on shear strength becomes increasingly significant as the ratio increases. This trend aligns with findings reported by Mohamed (2019), who observed similar behavior in rectangular beams. Conversely, Widjaja et al. (2019) reported that the effect of a/d ratio on the shear strength of rectangular beams diminishes at higher values.

Table 4.2: Experimental shear strengths for beams with different shear span-to-depth ratios (a/d)

Beam ID	a/d	f_c' (MPa)	ρ (%)	V_{exp} (kN)	a/d percent increase (%)	Load percent decrease (%)
T-2-50-16	2	50.98	1.1	55	-	-
GT-2.7-50-16	2.7	50.58	1.1	50	35.00	9.09
GT-3.5-50-16	3.5	51.71	1.1	30.75	78.5	44.1

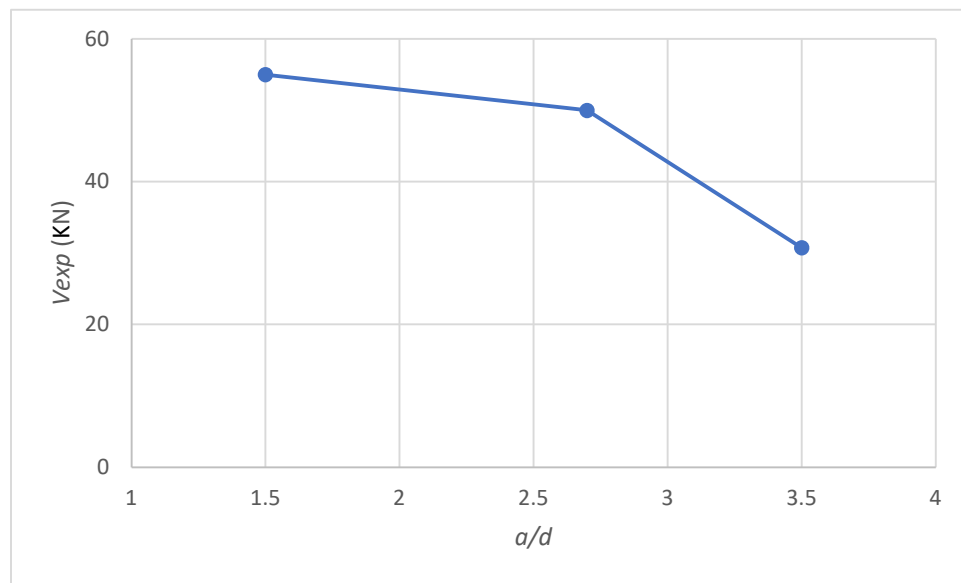


Figure 4.8: Effect of shear span-to-depth ratio on shear strength

In addition, the results of Group Two with different compressive strengths show that increasing the compressive strength of concrete (f_c') noticeably affected the shear capacity of the T-beams, as Table 4.3 illustrates. According to the results, increasing the f_c' from 42 to 51 MPa (21% increase) increased the shear strength of the T-beam by approximately 30%, while a further increase to 61 MPa (42.9% increase) increased the shear strength by only 38.3%. This illustrates that as the compressive strength increases, its influence on the shear strength decreases, as shown in Figure 4.9. A similar trend was observed in a study conducted by Alam (2010) to investigate the shear behavior of simply supported rectangular beams reinforced with GFRP. This supports the conclusion that the relation between the compressive strength of concrete and the shear strength is not linear. However, the author found that increasing the compressive strength from 40 to 65 MPa (62.5% increase) increased the shear strength by 23.9%, which suggests that the concrete compressive strength has a greater impact on shear strength in T-beams than rectangular beams.

Table 4.3: Experimental shear strengths for beams with different concrete strengths (f_c')

Beam ID	a/d	f_c' (MPa)	ρ (%)	V_{exp} (kN)	f_c' percent increase (%)	Load percent increase (%)
GT-3.5-40-16	3.57	42.61	1.1	23.5	-	-
GT-3.5-50-16	3.57	51.71	1.1	30.75	21.36	30.85
GT-3.5-60-16	3.57	60.88	1.1	32.5	42.9	38.3

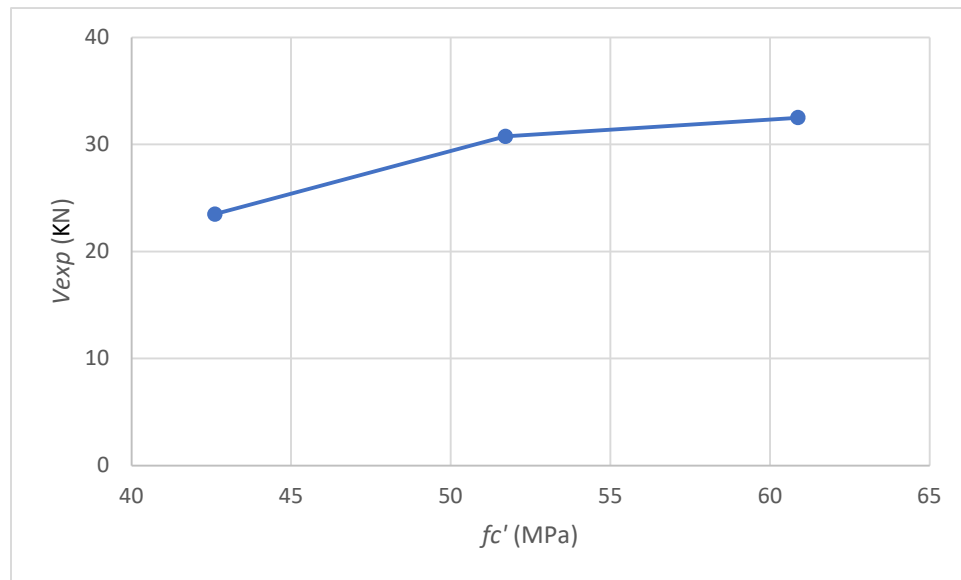


Figure 4.9: Effect of concrete strength on shear strength

Furthermore, Table 4.4 represents the shear strengths for T-beams with different reinforcement ratios. The increase of the reinforcement ratio from 0.7 to 1.1% (57%) and to 1.5% (114.3%) increased the shear strength only by 7.9 and 10.53%, respectively. This could be attributed to the placement of the bars in the bottom zone, where shear stress is minimal. Additionally, the contribution of GFRP as longitudinal reinforcement to the shear resistance is limited due to its low stiffness and limited dowel action, which results from its low modulus of elasticity (ACI 440.1R-15). Similarly, Alam (2010) recorded that increasing the longitudinal reinforcement ratio from 0.86 to 1.43 % (66.3%) only increased the shear capacity by 7.3%. Besides that, another study by Tureyen & Frosch's (2002) showed that increasing the GFRP longitudinal reinforcement ratio in a rectangular beam sample by 113% increased the shear capacity by 26%. Therefore, it can be noticed that the influence of the GFRP longitudinal reinforcement ratio on the shear strength in T-sections and rectangular sections is less than the effect of the effective depth-to-span ratio and the compressive strength of concrete, as illustrated in Figure 4.10.

Table 4.4: Experimental shear strengths for beams with different reinforcement ratios (ρ)

Beam ID	a/d	f_c' (MPa)	ρ (%)	V_{exp} (kN)	p percent increase (%)	Load percent increase (%)
GT-3.5-50-13	3.55	50.13	0.7	28.5	0.00	0.00
GT-3.5-50-16	3.57	51.72	1.1	30.75	57.14	7.89
GT-3.5-50-19	3.59	50.28	1.5	31.5	114.3	10.53

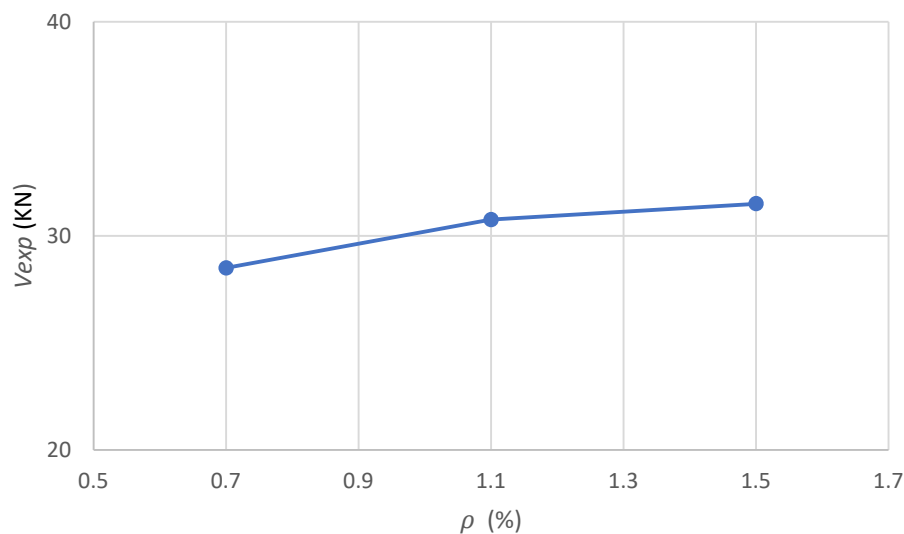


Figure 4.10: Effect of reinforcement ratio on shear strength

4.5 Code Comparison:

To study the applicability of the current major standards on T-beam reinforced with GFRP, the experimental results were compared with the predicted shear capacity by The American Concrete Institute (ACI 440.1R-15), Japan Society of Civil Engineers (JSCE 1997), Canadian Guideline (CSA S806-12) and ISIS Canada Research Network (ISIS 2007). Additionally, Alam (2010) and Tureyen et al. (2006) design methods have been included in the comparison, and the results are shown in Table 4.5. This study does not account for strength reduction factors, resistance factors of the materials, as well as the limitations on concrete strength, reinforcement ratio, and beam size.

Table 4.5: Experimental and code predictions for shear strength

Beam ID	V_{exp} (kN)	ACI 440.1R-15 (kN)	CSA S806-12 (kN)	ISIS (2007) (kN)	JSCE (1997) (kN)	Tureyen et al. (2006) (kN)	Alam (2010) (kN)
GT-2-50-16	55	16.90	48.82	25.28	24.36	23.90	41.76
GT-2.7-50-16	50	16.86	33.52	25.18	24.29	23.85	34.05
GT-3.5-50-16	30.75	16.96	29.36	25.46	24.47	23.99	28.57
GT-3.5-60-16	32.5	17.73	31.01	27.63	25.84	25.07	31.00
GT-3.5-40-16	23.5	16.09	27.53	23.11	22.94	22.76	25.94
GT-3.5-50-19	31.5	21.56	34.09	27.75	29.11	30.49	33.71
GT-3.5-50-13	28.5	14.14	26.16	25.62	21.37	20.00	24.90

According to Table 4.6, CSA S806-12 provides the best results compared to all other design standards. In this standard, the mean of the experimental to the calculated shear ratio (V_{exp}/V_{calc}) is 1.08, with a 20% standard deviation and an 18.8% coefficient of variation. In comparison with all other standards, these values are the lowest, indicating that the CSA S806-12 shear prediction is the most accurate and consistent. This can be attributed to the fact that CSA S806-12 takes into account the concrete compressive strength, reinforcement ratio, and the shear-span-to-depth ratio. The standard, however, overestimated the shear strength of beams GT-3.5-40-16 and GT-3.5-50-19, whose V_{exp}/V_{calc} values were 0.85 and 0.92, respectively.

As can be seen from Figures 4.11-4.17, the results obtained by JSCE (1997) and ISIS (2007) methods are approximately similar for all beam samples. Based on the results of both methods, the mean value of V_{exp}/V_{calc} is approximately 1.4, with a standard deviation of 47% and

a coefficient of variation of 33%. This implies that they are less accurate and consistent than the Canadian Standard S806-12. The inconsistency of both methods appeared significantly in members with a/d of 2 and 2.7. This is clearly because neither standard considers the shear-span-to-depth ratio. Nevertheless, both methods did not provide unsafe results, and the remaining beams with a/d of approximately 3.5 were not highly underestimated since their V_{exp}/V_{calc} ratio was not far from 1.

Furthermore, ACI 440.1R-15 has the lowest accuracy and consistency among all the other design standards. Based on the results of this method, the average and standard deviation are 2.11 and 71%, respectively. The method did not show any unsafe results. However, the majority of the results were highly unconservative. On the other hand, Tureyen et al. (2006) model for T-beams showed a slight improvement in accuracy and consistency compared with ACI 440.1R-15. However, this model provides higher conservative results than CSA S806-12, JSCE (2007), and ISIS (2007) methods. The average and standard deviation of the proposed model are 1.5 and 50%, respectively. In addition, Alam's (2010) proposed equation showed similar results to those of CSA S806-12. The average and standard deviation of the proposed model are 1.1 and 20%, respectively. This indicates that the accuracy and consistency of this equation are higher than JSCE, ISIS, and ACI 440.1R-15. However, similar to CSA S806-12, this model also provided unsafe results for beams GT-3.5-40-16 and GT-3.5-50-19.

Table 4.6: Comparison between experimental results and different design codes and models

Beam ID	V_{u-exp} (KN)	ACI 440.1R-15	CSA S806-12	ISIS (2007)	JSCE (1997)	Tureyen et al. (2006)	Alam (2010)
GT-2-50-16	55	3.25	1.13	2.18	2.26	2.30	1.32
GT-2.7-50-16	50	2.97	1.49	1.99	2.06	2.10	1.47
GT-3.5-50-16	30.75	1.81	1.05	1.21	1.26	1.28	1.08
GT-3.5-60-16	32.5	1.83	1.05	1.18	1.26	1.30	1.05
GT-3.5-40-16	23.5	1.46	0.85	1.02	1.02	1.03	0.91
GT-3.5-50-19	31.5	1.46	0.92	1.14	1.08	1.03	0.93
GT-3.5-50-13	28.5	2.02	1.09	1.11	1.33	1.43	1.14
Average		2.11	1.08	1.40	1.47	1.50	1.13
Standard deviation		0.71	0.20	0.47	0.49	0.50	0.20
COV %		33.77	18.82	33.61	33.22	33.77	18.01
% of unsafe results		0.00	28.57	0.00	0.00	0.00	28.57

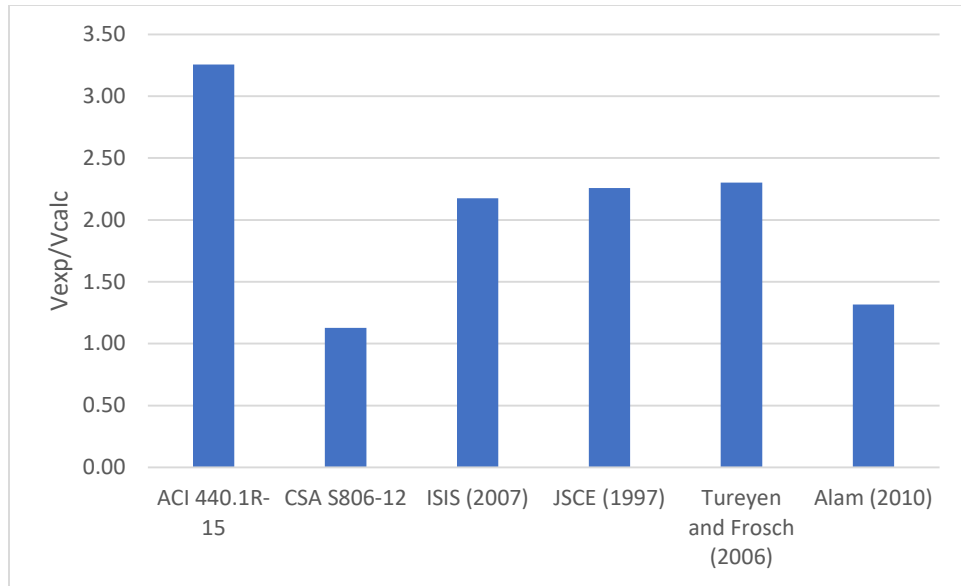


Figure 4.11: Comparison of the experimental shear strength of beam GT-2-50-16 with different design codes and models

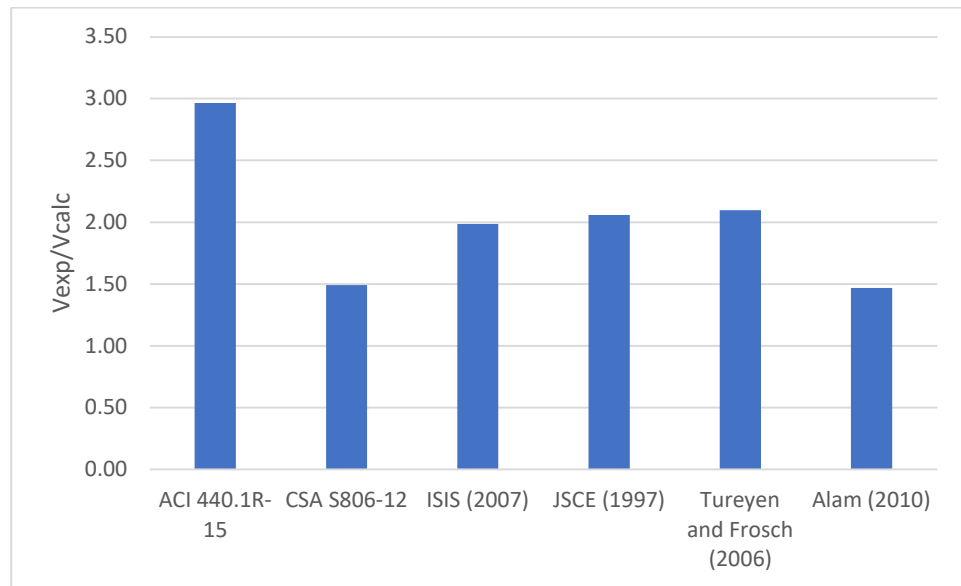


Figure 4.12: Comparison of the experimental shear strength of beam GT-2.7-50-16 with different design codes and models

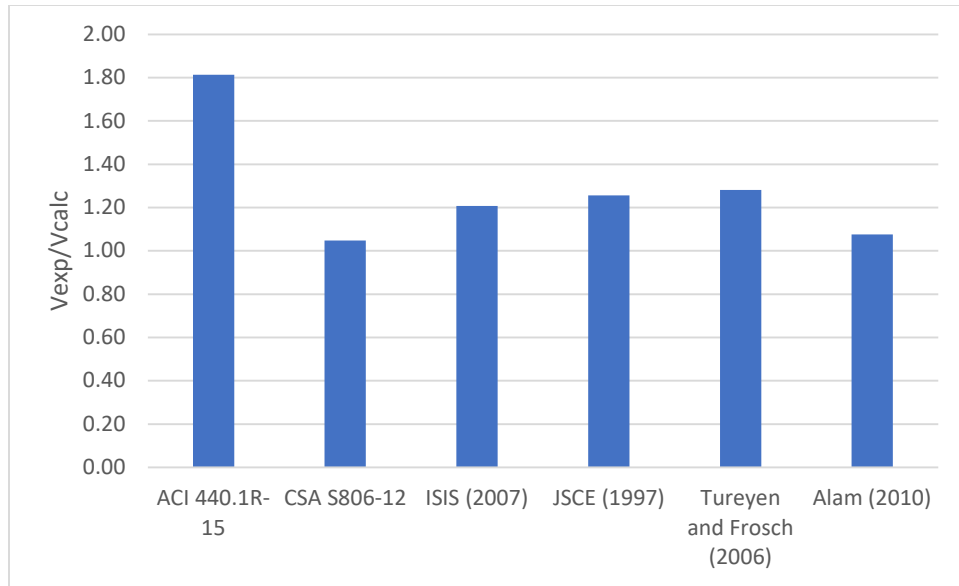


Figure 4.13: Comparison of the experimental shear strength of beam GT-3.5-50-16 with different design codes and models

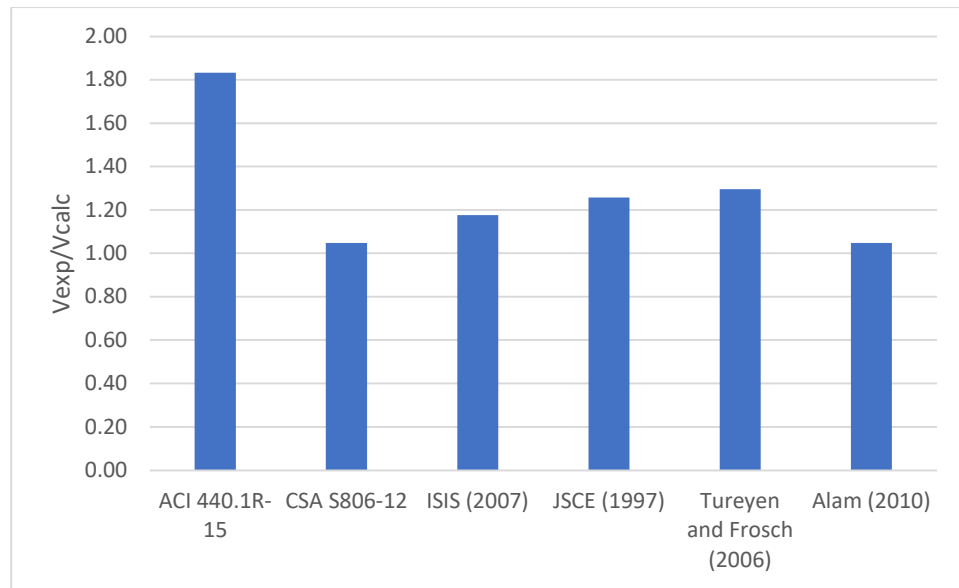


Figure 4.14: Comparison of the experimental shear strength of beam GT-3.5-60-16 with different design codes and models

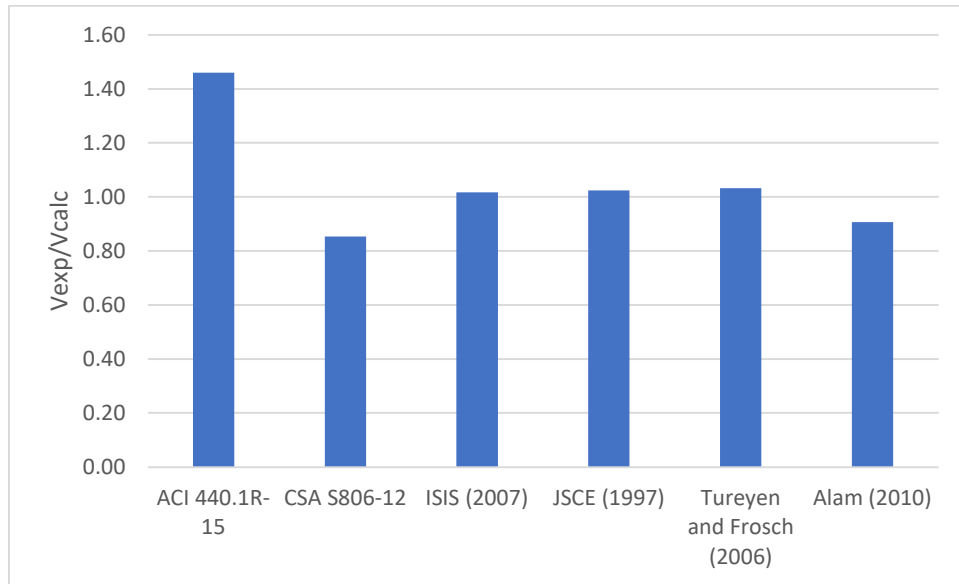


Figure 4.15: Comparison of the experimental shear strength of beam GT-3.5-40-16 with different design codes and models

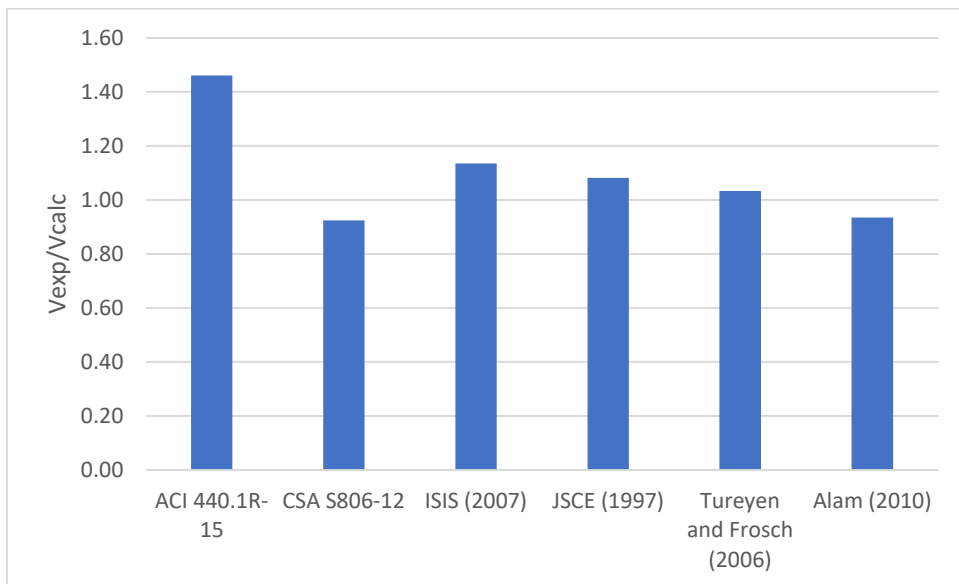


Figure 4.16: Comparison of the experimental shear strength of beam GT-3.5-50-19 with different design codes and models

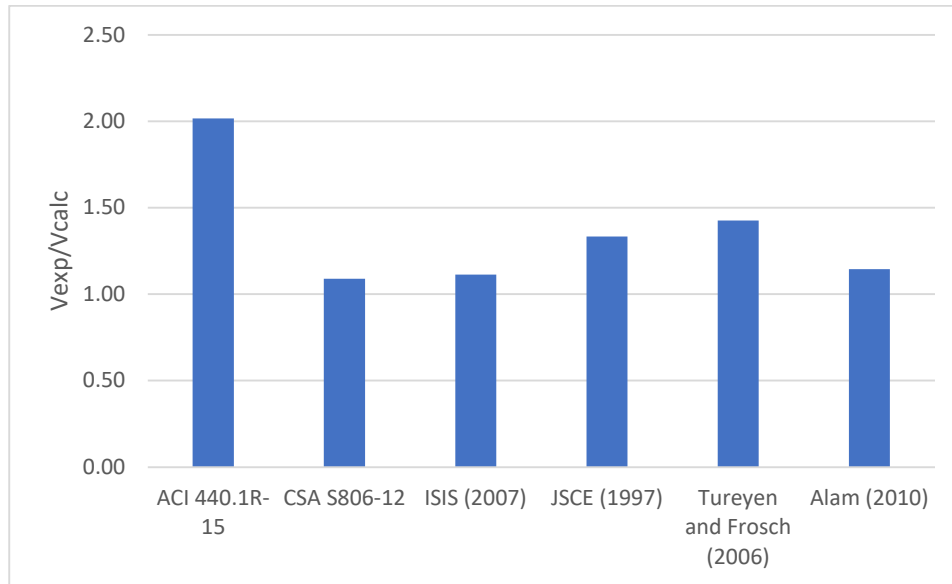


Figure 4.17: Comparison of the experimental shear strength of beam GT-3.5-50-13 with different design codes and models

Chapter Five

Finite Element Analysis

5.1 Introduction:

This chapter presents the analysis performed on all the beams tested in this study. A nonlinear finite element program, Abaqus, is used to model the seven T-beams reinforced with GFRP as longitudinal reinforcement. During the simulation, all the beams have the same properties and parameters as those used in the laboratory experiment. The present study uses one concrete material model, specifically the concrete damage plasticity model, but three values of concrete compressive strength are defined. Moreover, the compressive stress-strain curve is derived based on an expression generalized by Thorenfeldt et al. (1987), while the tension-stiffening model is derived using the model proposed by Alam and Hussein (2020) and Hordijk (1991). A comparison between the software and the experimental results contributes to defining the proper value for the input parameters, thereby improving the accuracy of the results and the software's precision. Therefore, this chapter aims to compare the experimental results with the finite element results obtained by Abaqus in terms of crack behavior, deflection, and shear strength.

5.2 Concrete Properties:

5.2.1 The Behavior of Concrete Under Compressive Stress:

Concrete is said to be loaded uniaxially when it is loaded throughout one axis perpendicular to its surface area. Therefore, the concrete compressive strength measured by the cylindrical compression test generally refers to uniaxial loading. Figure 5.1 shows a typical stress-strain curve for concrete subjected to uniaxial compressive stress. The curve shows that the stress-strain relation starts elastically linear. Some studies state that the linear stress-strain relation continues up to 30% of the ultimate compressive strength (MacGregor and Wight 2012). After the yielding point, the plastic behavior begins with an increase in the yield point, indicating a stress hardening until the ultimate stress value; beyond that value, a descending yield stress value starts, presenting strain softening. In this study, the compressive stress-strain curve is obtained based on an expression generalized by Thorenfeldt et al. (1987) as follows:

(5.1)

$$\frac{f_c}{f_c'} = \frac{n_c(\varepsilon_c/\varepsilon_0)}{n_c - 1 + (\varepsilon_c/\varepsilon_0)^{nk}}$$

Where,

f_c' = ultimate compressive strength from cylinder test

E_c = initial tangent modulus (Figure 5.1)

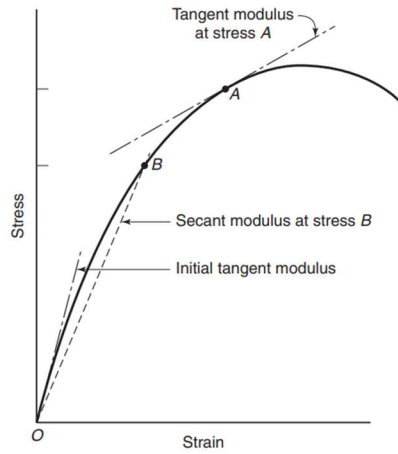


Figure 5.1: Typical compressive stress-strain curve of concrete (MacGregor and Wight 2012)

There are several ways to calculate the strain at ultimate stress (ε_0), the curve-fitting factor (n_c), and the stress decay factor (k). The equations are given by Collins & Mitchell (1991) are used in this study as follows:

$$\varepsilon_0 = \frac{f_c'}{E_c} \left(\frac{n-1}{n} \right) \quad (5.2)$$

$$n_c = 0.8 + \left(\frac{f_c'}{17} \right) \quad (5.3)$$

k is a controlling factor for the ascending and descending slopes. It is equal to 1 if $\varepsilon_c/\varepsilon_0$ ratio is less than 1, and if $\varepsilon_c/\varepsilon_0$ ratio is greater than 1, the k factor is calculated as the following:

$$k = 0.67 + \left(\frac{f_c'}{62}\right) \geq 1.0 \quad (5.4)$$

In addition, the initial tangent modulus (E_c) is obtained based on ACI 318 as equation 5.5 shows.

$$E_c = 4700\sqrt{f_c'} \quad (5.5)$$

5.2.2 The Behavior of Concrete Under Tensile Stress:

Concrete stiffening is the capability of concrete to resist tension between cracks and contribute to the stiffness of the member (ACI 224.2R-92). In this study, two methods were used to simulate the tension-stiffening behavior of concrete in Abaqus. The first method was based on stress-cracking displacement data (fracture energy approach) obtained according to Hordijk (1991) as discussed in the literature. The second method was stress-cracking strain data (Concrete softening behavior) based on the proposed model by Alam and Hussein (2020). This approach was developed based on Cope et al. (1979) simplified bilinear tension stiffening model. The proposed model is as follows:

$$f_t = \begin{cases} f_t' \frac{\varepsilon}{\varepsilon_{cr}} & \text{for } \varepsilon < \varepsilon_{cr} \\ \frac{f_t'}{10} (11 - \varepsilon/\varepsilon_{cr}) & \text{for } \varepsilon_{cr} < \varepsilon < \varepsilon_1 \\ \frac{f_t'}{2} \left(\frac{\varepsilon - \varepsilon_1}{\varepsilon_1 - \varepsilon_2} + 1 \right) & \text{for } \varepsilon_1 < \varepsilon < \varepsilon_2 \end{cases} \quad (5.6)$$

Where,

$$\varepsilon_1 = 6\varepsilon_{cr}$$

$$\varepsilon_2 = k_3\varepsilon_u, \text{ and } \varepsilon_u = \text{the ultimate strain of GFRP bars.}$$

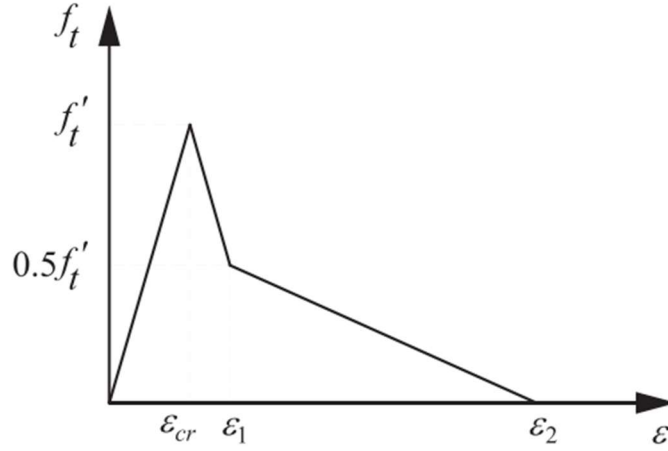


Figure 5.2: Tension stiffening model for FRP reinforced members (Alam and Hussein 2020)

Figure 5.2 shows the behavior of concrete under uniaxial tensile stress. It can be noted that the concrete is assumed linear until the cracking stress (f_t'). After that, a softening stress-strain response occurs.

5.2.3 Concrete damage plasticity

Abaqus concrete models are continuum plasticity-based damage models. During the simulation, it is assumed that either tensile cracking or compressive crash will cause the concrete to fail. Figure 5.3 shows the assumed tensile and compressive behavior of concrete by Abaqus. Furthermore, the figure illustrates the expected response of concrete to uniaxial compressive and tensile loading for both damaged and undamaged plasticity models. Based on the concrete damage plasticity model, beyond the yielding point of concrete, the total strain is the sum of elastic (ϵ^{el}) and plastic strain (ϵ^{pl}), and the uniaxial compressive and tensile responses of concrete are as follows (Abaqus Design Manual 2016):

$$f_c = (1 - d_c)E_o(\epsilon_c - \epsilon_c^{pl}) \quad (5.7)$$

$$f_t = (1 - d_t)E_o(\epsilon_t - \epsilon_t^{pl}) \quad (5.8)$$

Where,

d_c and d_t = scalar degradation factors for compression and tension, respectively

E_o = initial tangent modulus

ε_c = the total strain of concrete under compression

ε_c^{pl} = the compressive equivalent plastic strain

ε_t = the total strain of concrete under tension

ε_t^{pl} = the tensile equivalent plastic strain

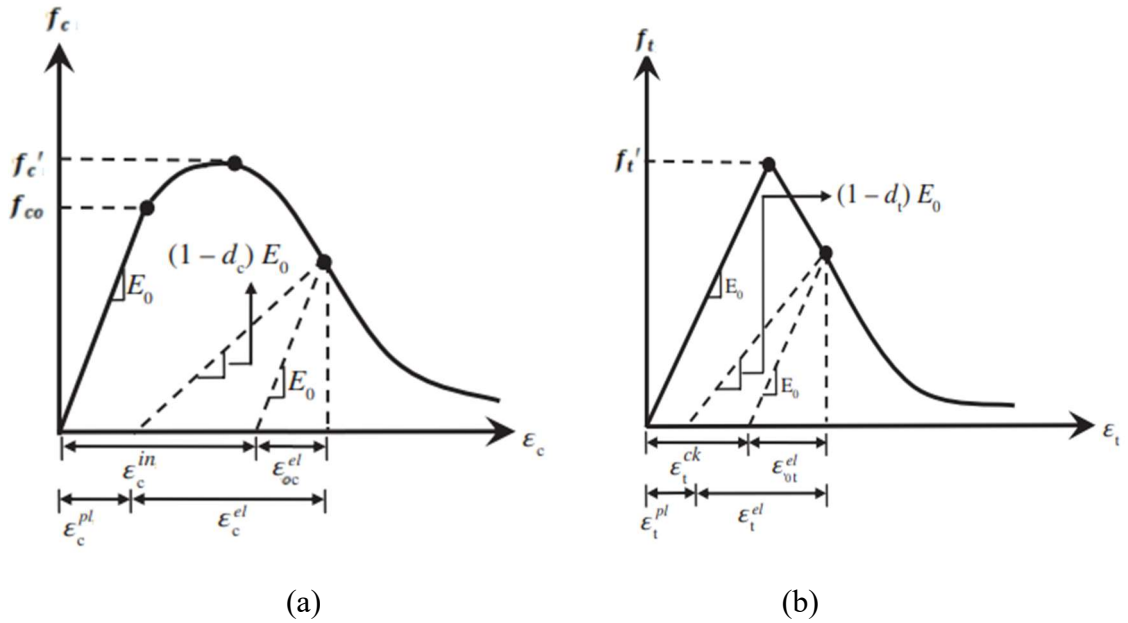


Figure 5.3: Response of concrete to uniaxial loading; (a) Compression (b) Tension (Hafezolghorani et al., 2017)

In this study, no cycling load is applied. Therefore, the undamaged material behavior is considered for concrete's response to the uniaxial compressive and tensile loading. Thus, the scalar degradation factors d_c and d_t are equal to zero. In addition, the stress-strain relationships for undamaged concrete under uniaxial compression and tension loading are given by:

$$f_c = E_o(\varepsilon_c - \varepsilon_c^{in}) \quad (5.9)$$

$$f_t = E_o(\varepsilon_t - \varepsilon_t^{ck}) \quad (5.10)$$

5.2.4 Reinforcement Properties:

As mentioned in Chapter 3, this study uses Sand-coated GFRP bars. In this study, three different bar sizes are modeled: 13, 16, and 19 mm. It is assumed that the bars will exhibit linear elastic properties until they fail. The ultimate stresses of each bar size were 886, 960, and 1038 MPa, respectively, as well as the modulus of elasticity, which were 45, 44, and 54 GPa, respectively. Table 3.7 presents detailed mechanical information from the regular laboratory test results conducted in the industrial solution factory. In addition, the passion ratio of the three sizes is assumed to be 0.2.

5.3 Modeling of the beams:

For this study, three-dimensional models of the seven T-beams were developed using Abaqus software. All the beams were simply supported; pin support was assigned to one end, whereas roller support was used on the other. Both supports were assigned 100 mm from each end, thus giving each beam an effective length of 1900 mm. As shown in Figure 5.4, the samples with a/d of 2 and 2.7 were four-point loaded, while the beam samples with a/d equal to 3.5 were three-point loaded.

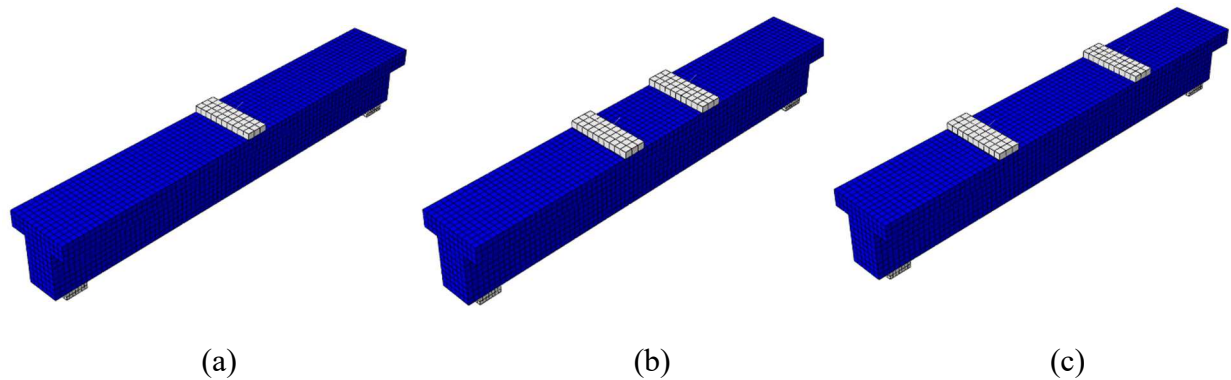


Figure 5.4: T-beam in 3D model; (a) $a/d = 3.5$ (b) $a/d = 2.7$ (c) $a/d = 2$

Three Concrete compressive strengths were defined in this study: 40, 50, and 60 MPa. The concrete compressive behavior was defined based on section 5.2.1, with the strain at the ultimate

compressive strength (ϵ_o) equal to 0.002 as recommended by Kent & Park (1971), and the ultimate strain (ϵ_o) was taken equal to 0.0035. In addition, each of the seven beams had two models with different concrete tensile data, as discussed in section 5.2.2. In the first model, the tensile stress-strain curve was defined based on Alam and Hussain (2020), while for the second model, Hordijk (1991) approach was used. The defined tensile and compressive stress-strain curves for the different concrete grades are shown in Figures 5.5-5.7. The figures show that the Alam and Hussain (2020) model generates a bi-linear tensile stress-strain curve, and Hordijk (1991) generates an exponential curve. It should also be noted that the tensile strength of concrete was calculated differently for each model. Model 1 concrete tensile strength was determined based on the CSA A23.3-04, while Model 2 concrete tensile strength was determined according to the ACI 318. The reason for that is due to the high values of shear capacity produced by Model 1 with concrete tensile strength calculated by ACI 318 code.

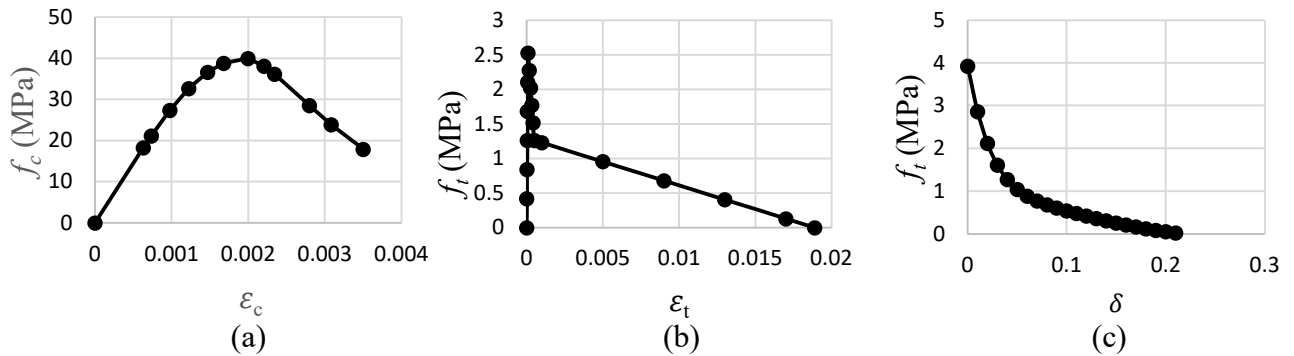


Figure 5.5: Theoretical results of 40 MPa concrete response to uniaxial loading; (a) Compression (b) Tension (Model 1) (c) Tension (Model 2)

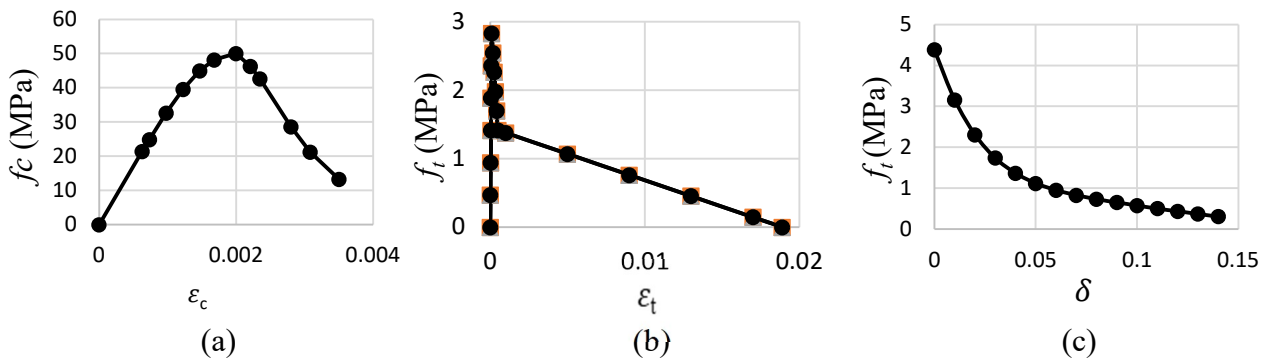


Figure 5.6: Theoretical results of 50 MPa concrete response to uniaxial loading; (a) Compression (b) Tension (Model 1) (c) Tension (Model 2)

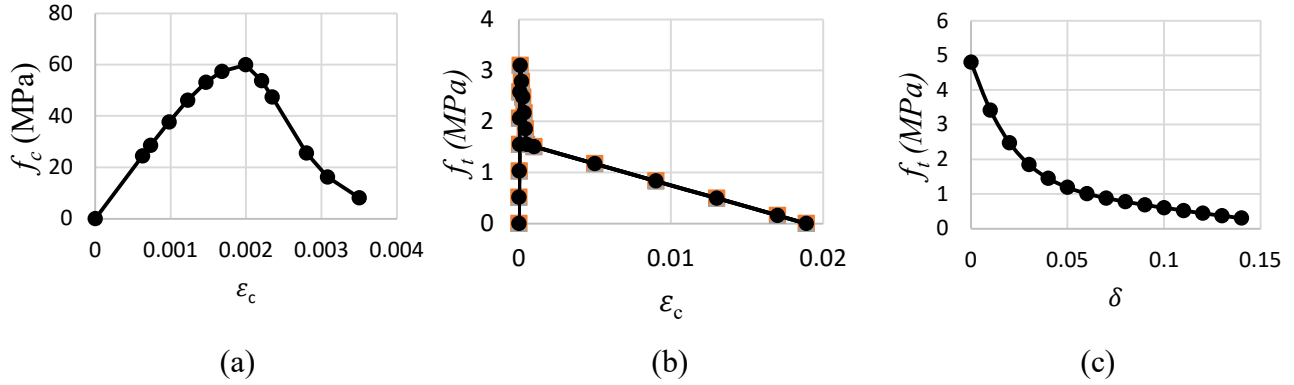


Figure 5.7: Theoretical results of 60 MPa concrete response to uniaxial loading; (a) Compression (b) Tension (Model 1) (c) Tension (Model 2)

In addition, FRP behavior is defined based on section 5.2.4. In all models, an embedded region constraint is applied for the reinforcement, which provides a full bond between the reinforcement and the surrounding concrete. Besides that, A mesh size of 25.0 mm was used in this study for both the concrete beam and the longitudinal bars, based on the findings of Alobaidi and Al-Zuhairi (2024), whose numerical simulation of RC T-beams strengthened with EB-CFRP composites demonstrated that this mesh size provided reliable results, while a 30 mm cubic element was used for the supports. Furthermore, the loading was applied in the form of 20 mm vertical displacement at the exact location of the laboratory experiment, as discussed in Chapter 3. Although the experiment's loading was static, the Finite element analysis was performed using a dynamic explicit procedure. In all cases, the dilation angle (ψ), which is related to the volume change of concrete during the plastic stage, was taken as 30 degrees for all compressive strengths.

5.4 Abaqus results

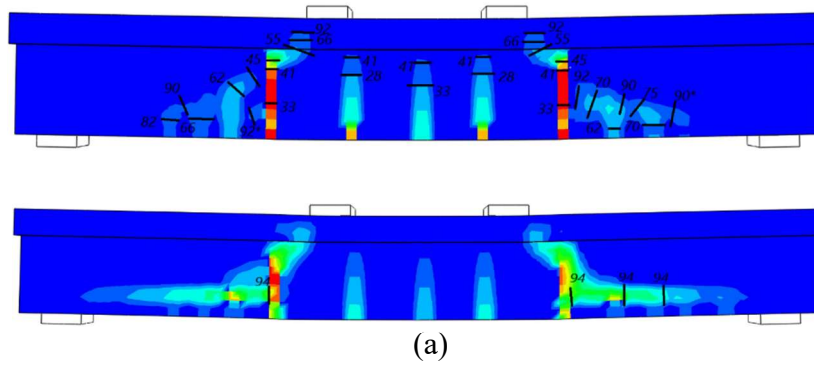
5.4.1 Crack pattern:

The values of the experimental cracking loads (P_{cr-exp}) and the finite element cracking loads (P_{cr-M1} & P_{cr-M2}) are shown in Table 5.1. In both models, it is apparent that the experimental and finite element results of the first cracking load were in reasonable agreement, with an average P_{cr-exp}/P_{cr-M} for models one and two equal to 0.99 and 0.93, respectively, and with a standard deviation of 1.08 and 1.01, respectively. Furthermore, both models had approximately a similar consistency with COV of 108.6 % for model 1 and 109% for model 2. Abaqus software presents the cracks'

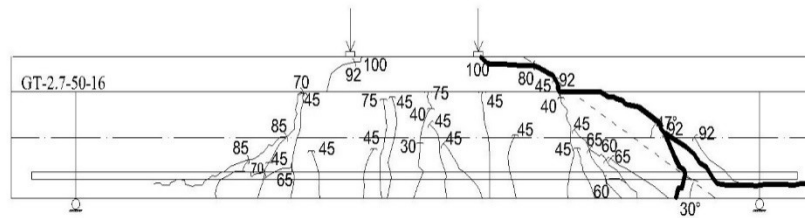
shape using the concrete's maximum and minimum principal plastic strain results. In general, the cracking patterns of both models were relatively similar to the experimental observations regarding failure crack type, shape, and location, as shown in Appendix D. However, the models did not always match the experimental observations regarding the magnitude of the applied load. However, when the ultimate load was remarkably predicted accurately in beams GT-2.7-50-16 and GT-3.5-40-16 in model one, the crack patterns were also similar to the experimental observations, as shown in Figures 5.8 and 5.9. It can be observed that the software succeeded in reasonably predicting the crack's shape, loads, and lengths. Besides that, the software also showed that the failure cracks are horizontal at the bottom of the shear span zone, then divert diagonally and propagate towards the loading point while also propagating back towards the support.

Table 5.1: First cracking load (FEA vs. experimental results)

Beam ID	P_{cr-exp} (kN)	Alam and Hussain (2020) (Model 1) (kN)	Hordijk (1991) (Model 2) (kN)	P_{cr-exp}/P_{cr-M1}	P_{cr-exp}/P_{cr-M2}
GT-2-50-16	45.00	46.27	53.66	0.97	0.84
GT-2.7-50-16	30.00	33.40	35.01	0.90	0.86
GT-3.5-50-16	25.00	25.46	27.66	0.98	0.90
GT-3.5-40-16	24.00	19.54	20.02	1.23	1.20
GT-3.5-60-16	27.00	27.54	29.26	0.98	0.92
GT-3.5-50-13	25.00	24.63	25.74	1.01	0.97
GT-3.5-50-19	25.00	28.12	31.90	0.89	0.78
Average				0.99	0.93
Standard deviation				1.08	1.01
COV %				108.60	109.00

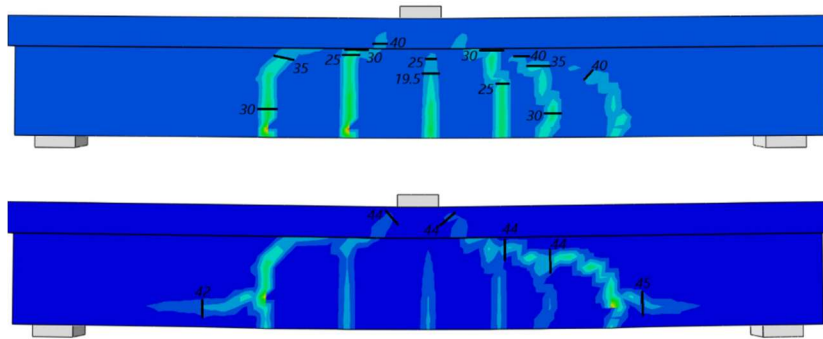


(a)

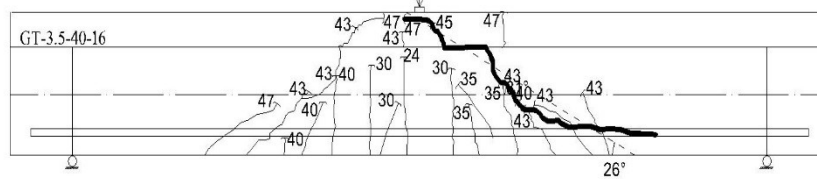


(b)

Figure 5.8: Crack Pattern of beam GT-2.7-50-16 (Model 1 vs. experimental results) (a) FEA (b) Experimental observation



(a)



(b)

Figure 5.9: Crack Pattern of beam GT-3.5-40-16 (Model 1 vs. experimental results) (a) FEA (b) Experimental observation

5.4.2 Deflection and ultimate load:

This section compares the ultimate load and load-deflection behavior of Model One and Model Two with the experimental results of seven GFRP-reinforced T-beams. With the exception of beams GT-3.5-50-19 and GT-3.5-60-16, the predicted load-deflection behaviors of both models approximately followed the same path as the experimental results. However, each beam's load-deflection behavior varied at the ultimate load level, except for beam GT-2.7-50-16. Despite the fact that beam GT-2.7-50-16 deflection recording was stopped at about 80 kN due to safety concerns, the beam recorded results showed a good agreement between the experimental results and the two models in terms of ultimate shear capacity deflection, which led to a conclusion that the beam experimental load-deflection behavior after 80 kN would continue to follow the same path as the two models. On the other hand, as Table 5.2 and Figures 5.10-5.12 demonstrate, both models overestimated the load and the deflection of most of the beams.

Table 5.2: FEA ultimate shear capacity results.

Beam ID	V_{u-exp} (kN)	Alam and Hussain (2020) (Model 1) (kN)	Hordijk (1991) (Model 2) (kN)	V_{u-exp}/V_{u-M1}	V_{u-exp}/V_{u-M2}
GT-2-50-16	55.00	72.65	68.19	0.76	0.81
GT-2.7-50-16	50.00	48.44	47.75	1.03	1.05
GT-3.5-50-16	30.75	40.15	36.12	0.77	0.85
GT-3.5-40-16	23.50	24.32	29.77	0.97	0.79
GT-3.5-60-16	32.50	41.07	37.87	0.79	0.86
GT-3.5-50-13	28.50	33.34	28.53	0.85	1.00
GT-3.5-50-19	31.50	48.93	45.40	0.64	0.69
Average				0.83	0.86
Standard deviation				0.91	0.94
COV %				109.19	108.94
% of unsafe results				85.71	71.43

For beams with different shear span-to-depth ratios, Figure 5.10 shows that both models overestimated the ultimate load and the deflection of beams with a/d of 2 and 3.5. Model One overestimated the beam's ultimate failure load by 32% and 30%, respectively, whereas Model Two was 24% and 17%, respectively, higher than the experimental ultimate load. Nevertheless, both

Models succeeded in predicting the beam's ultimate load and deflection behavior with a/d equal to 2.7.

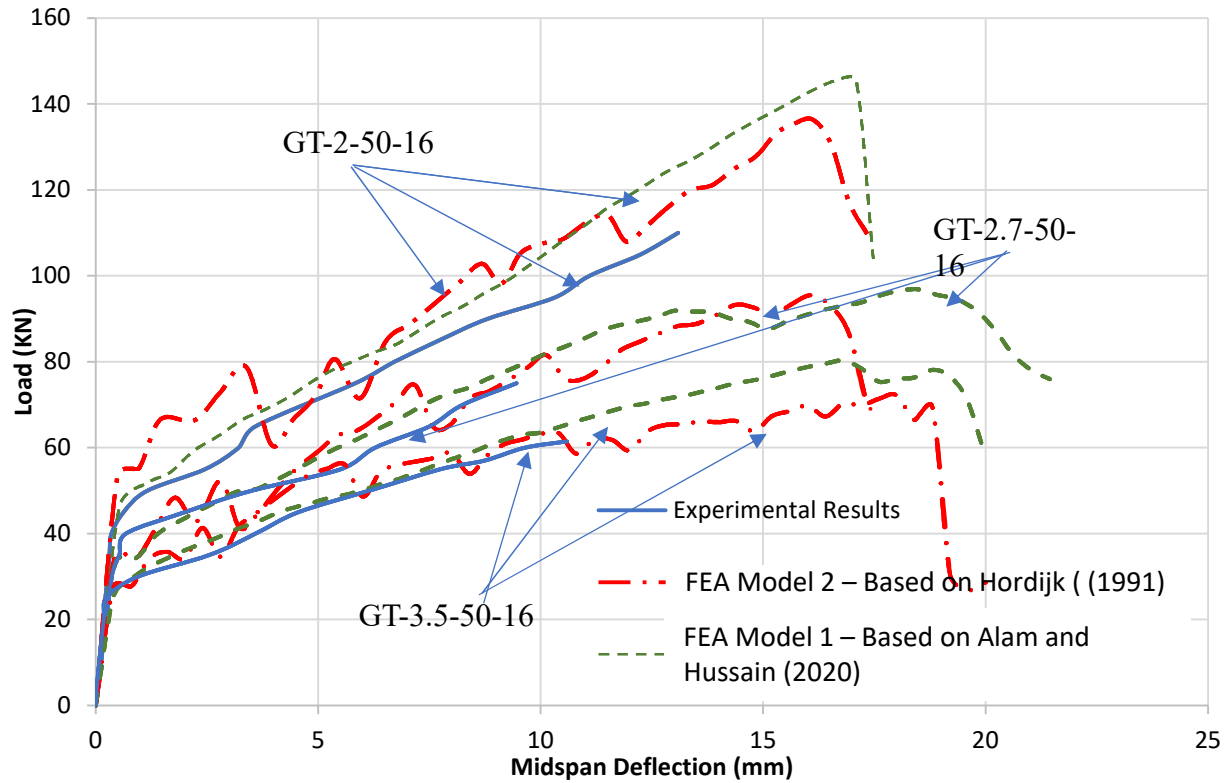


Figure 5.10: Comparative load-deflection behaviors of test results and FEA for beams with different a/d

Figure 5.11 illustrates the load-deflection behavior of beams with concrete strengths of 40 and 50 MPa. The figure shows that Model One overestimated the deflection of both beams, but for the beam with 40 MPa concrete strength, the model provided a good estimation of ultimate shear strength. On the contrary, both failure load and deflection were overestimated by Model Two. The Model overestimated the failure load of the beams with 40, 50, and 60 MPa by approximately 27%, 17%, and 16%, respectively.

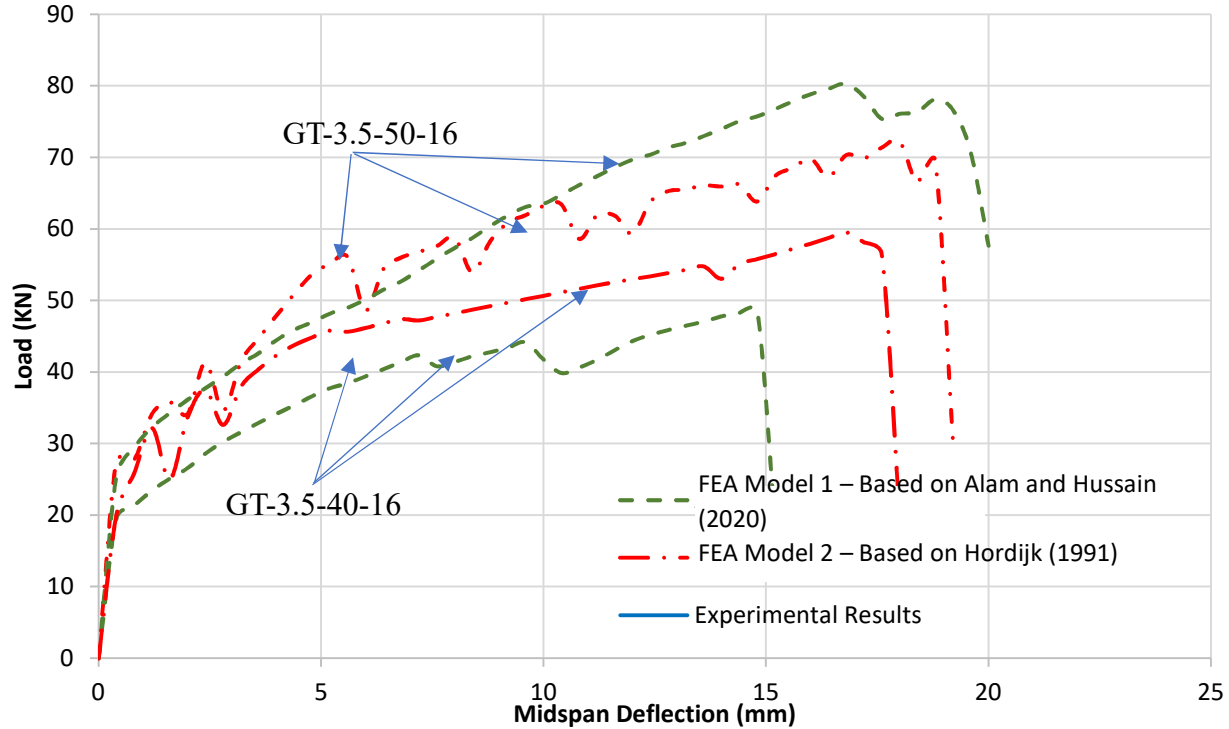


Figure 5.11: Comparative load-deflection behaviors of test results and FEA for beams with different f'_c

The load-deflection behavior of reinforcement ratios of 0.7%, 1.1%, and 1.5% are presented in Figure 5.12. Similar to the other two parameters, both models overestimated the failure load and the deflection for all three reinforcement ratios used in this study except for the reinforcement ratio of 0.7%, where Model Two accurately predicted the ultimate shear strength, as Table 5.2 demonstrates. In addition, Model Two load-deflection behavior demonstrated continuous load drops in most of the beams during the vertical displacement test. A possible explanation for this could be the occurrence of new cracks and the increase in crack size. Unlike Model Two, Model One showed a smooth ascending loading curve. Furthermore, the poor accuracy of the FEA models in predicting deflection and ultimate load, particularly in the post-peak region, is likely due to the fact that the compressive and tensile behaviors used in this study are based on theoretical estimations rather than material tests.

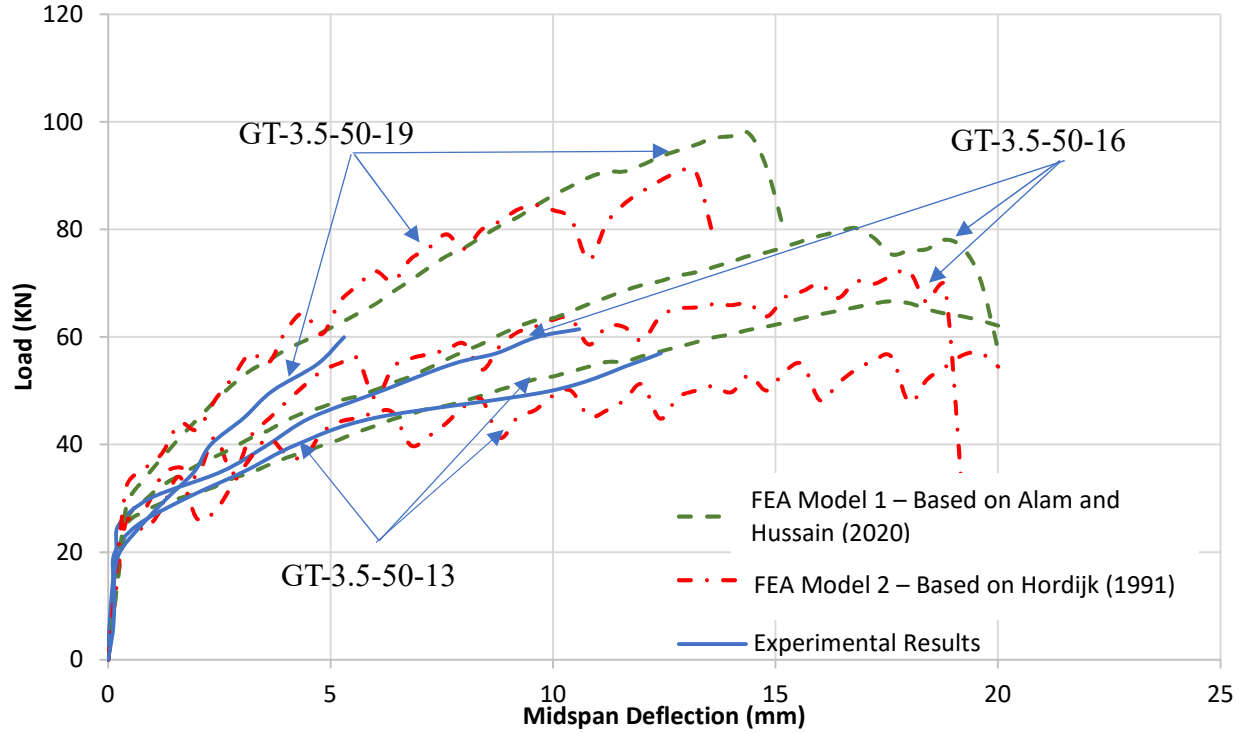


Figure 5.12: Comparative load-deflection behaviors of test results and FEA for beams with different ρ

5.4.3 GFRP stress-strain curve:

The stress-strain response of the GFRP reinforcement bars was not measured directly during the laboratory experiments due to the lack of strain gauges and LVDTs. Instead, this response was obtained from the Abaqus finite element simulations, offering insight into the internal behavior of the reinforcement under flexural loading. Figure 5.13 confirms that the modulus of elasticity used in the simulations matches the values provided in Table 3.7, verifying that Abaqus accurately implemented the bar-specific material properties from the manufacturer.

The stress-strain curves show that none of the GFRP bars reached their ultimate tensile stress throughout the loading process. This indicates that the reinforcement did not govern the failure. This is consistent with the experimental observations, where the concrete crushed before the GFRP reinforcement could rupture, meaning the T-sections behaved in a compression-controlled manner.

According to ACI 440, if the reinforcement ratio exceeds the balanced reinforcement ratio, the section is classified as compression-controlled, and failure is expected to occur due to concrete crushing. Conversely, if the reinforcement ratio is below the balanced value, the section is tension-controlled, and failure occurs due to rupture of the GFRP reinforcement. In this study, all the reinforcement ratios used were greater than the balanced reinforcement ratio, which explains why compression-controlled behavior was observed consistently in both the numerical model and the experimental tests.

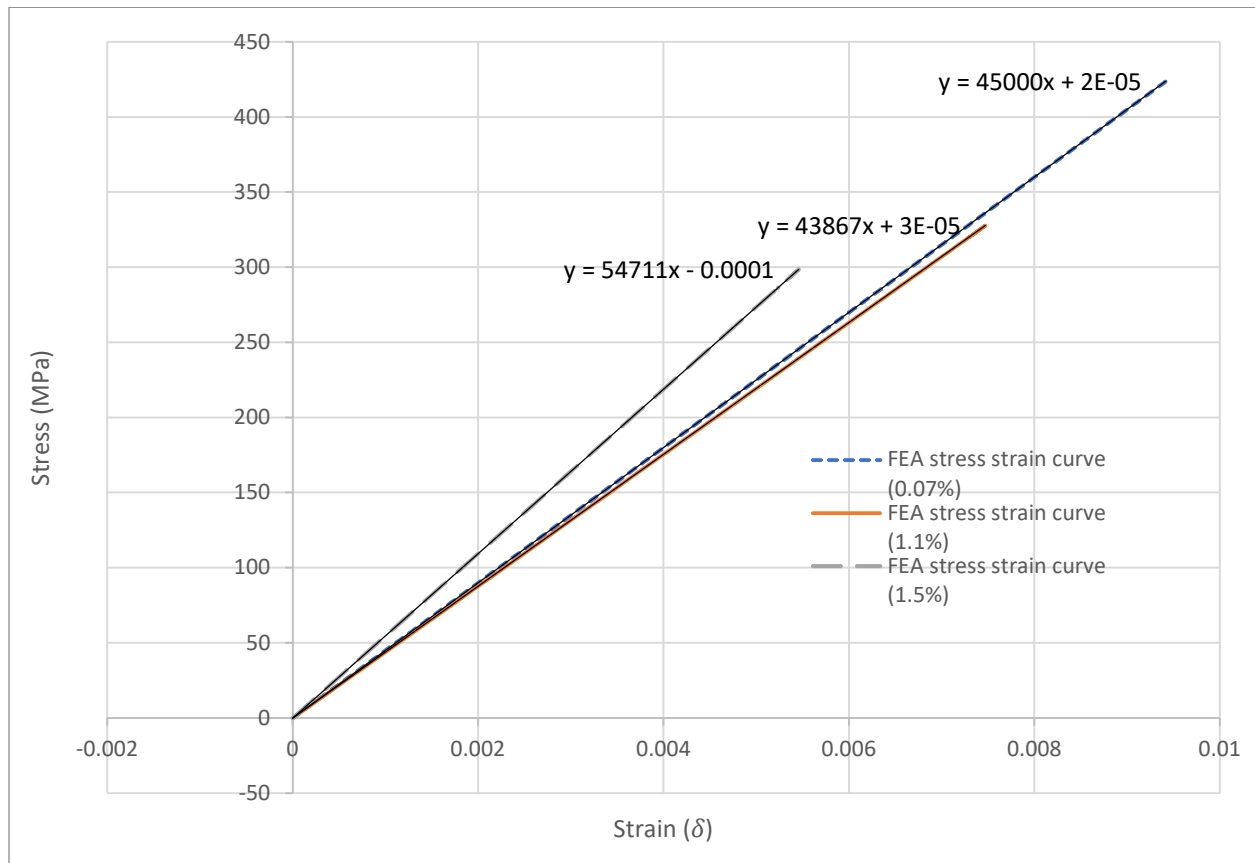


Figure 5.13: FEA GFRP stress-strain curves

Chapter Six

Conclusion and Recommendations

6.1 Introduction:

GFRP is one of the major alternative solutions for replacing steel reinforcement due to its high corrosion resistance. This study investigated the shear behavior of T-section concrete beams reinforced with GFRP as longitudinal reinforcement without transverse reinforcement. Seven beams were constructed in the Eastern Precast Factory and then delivered to the University of Bahrain campus. All the beams were tested to failure under three-point loading, except for those with an a/d ratio of 2 and 2.7, which were tested under four-point loading. Besides that, a Finite element analysis using Abaqus software was performed on the seven beams to compare it with the experimental results.

6.2 Experimental Study:

In this experiment, all the beams had a constant cross-sectional area and a length of 2 meters. The variables were the shear span-to-depth ratio (a/d), concrete compressive strength (f_c'), and the longitudinal reinforcement ratio (ρ). The seven beams were divided into three groups; each group consisted of three beams and focused only on one variable. In group one, the focus was to study the effect of different span-to-depth ratios, which were 2, 2.7, and 3.57. In the second group, the concrete compressive strength's effect was studied using f_c' approximately equal to 40, 50, and 60 MPa. Finally, the effect of the longitudinal reinforcement ratio was investigated using ratios of 0.007, 0.011, and 0.015.

The following conclusion can be drawn according to the experimental results:

- 1) The experimental and the predicted first cracking are approximately equal for all the beams
- 2) The first crack of all the beams was due to flexural stress and the mode of failure for all the beams is shear failure.

- 3) The failure in most of the beams was sudden and very rapid; in some beams, the crack that caused the failure occurred just seconds before the collapse of the beam. Besides that, all the beams exhibited shear splitting at the shear span during the failure except the beam GT-3.5-60-16 which has the highest concrete compressive strength in this experiment.
- 4) Increasing a/d slightly decreased the angle of the failure crack, reduced the cracking spacings, and limited the number of flexural cracks.
- 5) Changing the compressive strength did not affect the crack spacing, number of cracks, and the angle of the failure crack significantly.
- 6) An increase in reinforcement ratio from 1.1 to 1.5% resulted in a noticeable reduction in the number of cracks and an increase in crack spacing.
- 7) At an applied load of around 60 kN, reducing the a/d ratio from 3.57 to 2.7 and 2.0 decreased beam deflection by 34% and 66%, respectively. In addition, increasing the reinforcement ratio from 0.007 to 0.011 and 0.015 reduced deflection by 30% and 60%, respectively. At a lower load level of 47 kN, increasing the compressive strength from 40 to 50 MPa reduced deflection by 41%. These results indicate that lower a/d ratios, higher reinforcement ratios, and increased concrete compressive strength all have a significant influence on deflection control and contribute effectively in improving GFRP-reinforced T-beams' stiffness.
- 8) The experimental results demonstrated that the shear capacity of the beams was strongly influenced by the shear span-to-effective-depth ratio (a/d) and the concrete compressive strength. Increasing the a/d ratio from 2.0 to 2.7 and 3.57 led to reductions in shear capacity of 9.09% and 44.1%, respectively, confirming that higher a/d ratios adversely affect shear performance. Increasing the concrete compressive strength from 40 to 50 MPa enhanced the shear capacity by 30.8%, while a further increase to 60 MPa resulted in only a 38.3% gain, indicating that the effect of compressive strength becomes less pronounced at higher levels. Furthermore, increasing the longitudinal reinforcement ratio from 0.7% to 1.1% and 1.5% led to relatively smaller improvements in shear strength, 7.89% and 10.53%, respectively. These findings suggest that, among the parameters studied, the a/d ratio and concrete compressive strength have a more significant impact on shear behavior than the longitudinal reinforcement ratio in GFRP-reinforced T-beams without shear reinforcement.

- 9) CSA S806-12 and Alam's (2010) equations provided the most accurate shear strength prediction with average V_{exp}/V_{calc} of 1.08 and 1.13, respectively, which makes it suitable and economical to be used for the estimation of the shear strength of T-sections with taking into consideration the necessary factor of safety. On the other hand, ACI 440.1R-15 had the poorest agreement with experimental results with an average V_{exp}/V_{calc} of 2.11. Furthermore, ISIS (2002), JSCE (1997), and Tureyen et al. (2006) had approximately similar predictions. However, they were inconsistent with the experimental results; this is because these methods do not consider the effect of the shear span-to-depth ratio.

6.3 Conclusions about finite element analysis:

For FEA, the same geometrical and mechanical properties used in the experimental part were applied in the modeling of the T-beams. Each beam was modeled twice. The difference between Model 1 and Model 2 for each beam was the response to the uniaxial tensile loading. The tensile stress-strain curve of Model 1 was developed according to Alam and Hussain's (2020) proposed model, and for Model 2, Hordijk's (1991) approach was followed. In addition, for both models One and Two, the compressive stress-strain curves of the three concrete strengths were developed using the expression of Thorenfeldt et al. (1987). The mesh size was set to be 25 mm cubic element, and the load was applied in the form of vertical displacement. The maximum vertical displacement applied was 20 mm.

The following is concluded based on the FEA results:

- 1) The FEA model's estimation for the first cracking load was good for most of the beams. The average P_{cr-exp}/P_{cr-M} of models one and two were found to be equal to 0.99 and 0.93, respectively, with a standard deviation of 1.08 and 1.01, respectively.
- 2) The FEA model's prediction of the diagonal shear crack was similar to the experimental observation for all the beams.
- 3) Model One had a good agreement with the experimental results in terms of the crack pattern of beams GT-2.7-50-16 and GT-3.5-40-16.

- 4) In both models, the predicted load-deflection behavior is in a good agreement with the experimental results prior to the ultimate load level. Despite this, the models showed higher deflections at ultimate loads in all beams than the experimental results.
- 5) Model One overestimated most of the ultimate load except for beams GT-2.7-50-16 and GT-3.5-40-16, and Model Two overestimated most of the ultimate load except for beams GT-2.7-50-16 and GT-3.5-60-16.
- 6) Models One and Two had similar averages of V_{exp}/V_{u-M} , equal to 0.83 and 0.86, respectively, with standard deviations of 0.91 and 0.94, respectively.

6.4 Recommendations for future work:

- 1) The following are the recommendations for further studies related to the shear behavior of beams reinforced with FRP:
- 2) The shear behavior of FRP-RC T-beams with different flange widths should be investigated to understand how the flange width affects the shear strength.
- 3) Experimental investigation on two layers of reinforcement effect of shear strength should be conducted to study the impact of different reinforcement arrangements.
- 4) Experimental studies on the effect of the Top reinforcement on the shear behavior of FRP-RC T-beams.
- 5) This study focused on the shear behavior of T-beam reinforced with GFRP. Carbon FRP (CFRP), Basalt FRP (BFRP), and Aramid FRP (AFRP) bars should also be investigated for T-sections.
- 6) As concentrated load was applied in this study, Further studies should be conducted on beams with distributed load.
- 7) For FEA, methods for the estimation of concrete mechanical properties should be further explored. As a further improvement, uniaxial tensile and compressive strength tests are better for determining material mechanical properties than theoretical calculations.

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Appendices

Appendix (1)

Design of Beam GT-3.5-50-16:

The flexural and shear capacity of beam GT-3.5-50-16 is calculated as follows:

1- Reinforcement ratio.

$$A_f = 2 \times 201.05 = 402.12 \text{ mm}^2.$$

$$b = b_w = 150 \text{ mm}.$$

$$d = h - \text{cover} - \text{dia}/2 = 300 - 40 - 16/2 = 252 \text{ mm}.$$

$$\rho_f = \frac{A_f}{bd} = \frac{A_f}{bd} = \frac{402.12}{150 \times 252} = 0.011$$

$$f_c' = 52 \text{ MPa}.$$

$$\text{Since } 28 < f_c' < 56 \quad \text{so} \quad \beta_1 = 0.85 - 0.05 \frac{f_c' - 28}{7} = 0.85 - 0.05 \frac{52 - 28}{7} = 0.693.$$

Since the calculations are for experimental purposes, the environmental reduction factors were ignored, so $f_{fu} = 964 \text{ MPa}$ and $E_f = 43867.1 \text{ MPa}$ as stated by the manufacturer.

$$\epsilon_{cu} = 0.0035.$$

$$\rho_b = 0.85 \beta_1 \frac{f_c}{f_{fu}} \frac{E_f \epsilon_{cu}}{E_f \epsilon_{cu} + f_{fu}} = 0.85 \times 0.693 \times \frac{52}{964} \frac{43867.1 \times 0.0035}{43867.1 \times 0.0035 + 964} = 0.0044$$

Since $\rho_f > \rho_b$, so the controlling limit state is crushing of concrete.

2- Tensile stress of GFRP bars at ultimate condition.

$$\begin{aligned} f_f &= \left(\sqrt{\frac{(E_f \epsilon_{cu})^2}{4} + \frac{0.85 \beta_1 f_c'}{\rho_f} E_f \epsilon_{cu}} - 0.5 E_f \epsilon_{cu} \right) \leq f_{fu} \\ &= \left(\sqrt{\frac{(43867.1 \times 0.0035)^2}{4} + \frac{0.85 \times 0.736 \times 52}{0.011} \times 43867.1 \times 0.0035} - 0.5 \times 43867.1 \times 0.0035 \right) = 548 \text{ MPa} \\ &\leq 964 \text{ MPa}. \end{aligned}$$

3- Nominal flexural strength.

$$a' = \frac{A_f f_f}{0.85 f_c' b} = \frac{402.12 \times 548}{0.85 \times 52 \times 300} = 17.8 \text{ mm} < h_f (75 \text{ mm}), \text{ so the compression stress is within the flange.}$$

$$M_n = A_f f_f \left(d - \frac{a'}{2} \right) = 402.12 \times 530 \times \left(252 - \frac{20}{2} \right) / 1000^2 = 57.2 \text{ kN.m}.$$

4- Ultimate load capacity due to flexural strength.

$$a/d = 3.5$$

$$a = 3.5d = 3.5 * 252 = 882 \text{ mm} = 0.882 \text{ m}.$$

$$\text{Based on Figure 3.15 } M = \frac{P}{2}a, \text{ so } P = 2\frac{M}{a} = 2 * \frac{57.2}{0.882} = 127 \text{ kN}.$$

5- Shear strength.

$$E_c = 4500 \sqrt{f'c} = 4500\sqrt{52} = 32359.97 \text{ MPa}.$$

$$n_f = \frac{E_f}{E_c} = \frac{43867.1}{32359.97} = 1.36$$

$$k = \sqrt{2\rho_f n_f + (\rho_f n_f)^2} - \rho_f n_f = \sqrt{2 * 0.011 * 1.36 + (0.011 * 1.36)^2} - 0.011 * 1.36$$

$$k = 0.159$$

$$V_c = \frac{2}{5} \sqrt{f'c} b_w (kd) = \frac{2}{5} * \sqrt{52} * 150 * 0.159 * 252 = 1696 \text{ N} = 16.96 \text{ kN}$$

5-Ultimate capacity based on shear strength.

Based on Figure 3.15, $P = V_c * 2 = 16.96 * 2 = 33.93 \text{ kN}$. < Ultimate load capacity due to flexural strength, so the specimen GT-3.5-50-16 will fail in shear prior to flexural.

Appendix (2)

Photos of the Failure Mode of the Beams



Figure A2. 1: Crack pattern of beam GT-2-50-16



Figure A2. 2: Crack pattern of beam GT-2.7-50-16



Figure A2. 1: Crack pattern of beam GT-3.5-50-16



Figure A2. 2: Crack pattern of beam GT-3.5-60-16



Figure A2. 3: Crack pattern of beam GT-3.5-40-16



Figure A2. 4: Crack pattern of beam GT-3.5-50-19



Figure A2. 5: Crack pattern of beam GT-3.5-50-13

Appendix (3)

Load-Deflection Behavior of the Beams (Experimental and FEA)

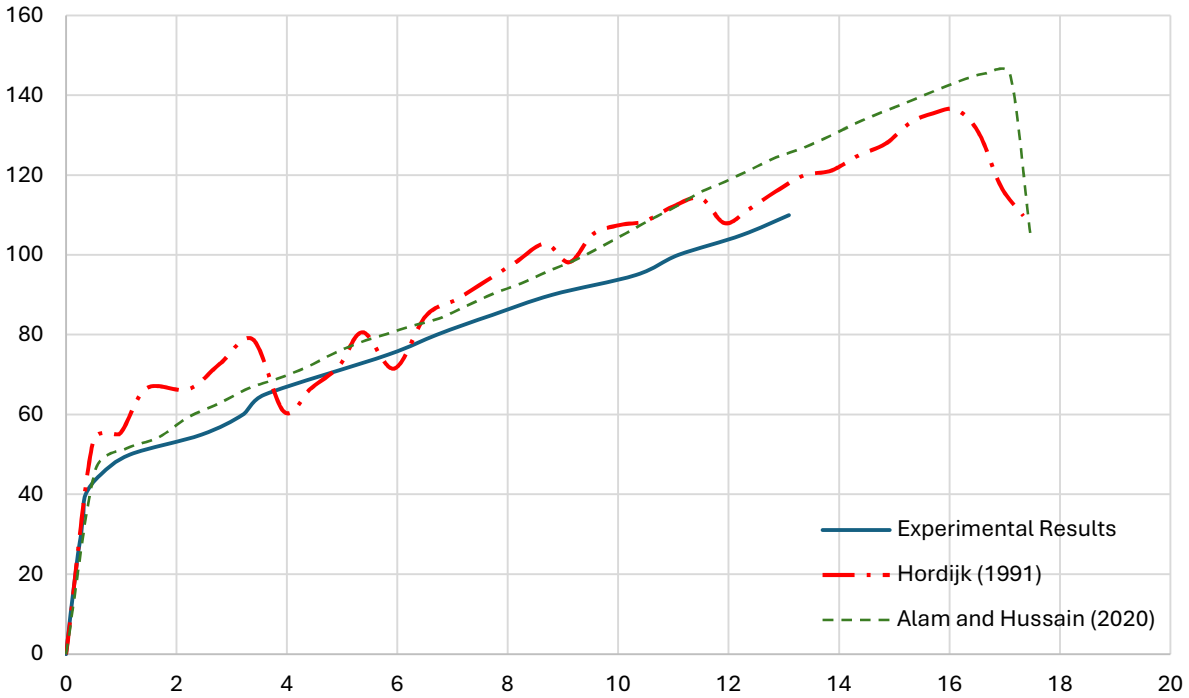


Figure A3. 1: Comparative load-deflection behaviors of test results and FEA for GT-2-50-16

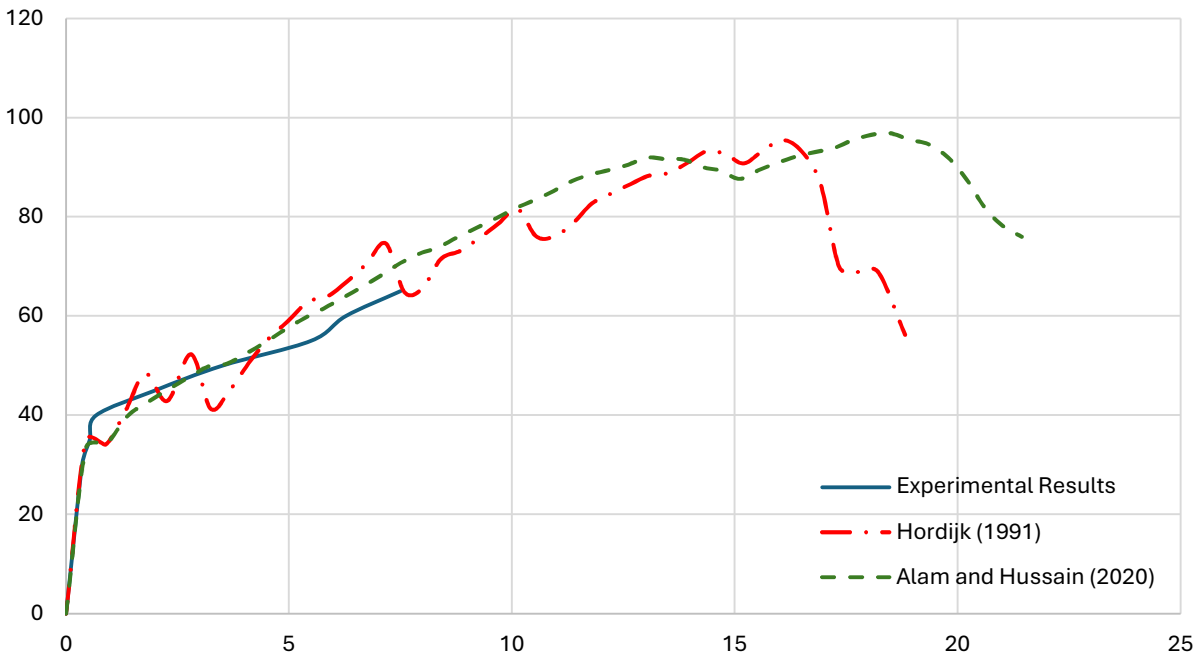


Figure A3. 2: Comparative load-deflection behaviors of test results and FEA for GT-2.7-50-16

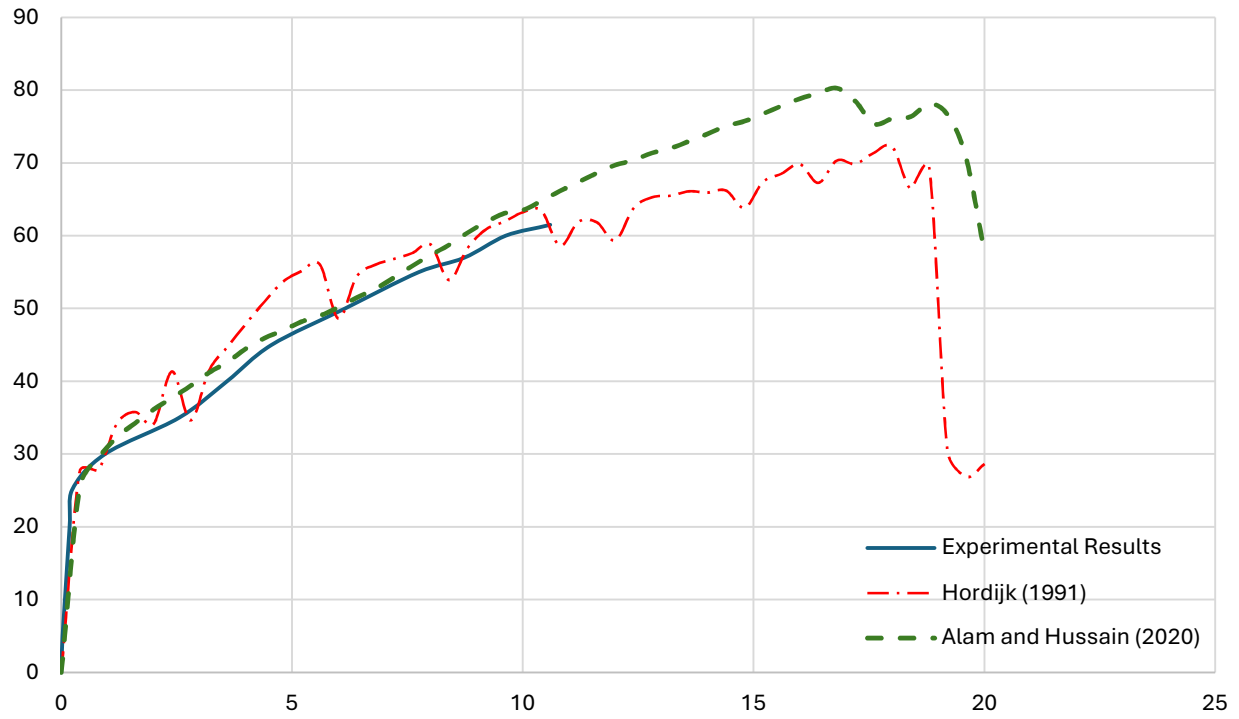


Figure A3. 3: Comparative load-deflection behaviors of test results and FEA for GT-3.5-50-16

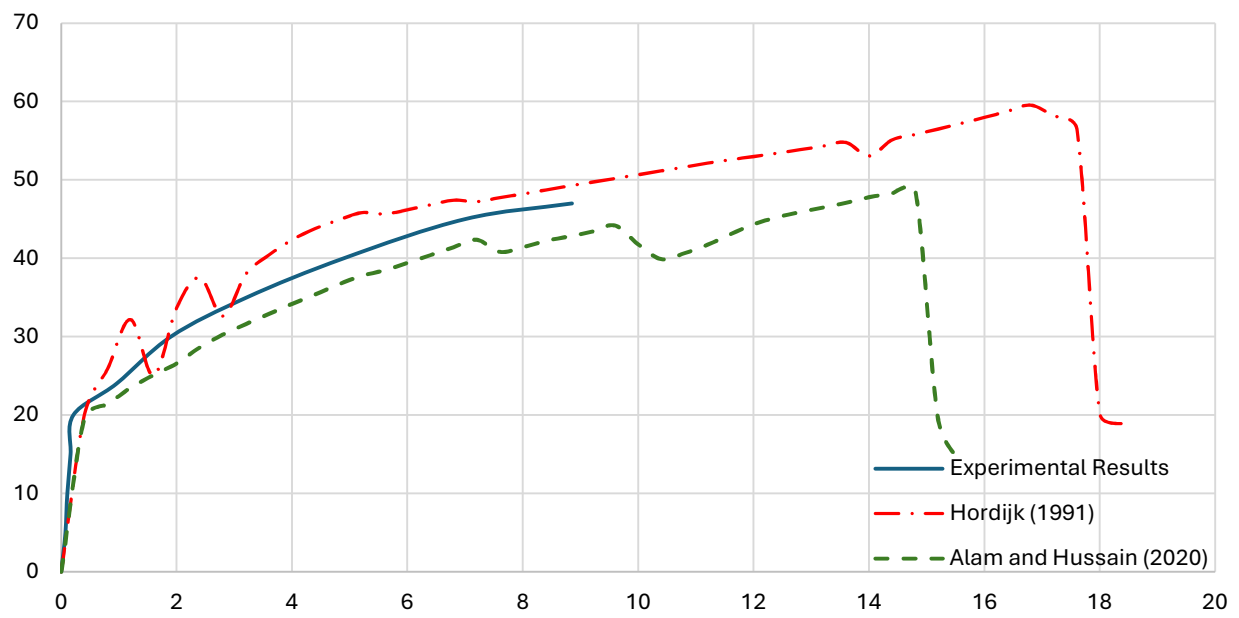


Figure A3. 4: Comparative load-deflection behaviors of test results and FEA for GT-3.5-40-16

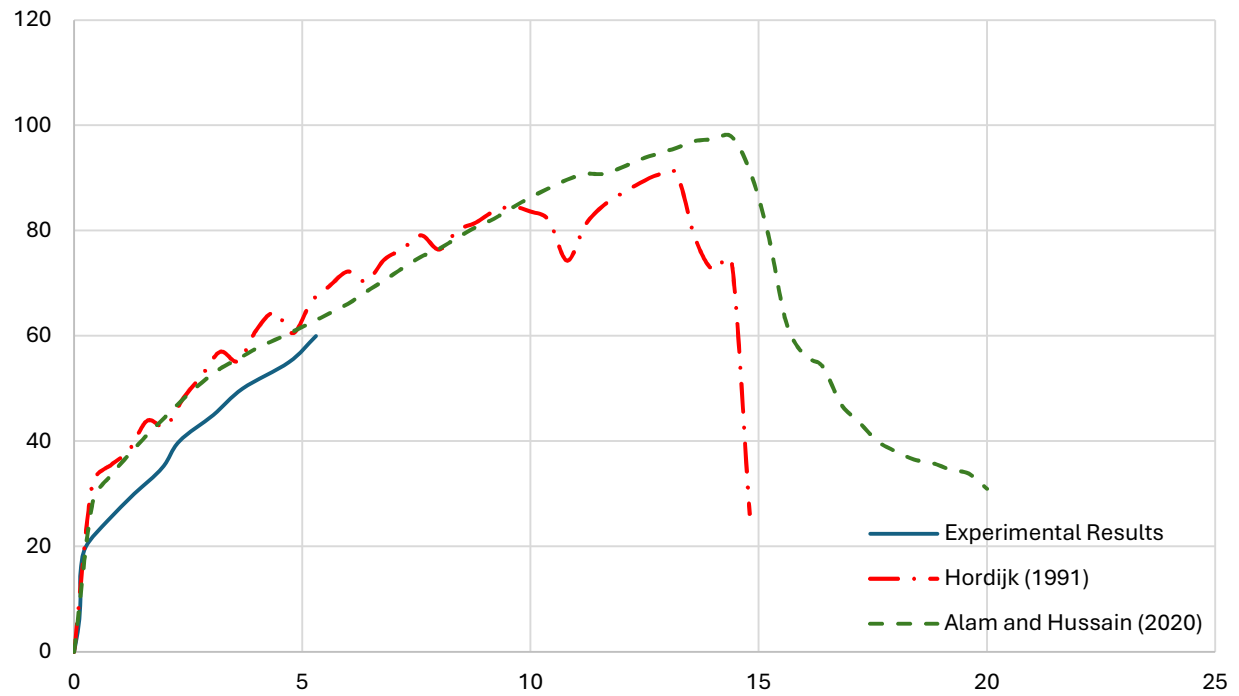


Figure A3. 5: Comparative load-deflection behaviors of test results and FEA for GT-3.5-50-19

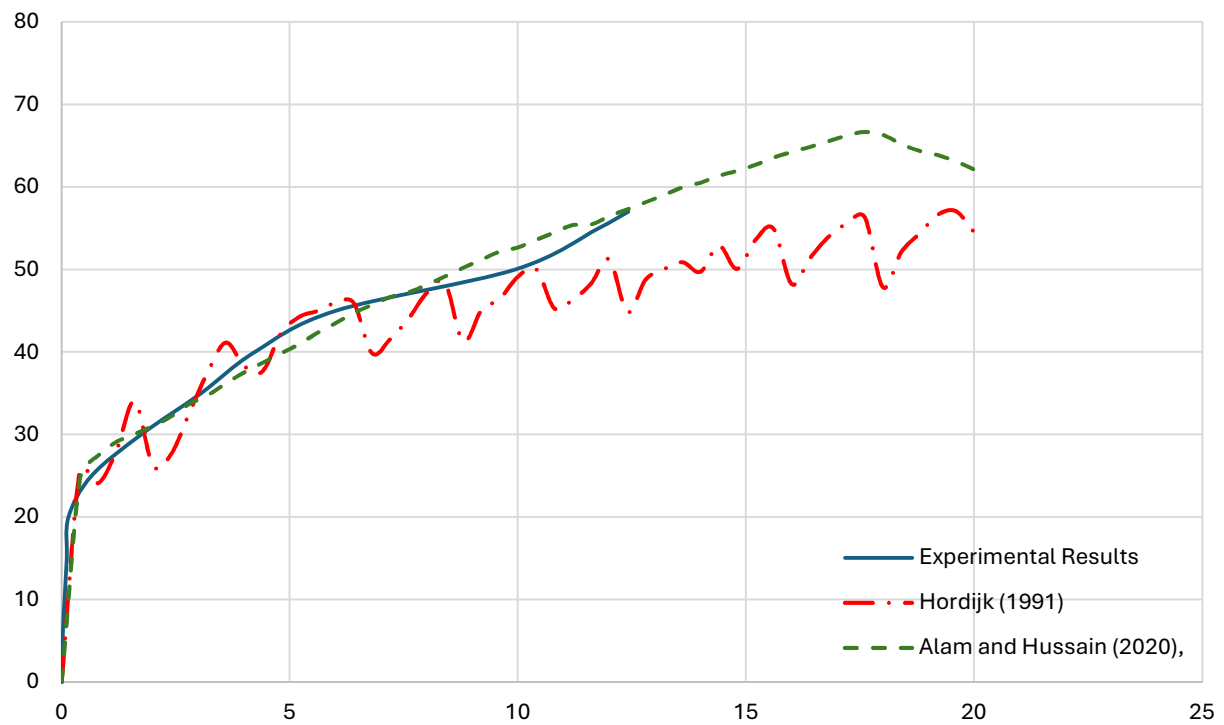


Figure A3. 6: Comparative load-deflection behaviors of test results and FEA for GT-3.5-50-13

Appendix (4)

Crack Pattern Obtained by FEA (Model 1)

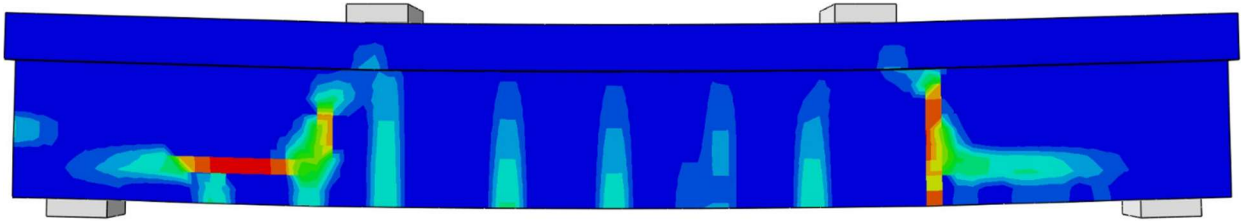


Figure A4. 1: Crack Pattern of beam GT-2-50-16 obtained by FEA Model 1

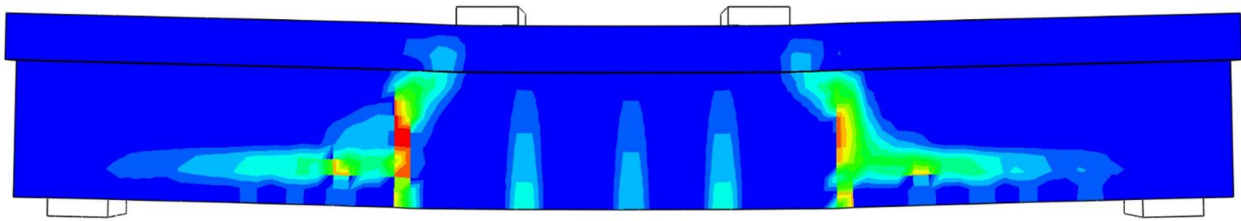


Figure A4. 2: Crack Pattern of beam GT-2.7-50-16 obtained by FEA Model 1

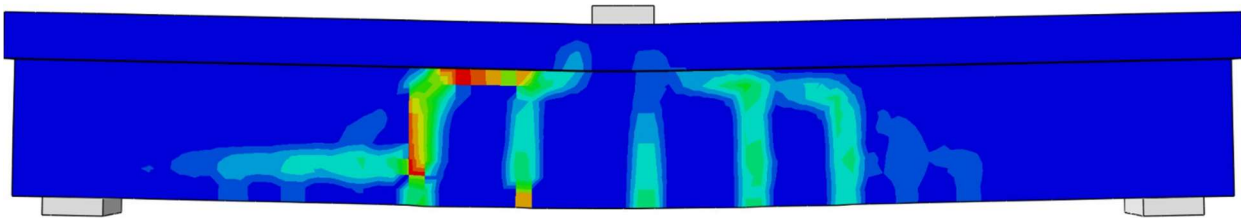


Figure A4. 3: Crack Pattern of beam GT-3.5-50-16 obtained by FEA Model 1

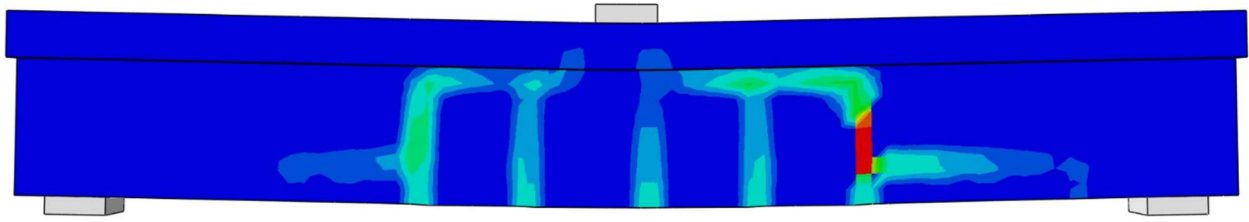


Figure A4. 4: Crack Pattern of beam GT-3.5-60-16 obtained by FEA Model 1

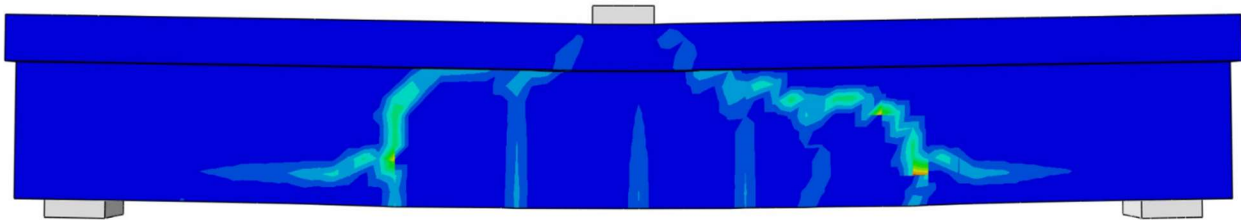


Figure A4. 5: Crack Pattern of beam GT-2.7-40-16 obtained by FEA Model 1

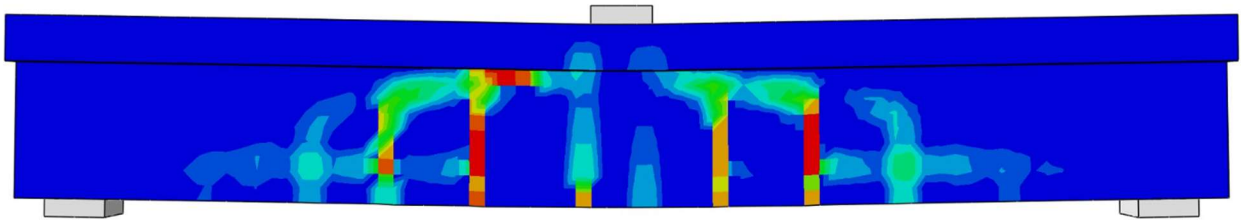


Figure A4. 6: Crack Pattern of beam GT-3.5-50-19 obtained by FEA Model 1

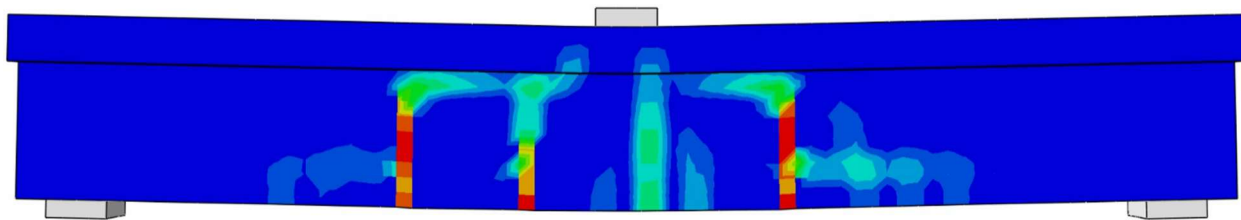


Figure A4. 7: Crack Pattern of beam GT-3.5-50-13 obtained by FEA Model 1

الملخص

التآكل هو أحد العيوب الأساسية لاستخدام الحديد كتسليح للخرسانة، وخاصة عند تعرضه لبيئات قاسية. أحد الخيارات البديلة لحل هذه المشكلة هي استخدام البوليمر المقوى بألياف الزجاج (GFRP)، والمعروف بقوته العالية وخفة وزنه ومقاومته الجيدة للتآكل. الجدير بالذكر أن خصائص البوليمر المقوى بألياف الزجاج الميكانيكية كمعامل المرونة مثلاً، تختلف عن خصائص الحديد، لذلك أجريت العديد من البحوث لدراسة سلوك القص للجسور الخرسانية ذات الشكل المستطيل (rectangular beams) المسلحة بألياف الزجاج المقوى. على الرغم من ذلك، لا زالت هنالك الحاجة لاستكشاف سلوكيات القص لجسور (T-beams) المسلحة بألياف الزجاج المقوى.

ركز هذا البحث على دراسة سلوك القص لسبعة جسور (T-beams) مسلحة في الجزء السفلي من قطاع الجسور بقضبان من ألياف الزجاج المقوى وبدون اضافة تسليح عرضي. تنقسم هذه الدراسة الى جزئين؛ يتكون الجزء الاول من تجربة عميلة. بينما الجزء الثاني عبارة عن محاكاة عن طريق استخدام تحليل العناصر المحدودة المعروف باختصار (FEA). في الجزء التجريبي، تم إنشاء سبعة عوارض (T-beams) في مصنع الشرقية للخرسانة الجاهزة ومن ثم تم نقلها واختبارها حتى الانهيار في المختبرات الانشائية التابعة لجامعة البحرين. كان لجميع الجسور المستخدمة في هذه التجربة مقطع عرضي وطول ثابتان، بالإضافة الى ذلك كانت جميعاً مسلحة في الجزء السفلي بقضبان من ألياف الزجاج المقوى ولم يتم اضافة اي تسليح عرضي او تسليح علوي لها. تم تقسيم العوارض السبعة إلى ثلاث مجموعات، في كل مجموعة تم تحديد عامل متغير واحد بينما تم الحفاظ على جميع المعاملات المتغيرة الأخرى ثابتة. المعامل المتغير الاول في هذه التجربة هو نسبة امتداد القص إلى العمق (a/d)، والتي تضمنت ثلاث قيم وهي 2.0 و 2.7 و 3.5. العامل المتغير الثاني هو قوة ضغط الخرسانة (f'_c)، والتي تضمنت ما يقرب من 40 و 50 و 60 ميجا باسكال، بينما المعامل المتغير الثالث هو ونسبة التسليح الطولي (ρ)، والتي كانت 0.7 و 1.1 و 1.5 في المئة.

تم عرض نتائج اختبار هذه الدراسة على شكل سبب الانهيار وقوة تحمل القص القصوى ومقدار الانحناء مقارنة بالحمل ونمط التشقق. علاوة على ذلك، تمت مقارنة القوى القصوى التجريبية بسعة القص مع القيم المتوقعة من قبل معهد الخرسانة الأمريكي (ACI 440.1R-15)، وجمعية المهندسين المدنيين اليابانية (JSCE 1997)، والمبادئ التوجيهية الكندية (CSA S806-12)، وشبكة أبحاث ISIS الكندية (ISIS 2007). بالإضافة إلى ذلك، تم تضمين المعادلات المقترحة من قبل الدكتور شاه علام وباحثين آخرين في المقارنة.

أظهرت نتائج الدراسة التجريبية أن تغيير المعلمات الثلاثة له تأثير كبير على الحمل الأقصى ونمط التشقق ومقدار الانحناء مقارنة بالحمل، بينما ظل القص سبب الانهيار لجميع العينات. بالإضافة إلى ذلك، وجد أن المبادئ التوجيهية الكندية (CSA S806-12) ومعادلة الدكتور شاه علام يتنبأ بقوة القص لعوارض (T-beams) المسلحة بقضبان ألياف الزجاج المقوى بدقة واستمرارية جيدين.

في جزء المحاكاة، تم إجراء تحليل العناصر المحدودة باستخدام برنامج (Abaqus). ثم تم مقارنة نتائج هذه المحاكاة بالنتائج التجريبية لتقييم دقة البرنامج في التنبؤ بقوة القص وأنماط التشقق و مقدار الانحناء مقارنة بالحمل وأنماط الانهيار. تمت محاكاة العوارض النموذجية باستخدام نفس الخصائص الميكانيكية المستخدمة في الدراسة التجريبية. تم استخدام نموذج مرونة الضرر الخرسانة لنمذجة جميع العوارض السبعة. تم تطوير نموذجين لكل جسر، وفي كل نموذج منهما تم استخدام نموذج مختلف لقوة شد الخرسانة، بينما ظلت جميع الخصائص الأخرى دون تغيير. أظهرت النتائج تنبؤات جيدة تتفق إلى حد ما مع النتائج التجريبية للحمل المتسبب للتشقق الأول ولنمط الانهيار. ولكن لم تظهر النماذج نتائج دقيقة لجميع العوارض فيما يتعلق بقوة التحمل ومقدار الانحناء. أخيرًا، أظهر استخدام نموذجين لكل جسر أن نموذج قوة شد الخرسانة يؤثر بشكل واضح على نتائج النموذج.

جامعة البحرين



قسم الهندسة المدنية

كلية الهندسة

سلوك الجسور الخرسانية المسلحة بالبوليمر المقوى بالاياف الزجاجية تحت إجهاد القص

أطروحة مقدمة كجزء من متطلبات الحصول على الماجستير في الهندسة المدنية

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أبريل 2025