

Behaviour of GFRP-RC Continuous Deep Beams without Web Reinforcement

by

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ABSTRACT

Steel-reinforced concrete (RC) deep beams are one of the major components in the superstructure of bridges, they are used for load distribution as transfer girders or bent caps and pile caps, because of their capability to sustain higher loads compared to slender beams. In North America, such elements are exposed to harsh weather, which makes them more susceptible to corrosion problems. Thus, this study aims at investigating the feasibility and performance of using the non-corrodible glass fiber-reinforced polymers (GFRPs) in reinforcing concrete deep beams.

The GFRP material is known for its relatively low modulus of elasticity, with respect to steel, and its linear elastic stress-strain relationship up to failure. Also, GFRP bars have different surface and bond characteristics than those of steel, which require a longer development length. These different properties affect the behaviour of GFRP-RC structures. The effects on simply-supported beams and slabs have been studied extensively, while, the behaviour of GFRP-RC continuous deep beams has not been studied yet.

In this study, twelve large-scale RC deep beam specimens were constructed and tested to failure; three simply-supported and nine continuous over two equal spans. All specimens were reinforced with GFRP headed-end bars. The test variables were the shear span-to-depth ratio, which included 1.0, 1.5, and 2.0; and the top longitudinal reinforcement ratio in continuous beams which included 1.2, 1.0, and 0.8%. The cross-sectional dimensions for all specimens were 590-mm deep $\times$ 250-mm wide. The overall length of the simply-supported specimens was 2,100, 2,600 and 3,100 mm while it was 3,500, 4,500, and 5,500 mm for the continuous beams. The test specimens were divided into four series. One series included the three simply-supported specimens, while the nine continuous specimens were divided into three series. The test results were presented in terms of

ultimate strength, cracking, deflection and strains in reinforcement. In addition, the test results were compared to the predicted values by the Canadian standards CSA/S806-12 (CSA 2012) and CSA/A23.3-14 (CSA 2014), and the American Code ACI 318-14 (ACI 2014).

The test results confirmed the formation of the Strut-and Tie-Model (STM). In addition, it was observed that increasing the shear span-to-depth ratio and increasing the top longitudinal reinforcement ratio led to a significant decrease in the load carrying capacity of the beam.

*In dedication for my Mom, Dad, and Sisters for making me who I am,
and my beloved wife-to-be for supporting me throughout this part of my life.
I hope this achievement gets me a step closer to making you all proud of me.*

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CHAPTER 1: INTRODUCTION

1.1 General

Steel bars have been the available material used to reinforce concrete members in the last two centuries. However, the use of steel bars is usually associated with corrosion problems, which are obvious in some reinforced concrete (RC) structures especially parking garages, bridges and other civil infrastructure exposed to harsh environmental conditions. Corrosion of the steel reinforcement leads to costly maintenance and service operations and, in the meantime, reduces the life span of the structures. Many techniques have been used to overcome corrosion of reinforcing steel such as the use of galvanized or stainless-steel bars, epoxy-coating and cathodic protection. However, none of these methods provided a radical solution to the steel corrosion problem. Recently, fiber-reinforced polymer (FRP) reinforcement has been established as an alternative to steel because of its non-corrosive nature. In addition, FRP bars have favorable properties such as high tensile strength-to-weight ratio and high fatigue resistance. Also, the transparency of FRP bars to magnetic and electrical fields makes them a viable alternative to steel reinforcement in applications sensitive to electromagnetic fields such as magnetic resonance imaging (MRI) units. On the other hand, FRP bars have different characteristics from those of steel such as low modulus of elasticity, linear stress-strain relationship until failure and low resistance in the transverse direction.

In the last two decades, many researchers investigated the feasibility of using FRP instead of steel in new structures as well as in retrofitting and strengthening of existing aging structures (ISIS Canada 2007). In these studies, the main objectives were to compare the behaviour of structural elements reinforced with FRP bars with their counterparts reinforced with steel. These studies also

focused mainly on simple structures such as simply-supported slender beams and one-way slabs (Matthys and Taerwe 2000; Zhang et al. 2004; El-Sayed et al. 2005). Based on the results of these research studies, expressions for the design of FRP-RC members were developed taking into account the difference in the mechanical and physical properties of FRP materials compared to those of steel bars.

1.2 Problem Definition

Reinforced concrete deep beams are structural members with a relatively small shear span-to-depth ratio ($a/d < 2.0$) compared to slender beams. Thus, the behaviour of such beams is dominated by shear failure. Such type of failure is undesirable in concrete structures as it is sudden and brittle. However, deep beams have a higher load capacity compared to slender beams. Consequently, deep beams are used in many applications such as transfer girders, pile caps, and foundation walls. It is worth mentioning that the arch action is the load transfer mechanism in deep beams unlike beam action in slender beams. In other words, the deep beams cannot be designed with linear analysis. At high loading levels, the load is transferred from the loading points to supports through compression struts at which stage, the longitudinal reinforcement works as a tie forming what is known as a Strut-and-Tie Model (STM). Figure 1.1 shows a schematic drawing for beam action in slender beams and arch action in deep beams.

The design procedure of the steel-RC deep beams using STM is based on the lower-bound theorem, which assumes that the main longitudinal steel reinforcement has yielded before the failure of any other element of the STM to ensure that the member has sufficient ductility to allow redistribution of forces in the assumed model (Tuchscherer et al. 2011; Wight and MacGregor 2009). However, FRP bars are elastic materials without yielding; therefore, the capacity of the FRP-RC deep beams

is still consistent with the lower-bound theorem since the failure is governed by crushing of the concrete before rupture of the FRP bars. High strains in the concrete and the FRP bars before failure allow for redistribution of the internal forces and provide sufficient deformability.

However, the differences in the mechanical and physical properties between FRP and steel, including lower modulus of elasticity, lower transverse strength, higher tensile strength, lower strain at failure, bar surface and bonding characteristics, result in a different shear behaviour in FRP-RC members compared to that of steel-RC counterparts. In general, RC members resist shear force by means of shear resistance of un-cracked concrete, aggregate interlock, dowel action of reinforcement, residual tensile stresses across crack, and arch action. Thus, the shear resistance of FRP-RC members is believed to be lower due to the deeper and wider cracks, which will adversely affect the contribution of un-cracked concrete and aggregate interlock. Moreover, the dowel action for FRP bars is always less than that of steel bars because of its lower transverse strength. In addition, using FRP instead of steel in deep beams, will result in higher strain which negatively affects the efficiency of the strut. Conversely, the contribution of the arch action mechanism to the shear strength in FRP-RC deep beams can be improved because of the relatively higher tensile strength of FRP bars (Omeman et al. 2008).

Furthermore, end anchorage failure is one of the common failure mechanisms in deep beams, due to the high and constant tensile stresses developed in the longitudinal reinforcement (tie) near supports after the formation of the STM as shown in Figure1.1. This type of failure is considered premature as it does not represent the failure of the main member in the STM (strut). Thus, a longer development length is required, or the reinforcement must be anchored by adequate hooks or using mechanical anchors (e.g. headed-end bars).

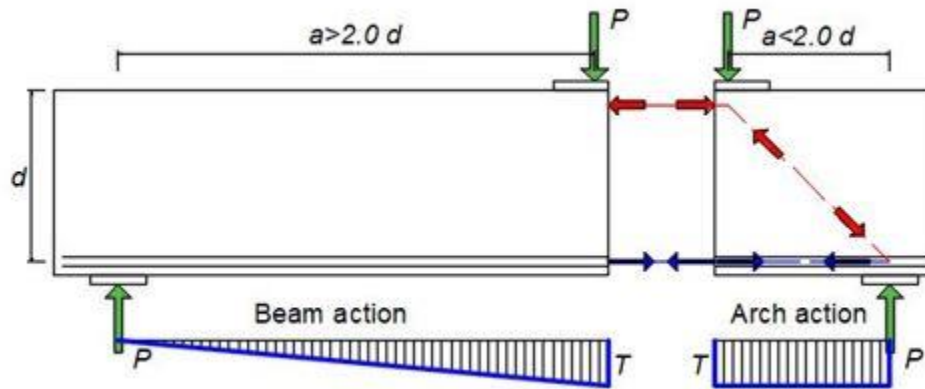


Figure 1.1: Beam action and arch action

In addition, based on the different reinforcement detailing and the middle support reaction in two-span continuous beams, a different STM is developed in the continuous deep beams as shown in Figure 1.2. Also, in continuous beams, the location of maximum shear force and bending moment coincide. The coexistence of high shear force and high bending moment at the interior shear span have a significant effect on development of cracks, which reduces the effective strength of the concrete strut. Therefore, the behaviour of continuous deep beams is different from that of simply-supported deep beams.

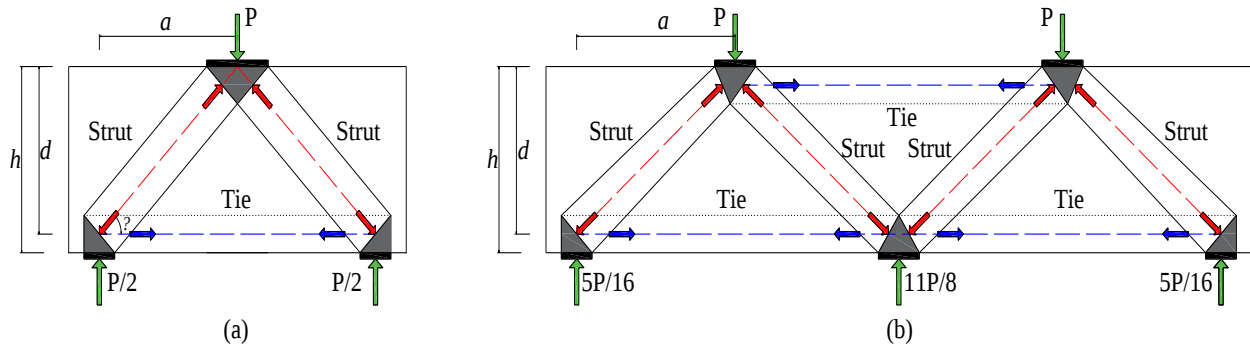


Figure 1.2: (a) STM in simply-supported deep beam and (b) STM in two span deep beam.

Moreover, to date, limited research has been carried out on FRP-RC simply-supported deep beams while there are no research studies on FRP-RC continuous deep beams. Therefore, this study aims at investigating the shear behaviour of concrete simply-supported and continuous deep beams reinforced with headed-end GFRP bars, and the ability of the headed end bars to provide adequate anchorage for the tie. Furthermore, this study examines whether or not the current strut-and-tie design provisions in relevant codes can be safely applied to continuous deep beams.

1.3 Research Significance

The shear behaviour of steel-RC continuous deep beams has been intensively investigated and it is well-established. Therefore, design provisions (strut-and-tie model) are provided in different codes in which steel-RC simply-supported and continuous deep beams are designed using the same design provisions. On the other hand, very limited studies were conducted to investigate the behaviour of simply-supported deep beams reinforced with FRP bars (Omeman et al. 2008; El-Sayed et al. 2012; Andermatt and Lubell 2013; Farghaly and Benmokrane 2013; Mohamed et al. 2017). Also, up to the author best knowledge, there is no research on the behaviour of continuous deep beams reinforced with FRP bars. This research program is dedicated to partially fill this

knowledge gap by evaluating the behaviour of continuous FRP-RC deep beams. In addition, the applicability of the shear design provisions in the current codes and guidelines is examined. Moreover, this study aims at investigating the feasibility of using headed-end bars in FRP-RC deep beams as a mean of providing the required development length of the tie reinforcement.

1.4 Objectives of Research

This research study is dedicated to investigating the behaviour of both simply-supported and continuous concrete deep beams reinforced with GFRP bars. The main objectives of this research are to:

- Study the behaviour of simply-supported RC deep beams reinforced with headed-end GFRP bars.
- Investigate the behaviour of continuous concrete deep beams reinforced with headed-end GFRP bars.
- Examine the ability of headed-end bars to provide sufficient anchorage for the tie in FRP-RC deep beams.

These objectives can be achieved by studying the effect of the shear span-to-depth ratio and the top longitudinal reinforcement ratio on the behaviour of such beams.

1.5 Research Scope

The scope of this study is restricted to concrete deep beams totally reinforced with glass fiber reinforced polymer (GFRP) reinforcing bars without web reinforcement. Among available types of FRP, GFRP is the most popular since it has the highest strain at failure and lowest price. Simply-supported deep beams and two-equal-span continuous deep beams with rectangular cross section

(250 × 590 mm) are examined in this study. The shear span-to-depth ratio ranged between 1.0 and 2.0. The beams are tested under one-point loading at the middle of each span.

1.6 Methodology

An experimental research program is designed to achieve the above-mentioned objectives. The study includes testing nine large-scale GFRP-RC deep beams continuous over two equal spans and three simply-supported GFRP-RC deep beams. The beams have 250 mm width and 590 mm height. The beams length varies because of varying shear span-to-depth ratio. The beams are reinforced with GFRP bars as longitudinal reinforcement.

The test parameters are:

1. Shear span-to-depth ratio (a/d) of 1.0, 1.5 and 2.0
2. Longitudinal reinforcement ratio of the top tie (0.8, 1.0 and 1.2%) while the bottom reinforcement ratio is kept constant at 1.0%.

1.7 Thesis Outline

This thesis consists of six chapters. Chapter 1 presents the problem definition, research significance, objectives, scope, and the methodology to achieve the objectives of this research.

Chapter 2 presents review of the previous work that identifies pertinent experimental investigations and establishes the main factors that affect the behaviour of steel- or FRP-RC deep beams. Also, the code provisions for RC deep beams are introduced in this chapter.

The experimental program is described in Chapter 3. In this chapter, the dimensions and reinforcement details of a total of 9 continuous and 3 simply-supported GFRP-RC deep beams are

described. Moreover, the properties of the used materials, details of instrumentations, test setup and procedure are provided in this chapter.

In Chapter 4, a comparison between the behaviour of the three simply-supported beams (having a longitudinal reinforcement ratio of 1.0% and variable a/d of 1.0, 1.5 and 2.0) and three continuous beams with similar bottom reinforcement and a/d ratios and a top reinforcement ratio of 0.8% is included to highlight the effect of the continuity on the capacity of GFRP-RC deep beams.

Chapter 5 is dedicated to explaining the behaviour of the continuous concrete deep beams reinforced with GFRP and the applicability of the STM to such beams. The effect of the shear span-to-depth ratio and the top reinforcement ratio on the behaviour of such beams is presented.

Chapter 6 contains a summary of the major findings of the program and the important conclusions based on the test results and analysis of the test specimens. Recommendations for future work are included as well.

CHAPTER 2: LITERATURE REVIEW

2.1 Introduction

Deep beams represent the main load-transfer elements in bridges, pile caps and in high-rise buildings. The behaviour of steel-reinforced concrete (RC) simply-supported and continuous deep beams has been extensively investigated. It was concluded that the best method to design such elements is the Strut-and-Tie Model (STM) but with different limits and formulae. Strut-and-tie model has been explained in many codes and guidelines including the Canadian standards (CSA/S806 2012; CSA/A23.3 2014; CSA/S6 2014) and the American codes (AASHTO LRFD 2007; ACI 318 2014).

There are many parameters that affect the shear behaviour and shear capacity of deep beams. These parameters include shear span-to-depth ratio, effective depth, concrete strength, longitudinal and transverse reinforcement ratios, and supports and loading conditions. In the following sections, the general behaviour of RC deep beams as well as the main findings in the available literature on simply-supported and continuous steel- and FRP-RC deep beams are discussed.

2.2 Behaviour of Deep Beams

Wight and MacGregor (2009) defined deep beam as a member in which the loads are transferred through a compression strut which is formed from loading point to supporting point. The loading point must be at a distance less than or equal to 2.0 times the member height from the support point. The magnitude of concentrated loads is not a condition needed so that the beam acts as a deep beam. In uniformly loaded members, the clear span (L_n) must be less than or equal to four times the height of the member (h). Figures 2.1 and 2.2 show examples of bent caps and

continuous deep beams in bridges. The deep beam is usually used to transfer the loads from one or two columns (or smaller beams) to the supports which can be columns, walls or piles. As mentioned before, deep beams cannot be designed using linear analysis as the case in slender beams; however, it is designed using Strut-and-Tie Model (STM). In deep beams, the elastic analysis can be used only before cracking. After cracking, a significant stress redistribution occurs as there are no tension forces can be transmitted across the cracks (Leonhardt and Walther 1966; ASCE-ACI Committee 426 1973; Schlaich et al. 1987; Oh and Shin 2001; Tuchscherer et al. 2014). This redistribution of stresses causes the formation of additional cracks which work as a guide to show the flow of the stresses and forces after cracking. Figure 2.3(a) shows the stresses trajectories in simply-supported deep beam where the dashed lines represent the flow of compression stresses and the solid lines represent the flow of tension stresses. The cracks are expected to be perpendicular to the flow of tension stresses as shown in Figure 2.3(b). Also, it is important to note that the flexural stress at the bottom is constant over much of the span (Oh and Shin 2001).



Figure 2.1: Bent caps

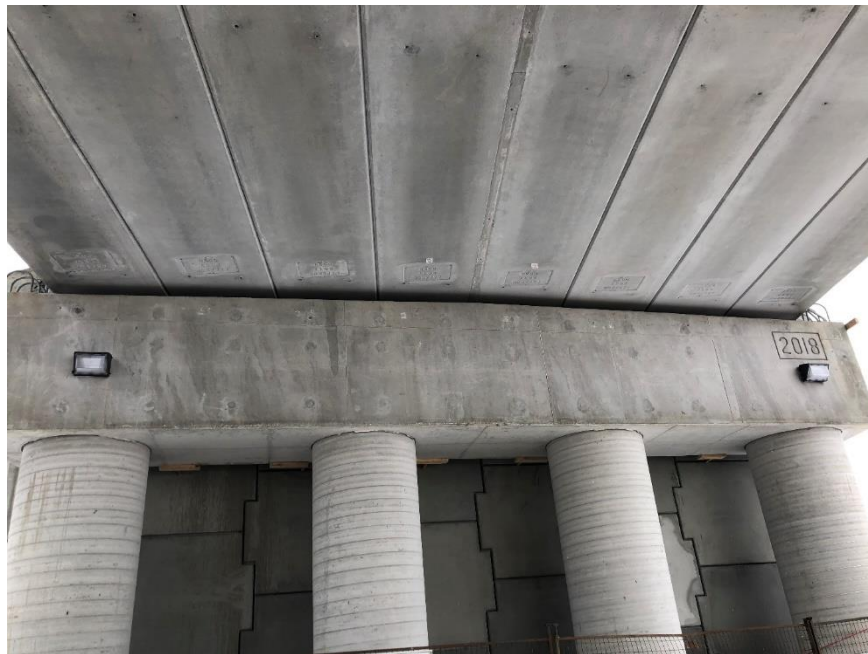


Figure 2.2: Three-span deep beam

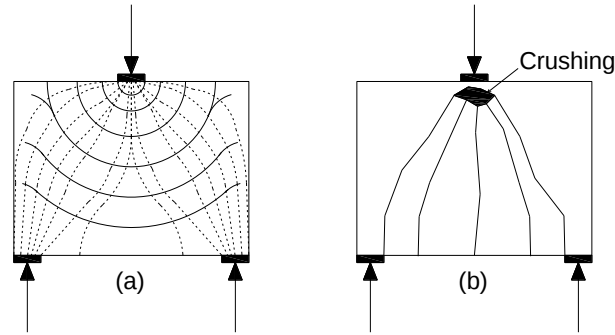


Figure 2.3: (a) Stress trajectories and (b) cracking of simply supported deep beam (reproduced from Wight and MacGregor 2009)

The ASCE-ACI Committee 426 (1973) reported that the shear failure in RC deep beams is different from that in RC slender beams due to the steeper cracks that propagate between the loading points and supports. Also, it was stated that the arch action is responsible for transmitting the vertical concentrated loads to the supports through the strut (Schlaich et al. 1987; Hsu 1988). Figure 2.4 shows the components of the STM in a simply-supported RC deep beam, which are the compression strut, the tie (longitudinal reinforcement) and the nodes (at which the strut and the tie intersect). It can be seen that the inclined force at the nodal zone at the support is resisted by vertical reaction from the support while the horizontal component of the compression force is resisted by the tie. Moreover, it was noted that RC deep beams are susceptible to failure because of slipping of the reinforcing bars that form the ties (anchorage failure). This failure does not reflect the real capacity of the STM.

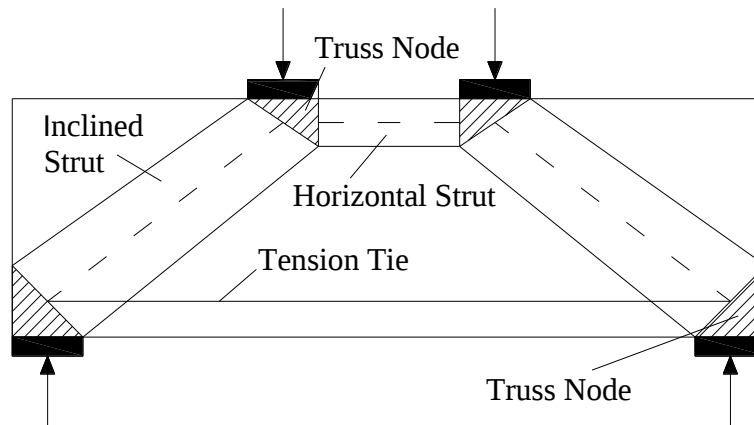


Figure 2.4: Arch action in deep beams with two loading points (reproduced from ASCE-ACI Committee 426 1973)

According to the ASCE-ACI Committee 426 (1973), many types of failure could occur after formation of an inclined crack in a deep beam without web reinforcement. The modes of failure of deep beams are shown in Figure 2.5 and are as follows:

- a) Anchorage failure of longitudinal reinforcement due to slipping of the bars.
- b) Crushing of concrete at the support.
- c) Flexural failure due to yielding or rupture of bars at the bottom tie.
- d) Tension failure of the arch rib.
- e) Crushing along the crack due to compression stress in strut.

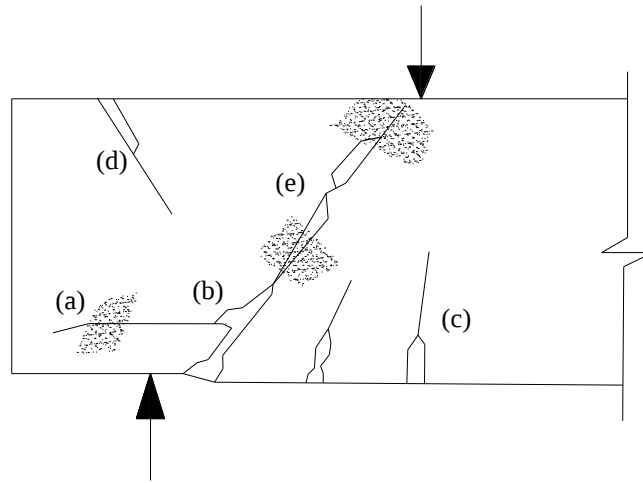


Figure 2.5: Types of failure in deep beams (reproduced from ASCE-ACI Committee 426 1973)

2.3 Strut-and-Tie Model

Schlaich et al. (1987) reported that the beams are mainly divided into two regions; B-region and D-region (Figure 2.6). The B-region is the region where Bernoulli hypothesis of plane strain distribution is valid. These B regions are designed based on internal stresses (bending moment, torsion, axial forces and shear). On the other hand, D-regions are mainly located at a distance equals to the effective depth (d) from the face of supports and loads. After cracking, the D-regions are designed by the STM.

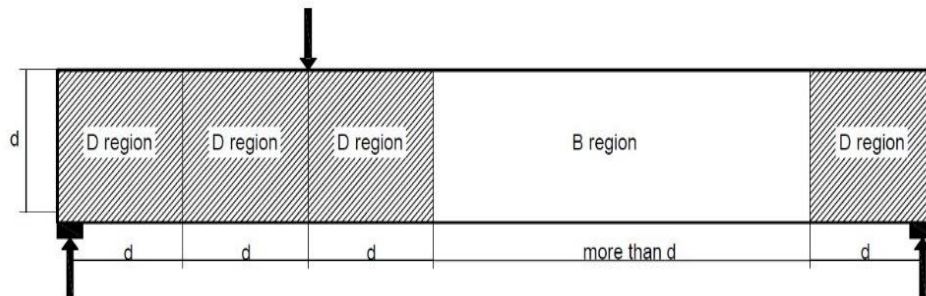


Figure 2.6: B-region and D-regions (reproduced from Schlaich et al. 1987)

The STM is widely used in many applications such as deep beams, corbels, dapped ends, post-tensioned anchorage zones, or other structural components with loading or geometric discontinuities. The STM condense all the stresses in compression and tension members and connect them by nodes. Figure 2.7 shows examples of the STM in different structural members.

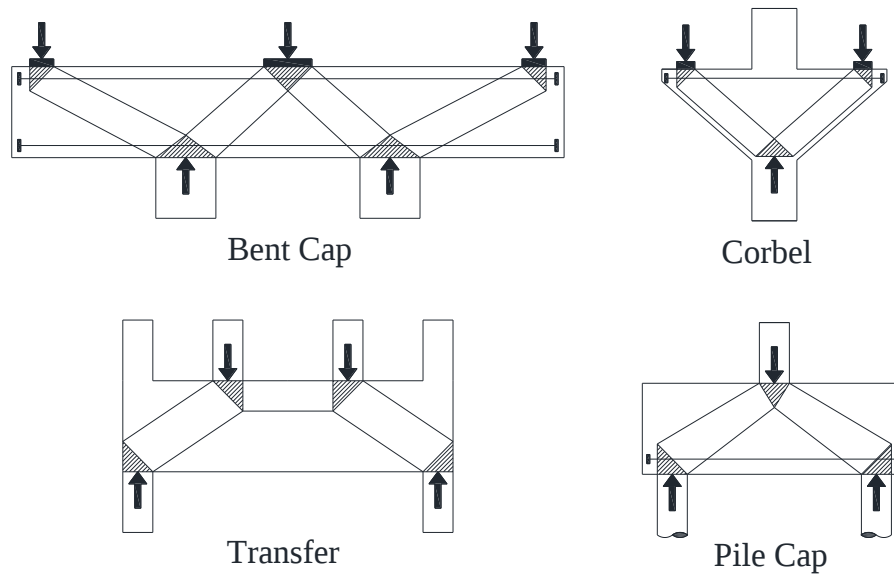


Figure 2.7: Examples of Strut-and-Tie Model in different structures (reproduced from Schlaich et al. 1987)

Wight and MacGregor (2009) stated that the STM in a simply-supported RC deep beam consists of two concrete compression struts, longitudinal reinforcement working as a tension tie and joints or “nodes” as shown in Figure 2.8. The nodes are surrounded by concrete area known as “nodal zone”. The nodal zones are responsible for transferring the force from the struts to ties and supports. The STM is mainly used in designing D-regions and it should satisfy the following conditions:

- a) It produces a system of forces which is in equilibrium with the loads.

- b) The factored member forces of the struts, ties and nodes do not exceed the design member strength at any section.
- c) Structures must be ductile to make the transition from elastic to plastic behaviour to redistribute the forces, achieve equilibrium, and not exceed the designed member strength.

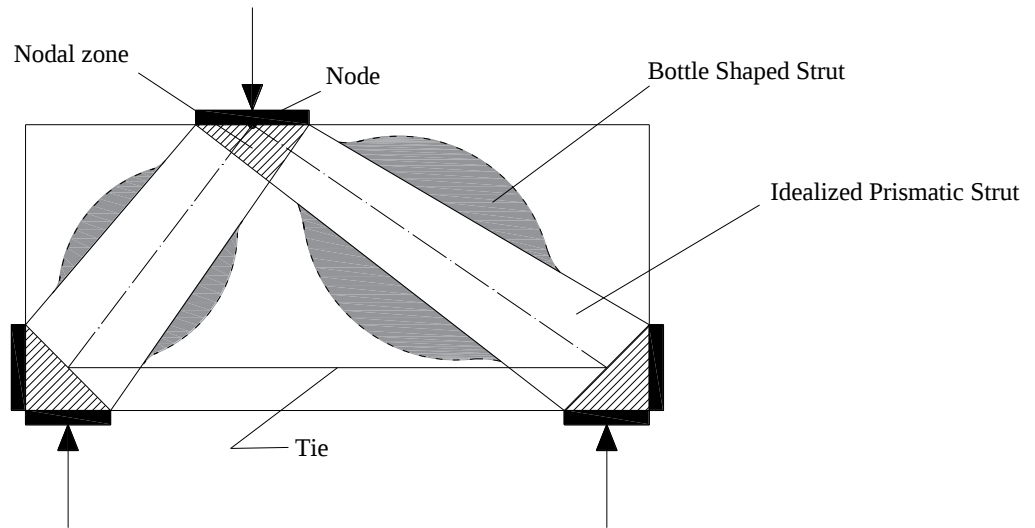


Figure 2.8: Strut-and-Tie model of a deep beam with one-point loading (reproduced from Wight and MacGregor 2009)

2.3.1 Struts

Struts are concrete compression members that transfer the load from loading point to support and surrounded by compression stress acting parallel to it (Wight and MacGregor 2009). It is worth mentioning that the failure of the strut reflects the real capacity of the STM. The compression stress fields in the concrete are wider at the middle of the strut than that at its ends. Therefore, the struts cross-section is changeable along its length (bottle shape struts); however, they are idealized by local strut and tie models as shown in Figure 2.8.

In RC deep beams without transverse reinforcement, the struts fail after diagonal crack occurs due to the transverse tension stresses. The factors that affect the compression strength of the struts are:

- a) Concrete compressive strength.
- b) Load duration effect (the reduction of compressive strength under sustained load).
- c) The transverse tensile strains.
- d) Longitudinal cracks.

2.3.2 Ties

Ties are the second major component in STM (Wight and MacGregor 2009). They represent one or several longitudinal bars in the same direction. The ties consist of longitudinal reinforcement surrounded by a prism of concrete as shown in Figure 2.8. The main role concrete prism is to transfer loads from strut or from bearing areas to ties through bond with reinforcement. The anchorage of the ties at the nodal zones is important part in the design of the STM, as such failure is considered as a premature failure. Ties must be anchored at the nodes by bond, hooks, or mechanical anchorage between the ends of the bar and the point at which the centroid of the tie reinforcement leaves the compressed extended portion of the nodal zone.

2.3.3 Nodes and nodal zones

Wight and MacGregor (2009) defined the nodes as the points where struts and ties meet in STM. They are idealized as pinned joints. Nodal zones are the concrete surrounding the nodes (Figure 2.8). There are always three or four forces acting on the node to achieve node equilibrium. Accordingly, nodal zones are classified as C-C-C, C-C-T or C-T-T depending on the type of forces acting on the node (Figure 2.9) where “C” and “T” stands for compression and tension, respectively.

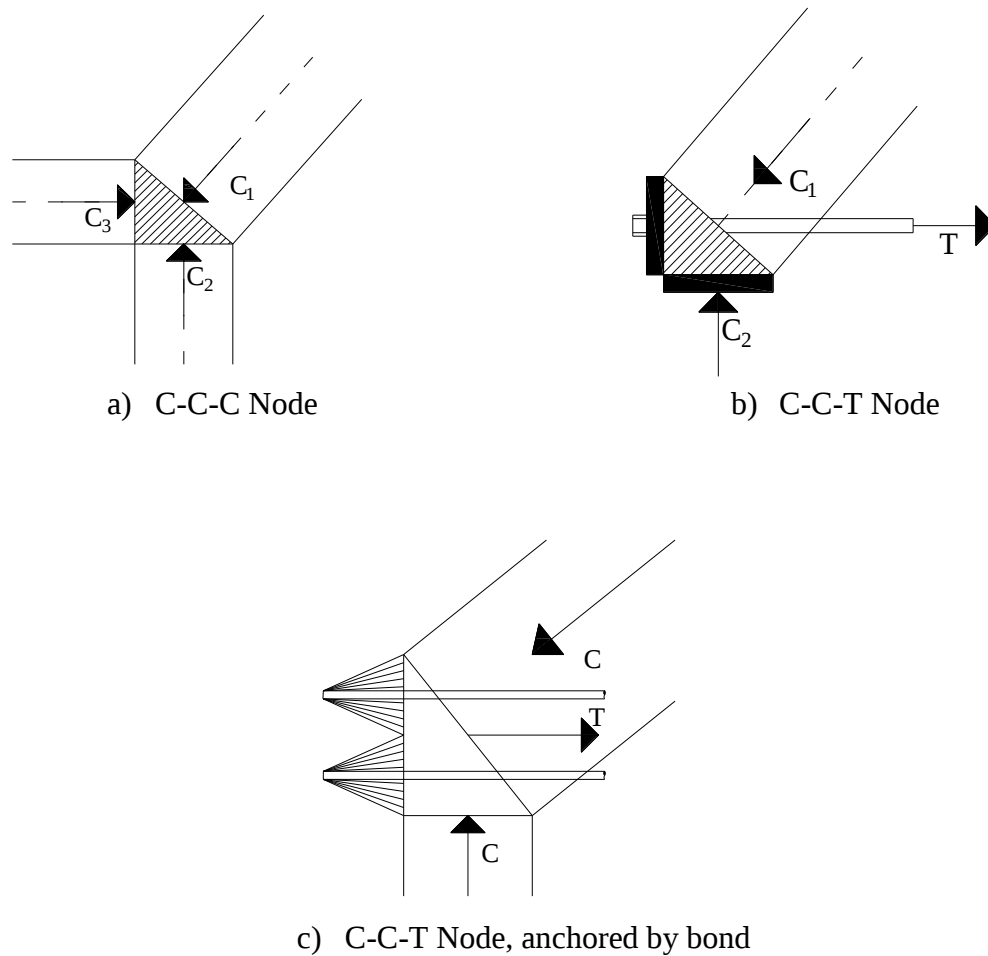


Figure 2.9: Hydrostatic nodal zones (reproduced from Wight and MacGregor 2009)

Nodal zones always fail by crushing of concrete due to compression forces from the strut and the bearing plates of loads or support. Therefore, it is important to make sure that the compression forces of the struts and of the bearing plates are within limits.

2.4 Simply-Supported Steel-RC Deep Beams

There are many factors affecting the behaviour and the strength of RC deep beams.

- 1) Shear span-to-depth ratio.
- 2) Section depth.

- 3) Concrete compressive strength.
- 4) Web reinforcement.
- 5) Longitudinal reinforcement ratio.

In the following subsections, the effect of each parameter on the behaviour of such beams is discussed.

2.4.1 Effect of shear span-to-depth ratio

Manuel et al. (1971) tested 12 steel-RC deep beams using four-point bending loading scheme with variation in the shear span-to-depth ratio (a/d) and clear span-to-depth ratio (l/d). It was reported that specimens with $a/d = 0.3$ exhibited web compression mode of failure, while specimens with $a/d = 0.65$ and $a/d = 1.0$ exhibited flexural failure by crushing of concrete between the two loading points, due the increase in the compressive stresses in the horizontal strut. It was also concluded that, decreasing a/d caused a significant increase in the ultimate capacity due to the change in the mode of failure, unlike the l/d which had insignificant effect on the ultimate capacity. In other words, the extent of the arch action decreased with increasing l/d for specimens with the same a/d (Smith and Vantsiotis 1983; Rogowsky et al. 1986; Oh and Shin 2001; Tan and Cheng 2006).

Oh and Shin (2001) tested 53 simply-supported RC deep beams. The beams had rectangular cross section of 120×560 mm or 130×560 mm. The variables were the concrete compressive strength, span-to-depth ratio, longitudinal reinforcement ratio, and horizontal and vertical web reinforcement ratios. It was concluded that a/d is a very important parameter affecting the behaviour of RC deep beams, as a change in the mode of failure with increasing the shear span-to-depth ratio was reported. In case of specimens with a/d of 0.5 and 0.85, two diagonal cracks formed between the two loading points and the supports and the failure was due to crushing of concrete strut between the two diagonal cracks. On the other hand, specimens with a/d of 1.25 and

2.0, failed due to splitting failure of the compression strut. It was also reported that the ultimate shear capacity of RC deep beams is mainly affected by a/d more than any other parameter as there was significant increase in the ultimate capacity with decreasing a/d from 2.0 to 0.5. This indicates that the tied arching action becomes less effective with increasing a/d because of the reduced angle between the inclined strut and longitudinal axis of the beam. Also, an increase in the axial stiffness of the specimens was reported with decreasing a/d with similar values of the mid-span deflection.

2.4.2 Effect of section depth

Tan and Lu (1999) reported that increasing the section depth (h) in geometrically similar specimens led to an increase in the propagation of cracks at the same shear stress with a negligible effect on the diagonal cracking load. However, a steep decrease in the shear strength with increasing h from 500 to 1000 mm was reported while no significant effect beyond this range was observed. It was also observed that in beams with $a/d = 1.0$, increasing h led to an increase in the mid-span deflection. Moreover, increasing the depth of the beams led to a less violent failure which was attributed to the propagation of more cracks in the deeper beams that, in turn resulted in increasing the released energy during the loading history. Zhang and Tan (2007) concluded that the size effect can be mitigated by using a proportional loading and support plates width to the beam height.

Bircher et al. (2014) performed an experimental study to examine the effect of section depth on the behaviour of deep beams. Twelve full-scale simply-supported RC beams were tested. The specimens had a/d of 1.2, 1.85 and 2.5 and a depth of 584, 1,067 and 1,905 mm. It was reported that the normalized shear strength decreased with increasing the section depth especially for deep beams having $a/d < 2.0$. In addition, for specimen with $a/d < 2.0$, increasing the section depth did

not affect the diagonal cracking load. However, in specimens with a/d of 2.5, a reduction in the diagonal cracking load was noted with an increase in the section depth. The normalized shear strength of the specimens decreased with increasing the depth in beams with constant shear span-to-depth ratio. For example, at $a/d = 1.2$, a significant decrease in the normalized shear strength was observed with increasing the depth.

2.4.3 Effect of compressive strength of concrete

Smith and Vantsiotis (1983) reported that beams with lower concrete compressive strength failed at lower load even with adding web reinforcement. This indicates the great influence of the concrete compressive strength on the load capacity of RC deep beams. It was also reported that the effect of the concrete compressive strength diminishes with increasing the shear span-to-depth ratio.

Oh and Shin (2001) carried out a study to investigate the effect of the concrete strength on the shear strength of RC deep beams. It was found that beams made of normal or high strength concrete with small shear span-to-depth ratio failed suddenly. Also, it was concluded that the failure mode and the ultimate load capacity depend on the shear span-to-depth ratio more than the compressive strength of concrete.

2.4.4 Effect of web reinforcement

Smith and Vantsiotis (1983) observed that RC deep beams without web reinforcement exhibited larger crack width at failure and more cracks than that in beams with web reinforcement. Also, web reinforcement increased the ultimate shear capacity by approximately 30%. The use of vertical web reinforcement only increased the ultimate shear capacity of the beams; however, this

increase diminished in beams with a/d less than 1.0. It was also noted that the effect of the vertical web reinforcement diminishes with increasing the horizontal web reinforcement ratio.

Oh and Shin (2001) observed that all RC deep beams without web reinforcement failed suddenly without any warning. The same was observed in beams with web reinforcement and low a/d ranging from 0.5 to 0.85. At higher a/d (1.25 and 2.0), web reinforcement showed a great role in preventing the sudden shear failure. The results showed that the ultimate shear capacity of the members increased slightly as the vertical web reinforcement (ρ_v), increased from 0.12 to 0.34%. Also, it was observed that the effect of shear reinforcement decreases as a/d values decrease. On the other hand, the horizontal web reinforcement (ρ_h) did not show a significant effect on the ultimate shear capacity with high strength concrete.

Birrcher et al. (2013) performed an experimental study to examine the minimum amount of web reinforcement needed in RC deep beams considering the strength and the serviceability of such beams. Twelve specimens with a/d of 1.2, 1.85 and 2.5 were tested to failure. For each a/d , the amount of web reinforcement in vertical and horizontal direction ranged from 0 to 0.3%. It was observed that, at a/d of 1.85 and 1.2, increasing the web reinforcement did not play any role in increasing the normalized shear strength. Moreover, at $a/d = 2.5$, increasing the reinforcement from 0 to 0.3% increased the shear strength of the specimens significantly. However, it was concluded that increasing web reinforcement had a great effect on serviceability. In beams with a/d of 1.2, 1.85 and 2.5, increasing the web reinforcement led to a reduction in the diagonal crack width while no change was observed in diagonal cracking load in these beams. Moreover, a 0.3% web reinforcement in both directions was recommended to enhance both the serviceability and ultimate strength.

2.4.5 Effect of longitudinal reinforcement ratio

Oh and Shin (2001) studied the effect of the longitudinal reinforcement ratio on 53 specimens. It was reported that increasing the longitudinal reinforcement ratio increased the shear capacity of the deep beams.

Garay and Lubell (2008) conducted a research to investigate the behaviour of concrete deep beams reinforced with high strength longitudinal reinforcement. Six beams divided into three groups were tested. The test variables were the shear span-to-depth ratio and longitudinal reinforcement ratio while all test beams had the same transverse reinforcement ratio. Three modes of failure were observed; shear failure, failure due to crushing of the top strut between loading points and diagonal strut failure. The experimental results showed that with increasing the longitudinal reinforcement ratio, stiffness of beam increased while the ductility decreased. Also, as the longitudinal reinforcement ratio increases a significant increase of load capacity was observed.

2.5 Continuous Steel-RC Deep Beams

The behaviour of continuous deep beams is different from that of simply-supported deep beams, due to high shear and bending stresses that occur in the interior shear span, resulting in development of more cracks and decrease in the effective strength of the concrete strut. Also, the developed tensile strains in longitudinal top and bottom reinforcement adversely affect the efficiency of the concrete strut (Rogowsky et al. 1986; Ashour 1997; Yang et al. 2007; Yang and Ashour 2007).

Rogowsky et al. (1986) reported the test results of 17 continuous steel-RC deep beams. The tested beams had either vertical web reinforcement or horizontal web reinforcement. It was observed that

using vertical web reinforcement changed the behaviour of specimens from brittle to ductile. Also, deep beams with high vertical web reinforcement, four times the minimum amount assigned by ACI building code (ACI Committee 318 1983), showed a better ductile behaviour and shear capacity compared to those of beams with less vertical web reinforcement. On the other hand, it was observed that the shear capacity decreased with increasing a/d in beams without web reinforcement, in beam with high or low horizontal web reinforcement, and in beams with low vertical web reinforcement. At the meantime, the horizontal web reinforcement had a very small effect on the strength of the test beams. This contradicted the predictions of the ACI code (ACI Committee 318 1983) where it was expected that the horizontal web reinforcement would have a greater effect on the capacity of the beam. Also, the ACI Committee 318 (1983) suggested that the continuous deep beams exhibit higher value of V_c than the simply-supported beams. This was attributed, in case of beams with heavy stirrups, to that even with a high number of stirrups there were loud sounds with the initiation of the inclined cracks which occurred at the interior shear span of the two spans in which the compression strut was formed. Moreover, the failure was due to either crushing of the compression strut or vertical opening of the inclined cracks that tended to pull the compression strut apart. In case of continuous deep beams without heavy stirrups, the inclined cracks initiated suddenly, followed by brittle failure before yielding of the longitudinal reinforcement. Beams having a/d lower than 1.5 exhibited compression strut failure with opening in the inclined cracks while beams with a/d higher than 1.5 showed diagonal tension failure with opening in inclined cracks. The longitudinal reinforcement strains were constant along the spans which confirm the formation of the tie. The failure of this specimen was brittle, as the specimens failed before yielding of the steel.

Ashour (1997) tested eight steel-RC continuous deep beams. The main parameters were a/d and the web reinforcement. The amount of web reinforcement used varied from none to high amount with different configurations. It was observed that the vertical reinforcement had a greater effect on the load carrying capacity of the specimens than the horizontal web reinforcement. It was also observed that the beams with horizontal web reinforcement only exhibited the lowest ductility while more ductile behaviour was observed in specimens with high a/d .

Chemrouk and Kong (2004) studied the behaviour of twelve two-span continuous steel-RC deep beams. The test parameters were the shear span-to-depth ratio (0.45 and 0.24), and the arrangement and ratio of web reinforcement. The flexural cracks formed first at the mid-span sections, then it started to propagate above the middle support. It was reported that continuous deep beams showed higher shear strength and a better behaviour after formation of diagonal cracks compared to that of identical single-span deep beams. Also, it was noted that due to the presence of web reinforcement, the mode of failure changed from diagonal shear failure to shear-bearing failure. Moreover, it was observed that the measured reaction at the middle support was lower than the theoretical reaction which means redistribution of internal forces took place.

Yang et al. (2007) carried out a study to investigate the behaviour of steel-RC deep continuous beams. Twenty-four specimens with different a/d , concrete strength and different amount, configuration and ratio of web reinforcement were tested. The authors observed that propagation of cracks was greatly affected by a/h , without significant effect from the shear reinforcement. For beams with $a/h = 0.5$, the diagonal crack was the first crack to propagate at about 40% of the ultimate capacity at the mid-height of the beam within the interior shear span. Then, the flexural cracks formed at mid span sections followed by flexural cracks over the middle support at 80% of the ultimate capacity. For beams with $a/h = 1.0$, the cracks firstly initiated over the middle support

followed by diagonal cracks. Afterwards, the flexural cracks of the mid-span regions formed with rare cracks in the exterior shear span for all beams. The stiffness of the beams was greatly affected by f_c' and a/h as the stiffness increased with increasing f_c' and decreasing a/h . On the other hand, no effect of the web reinforcement on the stiffness was observed. The experimental values of the support reaction were compared to the theoretical reactions and it was observed that before the formation of the diagonal crack, the measured reactions at supports were approximately similar to the predicted values. However, after the formation of diagonal cracks, the reaction of the middle support increased or decreased by 7% and 12% for beams with $a/h = 0.5$ and $a/h = 1.0$, respectively. The diagonal crack width was affected by both a/h and the web reinforcement. In other words, with larger a/h , the angle of inclination of the diagonal crack decreased; therefore, the vertical web reinforcement was more effective in controlling the crack. Also, with lower a/h , the angle increased, and the crack tended to be vertical. In this case, the crack is restrained by the horizontal web reinforcement. It was observed that the strains in all bars were low before the formation of the diagonal crack. The vertical web reinforcement reached the yielding strain first in beams with higher a/h while the horizontal bars reached the yielding point first in beams with low a/h . The ultimate shear capacity of the specimens was mainly affected by a/h and the configuration of the web reinforcement.

Yang et al. (2007) tested 12 steel-RC continuous deep beams to study the effect of increasing the section depth. The main studied parameters were the beam depth (varied from 400 to 720 mm), concrete compressive strength and shear span-to-depth ratio. All beams had the same longitudinal top and bottom reinforcement without web reinforcement. The failure mode and cracking pattern were similar when using normal or high concrete strength, but they were more influenced by section depth and a/d . With increasing h , the number and the depth of cracks decreased. It was

also observed that increasing a/d increased the width of cracks and reduced the stiffness of specimens significantly. Also, specimens with $a/h = 1.0$ failed soon after the formation of diagonal cracks. A significant decrease in the ultimate strength with increasing section depth was noted. This decrease was more pronounced in specimens with high strength concrete. Moreover, it was noted that the flexural cracks over the middle support and in the spans as well as the development of tensile strains in the longitudinal top and bottom reinforcement have a prominent influence on the reduction of the effective strength of concrete struts within the interior shear span.

2.6 Simply-Supported FRP-RC Deep Beams

2.6.1 Fiber reinforced polymers

Concrete structures are normally reinforced with non-prestressed and prestressed steel reinforcement that is susceptible to corrosion, which results in a threat to the integrity and safety of structures. Due to the costly rehabilitation work, research has been conducted to find technical solutions to this issue.

In the last few decades, researchers found that using Fiber Reinforced Polymers (FRP) reinforcement is a promising alternative to steel reinforcement, as FRP bars are made of non-corrodible materials. FRP is a composite material made of fibers embedded in a polymeric resin. The use of FRP in the construction industry became more popular due to the improved durability, reduced maintenance costs, saving from easier transportation and improved onsite productivity, and low relaxation characteristics (ISIS Canada 2007; Mallick 2007; ACI 440.1R 2015).

The mechanical properties of FRP differ from that of steel bars. FRP bars are characterized by their high tensile strength with low strain at failure and no yielding plateau. In addition, FRP

materials are anisotropic and this affects the shear strength and the dowel action of FRP bars. Accordingly, a change in the traditional design philosophy of FRP-reinforced concrete structures was necessary to account for the above-mentioned characteristics and to consider the deformability of such structures (ISIS Canada 2007 and ACI 440.1R 2015).

Strength and stiffness of FRP bars depend on many factors. The most effective factors are fiber volume fraction and the bar manufacturing process. In addition, the tensile strength is function of FRP bar diameter. Due to shear lag, fibers located near the center of cross section are not subject to the same stress as those fibers oriented near the outer surface of FRP bar. This phenomenon results in reduced strength and efficiency in larger diameter bars. Thus, the tensile strength of a particular FRP bar should be obtained from the bar manufacturer. In general, the tensile strength of FRP bars is much higher than that of steel bars. On the other hand, FRP bars has lower modulus of elasticity than that of steel bars.

The shear resistance of FRP bars is affected by the orientation of the fibers in the off-axis direction across the layers of fibers depending on the degree of offset of layers. That is why most of FRP bars are relatively weak in inter-laminar shear where layers of unreinforced resin lie between layers of fibers (ACI Committee 440 2008). As well as, the dowel resistance of FRP bars for shear is low because of their low transverse modulus and strength.

The bond properties of reinforcement have significant effect on design of members as the design tensile strength, deflection control, crack width calculations, and development length estimations are all dependent on the bond strength. In RC members the tension forces transfer from the concrete to the reinforcement through the bond between concrete and bars. Bond between FRP reinforcement and concrete develops similar to that of steel reinforcement and depends on FRP

type, surface deformation, and the shape of the FRP bar. As mentioned previously, the surface of the rods can be spiral, straight, sanded-straight, sanded-braided, and deformed so that the bond of these bars with concrete is equal to, or better than, the bond of steel bars (ISIS Canada 2007; ACI 440.1R 2015).

There are four types of FRP reinforcing bars according to the type of fiber used in the manufacturing process: glass (G) FRP, carbon (C) FRP, aramid (A) FRP and basalt (B) FRP. GFRP is the most common used type of FRPs, due to its low cost, high tensile strength, high chemical resistance, great impregnation in the resin, low thermal conductivity, high impact resistance and excellent insulating properties. Notwithstanding, the disadvantages are low stiffness, low fatigue resistance and high density. GFRP bars are available in different sizes, ranging from 6 mm in diameter to 32 mm; i.e. from No.6 to No.32 bars. GFRP bars have a sand-coated or ripped external layer, to create rough surface. The tensile strength of GFRP bars range between 483 – 1,600 MPa. While the modulus of elasticity is 35–65 GPa, and the ultimate strain is in the range of 1.2 – 3.1 %.

2.6.2 Simply-supported FRP-RC deep beams.

Omeman et al. (2008) investigated the behaviour of the FRP-RC deep beams. The variables were the shear span-to-depth ratio, effective depth, reinforcement ratio, and concrete compressive strength. Eight specimens reinforced with CFRP and four steel-RC specimens were tested. The test results were compared in terms of crack patterns, failure modes, load-strain relationship, shear strength, and deflection. It was concluded that using CFRP as a tie reinforcement greatly enhanced the shear strength and increased the deflection of such beams, due to its relatively low modulus of elasticity. The higher tensile strength of CFRP with respect to steel, resulted in comparable or

higher shear strength, by enhancing the arch action through a more effective tensile tie. Moreover, the lower modulus of elasticity of CFRP, resulted in wider cracks whereas it was similar in terms of crack orientation, propagation, and length. A significant increase in the shear strength of the CFRP-RC specimens occurred with increasing the effective depth, reinforcement ratio, compressive strength, and decreasing the shear span-to-depth ratio. Modifying the effective compressive strength of concrete struts was recommended based on the experimental results, to take into account the differences between the mechanical properties of steel and CFRP bars that are expected to affect the cracking of concrete struts.

El-Sayed et al. (2012) carried out an experimental investigation to study the behaviour of concrete deep beams reinforced with GFRP or CFRP bars. Ten full-scale specimens were tested to failure. All specimens had neither compression reinforcement nor shear reinforcement. The main variables were the longitudinal reinforcement ratio (ρ), shear span-to-depth ratio (a/d) and the modulus of elasticity of the longitudinal reinforcing bars. All specimens showed a bilinear load-deflection relationship. Prior to cracking, all the beams showed approximately same load deflection plot. After cracking, it was observed that the flexural stiffness increases with increasing ρ for constant a/d . Also, specimens reinforced with CFRP bars showed flexural stiffness greater than that of specimens reinforced with GFRP bars due to the higher modulus of elasticity of the CFRP bars compared to that of GFRP bars. On the other hand, for specimens with same ρ , significant increase in load capacity was observed with decreasing a/d . One of the specimens exhibited a flexural failure by crushing of concrete between the two loading points which proved that arch mechanism had developed in this beam. It was also observed that the strains in the reinforcing bars and concrete decreased with increasing the ρ . Moreover, decreasing the shear span-to-depth ratio decreased the strains in both bars and concrete measured at the same load level. However, the

strain in the longitudinal reinforcement was uniformly distributed along the bar length which proves the formation of arch action.

Andermatt and Lubell (2013) carried out a study to investigate the shear behaviour of deep beams reinforced with GFRP without web reinforcement. The experimental program included testing of twelve specimens with varied depth, shear span-to-depth ratio, longitudinal reinforcement ratio, and concrete strength. The reinforcement ratio of the specimen where chosen such that, at service limit state, the stresses in the reinforcement do not exceed 25% of the specified tensile strength of the GFRP bars. It was noted that crack patterns in the specimens indicated the formation of the arch mechanism. Inclined cracks developed, between the supports and loading points, which showed the change in internal forces flow from beam action to arch action, similar to the behaviour of steel-RC deep beams. It was also observed that, before cracking, the flexural stiffness of all beams having the same depth is similar. On the other hand, after cracking, the stiffness of specimens decreased with increasing a/d . Also, with increasing the reinforcement ratio, the stiffness and the capacity of the specimens increased. Moreover, it was reported that as the a/d decreased, the normalized shear capacity increased significantly similar to that in conventional beams reinforced with steel. Also, increasing the longitudinal reinforcement ratio ρ increased the shear capacity while increasing the concrete compressive strength (f_c') decreased the normalized shear capacity regardless a/d , ρ , or h . With increasing a/d , the effect of the concrete compressive stress diminished.

Farghaly and Benmokrane (2013) investigated the behaviour of four full-scale deep beams reinforced with GFRP and CFRP bars. The investigated parameters were the longitudinal reinforcement type and ratio and concrete compressive strength. It was observed that the longitudinal reinforcement has a significant effect on the width of the diagonal crack. Increasing

the reinforcement ratio by 80% decreased the diagonal crack width by 43% and 51% for GFRP and CFRP-RC beams. Also, increasing the reinforcement ratio increased the ultimate capacity even without the existence of any web reinforcement. In specimens with the same axial stiffness, increasing the compressive strength of the concrete by 27% increased the cracking and ultimate loads by 15% and 21%, respectively. The axial stiffness of the reinforcement had no effect on the stiffness of the specimens. In addition, it was reported that the STM was formed which is supported by the uniform strains along the longitudinal reinforcement.

Mohamed et al. (2017) carried out a study on ten full-scale concrete deep beams reinforced with GFRP bars. The main variables were the configuration of web reinforcement and shear span-to-depth ratio ($a/d = 1.47, 1.13, \text{ and } 0.83$). It was observed that the number and width of shear cracks increased with increasing a/d ratio. Also, for beams without web reinforcement, there was only one diagonal crack while beams with web reinforcement had parallel cracks adjacent to the first main diagonal crack. A horizontal strut was formed due to the use of two loading point. The use of either vertical or horizontal web reinforcement only has no effect on the first flexural cracking load, the first shear cracking load, and the main diagonal cracking load for each group of specimens with the same a/d . On the other hand, the use of both horizontal and vertical web reinforcement significantly increased the main diagonal cracking load. The use of horizontal web reinforcement only decreased the ultimate load capacity and this effect diminished with adding vertical web reinforcement. The decrease of ultimate load capacity in case of using horizontal web reinforced only was attributed to degradation of the compressive strength in the concrete strut due to high tensile strain in the horizontal bars affecting the strength and stiffness of the concrete diagonal strut. These strains generated radial tensile forces around the bars which led to decrease of the compressive strength of the strut. The vertical-only web reinforcement provided more significant

crack-width control than the horizontal-only web reinforcement. The presence of web reinforcement horizontal or vertical significantly decreased the crack width at ultimate load compared to that in beams without web reinforcement with the same a/d .

2.7 Code Provision for RC Deep Beams

Over many decades, new approaches of shear design of concrete structures were developed in many codes. The current CSA/S806-12 (CSA 2012), CSA/S6-14 (CSA 2014a), CSA/A23.3-14 (CSA 2014b) and ACI 318-14 (ACI Committee 318 2014) codes use the same approach of design; however, with different formulae. This section summarizes the design procedures for deep beams by the use of STM implemented in each code provision.

2.7.1 Code provisions for steel-RC deep beams

2.7.1.1 The American Code ACI 318-14

According to ACI 318-14 (ACI Committee 318 2014), deep beams are structural members loaded on one face and supported on the opposite face so that the compression strut can be developed between the load and support. This strut will develop under two conditions:

- 1) Clear span does not exceed four times the overall member depth (h).
- 2) Concentrated loads exist within a distance $2h$ from the face of the support.

In addition, the ACI 318-14 recommends that deep beams should be designed using either nonlinear analysis or using the STM.

According to ACI 318 (2014), the internal forces in strut and tie should be in equilibrium with the loads and reactions at supports. Also, the angle between strut and tie cannot be less than 25 degrees.

The compressive strength of strut is calculated by Equations [2.1] and [2.2].

Strut without longitudinal reinforcement

$$F_{ns} = f_{ce} A_{cs} \quad [2.1]$$

Strut with longitudinal reinforcement

$$F_{ns} = f_{ce} A_{cs} + A_s' f_s' \quad [2.2]$$

Where,

F_{ns} is calculated at each end of the strut and taken as the lesser value,

A_{cs} is the cross-sectional area at the end of the strut,

A_s' is the area of compression reinforcement along the length of the strut,

f_s' is the stress in the compression reinforcement at the nominal axial strength of the strut and it should be equal to f_y

f_{ce} is the effective concrete compressive strength in a strut and it is calculated by Equations [2.3].

$$f_{ce} = 0.85 \beta_s f_c' \quad [2.3]$$

where β_s is the strut efficiency factor defined in Table 2.1.

Table 2.1: Efficiency factors for ACI 318 (2014) Strut

Strut efficiencies ($0.85 f_c'$)	β_s
Strut with uniform cross section over its length	1.00
Bottle-shaped struts with reinforcement	0.75
Bottle-shaped struts without reinforcement	0.60
Struts in tension members	0.40
All other cases	0.60

According to ACI 318 (2014), web reinforcement in deep beams has a great effect in confining the strut and it increases the value of f_{ce} which, in turn, increases the value of F_{ns} . In bottle shaped struts, β_s increase from 0.6 to 0.75 in case of using web reinforcement which helps in resisting the transverse tension forces of struts resulting from spreading of the compressive force in the strut shall cross the strut axis. The web reinforcement is calculated using Equation [2.4].

$$\sum \frac{A_{si}}{b_s s_i} \sin \alpha_i \geq 0.003 \quad [2.4]$$

where A_{si} is the total area of distributed reinforcement at spacing s_i in the i -th direction of reinforcement crossing a strut at an angle α_i to the axis of a strut, and b_s is the width of the strut as shown in Figure 2.10.

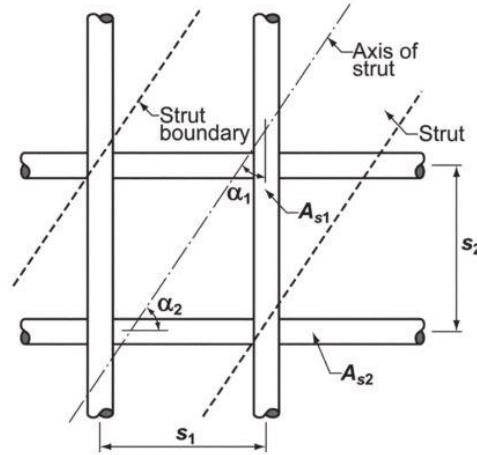


Figure 2.10: Reinforcement crossing a strut (ACI 318 (2014)).

According to ACI 318 (2014), the tie reinforcement can be non-prestressed or prestressed and the strength of ties F_{nt} is calculated by Equation. [2.5].

$$F_{nt} = A_{ts} f_y + A_{tp} (f_{se} + \Delta f_p) \quad [2.5]$$

where $(f_{se} + \Delta f_p)$ shall not exceed f_{py} , and A_{tp} is zero for non-prestressed members.

Also, the centroid of the tie reinforcement should be corresponding to the assumed tie in the STM. The reinforcement of the tie should be anchored by mechanical devices, post-tensioning anchorage devices, standard hooks, or straight bar development.

According to ACI 318 (2014) the strength of nodes F_{nn} in STM is calculated by Equation. [2.6].

$$F_{nn} = f_{ce} A_{nz} \quad [2.6]$$

where f_{ce} is effective compressive strength of the concrete and calculated by Equation [2.7], A_{nz} is the area of nodal zone and it's taken the smaller value:

- a) Area of the face of the nodal zone perpendicular to the line of action of F_{us} .

- b) Area of a section through the nodal zone perpendicular to the line of action of the resultant force on the section.

$$f_{ce} = 0.85\beta_n f'_c \quad [2.7]$$

where β_n is the node efficiency factor defined in Table 2.2.

Table 2.2: Efficiency factors for ACI 318 (2014) Nodes

Nodes efficiencies ($0.85 f'_c$)	β_n
Nodes bounded by compression or bearing CCC node	1.00
Nodes anchoring one tie CCT node	0.80
Nodes anchoring more than one tie CTT and TTT nodes	0.60

ACI 318 (2014) stated that the dimensions of deep beams shall be selected according to Equation [2.8].

$$V_u \leq \phi 10 \sqrt{f'_c} b_w d \quad [2.8]$$

Also, it was recommended that the area of web reinforcement in transverse direction and longitudinal direction shall be at least equal to $0.0025b_ws$, where s is spacing of the distributed reinforcement.

2.7.1.2 The Canadian Standards CSA/A23.3

The Canadian standards CSA/A23.3-14 (CSA 2014b) defines the deep beam as a flexural member with shear span-to-depth ratio less than two. In members with non-linear distribution of strain, it

is more preferable to be designed by STM. The STM is defined as a method of investigating strength of reinforced concrete members by idealizing the member as a series of reinforcing steel tensile ties and concrete compressive struts connected at nodes to form a truss able to transfer loads to supports directly.

According to CSA/A23.3-14 (2014b), the dimension of strut should be large enough so the calculated compression stresses in the strut does not exceed the allowable stress $\phi_c f_{cu}$ where ϕ_c is the factored concrete strength and f_{cu} is calculated by the following Equations

$$f_{cu} = \frac{f_c'}{0.8 + 170\varepsilon_1} \quad [2.9]$$

where,

$$\varepsilon_1 = \varepsilon_s + (\varepsilon_s + 0.002) \cot \theta_s^2 \quad [2.10]$$

θ_s is the angle of inclination between strut and tie.

ε_s is the tensile strain in tie.

When the specific yield strength of reinforcement is less than 400 MPa, Equation [2.11] can be used.

$$f_{cu} = \frac{f_c'}{1.14 + 0.68 \cot \theta_s^2} \quad [2.11]$$

According to CSA/A23.3-14 (2014b), the area of reinforcement in the tie should be large so that the tensile stresses in the tie do not exceed the allowable stresses $\phi_s f_y A_{st} + \phi_p(f_{po} + 400) A_p$. Also, the tie reinforcement shall be anchored to the nodal zones by straight bars, standard hooks or by headed bars.

According to CSA/A23.3-14 (2014b), the compressive strength in node region shall not exceed any of the following:

- a) $0.85 \phi_c m f'_c$ in node regions bounded by struts and bearing areas.
- b) $0.75 \phi_c m f'_c$ in node regions anchoring a tie in only one direction.
- c) $0.65 \phi_c m f'_c$ in node regions anchoring ties in more than one direction.

where m is the confinement modification factor taken as $\sqrt{A_2/A_1}$ but not more than 2.0. However, the stresses in node region are considered satisfying if

- a) The bearing stress on the node regions produced by concentrated loads or reactions does not exceed the stress limits.
- b) The tie reinforcement is uniformly distributed over an effective area of concrete at least equal to the tie force divided by the stress limits specified.

The CSA/A23.3-14 (CSA 2014b) mentioned that deep structural members and slabs need crack control reinforcement which is an orthogonal grid of reinforcement near to each face of the member. The ratio of this reinforcement to concrete gross area should be more than 0.002 in each direction with a minimum spacing of 300 mm. In deep beams, the crack control reinforcement can be also used as a tie reinforcement.

2.7.2 Code provisions for FRP reinforced concrete deep beams

The CSA/S806-12 (CSA 2012) includes provisions for design of deep beams using STM. These provisions represent a modified version of those found in the CSA/A23.3-04 (CSA/A23.3 2004) code for steel-RC deep beams. However, the allowable tensile strength of tie is reduced to $0.65 \phi_f$

$A_f f_{fu}$. Moreover, according to CSA/S806-12 (CSA 2012), the minimum ratio of the orthogonal reinforcement is 0.004 for GFRP and AFRP, and 0.003 for CFRP with minimum spacing of 200 mm for GFRP and AFRP, and 300 mm for CFRP.

On the other hand, ACI 440.1R-15 (2015) guidelines do not provide design provisions for deep beams reinforced with FRP bars.

CHAPTER 3: EXPERIMENTAL PROGRAM

3.1 Introduction

Based on the literature presented in the previous chapter, it is evident that the behaviour of RC deep beams is affected by the shear span-to-depth ratio, beam depth, web reinforcement, concrete compressive strength, and longitudinal reinforcement ratio. As the behaviour of the beam changes from beam-action to tied-arch action when the shear span-to-depth ratio is less than 2.0 (ACI 318 2014; CSA/S6 2006; Wight and MacGregor 2009), the effect of the shear span-to-depth ratio and the longitudinal reinforcement ratio in both simply-supported and continuous GFRP-RC deep beams is investigated. Also, in the Strut-and-tie method, the longitudinal reinforcement works as a tie and the strain level in the reinforcement affects the efficiency of the strut; therefore, the effect of the tie reinforcement ratio/arrangement on the shear capacity of such beams is investigated. Moreover, the redistribution of the internal forces in the test beams is evaluated.

The behaviour of GFRP-RC simply-supported and continuous deep beams is investigated considering the following parameters:

- 1- Shear span-to-depth ratio (1, 1.5 and 2) in both simply-supported and continuous deep RC beams.
- 2- Longitudinal reinforcement ratio at the hogging moment region of 0.8%, 1.0%, and 1.2% while keeping the reinforcement ratio at the sagging moment region constant at 1.0%.

3.2 Test Beams and Design Concept

In this study, a total of twelve large-scale concrete deep beams reinforced longitudinally with GFRP bars were constructed and tested to failure. All test specimens had a rectangular cross section with a width of 250 mm and a height of 590 mm. The length of the simply-supported beams was 2,100, 2,600, and 3,100 mm, while the length of the continuous beams was 3,500, 4,500, and 5,500 mm for beams with shear span-to-depth ratio of 1.0, 1.5, and 2.0, respectively. None of the test specimens had web reinforcement. The test specimens were divided into 4 series, one for simply-supported beams and 3 for continuous beams. In all test specimens, the bottom reinforcement ratio was kept constant at 1.0%. Series I had three simply-supported RC deep beams with a bottom longitudinal reinforcement ratio (ρ_{bot}) of 1.0%, and a clear shear span-to-depth ratio of 1.0, 1.5 and 2.0. The continuous RC deep beams were divided into three series depending on the clear shear span-to-depth ratio (Series II beams had $a/d = 1.0$, Series III beams had $a/d = 1.5$, and Series IV had $a/d = 2.0$). In each series, the variable was the top longitudinal reinforcement ratio which was 0.8, 1.0 and 1.2%. It should be noted that the top reinforcement ratio of 1.2% was selected to satisfy the requirement of the elastic internal forces, then reduced ratios of 1.0 and 0.8% were used to evaluate the capacity of the beams at such ratios. The bottom longitudinal reinforcement was kept constant at 1.0%.

The beams were designed according to the Canadian standard CSA/S806-12 (CSA 2012). The STM for the continuous deep beams was selected based on a suggested model by Yang et al. (2007) for the continuous deep beams as shown in Figure 3.1. This STM assumes the formation of a tie in the top and bottom longitudinal reinforcement. In addition, the load is transferred to the support through struts in exterior and interior shear spans. Marti (1985) noted that the width of the strut is in proportion to the stress on the node face to make the stresses in the node region constant. Thus,

the width of the loading plate is divided according to the ratio of the exterior reaction to the applied load (β) as shown in Figure 3.1. The ratio between the exterior reaction-to-applied load was determined using 2-D finite element (FE) analysis assuming a unit force acting at the loading points. The β values of the test beams were 0.344, 0.332, and 0.326 when a/d ratios were 1.0, 1.5, and 2.0, respectively. The design procedures were based on the recommendations of CSA/S806-12 as discussed in the code provisions section. Detailed design calculations of all test beams can be found in Appendix A.

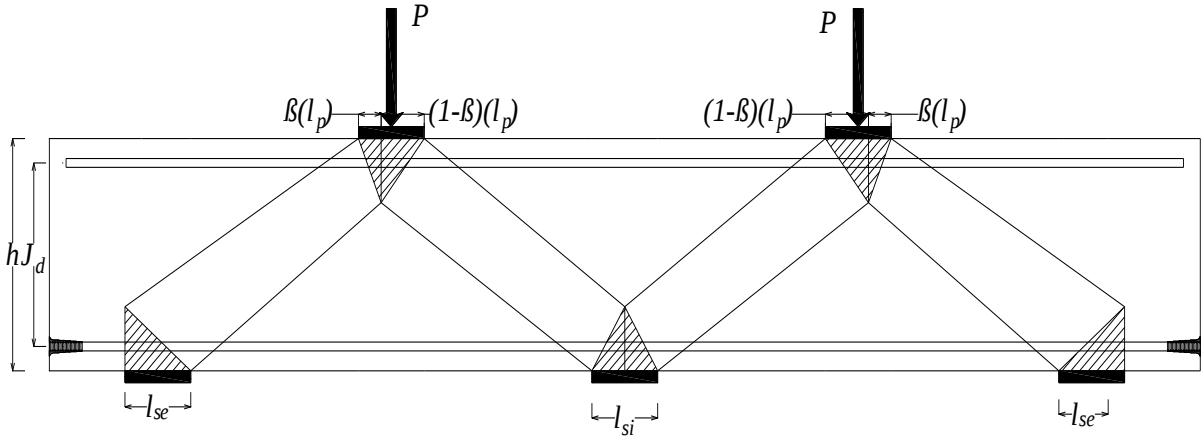


Figure 3.1: Proposed strut-and-tie Model

The test beams are labeled based on the parameters included in this study. The name of each beam can be explained as follows. The first letter denotes the beam type (“S” for simply-supported beams and “C” for Continuous beams). The second character is a number that indicates the clear shear span-to-depth ratio of the beam (“1.0, 1.5, and 2.0” for beams with $a/d = 1.0, 1.5$ and 2.0 , respectively). The third character is a number that indicates the ratio of the top-to-bottom longitudinal reinforcement ratio (“0.8, 1.0 and 1.2” for $\rho_{top} / \rho_{bot} = 0.8, 1.0$ and 1.2 , respectively). For example, specimen “C-1.0-1.2” is a continuous deep beam that has a clear shear span-to-depth

ratio of 1.0 and the top-to-bottom longitudinal reinforcement ratio is 1.2. The dimensions, geometry and reinforcement of the test specimens are shown in Figure 3.2 and Tables 3.1 and 3.2.

Table 3.1: Details of test specimens.

Series No.	Beam ID	Depth (mm)	a/d	Bottom Tie		Top Tie		f_c' (MPa)
				ρ_b (%)	Bars	ρ_t (%)	Bars	
Series I	S-1.0-0.0	521.3	1.0	1.0	3#20+2#15	—	—	43
	S-1.5-0.0	521.3	1.5	1.0	3#20+2#15	—	—	46
	S-2.0-0.0	521.3	2.0	1.0	3#20+2#15	—	—	46
Series II	C-1.0-1.2	521.3	1.0	1.0	3#20+2#15	1.2	3#20+3#15	49
	C-1.0-1.0	521.3	1.0	1.0	3#20+2#15	1.0	3#20+2#15	48
	C-1.0-0.8	521.3	1.0	1.0	3#20+2#15	0.8	5#15	49
Series III	C-1.5-1.2	521.3	1.5	1.0	3#20+2#15	1.2	3#20+3#15	49
	C-1.5-1.0	521.3	1.5	1.0	3#20+2#15	1.0	3#20+2#15	49
	C-1.5-0.8	521.3	1.5	1.0	3#20+2#15	0.8	5#15	49
Series IV	C-2.0-1.2	521.3	2.0	1.0	3#20+2#15	1.2	3#20+3#15	47
	C-2.0-1.0	521.3	2.0	1.0	3#20+2#15	1.0	3#20+2#15	47
	C-2.0-0.8	521.3	2.0	1.0	3#20+2#15	0.8	5#15	48

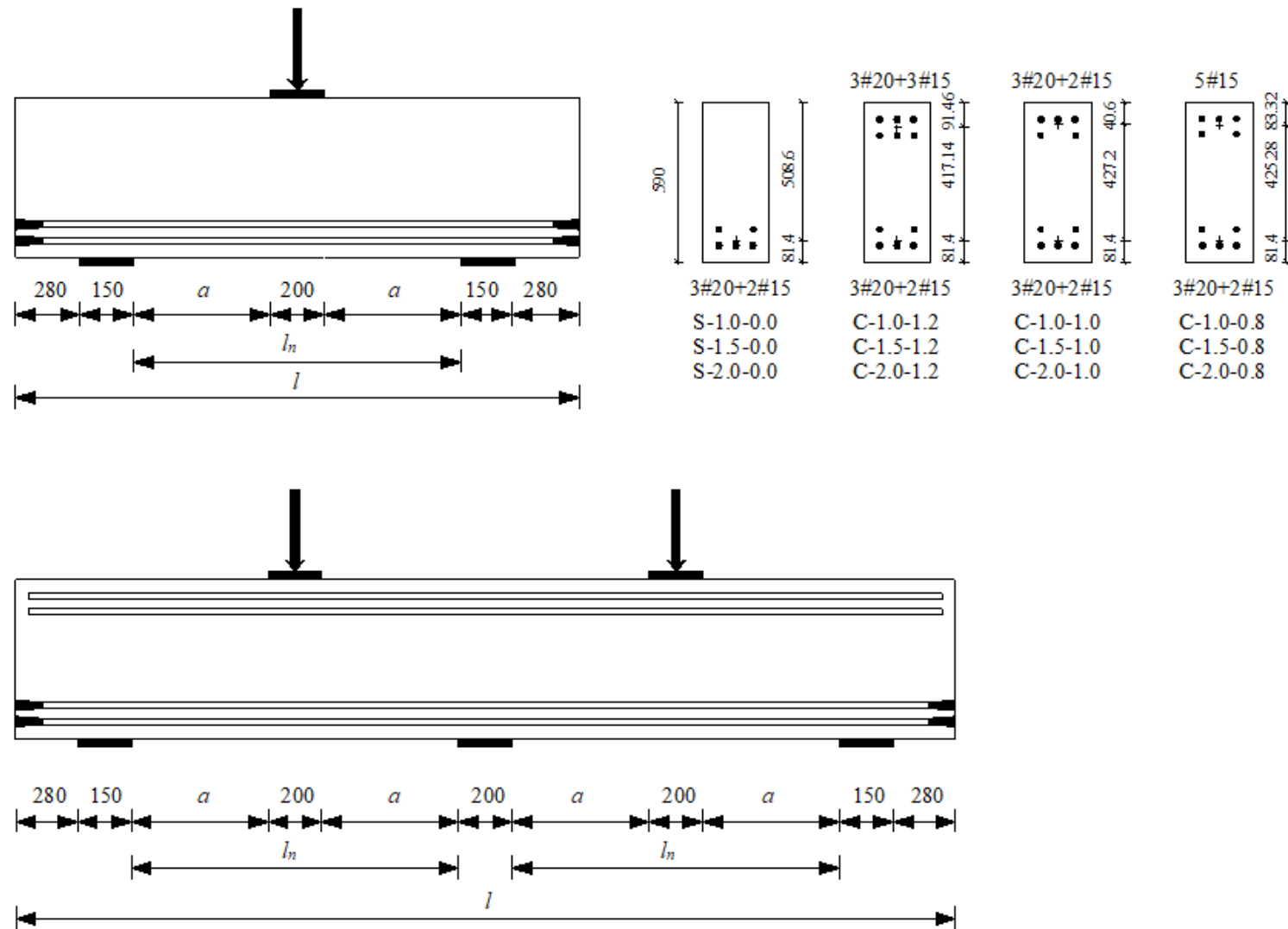


Figure 3.2: Details of testing specimens

Table 3.2: Dimensions of test specimens

Series No.	Beam ID	a^* (mm)	d (mm)	ℓ_n^{**} (mm)	ℓ (mm)
Series I	S-1.0-0.0	520	521.3	1,240	2,100
	S-1.5-0.0	770	521.3	1,740	2,600
	S-2.0-0.0	1020	521.3	2,240	3,100
Series II	C-1.0-1.2	520	521.3	1,240	3,500
	C-1.0-1.0	520	521.3	1,240	3,500
	C-1.0-0.8	520	521.3	1,240	3,500
Series II	C-1.5-1.2	770	521.3	1,740	4,500
	C-1.5-1.0	770	521.3	1,740	4,500
	C-1.5-0.8	770	521.3	1,740	4,500
Series IV	C-2.0-1.2	1,020	521.3	2,240	5,500
	C-2.0-1.0	1,020	521.3	2,240	5,500
	C-2.0-0.8	1,020	521.3	2,240	5,500

* a is the clear shear span

** ℓ_n is the clear span between the supports

3.3 Materials

To construct the specimens, both concrete and GFRP bars are used. All specimens are constructed in the Heavy Structures Laboratory at the University of Manitoba using ready-mix normal-weight concrete with a target 28-day strength of 35 MPa. The concrete had a nominal maximum aggregate size of 20 mm with a target slump of 100 to 120 mm. Ten standard cylinders 100×200 mm were cast from each batch and tested to determine the concrete compressive strength at 28 days and on the day of testing (provided in Table 3.1).

Sand-coated GFRP pultruded bars were used as main longitudinal reinforcement. Straight GFRP bars were used for the top reinforcement while headed-end GFRP bars were used for the bottom reinforcement. The properties of the used GFRP bars (the cross-sectional area, the tensile strength, modulus of elasticity and ultimate strain) provided by the manufacturer are listed in Table 3.3.

Table 3.3: Mechanical properties of GFRP bars

Bar size.	Diameter (mm)	Area (mm ²)		Tensile Strength (MPa)	Modulus of Elasticity (GPa)	Ultimate Strain (%)
		Nominal	CSA/S806-12 Annex A			
#15	15.9	198	291	1,184	62.6	1.89
#20	19.0	285	394	1,105	63.7	1.73

The headed ends are made of thermoplastic matrix reinforced with short glass fiber cast at the end of bars at high temperature. The bars inside the head is threaded to increase mechanical interlock with the head as shown in Figure 3.3a. The head is approximately 100 mm in length with a maximum outer diameter of 50 mm at the end (Figure 3.3b). Benmokrane et al. (2016) carried out a study to investigate the behaviour of headed-end bars. It was concluded that the headed-end improved the mechanical interlock to the bar surface which led to the development of higher tensile strength compared to straight end bars. Moreover, headed-end bars showed a 90% higher pull out capacity compared to straight bars.

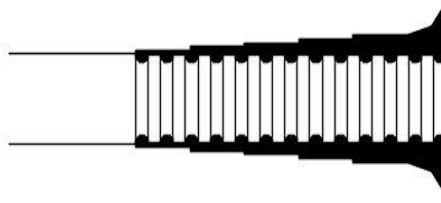


Figure 3.3: Details and overview of the bar-head (Benmokrane et al. 2015)



Figure 3.4: Photos of the used headed GFRP bars

3.4 Construction of Test Beams

The first step of construction was the installation of strain gauges at the specified locations on GFRP reinforcing bars. Then reinforcement cages were assembled including the instrumented bars. The following step was to build formworks for the different beam sizes using plywood sheets. Afterwards, the reinforcement cages were assembled and placed into the formwork, in preparation for concrete casting. To install the strain gauges on bars, abrading and cleaning were done to the bar surface; then, the strain gauges were attached to the bars at the assigned locations by special glue designed for this purpose. A thin layer of silicone coating was provided on the strain gauges to protect the strain gauges against moisture, impact or damage during casting. Before placing the reinforcement cages, the formwork was coated with a layer of oil to ease the beam removal after casting and curing of the concrete. Plastic chairs were used to maintain the effective depth of the

reinforcement and the concrete cover. In addition, while casting, the concrete was vibrated using an electrical vibrator. The surface of the concrete was finished after casting to a smooth surface and then covered with a plastic sheet. Moreover, twenty 100×150 -mm and six 150×300 -mm cylinders were cast in plastic molds over three layers while compacting each layer with the standard steel rod for 25 times. The curing process for the constructed beams and the concrete cylinders started the next day and the concrete surface was kept wet for seven days. Figure 3.5 shows some of the construction stages of the test beams.



a) Simply-supported beams



b) Continuous beams



c) Beams in curing

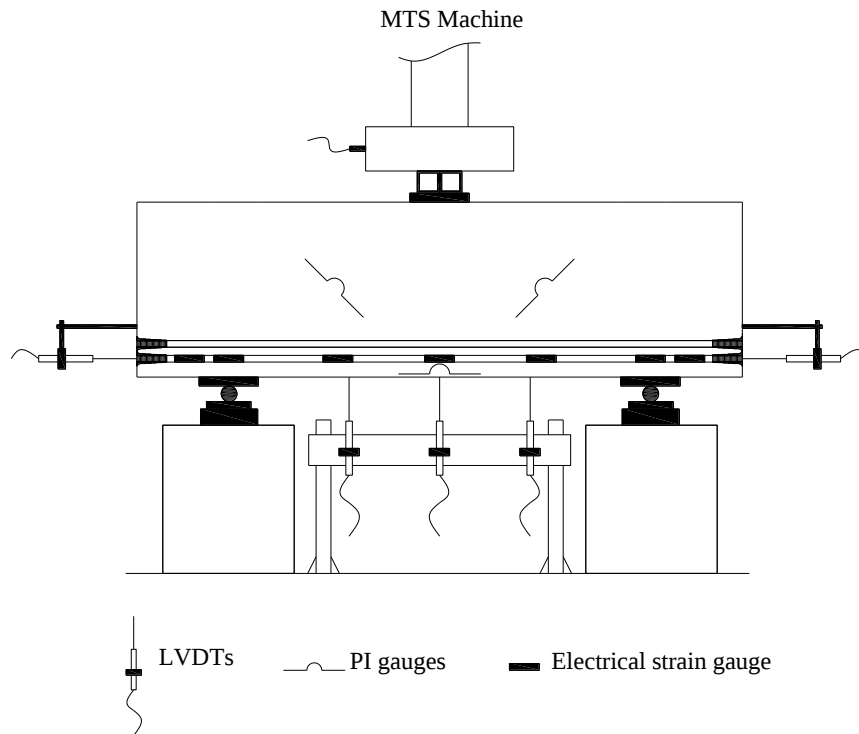
Figure 3.5: Construction stages of test specimens

3.5 Test Setup and Instrumentation

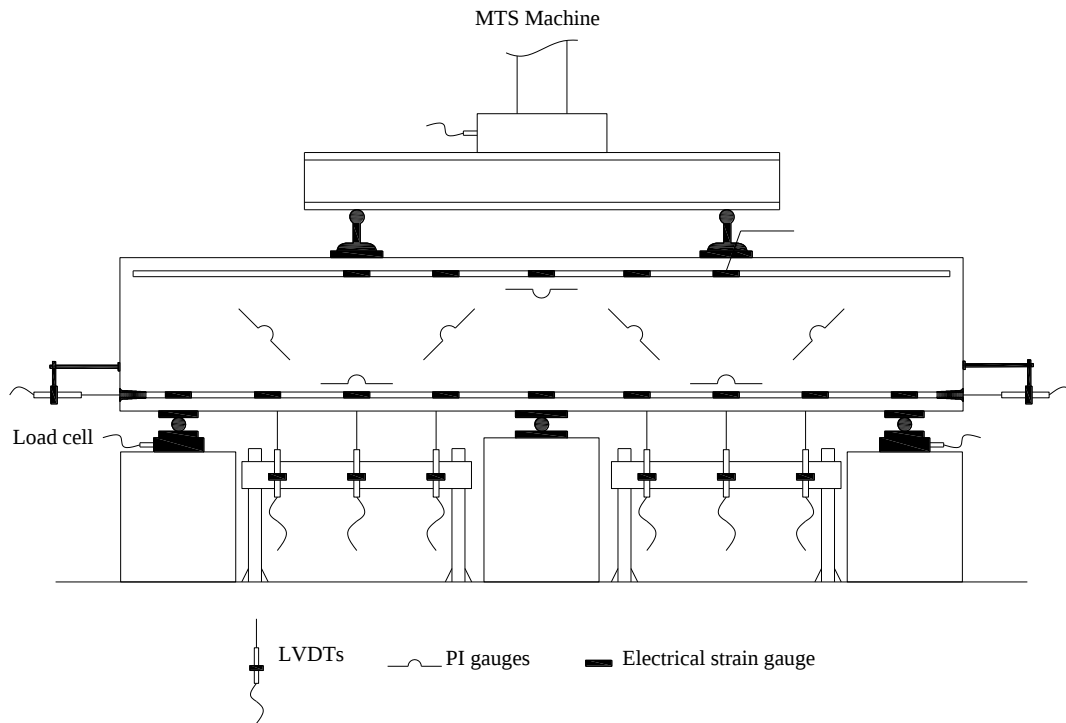
The simply-supported beams had one hinged support at one end and one roller support at the other end. However, the continuous beams had two equal spans and were supported on one hinged support at the middle and two roller supports at both ends. The beams were tested under one-point load in each span with varied shear span-to-depth ratio. A 5000-kN MTS machine was used to apply a monotonic concentrated load that was transmitted to the beam through a rigid steel spreader beam. In all tests, a load-controlled rate of 10 kN/min was used to ensure equal loads to the two spans.

Two load cells were used to measure the reactions at the end supports for the continuous deep beams. Also, deflection was measured, using linear variable displacement transducers (LVDTs), at three different locations in each span, at mid-span and at the mid-shear span points. In addition, high-accuracy LVDTs were used to monitor the slip, if any, at the end of one bar in the bottom reinforcement to evaluate the ability of the headed-end to ensure adequate anchorage. To measure

the strains in the reinforcement, seven strain gauges were installed on the longitudinal reinforcement of the simply-supported specimens. On the other hand, fourteen strain gauges were installed on the top and bottom longitudinal reinforcement of the continuous deep beams. The strain gauges were placed at the loading and supporting points and at the middle of each shear span, to measure the strains along the entire length of the specimens. In addition, 200-mm long PI gauges were attached to the concrete side surface to measure the width of the diagonal cracks in the shear spans between the loading and the supporting points and at the mid-span to monitor the flexural crack width. The applied load, reactions, displacements, crack widths, and strain gauges readings were electronically recorded during the test using a data acquisition system monitored by a computer. Figure 3.6 shows the test setup and instrumentations of test beams.



a) Simply-supported beams test set-up



b) Continuous beams

Figure 3.6: Test set-up and instrumentations.

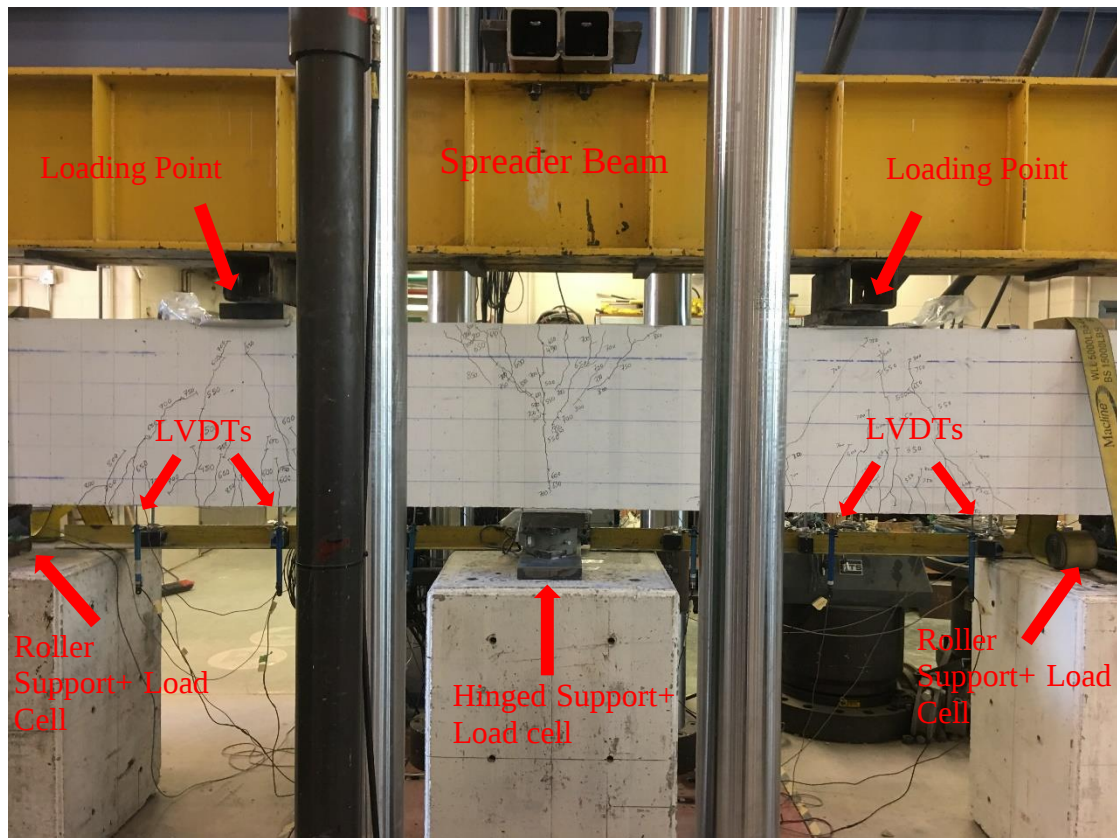


Figure 3.7: Beam in test set-up.

CHAPTER 4: RESULTS AND DISCUSSION OF SERIES I

4.1 General

This chapter presents the behaviour of simply-supported deep beams reinforced with GFRP bars. It also includes a comparison between simply-supported beams of Series I and the continuous beams with the same shear span-to-depth ratio having top reinforcement ratio of 0.8% from series II, III, and IV. All beams were tested under one-point loading arrangement in each span. The test variable is the shear span-to-depth ratio (1.0, 1.5 and 2.0.) in both simply-supported and continuous RC deep beams. The main objectives of this series were to evaluate the shear capacity of GFRP-RC continuous deep beams and to compare their behaviour to that of simply-supported deep beams. The behaviour of the test beams, in terms of cracking patterns, load-deflection curves, and strain variations in the reinforcement bars, is described in this chapter.

4.2 General Behaviour, Cracking and Mode of Failure

In simply-supported beams, flexural cracks were observed first at the mid-span section, at early loading stage (18-33% of ultimate load). The cracks propagated vertically to a distance of about 90% of the beam height ($0.9h$). With further loading, flexural-shear cracks started to propagate in the shear span inclined towards the loading point and parallel to the compression strut. For the continuous specimens, flexural cracks propagated first over the middle support followed by similar cracks in the sagging moment region. As loading progressed, flexural-shear cracks developed in the interior shear span at the vicinity of the middle support, then more cracks formed in the interior and exterior shear spans. A diagonal shear crack between the loading and supporting points parallel to the compression strut formed at a load of approximately 65-88 % of the ultimate load. With further increase in the load, the width of the diagonal crack

increased until failure of specimen. Generally, the behaviour of the simply-supported deep beams was very similar to the behaviour of the continuous deep beams, in terms of the cracking pattern. However, the continuous deep beams had developed lower number of cracks in the mid-spans compared to simply-supported deep beams. In addition, as the shear span-to-depth ratio increased, more cracks formed over the middle support in continuous beams. Cracks pattern for the specimens is provided in Figure 4.1.

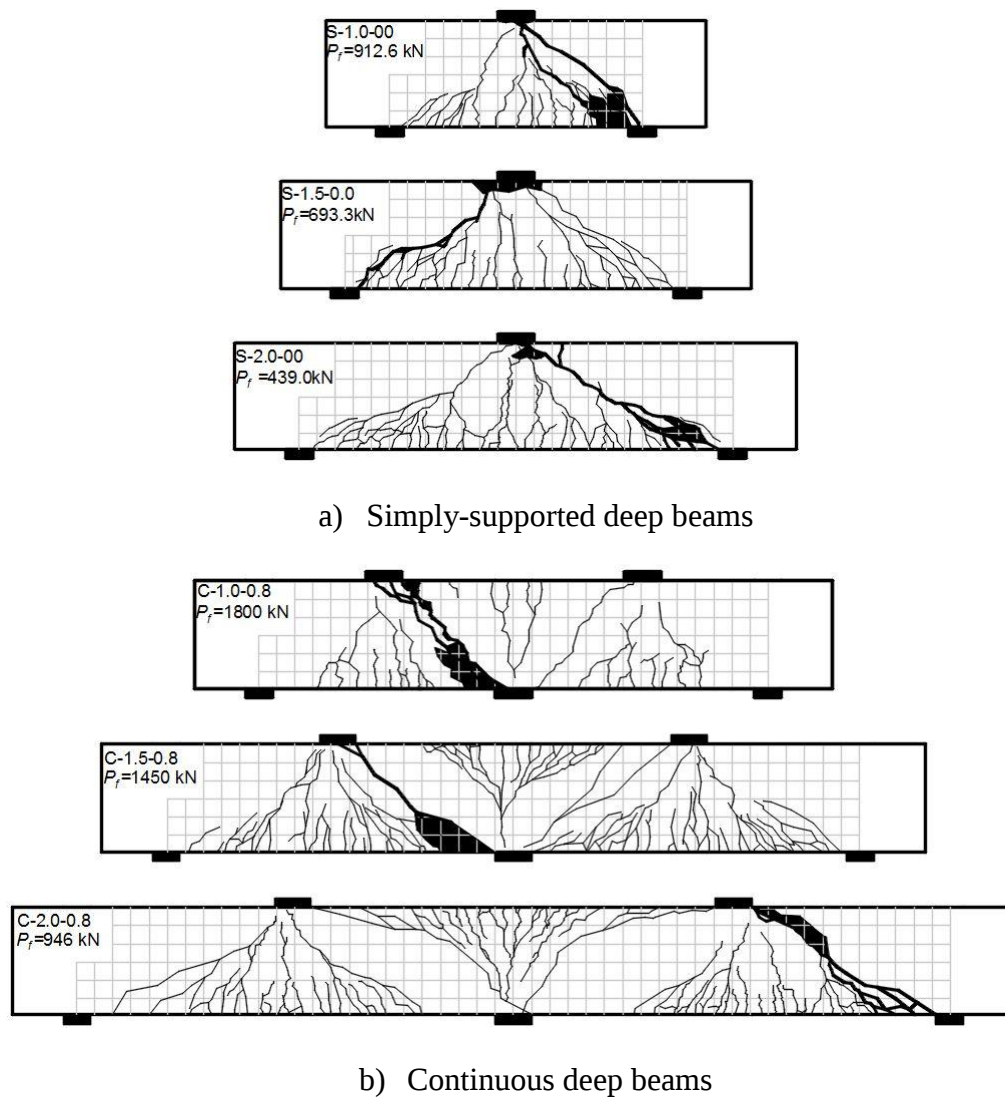


Figure 4.1: Crack pattern at failure of test beams.

Three types of failure were noted in this study. Failure of diagonal compression strut in the region between the loading and supporting plates occurred in specimens S-1.0-0.0, and C-1.0-0.8. This type of failure occurred in a noisy, brittle manner. Shear compression failure occurred in specimens S-1.5-0.0 and C-1.5-0.8 where failure was characterized by crushing of concrete at the tip of the diagonal crack in the flexural compression zone. The diagonal crack extended between the internal edges of the loading and supporting plates. A concrete diagonal tension failure occurred in specimens S-2.0-0.0 and C-2.0-0.8. Figure 4.2 shows photos for the failure modes of all test specimens. It was noted that the shear failure became more brittle with decreasing a/d . In addition, no premature anchorage failure of the tension reinforcement or bearing failure at the supports was observed. Similar modes of failure were reported in a previous study by Andermatt and Lubell (2013) for specimens with a/d ranging between 1.07 and 2.08.

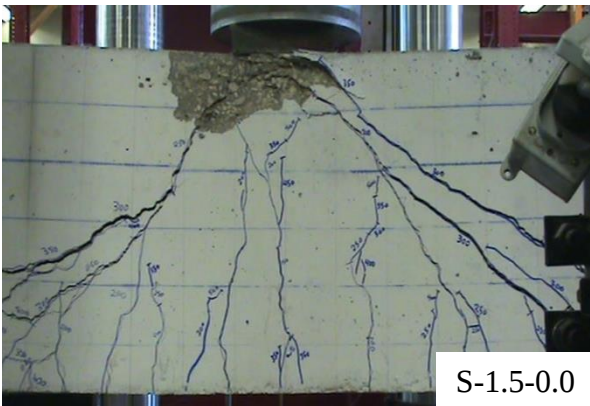
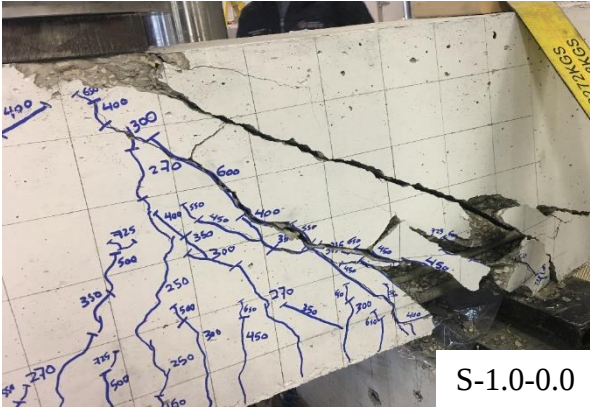
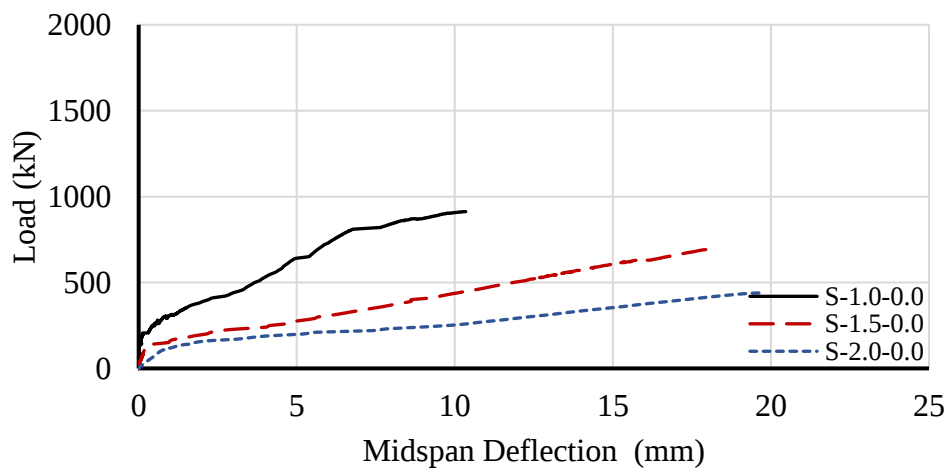


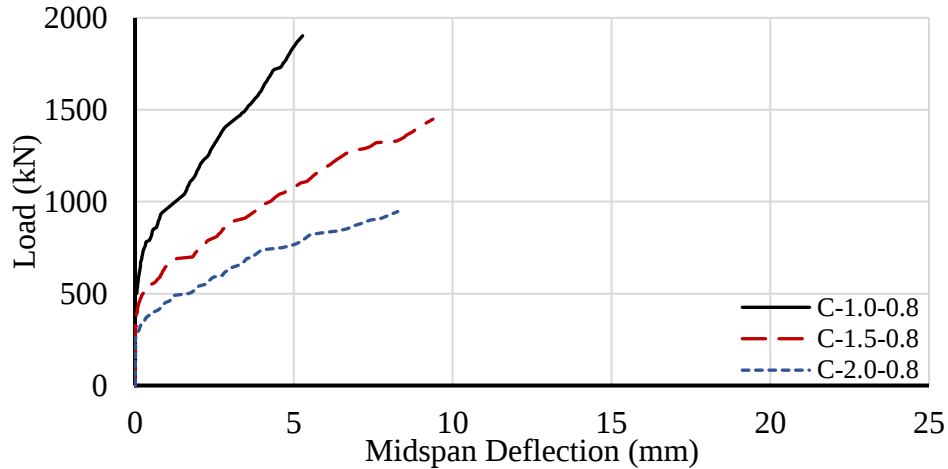
Figure 4.2: Modes of failure of test beams.

4.3 Load-Deflection Behaviour

The total applied load versus the deflection plot for all test specimens is shown in Figure 4.3. Generally, in all test specimens, the load-deflection relationship was approximately bilinear. The behaviour of the specimens were similar before the formation of the flexural cracks where the deflection was of very small values. After cracking, a reduction in the flexural stiffness of the specimens was observed but with different tendencies, which is attributed to the different a/d of each specimen. Decreasing a/d resulted in a reduction of the deflection exhibited by the beams. In other words, the influence of changing a/d on the load-deflection curve was that specimen with lowest $a/d = 1.0$ exhibited the steepest load-deflection curve followed by specimens with $a/d = 1.5$ and $a/d = 2.0$. This is true for both simply-supported and continuous beams; however, the continuous beams showed less deflection compared to their simply-supported beams counterparts. At failure, the maximum mid-span deflection of the continuous deep beams was 40-50% of the maximum mid-span deflection of the simply-supported counterparts, as shown in Table 4.1. This can be attributed to the continuity effect and that the sagging region of the continuous beams showed less cracks than simple beams.



a) Simply-supported deep beams



b) Continuous deep beams

Figure 4.3: Load-deflection relationship for test beams

4.4 Load Capacity

This study shows that decreasing a/d considerably affected the load capacity of deep beams reinforced with GFRP bars as shown in Figure 4.4. It was found that, increasing a/d from 1.0 to 1.5 resulted in decreasing the shear capacity by 24 and 23% in simply-supported and the continuous deep beams, respectively. Similarly, a reduction of 37 and 35% of the shear capacity was observed with increasing a/d from 1.5 to 2.0 for simply-supported and the continuous deep beams, respectively. The reduction in the load capacity resulted from decreasing the inclination angle of the strut which negatively affects the efficiency of the compression strut. However, insignificant difference was noticed between the capacity of the simply-supported beams compared to the load capacity of one span in the continuous counterpart beams. The load capacity of the continuous deep beam was higher than its simply-supported counterpart by 4, 5 and 8% in specimens with $a/d = 1.0$, 1.5, and 2.0, respectively.

Similar behaviour was reported in studies carried out to investigate the behaviour of simply-supported FRP-RC deep beams (El-Sayed et al. 2012, Andermatt and Lubell 2013). It was reported that increasing the shear span-to-depth ratio from 1.08 to 1.48 resulted in decreasing the load capacity by 37%. In addition, increasing the shear span-to-depth ratio from 1.48 to 2.08 decreased the load capacity by 46%.

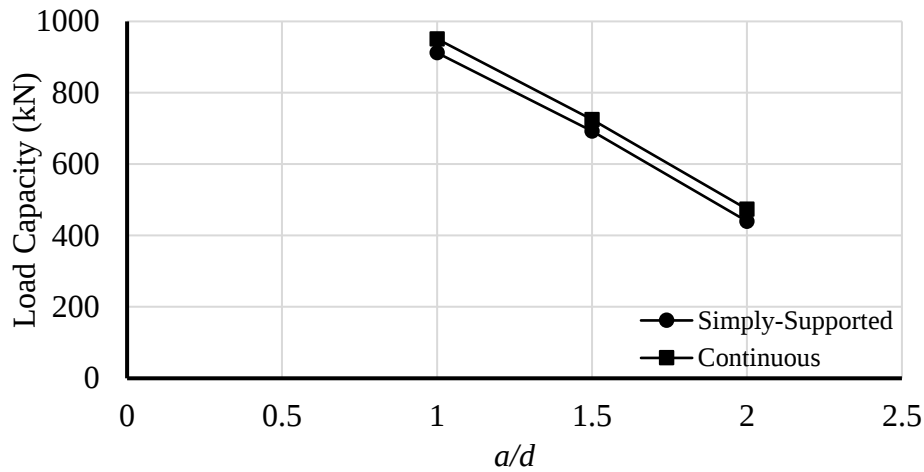


Figure 4.4: Load capacity versus shear span to depth ratio

4.5 Strain in Reinforcement and Strain Profiles.

Figures 4.5 and 4.6 show the applied load versus the measured mid-span strains in the bottom reinforcement of simply-supported and continuous beams as well the strain in the top reinforcement over the middle support in continuous beams. The load-strain plots for the test beams exhibited similar characteristics. For all specimens, after cracking, the strains varied linearly with increasing the load up to failure. The results showed that increasing a/d affected significantly the strains in the reinforcement in both simply-supported and continuous deep beams. Increasing a/d resulted in increasing the strains in the reinforcement at the same load level. At failure, increasing a/d from 1.0 to 1.5, resulted in decreasing the developed tensile

strains by 11%. Further increase in a/d to 2.0 led to a 21% reduction in the developed tensile strains in simply-supported specimens. However, in continuous beams, increasing a/d from 1.0 to 1.5, resulted in increasing the developed tensile strains by 33%. On the other hand, increasing a/d from 1.5 to 2.0 decreased the developed tensile strain by 18%. It was noticed that the simply-supported specimens developed higher tensile strains at failure compared to the continuous beams; however, simply-supported and continuous deep beams with the same a/d had a comparable load capacity. As shown in Table 4.1, the developed tensile strains in the bottom reinforcement in the continuous deep beams were 45-67 % of that of the strains developed in bottom reinforcement of their simply-supported counterpart beams at mid-span section. Also, it is worth mentioning that, in continuous beams, the maximum measured strain in the top reinforcement was 80, 88, and 110 % of the developed strains in the bottom reinforcement at the mid-span section, for specimens with shear span-to-depth ratio of 1.0, 1.5, and 2.0, respectively.

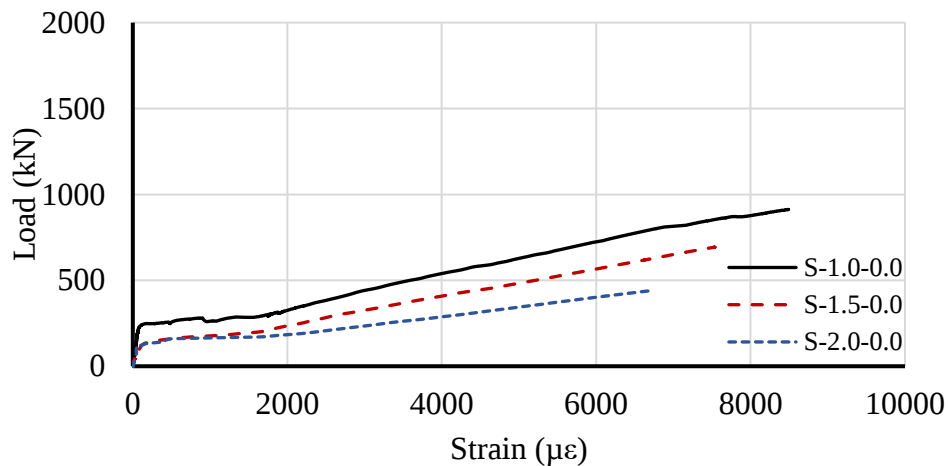
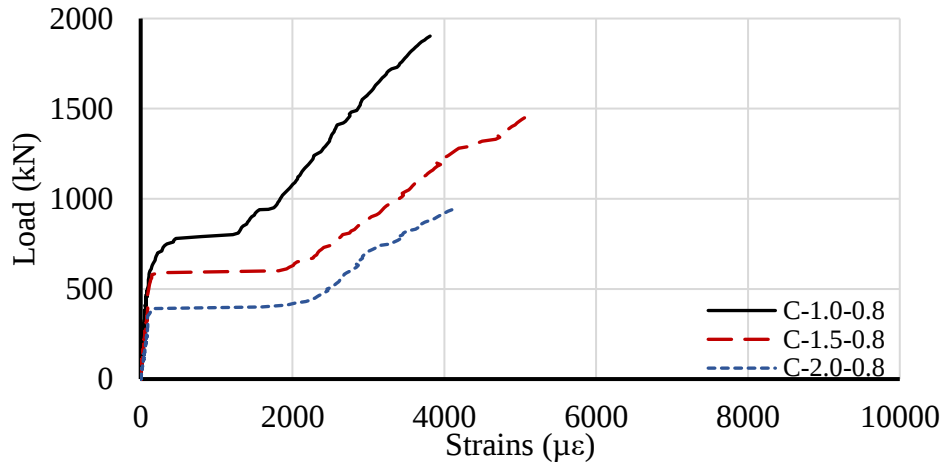
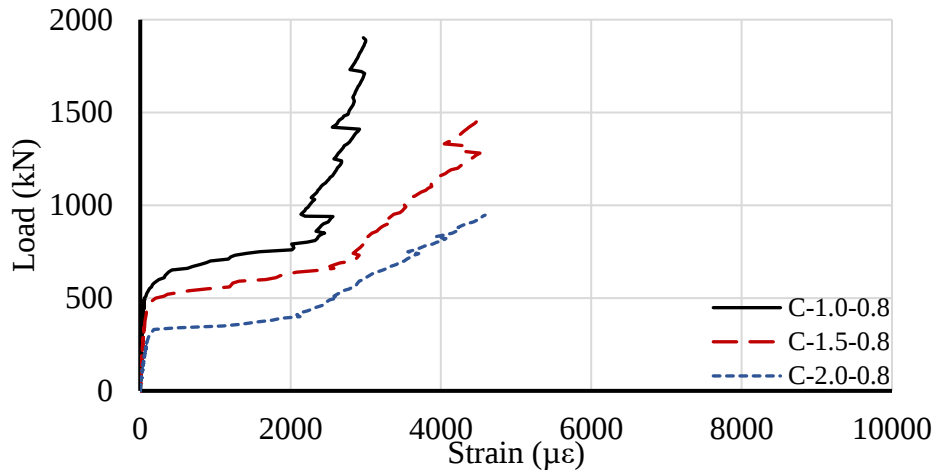


Figure 4.5: Strain at mid-span section in simply-supported deep beams



a) Strains in bottom reinforcement at mid-span section

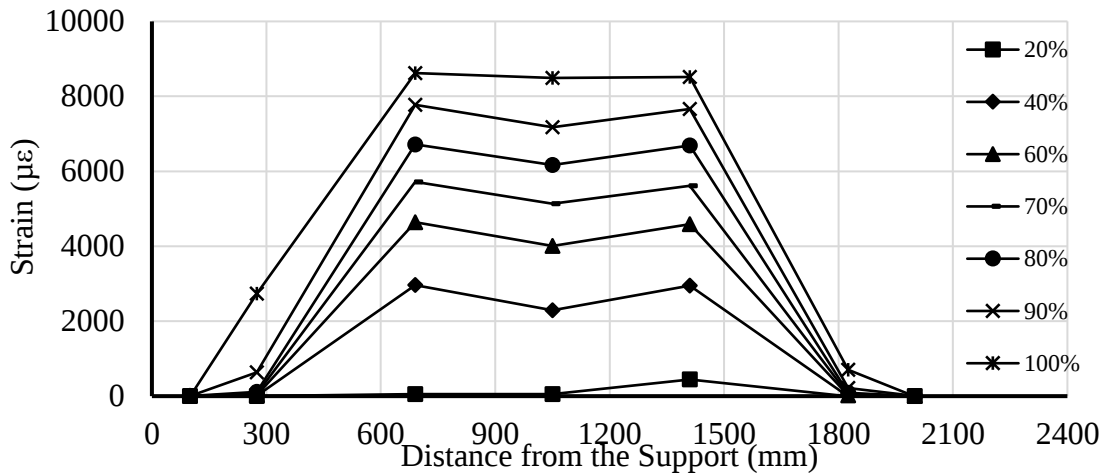


b) Strain in top reinforcement over intermediate support

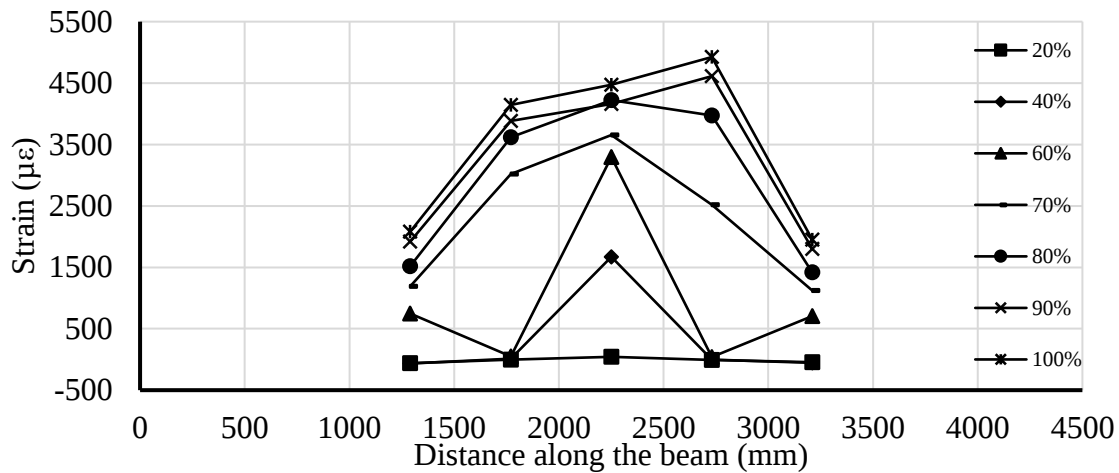
Figure 4.6: Developed strains in reinforcement in continuous beams

Figures 4.7 shows the strain profiles of selected specimens where the strains along the beams indicated the development of the STM in the test specimens. For all specimens, very low tensile strains developed in the reinforcement before the propagation of the first flexural crack. With further loading, a rapid increase in the development of the tensile strains at the flexural cracks resembling the bending moment diagram for both the simply-supported and continuous deep beams. As loading progressed, flexural-shear cracks propagated in the shear spans resulted in increasing the strains of the reinforcement near supports. A significant increase in the strains in

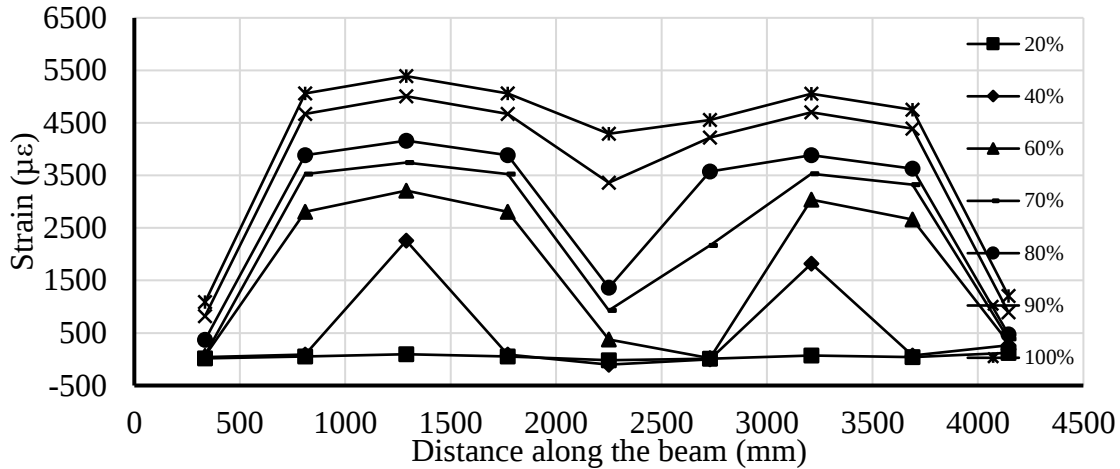
the bottom tie over the middle support occurred, after the development of the diagonal shear crack in the interior shear span in the continuous beams. It can be noted that the strains tend to be similar along the entire length between supports after the development of the tied arch action. The distribution of the reinforcement strain is an indicator of whether a tied arch action is developed in the specimen and to what extent.



a) S-1.5-0.0



b) Strain profiles for top reinforcement in C-1.5-0.8



c) Strain profiles for bottom reinforcement in C-1.5-0.8

Figure 4.7: Strain profiles of tested specimens

Table 4.1: Summary of test results

Specimen	Mode of Failure*	Failure Load (kN)	Reinforcement Strain at failure (micro-strain) @ mid-span	Mid-span Deflection at Failure (mm)
S-1.0-0.0	S	913	8,490	10.3
S-1.5-0.0	SC	693	7,530	18.1
S-2.0-0.0	DT	439	6,690	19.6
C-1.0-0.8	S	1902	3,810	5.3
C-1.5-0.8	SC	1449	5,060	9.4
C-2.0-0.8	DT	947	4,130	8.3

*DT is diagonal concrete tension failure; SC is shear compression failure; S is compression strut failure.

4.6 Headed-end Bars

The development of the STM in deep beams was confirmed by the development of high tensile strains over the supports and cracks propagation. These strains result from the tensile stresses

propagated in the tie between nodes at supports. Due to these tensile strains, a longer development length is required at supports than in the case of slender beams such that reinforcement can develop the stresses. In previous studies, the development length ranged from 400 to 1000 mm for simply-supported deep beams with height ranging from 400 to 1200 mm (El-Sayed et al. 2012; Farghaly and Benmokrane 2013; Mohamed et al. 2017). Andermatt and Lubell (2013) provided overhang length ranging from 400 mm to 800 mm for specimens with height of 600 mm to prevent the end anchorage failure, as it does not indicate the actual capacity of deep beams. In this study, the required development length of each specimen was calculated based on the Canadian standards (CSA/S806-12 2012), which ranged between 330 to 430 mm. It is worth mentioning that these development lengths were at the strain level developed in the tie at the predicted loads. However, only 280 mm was provided as a development length to study the efficiency of the headed-end bars in developing the required tensile stresses in a shorter overhang length. It is worth mentioning that no slippage in the bottom headed-end bars was observed. Therefore, the headed-end bars provided an effective anchorage method for deep beams considering the achieved loads and strains in such beams.

4.7 Code Comparison

In this section, the shear strength of the tested beams is compared to the predicted shear strength by the Canadian standards (CSA/S806-12 2012) and the ACI code for steel-RC structures (ACI Committee 318 2014) using the STM analysis

The CSA/S806-12 STM yielded better prediction of the shear capacity of the specimens with almost similar P_{exp}/P_{pre1} for the three simply-supported specimens with an average of 1.6. However, for the continuous specimens, the predicted loads by the STM was much lower than the

experimental failure loads as the average of P_{exp}/P_{pre1} for the continuous deep beams was 2.7. Thus, it is recommended based on the test results of the three continuous deep beams, to revise the design equations of the STM to take into account the different boundary conditions. On the other hand, the predicted capacities by ACI 318-14 was lower than the experimental capacities of the simply-supported deep beams with an average of 0.84 which is similar to the values noted in a previous study by Andermatt and Lubell (2013). For the continuous deep beams, the predicted values capacities by ACI 318-14 were higher than the experimental capacities for specimens with a/d of 1.0 and 1.5 by 8 and 4%, respectively; however, for the continuous specimens with a/d of 2.0, the predicted value was lower than the experimental one, which can be attributed to the relatively high a/d which is closer to the slender beam category.

Table 4.2: Comparison between the experimental and predicted failure loads.

	Experimental	CSA/S806-12		ACI 318-14	
Specimen	Load P_{exp} , (kN)	STM P_{pre1} , (kN)	P_{exp}/P_{pre1}	STM P_{pre2} , (kN)	P_{exp}/P_{pre2}
S-1.0-0.0	913	601	1.52	957	0.95
S-1.5-0.0	693	423	1.64	794	0.87
S-2.0-0.0	439	270	1.63	620	0.70
C-1.0-0.8	951*	442	2.15	880	1.08
C-1.5-0.8	725*	257	2.82	695	1.04
C-2.0-0.8	474*	152	3.11	531	0.90

* load in one span of the continuous beams

CHAPTER 5: RESULTS AND DISCUSSION OF SERIES II, III, AND IV.

5.1 General

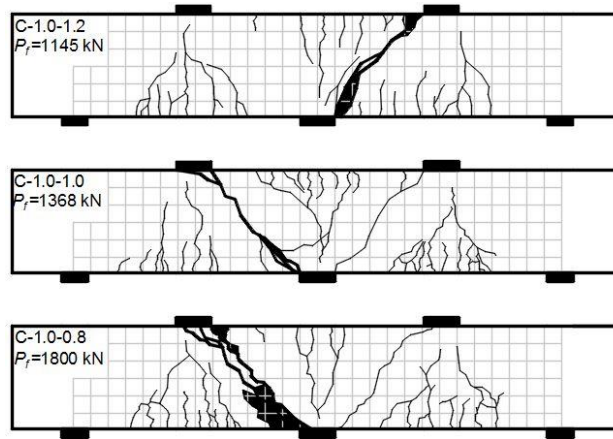
This chapter presents the analysis and discussion of the experimental results of the test beams in Series II, III, and IV. As mentioned in Chapter 3, each series includes three large-scale two-span continuous GFRP-RC deep beams. All beams were tested under one-point loading arrangement in each span. The test variables are the top longitudinal reinforcement ratio and the shear span-to-depth ratio. The main objectives of this series are to evaluate the shear capacity of GFRP-RC continuous deep beams and to examine the applicability of the STM to such beams. The behaviour of the test beams was carefully monitored during the test. The behaviour of the test beams, in terms of cracking patterns, load-deflection curves, and strain variations in the reinforcement bars, is described in this chapter.

5.2 General Behaviour, Cracking and Mode of Failure

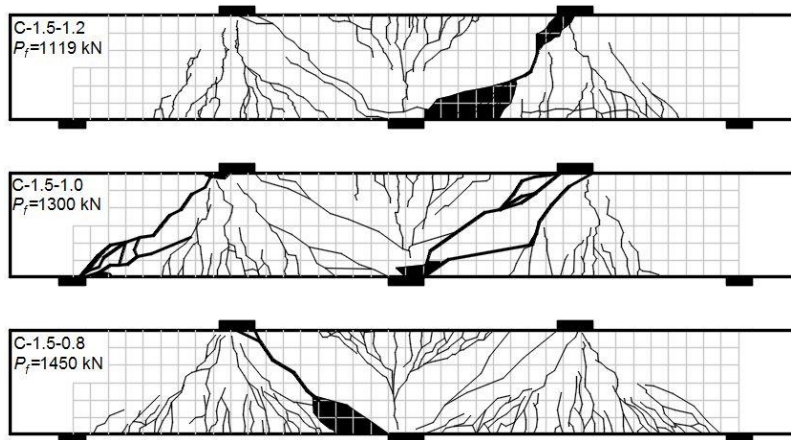
At initial stages of loading, flexural cracks propagated in the hogging moment zone followed by flexural cracks in the sagging moment zone. Such cracks propagated vertically to a distance of about $0.8h$ in series II beams (C-1.0-1.2, C-1.0-1.0, and C1.0-0.8) and $0.9h$ of the specimens in series III and IV beams (C-1.5-1.2, C-1.5-1.0, C-1.5-0.8, C-2.0-1.2, C-2.0-1.0, and C-2.0-0.8) at 19 to 38% of failure load. At higher loads, flexural-shear cracks developed in the interior shear span at the vicinity of the intermediate support, then more cracks formed in the interior and exterior shear spans. With further loading, a diagonal shear crack developed suddenly between the loading points and the intermediate support. The width of the diagonal crack continued to increase with applying additional load until failure. Specimens C-1.0-1.2 and C-1.0-1.0 failed after the

propagation of a diagonal shear crack in one shear span only. In specimen C-1.0-0.8, another diagonal shear crack formed at 73% of the failure load that kept increasing in width up to failure. Specimens C-1.5-1.2 and C-1.5-0.8 exhibited a diagonal shear crack at 88 and 89% of failure load, respectively. However, they failed by another diagonal shear crack that suddenly propagated in the other interior shear span just before failure. On the other hand, specimen C-1.5-1.0 had two diagonal shear cracks in the interior shear spans at 66 and 80% of failure loads, and another two diagonal shear cracks in the exterior shear spans at 83 and 86% of failure load. Specimen C-2.0-1.2 exhibited a diagonal shear crack at 72% of failure load in the interior shear span. However, it failed by another diagonal shear crack that propagated in the exterior shear span before failure. On the other hand, the diagonal shear crack suddenly propagated in the interior shear spans of specimen C-2.0-1.0 at 95% of failure load. Specimen C-2.0-0.8, had a diagonal shear crack at the exterior shear span at 90% of failure load which kept increasing in width until failure. In general, the two spans showed similar behaviour in terms of propagation of cracks before failure. Figure 5.1 shows the cracking pattern near failure of test beams.

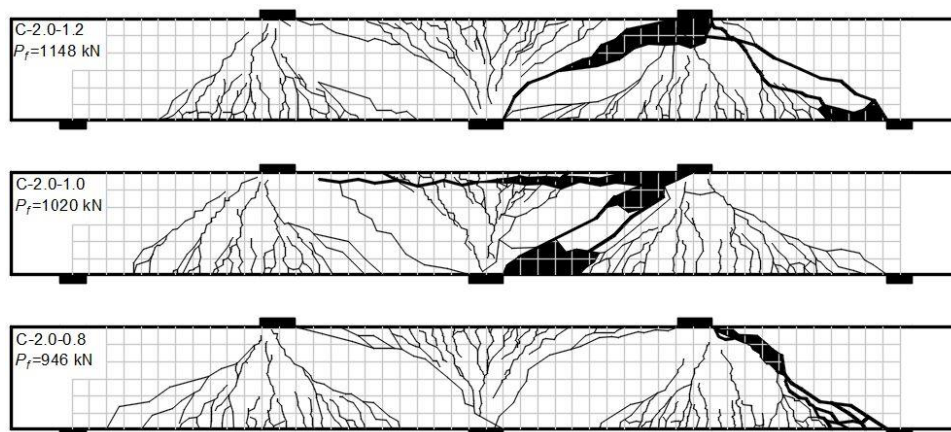
All specimens failed in a sudden and violent manner. No premature failure was observed due to anchorage of the tension reinforcement or due to bearing at the supports. Specimens C-1.0-1.2, C-1.5-0.8, and C-1.5-1.2 exhibited a shear compression failure. However, specimens C-1.0-1.0 and C-1.5-1.0 failed due to compression strut failure after the formation of a diagonal crack in the compression strut zone. Also, diagonal tension failure was observed in specimen C-2.0-0.8. Figure 5.2 shows photos of the test beams at failure. Similar modes of failure were reported for simply-supported deep beams in a study by Andermatt and Lubell (2013).



a) Series II



b) Series III



c) Series IV

Figure 5.1: Cracking pattern at failure of test beams.

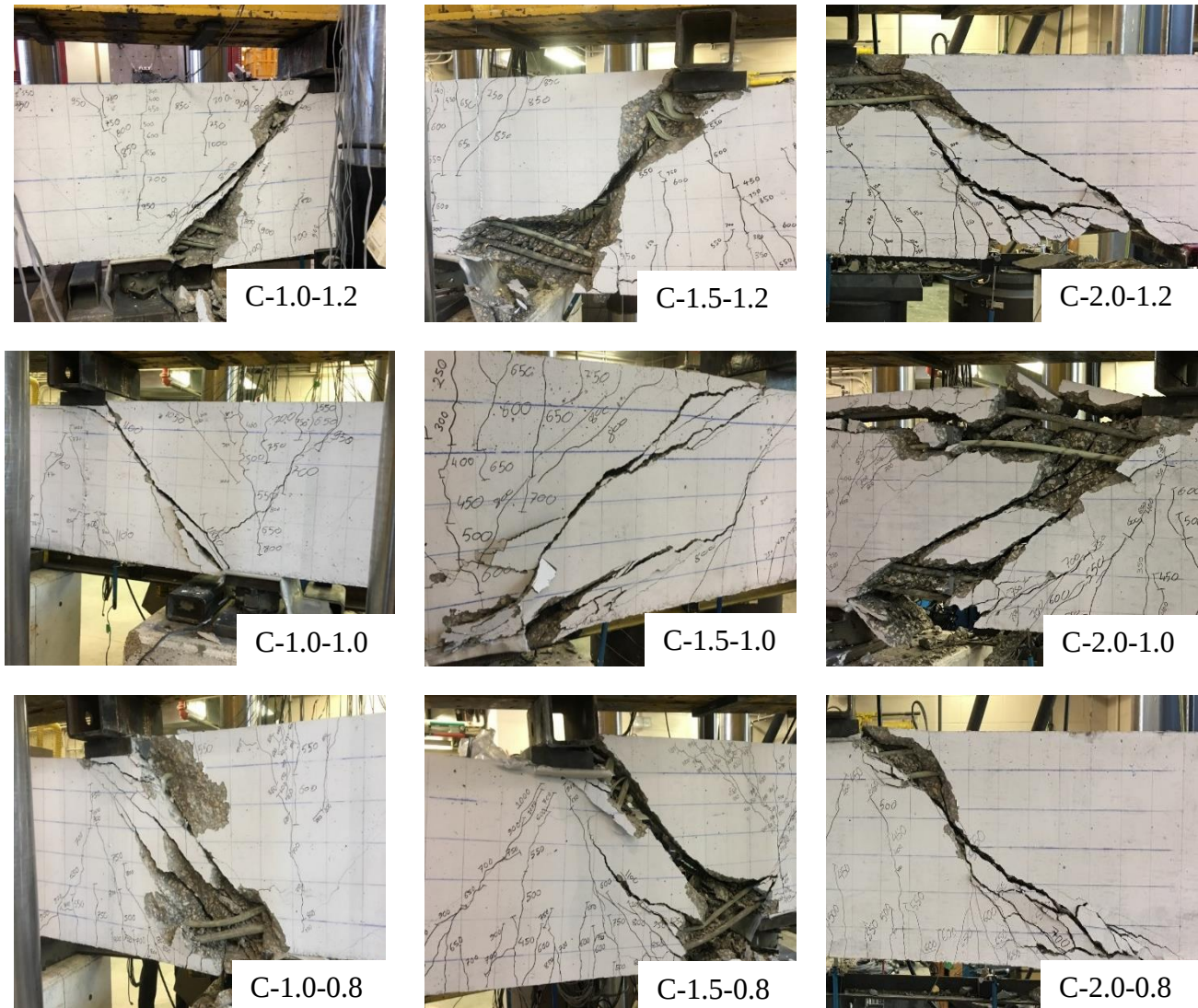
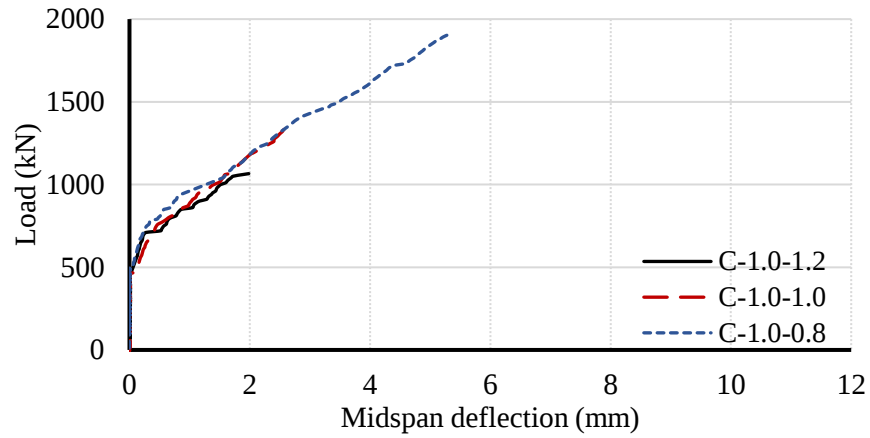


Figure 5.2: Modes of failure of test specimens

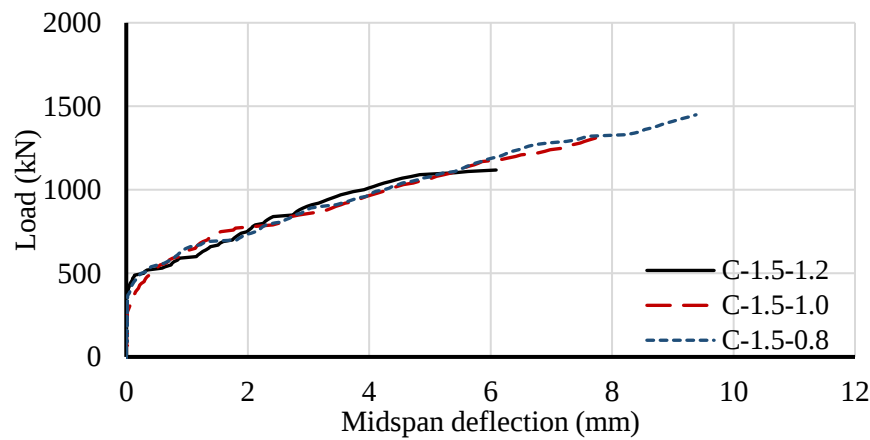
5.3 Load-Deflection Behaviour

The total applied load versus the deflection plot for specimens of series II, III, and IV is depicted in Figure 5.3. The behaviour of the specimens were similar before the formation of the flexural cracks where the deflection was of very small values. After cracking, a reduction in the flexural stiffness of the specimens was observed. It is well-established that the mid-span deflection of continuous beams depends on the effective moment of inertia for both the sagging and hogging

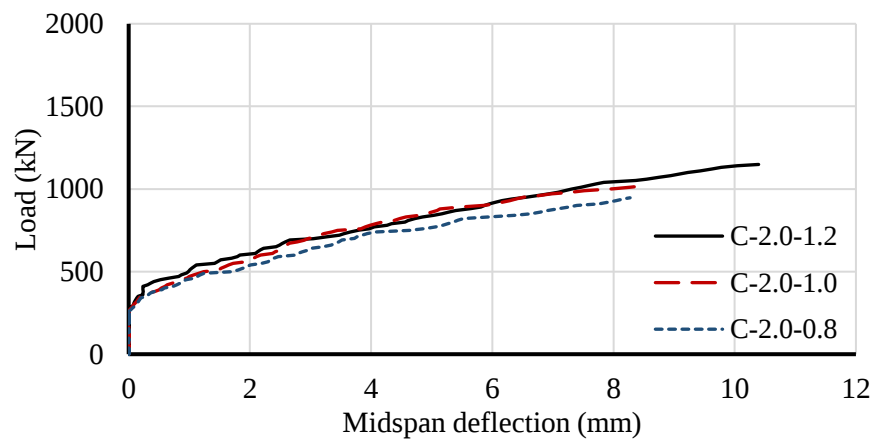
moment sections. However, it seems that the post-cracking deflection of the test specimens was independent of the amount of the top longitudinal reinforcement where all the specimens in each series showed approximately the same reduction in stiffness. On the other hand, increasing a/d resulted in significant reduction in the stiffness and increased the deflection values at failure. Table 5.1 shows the maximum mid-span deflection at failure for all specimens. It can be noted that increasing a/d from 1.0 to 1.5 increased the maximum deflection at failure by 200, 140 and 77% for specimens with top longitudinal reinforcement ratio 1.2, 1.0 and 0.8%, respectively. Further increase in a/d from 1.5 to 2.0 showed a lesser increase in deflection values as the maximum deflection at failure increased by 70, 10, and 13% for specimens with top longitudinal reinforcement ratio 1.2, 1.0 and 0.8%, respectively. In addition, in specimens with a/d of 1.0 and 1.5, decreasing the top longitudinal reinforcement ratio increased the maximum deflection at failure load. Decreasing the top longitudinal reinforcement ratio from 1.2 to 1.0 led to an increase in the deflection at failure by 60 and 26% in specimens with a/d of 1.0 and 1.5, respectively. Further decrease in the top longitudinal reinforcement ratio from 1.0 to 0.8% increased the deflection by 65 and 22% for specimens with a/d of 1.0 and 1.5, respectively. On the other hand, in Series IV ($a/d = 2.0$), decreasing the top longitudinal reinforcement ratio from 1.2 to 1.0% decreased the maximum deflection at failure by 18% while no effect was noticed with further decreasing in the top longitudinal reinforcement ratio.



a) Series II



b) Series III



c) Series IV

Figure 5.3: Load-deflection relationship of test beams.

Table 5.1: Summary of test results

Specimen	Failure Load (kN)	Top reinforcement strain @ failure ($\mu\epsilon$)	Bottom reinforcement strain @ failure ($\mu\epsilon$)	Midspan deflection @ failure (mm)
C-1.0-1.2	1,065	1,510	2,610	2.0
C-1.0-1.0	1,382	1,890	3,180	3.2
C-1.0-0.8	1,902	3,000	4,110	5.3
C-1.5-1.2	1,119	2,870	3,810	6.1
C-1.5-1.0	1,310	3,890	4,450	7.7
C-1.5-0.8	1,450	4,520	5,060	9.4
C-2.0-1.2	1,148	3,850	4,830	10.4
C-2.0-1.0	1,020	3,940	4,180	8.5
C-2.0-0.8	946	4,590	4,130	8.3

5.4 Load Capacity and Redistribution of Internal Forces

This study shows that decreasing the top longitudinal reinforcement ratio has a significant influence on the load capacity of continuous deep beams reinforced with GFRP especially in specimens with low a/d , as shown in Figure 5.4. For series II ($a/d = 1.0$), the test results showed that decreasing the top longitudinal reinforcement ratio from 1.2 to 1.0 resulted in an increase in the load capacity by 30%. Similarly, an increase of 37% of the load capacity was observed with decreasing the top longitudinal reinforcement ratio from 1.0 to 0.8. Also, for series III beams ($a/d = 1.5$), the load capacity increased by 16% with decreasing top longitudinal reinforcement ratio from 1.2 to 1.0. Further decreasing of the top longitudinal reinforcement

ratio from 1.0 to 0.8 resulted in increasing the load capacity by 12%. The enhancement in the load capacity can be attributed to the redistribution of the internal forces which improved as the top longitudinal reinforcement ratio decreased as it is discussed below. In other words, reducing the stiffness of the top tie, resulted in decreasing the force attracted by the top tie and increasing the force attracted by the bottom ties which can be proven by the crack propagation and the developed tensile strains in the reinforcement. Also, a well-developed strain profile occurred in the top and bottom ties of specimen with top longitudinal reinforcement ratio of 0.8% (C-1.0-0.8, C-1.5-0.8, and C-2.0-0.8) compared to specimen with top longitudinal reinforcement ratio of 1.0 and 1.2 % (C-1.0-1.2, C-1.0-1.0, C-1.5-1.2, C-1.5-1.0, C-2.0-1.2, and C-2.0-1.0). However, for series IV ($a/d = 2.0$), the test results show that decreasing the top longitudinal reinforcement ratio from 1.2 to 1.0% resulted in decreasing the load capacity by 10%. Further decrease in the ρ_t from 1.0 to 0.8 resulted in an insignificant decrease in the load capacity by 7%. This might be due to the smaller amount of the redistribution of the internal forces in the test beams of series IV.

In addition, increasing a/d from 1.0 to 1.5 resulted in decreasing the load capacity by 24 and 6% in specimens with top longitudinal reinforcement ratio of 0.8 and 1.0%, respectively. Moreover, the load capacity decreased by 35 and 22% with increasing a/d from 1.5 to 2.0 for specimens with the top longitudinal reinforcement ratio of 0.8 and 1.0%, respectively. However, no significant change in the load was observed with changing a/d ratio in beams having a top longitudinal reinforcement ratio of 1.2%.

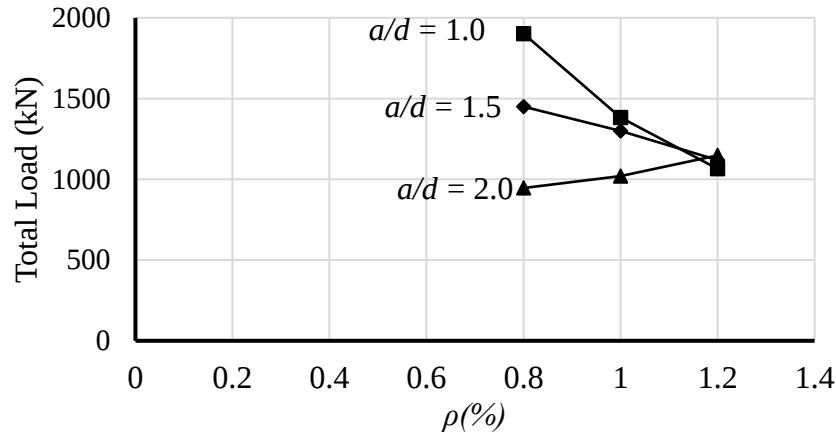
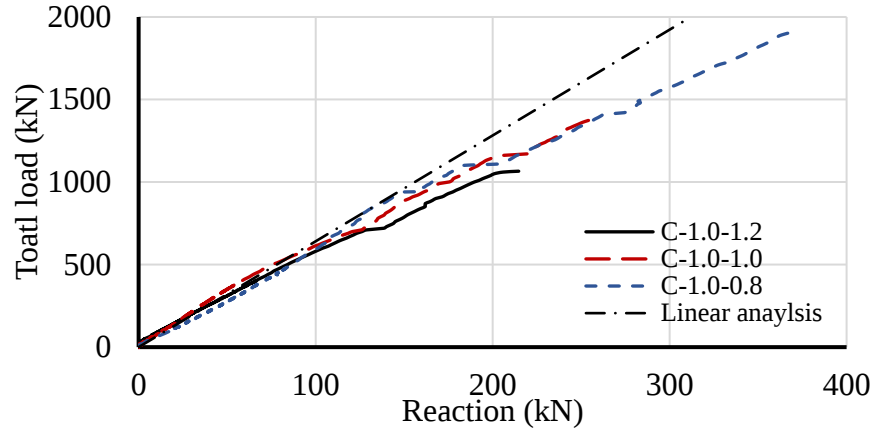
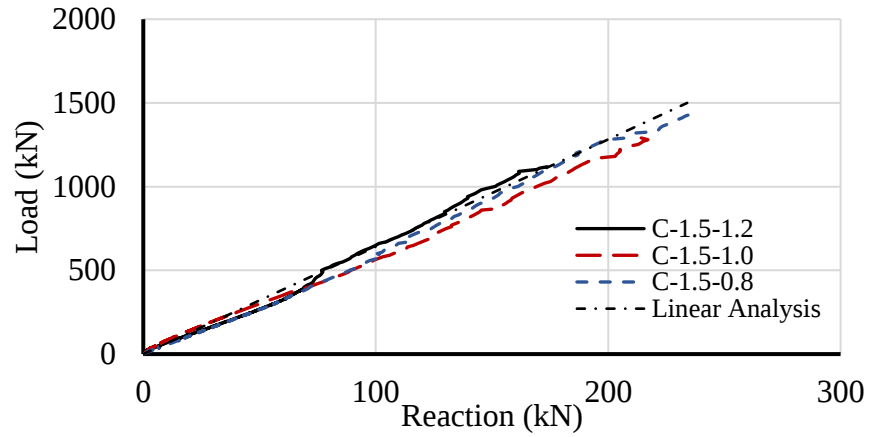


Figure 5.4: Load capacity versus top longitudinal reinforcement ratio.

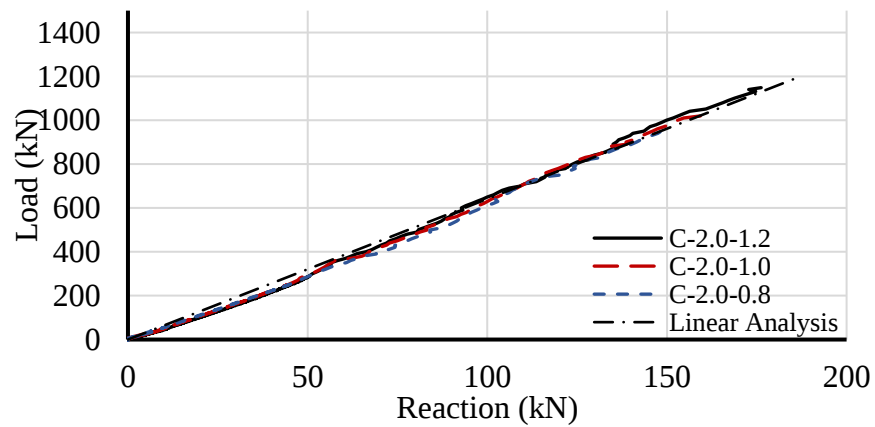
Figure 5.5 shows the total applied load against measured exterior reactions. On the same figure, the elastic exterior reaction obtained from a linear analysis is also presented. Up to the propagation of the first flexural crack, the measured exterior reaction in all beams showed similar values compared to the elastic reaction. However, after the formation of cracks at the hogging and sagging moment regions, the measured exterior reaction was higher than the elastic one. The deviation between the measured exterior reaction and the elastic one kept increasing with the propagation of minor flexural cracks and the development of the diagonal shear cracks. The maximum difference between the experimental and the analytical support reactions increased by 22-27% at failure load in series II ($a/d = 1.0$). For series III ($a/d = 1.5$), the experimental reaction was higher than the analytical reactions by 6 and 7% for specimens C-1.5-1.0 and C-1.5-0.8 at failure, respectively. On the other hand, specimen C-1.5-1.2, which exhibited a reversed redistribution of the internal forces with the propagation of the flexural cracks; however, before failure, the exterior reaction was 6% higher than the elastic reaction with the propagation of the diagonal shear crack. However, in series IV ($a/d = 2.0$), no significant difference was noticed between the measured and elastic exterior reaction. It can be noted from the test results that increasing a/d resulted in reduction of the redistribution of the internal forces.



a) Series II



b) Series III

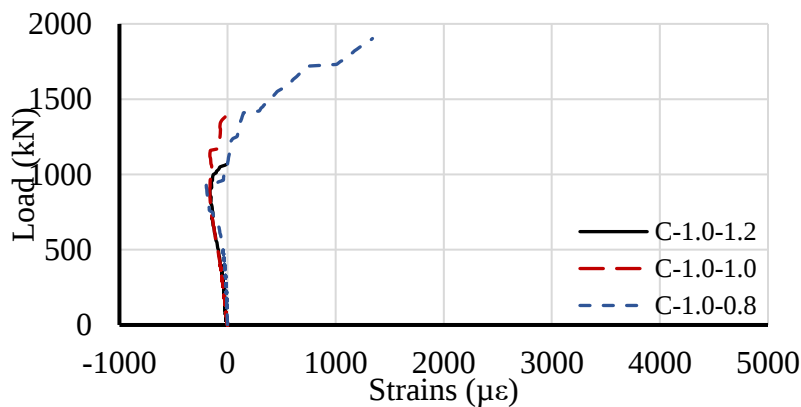


c) Series IV

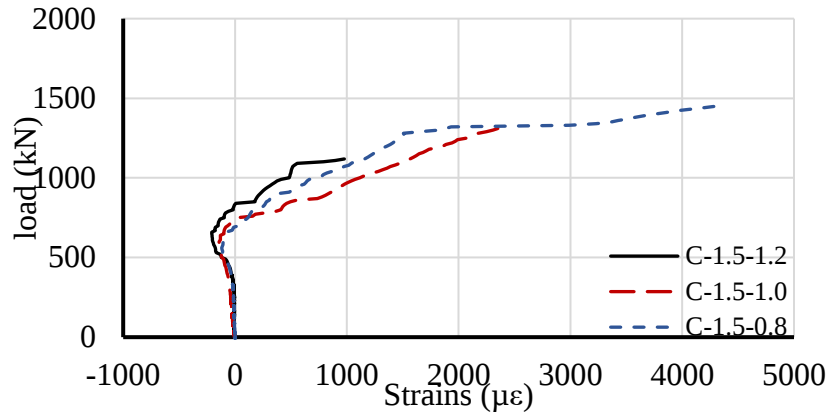
Figure 5.5: Exterior support reaction against total load.

5.5 Strain in Reinforcement and Strain Profiles

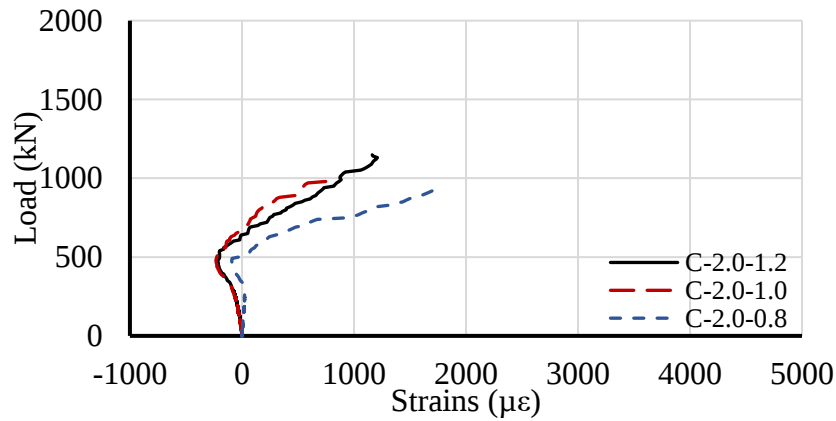
Figures 5.6 to 5.8 show the relationship between total applied load and the strain developed in the longitudinal bottom and top reinforcement of the beams tested at different locations. Both the top reinforcement at the loading point and bottom reinforcement at supporting locations developed a compressive strain until the development of the first diagonal crack at which the strains changed dramatically from compression to tension strains as shown in Figure 5.6. It can be noted that decreasing the top reinforcement ratio from 1.2 to 1.0% did not significantly affect the developed strains at failure in the bottom reinforcement over the intermediate support especially in specimens with a/d of 1.0 and 2.0. On the other hand, in series III ($a/d = 1.5$), decreasing the top reinforcement from 1.2 to 1.0% resulted in an increase in the strain at failure from 900 to 2,350 micro-strain at failure. However, decreasing the top reinforcement ratio from 1.0 to 0.8% changed the developed strain from compression (-50 micro-strain) in specimen C-1.0-1.0 to tension (1,320 micro-strain) in specimens C-1.0-0.8. In the meantime, it increased the developed tensile strain by 83 and 120% in specimens with $a/d = 1.5$ and 2.0. Similarly, increasing a/d resulted in increasing the developed tensile strains in the reinforcement at the same load level, over the intermediate support as shown in Figure 5.6.



a) Series II



b) Series III



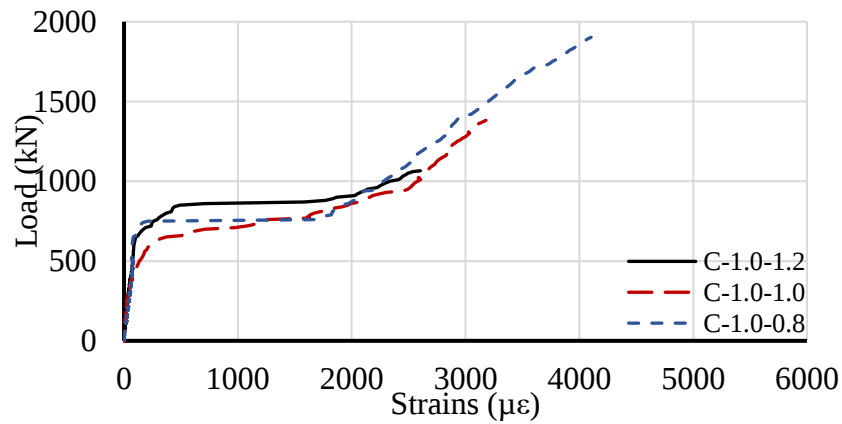
c) Series IV

Figure 5.6: Strain in bottom reinforcement over the intermediate support

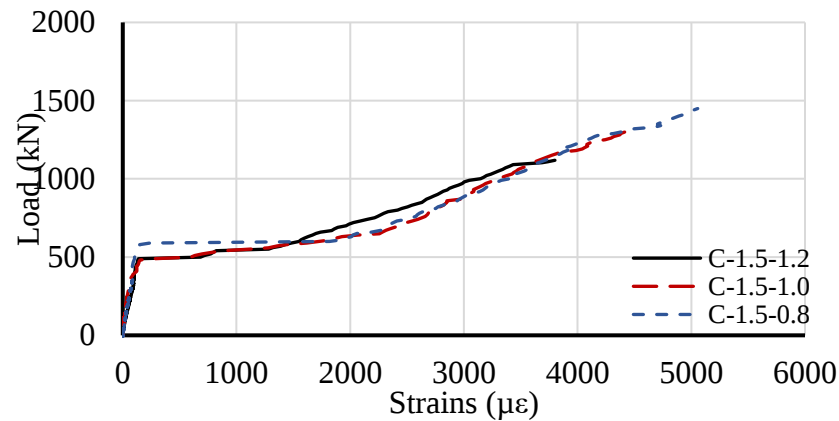
It was noted that decreasing the top longitudinal reinforcement ratio showed significant effect on the developed tensile strain in the bottom reinforcement at midspan at failure load specifically in series II and III. In series II ($a/d = 1.0$) test beams, decreasing the top reinforcement ratio from 1.2 to 1.0% and from 1.0 to 0.8%, resulted in increasing the developed tensile strains in the bottom reinforcement by 23 and 28%, respectively. In addition, for series III ($a/d = 1.5$) test beams, decreasing the top reinforcement ratio from 1.2 to 1.0% and from 1.0 to 0.8%, resulted in increasing the developed tensile strains in the bottom reinforcement by 16 and 12%, respectively.

However, decreasing the top longitudinal reinforcement ratio did not affect the developed tensile strain in the bottom reinforcement at midspan in series IV ($a/d = 2.0$) as shown in Figure 5.7.

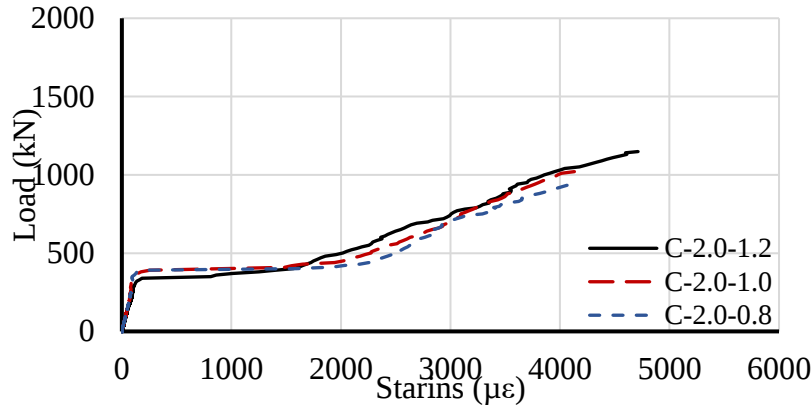
On the other hand, increasing a/d increased the developed strains at failure in the bottom reinforcement at the mid-span, especially for specimens with top reinforcement ratio of 1.2 and 1.0% as the developed tensile strains increased by 46 and 27% with increasing a/d from 1.0 to 1.5 for specimens with top reinforcement ratio 1.2 and 1.0%, respectively. Generally, at the same load level, increasing a/d increased the developed tensile strains in the reinforcement. Table 5.1 shows the maximum tensile strains in the top and bottom reinforcement at failure for all specimens.



a) Series II



b) Series III

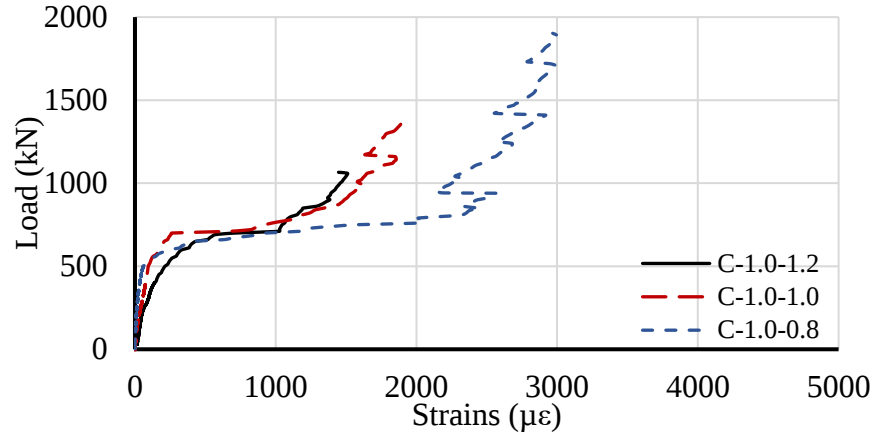


c) Series IV

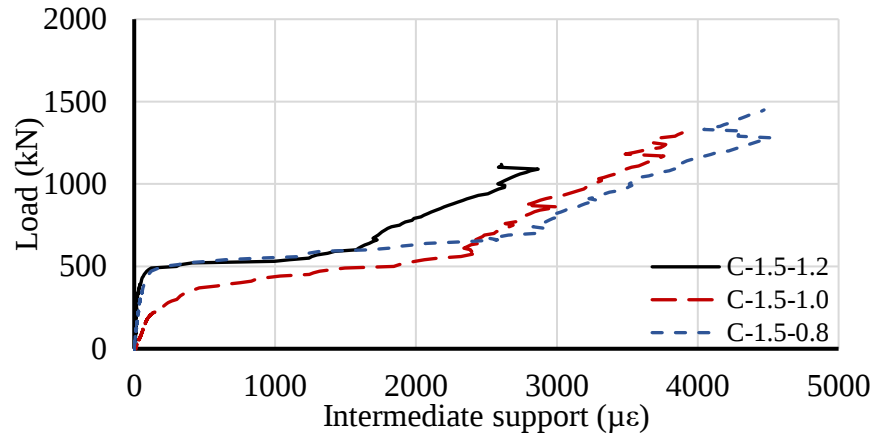
Figure 5.7: Strain bottom reinforcement at midspan.

On the other hand, decreasing top reinforcement ratio significantly affected the developed tensile strain in the top reinforcement. In other words, specimens with relatively low top reinforcement ratio developed higher tensile strains at the same load level and at failure. It was noticed that decreasing top reinforcement ratio from 1.2 to 1.0% increased the tensile strains at failure by 26, 34, and 2.6% in specimens with $a/d = 1.0, 1.5$, and 2.0 , respectively. Further decreasing in the top reinforcement ratio to 0.8%, resulted in increasing the strains at failure by 58, 15, and 18% in specimens with $a/d = 1.0, 1.5$, and 2.0 , respectively.

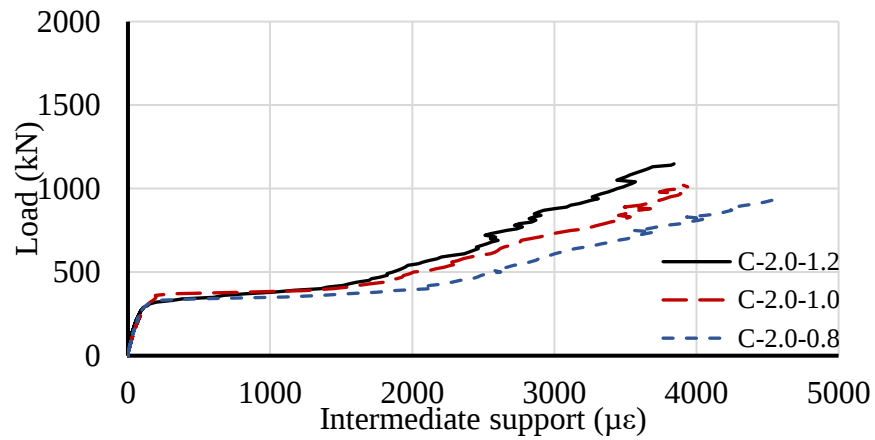
Moreover, increasing a/d increased the developed tensile strains at the same load level for the top reinforcement as in the bottom reinforcement. In other words, increasing the a/d by 50%, increased the strains at failure by 93, 105, and 50% for specimens with top reinforcement ratio of 1.2, 1.0, and 0.8%, respectively. No significant effect, in terms of tensile strains at failure in the top reinforcement, was noticed with increasing a/d from 1.5 to 2.0. Figures 5.8 shows the load-strain relationship for the top reinforcement over the intermediate support.



a) Series II



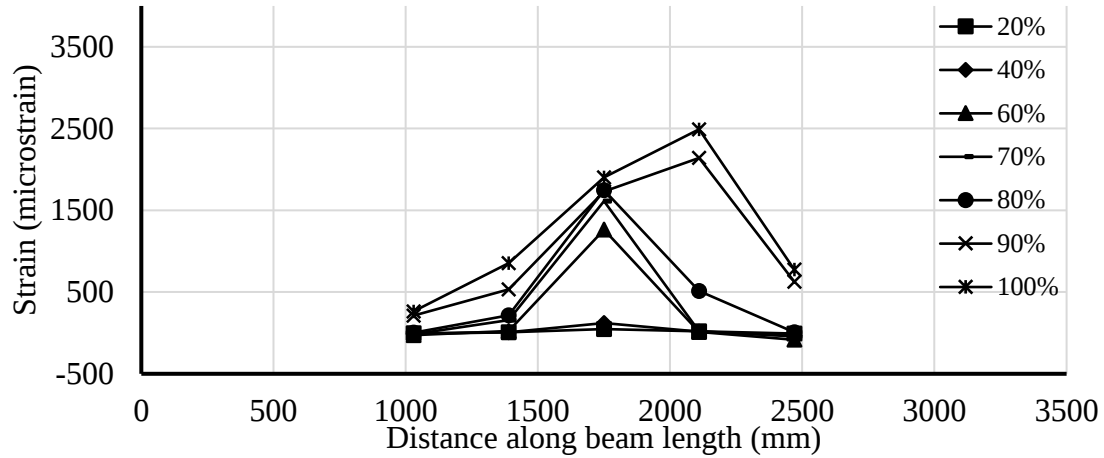
b) Series III



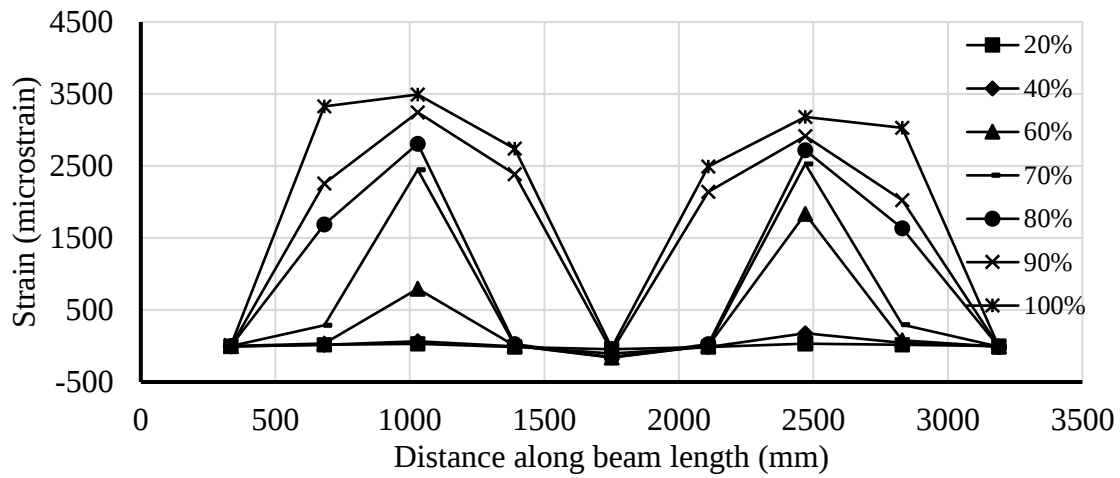
c) Series IV

Figure 5.8: Strains in top reinforcement over the intermediate support

Figures 5.9 to 5.14 shows the reinforcement strain distribution in selected specimens from series II, III, and IV. Generally, the developed strains in the reinforcement was very small until the formation of the flexural cracks, at hogging moment zone then sagging moment zone where a rapid increase in the strains at the locations of the flexural cracks representing the distribution of the bending moment of such loading case. With further loading, the development of strains increased along the bar length confirming the formation of the STM, which was more pronounced in specimens with top reinforcement ratio of 0.8 and 1.0%, in these specimens, the reinforcement strains over the intermediate support changed dramatically from compression to tension reaching 40-80 % of the maximum developed tensile strain in bottom bars at mid-span section in specimens C-1.0-0.8, C-1.5-1.0, C-1.5-0.8, and C-2.0-0.8. Similar behaviour occurred at the loading points and exterior supports in some specimens. In addition, after the formation of the diagonal shear crack, the tensile strains increased rapidly in the shear spans in the location of the diagonal shear crack. Moreover, the strain profiles for all the specimens confirm the development of a STM.

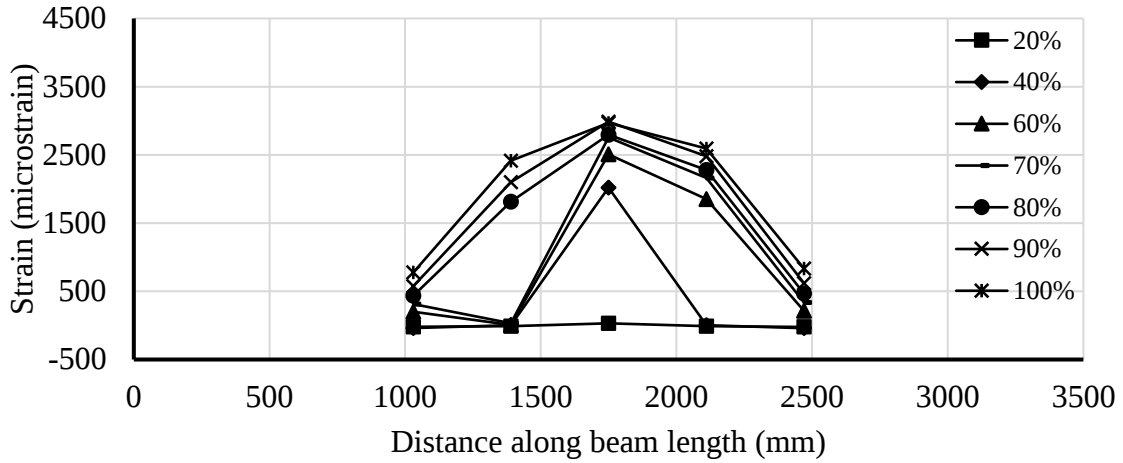


a) Top reinforcement

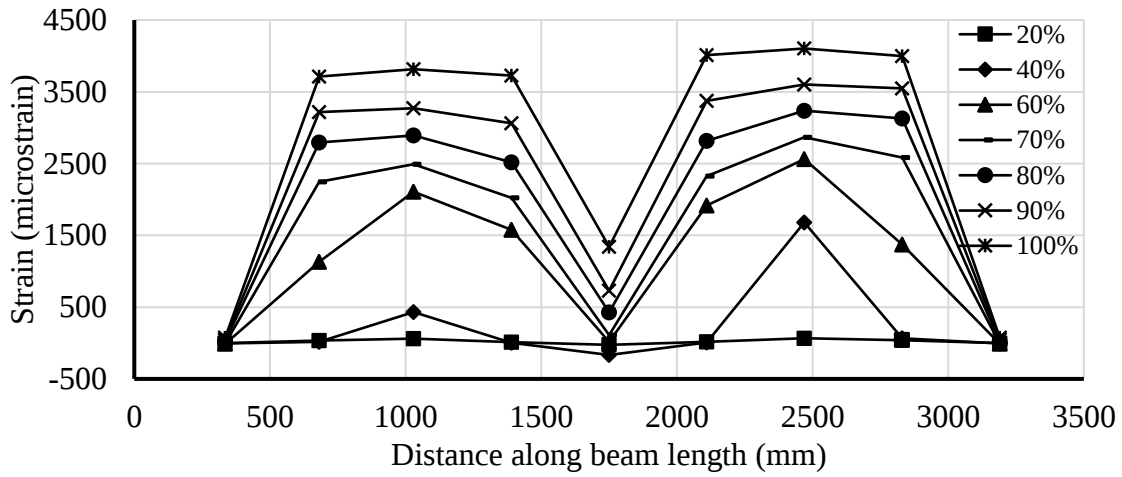


b) Bottom reinforcement

Figure 5.9: Strain profiles for C-1.0-1.0

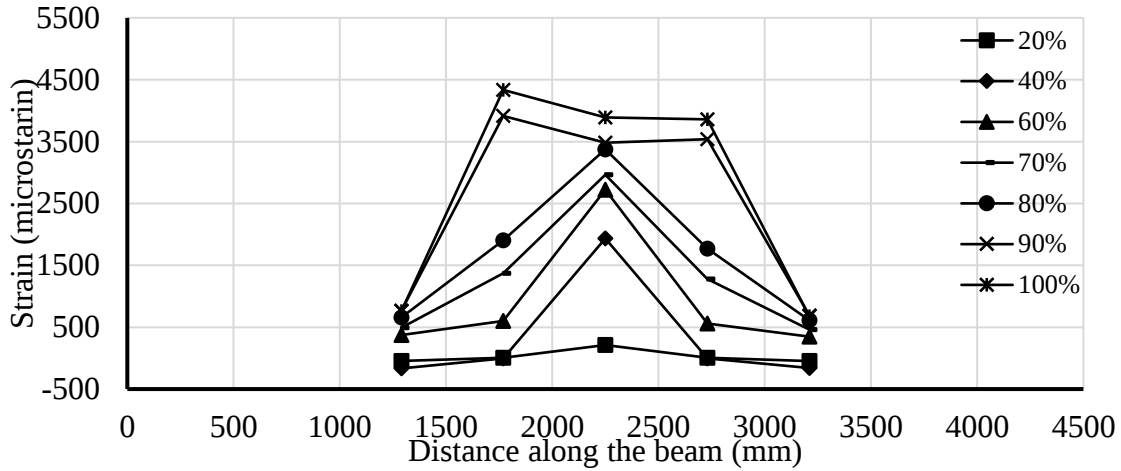


a) Top reinforcement

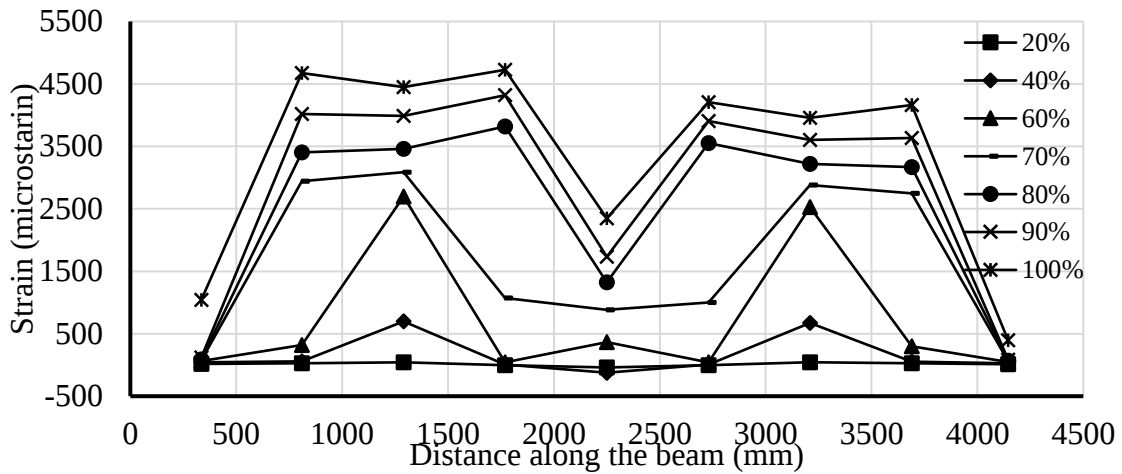


b) Bottom reinforcement

Figure 5.10: Strain profiles for C-1.0-0.8

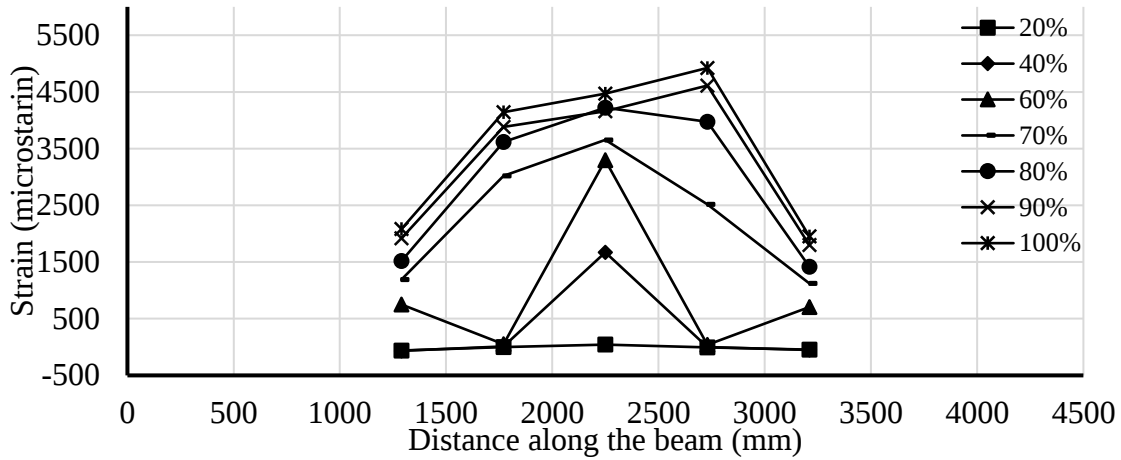


a) Top reinforcement

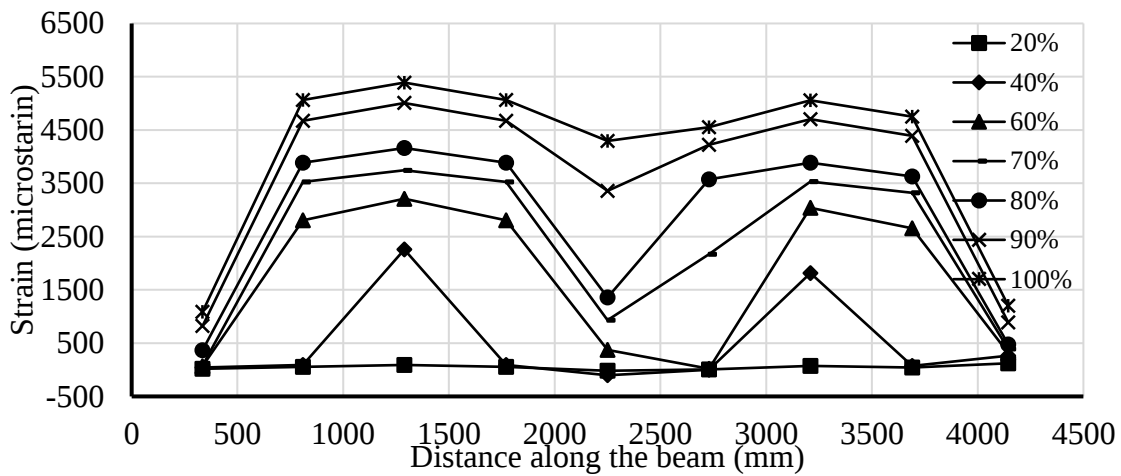


b) Bottom reinforcement

Figure 5.11: Strain profiles for C-1.5-1.0

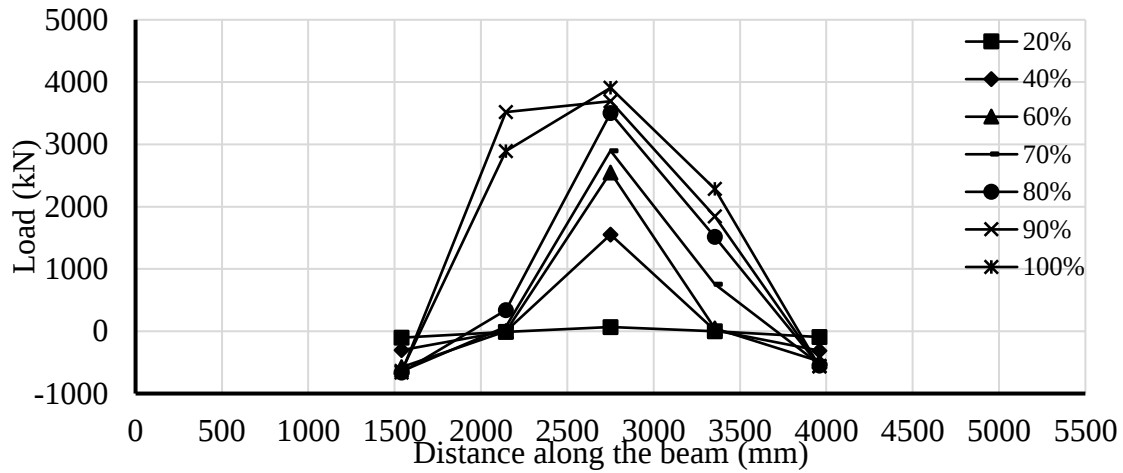


a) Top reinforcement

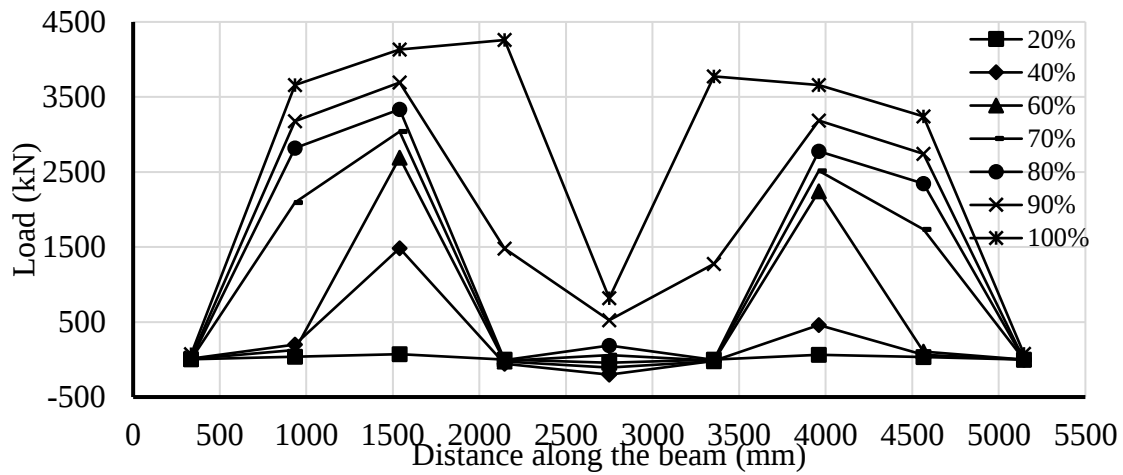


b) Bottom reinforcement

Figure 5.12: Strain profiles for C-1.5-0.8

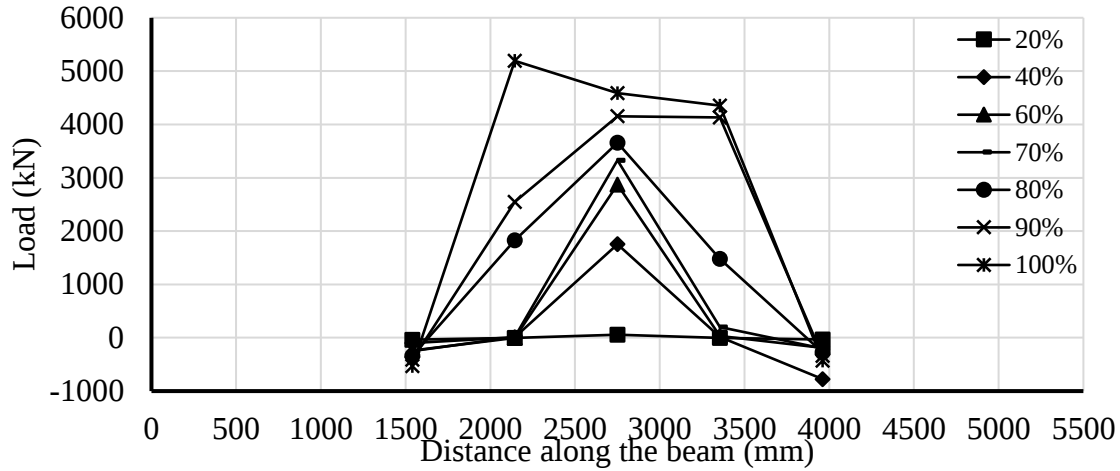


a) Top reinforcement

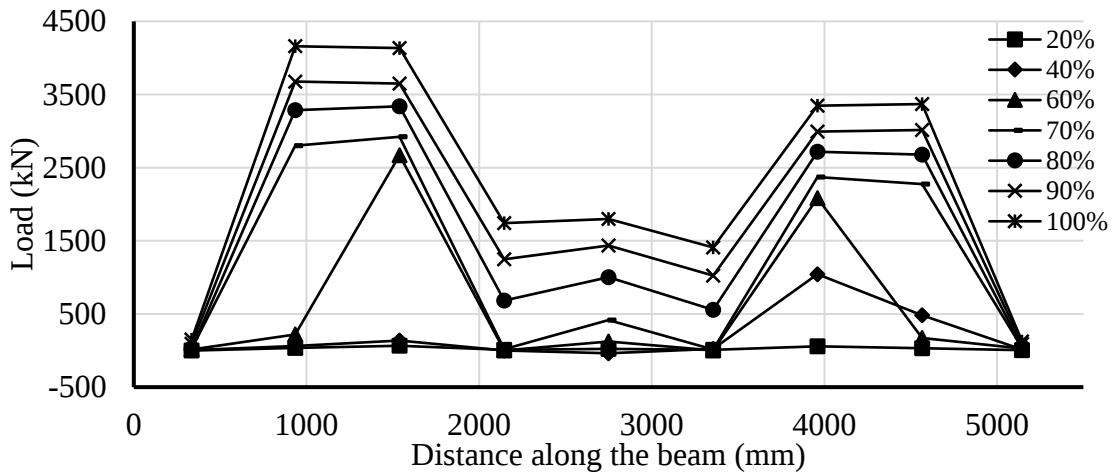


b) Bottom reinforcement

Figure 5.13: Strain profiles for C-2.0-1.0



a) Top reinforcement



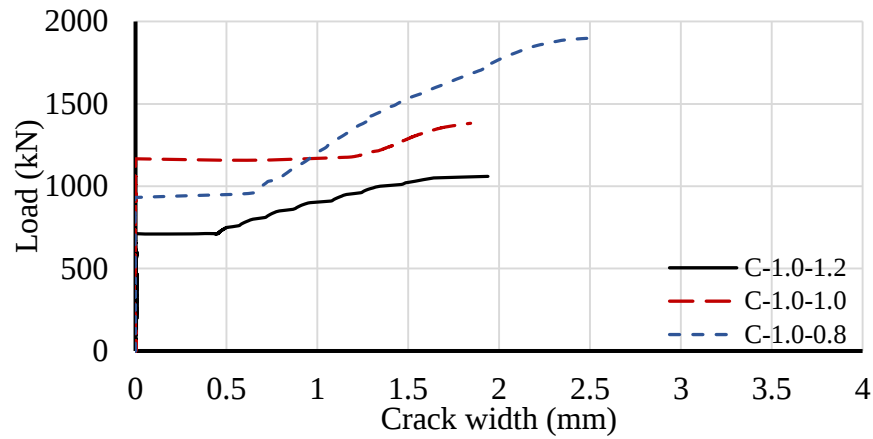
b) Bottom reinforcement

Figure 5.14: Strain profiles for C-2.0-1.0

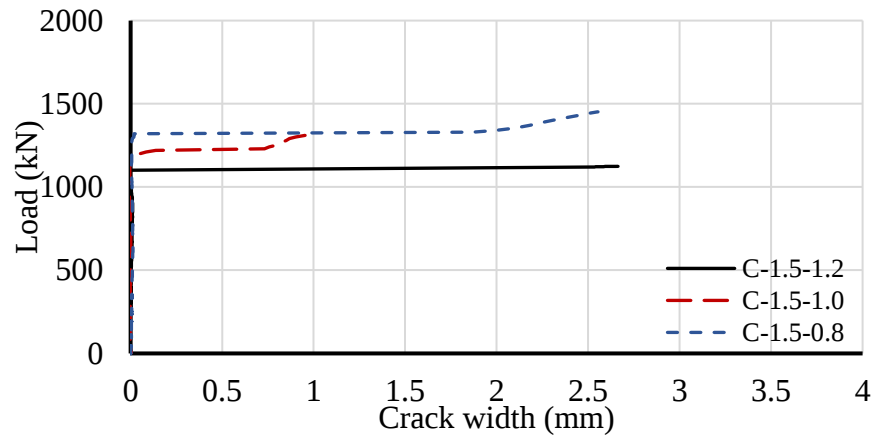
5.6 Width of Diagonal Shear Crack

All specimens exhibited the same behaviour in terms of crack width development. As shown in Figure 5.15, a crack propagated suddenly in each specimen between the edges of the loading point and the intermediate support. The width of the diagonal shear crack kept increasing reaching almost 2.5 mm at failure of the specimen. Specimens C-1.5-1.2, C-2.0-1.2, and C-2.0-1.0 failed

suddenly after the development of the diagonal shear crack. No significant effect was observed due to changing the top longitudinal reinforcement ratio from 1.2 to 0.8%. In all specimens, after the formation of the diagonal shear crack, the width of the crack increased with loading until failure.



a) Series II



b) Series III

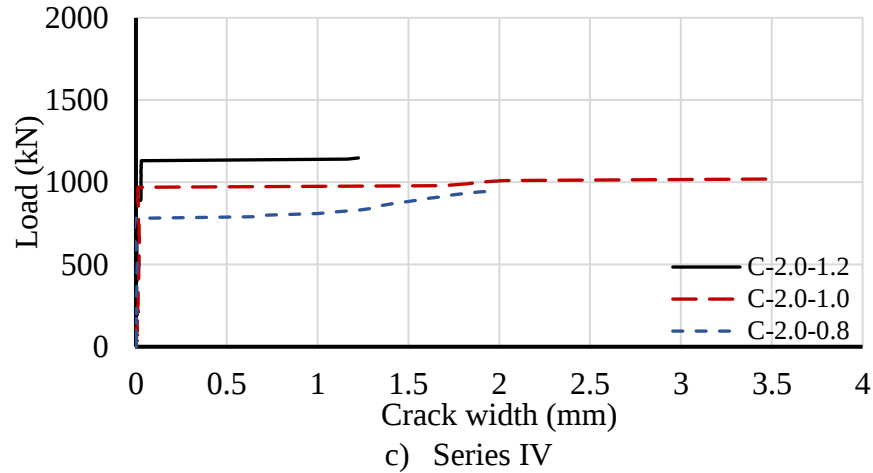


Figure 5.15: Load versus diagonal crack width.

5.7 Code Comparison

In order to investigate the applicability of the STM on continuous FRP-RC deep beams, the test results of the tested specimens were compared to the predicted shear capacity by the Canadian standards for FRP-RC structures (CSA/S806, 2012) and the ACI code for steel-RC structures (ACI Committee 318, 2014) using the STM analysis.

Table 5.2: Experimental and code predicted capacity of test beams

Specimen	Experimental	CSA/S806-12	ACI 318-14
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	Failure mode*	Diagonal Shear Crack (kN)	Failure Load P_{exp} , (kN)	STM P_{pre1} , (kN)	$\frac{P_{exp}}{P_{pre1}}$	STM P_{pre2} , (kN)	$\frac{P_{exp}}{P_{pre2}}$
C-1.0-1.2	SC	710	1,065	1,000	1.07	1,802	0.60
C-1.0-1.0	S	1,180	1,382	958	1.44	1,750	0.79
C-1.0-0.8	S	950	1,902	884	2.15	1,760	1.10
C-1.5-1.2	SC	1,000	1,119	600	1.87	1,425	0.78
C-1.5-1.0	S	1,200	1,310	560	2.33	1,382	0.95
C-1.5-0.8	SC	1,290	1,450	520	2.79	1,390	1.05
C-2.0-1.2	S	1,143	1,148	350	3.28	1,100	1.04
C-2.0-1.0	S	980	1,020	330	3.09	1,044	0.98
C-2.0-0.8	DT	790	946	302	3.13	1,061	0.89

*DT is diagonal concrete tension failure; SC is shear compression failure; S is compression strut failure.

The STM in the CSA/S806-12 (CSA 2012) gave reasonable predictions to the failure loads in series II ($a/d = 1.0$) with an average of $P_{exp}/P_{pre1} = 1.55$. However, it underestimated the failure loads for beams in series III ($a/d = 1.5$) and IIV ($a/d=2.0$), where the average of P_{exp}/P_{pre1} was 2.33 and 3.16, respectively. In addition, in series II and III, the value of P_{exp}/P_{pre1} increased with decreasing ρ_t , as P_{exp}/P_{pre1} increased by 34 and 24% in series II and III by decreasing ρ_t from 1.2 to 1.0%, respectively. In addition, the test results of the tested specimens were compared to the predicted shear capacity by the ACI Code for steel-RC structures (ACI Committee 318 2014). It was noticed that STM design provisions by the ACI 318 2014 were much closer in predicting the load capacity of the GFRP-RC deep beams with an average $P_{exp}/P_{pre2} = 0.91$. This can be attributed to the different design criteria used in the ACI 318 2014, as the capacity of the strut is calculated

based on the efficiency factor which depends on the location and geometry of the strut, neglecting the developed tensile strains in the tie which are considered in the CSA/S806-12. However, these predicted results were lower than the experimental values in most specimens as shown in Table 5.2.

CHAPTER 6: CONCLUSIONS AND FUTURE WORK

6.1 Summary and Conclusions

In this study, the behaviour of GFRP-RC simply-supported and continuous deep beams was investigated. Twelve large-scale RC deep beams were constructed and tested to failure under one-point load in each span. All specimens had the same cross-section dimensions of 250×590 mm. The variables were the shear span-to-depth ratio and the longitudinal top reinforcement ratio. The specimens were divided into four series; each series consisted of three beams. Series I investigated the effect of the shear span-to-depth ratio on the behaviour of simply-supported GFRP-RC deep beams, while Series II, III, IV investigated the effect the shear span-to-depth ratio and the longitudinal top reinforcement ratio on the behaviour of the continuous GFRP-RC deep beams.

Based on the results of the test specimens, the following conclusions can be drawn:

6.1.1 Conclusions from simply-supported deep beams.

- 1) All specimens failed after the formation of a major diagonal shear crack extending from the inside edge of the support plate toward the loading plate.
- 2) Decreasing the shear span-to-depth ratio resulted in reduction in the post-cracking stiffness of test beams and increased the maximum mid-span deflection at failure.
- 3) The developed strains and strain profiles along the reinforcement and the crack orientations confirmed the formation of the strut-and-tie mechanism.
- 4) Increasing the shear span-to-depth ratio from 1.0 to 1.5 and 2.0 decreased the load capacity of the simply-supported deep beams significantly by 24% and 37%, respectively. This can be

attributed to the decrease in the angle of inclination of the strut which adversely affects the efficiency of the compression strut.

- 5) Headed-end bars were able to provide sufficient anchorage in all specimens, as no sign of end-anchorage failure was observed in all tested specimens. In addition, no sign of slippage of the bottom reinforcement from the bar head was noted at failure.
- 6) The predicted values by the CSA/S806-12 STM was closer to the experimental load capacity for the three simply-supported specimens with an average of 1.6. However, the predicated values by the ACI code for steel-RC structures (ACI Committee 318, 2014) were lower than the experimental load with an average of 0.87. This can be attributed to the different design criteria provided in the ACI code which does not take into consideration the developed tensile strains in ties.

6.1.2 Conclusions from continuous deep beams.

- 1) All specimens exhibited brittle shear failure after the formation of the diagonal shear crack in the interior shear span with the exception of specimens C-2.0-0.8 which failed after the propagation of the diagonal shear crack in the exterior shear span only. Shear compression failure, compression strut failure and diagonal tension failure were the observed mode of failures.
- 2) The development of the arch action was confirmed with the propagation of cracks and the developed strains in the top and bottom longitudinal reinforcements. The measured strain indicated the development of ties in the top and bottom longitudinal reinforcement, which was more pronounced in specimens with top longitudinal reinforcement ratio of 0.8 and 1.0%.

- 3) Increasing the shear span-to-depth ratio reduced the post-cracking stiffness significantly and increased the maximum deflection at failure. However, decreasing the top longitudinal reinforcement ratio did not show any effect on the post-cracking stiffness.
- 4) Decreasing the top longitudinal reinforcement ratio from 1.2 to 0.8% increased the load capacity by 76 and 30% of the test continuous deep beams with shear span-to-depth ratio of 1.0 and 1.5, respectively. Due to the redistribution of internal forces that occurs with decreasing stiffens of top longitudinal reinforcement, which was indicated by the increase in the reaction value at the exterior supports. However, this effect diminished in specimens with shear span-to-depth ratio of 2.0.
- 5) Increasing the shear span-to-depth ratio from 1.0 to 2.0 reduced the load capacity of the tested continuous deep beams with top longitudinal reinforcement ratio of 0.8 and 1.0% by 50 and 26%, respectively. On the other hand, no effect was observed for specimen with top longitudinal reinforcement ratio of 1.2%.
- 6) For specimens with shear span-to-depth ratio of 1.0, a significant redistribution of the internal forces after the propagation of cracks was observed. However, increasing the shear span-to-depth ratio adversely affected the redistribution of the internal forces, in specimens with shear span-to-depth ratio of 1.5 and 2.0
- 7) Decreasing the top longitudinal reinforcement ratio increased the developed tensile strain in the top and bottom reinforcement at different location at failure.
- 8) The predicted failure loads by the ACI code for steel-RC structures (ACI Committee 318 2014) were much closer to the experimental failure loads than the predicated values by the Canadian standards for FRP-RC structures (CSA/S806 2012) with an average of 0.91 and 2.35, respectively.

- 9) No signs of end-anchorage failure or slipping of reinforcement was noticed in this study.

6.2 Future Work Recommendations

The following are suggestions for further studies on the behaviour of FRP-RC deep beams:

- 1) The behaviour of FRP-RC continuous deep beams with a wider range of reinforcement ratios should be experimentally investigated to have a better understanding of the effect of different arrangement/reinforcement ratios of the top and bottom ties on the behaviour of such beams
- 2) The effect of different loading configurations (two loading point per span or uniform distributed loading) on the behaviour of FRP-RC continuous deep beams has to be studied, to verify the formation of the STM under these loading configurations.
- 3) Experimental investigation on the size effect on the behaviour of FRP-RC continuous deep beams should be conducted.
- 4) The effect of the web reinforcement ratio and configuration on the behaviour of FRP-RC continuous deep beams needs to be studied.
- 5) As GFRP was used as internal reinforcement in this study, more experiments should be conducted on deep beams reinforced with Carbon FRP (CFRP), Basalt FRP (BFRP) and Aramid FRP (AFRP) bars, to examine the effect of using reinforcement with a wide range of tensile strength and modulus of elasticity. In addition, similar tests can be conducted on Fibre Reinforced Concrete (FRC) deep beams, to understand the effect of bridging of the cracks on the capacity of such beams.

REFERENCES

- ACI Committee 318. (1983). "Building Code Requirements for Structural Concrete" (ACI 318-83) and Commentary (ACI 318R-83), *American Concrete Institute*, Detroit, MI, 1-432
- ACI Committee 318. (2005). "Building Code Requirements for Structural Concrete" (ACI 318-05) and Commentary (ACI 318R-05), *American Concrete Institute*, Detroit, MI, 1-432
- ACI Committee 318. (2014). "Building Code Requirements for Structural Concrete" (ACI 318-14) and Commentary (ACI 318R-14), *American Concrete Institute*, Detroit, MI, 1-519
- ACI Committee 440.1R (2015). "Guide for the Design and Construction of Structural Concrete Reinforced with FRP Bars." *ACI 440.1R-15*, *American Concrete Institute*, Detroit, MI, 1-88
- Andermatt, M. F., and Lubell, A. S. (2013a). "Behavior of Concrete Deep Beams Reinforced with Internal Fiber-Reinforced Polymer-Experimental Study." *ACI Structural Journal*, 110(4): 585–594.
- Andermatt, M. F., and Lubell, A. S. (2013b). "Strength Modeling of Concrete Deep Beams Reinforced with Internal Fiber-Reinforced Polymer." *ACI Structural Journal*, 110(4): 595–606.
- ASCE-ACI Committee 426. (1973). "Shear and Diagonal Tension." *ASCE, Journal of Structure Division*, (6): 1091–1187.
- Ashour, A. F. (1997). "Tests of Reinforced Concrete Continuous Deep Beams." *ACI Structural Journal*, 94(1): 3–11.

- Benmokrane, B., Mohamed, H. M., Manalo, A., and Cousin, P. (2016). "Evaluation of Physical and Durability Characteristics of New Headed Glass Fiber – Reinforced Polymer Bars for Concrete Structures." *ASCE, Journal of Composites for Construction*, 21(2): 1-12.
- Birrcher, D. B., Tuchscherer, R. G., Huizinga, M., and Bayrak, O. (2013). "Minimum Web Reinforcement in Deep Beams." *ACI Structural Journal*, 110(2): 297–306.
- Birrcher, D. B., Tuchscherer, R. G., Huizinga, M., and Bayrak, O. (2014). "Depth Effect in Deep Beams." *ACI Structural Journal*, 111(4): 731–740.
- Birrcher, D., Tuchscherer, R., Huizinga, M., and Bayrak, O. (2009). "Depth Effect in Reinforced Concrete Deep Beams." *Structures Congress*, Austin, Texas, USA: 1595–1603.
- Breña, S. F., and Roy, N. C. (2009). "Evaluation of Load Transfer and Strut Strength of Deep Beams with Short Longitudinal Bar Anchorages." *ACI Structural Journal*, 106(5): 678–689.
- Chemrouk, M., and Kong, F. K. (2004). "High Strength Concrete Continuous Deep Beams – With Web Reinforcement and Shear-Span Variations." *Advances in Structural Engineering*, 7(3): 229–244.
- CSA. (2012). "Design and Construction of Building Structures with Fibre-Reinforced Polymer". CSA/S806-12. *Canadian Standards Association (CSA)*, Tronto, ON, 1-87.
- CSA. (2014). "Design of Concrete Structures." CSA/A23.3-14. *Canadian Standards Association (CSA)*, Tronto, ON, 1–290.

- El-Sayed, A., El-Salakawy, E., & Benmokrane, B. (2005). "Shear strength of one-way concrete slabs reinforced with fiber-reinforced polymer composite bars." *Journal of Composites for Construction*, 9(2), 147-157.
- El-Sayed, A. K., El-Salakawy, E. F., and Benmokrane, B. (2012). "Shear Strength of Fibre-Reinforced Polymer Reinforced Concrete Deep Beams Without Web Reinforcement." *Canadian Journal of Civil Engineering*, 39(5): 546–555.
- Farghaly, A., and Benmokrane, B. (2013). "Shear Behavior of Large Reinforced Concrete Beams without Web Reinforcement." *ASCE, Journal of Composites for Construction*, 17(6): 1–10.
- Fung-Kew Kong David P. Kirby, and David R. Short, P. J. R. (1972). "Deep Beams with Inclined Web Reinforcement." *ACI Journal Proceedings*, 69(3): 172–178.
- Garay, J. D., and Lubell, A. S. (2008). "Behavior of Concrete Deep Beams with High Strength Reinforcement." *Structures Congress*, Vancouver, British Columbia, Canada: 1–10.
- Hsu, T. T. C. (1988). "Softened Truss Model Theory for Shear and Torsion." *ACI Structural Journal*, 85(6): 624–635.
- ISIS Canada. (2007). *"Reinforcing Concrete Structures with Fibre Reinforced Polymers"*. Winnipeg, Manitoba, Canada. 1-151.
- Kani, G. N. J. (1967). "How Safe are Our Large Reinforced Concrete Beams." *ACI Journal Proceedings*, 64(3): 128–141.
- Kong, F. K., Robns, P. J., and Cole, D. F. (1970). "Web Reinforcement Effects on Deep Beams." *ACI Journal Proceedings*, 67(12): 1010–1018.

- Lu, W.-Y., Lin, I.-J., and Yu, H.-W. (2013). "Shear Strength of Reinforced Concrete Deep Beams." *ACI Structural Journal*, 110(4): 671–680.
- Mallick, P. K. (2007). "*Fiber-reinforced composites: materials, manufacturing, and design.*" Dearborn, Michigan, USA. 1-45
- Manuel, R., Slight, B., and Suter, G. (1971). "Deep Beam Behavior Affected by Length and Shear Span Variations." *ACI Journal Proceedings*, 68(12): 954–958.
- Matthys, S., and Taerwe, L. (2000). "Concrete slabs reinforced with FRP grids. I: One-way bending." *Journal of Composites for Construction*, 4(3), 145-153.
- Mohamed, K., Farghaly, A.S. and Benmokrane, B. (2017). "Effect of Vertical and Horizontal Web Reinforcement on the Strength and Deformation of Concrete Deep Beams Reinforced with GFRP Bars." *ASCE, Journal of Structural Engineering*, 143(8): 1-14.
- Oh, J. K., and Shin, S. W. (2001). "Shear Strength of Reinforced High-Strength Concrete Deep Beams." *ACI Structural Journal*, 98(2): 164–173.
- Omeman, Z., Nehdi, M., and El-Chabib, H. (2008). "Experimental Study on Shear Behavior of Carbon-Fiber-Reinforced Polymer Reinforced Concrete Short Beams Without Web Reinforcement." *Canadian Journal of Civil Engineering*, 35(1): 1–10.
- Ramakrishnan, V., and Ananthanarayana, Y. (1968). "Ultimate Strength of Deep Beams in Shear." *ACI Journal Proceedings*, 65(2): 87–98.
- Rogowsky, D. M., MacGregor, J. G., and Ong, S. Y. (1986). "Design of Reinforced Concrete Deep Beams." *ACI Journal Proceedings*, 83(4): 614–623.

- Roy, N. C., and Breña, S. F. (2008). "Behavior of Deep Beams with Short Longitudinal Bar Anchorages." *ACI Structural Journal*, 105(4): 460–470.
- Schlaich, J., Schafer, K., and Jennewein, M. (1987). "Toward a Consistent Design of Structural Concrete." *PCI Journal*, 32(3): 74–150.
- Smith, K. N., and Vantsiotis, A. S. (1982). "Shear Strength of Deep Beams." *ACI Journal Proceedings*, 79(3): 201–213.
- Tan, K. H., and Cheng, G. H. (2006). "Size Effect on Shear Strength of Deep Beams: Investigating with Strut-and-Tie Model." *ASCE, Journal of Structural Engineering*, 132(5): 673–685.
- Tan, K. H., Kong, F. K., Teng, S., and Weng, L. W. (1997). "Effect of Web Reinforcement on High-Strength Concrete Deep Beams." *ACI Structural Journal*, 94(5): 572–581.
- Tan, K. H., and Lu, H. Y. (1999). "Shear Behavior of Large Reinforced Concrete Deep Beams and Code Comparisons." *ACI Structural Journal*, 96(5): 836–845.
- Tang, C. Y., and Tan, K. H. (2004). "Interactive Mechanical Model for Shear Strength of Deep Beams." *ASCE, Journal of Structural Engineering*, 130(10): 1534–1544.
- Tuchscherer, R., Birrcher, D., Huizinga, M., and Bayrak, O. (2011). "Distribution of Stirrups Across Web of Deep Beams." *ACI Structural Journal*, 108(6): 108–115.
- Wight, J. K. and MacGregor, J. G. (2009). "Reinforced Concrete: Mechanics and Design." (6th edition), Pearson Prentice Hall, Upper Saddle River, New Jersey: 1-1157.

- Yang, K. H., and Ashour, A. F. (2007). "Influence of Section Depth on the Structural Behaviour of Reinforced Concrete Continuous Deep Beams." *Magazine of Concrete Research*, 59(8): 575–586.
- Yang, K. H., Chung, H. S., and Ashour, A. F. (2007). "Influence of Shear Reinforcement on Reinforced Concrete Continuous Deep Beams." *ACI Structural Journal*, 104(4): 420–429.
- Zhang, B., Masmoudi, R., & Benmokrane, B. (2004). "Behaviour of one-way concrete slabs reinforced with CFRP grid reinforcements." *Construction and Building Materials*, 18(8), 625–635.
- Zhang, N., and Tan, K. H. (2007). "Size Effect in RC Deep Beams: Experimental Investigation and STM Verification." *Elsevier, Engineering Structures*, 29(12): 3241–3254.
- Zhang, N., and Tan, K. H. (2010). "Effects of Support Settlement on Continuous Deep Beams and STM Modeling." *Elsevier, Engineering Structures*, 32(2): 361–372.

APPENDIX A:
DESIGN OF TEST BEAMS

A.1. Design Criteria for Simply-Supported Deep Beams

A.1.1 Design of specimens S-1.0-0.0

The simply-supported deep beams were design by the STM analysis provided by the Canadian standard (CSA/S806 2012). The main objective was to better understand of the behaviour of FRP-RC simply-supported deep beams through investigating the effect of changing the shear span-to-depth ratio on the capacity of the deep beams. The used STM in designing series I specimens is shown in Figure A.1.

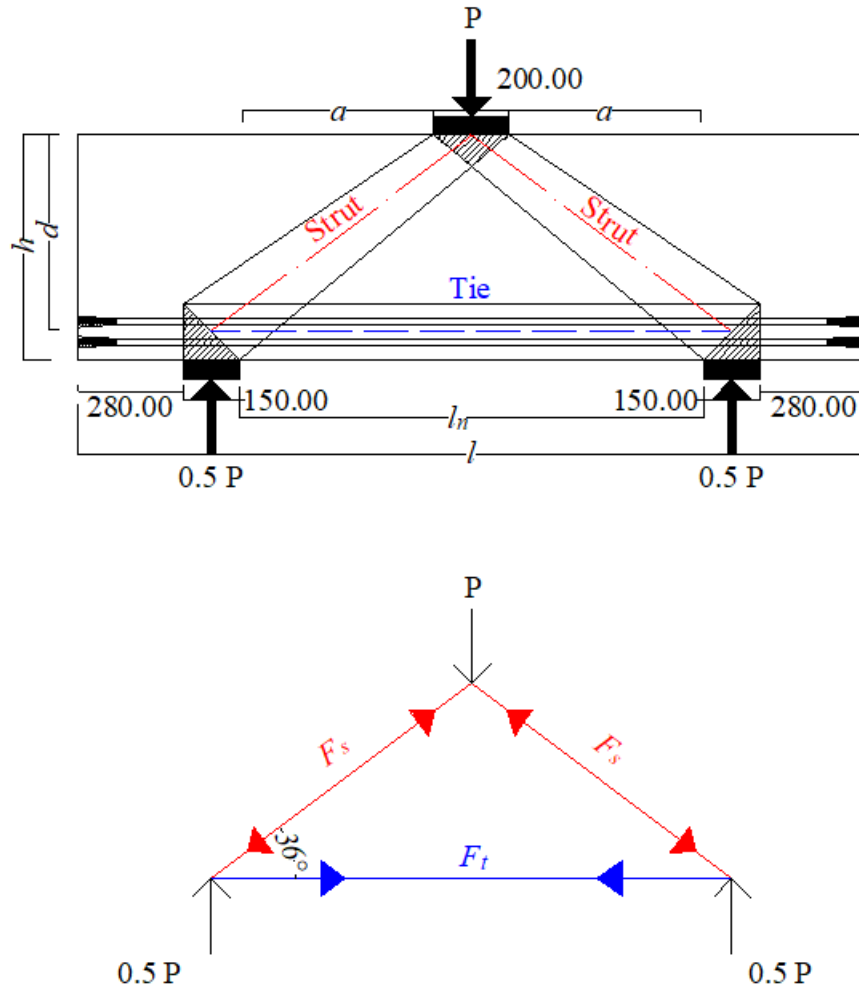


Figure A.1: Proposed STM for the simply-supported deep beams.

- $b = 250$ mm,
- $h = 590$ mm,
- $d = 521.3$ mm,
- $a = 520$ mm,
- $l_n = 1240$ mm,
- $l = 2100$ mm,
- loading Plate (L_p) = 200 mm,

- Supports plate (L_s) = 150 mm,
- $\theta = 36^\circ$
- Target concrete strength were $f'_c = 35$ MPa. The concrete strain before crushing was taken as 0.0035.
- GFRP bars properties:
 - Size No.5, nominal diameter = 15.9 mm, nominal area = 198 mm², tensile strength $f_{frpu} = 1,184$ MPa, $E_{frp} = 62.6$ GPa, and $\varepsilon_{frpu} = 1.89$ %
 - Size No.6, nominal diameter = 19.0 mm, nominal area = 285 mm², tensile strength $f_{frpu} = 1,105$ MPa, $E_{frp} = 63.7$ GPa, and $\varepsilon_{frpu} = 1.73$ %

A.1.1.1 Dimensions of struts and ties.

The longitudinal reinforcement ratio (ρ_b) was assumed to be 1.0%. Three No.6 bars and two No.5 bars are used as longitudinal reinforcement. The calculations of the dimension of the struts and ties can be determined according to clause 8.5.2.2. (CSA/S806 2012), as shown in Figure A.2,

- $A = (3 \times 285) + (2 \times 198) = 1251 \text{ mm}^2$,
- $W_t = (h-d) \times 2 = (590-521.3) \times 2 = 137.40 \text{ mm}$,
- $W_s = \sin \theta \times L_s + \cos \theta \times W_t = (0.6 \times 150) + (0.8 \times 137.4) = 199.92 \text{ mm}$

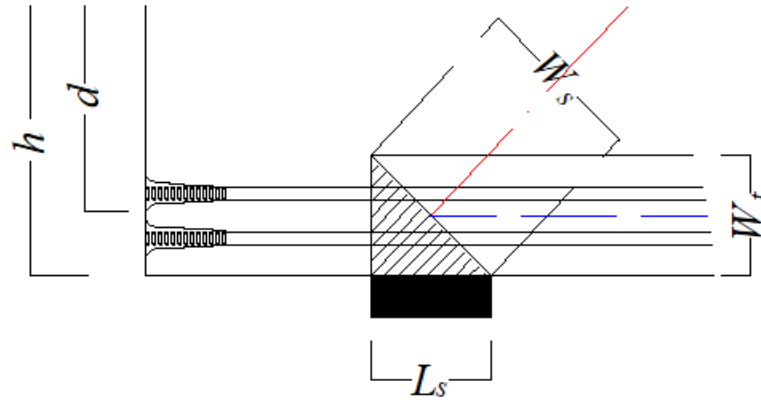


Figure A.2: Widths calculations for struts and ties.

A.1.1.2 Forces in struts and ties

As the strut is the main element in the STM, all specimens were design to exhibit strut failure. Thus, the maximum capacity of the specimens will be calculated based on the maximum capacity of the strut. Then, all the stresses in nodes and ties will be compared to the allowable values. All test beams were designed according the CSA standards (CSA/S806 2012) according to clause 8.5.2. In designing the test beams, all material safety factors were taken equal to unity.

- Force in strut (F_s) $\leq \phi_c f_{cu} A_{cs}$, according to clause 8.5.2.1 where,
 - $\phi_c = 1.0$
 - $A_{cs} = W_s \times b = 199.92 \times 250 = 49980 \text{ mm}^2$ (Clause 8.5.2.2)
 - $f_{cu} = \frac{f'_c}{0.8 + 170\epsilon_1} \leq f'_c$ (Clause 8.5.2.4)
- $$\epsilon_1 = \epsilon_f + (\epsilon_f + 0.002) \cot^2 \theta$$

First, ε_f will be assumed to determine ε_1 and calculate F_s . Based on the calculated F_s , the corresponding load capacity can be calculated from the equilibrium of joints at the top node as shown in Figure A.3, and the corresponding force in tie can be calculated from the equilibrium of joints at the bottom node as shown in Figure A.4, at which the actual strain ε_f can be compared to the assumed. Assume $\varepsilon_f = 0.003$

$$\varepsilon_1 = 0.003 + (0.003 + 0.002) \cot^2 36 = 0.0119$$

$$f_{cu} = \frac{35}{0.8 + 170 \times 0.0119} = 12.40 \text{ MPa}$$

$$F_s = 12.4 \times 49980 = 619.8 \text{ kN}$$

From the equilibrium of the top node joint,

$$P = 2 \times \sin \theta \times F_s = 2 \times 0.6 \times 619.8 = 744 \text{ kN}$$

From the equilibrium of the bottom joint,

$$F_t = F_s \times \cos \theta = 619.8 \times 0.8 = 495.84 \text{ kN}$$

$$\varepsilon_f = \frac{F_t \times 1000}{A_b \times E_{fp}} = \frac{495.84 \times 1000}{1251 \times 63500} = 0.00624 \neq \text{Assumed } \varepsilon_f$$

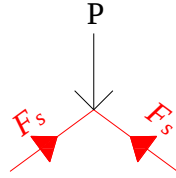


Figure A.3: Top node joint

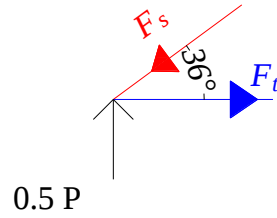


Figure A.4: Bottom node joint

Assume $\varepsilon_f = 0.0048$

$$\varepsilon_1 = 0.0048 + (0.0048 + 0.002) \cot^2 36 = 0.0169$$

$$f_{cu} = \frac{35}{0.8 + 170 \times 0.0169} = 9.53 \text{ MPa}$$

$$F_s = 9.53 \times 49980 = 476 \text{ kN}$$

From the equilibrium of the top node joint,

$$P = 2 \times 0.6 \times 476 = 571 \text{ kN}$$

From the equilibrium of the bottom joint,

$$F_t = 476 \times 0.8 = 380.8 \text{ kN}$$

$$\varepsilon_f = \frac{F_t \times 1000}{A_b \times E_{frp}} = \frac{380.8 \times 1000}{1251 \times 63500} = 0.00479 = \text{Assumed } \varepsilon_f$$

A.1.1.3 Check allowable stresses in ties and nodes

According to clause 8.5.3.1, tensile force in the tie should not exceed $0.65 \phi_t A_b f_{fu}$

$$F_t = 380.8 \text{ kN} < 0.65 \times 1 \times 1251 \times 1136.6 = 924.2 \text{ kN}$$

Actual force in tie is less than the allowable according to the CSA standards (CSA/S806 2012).

According to clause 8.5.3.1, the calculated concrete compressive stress in the node regions shall not exceed $0.85 \phi_c f'_c$ in node regions bounded by struts and bearing areas, and $0.75 \phi_c f'_c$ in node regions anchoring a tie in only one direction.

The allowable concrete compressive stress at the top node (σ_{all}) = $0.85 \times 1 \times 35 = 29.75 \text{ MPa}$

$$\text{Actual stress at strut face} = 476 \times 1000 / 49980 = 9.52 \text{ MPa} < \sigma_{all}$$

$$\text{Actual stress at bearing face} = 571 \times 10000 / 50000 = 11.42 \text{ MPa} < \sigma_{all}$$

The allowable concrete compressive stress at the bottom node (σ_{all}) = $0.75 \times 1 \times 35 = 26.25 \text{ MPa}$

$$\text{Actual stress at strut face} = 476 \times 1000 / 49980 = 9.52 \text{ MPa} < \sigma_{all}$$

$$\text{Actual stress at bearing face} = 285.5 \times 10000 / 37500 = 7.613 \text{ MPa} < \sigma_{all}$$

A.1.1.4 Development length calculation

According to the CSA standards (CSA/S806 2012), clause 9.3 the development length l_d , shall be calculated by the provided equation is clause 9.3.2,

$$l_d = 1.15 \frac{k_1 k_2 k_3 k_4}{d_{cs}} \frac{f_F}{\sqrt{f'_c}} A_b = 1.15 \frac{1 \times 0.8 \times 1 \times 1 \times 1}{39.3} \frac{304.5}{\sqrt{35}} \times 1251 = 406 \text{ mm}$$

A.2. Design Criteria for continuous Deep Beams

A.2.1. Design of specimens C-1.0-1.2

The continuous deep beams were design by the STM analysis provided by the Canadian standard (CSA/S806 2012). The used STM was selected based on a suggested model by Yang et al. (2007) for the continuous deep beams. The main objective was to examine the applicability of the strut-and-tie model in GFRP-RC continuous deep beams. The used STM in designing series II, III, and IV specimens is shown in Figure A.5.

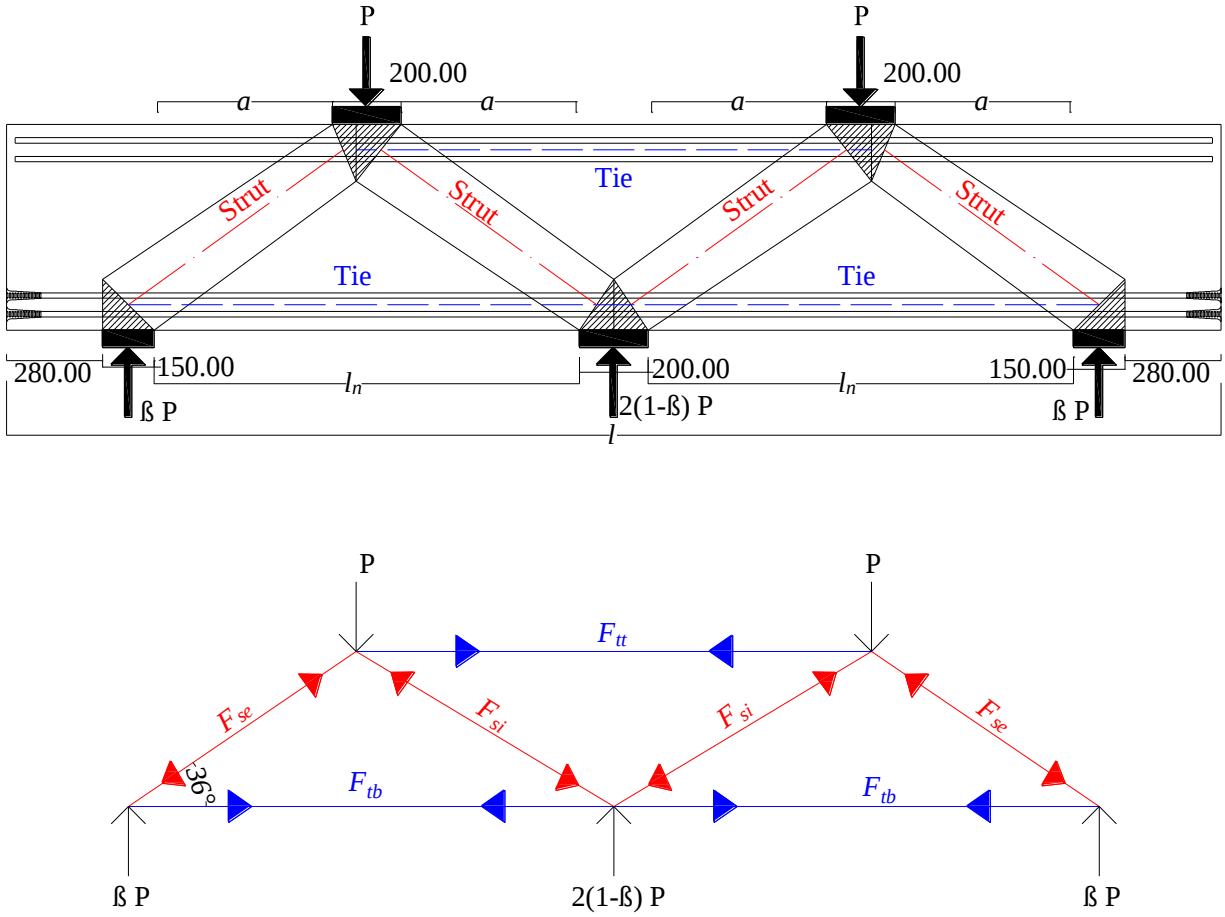


Figure A.5: Proposed STM for continuous deep beams.

- $b = 250$ mm,
- $h = 590$ mm,
- $d = 521.3$ mm,
- $a = 520$ mm,
- $l_n = 1240$ mm,
- $l = 3500$ mm,
- loading Plate (L_p) = 200 mm,

- Internal supports plate (L_{si}) = 200 mm,
- External supports plate (L_{se}) = 150 mm,
- $\theta = 36^\circ$
- $\beta = 0.344$
- Target concrete strength were $f'_c = 35$ MPa. The concrete strain before crushing was taken as 0.0035.
- GFRP bars properties:
 - Size No.5, nominal diameter = 15.9 mm, nominal area = 198 mm², tensile strength $f_{frpu} = 1,184$ MPa, $E_{frp} = 62.6$ GPa, and $\varepsilon_{frpu} = 1.89$ %
 - Size No.6, nominal diameter = 19.0 mm, nominal area = 285 mm², tensile strength $f_{frpu} = 1,105$ MPa, $E_{frp} = 63.7$ GPa, and $\varepsilon_{frpu} = 1.73$ %

A.2.1.1 Dimensions of struts and ties.

The longitudinal reinforcement ratio (ρ_b) was assumed to be 1.0%. Three No.6 bars and two No.5 bars were used as bottom longitudinal reinforcement. Based on conventional elastic analysis for a two-equal spans slender beam, the negative moment at the intermediate support will be 1.2 times the positive bending moment at midspan. Thus, the top longitudinal reinforcement was decided to be 1.2 times the bottom longitudinal reinforcement. Therefore, three No.6 bars and three No.5 bars were used as top longitudinal reinforcement. The dimension of the struts and ties can be determined based on a suggested model by Yang et al. (2007) for the continuous deep beams. The average width of interior strut is W_{si} and exterior strut is W_{se} .

$$W_{se} = \frac{(w'_t + 2c') \cos \theta + (l_{se} + \beta l_p) \sin \theta}{2}$$

$$W_{si} = \frac{(w_t' + 2c') \cos \theta + (0.5(l_{si}) + (1.0 - \beta)(l_p))}{2}$$

where w_t' the smaller of the height of the plate anchored to longitudinal bottom reinforcement w_t and twice distance from the bottom surface to the centroid longitudinal bottom reinforcement $2c$, c' is the distance from the top surface to the centroid of the longitudinal top reinforcement.

- $w_t' = 137.4$ mm,
- $W_{se} = 183.14$ mm,
- $W_{si} = 186.48$ mm,

A.2.1.2 Forces in struts and ties

As the strut is the main element in the STM, all specimens were design to exhibit strut failure. Thus, the maximum capacity of the specimens will be calculated based on the maximum capacity of the interior strut. Then, all the stresses in nodes, exterior strut and ties will be compared to the allowable values. STM of the tested beams were designed according the CSA standards (CSA/S806 2012) according to clause 8.5.2. In designing the test beams, all material safety factors were taken equal to unity.

- Force in strut (F_s) $\leq \phi_c f_{cu} A_{cs}$, according to clause 8.5.2.1 where,
 - $\phi_c = 1.0$
 - $A_{csi} = W_{si} \times b = 186.48 \times 250 = 466200 \text{ mm}^2$ (Clause 8.5.2.2)
 - $f_{cu} = \frac{f_c'}{0.8 + 170\epsilon_1} \leq f_c'$ (Clause 8.5.2.4)

$$\epsilon_1 = \epsilon_f + (\epsilon_f + 0.002) \cot^2 \theta$$

First, ε_f will be assumed to determine ε_1 and calculate F_{si} . Based on the calculated F_{si} , the corresponding load capacity and the forces of the other elements in the strut and tie model can be calculated from the equilibrium of the joints at the supports and loading points, as shown in Figures A.6, A.7 and A.8, at which the actual strain ε_f can be compared to the assumed. Thus, ε_f of the longitudinal top reinforcement is assumed to be 0.0032.

$$\varepsilon_1 = 0.0032 + (0.0032 + 0.002) \cot^2 36 = 0.016$$

$$f_{cu} = \frac{35}{0.8 + 170 \times 0.016} = 9.92 \text{ MPa}$$

$$F_{si} = 9.92 \times 46620 = 463 \text{ kN}$$

From the equilibrium of the intermediate support joint

$$P = \frac{F_{si} \sin \theta}{1 - \beta} = \frac{463 \times \sin 36}{1 - 0.344} = 379 \text{ kN}$$

From the equilibrium of the exterior support joint

$$F_{se} = \frac{\beta P}{\sin \theta} = \frac{0.344 \times 379}{\sin 36} = 242 \text{ kN}$$

$$F_{tb} = F_{se} \cos \theta = 242 \times \cos 36 = 205 \text{ kN}$$

$$\varepsilon_{tb} = \frac{F_{tb} \times 1000}{A_b \times E_{frp}} = \frac{205 \times 1000}{1251 \times 63500} = 0.00258 \neq \text{Assumed } \varepsilon_f$$

From the equilibrium of the loading point joint

$$F_{tt} = (1 - 2\beta)P \cot \theta = (1 - 2 \times 0.344)379 \times \cot 36 = 186 \text{ kN}$$

$$\varepsilon_{ft} = \frac{F_{tt} \times 1000}{A_b \times E_{fcp}} = \frac{186 \times 1000}{1449 \times 63500} = 0.00202 \neq \text{Assumed } \varepsilon_f$$

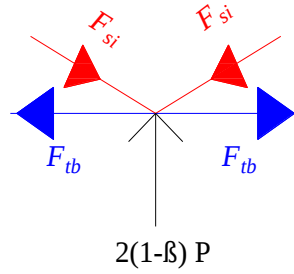


Figure A.6: Intermediate support joint

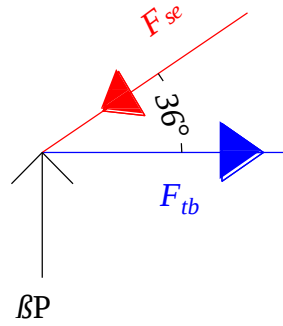


Figure A.7: Exterior support joint

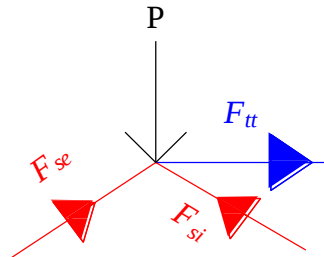


Figure A.8: Loading point joint

Assume $\varepsilon_f = 0.00277$

$$\varepsilon_1 = 0.00277 + (0.00277 + 0.002) \cot^2 36 = 0.0146$$

$$f_{cu} = \frac{35}{0.8 + 170 \times 0.0146} = 10.69 \text{ MPa}$$

$$F_{si} = 10.69 \times 46620 = 498 \text{ kN}$$

From the equilibrium of the intermediate support joint

$$P = \frac{F_{si} \sin \theta}{1 - \beta} = \frac{498 \times \sin 36}{1 - 0.344} = 408 \text{ kN}$$

From the equilibrium of the exterior support joint

$$F_{se} = \frac{\beta P}{\sin \theta} = \frac{0.344 \times 408}{\sin 36} = 261 \text{ kN}$$

$$F_{tb} = F_{se} \cos \theta = 261 \times \cos 36 = 220 \text{ kN}$$

$$\varepsilon_{fb} = \frac{F_{tb} \times 1000}{A_b \times E_{frp}} = \frac{220 \times 1000}{1251 \times 63500} = 0.0027740 = \text{Assumed } \varepsilon_f$$

From the equilibrium of the loading point joint

$$F_{tt} = (1 - 2\beta)P \cot \theta = (1 - 2 \times 0.344)408 \times \cot 36 = 200 \text{ kN}$$

$$\varepsilon_{ft} = \frac{F_{tt} \times 1000}{A_b \times E_{frp}} = \frac{200 \times 1000}{1449 \times 63500} = 0.0021742$$

A.2.1.3 Check allowable stresses

According to clause 8.5.2.1, the capacity of the strut should not exceed $\phi_c f_{cu} A_{cs}$.

$$F_{se} = 261 \text{ kN} < 10.69 \times 45785 = 489 \text{ kN}$$

Actual force in the exterior strut is less than the allowable according to the CSA standards (CSA/S806 2012).

According to clause 8.5.3.1, tensile force in the tie should not exceed $0.65 \phi_t A_b f_{tu}$

$$F_{tt} = 200 \text{ kN} < 0.65 \times 1 \times 1449 \times 1136.6 = 1071 \text{ kN}$$

$$F_{tb} = 220 \text{ kN} < 0.65 \times 1 \times 1251 \times 1136.6 = 924.2 \text{ kN}$$

Actual force in top and bottom ties is less than the allowable according to the CSA standards (CSA/S806 2012).

According to clause 8.5.3.1, the calculated concrete compressive stress in the node regions shall not exceed $0.85 \phi_{fc}'$ in node regions bounded by struts and bearing areas, and $0.75 \phi_{fc}'$ in node regions anchoring a tie in only one direction.

The allowable concrete compressive stress for the loading point node (σ_{all}) = $0.75 \times 1 \times 35 = 26.25$ MPa

$$\text{Actual stress at the exterior strut face} = 261 \times 1000 / 45785 = 5.7 \text{ MPa} < \sigma_{all}$$

$$\text{Actual stress at the interior strut face} = 498 \times 1000 / 46620 = 10.68 \text{ MPa} < \sigma_{all}$$

$$\text{Actual stress at bearing face} = 408 \times 10000 / 50000 = 8.16 \text{ MPa} < \sigma_{all}$$

The allowable concrete compressive stress for the exterior support node (σ_{all}) = $0.75 \times 1 \times 35 = 26.25$ MPa

Actual stress at exterior strut face = $261 \times 1000 / 45785 = 5.7$ MPa < σ_{all}

Actual stress at bearing face = $140.4 \times 10000 / 37500 = 3.74$ MPa < σ_{all}

The allowable concrete compressive stress for the intermediate support node (σ_{all}) = $0.75 \times 1 \times 35 = 26.25$ MPa

Actual stress at the interior strut face = $498 \times 1000 / 46620 = 10.68$ MPa < σ_{all}

Actual stress at bearing face = $535.3 \times 10000 / 50000 = 10.71$ MPa < σ_{all}