REHABILITATION OF REINFORCED CONCRETE BEAMS WITH SPRAYED GLASS FIBER REINFORCED POLYMERS

by

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ABSTRACT

Worldwide, a great deal of research is currently being conducted concerning the use of fibre reinforced plastic wraps or laminates in the repair and strengthening of reinforced concrete members. It has been shown that such techniques can be both effective and economical when compared to the existing practice of retrofitting with steel plates. One of the most significant advantages of such systems is the labour cost savings achieved through their ease of application. A novel technique which further simplifies the application procedure is to apply the fibre using a spraying process. By spraying the fibres onto the member surface concurrently with a suitable matrix resin, a two dimensional random distribution of discontinuous fibres is obtained.

This work reports the first usage of a sprayed glass fibre reinforced plastic (GFRP) retrofit system on reinforced concrete beams. Using E-glass fibres embedded in a polyester matrix, it was shown that significant increases in load carrying capacity, member stiffness and fracture energy can be achieved with such a system. A comparison of results obtained from small scale specimens with those available from current literature was performed. The sprayed glass FRP procedure produced results similar to those obtained with FRP fabric wraps or plate bonding techniques for flexural strengthening. When used for shear strengthening, the sprayed FRP outperformed all of the other techniques. An analysis technique was developed to predict the ultimate load carrying ability of FRP retrofitted beams as well as their load-deflection behaviour up to failure.
ABSTRACT

In the rehabilitation of large scale bridge channel beams, the sprayed FRP technique was found to be more effective than a commercially available continuous fabric system at upgrading structural properties. An economic comparison revealed that the material costs associated with the sprayed approach can result in a significant savings compared to that same system.
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LIST OF SYMBOLS

a = depth of compression block
A_f = area of fiber
A_{fb} = area of FRP on bottom of beam
A_s = area of longitudinal reinforcing steel
A_v = area of steel shear reinforcement
b = b_w = width of beam
c = depth from top of beam to neutral axis
C_c = compressive force in concrete
C_{fst} = compressive force in FRP on side of beam above neutral axis
C_{ft} = compressive force in FRP on top of beam
C_v = strength correction factor for voids
d = d_r = effective depth from top of beam to centroid of reinforcing steel
d_{frp} = effective depth from top of beam to centroid of bottom FRP
E_l = longitudinal elastic modulus of a continuous unidirectional fiber composite
E_2 = transverse elastic modulus of a continuous unidirectional fiber composite
E_f = elastic modulus of fiber
E_{f_{frp}} = elastic modulus of a random discontinuous fiber composite
E_m = elastic modulus of matrix
f'_c = ultimate compressive strength of concrete
f'_{cc} = confined compressive strength of concrete
f_l = f_{fb} = f_{fst} = f_{frp} = ultimate tensile strength of FRP
f_y = yield strength of steel
F_{lt} = ultimate longitudinal tensile strength of a continuous unidirectional fiber composite
F_{2l} = ultimate transverse tensile strength of a continuous unidirectional fiber composite
F_6 = in-plane shear strength of a continuous unidirectional fiber composite
G_f = shear modulus of fiber
G_m = shear modulus of matrix
h = height of beam
h_f = height of FRP on side of beam
K = confined strength ratio
K_e = confinement effectiveness coefficient
L = length of beam
L_f = length of fiber
M_{max} = maximum moment
M_r = moment resistance of beam
P_{ult} = ultimate load carrying capacity of beam
R = mean fiber separation in composite
R_c = radius of curvature of beam
LIST OF SYMBOLS

$R_f =$ radius of fiber
$s =$ spacing of shear stirrups
$t = t_f =$ thickness of FRP
$T_{fb} =$ tensile force in FRP on bottom of beam
$T_{fsb} =$ tensile force in FRP on side of beam below neutral axis
$T_s =$ tensile force in steel
$V_c = V_{ck} =$ shear resistance contribution of concrete
$V_C =$ volume of catalyst
$V_f =$ volume fraction of fiber
$V_{fmin} =$ minimum effective volume fraction of fiber
$V_F =$ volume of fiber
$V_{fp} = V_{fg} =$ shear resistance contribution of FRP
$V_m =$ volume fraction of matrix
$V_{r} =$ shear resistance of concrete beam
$V_R =$ volume of resin
$V_s = V_{sg} =$ shear resistance contribution of steel reinforcement
$V_v =$ volume fraction of voids
$\sigma_l =$ longitudinal tensile stress in a continuous unidirectional fiber composite
$\sigma_f =$ stress in fiber
$\sigma_{f}^\prime =$ stress in fiber at $e_{mu}$
$\sigma_{fu} =$ ultimate strength of fiber
$\sigma_m =$ stress in matrix
$\sigma_{mu} =$ ultimate strength of matrix
$e_c =$ strain in concrete
$e_{ct} =$ strain at extreme upper surface of concrete beam
$e_{cu} =$ ultimate compressive strain of concrete
$e_{cuf} =$ ultimate confined compressive strain of concrete
$e_{fb} =$ strain in FRP on bottom of beam
$e_{fsb} =$ strain in FRP on side of beam below neutral axis
$e_{fst} =$ strain in FRP on side of beam above neutral axis
$e_{ft} =$ strain in FRP on top of beam
$e_{fu} =$ ultimate strain of fiber
$e_{mu} =$ ultimate strain of matrix
$e_s =$ strain in steel
$v_f =$ Poisson’s ratio of fiber
$v_m =$ Poisson’s ratio of matrix
$\delta = \delta_c =$ deflection at midspan
$\delta_1, \delta_2 =$ settlement in testing supports
$\Delta =$ adjusted deflection at midspan
LIST OF SYMBOLS

\( \tau_m = \) shear stress in matrix
\( \tau_{mu} = \) ultimate shear strength of matrix
\( \rho_f = \) density of fiber
\( \rho_{frp} = \) density of FRP composite
\( \rho_m = \) density of matrix
\( \rho_x = \) transverse reinforcement area ratio in x-direction
\( \rho_y = \) transverse reinforcement area ratio in y-direction
\( \phi = \) curvature of beam
\( \phi_c = \) resistance factor for concrete
\( \phi_t = \) resistance factor for FRP
\( \phi_s = \) resistance factor for steel
\( \beta = \) coefficient of diffusion
\( \eta_L = \) efficiency factor for discontinuous fibers
\( \eta = \) correction factor for fiber shape
\( \zeta = \) shape parameter of fiber
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For Edith;

Without whose help, support, consoling and comforting ... I would never have made it. And without whose love ... I wouldn't have cared. Thank you from the bottom of my heart.
1

Introduction

1.1 - Overview

The research project described within this thesis deals with the strengthening or rehabilitation of existing reinforced concrete beams through the use of a sprayed fiber reinforced polymer composite. By applying a coating of fiber reinforced polymer to the outer surface(s) of a concrete beam to provide additional reinforcement, it is possible to increase the load carrying capacity, stiffness and energy absorption potential of the member.

1.2 - Problem Description

One of the biggest problems facing the construction industry today is the structural inadequacy of a large proportion of our transportation infrastructure. Throughout the world, transportation agencies are constantly faced with the need to upgrade, repair and maintain deteriorating structures, a task which is made even more difficult by shrinking resources,
budgets and manpower. A great many of these structures were designed in the 1950s and 60s and no longer meet the demands placed on them. As an example of the magnitude of this problem, over 40% of all bridges in the United States are currently considered to be structurally deficient.

Such structural deficiency could be a result of several factors. From a repair viewpoint, deterioration or damage to the structure from durability related issues or vehicle impact are probably the most significant factors. Revisions to building codes or changes in the original purpose of a structure, such as the need to carry heavier loads or higher traffic volumes can spawn the need to upgrade its structural capacity. In addition, the greater importance placed upon the seismic response of structures in recent years also adds to the need for rehabilitation of older structures.

Of particular relevance to the upgrading or strengthening issue is the shear strength of reinforced concrete beams. Existing beams can be deficient in shear strength for several reasons, including shear deficient design procedures in older codes, increased service loads on the structure and corrosion of the shear stirrups (which are typically placed outside the flexural reinforcement and are thus protected by less cover concrete).

The obvious solution to this problem is to simply demolish the deficient structures and replace them with new ones. Unfortunately, the economics of today’s world make this solution impractical. The only remaining alternative then, is to repair and/or upgrade these structures to modern standards. Some form of external reinforcement is needed to repair and
strengthen these deficient structures.

1.3 - Strengthening With Steel Plates

In the past, post-reinforcement of concrete structures was primarily accomplished through the use of steel plates bonded to the surface of the structural members. Though it has been in use for over 25 years now, there are a number of concerns associated with this technique.

Difficulties in handling the heavy steel plates, susceptibility to corrosion at the steel/adhesive interface, difficulties in forming acceptable butt joints in the field (necessary due to the limited available lengths of steel plate) and the sheer amount of labor and time involved make this technique much less attractive.\(^1,3,5,6-8,11,16-19\)

1.4 - Strengthening With Fiber Reinforced Polymers

In recent years, a great deal of research has been conducted into the use of fiber reinforced polymers in place of steel for the strengthening of reinforced concrete members. A fiber reinforced polymer (FRP) is a composite material made up of two distinct, independent elements. The primary structural component is the fibers, which are encapsulated by a matrix composed of some type of polymer. The basic theory behind the creation of such a material is to use the fibers, which are significantly stronger than the matrix in tension, to carry any tensile stresses applied to the composite material. A variety of different fiber types can be used for this purpose, with the most common being glass, carbon and aramid.
The polymer which makes up the matrix portion of the composite typically serves a dual purpose:

1. protect the fibers from external damage or environmental attack and
2. transfer tensile stresses from one fiber to another and from an external load source to the fibers.

For fiber reinforced polymers loaded in flexure, the polymeric matrix is also responsible for providing compressive strength in the region above the neutral axis while the fibers provide the tensile strength necessary below the neutral axis, much like the roles of concrete and steel in reinforced concrete beams. Due to a low compressive strength of most polymers and the inefficiency of the fibers in such loading configurations, fiber reinforced polymers are rarely used solely to resist compressive stresses or moments. Common matrix polymers include polyesters, vinyl esters and epoxies.

There are a number of significant advantages associated with these materials. Unlike steel, FRPs possess excellent corrosion resistance. They also exhibit good fatigue resistance and a very low coefficient of thermal expansion in the fiber orientation. High specific stiffness and strength characteristics contribute to structural efficiency and their low densities make physical implementation much easier. Their electromagnetic neutrality (electrical neutrality as well in the cases of glass and aramid fibers) can be a useful benefit in certain types of structures. The higher costs associated with these FRP materials is typically offset by savings in labour when compared to the difficulty of steel plate jacketing.
Up to now, the majority of research into FRP strengthening has revolved around the use of laminated plates or woven fiber fabrics which are epoxy-bonded to the concrete surface. Strengthening of unreinforced concrete specimens with sprayed FRP was first studied at the University of British Columbia by Banthia et al. The work presented here is part of a continuing research program that investigates the effectiveness of this technique on reinforced concrete members.

1.5 - Study Objective

The scope of this project was to investigate the use of sprayed fiber reinforced polymers as a strengthening and repair method for existing reinforced concrete beams. The primary objective was to determine whether such a system could be a feasible alternative to existing techniques.
2

Literature Survey

2.1 - Introduction

As mentioned in Chapter 1, the research performed throughout this project involved the use of a sprayed FRP for the rehabilitation of reinforced concrete members. Since this is a novel approach to the problem of rehabilitation, there has been limited research performed using this technique and hence a limited number of publications available referring to it. Though the sprayed FRP technique itself has not been investigated extensively before, it is essentially an extension of previous work done on other rehabilitation techniques. Fundamentally, these various approaches are all alike in that they all involve the attachment of additional reinforcement to the outside of existing reinforced concrete members. This chapter will discuss some of this other research and cover the different techniques used.

The initial work in this field revolved around the use of steel plates epoxy bonded to the face of the concrete structure. This concept has moved well beyond the research stage and
has been used in practice for many years. The steel plates were typically bonded to the
tension face of concrete members and were effective at providing increased load carrying
ability and flexural stiffness, while at the same time reducing deflections and cracking in the
member.\textsuperscript{5,17}

Though, structurally, steel plate bonding is a very effective rehabilitation technique, there are
a number of serious problems associated with the use of steel in this manner; difficulties in
handling the heavy plates, susceptibility to corrosion at the steel/adhesive interface,
difficulties in forming acceptable butt joints in the field and the requirement of large
amounts of labor and time.\textsuperscript{1,3,5,6,8,11,16,19} Still, the use of steel plate bonding remained the
technique of choice for several years before alternatives began to appear.

The single largest problem with steel plate bonding is the interfacial corrosion issue since it
leads directly to bond failure and exfoliation of the steel plates. A material was needed that
could provide the necessary structural properties, while at the same time avoiding this
corrosion problem. Thus, in the mid 1980s the focus of plate bonding research shifted toward
fiber reinforced polymers (FRPs). Their immunity to the corrosion problems inherent with
ferrous metals made these materials an ideal substitute from a durability standpoint. Adding
their high specific stiffness and strength characteristics, as well as their low densities, made
FRPs a very appealing alternative to steel.\textsuperscript{1,3,7,9,10,16,17,20-22} The initial research into the use of
FRPs was conducted by U. Meier \textit{et al} the Swiss Federal Laboratories for Materials Testing
and Research (EMPA).\textsuperscript{21,34,35} This original research investigated the use of laminated FRP
plates as the reinforcement material.
Following the promising results reported at EMPA, a number of other researchers around the world began their own investigations into this new area, though the motivation behind such work tended to vary from one region to another.\textsuperscript{10} One of the earliest and largest programs originated in Japan where the primary interest seemed to be in developing a simpler and more efficient technique for upgrading the transportation infrastructure. In North America, the emphasis was on the repair of decaying structures, primarily due to corrosion of steel reinforcement, though seismic upgrading was also a major thrust on the west coast. Many European researchers began looking at implementing the approach to upgrade or restore damaged historical structures.

Other deviations began to appear as different fibers, resins and adhesives were investigated. One of the major advances came from Japan, where the laminated FRP plates were replaced by much thinner pre-impregnated sheets that significantly simplified field application.\textsuperscript{8} Further modifications followed as woven or unidirectional fabrics became popular. Most of the commercially available systems today consist of such fabrics which are applied dry and impregnated manually by the contractor.

\section*{2.2 - FRP Materials}

As mentioned in Section 1.1, the fiber types used in FRP strengthening procedures include carbon, glass and aramid, with carbon being by far the most popular due to its superior stiffness characteristics.\textsuperscript{19,21,36} Typical matrix resins include epoxies, vinyl esters and polyesters, with epoxies being the material of choice due to their superior bonding
capabilities with both the concrete and the fiber reinforcement and improved durability.\textsuperscript{36} Between these two variables alone, a significant number of different FRP materials have thus far been investigated.

There are also a number of different formats that the FRP retrofit materials are used in. FRP plate bonding techniques are typically performed using 0.5-2 mm thick unidirectional FRP laminates.\textsuperscript{16} The FRP tow sheets preferred by the Japanese researchers (among others) are significantly thinner, only 0.1 to 0.5 mm thick.\textsuperscript{17} Both formats can be fabricated to the desired width, whether it be the full width of the retrofitted member or narrow strips spaced at intervals similar to shear stirrups.

These FRP plates and sheets are manufactured using the pultrusion process and are thus unidirectional with fibers running only in the longitudinal direction. As a result, the FRPs are anisotropic, with their strength in the fiber direction being very high but with very little strength perpendicular to the fiber direction. They do not exhibit a definite yield point, but instead tend to behave linearly elastic up to failure.\textsuperscript{8} Inherent in the manufacturing process, these materials can be made to any required length, with the sheet materials being available on endless bobbins, effectively eliminating the need for joints.\textsuperscript{7}

Fabric reinforcement for FRPs comes in a multitude of weave, fiber orientation and weight combinations. In many cases, the fabrics are composed of unidirectional fibers with lines of stitching running perpendicular to the fiber direction simply to hold the material together. A combination of orientations can also be manufactured simply by aligning successive layers
of unidirectional fabrics at varying orientations and then stitching these layers together. Originally, woven fabrics were used but later rejected since the changes in fiber direction as they pass around each other can lead to reinforcement inefficiencies and stress concentrations.

Application of a fabric reinforced FRP generally consists of these basic steps:  

- preparing concrete surface (cleaning, sealing cracks, rust proofing existing steel reinforcement, smoothing, etc.),
- applying primer coat or coupling agent,
- applying resin undercoat,
- placing fabric sheet,
- applying resin top coat and working the resin into the fabric and
- applying finish coat or protective layer.

The laminated FRP plates or pre-impregnated sheets, on the other hand, require only that the concrete surface be prepared (as above) and a layer of epoxy adhesive spread over the surface onto which the FRP is placed. However, in order to optimize the composite action between the FRP and concrete, the FRP surface must first ground to remove the outermost matrix-rich layer and expose the fibers. Prior to placement, the bonding surface of the FRP must be carefully cleaned with a solvent.
2.3 - Previous Research

As previously mentioned, FRPs were first used as external reinforcement to strengthen reinforced concrete beams at the Swiss Federal Laboratories for Materials Testing and Research (EMPA). This groundbreaking research was conducted by U. Meier et al. and examined the effectiveness of epoxy bonding thin CFRP plates to the underside of reinforced concrete beams.

Meier et al. found that FRP strengthening of short, under-reinforced RC beams (2 m in length) with CFRP laminates resulted in a significant increase in the ultimate load (nearly 100%) which was accompanied by a corresponding decrease in the deflection at this load (~50%). In the case of a seven metre long RC beam containing properly designed steel reinforcement, the increase in ultimate load was 22%. The reduction in beam deflection at failure was a direct result of the CFRP’s higher stiffness compared to steel.

Since these beams were under-reinforced in flexure, the failure mode they exhibited was tensile fracture of the CFRP in the tension zone at the bottom of the member. Meier described the failure process of these beams as follows:

"After the appearance of the first cracks in the concrete, the internal steel reinforcement and the external CFRP laminate are carrying the tensile stresses. A soon as the internal steel bars reach yielding, only the CFRP laminate contributes to an additional increase in the load. Finally, the laminate fails in a brittle manner (tensile failure). ....there is less rotation of the strengthened beams but it is still sufficient to predict impending failure."
not always produce the same results due to differences in beam reinforcement. The failure modes that were encountered by Meier et al include: $^3,7,17,34,35$

- Tensile failure of the CFRP.
- Compressive failure of the concrete in the compression zone of the beam.
- Progressive delamination of the CFRP due to an uneven concrete surface.
- Abrupt delamination at the location of shear cracks in the concrete (see Figure 2.1).
- Interlaminar shear failure within the CFRP sheet.
- Tensile failure of the reinforcing steel in the tension zone.
- Shear failure of the concrete in the shear span.

There were also a number of other failure modes that were hypothesized, though they were not actually encountered, including: $^7,34,35$

- Cohesion failure within the adhesive.
- Adhesion failure at the CFRP/adhesive interface.
- Adhesion failure at the concrete/adhesive interface.

Figure 2.1 illustrates the initiation of delamination that can be caused due to formation of a shear crack in the concrete. This effect became more pronounced as the thickness of the laminate was increased. $^{17}$
Another effect induced by the CFRP laminate involves the pattern of flexural cracking exhibited by the beams. It was discovered that the cracks became more evenly distributed and that the total crack width was reduced. Though shear cracking tended to initiate delamination (as described above), flexural cracks were spanned by the CFRP sheet and did not influence loading capacity. This trait bodes well for the use of such strengthening techniques for the repair of damaged members.

Meier et al also examined the economic factors involved in using FRP strengthening to replace steel plates. At the time (1991), carbon fiber reinforced polymers were estimated to be about 50 times more expensive, per kilogram, than steel plate. However, the
difference in mass needed to carry out a rehabilitation project can quickly make up for a great deal of this difference. For example, the repair of the Ibach bridge in Lucerne, Switzerland would have required 175 kg of steel plating. When the bridge was actually retrofitted, this steel was replaced with only 6.2 kg of CFRP. 

Moreover, it must also be considered that in this type of construction, the cost of materials is typically only 20% of the total cost while the costs associated with labor amount to the remaining 80%. In the Ibach bridge upgrade, the much lighter pieces of material allowed the work to be performed from a mobile platform, as opposed to the heavier scaffolding that would have been necessary for steel plate jacketing. As a more specific example, a 94 kg steel plate that would have been required in the project was replaced by a CFRP laminate weighing only 4.5 kg. Such weight reductions would lead to very large savings in labor costs.

Additionally, when considering the price of CFRP, nearly two thirds is governed by the price of the fibers themselves, with the rest controlled by the cost of the matrix, processing and overhead. Currently, the price of reinforcing fibers is relatively high, though it should drop as such large scale usage becomes more prevalent.

Chajes et al investigated the FRP bonding technique for use in a combination of flexural and shear strengthening. The beams in this study were intentionally designed so as to be under-reinforced and fail in flexure. CFRP sheets were then incrementally added to the bottom (flexural reinforcement) and sides (shear reinforcement). The reinforcing material
used in this case was a 0.11 mm thick unidirectional fiber CFRP tow-sheet.

Applying a single sheet to the bottom face of the beam resulted in a significant increase in ultimate load of nearly 160%, though it also resulted in a change in failure mode from concrete crushing in the compression zone to shear. A second sheet was then added in the transverse direction, with the fibers running at 90° to the beam axis. This sheet was also continued up both sides of the member to provide shear reinforcement. Though the increase in load carrying ability was only slightly higher (a 175% improvement) due to the extra transverse layer, it did manage to prevent a shear failure. These specimens failed due to tensile fracture of the CFRP in the tension zone.

Further studies by Chajes et al.\textsuperscript{38} examined the effect of different fiber types on shear strengthening of reinforced concrete beams. Three fiber types were investigated, including glass, carbon and aramid. Bi-directional fiber fabrics were applied to the bottom and sides of the beams and oriented with the fiber directions at 0/90°. The ultimate strength was increased by 80, 85 and 88%, respectively, by the aramid, glass and carbon fiber fabrics. A further pair of beams was retrofitted with the carbon fiber fabric, this time oriented at ±45°, which produced a 122% increase in peak load. This study again indicated that an applied FRP material can act as shear, as well as flexural, reinforcement but also showed that a 45° fiber orientation is more effective for such strengthening.

The authors identified a number of drawbacks related to FRP plate bonding, including the necessity of extensive surface preparation to provide a flat surface for bonding, increased
costs related to the production of large FRP plates and difficulties in achieving a sufficient
bond between the concrete and FRP. Continuous fiber fabrics, on the other hand, can
conform to minor irregularities in the surface, are available in rolls of virtually any length
and tend to develop much stronger bonds with the concrete. ¹ This improved bonding ability
is thought to be related to the fact that the adhesive actually penetrates into the reinforcement
fabric which it cannot do with pre-made FRP plates.

Triantafillou et al conducted a great deal of research into a number of facets related to
reinforced concrete beams strengthened with FRPs.¹¹,¹⁸,¹⁹,²¹,²²,³⁹,⁴¹ They looked at both shear
and flexural strengthening, used FRPs to prestress concrete beams, and made attempts to
model the various failure mechanisms.

These authors first identified a number of failure mechanisms associated with such FRP
retrofit techniques, including:

- reinforcement yielding followed by FRP fracture.
- reinforcement yielding followed by concrete crushing.
- concrete crushing.
- debonding.¹¹,¹⁹

After further investigation into each of these major failure mechanisms, expressions were
derived to predict the ultimate bending moments for each case.¹⁹,²¹ The equations related to
the first three of these failure modes are:
Reinforcement yielding followed by FRP fracture.

\[
\frac{M_u}{bd^2f'_c} = \frac{f_y}{f'_c} \rho_s \left[ 1 - \frac{\bar{y}}{d} \right] + \frac{E_{fc}e_{fc}^*}{f'_c} \rho_{fc} \left[ \frac{h}{d} - \frac{\bar{y}}{d} \right]
\]

Reinforcement yielding followed by concrete crushing.

\[
\frac{M_u}{bd^2f'_c} = 0.85\beta_1 \frac{c}{d} \left[ \frac{h}{d} - \frac{\beta_1 c}{2d} \right] - \frac{f_y}{f'_c} \rho_s \left[ \frac{h}{d} - 1 \right]
\]

Concrete crushing.

\[
\frac{M_u}{bd^2f'_c} = 0.7225 \frac{c}{d} \left[ \frac{h}{d} - 0.425 \frac{c}{d} \right] - 0.003 \frac{1 - \frac{c}{d}}{\frac{c}{d}} \frac{E_s}{f'_c} \rho_s \left[ \frac{h}{d} - 1 \right]
\]

where:

- \( M_u \) = ultimate bending moment
- \( b \) = beam width
- \( h \) = beam depth
- \( d \) = beam effective depth
- \( c \) = beam neutral axis depth
- \( \bar{y} \) = distance from centroid of concrete stress distribution to top fiber
- \( f'_c \) = concrete compressive strength
- \( f_y \) = steel yield stress
- \( \rho_s = A_s/bd \) = area fraction of steel reinforcement
- \( \rho_{fc} = t/d \) = area fraction of fiber composite
- \( E_{fc} \) = FRP plate Young's modulus
- \( e_{fc}^* \) = FRP plate tensile failure strain
- \( E_s \) = steel Young’s modulus

The authors also investigated the debonding failure mode, concluding that debonding occurs due to the catastrophic propagation of a crack along the concrete/FRP interface. Such cracks could be induced by a number of factors, including inconsistencies in the distribution of the adhesive, cracking of the concrete, peeling-off of the composite from concrete faces that are
not perfectly flat and fatigue loading.\textsuperscript{19}

Since the authors considered only FRP applied to the bottom face of the beam, and this FRP is loaded in tension, they could assume that the bonding agent is primarily subjected to shear stresses. Thus, the adhesive must possess sufficient shear strength to provide the necessary bond between the concrete and FRP. It was then concluded that propagation of the interfacial crack would resemble a mode II fracture.\textsuperscript{11}

They went on to model the debonding failure mechanism and derive an expression to predict the ultimate moment capacity of a beam which fails due to debonding. Further research suggested that such a failure mode could be avoided altogether by providing a clamping mechanism at either end of the laminate.\textsuperscript{21}

Another study by Triantafillou \textit{et al} investigated the variability inherent in the design variables associated with CFRP strengthening of reinforced concrete beams.\textsuperscript{21,39} The variables considered in this study included geometric parameters of concrete (such as beam width, depth and reinforcement location) as well as strength and elasticity values of the materials involved. It was found that the flexural strength of a retrofitted beam was influenced the most by variations in concrete strength and CFRP failure strain and area fraction.

Saadatmanesh \textit{et al} also performed a great deal of research into the strengthening of reinforced concrete beams with FRPs. Their research investigated a number of aspects,
including shear strengthening, flexural strengthening, analytical and predictive approaches and the properties of various FRP materials themselves. The majority of results reported by the authors coincide with previously discussed research, with the exception of the modeling techniques employed. The authors used a finite element approach to predict the effect of the FRP and were quite successful in simulating load-deflection curves similar to those reported in experimental studies.

Probably the most concise attempt to develop a design procedure for FRP retrofit of reinforced concrete beams is now in the process of final revisions of ISIS, which is working on a design manual which will provide designers of rehabilitation projects with useful guidelines and procedures to effectively and safely carry out their work. This design manual applies not only to beam retrofit but to column strengthening as well. It also goes well beyond the actual design of FRP retrofits by discussing the evaluation of existing structures, the proper usage of FRP materials (including their specification, handling, storage and installation) and methods for ensuring that the entire rehabilitation project work is carried out properly. This document is till in the draft stages, though a final version is expected soon.

A very important issue that arises when dealing with the use of FRPs in this manner is the performance of the adhesive bond between the FRPs and the concrete. A number of researchers have studied the characteristics of this bond. Many of the test methods and theories developed by this work will prove helpful in future attempts to investigate the bond created by the sprayed FRP technique. However, there are a number of significant physical
differences between the bonds created by fabric or FRP plate bonding and those inherent in the sprayed FRP approach.

The application of FRP fabrics or plates typically involve an adhesive layer which is used to bond the fabric or plate to the concrete surface. Since this adhesive is applied separately from the FRP itself it essentially creates two bonds; one between the adhesive layer and the concrete and the second between the adhesive and the applied FRP. Achieving a good bond between the adhesive and the FRP is relatively easy due to the compatibility between the adhesive and the FRP matrix resin. It is the bond between the adhesive and the concrete which is the more difficult case and thus of utmost importance. With the sprayed FRP technique, the adhesive and the matrix resin are the same entity, resulting in a single bond which leads to a number of difficulties from a testing standpoint.

For the former case of bonding the fabrics or FRP plates to concrete, testing the adhesive’s bonding ability with concrete is a relatively simple matter. Typically, the adhesive is used to bond a steel plate or frame to the concrete and then apply load to the steel component to induce bond failure. Other researchers have used actual FRP plates bonded to the surface, applying the load to the FRP plate itself. In both approaches, the advantage is in the use of a loading apparatus with a precisely defined shape and size (the steel or FRP plate).

In the sprayed FRP technique, the finished FRP plate is not as well defined geometrically. The thickness is difficult to control to the exacting tolerances that would be needed to endure
proper stress transfer to the bond during loading. Even worse, it is virtually impossible to control the contact area between the sprayed FRP and the concrete due to overspray beyond the desired limits. Though this can be alleviated by masking the surface around the desired contact area, it still leaves the problem of trying to get an even stress transfer from the loading apparatus to the bond through a material where geometric uniformity is difficult to achieve.

To make matters more difficult, this stress transfer mechanism is also affected by the nature of the fiber reinforcement, which is discontinuous and randomly oriented. As a result, it is necessary to include the FRP in the test in order to get useful results so simply testing the resins ability to bond to concrete is insufficient.

There has also been a number of attempts at analyzing the mechanisms involved in bond failures. These studies have indicated two bond failure modes that typically occur; peeling failure\textsuperscript{17,61} and shear failure.\textsuperscript{52} Karbhari\textsuperscript{62} suggests that the peel test is a more representative measure of the actual bond strength as it relates to FRP retrofits in the field since this is the most common mode of debonding failure encountered. In general, the issues of bond strength and bond failure have been investigated but the consensus opinion appears to indicate that there is a very large variety of adhesive materials with an even larger variation in bond properties. This variability is further complicated by the influence of concrete surface condition preparation.
Karbhari et al. went on to examine the effect of environmental exposure on the FRP-concrete bond strength. Exposure conditions included in his work included immersion in fresh water, immersion in sea water, exposure to freezing temperatures and exposure to freeze-thaw cycling. In general, all exposure conditions had detrimental effects on the bond strength, though the magnitude of the deterioration varied widely depending on the adhesive as well as the FRP material involved. It was noticed that carbon fiber based systems were less susceptible to environmental exposure, thus contributing to the resistance of these materials to debonding.

Another investigation into the effect of environmental exposure on FRP retrofit performance and on the FRP-concrete bond was conducted by Homam et al. This study included exposure to alkaline solutions, high temperature cycling and freeze-thaw cycling. Their results indicated that both the CFRP and GFRP themselves are well suited to resist these environmental conditions but that the FRP-concrete bond was adversely affected by high alkalinity and both temperature cycling ranges for both materials.

Further durability studies can be divided into two major groups; those that examine the durability of the FRP material and its components and those that investigate the effect of environmental exposure on the performance of an FRP retrofitted member. In the former group, it quickly becomes obvious that carbon fiber is the reinforcement of choice since glass fiber is particularly susceptible to alkaline environments and UV radiation. Regarding the matrix resins, it appears that epoxies are the most durable, followed by polyesters and vinyl esters.
Research into the effect of long-term environmental exposure on the performance of retrofitted structural members reveals an extremely important phenomenon. The durability of the FRP materials themselves quickly becomes a secondary issue since, in virtually every combination of FRP and adhesive, the FRP-concrete bond degrades far faster, resulting in a debonding failure.\textsuperscript{69-73}

A new field of research beginning to develop in this area is the use of non-destructive testing methods for determining the presence and extent of damage to the FRP retrofit materials and, more applicably, to the FRP-concrete bond.\textsuperscript{74,75} Such test methods are particularly useful for detecting delaminations, whether they occur within FRP material itself or at the interface between the FRP and concrete.

In addition to the researchers mentioned here, there are many others who have investigated the use of FRPs for repairing and strengthening reinforced concrete beams. The results of these numerous research projects all point to the same conclusion - the use of fiber reinforced polymers to strengthen reinforced concrete beams is an effective, economical and very promising technique. Further discussion of these other investigations will be curtailed due to the repetitive nature of the findings and due to the fact that many are aimed at very specific issues that do not directly apply to the sprayed FRP material used in this project.
3.1 - Concrete Materials

The following section consists of a description of the general properties and characteristics related to the various components used to make the concrete used throughout this research project.

3.1.1 - Portland Cement

There were two brands of cement used in the concrete cast for this project. The first brand was manufactured by Lafarge Canada Inc., the second by Tilbury Cement Ltd. Both were classified as Type 10 Normal Portland Cement. During the batching and mixing operation, care was taken to ensure only fresh cement was used in order to avoid possible detrimental effects or inconsistencies in test results associated with the use of partially hydrated cement.
3.1.2 - Water

All mixing water used in the concrete was taken directly from the City of Vancouver drinking water supply. This water source is considered acceptable for use in concrete.

3.1.3 - Aggregates

The coarse aggregate used in the concrete for this research project was a combination of 12.5 and 22 mm torpedo gravels obtained from the Lafarge Canada Inc. Vancouver ready mix plant. These two gravels were combined in equal proportions to make up the total coarse aggregate content. This aggregate was found to have a relative density 2.71, an SSD absorption value of 1.24% and a dry rodded density of 1550 kg/m³.

The fine aggregate consisted of Lafarge Concrete Sand also obtained from the Lafarge Canada Inc. Vancouver ready mix plant. This sand was found to have a relative density of 2.70, an SSD absorption value of 1.00% and a fineness modulus of 2.35.

3.1.4 - Admixtures

A high range water reducing admixture was required during beam casting due to the limited spacing between reinforcing bars and between the outside reinforcing bars and mold walls. The water reducer used in this project was WRDA-19, manufactured by W.R. Grace & Co. Addition rates varied depending upon mix composition. These rates are discussed later in
Section 3.2 along with mix proportions.

3.2 - Mix Proportioning

The were two different mix designs used within this research project, one for normal strength concrete (~45 MPa) and another for high strength concrete (~80 MPa). The majority of specimens were cast using the former design, with a single group of specimens cast using the high strength mix (see Section 6.1.5.2.2.). The proportions of constituent materials for the normal strength mix are provided in Table 3.1 while those for the high strength are shown in Table 3.2.

<table>
<thead>
<tr>
<th>Component</th>
<th>Ratios</th>
<th>kg per m$^3$ of Concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td>Water</td>
<td>0.50</td>
<td>175</td>
</tr>
<tr>
<td>Cement</td>
<td>1.00</td>
<td>350</td>
</tr>
<tr>
<td>Fine Aggregate</td>
<td>2.00</td>
<td>700</td>
</tr>
<tr>
<td>Coarse Aggregate</td>
<td>3.50</td>
<td>1225</td>
</tr>
<tr>
<td>Water Reducer</td>
<td>Dosage Rate = 15 ml/kg CM</td>
<td></td>
</tr>
</tbody>
</table>

A fine aggregate proportion of 36% by mass of the total aggregate content was used in the normal strength concrete, though this proportion was increased to 60% in the high strength mix. The addition of silica fume to the high strength mix actually resulted in a reduction in water reducer demand, even though the water to cementitious materials (W/CM) ratio was reduced from 0.50 to 0.33.
<table>
<thead>
<tr>
<th>Component</th>
<th>Ratios</th>
<th>kg per m³ of Concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td>Water</td>
<td>0.33</td>
<td>202</td>
</tr>
<tr>
<td>Cement</td>
<td>0.86</td>
<td>526</td>
</tr>
<tr>
<td>Silica Fume</td>
<td>0.14</td>
<td>86</td>
</tr>
<tr>
<td>Fine Aggregate</td>
<td>1.57</td>
<td>961</td>
</tr>
<tr>
<td>Coarse Aggregate</td>
<td>1.04</td>
<td>636</td>
</tr>
<tr>
<td>Water Reducer</td>
<td></td>
<td>Dosage Rate = 12 ml/kg CM</td>
</tr>
</tbody>
</table>

3.3 - GFRP Materials

The following section is a compilation of the general descriptions and characteristics of the various materials used to make the glass fiber reinforced plastics for this research project. Materials used in both the spraying technique and the wrapping procedure are included.

3.3.1 - Resin

Though there were actually three different resins used within this project, only two of them will be discussed in this section. Since the third resin is part of the fully comprehensive MBrace® retrofit system, it will be covered in Section 3.3.6., which deals with that system.

The primary resin used throughout the research project, regardless of spraying location, was the K-1907 polyester resin manufactured by Ashland Chemical Canada Ltd. Physical properties for a clear casting fabricated from this resin, as per the manufacturer’s literature, are listed below in Table 3.3.
Table 3.3: Properties of K-1907 polyester resin.

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density</td>
<td>1.07</td>
<td>g/cm³</td>
</tr>
<tr>
<td>Tensile Strength</td>
<td>75.8</td>
<td>MPa</td>
</tr>
<tr>
<td>Elastic Modulus</td>
<td>3.77</td>
<td>GPa</td>
</tr>
<tr>
<td>Shear Strength</td>
<td>48</td>
<td>MPa</td>
</tr>
<tr>
<td>Elongation @ Failure</td>
<td>2.4</td>
<td>%</td>
</tr>
</tbody>
</table>

It must be noted that these properties refer to a clear casting, as opposed to a sprayed material. Actual laboratory testing of specimens fabricated from a sheet of clear sprayed resin varied significantly. Refer to the material properties testing chapter (Section 5.2) for further details.

The secondary resin used in the initial phases of the project, in which the beams were sprayed elsewhere was a polyurethane resin, also manufactured by Ashland Chemical Canada Ltd. Both of these resins were supplied in a fully promoted form to simplify application.

3.3.2 - Catalyst

The catalyst which was used to initiate curing of the resin was Methyl Ethyl Ketone Peroxide (MEKP), again manufactured by Ashland Chemical Canada Ltd. under the product name Lupersol DDM-9. The typical addition rate used throughout the project was 3% by volume.
This resulted in a gel time of approximately 15 minutes at 20°C, sufficient to allow compaction of the sprayed material but quick enough so that the specimens could be moved within an hour of spraying without fear of damaging the GFRP coating.

During warm dry weather the spraying operation was performed outdoors, often under significantly higher temperatures. In such case, the catalyst content was reduced to 2% or even 1%, depending on conditions. Generally, a 15 minute gel time was targeted.

3.3.3 - Coupling Agent

The coupling agent used to improve the GFRP to concrete bond was Derakane® 8084, a vinyl ester resin manufactured by The Dow Chemical Company and supplied in a fully promoted form. Original implementation of this resin as a coupling agent was done on the advice of the supplier, Ashland Chemical Canada Ltd., when consulted as to potential methods of improving the FRP to concrete bond. Though data is not available with respect to the actual adhesion strength of the resin to concrete, the supplier did point out that this is a common use for the Derakane® 8084 resin and that it would indeed be the most suitable resin for our purpose. Selected physical and mechanical properties of Derakane® 8084 are provided in Table 3.4.

3.3.4 - Glass Fiber Rovings

The glass fiber used in the GFRP spraying process was Advantex® 360RR chopper roving
manufactured by Owens Corning. Advantex® is an improved form of E-glass which combines the mechanical properties of E-glass with the corrosion resistance of E-CR glass. The *roving* format refers to a number of continuous glass filaments which are gathered together into a single bundle or yarn, without introduction of a mechanical twist. These rovings are then wound and packaged in a tubeless configuration specifically designed for use with the chopper gun application technique used here. Physical properties, as per the manufacturer's literature, are listed in Table 3.5.

**Table 3.4**: Properties of Derakane 8084® vinyl ester resin.

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density</td>
<td>1.15</td>
<td>g/cm³</td>
</tr>
<tr>
<td>Tensile Strength</td>
<td>72</td>
<td>MPa</td>
</tr>
<tr>
<td>Elastic Modulus</td>
<td>4.6</td>
<td>GPa</td>
</tr>
<tr>
<td>Elongation @ Failure</td>
<td>10.0</td>
<td>%</td>
</tr>
<tr>
<td>Adhesive Strength to Metals</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Carbon Steel</td>
<td>1430</td>
<td>psi</td>
</tr>
<tr>
<td>304 Stainless Steel</td>
<td>1530</td>
<td>psi</td>
</tr>
<tr>
<td>2024T3 Aluminum</td>
<td>970</td>
<td>psi</td>
</tr>
</tbody>
</table>

**Table 3.5**: Properties of Advantex® glass fiber.

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density</td>
<td>2.62</td>
<td>g/cm³</td>
</tr>
<tr>
<td>Diameter</td>
<td>11</td>
<td>μm</td>
</tr>
<tr>
<td>Tensile Strength</td>
<td>3100-3800</td>
<td>MPa</td>
</tr>
<tr>
<td>Elastic Modulus</td>
<td>80-81</td>
<td>GPa</td>
</tr>
<tr>
<td>Elongation @ Break</td>
<td>4.6</td>
<td>%</td>
</tr>
</tbody>
</table>
3.3.5 - Glass Fiber Fabrics

Though there were four different glass fiber fabrics used in this project, only two will be discussed in this section. The other two are part of the MBrace® retrofit system and will be included in the section dealing solely with that system (Section 4.3.6). The two which will be covered are the NEWF 230 and NEMP 240 fabrics manufactured and supplied by Brunswick Technologies Inc. These particular products were chosen for strengthening laboratory scale specimens because they are composed of the exact same glass fiber used in the spraying operation (Advantex®).

Both fabrics are knitted, as opposed to woven, which means that two planes of glass fiber rovings running in orthogonal directions are laid one on top of the other and then stitched together. According to the manufacturer, weaving induces stress concentrations at the crossover points due to the necessary change in direction of the fibers as they pass around those running in the other direction. Knitting allows all of the fibers to run in a straight line, with the stitching thread holding the two orthogonal planes together.

The nomenclature used for the two fabrics (NEWF 230 and NEMP 240) also identifies the arrangement and weight of the material. The first letter identifies the fabric type, with N representing a knitted fabric. The second letter identifies the fiber type, with E being E-glass. The final two letters indicate the fiber directions, where:
M = minus axis layer (-45°)
P = plus axis layer (+45°)
W = warp axis layer (-0°)
F = fill axis layer (-90°)

The number following the letter combination identifies the weight of the fabric in ounces (and tenths) per square yard. Thus, the NEWF 230 would be a 23.0 oz/yd² knitted E-glass fabric with fibers running at 0° and 90°. NEMP 240 would be a 24.0 oz/yd² knitted E-glass fabric with fibers running at ±45°.

3.3.6 - MBrace® System

The MBrace® Composite Strengthening System is an all-inclusive retrofit process produced by Master Builders Technologies, Inc. The system includes a coupling agent (MB Primer), an epoxy matrix resin (MB Saturant) and a 27 oz/yd² unidirectional E-glass sheet (EG 900). Physical and mechanical properties of the MB Saturant epoxy resin, as per the manufacturer’s literature, are listed in Table 3.6.

| Table 3.6: Properties of MB Saturant® epoxy resin. |
|-----------------|---------|---------|
| Property        | Value   | Unit    |
| Density         | 0.98    | g/cm³   |
| Tensile Strength| 54.0    | MPa     |
| Elastic Modulus | 3.03    | GPa     |
| Elongation @ Failure | 2.5 | %       |
3.3.7 - Solvent

An effective solvent is always useful for cleaning purposes when working with resins, but in the GFRP spraying operation it is an absolute necessity. The spraying equipment requires periodic flushing to prevent resin from hardening within the nozzle of the gun. The solvent used throughout this project was acetone, supplied by Ashland Chemical Canada Ltd.
4.1 - Introduction

There are a variety of techniques available for fabricating fiber reinforced polymer components or, in this case, applying an FRP coating to an existing object. Some of the most common include hand lay-up, filament winding, pultrusion and spraying. Hand lay-up, filament winding and spraying can be applied directly to the surface being strengthened while the pultrusion method is normally used to produce FRP plates which are later bonded to the surface with some form of adhesive.

The sprayed FRP system used throughout this project is not a new technology. It has been used for years in the boat building and automotive industries, as well as for many other more specialized applications. It is the application of this technology to the structural retrofit of reinforced concrete which is unique to this research. Due to the novel nature of this technique, a brief description of the equipment, its operation and the resulting FRP follows.
4.2 - Equipment

There are a number of different manufacturers of FRP spraying equipment. Though there are slight variations between these types of equipment, they all possess the same general components. The equipment used throughout this research project was a Venus-Gusmer H.I.S. Chopper Unit equipped with a Pro Gun spray gun.

Figure 4.1: GFRP spraying equipment.

Figure 4.1 is a photograph of a typical setup for this particular piece of spraying system. The entire apparatus is completely self-contained and mounted on a portable cart for easier on-site operation. There are three basic components; the resin pump (R), the catalyst pump (C) and the spray or chopper gun unit (G). The only external equipment needed is a compressed air source (0.5 m³/min minimum). All three of the major components are completely driven by the compressed air supply. No electrical power supply is required unless the optional resin
heater is added for use in cooler weather (recommended below 16°C).

One unique feature of this particular piece of apparatus is the configuration of the two pumps (catalyst and resin). The catalyst pump is actually mounted on the side of the resin pump using a *slave arm* arrangement, as depicted in Figure 4.2.

![Figure 4.2: GFRP spraying equipment - slave arm arrangement.](image)

The slave arm apparatus is made up of three parts; the upper and lower rails and the vertical strut which connects the two together. The lower rail of the slave arm is fixed in a horizontal position as shown in the figure. The outer end of the upper rail is connected to the lower rail via the vertical strut while the inner end is attached to the piston of the resin pump. As the resin pump cycles, the inner end of the upper rail moves up and down with it, thus driving the catalyst pump.
There are two distinct advantages to this arrangement. First, it eliminates the need for a separate air supply feed to the catalyst pump. Second, it provides a simple method for adjusting the catalyst proportion of the final resin/catalyst mixture. The distance between the upper and lower rails decreases as the catalyst pump is moved outward. This shortens the stroke of the pump and reduces the amount of catalyst being supplied to the spray gun.

Since the amount of resin is unaffected, the proportion of catalyst in the final mixture is controlled by the position of the catalyst pump. With this particular piece of equipment, the catalyst content can be varied from a minimum of 0.75% to a maximum of 3.0% (±0.1%). The proportion of catalyst in the final product directly affects the curing time required for the composite to harden. Since curing time is also affected by temperature, the catalyst content can be varied to offset this effect.

Since the resin and catalyst pumps have separate outlets, the two components are transported to and fed into the spray gun separately. Figure 4.3 consists of a photo of the spray gun. The resin (R) and catalyst (C) lines are labelled in the photo, as are the spraying gun (S) and the chopper unit (Ch) mounted on top of the gun. There are also inlets for air, which is needed to power the chopper unit and assist trigger operation and solvent, which is used to flush the resin and catalyst from the gun between uses. It should be noted that the resin and catalyst actually pass through the gun block separately and do not actually come into contact until they reach the mixing nozzle at the front of the gun (not shown in photo). Here they are forced through a turbulent mixer immediately before exiting the gun as a mixture.
The primary advantage of this arrangement is ease of cleaning. Since the resin and catalyst only come into contact with each other within the spray nozzle, this is the only part that needs to be removed and cleaned during shutdown. The equipment does not require disassembly or flushing unless it is expected to remain idle for extended periods.

The glass fiber is supplied in a roving format, which is simply a large number of fibers bundled together and wound onto a spool. One or two rovings (depending upon the desired fiber content range) are fed into the chopper unit, where they pass between a pair of rollers. One of the rollers has a number of blades embedded in its surface at evenly spaced intervals around its circumference. These blades break the glass fiber into shorter lengths, with that length dependent entirely upon the number and spacing of blades on the wheel, allowing production of fibers of consistent length, adjustable from 8 to 48 mm.
The chopped fibers are forced out the front of the chopper unit by a combination of their own momentum (as they are pulled through by the rollers) and air flow (since air is also fed into the chopper housing) from the rear. The air motor which powers the rollers has an adjustable air feed which allows the operator to control the speed at which the rollers spin. Combining this adjustment option with the choice between single or dual roving feeds provides a large range in the amount of chopped fiber being produced, thus affecting the overall fiber volume fraction of the finished composite.

4.3 - General Operation

Operation of the GFRP spraying equipment is actually quite simple. The only difficult part of the process is being able to judge the thickness of the material that has been applied since there is no effective way to actually measure it during application. Also, since the rate of increase in thickness depends on how fast the operator moves the gun and how much each pass overlaps the previous one, it will be entirely operator dependent. In other words, being able to determine the thickness applied during spraying will take practice and experience until the operator develops a feel for it using their own technique.

In operation, the gun sprays the resin/catalyst mixture from the lower spray nozzle while, at the same time, spraying the fibers from the top-mounted chopper unit. These two streams or fans combine and continue on to the spraying surface together (Figure 4.4). The result is a two dimensional random distribution of fibers encapsulated by a fully catalysed resin. This approach allows the operator to build up the GFRP material to whatever thickness is
required. After spraying, a ridged aluminum compaction roller is used to force out any entrapped air voids and to work the material into a consistent thickness.

![GFRP spraying operation](image)

**Figure 4.4:** GFRP spraying operation.

One of the difficulties that may arise during application is the need to apply the FRP around a corner. Though this procedure is much easier with the sprayed FRP technique than with conventional fabric wrapping procedures, it still poses a problem since the fibers tend to straighten again after being rolled around the corner. This problem is discussed further in Section 6.1.3 which details preparation of the laboratory test specimens.

Some precautions that need to be taken include assuring that the air supply is free of moisture and that the sprayed material remains dry until it has completely cured. Overspray
can also be a problem due to the width of the spray fan and proper masking techniques should be used. Finally, as with any operation involving potentially hazardous materials, proper safety procedures and equipment must be utilized.
5.1 - Introduction

In order to properly characterize the effect of using sprayed glass fiber reinforced polymers for retrofitting existing concrete beams, it is first necessary to define their material properties. This chapter deals with a number of experimental investigations aimed at determining the relevant physical and mechanical properties of this type of composite.

5.2 - Tensile Properties

As an external retrofit material applied in a thin skin to the concrete surface, the sprayed GFRP will be primarily used to carry tensile stresses, in addition to some shear. Whether these materials are placed on the bottom face of the beam to assist in flexure or applied to the side to reinforce the beam in shear, the performance of the GFRP itself will be governed by its tensile properties. Thus, it is imperative that the tensile properties of these materials are
known. The first step is to experimentally measure the tensile strength of the sprayed GFRP. To this end, a series of tensile strength coupons was fabricated.

5.2.1 - Test Procedure

ASTM does not publish a specific test method which they deem to be applicable to this particular type of material. However, there are two standards which are close. The first of these is D628 *Tensile Properties of Plastics*, which states (Note 4) that composites containing discontinuous fibers with high moduli (E > 20 GPa) are to be tested under Test Method D3039 *Tensile Properties of Fibre-Resin Composites*. The latter test method, however, states in its Scope section that it applies only to unidirectional composites or laminates of symmetric, orthotropic construction. Thus, neither standard applies to this particular material.

The majority of test results reported by the manufacturers of the materials used refer to the ASTM D628 method. Thus, the tensile specimens in this research program were tested in general accordance with that standard, the single variation from the prescribed procedure being specimen size. Since the specimen size dictated by the ASTM standard was considered to be too small to accurately predict the tensile strength of composites containing the longer fiber lengths, the specimens were scaled upward to a more applicable size. Figure 5.1 details the dimensions of the actual specimens used.

The problem inherent in the original ASTM D638 specimen size involves the specified width
of the narrow section, which is stipulated as 10 mm. Since the tensile strength coupons to be tested contained fibers up to 48 mm in length, a 10 mm width would result in any fibers oriented at a significant angle being discounted from the overall composite strength. Thus, the specimen was scaled upward to make this narrow section 60 mm in width.

![Diagram of GFRP tensile strength specimen dimensions.](image)

**Figure 5.1:** GFRP tensile strength specimen dimensions.

Specimen thickness was not defined prior to specimen fabrication. Rather, the sprayed FRP plates from which the specimens were to be cut were built up to any arbitrary thickness. Specimen thickness was then measured during testing, with the actual measured value being used in the determination of FRP strength.

Fabrication of the tensile strength coupons involved spraying a flat sheet or plate of GFRP material from which the coupons could later be cut. This was accomplished by spraying the glass fiber and resin onto a pane of glass (Figure 5.2), which was first covered with a thin sheet of plastic. The plastic was used to prevent the sprayed resin from bonding to the glass.
surface. After hardening, the GFRP plate could be removed from the glass and the plastic sheet peeled off the back. The tensile strength coupons were then cut out and machined to the proper tolerances.

The top surface of the composite plate was then ground in order to remove the outermost resin layer. This step has no significant effect on the load carried by the specimen but will affect the measurement of cross-sectional area used in strength calculations. The removed layer consists solely of resin which is forced to the surface during the rolling or compaction process. Since the thickness of this resin layer (0.5 - 1.0 mm) is relatively constant regardless of overall composite thickness, it would act to distort calculated strength values by making overall thickness proportionally higher for thinner plates. As a result, it was decided that this layer should be removed prior to testing. Thus, only that portion of the composite containing reinforcing fibers was counted in the determination of tensile strength.

To examine the effect of fiber length on composite tensile properties, a number of different fiber lengths were tested (8, 16, 24, 32 and 48 mm). For each of these lengths, six tensile strength coupons were fabricated and tested. In order to determine whether the distribution of fibers truly promoted planar isotropic behavior in the composite material, these specimens were cut from the GFRP sheet in three different orientations, as shown in Figure 5.3. The specimen numbers indicated in the figure correspond to the specimen ID numbers shown in the individual specimen result table (Appendix A).
Figure 5.2: GFRP tensile strength specimen spraying.

Figure 5.3: GFRP tensile strength specimen fabrication orientation.
In addition, four coupons were fabricated from a sheet of GFRP which had been prepared using the original spraying equipment at GU Manhole Liners Ltd. Due to a limited amount of material, only two orientations were tested for this material (horizontal and vertical).

Testing of the coupons was carried out using a Baldwin 400 kip U.T.M. set to its low load range (16000 lbs. maximum capacity). Friction wedge grips were used to grip the ends of the specimens during testing and care was taken to ensure that the specimen remained completely vertical throughout the test. Elongations were measured using an LVDT mounted on an extensometer, which was then attached to the specimen. Figure 5.4 depicts the testing apparatus setup.

Figure 5.4: GFRP elongation measurement apparatus.
Both applied load and elongation were constantly monitored and recorded using a data acquisition unit and PC. Following completion of testing, the stress-strain data was calculated and plotted. From this information, the ultimate tensile strength and modulus of elasticity was determined for each specimen.

5.2.2 - Results & Discussion

The average tensile properties are shown in Table 5.1 while the average ultimate tensile strengths and moduli of elasticity are depicted graphically in Figure 5.5 for each fiber length. Since specimen orientation had no effect on composite strength, these values represent the average of all specimens tested for each fiber length.

<table>
<thead>
<tr>
<th>Fiber Length (mm)</th>
<th>Matrix Resin</th>
<th>Ultimate Tensile Strength (MPa)</th>
<th>Elastic Modulus (GPa)</th>
<th>Elongation at Failure (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>8</td>
<td>Polyester/Polyurethane</td>
<td>35</td>
<td>5.8</td>
<td>0.66</td>
</tr>
<tr>
<td>None**</td>
<td>Polyester</td>
<td>45</td>
<td>3.7</td>
<td>2.40</td>
</tr>
<tr>
<td>8</td>
<td>Polyester</td>
<td>54</td>
<td>8.1</td>
<td>1.14</td>
</tr>
<tr>
<td>16</td>
<td>Polyester</td>
<td>82</td>
<td>9.6</td>
<td>1.23</td>
</tr>
<tr>
<td>24</td>
<td>Polyester</td>
<td>94</td>
<td>10.4</td>
<td>1.27</td>
</tr>
<tr>
<td>32</td>
<td>Polyester</td>
<td>104</td>
<td>10.5</td>
<td>1.43</td>
</tr>
<tr>
<td>48</td>
<td>Polyester</td>
<td>108</td>
<td>11.8</td>
<td>1.32</td>
</tr>
</tbody>
</table>

* - Fabricated from GFRP prepared by GU Manhole Liners Ltd.
** - Clear casting containing no fiber.
The values for elastic modulus reported in Table 5.1 refer to a secant modulus. This approach was used due to the nonlinear behavior of some of the FRP materials which made determination of the actual initial modulus very difficult. The choice of secant endpoints was based on a similar approach, used for determining the elastic modulus of concrete, which uses endpoints of 50 μstrain and 40% of ultimate stress. Though the same upper limit was used (40% of the ultimate stress), the lower point was raised to 500 μstrain for three reasons. First, the precision of the recorded data made definition of the 50 μstrain level very difficult since this value was very close to the resolution of the equipment used. Second, the typical strains exhibited by the sprayed FRP are much higher than those associated with concrete. Finally, the initial part of the stress-strain curve tends to be very noisy as the loading apparatus and specimen seat themselves. The choice of 500 μstrain as the secant endpoint assured that this region would not be included in the elastic modulus determination.

As indicated by the results, both the composite tensile strength and modulus of elasticity increase with fiber length. The coefficient of variation within each set of specimens (for all three values) tended to decrease as fiber length increased. It is believed that this trend toward improved consistency with increased fiber length is due to the greater ease of compaction associated with longer fiber lengths. Since this phenomenon also affects the density of the material, it will be discussed further in the section on density (Section 5.2).
Figure 5.5: GFRP average tensile properties.

Figure 5.6: GFRP stress-strain response.
Typical stress-strain curves for the sprayed GFRP material are shown in Figure 5.6. For the polyester matrix specimens, it is interesting to note that the stress-strain response became more linear as the fiber length, and thus strength, was increased. This effect can be attributed to better stress transfer between the fibers as the fiber length increases. A decrease in fiber length means that the stresses must be transferred from one fiber to another more often along the length of the composite. Such an increase in the number of stress transfers essentially increases the effect of the matrix on the overall composite strength since it is the matrix that must act as the stress transfer mechanism. It is this increased matrix contribution that induces the increasing non-linearity evident as the fiber length decreases.

5.3 - Density

One physical property of GFRP that becomes important when considering the use of such materials as a retrofit technique is its density. Inherent in the concept of retrofitting an existing structure is the fact that more material, and thus more dead load, is being added to the structure. If this extra material is to be accounted for in design calculations, it is first necessary to know its density.

5.3.1 - Test Procedure

Determination of composite density requires only a small amount of the actual material, typically around 25 cm$^2$ square (25-35 g mass). Additionally, it is not necessary to use a single piece to perform the test. Due to size limitations of the balance used to determine the
specimen masses, it was decided that each sample would be made up of two or three smaller pieces. These pieces were easily obtained from the scraps left over during fabrication of the tensile specimens described above.

The density was determined for each variation in fiber length to determine whether fiber length has any influence on the result. A total of six separate samples were prepared and tested for each such variation. The actual determination of densities was performed in accordance with ASTM D792 Specific Gravity (Relative Density) and Density of Plastics by Displacement.

5.3.2 - Results & Discussion

A full tabulation of results for each specimen tested has been included in Appendix A. Average densities for each fiber length are summarized below in Table 5.2.

The coefficient of variation was below 3.0% for all fiber lengths, indicating very good consistency within each group of results. Examining the polyester matrix composites indicates an upward trend in density as fiber length increases. The variation in density over this range is not statistically significant (COV = 3.0% over entire range) but is intriguing due to the consistency exhibited by the increase. From a material application point of view (i.e. the spraying process itself) the final composite density should be consistent.
<table>
<thead>
<tr>
<th>Fiber Length (mm)</th>
<th>Matrix Resin Type</th>
<th>Density (kg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>8*</td>
<td>Polyester/Polyurethane</td>
<td>1263</td>
</tr>
<tr>
<td>None**</td>
<td>Polyester</td>
<td>1070</td>
</tr>
<tr>
<td>8</td>
<td>Polyester</td>
<td>1289</td>
</tr>
<tr>
<td>16</td>
<td>Polyester</td>
<td>1343</td>
</tr>
<tr>
<td>24</td>
<td>Polyester</td>
<td>1374</td>
</tr>
<tr>
<td>32</td>
<td>Polyester</td>
<td>1382</td>
</tr>
<tr>
<td>48</td>
<td>Polyester</td>
<td>1394</td>
</tr>
</tbody>
</table>

* - Fabricated from GFRP prepared by GU Manhole Liners Ltd.
** - Clear casting containing no fiber.

The respective quantities of fiber and resin deposited on the surface during the spraying operation should be consistent regardless of fiber length. The rate of fiber movement through the chopper unit is constant and will thus produce a constant volume of chopped fiber over a given time period. The quantity of resin sprayed onto the surface is entirely independent of fiber production and is thus unaffected as well.

The only other component of the fabrication procedure which could account for this discrepancy is the rolling or compaction of the deposited material. Throughout the investigation into different fiber lengths, it was noted that the shorter lengths were much harder to roll out since the physical application of the roller tended to move the fibers around a great deal. This would result in fibers being pushed or pulled into piles if too much pressure was applied. Thus, less pressure could be applied to the GFRP during the compaction process. With the longer fibers lengths, however, this problem did not occur.
More pressure could be applied without actually moving the fibers.

It is likely that this difference in compaction efficiency could be responsible for the noted variation in density with respect to fiber length. If so, a similar trend should appear in measurements of fiber volume fraction. The volume proportion occupied by the fiber would be lower in cases where the compaction efficiency was inferior since the non-fiber volume would be higher if air voids remained in the system. This phenomenon is discussed further during the analysis of the FRP mechanical properties Section 5.5.

5.4 - Fiber Volume Fraction

The fiber volume fraction of a composite is typically determined on a mass basis by simply removing the matrix material and measuring the remaining mass of fiber. This mass fraction can then be converted to volume fraction if the densities of the fiber and the overall composite are known.

5.4.1 - Test Procedure

There are two different ASTM Standard Test Methods which can be used to determine the fiber content of polymer matrix composites. ASTM D3171 Fiber Content of Resin-Matrix Composites by Matrix Digestion uses an acidic solution to dissolve the matrix material while ASTM D2584 Ignition Loss of Cured Reinforced Resins uses high temperature to burn off the resin material.
For a couple of reasons, it was decided that the latter test method would be used for the composites utilized in this testing program. Firstly, the glass fiber itself is completely unaffected by the high temperatures required in the test, though it will lose mass when exposed to strong acidic solutions. Secondly, the ignition loss procedure is far safer and easier to perform, with minimal chance of operator error.

5.4.2 - Results & Discussion

There is one important concept to note with respect to the results obtained from this test. Since the polyester resin itself is completely decomposed to volatile materials under the high temperatures involved, no trace of the resin will remain. Thus, the reported fiber proportion of each composite should be accurate. However, the matrix proportion can not truly be determined from these values unless the quantities of any other volatiles present (i.e. water, residual solvent) are small enough to be considered insignificant. Results have been tabulated in Table 5.3.

<table>
<thead>
<tr>
<th>Fiber Length (mm)</th>
<th>Mass Fraction (%)</th>
<th>Volume Fraction (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>8</td>
<td>34.5</td>
<td>17.0</td>
</tr>
<tr>
<td>16</td>
<td>34.3</td>
<td>17.6</td>
</tr>
<tr>
<td>24</td>
<td>34.4</td>
<td>18.1</td>
</tr>
<tr>
<td>32</td>
<td>35.2</td>
<td>18.6</td>
</tr>
<tr>
<td>48</td>
<td>35.8</td>
<td>19.0</td>
</tr>
</tbody>
</table>

Table 5.3: Average fiber content.
As for the original GFRP prepared by GU Manhole Liners Ltd., this test proved to be ineffective at determining fiber volume fraction due to the inclusion of a fire retarding filler during production. This filler prevented the resin from being completely decomposed at the temperatures that could be reached by the equipment used. According to personnel at GU Manhole Liners Ltd., however, external testing performed in the past indicated that the volume fraction of their material is typically in the 8% range.

Similar to the trend with densities, there is also an upward trend in fiber volume fraction as fiber length increases. This is likely a function of compaction as well. Inferior compaction would result in a larger overall volume of material, with the extra volume being included in the non-fiber portion of the composite.

5.5 - Theoretical Analysis

In this section, predictive tools are derived to estimate the tensile strength and elastic modulus of sprayed fiber reinforced polymers based upon the mechanical properties of their constituent materials and the proportions at which they are combined.

5.5.1 - Tensile Strength

According to Barbero\textsuperscript{7}, the tensile strength of a fiber reinforced plastic containing randomly oriented continuous fibers can be determined from the strength parameters of a fictitious unidirectional material made up of the same materials and containing the same volume
fraction of fiber. The first strength value that must be determined is the longitudinal tensile strength of the fictitious unidirectional continuous fiber composite. When such a material is subjected to a tensile load, the most common model used to determine the longitudinal stress carried by the composite is the Rule of Mixtures expression, which states that the total stress is simply a proportional sum of the stress carried by the fibers plus the stress carried by the matrix.

\[ \sigma_1 = \sigma_f V_f + \sigma_m V_m \]

However, this expression only applies to the distribution of stresses while both materials are contributing. Once the maximum load or strain level of either the matrix or fiber is reached, the Rule of Mixtures must be modified to account for the change in load distribution. In the case of a polyester matrix reinforced with glass fibers, the fibers are far stiffer than the matrix material and also more ductile (i.e. they exhibit higher elongation). This combination is referred to as a stiff fiber-brittle matrix system. With such a combination, the matrix material will always fail first as the applied load is increased. Figure 5.7 indicates the relative ideal performances of these two materials.

Once the strain in the composite material reaches the ultimate strain of the polyester resin \((e_{mu})\), the matrix will fail and shed its portion of the load to the fibers. Since the entire applied load must now be transferred to the fibers, the load associated with this point can be determined by calculating the stress carried by the fibers at the same strain level.
For the polyester/glass combination used here, this value becomes

\[ \sigma'_f = E_f \epsilon_{mu} \]

Whether the composite itself fails at this point depends entirely upon whether the fibers are capable of carrying the additional load that is shed by the matrix. At very low fiber volume fractions, the matrix may very well be carrying more load than the fibers are capable of supporting, thus resulting in immediate failure when the matrix reaches its failure strain. In this scenario, the overall strength of the composite will be the ultimate strength of the matrix plus the stress in the fibers at the same applied strain. This value is represented by writing the Rule of Mixtures equation at this specific strain level

\[ F_{tr} = \sigma'_f V_f + \sigma_{mu} V_m \]
At higher volume fractions, the fibers are capable of carrying the additional load transferred to them by the matrix when it fails. In this scenario, the matrix can no longer carry any load and the overall composite strength is equal to the load carrying ability of the fibers themselves.

\[ F_{lt} = \sigma_{fu} V_f \]

Plotting these two governing equations against the fiber volume fraction of the composite results in the graph shown in Figure 5.8.

![Graph](image)

**Figure 5.8:** Variation of FRP tensile strength with fiber volume fraction.

The point where these two lines cross represents the minimum fiber volume fraction beyond which composite failure no longer occurs at the same time as matrix failure. In other words,
volume fractions above this point provide sufficient fibers to carry the load transferred to them when the matrix fails.

The actual value of this minimum volume fraction can be computed as the intersection between the two equations and can be expressed as

\[ V_{f_{\text{min}}} = \frac{\sigma_{mu}}{\sigma_{fu} - \sigma'_f + \sigma_{mu}} \]

For a glass fiber/polyester matrix combination, the minimum fiber volume fraction is about 3%. It should be noted that this value, in fact the entire concept described here, relates to continuous unidirectional fiber systems. In a general sense, randomly oriented or discontinuous fibers will not be governed by the same minimum fiber volume fraction since they require the contribution of the matrix to transfer stresses from one fiber to another. In an ideal continuous unidirectional fiber composite, the load is applied directly to the fibers themselves. A randomly oriented or discontinuous fiber system requires that the matrix transfer the applied load from one fiber to another along the length of the composite.

However, the essence of the model is a relationship between the properties of a randomly oriented fiber material and those of a fictitious unidirectional material made up of the same materials. Since the fiber volume fractions obtained in the spraying operation are typically well above the 3% minimum volume fraction determined for the polyester/glass combination, it will be assumed that the latter of the two equations for longitudinal tensile
stress applies and thus

\[ F_{ts} = \sigma_{tu} V_f \]

Furthermore, for our case the resulting value still needs to be adjusted to take into account the effect of the discontinuity of the fibers used in the sprayed FRP technique. This is typically accomplished through the use of an efficiency factor which relates the effectiveness of the load transfer between the discontinuous fibers with a continuous fiber system. Cox\textsuperscript{78} developed an expression for such an efficiency factor (\( \eta_L \)), which took the form

\[ \eta_L = 1 - \frac{\tanh(\beta L_f / 2)}{\beta L_f / 2} \]

where \( L_f \) is the length of the discontinuous fibers and \( \beta \) is defined as

\[ \beta = \frac{G_m 2\pi}{E_f A_f \ln(R/R_f)} \]

in which \( A_f \) is the cross sectional area of the fiber, \( R_f \) is the radius of the fiber and \( R \) is the mean separation between fibers. For the purpose of this analysis, the packing geometry of the fibers was assumed to be a square array in which the mean fiber separation \( R \) can be determined using the expression
The efficiency factor $\eta_L$ represents the effect of discontinuous fibers as opposed to continuous fibers and will thus be applicable to the longitudinal strength calculations only. The expression for longitudinal tensile strength becomes

$$\sigma_L = \eta_L \sigma_{fL} V_f$$

The transverse tensile strength of a unidirectional fiber reinforced composite is not significantly affected by fiber strength but is instead controlled by the strength of the matrix, the matrix-fiber bond strength, and the presence of voids or defects in the materials. The basic form of this relationship is

$$\sigma_{2t} = \frac{\sigma_{mu} E_2}{E_m K}$$

where $K$ is a strain concentration factor that is designed to account for the strain concentrations in the matrix surrounding the fibers. Gibson suggests a relationship for $K$ which takes the form

$$K = \frac{1}{d_f \left[ \frac{E_m}{E_f} - 1 \right] + 1}$$

where $d_f$ is the fiber diameter and $R$ is the mean fiber spacing. The ratio $d_f/R$ is directly related to the fiber volume fraction. Barbero breaks the stress concentration factor
into two expressions, one to embody the effect of fiber volume fraction and the other to account for the detrimental effect of voids.

This equation takes the form

\[ F_{2t} = \sigma_{mu} C_v \left( 1 - V_f^{1/3} \right) \frac{E_2}{E_m} \]

where \( C_v \) represents the void effect and is expressed as

\[ C_v = 1 - \frac{4V_v}{\pi \left( 1 - V_f \right)} \]

A similar expression proposed by Chamis eliminates the dependence on \( E_2 \) and results in

\[ F_{2t} = \sigma_{mu} C_v \left[ 1 + \left( V_f - \sqrt{V_f} \right) \left( 1 - \frac{E_m}{E_f} \right) \right] \]

For the sprayed FRP material used throughout this research program, it was not possible to experimentally determine the volume fraction of voids within the material. For the purpose of this analysis, however, an attempt was made to determine the void ratio empirically, based on density variations among the different fiber lengths.

As mentioned earlier in Section 5.2.2, the density of the sprayed FRP material tended to
increase with fiber length. Since the actual delivery of the two components to the spraying surface remains constant regardless of fiber length, it could be concluded that this variability in density is due to the degree of compaction achieved with the different fiber lengths. Such a conclusion is supported by the difficulties encountered during the rolling out process, also described in Section 5.2.2.

Inferior compaction would result in a larger number of voids in the final composite and thus a reduction in transverse tensile strength. Since measurement of the void volume fraction could not be made directly, an empirical approach was devised. Figure 5.9 depicts the measured composite densities plotted against fiber length. With the aid of a statistical curve fitting program, a relationship was obtained which matched the data extremely well ($r^2 = 0.9998$), as indicated in the figure.

$$y = a + b \cdot e^{x \cdot p (-0.5 \ln (x / c) / d)^2}$$

$$a=1070.0092 \quad b=324.85795 \quad c=57.242277 \quad d=2.2101607$$

$$r^2=0.999750308$$

**Figure 5.9:** Composite density by fiber length.
Examining the graphical representation of this relationship, it appears that the density not only increases with fiber length but that the rate of increase actually slows at the high end of the fiber lengths used. This corresponds exactly with compaction difficulties encountered in the laboratory. The 8 mm fibers were very difficult to roll out but this task became progressively easier as the fiber length increased. With the 48 mm fibers, it was possible to put a great deal more pressure on the roller during the compaction process.

Upon further investigation, it was discovered that the fitted equation actually becomes asymptotic to a density of 1395 kg/m$^3$, which is only slightly higher than the 1394 kg/m$^3$ measured for the 48 mm fiber length. For the purpose of this analysis, therefore, it was assumed that full compaction (i.e. no voids remaining) would produce a density of 1395 kg/m$^3$ and that any discrepancy would represent the voids present in the material. Furthermore, the basic Rule of Mixtures for combining materials of different densities can be used to determine the expected fiber volume fraction given a final composite density of 1395 kg/m$^3$.

\[
\rho_{fp} = \rho_f V_f + \rho_m V_m
\]

This results in a theoretical fiber volume fraction of 0.209, nine percent higher than the 0.190 actually measured for the 48 mm fiber length. Implementation of the empirical relationship described in Figure 5.9 results in the volume fractions shown below in Table 5.4.
<table>
<thead>
<tr>
<th>Fiber Length (mm)</th>
<th>Volume Fraction (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Fiber</td>
</tr>
<tr>
<td>8</td>
<td>17.0</td>
</tr>
<tr>
<td>16</td>
<td>17.6</td>
</tr>
<tr>
<td>24</td>
<td>18.1</td>
</tr>
<tr>
<td>32</td>
<td>18.6</td>
</tr>
<tr>
<td>48</td>
<td>19.0</td>
</tr>
</tbody>
</table>

Determination of the inplane shear strength of a unidirectional continuous fiber reinforced composite is very similar to the procedure for calculating transverse tensile strength. The matrix tensile strength and the elastic moduli in the previous equation are replaced with the matrix shear strength and the shear moduli of the components to give

\[ F_6 = \tau_{mu} C_v \left[ 1 + \left( V_f - \sqrt{V_f} \right) \left( 1 - \frac{G_m}{G_f} \right) \right] \]

The shear moduli of the fiber and matrix \((G_f \text{ and } G_m)\) were calculated from their respective elastic moduli and Poisson’s ratios using the relationship

\[ G = \frac{E}{2(1 + \nu)} \]

Upon examination of a number of material property sources, it was found that the Poisson’s ratios for glass fiber and polyester resin tend to be very consistent and were taken to be \(\nu_f = 0.22\) and \(\nu_m = 0.38\).
Once values are obtained for the longitudinal tensile, transverse tensile and inplane shear strengths ($F_{lt}$, $F_{2t}$ and $F_6$, respectively) of the fictitious continuous, unidirectional composite, the tensile strength of the randomly oriented continuous fiber composite can be determined. The relationship used for making this transition is represented by one of the following equations:\[77\]

\[
F_{frp} = \frac{4\alpha_1 F_{2t}}{\pi} \left[1 + \frac{1}{2} \ln \left( \frac{F_{lt}}{\alpha_2 F_{2t}} \right) \right] \text{ for } \alpha_1 \leq \alpha_2
\]

\[
F_{frp} = \frac{4F_{2t}}{\pi} \sqrt{\frac{F_{lt}}{F_{2t}}} \text{ for } \alpha_1 > \alpha_2
\]

where the terms $\alpha_1$ and $\alpha_2$ are defined as

\[
\alpha_1 = \frac{F_6}{F_{2t}} \quad \alpha_2 = \sqrt{\frac{F_{lt}}{F_{2t}}}
\]

According to Hahn\[82\], this predictive relationship tends to work well for fiber reinforced polymers with fiber volume fractions of up to 20%. Using Barbero's approach to determine the tensile strengths of the sprayed FRP used in this project produced the results shown in Figure 5.10. Actual calculations and further details have been included in Appendix A.
Figure 5.10: Comparison of theoretical vs. experimental tensile strengths of sprayed FRP composites.

The variation between theoretical values and experimental results is well below the individual coefficients of variation reported for the different fiber lengths during laboratory testing, with the sole exception being the 8 mm fiber length. Actually, all of the theoretical predictions, again with the exception of the 8 mm fiber length, are within 10% of their respective experimental values. This appears to indicate that this approach is an effective means for predicting the tensile strength of this specific combination of materials, at least at the low fiber volume fractions used here.
5.5.2 - Elastic Modulus

Estimation of the elastic modulus of the sprayed FRP material utilizes a similar approach to that followed in the previous section. Basically, the elastic properties of a fictitious unidirectional composite are again determined independently and then combined to produce the properties of the randomly oriented fiber system.

The elastic modulus in the fiber direction of a unidirectional fiber reinforced matrix is normally assumed to follow the Rule of Mixtures equation

\[ E_1 = E_f V_f + E_m V_m \]

In order to modify this expression to account for the discontinuity of the fibers, the same efficiency factor used earlier in the tensile strength portion of the analysis is implemented to correct the fiber contribution portion.

\[ E_1 = \eta E_f V_f + E_m V_m \]

The transverse modulus of a unidirectional fiber reinforced composite is typically determined using the inverse Rule of Mixtures, which takes the form.

\[ \frac{1}{E_2} = \frac{V_m}{E_m} + \frac{V_f}{E_f} \]
However, unless the fiber volume fraction is very high, the fibers do not contribute to the transverse stiffness of the composite. $E_2$ is thus a matrix dominated property and the above equation does not provide accurate results for low fiber volume fractions. A more accurate prediction can be obtained with the Halpin-Tsai expression

$$E_2 = E_m \left[ \frac{1 + \zeta \eta V_f}{1 - \eta V_f} \right]$$

where $\zeta$ is an empirical parameter that can be approximated by 2.0 for circular or square fibers and $\eta$ is a correction factor of the form

$$\eta = \left( \frac{E_f / E_m}{E_f / E_m} - 1 \right) \left( \frac{E_f / E_m}{E_f / E_m} + \zeta \right)$$

Combining the expressions for longitudinal and transverse stiffness of the fictitious unidirectional fiber composite to produce a relationship for randomly oriented fibers involves idealizing the composite as a laminate with numerous layers, each with a slightly different orientation. The resulting expression derived by Barbero is

$$E_{frp} = \frac{E_1^2 + 4E_1G_{12}\Delta + 2E_1E_2 + 8\nu_{12}E_2G_{12}\Delta - 4\nu_{12}^2E_2^2 + 4E_2G_{12}\Delta + E_2^2}{\Delta \left( 3E_1 + 2\nu_{12}E_2 + 3E_2 + 4G_{12}\Delta \right)}$$
where \( \Delta = 1 - \nu_{12} \nu_{21} \) and \( G_{12}, \nu_{12} \) and \( \nu_{21} \) are determined using the expressions

\[
G_{12} = G_m \left[ 1 + \frac{V_f \left( 1 + G_m/G_f \right)}{G_m/G_f + S_3 \left( 1 - G_m/G_f \right)} \right]
\]

\[
S_3 = 0.49247 - 0.47603V_f - 0.02748V_f^2
\]

\[
\nu_{12} = \nu_f V_f + \nu_m V_m
\]

\[
\nu_{21} = \frac{\nu_{12} E_2}{E_1}
\]

According to Hull, the expression for \( E_{frp} \) can be approximated by the much simpler expression.

\[
E_{frp} = \frac{3}{8} E_1 + \frac{5}{8} E_2
\]

Both the Barbero and Hull relationships were used to predict the composite elastic modulus for the sprayed FRP material. The results are shown in Figure 5.11 while further details have been included in Appendix A.
Surprisingly, the approximation proposed by Hull actually produced better results when compared with the experimentally measured values. The theoretical results were generally within the coefficient of variation found during laboratory testing of the sprayed composite material, with the exception of the FRP containing 8 mm fibers. As with the tensile strength predictions discussed in the previous section, this approach appears to be an effective means for predicting the elastic modulus of this specific combination of materials.
6.1 - Introduction

The primary testing done for this research project involved the retrofit or rehabilitation of existing reinforced concrete beams. It should be noted here that the terms *retrofit* and *rehabilitation* are used interchangeably to describe the upgrading of existing structures. Throughout this text, these terms are meant to include both the strengthening and repair of such structures.

The majority of the beams tested in this research were small in size due to the large number of specimens required. This smaller size allowed the researcher to prepare all of the required specimens in the laboratory, with the sole exception of the final project phase described in Section 6.3. The final set of specimens were full scale bridge channel beams generously provided by the Ministry of Transportation and Highways of British Columbia.
6.2 - Laboratory Specimens

Due to the large number of laboratory size specimens tested and the number of variables studied throughout this project, the results will be broken down into the different project phases, as shown in Table 6.1 below.

<table>
<thead>
<tr>
<th>Project Phase</th>
<th>Specimen Type</th>
<th>Variables Considered</th>
<th>Section</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Flexural Strengthening</td>
<td>Beam Condition (Damaged/Undamaged) Concrete Strength (Normal/High)</td>
<td>6.2.4</td>
</tr>
<tr>
<td>2</td>
<td></td>
<td>Beam Condition (Damaged/Undamaged)</td>
<td>6.2.5.2</td>
</tr>
<tr>
<td>3</td>
<td>Shear Strengthening</td>
<td>FRP Strength FRP Configuration Reinforcement Configuration</td>
<td>6.2.5.3</td>
</tr>
<tr>
<td>4</td>
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<td>GFRP Thickness</td>
<td>6.2.5.4</td>
</tr>
<tr>
<td>5</td>
<td></td>
<td>Fiber Length</td>
<td>6.2.5.5</td>
</tr>
<tr>
<td>6</td>
<td></td>
<td>Comparison with Fabric Wraps</td>
<td>6.2.5.6</td>
</tr>
</tbody>
</table>

6.2.1 - Specimen Preparation

All laboratory specimens were identical in size and proportion, with the only difference between the flexural strengthening and shear strengthening specimens being the reinforcement design. These beams were 1 m in length, 96 mm in width and 125 mm in overall depth. Casting was done on a vibrating table to ensure proper consolidation of the concrete and to ensure flow around and between reinforcing bars.
Specimens were demoulded after one day and immersed in lime saturated water. At 28 days of age, the beams were removed from the curing bins and set out to dry under normal laboratory conditions. A minimum of one week of such drying was allowed prior to any testing, preloading, sandblasting or spraying.

6.2.2 - Testing Procedure

For the laboratory cast specimens, all of the beams were tested in third-point loading using a Baldwin 400 kip universal testing machine. These beams were 1 m in total length and were tested over a 900 mm span, using the loading arrangement shown schematically in Figure 6.1.

![Laboratory scale beam test setup](image)

**Figure 6.1:** Laboratory scale beam test setup.
Though this testing procedure was not specifically intended to fully conform to any particular ASTM standard, the load application rate used throughout the testing program was derived from ASTM C78 *Flexural Strength of Concrete*. This standard specifies a rate of increase in the flexural stress of 0.86 – 1.21 MPa/min for flexural testing. The flexural stress in the concrete $R$ was determined as:

$$ R = \frac{Pl}{bd^2} $$

where:
- $R$ = flexural stress in concrete (MPa)
- $P$ = applied load (N)
- $l$ = span length (mm)
- $b$ = specimen width (mm)
- $d$ = specimen depth (mm)

Rearranging this equation for the applied load $P$ gives

$$ P = \frac{Rbd^2}{l} $$

Substituting the previously mentioned ASTM C78 flexural stress range endpoints (0.86 and 1.21 MPa), along with values for $b$ (96 mm), $d$ (125 mm) and $l$ (900 mm) allows determination of a recommended range for load application rate. As a result, a loading range of 1433-2017 N/min (322-453 lbs/min), with a target of 1780 N/min (400 lbs/min), was used for the small scale specimens in this research project. Though automation of this load rate was not possible with the equipment used, the load rate was monitored visually throughout the testing and was maintained within this range as consistently as possible.
The reported deflection value $\Delta$ for each specimen is a corrected quantity which was taken to be the mid-span deflection $\delta_c$ less the average support settlement indicated by the other two LVDTs ($\delta_1$ and $\delta_2$).

$$\Delta = \delta_c - \frac{\delta_1 + \delta_2}{2}$$

In the early stages of loading, the settlement values ($\delta_1$ and $\delta_2$) were as high as 5% of the midspan deflection, though the amount of settlement quickly leveled off and became virtually constant. This maximum settlement was attained even though the load was still increasing, and the settlement deflections typically amounted to about 1% of midspan deflection at the same load level and much less than 1% of the final midspan deflection. The upper LVDTs used to measure this settlement were removed once the settlement leveled off. This was done to protect these sensitive devices from damage due to any sudden upward movement of the specimen during failure. During the initial phases of testing, it was determined that a load of 10000 lbs was sufficient to reach maximum settlement and the upper LVDTs were removed once the applied load reached this level.

To allow comparisons with respect to the performance of the different beams, data acquisition software was used to monitor and collect both the applied load and the three LVDT readings. After adjusting the measured deflection values, load-deflection curves were produced for each specimen. From these curves, the initial member stiffness and peak load were identified. Another quantity calculated from this data was the energy absorbed by the specimen during loading, with energy absorption being represented by the area under the
load-deflection curve. The final piece of information that was noted for each specimen was the mode of failure, which consisted of the event causing the load to drop after reaching its ultimate value.

There is one major shortcoming to the use of this definition of “failure”. In many cases the failed beam was still capable of carrying far more load than the peak load associated with its control specimen. Though such beams would obviously need to be repaired, labeling it as failed when it is still carrying significantly more load than it was capable of before being retrofitted is surely a loose definition of the word.

6.2.3 - Retrofit Schemes

In order to determine the effectiveness of the sprayed GFRP system for rehabilitation of reinforced concrete beams, it was necessary to examine several different approaches or configurations that could be implemented. Specifically, the number of surfaces around the beam to be sprayed and the effect of the various combinations of these surfaces was of utmost interest. Throughout the project, a total of five different retrofit schemes were studied. These schemes are depicted schematically in Figure 6.2.

Generally, it was anticipated that spraying the sides of the member would contribute to the shear strength of the beam while spraying the bottom face would provide flexural reinforcement. The various retrofit schemes were thus designed so as to provide information for each of these cases. To provide shear reinforcement only, the two vertical sides of the
beam were sprayed (Scheme A) while pure flexural reinforcement was provided by spraying only the bottom of the beam (Scheme E).

Schemes C and D provided a combination of shear and flexural reinforcement and are more typical of the type of retrofit procedure that would be used in the field. Scheme B was added to the early phases of the project to help determine the contribution, if any, of the upper plate in the Scheme D specimens. The retrofit scheme of the most interest from a practical standpoint is Scheme C due to the inability, in most cases, to gain access to the upper surface of in situ reinforced concrete beams. Such access would often be blocked by the roof deck, bridge deck or floor slab that the beam is supporting.

One difficulty that was encountered during the retrofit process was the inability of the fibers to stay in place when bent around sharp corners. The stiffness of these fibers was sufficient to cause a tendency for them to straighten after being bent around a corner. This would cause a continuous air pocket to form along the edge of the beam and prevent the fibers from bonding to the adjacent face. A special application technique was developed to offset this problem. Figure 6.3 depicts the procedure that was used throughout this project.
The first step was simply to spray only one surface of the beam at a time. Prior to spraying any particular surface, the adjacent faces were masked with masking tape to prevent any resin overspray from accumulating. The first surface, typically the bottom, was sprayed with no attempt to force the fibers around the corners (Step 1). They were simply allowed to project outward from the edge of the surface. A ribbed aluminum roller was used to compact the material as described in Section 4.2. (Step 2). After the resin had hardened, the projecting edges were trimmed off and a 45° bevel was ground into the edge of the GFRP plate (Step 3). The beam was then flipped 180° and steps 1 through 3 were repeated for the top face, if necessary for the particular retrofit scheme being used.

Once the GFRP applied to the top and bottom faces had cured sufficiently, the beam was
turned 90° (Step 4) and one of the sides side was sprayed. This time, during the compaction process, the GFRP material was rolled around the 45° bevels at the corners (Step 5), allowed to cure, and any excess material was then trimmed (Step 6). The beam was then flipped 180° and steps 4 through 6 were repeated for the opposite side.

The above description details the technique used when spraying all four faces of the beam, as in Scheme D. For the other schemes, steps were omitted as needed depending on the number of faces sprayed and the number of corners where sprayed faces met. For retrofit schemes where sprayed faces were adjacent to unsprayed faces (i.e. Schemes A, B, C, and E), the edge of the plates were simply trimmed off square without being beveled.

Following completion of the spraying process, the retrofitted beams were left for a minimum of 48 hours under laboratory conditions. Though the exposure temperature does affect the setting time of the resin, this effect is insignificant in this case since full resin strength is reached in a matter of hours even at relatively low temperature. The 48 hour delay was chosen more for scheduling reasons than due to any concerns over curing time.
6.2.4 - Flexural Strengthening

6.2.4.1 - Beam Design

To examine the effectiveness of the sprayed GFRP material in the flexural rehabilitation of reinforced concrete members, a series of 20 beams was cast using normal strength (45 MPa) concrete. These beams contained full reinforcement cages (flexural and shear reinforcement) designed in accordance with CSA Standard A23.3-94 to produce a typical flexural failure mode. Details of the beam cross section and reinforcement sizes are provided in Figure 6.4 while stirrup location and spacing are indicated in Figure 6.5.

![Figure 6.4: Flexural strengthening - beam design.](image-url)
Note that stirrups were only included in the shear spans of the specimen. They were not necessary in the center span since the shear stress in this region is zero under third point loading. During casting, 25 mm cubes of hardened concrete were used as chairs to support the reinforcing cages at their ends. A total of four mixes (with 5 beams per mix) were required to produce the 20 beams due to limitations on the maximum concrete batch size.

6.2.4.2 - Testing Program

The 20 beams in this phase were divided into 7 groups to allow investigation of three different retrofit schemes and two specimen conditions (damaged vs. undamaged). Of the retrofit schemes previously described in Figure 6.2, only Schemes C, D and E were used in this phase. Scheme A was omitted since it is primarily designed for shear reinforcement and would be largely ineffective in flexural reinforcing. Table 6.2 provides details for the 7 different specimen groups in this phase.
Table 6.2: Flexural strengthening specimen groups.

<table>
<thead>
<tr>
<th>Retrofit Scheme</th>
<th>Testing Condition</th>
<th>Number of Specimens</th>
</tr>
</thead>
<tbody>
<tr>
<td>None</td>
<td>Control</td>
<td>4</td>
</tr>
<tr>
<td>C</td>
<td>Damaged</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>Undamaged</td>
<td>4</td>
</tr>
<tr>
<td>D</td>
<td>Damaged</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>Undamaged</td>
<td>2</td>
</tr>
<tr>
<td>E</td>
<td>Damaged</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>Undamaged</td>
<td>2</td>
</tr>
</tbody>
</table>

Care was taken to ensure that each of the groups containing four specimens (Groups 1-3) was made up of one specimen from each mix. The remaining groups (containing two specimens each) were made up of two specimens from the same mix. Complete information for each beam has been compiled in Appendix B.

The beams indicated in Table 6.2 as being damaged were subjected to a preloading stage before rehabilitation with sprayed GFRP. Beams were tested in both the damaged and undamaged condition in order to determine the effectiveness of sprayed GFRP in both strengthening and repair applications. Preloading consisted of subjecting the beams to third point loading until the center point deflection reached 5 mm (L/180). This damage level was also chosen during testing of the shear strength deficient beams described in Section 6.2.5. For those specimens, it corresponded to the appearance of the first shear crack, though it preceded beam failure. For the flexural strengthening beams in this phase, this preloading...
resulted in the appearance of several fine flexural cracks (approximately 0.1 mm in width) at this deflection level.

Following the preloading or damaging stage, the surfaces of both the damaged and undamaged specimens were sandblasted in an attempt to optimize the GFRP-concrete bond and then sprayed as described in Section 6.2.3. Spraying for this phase of the project was performed by the author at the University of British Columbia.

This was the first set of specimens prepared with the equipment purchased for this project and the fiber length was, at that time, limited to 32 mm. The material properties for this material are discussed further in Chapter 5 ($l_f = 32$ mm, $\sigma_{ult} = 104$ MPa, $E = 10.5$ GPa, $\Delta_f = 1.43\%$, $V_f = 18\%$). Application thickness was approximately 8 mm.

6.2.4.3 - Results & Discussion

The test results for the flexural strengthening phase are compiled below in Table 6.3. Note that the values reported in the table represent the averages of all the beams in each group.

Presented along with each test result is the percent change (‘+’ being an increase and ‘-’ representing a decrease) induced by the retrofitting process, calculated with respect to the control specimens. As discussed previously in Section 6.2.2, the failure modes reported in Table 6.3 refer to the event occurring as the load reaches its peak value and then drops off. For example, Table 6.3 indicates that the Scheme E specimens failed due to a debonding of
the GFRP plate from the concrete. This is the event that caused the load to drop from its peak value. In actuality, once the GFRP debonds, the beam reverts to its unretrofitted properties (as per the control specimens) and the final failure would thus be due to yielding of the steel reinforcement.

<table>
<thead>
<tr>
<th>Retrofit Scheme</th>
<th>Testing Condition</th>
<th>Stiffness</th>
<th>Strength</th>
<th>Failure Mode*</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Initial (kN/mm)</td>
<td>Change (%)</td>
<td>Peak (kN)</td>
</tr>
<tr>
<td>None</td>
<td>Undamaged</td>
<td>11.9</td>
<td>-</td>
<td>63</td>
</tr>
<tr>
<td></td>
<td>Damaged</td>
<td>16.0</td>
<td>+34</td>
<td>80</td>
</tr>
<tr>
<td></td>
<td>Undamaged</td>
<td>15.0</td>
<td>+26</td>
<td>86</td>
</tr>
<tr>
<td>E</td>
<td>Damaged</td>
<td>22.6</td>
<td>+89</td>
<td>126</td>
</tr>
<tr>
<td></td>
<td>Undamaged</td>
<td>21.6</td>
<td>+81</td>
<td>136</td>
</tr>
<tr>
<td>C</td>
<td>Damaged</td>
<td>29.8</td>
<td>+151</td>
<td>245</td>
</tr>
<tr>
<td></td>
<td>Undamaged</td>
<td>24.3</td>
<td>+104</td>
<td>230</td>
</tr>
<tr>
<td>D</td>
<td>Damaged</td>
<td>15.0</td>
<td>+26</td>
<td>150</td>
</tr>
</tbody>
</table>

* - Failure modes: Y = Rebar Yielding, S = Shear, C = Concrete Crushing, B = GFRP Bond, P = GFRP Plate

Application of the sprayed GFRP coating had a significant effect on member stiffness, with that increase growing larger as the degree of confinement increased (i.e. more sides coated). For the undamaged specimens, the Scheme E retrofit induced a 26% improvement in initial stiffness while the Scheme C approach outperformed that by more than three times with an 81% increase. The full confinement of the Scheme D retrofitted beams virtually doubled the stiffness of the control specimens with a 104% increase. This improvement in member stiffness is especially beneficial from the viewpoint of load distribution.
A very interesting trend appeared in these results as well. In all three retrofit schemes, the repaired beams (i.e. damaged specimens) showed slightly higher improvements in stiffness than the strengthened beams (i.e. undamaged specimens). The differences were, for the most part, fairly small relative to the improvement over the control specimens and cannot be considered statistically significant due to the small sample size, but were intriguing due to the consistency in the results. One possible explanation that may explain this phenomenon is the effect of the polyester resin on the cracks. It was noted that the resin does enter the damage cracks during the spraying process. This penetrating resin may be acting as reinforcement for the crack by adhesively bonding to both sides and thus resisting further crack opening.

The second test result to be examined was the peak load carried by the specimens immediately before failure. This is probably the most important value determined for each beam since it signifies the ultimate load carrying capability of the beam and represents the point beyond which failure occurs. As with the initial stiffness values, the peak load exhibited by the specimens also increased with the number of sides coated. Contrary to the stiffness results, however, the damaged specimens did not consistently outperform the undamaged beams. In fact, there was very little difference between the two groups.

Other than the actual calculated values reported thus far in this section, there is a great deal to be learned from the load-deflection curves of the different specimen types. The load-deflection curves for the undamaged/strengthened specimens are shown in Figure 6.6 while the respective curves for the damaged/repaired beams are provided in Figure 6.7.
Figure 6.6: Flexural strengthening specimens - load-deflection curves (undamaged/strengthened beams).

Figure 6.7: Flexural strengthening specimens - load-deflection curves (damaged/repaired beams).
These figures include all of the beams tested in this phase of the project and have been included primarily to indicate the consistency between identical specimens within each specimen group (i.e. each retrofit scheme and testing condition combination).

For each of these specimen groups, a typical curve was determined using a curve fitting software package. These typical curves are shown in Figure 6.8. As can be seen from this figure, there is no significant difference in performance between the damaged and the undamaged specimens. Actually, there is not even a consistent trend as to which of these two subgroups performs better.

![Figure 6.8: Flexural strengthening specimens - typical load-deflection curves (strengthened vs. repaired specimens).](image)

For the Scheme D retrofit approach, the damaged beams seem to be slightly stronger while the Scheme C specimens performed better in the undamaged condition. In the Scheme E
beams, on the other hand, the curves actually crossed, though only after the damaged specimen curve reached its peak load and began to drop.

Removing the curves related to the damaged specimens provides a very clear picture of the improvement in performance exhibited by the retrofitted beams (Figure 6.9). The increases in both stiffness and peak load are easily discerned in this figure.

Figure 6.9: Flexural strengthening specimens - typical load-deflection curves.

Another important property related to beam performance is the energy absorbed by the specimen during loading, which is represented by the area under the load-deflection curve. Figure 6.10 indicates that the energy absorbed by the retrofitted specimens increased quite dramatically as well.
A better comparison can be obtained by determining the energy absorbed at similar deflection levels. Figure 6.10 depicts the cumulative absorbed energy for each retrofit scheme. Now it becomes obvious that the Scheme E beams absorb more energy than the control specimens at all deflection levels. The Scheme C and D specimens perform incrementally better.

Another issue that had a significant effect on the results reported in this section is the failure mode exhibited by the test specimens. As mentioned in Table 6.3, the control specimens failed due to yielding of the longitudinal reinforcement, as expected in an under-reinforced beam. In practice, this failure mode involves the formation of fine flexural cracks along the bottom face of the beam. These cracks grow wider and longer as the steel yields, causing the
neutral axis of the member to move upward, eventually leading to crushing of the concrete at the upper surface of the beam. In a balanced section, the flexural cracking and the concrete crushing should begin at the same time, though it is more typical for the failure to be initiated by the flexural cracks as described above.

In the Scheme C specimens, the extra tensile reinforcement provided by the GFRP applied to the bottom face of the beam disrupted the balanced reinforcement condition. As would be expected for an over-reinforced member, these beams tended to fail due to crushing of the concrete in the compression zone at the upper surface of the beam before steel yielding. A typical example of this failure is shown in Figure 6.11. This photograph shows the crushed concrete in the compression zone.

![Figure 6.11: Flexural strengthening specimens - typical failure of Scheme C specimens.](image-url)
It should be noted here that this photograph was taken well after failure of the specimen to provide a better view of the crushing zone once the GFRP had buckled. The debonding of the GFRP which is evident along the side of the beam occurred due to the large deflections after member failure and did not contribute to it.

The Scheme D specimens failed due to tensile fracture of the sprayed GFRP in the tension zone at the bottom surface of the beam. Figure 6.12 provides a typical example of this failure mode. This photograph shows the side of the beam shortly after failure. Initially, the GFRP fails in tension at the bottom face, though the GFRP on the sides of the beam does not immediately fail. Actually, the beam is still capable of carrying a significant amount of load at this point.

As the load-deflection curve in Figure 6.9 indicates, even after failure of the GFRP takes place (indicated by the large drop in load immediately following the ultimate load) these beams can still carry a load of about 100 kN. This residual strength remains virtually constant as cracks begin to propagate upward form both ends of the original failure. Eventually, the GFRP on the upper surface buckles and the load carrying capability drops to negligible values. The deflections associated with this point are extremely large and virtually impossible to obtain with the available loading apparatus.
Though both the Scheme C and D specimen failure modes tended to take full advantage of the applied GFRP, a significant problem transpired in the Scheme E specimens where the GFRP was applied to the bottom face only. These beams tended to fail due to debonding of the GFRP from the bottom face, resulting in specimen failure occurring before the GFRP could reach its full effectiveness. The photograph in Figure 6.13 depicts a typical example of this failure mode.
Again, this photograph was taken after specimen failure and shows both flexural cracking and crushing of the concrete. Both of these damage mechanisms, however, occurred after the initial failure of the specimen due to debonding of the GFRP. Also apparent in the photograph is the debonded GFRP layer along the bottom. This type of debonding begins at one end of the GFRP and progresses inward toward the midspan, occurring very quickly and leading to the appearance of flexural cracking and eventual crushing of the concrete.
6.2.5 - Shear Strengthening

The largest single portion of this project involved the strengthening of beams intentionally designed to be deficient in shear strength. Shear strength deficiency is particularly a problem in structures designed in accordance with older building codes since newer codes place more emphasis on this aspect of their performance. It can also be quite common in structures exposed to chlorides, either from deicing salts or seawater, since corrosion of the reinforcement will begin at the outermost steel, the shear stirrups. Effectively, the shear capacity of the section will begin to deteriorate immediately.

A number of different investigations were conducted in this part of the research program and these are summarized below in Table 6.4.

<table>
<thead>
<tr>
<th>Project Phase</th>
<th>Specimen Type</th>
<th>Variables Considered</th>
<th>Section</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>Beam Condition</td>
<td>Beam Condition (Damaged/Undamaged)</td>
<td>6.2.5.2</td>
</tr>
<tr>
<td>3</td>
<td>Shear Strengthening</td>
<td>FRP Strength</td>
<td>6.2.5.3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>FRP Configuration</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Reinforcement Configuration</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>GFRP Thickness</td>
<td>GFRP Thickness</td>
<td>6.2.5.4</td>
</tr>
<tr>
<td>5</td>
<td>Fiber Length</td>
<td>Fiber Length</td>
<td>6.2.5.5</td>
</tr>
<tr>
<td>6</td>
<td>Comparison with Fabric Wraps</td>
<td>Comparison with Fabric Wraps</td>
<td>6.2.5.6</td>
</tr>
</tbody>
</table>

6.2.5.1 - Beam Design

To ensure shear failure in these beams, the flexural reinforcement was enlarged from the two
10M bars used in the flexural strengthening specimens) to two 15M bars and the shear stirrups completely eliminated - a worst case scenario. Typical beam dimensions and details are shown in Figure 6.14.

![Figure 6.14: Shear strengthening - beam design.](image)

With the exception of one subgroup of beams to be discussed in this section, all of the beams dealt with in this section were cast with normal strength concrete exhibiting a design compressive strength of 45 MPa. As mentioned previously in Section 6.2.1., these beams were 1 m in length, 96 mm in width and 125 mm in overall depth.

6.2.5.2 - Initial Study on Repair & Strengthening

This section describes the initial round of testing for shear strength deficient beams, which was designed to examine the effectiveness of four different retrofit schemes in both the
strengthening and repair of shear strength deficient beams. There was a total of 31 beams included in this phase, 22 cast from normal strength concrete and a further 9 cast from high strength concrete. As previously reported in Section 3.2, the design strengths for normal and high strength concretes were 45 and 80 MPa, respectively. These two sets of specimens will be discussed separately, the normal strength specimens now and the high strength specimens later in this section.

6.2.5.2.1 - Normal Strength Concrete

The 22 normal strength beams in this phase were divided into 9 groups to allow investigation of four different retrofit schemes and two specimen conditions (damaged vs. undamaged). Of the retrofit schemes previously described in Figure 6.2, Schemes A, B, C, and D were used. Scheme E was omitted since it is primarily designed for flexural reinforcement and was not expected to contribute significantly to the shear strength of the member. Table 6.5 provides details for these 9 different specimen groups.

The beams indicated in Table 6.5 as being damaged were subjected to the same preloading stage described in Section 6.2.4.2, which consisted of subjecting the beams to third point loading until the center point deflection reached 5 mm (L/180). For these specimens, this deflection level corresponded to the appearance of the first shear crack, though it preceded beam failure in most cases.
Unfortunately, the difference in load between the appearance of this initial crack and full failure of the member in shear was very small. Occasionally, a beam would fail before the loading could be stopped. Such beams were excluded from the study. There was also a second problem involving confusion on the part of the spray operator concerning specimen identification which resulted in the specimens being unevenly distributed between groups (both the normal strength specimens discussed here and the high strength concrete specimens covered in the next section).

Following the preloading or damaging stage, the surfaces of both the damaged and undamaged specimens were sandblasted in an attempt to optimize the GFRP-concrete bond and then sprayed as described in Section 6.2.3. Spraying for this phase of the project was
performed by GU Manhole Liners Ltd. of Aldergrove, BC. Strength properties for the applied GFRP material are discussed in Chapter 5 (\(l_f = 8\) mm, \(\sigma_{ult} = 35\) MPa, \(E = 5.8\) GPa, \(\Delta_f = 0.66\%)\), \(V_f = 8\%\)). The GFRP was applied in a coating approximately 6 mm thick.

Test results for the normal strength specimens are compiled in Table 6.6. The values reported in the table represent the averages of all the beams in each group.

<table>
<thead>
<tr>
<th>Retrofit Scheme</th>
<th>Testing Condition</th>
<th>Stiffness</th>
<th>Strength</th>
<th>Absorbed Energy</th>
<th>Failure Mode*</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Initial (kN/mm)</td>
<td>Change (%)</td>
<td>Peak (kN)</td>
<td>Change (%)</td>
</tr>
<tr>
<td>None</td>
<td>Undamaged</td>
<td>17.5</td>
<td>-</td>
<td>48</td>
<td>-</td>
</tr>
<tr>
<td>A</td>
<td>Damaged</td>
<td>15.0</td>
<td>-14</td>
<td>77</td>
<td>60</td>
</tr>
<tr>
<td></td>
<td>Undamaged</td>
<td>15.6</td>
<td>0</td>
<td>83</td>
<td>73</td>
</tr>
<tr>
<td>B</td>
<td>Damaged</td>
<td>25.2</td>
<td>44</td>
<td>82</td>
<td>71</td>
</tr>
<tr>
<td></td>
<td>Undamaged</td>
<td>22.2</td>
<td>27</td>
<td>93</td>
<td>94</td>
</tr>
<tr>
<td>C</td>
<td>Damaged</td>
<td>24.6</td>
<td>41</td>
<td>132</td>
<td>175</td>
</tr>
<tr>
<td></td>
<td>Undamaged</td>
<td>23.6</td>
<td>35</td>
<td>126</td>
<td>163</td>
</tr>
<tr>
<td>D</td>
<td>Damaged</td>
<td>21.3</td>
<td>22</td>
<td>131</td>
<td>173</td>
</tr>
<tr>
<td></td>
<td>Undamaged</td>
<td>23.6</td>
<td>35</td>
<td>123</td>
<td>156</td>
</tr>
</tbody>
</table>

- Failure modes: \(Y =\) Rebar Yield, \(S = \) Shear, \(C = \) Concrete Crushing, \(B = \) GFRP Bond, \(P = \) GFRP Plate

Examining the results from Table 6.6, it appears that the GFRP applied to the sides of the beam has no effect on stiffness. The Scheme B and Scheme C specimens, however, indicate that GFRP applied to either the top or bottom surface will make the specimen more rigid.
The lack of further increases in stiffness among the Scheme D specimens implies that spraying both of these surfaces does not produce a cumulative effect. It also confirms the Scheme A results by showing that coating the side faces makes no contribution to stiffness. As found in the flexural strengthening phase, there appears to be no significant difference between the repaired and strengthened beams.

Unlike the initial stiffness values, the peak load exhibited by the specimens increased for every retrofit scheme. Similarly to stiffness, however, is the appearance of two distinct groups of improvement. The Scheme A and B specimens reported similar peak load increases while the Scheme C and D retrofits produced nearly identical improvements. These two groups can also be identified by the placement of the retrofitted sides (i.e. the two groups each receive contributions from specific retrofitted surfaces). The improvements in the Scheme A and B specimens seems to be due solely to the two vertical sides while the increase in load carrying ability exhibited by the Scheme C and D specimens seems to be a combination of this and the addition of the bottom face.

Though the stiffness results indicated that the top and bottom faces have virtually the same effect on member stiffness, they do not have the same effect on ultimate load. In the latter case, the top face does not appear to contribute significantly to load carrying ability while the bottom face provides a significant amount of strength.

Again, the peak load results indicate no consistent difference between the GFRP strengthened specimens and those damaged and then repaired with sprayed GFRP. The
Scheme A and B beams reported slightly higher values for the undamaged beams while the Scheme C and D retrofits exhibited the opposite, slightly lower values for the undamaged members. None of these differences can be considered to be significant.

The results for energy absorbed by the specimens up to peak load show that the Scheme A specimens exhibited a 300% increase in absorbed energy with respect to the control specimens while the Scheme B specimens showed an increase of only 270%. Though this difference is not statistically significant, there is a specific reason for its occurrence. As will be shown later in this section, the drop in absorbed energy between the Scheme A specimens to those retrofitted using Scheme B can be directly attributed to the increased stiffness in the Scheme B members. The resulting shift in the load-deflection curve causes the area under that curve to be reduced. This additional stiffness also appeared in the Scheme C and D beams but its effect on the energy absorbed was overshadowed by the much larger increases in energy absorption exhibited by these specimens.

Again there was no consistent difference between the damaged and undamaged specimens with respect to energy absorbed. A significant difference was reported for the Scheme B specimens but the trend was not consistent among the other schemes and was thus considered to be an anomaly.

For each of the specimen groups, a typical curve was determined using curve fitting software. These typical curves are shown in Figure 6.15. Further figures showing each particular combination beam separately, along with the average or typical value for each, can
be found in Appendix C.

![Deflection vs Load Curve](image)

**Figure 6.15:** Shear strengthening - Phase 2 typical load-deflection curves (initial study, strengthened vs. repaired specimens, normal strength concrete).

Eliminating the curves representing the damaged specimens leaves Figure 6.16. This diagram shows the distinct differences among the four different retrofit schemes. First, the increased stiffness in the Scheme B, C and D members, with respect to the control specimens and Scheme A beams, is readily apparent. Second, the increase in peak load which was virtually the same for the Scheme A and B specimens, as well as for the Scheme C and D beams, is also evident. Finally, the reason for the drop in absorbed energy between the Scheme A Scheme B retrofits is also clear. The increased stiffness of the Scheme B specimens shifted the load-deflection curve leftward toward the y-axis, resulting in a reduction in the measured area under the curve.
Figure 6.16: Shear strengthening - Phase 2 typical load-deflection curves (initial study, normal strength concrete).

Figure 6.17: Shear strengthening - Phase 2 typical energy absorption curves (initial study, normal strength concrete).
It must be pointed out that this apparent reduction in absorbed energy only exists because the comparison was made between the energies absorbed up to the point of ultimate load. Comparing the cumulative energy absorbed at similar deflection levels (as shown in Figure 6.17) makes it obvious that the Scheme B beams actually absorbed more energy than the Scheme A specimens at all deflection levels.

Actually, the Scheme B, C and D specimens all absorbed energy at the same rate, up to the successive points where the first two specimen types failed. The Scheme A specimens, on the other hand, were virtually identical to the control beams prior to failure of the latter specimens.

Before discussing the failure modes of the various retrofitted specimens, it is first necessary to describe the failure mode exhibited by the control specimens. These specimens exhibited a typical shear failure as depicted in Figure 6.18 below.

Looking at the typical failure modes indicated in Table 6.6 reveals that a significant problem arose with respect to the bond strength of the sprayed GFRP material. Other than the Scheme D specimens, all of the retrofitted beams tended to fail due to debonding of the GFRP. Though this should not affect the shape of the load-deflection curves up to the point of bond failure, it does essentially act to stop the curve at that point, preventing the GFRP from reaching its full potential.
A typical example of this type of debonding is illustrated in Figure 6.19 which shows a top view of a beam retrofitted using the Scheme C arrangement. As can be seen along the outer edges of the specimen, the GFRP material has separated from the concrete surface. This bond failure occurred before the shear splitting also evident in the photograph.

The Scheme D specimens failed due to tensile fracture of the GFRP applied to the bottom face of the beam, as depicted in Figure 6.20. This photograph shows a side view of the center portion of a Scheme D specimen which failed in this mode. The failure was initiated by tensile fracture of the bottom GFRP, followed by cracks progressing upward on both sides and finally culminating in buckling of the GFRP on the top surface.
Figure 6.19: Shear strengthening - Phase 2 typical GFRP debonding, Scheme C specimen top view (initial study).

Figure 6.20: Shear strengthening - Phase 2 typical GFRP tensile failure in Scheme D specimens, side view (initial study).
To determine whether the sprayed GFRP rehabilitation technique would be effective for concrete members cast from high strength concrete (which tends to be more brittle), a second series of specimens was added to this project phase. This series consisted of 9 beams which were divided into 5 groups to allow investigation of the two most effective retrofit schemes along with both the strengthening and repair approaches. Of the retrofit schemes previously described in Figure 6.2.3, only Schemes C and D were investigated. Table 6.7 provides details for the 5 different specimen groups in this phase.

<table>
<thead>
<tr>
<th>Retrofit Scheme</th>
<th>Testing Condition</th>
<th>Number of Specimens</th>
</tr>
</thead>
<tbody>
<tr>
<td>None</td>
<td>Control</td>
<td>2</td>
</tr>
<tr>
<td>C</td>
<td>Damaged</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>Undamaged</td>
<td>1</td>
</tr>
<tr>
<td>D</td>
<td>Damaged</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td>Undamaged</td>
<td>2</td>
</tr>
</tbody>
</table>

The beams indicated in Table 6.7 as being damaged were subjected to the same preloading stage originally described in Section 6.2.4.2. The problem previously discussed in Section 6.2.5.2.1. concerning premature failure of the beam during the preloading stage was even
more prevalent with the high strength concrete specimens. Since these beams could no longer be used in the study, the number of remaining specimens was limited. Though the original plan called for the testing of three retrofit schemes (A, C and D), the number of specimens remaining resulted in the study being limited to the two retrofit schemes included in the table.

Following the preloading, the surfaces of all specimens were sandblasted and then sprayed as previously described in Section 6.2.3. Spraying was again performed by GU Manhole Liners Ltd. of Aldergrove, BC. Strength properties for the applied GFRP material were discussed in Section 5.2 ($\sigma_{ult} = 35$ MPa, $E = 5.8$ GPa, $\Delta_t = 0.66\%$, $V_f = 8\%$, $t = 6$ mm).

Test results for the high strength specimens are compiled in Table 6.8. The values reported in the table represent the averages of all the beams in each group. Both the Scheme C and Scheme D specimens exhibited significant increases in member stiffness, with the repaired specimens showing a larger improvement in both cases. The Scheme C retrofit actually produced larger improvements than the Scheme D retrofit, though the difference between the two was deemed to be statistically insignificant, primarily due to the small sample size.

Complete data for these specimens has been included in Appendix D. The peak load results indicate that, again, there is a large increase in load carrying ability imparted by both retrofit schemes but that there is very little difference between them or between the damaged and undamaged specimens.
Table 6.8: Shear strengthening - Phase 2 test results (initial study, high strength concrete).

<table>
<thead>
<tr>
<th>Retrofit Scheme</th>
<th>Testing Condition</th>
<th>Stiffness</th>
<th>Strength</th>
<th>Absorbed Energy</th>
<th>Failure Mode*</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Initial</td>
<td>Peak</td>
<td>To Peak</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>(kN/mm)</td>
<td>(kN)</td>
<td>(N·m)</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Change</td>
<td>Change</td>
<td>Change</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>(%)</td>
<td>(%)</td>
<td>(%)</td>
<td></td>
</tr>
<tr>
<td>None</td>
<td>Undamaged</td>
<td>21.0</td>
<td>52</td>
<td>77</td>
<td>S</td>
</tr>
<tr>
<td>C</td>
<td>Damaged</td>
<td>30.7</td>
<td>144</td>
<td>674</td>
<td>B</td>
</tr>
<tr>
<td>C</td>
<td>Undamaged</td>
<td>27.8</td>
<td>152</td>
<td>785</td>
<td>P</td>
</tr>
<tr>
<td>D</td>
<td>Damaged</td>
<td>27.6</td>
<td>143</td>
<td>1017</td>
<td>P</td>
</tr>
<tr>
<td>D</td>
<td>Undamaged</td>
<td>24.9</td>
<td>152</td>
<td>1182</td>
<td>P</td>
</tr>
</tbody>
</table>

* - Failure modes: Y = Rebar Yielding, S = Shear, C = Concrete Crushing, B = GFRP Bond, P = GFRP Plate

Though the performance of the Scheme C and D specimens were, for all practical purposes, identical with respect to member stiffness and ultimate load carrying ability, the amount of energy absorbed by the specimens up to peak load was significantly different. The Scheme D retrofit produced much larger increases in absorbed energy, with the undamaged specimens performing much better than the repaired members for both schemes. It should also be noted here that the degree of improvement observed in the high strength concrete members was very similar to that exhibited by the normal strength concrete specimens.

Typical load-deflection curves were determined for each specimen group and are shown in Figure 6.21. As can be seen from this figure, the strengthened (undamaged) beams in each group performed slightly better than their repaired (damaged) counterparts.
Also visible in this diagram is the difference between the Scheme C and D specimens. It is apparent that the Scheme D members are capable of carrying load further beyond the point of reinforcement yielding than those from Scheme C before reaching failure. This explains the increase in energy absorption exhibited by the Scheme D beams even though the ultimate load did not change.

Figure 6.22 depicts the cumulative energy absorption for each of the retrofit schemes, as well as the control specimens. The absorption rates for the Scheme C and Scheme D beams were much higher than that of the control specimens and were very similar to each other up to the point where the Scheme C specimens failed.
Illustrations of failure modes for the high strength specimens are not included since these modes were identical in appearance to those reported for the normal strength specimens in the previous section. The single variation was the tendency of the strengthened Scheme C specimens to fail due to tensile fracture of bottom GFRP (similar to the Scheme D beams) as opposed to the debonding failure previously described.
6.2.5.3 - Effect of FRP Strength

The specimens discussed in the previous section (6.2.5.2) were sprayed by GU Manhole Liners Ltd. of Aldergrove, BC. After completion of that phase of the testing program, the University of British Columbia Department of Civil Engineering purchased its own GFRP spraying equipment, as described in Section 4.2. From this point on, all of the sprayed GFRP applications discussed were performed by the author using this new equipment.

A longer chopped fiber length (32 mm) and a different resin (polyester) were implemented along with the new equipment, resulting in a higher GFRP strength and elastic modulus (as discussed in Section 5.2.2). Consequently, it was decided that a portion of the initial study be repeated to see the effect of these material improvements. As a result, this portion of the study was designed to reexamine the effectiveness of the three primary retrofit schemes used in the initial study (Schemes A, C and D). Scheme B, which involved coating the sides and top of the specimen, was omitted due to its inapplicability to real-life applications. Also, since the previous testing failed to indicate any significant difference between strengthened and repaired specimens, it was decided to reduce the number of test specimens by using only undamaged specimens. Conversely, this project phase was expanded to investigate two other factors.

First, there was some concern whether the development length of the reinforcement was sufficient to prevent debonding of the concrete from the steel since this failure mode had been observed in a few specimens. To investigate this problem, the use of hooked
reinforcement was included in the testing program. Basically, the ends of the reinforcing bars were bent upwards at a 90° angle and continued in this direction for another 50 mm.

Second; another issue that was raised was the continuity of the bottom GFRP over the entire length of the member. It was felt that continuation of the GFRP over the supports in the laboratory specimens may result in a clamping action as the downward loading force pinned the GFRP between the concrete beam and the support. Such a clamping action might work toward preventing GFRP from debonding. In a field application of such materials, the GFRP would have to be discontinued at the beam supports (i.e column, girder, etc.) due to lack of access. To investigate this problem, some of the specimen configurations were duplicated, except that the bottom GFRP was abbreviated (ground off) short of the supports.

Additionally, one of the primary problems encountered to this point in the testing program was the premature debonding of the GFRP material from the concrete surface. Consultation with the resin manufacturer (Ashland Chemical Canada Ltd.) resulted in their suggestion to apply a vinyl ester resin to the concrete surface before commencing the spraying operation. According to the manufacturer, vinyl ester possesses superior bonding characteristics with concrete and should act as a coupling agent between the concrete and the polyester resin used in the spraying operation. This vinyl ester resin is further discussed in Section 3.3.3.

In operation, the vinyl ester was prepared (catalysed) and applied by hand with a simple paint brush. It was allowed to cure until ‘tacky’, when it was still sticky to the touch but would no longer leave any residue behind on the tester’s finger. At this point, the regular
spraying operation, as described in Section 4.3, was performed.

There was a total of 16 beams included in this phase, all cast from normal strength (45 MPa) concrete. These specimens were divided into 8 groups to allow investigation of the three retrofit schemes (A, C and D), two lengths of bottom face GFRP (full length and abbreviated) and two shapes of reinforcement (straight and hooked). The resulting 8 groups are detailed in Table 6.9 below.

<table>
<thead>
<tr>
<th>Retrofit Scheme</th>
<th>Retrofit Details</th>
<th>Reinforcement Details</th>
<th>Number of Specimens</th>
</tr>
</thead>
<tbody>
<tr>
<td>None</td>
<td>None</td>
<td>Straight</td>
<td>3</td>
</tr>
<tr>
<td>A</td>
<td>Full Length</td>
<td>Straight</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>Full Length</td>
<td>Hooked</td>
<td>2</td>
</tr>
<tr>
<td>C</td>
<td>Full Length</td>
<td>Straight</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>Full Length</td>
<td>Hooked</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>Abbreviated</td>
<td>Hooked</td>
<td>2</td>
</tr>
<tr>
<td>D</td>
<td>Full Length</td>
<td>Straight</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>Abbreviated</td>
<td>Hooked</td>
<td>1</td>
</tr>
</tbody>
</table>

The surfaces of all specimens were sandblasted in an attempt to optimize the GFRP-concrete bond and then sprayed as described in Section 6.1.2. Spraying for this phase of the project was performed by the author at the University of British Columbia. The mechanical properties for this material are discussed further in Chapter 5 \( (l_r = 32 \text{ mm}, \sigma_{ult} = 104 \text{ MPa}, E = 10.5 \text{ GPa}, \Delta_r = 1.43 \%, V_r = 18 \%) \). Application thickness was approximately 6 mm.
Averaged test results for the Phase C specimens are compiled in Table 6.10, with the values reported being the averages of all the beams in each group. Full details are included in Appendix E.

<table>
<thead>
<tr>
<th>Retrofit Schem</th>
<th>Retrofit Details**</th>
<th>Rebar Details</th>
<th>Stiffness</th>
<th>Strength</th>
<th>Absorbed Energy</th>
<th>Failure Mode*</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Initial (kN/mm)</td>
<td>Change (%)</td>
<td>Peak (kN)</td>
<td>Change (%)</td>
</tr>
<tr>
<td>None</td>
<td>None</td>
<td>Straight</td>
<td>18.7</td>
<td>-</td>
<td>47</td>
<td>-</td>
</tr>
<tr>
<td>A</td>
<td>Full</td>
<td>Straight</td>
<td>22.3</td>
<td>19</td>
<td>110</td>
<td>133</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Bent</td>
<td>21.3</td>
<td>14</td>
<td>110</td>
<td>133</td>
</tr>
<tr>
<td>C</td>
<td>Full</td>
<td>Straight</td>
<td>27.6</td>
<td>47</td>
<td>141</td>
<td>200</td>
</tr>
<tr>
<td></td>
<td>Abbr</td>
<td>Bent</td>
<td>30.8</td>
<td>65</td>
<td>141</td>
<td>200</td>
</tr>
<tr>
<td>D</td>
<td>Full</td>
<td>Straight</td>
<td>30.9</td>
<td>65</td>
<td>287</td>
<td>510</td>
</tr>
<tr>
<td></td>
<td>Abbr</td>
<td>Straight</td>
<td>28.4</td>
<td>52</td>
<td>283</td>
<td>502</td>
</tr>
</tbody>
</table>

* - Failure modes: Y = Rebar Yield, S = Shear, C = Concrete Crushing, B = GFRP Bond, P = GFRP Plate
** - Full = Full length FRP on bottom, Abbr = FRP does not extend over support.

Examining the stiffness values reveals that, again, the Scheme A retrofit has very little effect on member stiffness while both the Scheme C and D retrofits induce significant increases. Considering the different variables investigated in this phase of the project, we can see that the Scheme A specimens exhibit a slight decrease in stiffness between the straight and hooked reinforcement configurations (for the full length GFRP application) while the same comparison in the Scheme C specimens shows completely the opposite trend.
Looking at the effect of GFRP length, we can see in the Scheme C specimens that the abbreviated length produced a higher stiffness than the corresponding full length version. The Scheme D beams, on the other hand, exhibited a drop in stiffness between the full length and the abbreviated GFRP specimens. It appears that neither factor can be said to produce a consistent, significant effect on initial member stiffness. The overall improvement in stiffness for each of these schemes was more pronounced than that of the Phase A specimens, however.

The peak load results exhibit a similar result. Here, there is no significant difference between full length and abbreviated GFRP or between straight and hooked reinforcement for any of the retrofit schemes. What is interesting to note, however, is the effect of retrofit scheme on the ultimate strength of the members.

Examining the energy absorbed up to peak load shows that the three retrofit schemes produced average increases of 567%, 719% and 6525% for Schemes A, C and D, respectively. The very large increase exhibited by the Scheme D specimens dwarfs that of the other two schemes, making comparison difficult.

Since no consistent, significant difference was observed between the two GFRP lengths investigated or between the two shapes of reinforcement used, these variables were discounted and the average test results were recalculated to include all specimens in each retrofit scheme. These revised results are shown in Table 6.11 below.
Table 6.11: Shear strengthening - Phase 3 combined test results (FRP strength effect).

<table>
<thead>
<tr>
<th>Retrofit Scheme Details</th>
<th>Retrofit Details</th>
<th>Stiffness</th>
<th>Strength</th>
<th>Absorbed Energy</th>
<th>Failure Mode*</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Rebar Details</td>
<td>Initial (kN/mm)</td>
<td>Change (%)</td>
<td>Peak (kN)</td>
<td>Change (%)</td>
</tr>
<tr>
<td>None</td>
<td>None</td>
<td>18.7</td>
<td>-</td>
<td>47</td>
<td>-</td>
</tr>
<tr>
<td>A</td>
<td>Full</td>
<td>21.8</td>
<td>16</td>
<td>110</td>
<td>133</td>
</tr>
<tr>
<td>C</td>
<td>Both</td>
<td>30.8</td>
<td>64</td>
<td>144</td>
<td>207</td>
</tr>
<tr>
<td>D</td>
<td>Both</td>
<td>30.1</td>
<td>61</td>
<td>285</td>
<td>507</td>
</tr>
</tbody>
</table>

* - Failure modes: Y = Rebar Yield, S = Shear, C = Concrete Crushing, B = GFRP Bond, P = GFRP Plate

The primary objective of this testing phase was to determine the effect of increasing the FRP strength from 35 MPa (in the initial study) to 104 MPa in this portion of the study. To allow a direct comparison of these two materials, Table 6.12 summarizes the change in results between the two sets of specimens. Note that for consistency, only the values corresponding to the undamaged/strengthened beams were used from the initial study results.

The control specimens from each testing phase are virtually identical so a direct comparison is possible. As expected, all of the specimens from this testing phase significantly outperformed their counterparts from the initial study, with the exception of the energy absorption values of the Scheme C beams. This apparent anomaly is related to the change in stiffness and failure mode exhibited by these members, as discussed below.
<table>
<thead>
<tr>
<th>Retrofit Scheme</th>
<th>Initial Stiffness (kN/mm)</th>
<th>Peak Strength (kN)</th>
<th>Absorbed Energy to Peak (N-m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>FRP Strength</td>
<td>Change (%)</td>
<td>FRP Strength</td>
</tr>
<tr>
<td>None</td>
<td>35</td>
<td>104</td>
<td>17.5</td>
</tr>
<tr>
<td>A</td>
<td>15.6</td>
<td>21.8</td>
<td>+40</td>
</tr>
<tr>
<td>C</td>
<td>23.6</td>
<td>30.8</td>
<td>+31</td>
</tr>
<tr>
<td>D</td>
<td>23.6</td>
<td>30.1</td>
<td>+28</td>
</tr>
</tbody>
</table>

* - Failure modes: Y = Rebar Yield, S = Shear, C = Concrete Crushing, B = GFRP Bond, P = GFRP Plate

These improvements are a direct result of the change in GFRP retrofit material. The new material has a higher strength and, apparently, produces a better bond with the concrete surface, as evidenced by the failure mode exhibited in the Scheme C specimens. The typical mode of failure is no longer debonding (though it did still occur in some specimens) but crushing of the concrete in the compression zone at the upper surface of the beams. The load limit at which debonding occurs is now very close to that required to induce concrete crushing, which is why some specimens did still debond while others failed in crushing.

Even though the Scheme A specimens still failed due to debonding, they also registered a large increase in load carrying ability over their initial study counterparts. This is also a direct result of the improved GFRP-concrete bond strength, which is capable of withstanding higher loads. The Scheme D specimens, which exhibited the largest increase by far, still failed due to tensile fracture of the bottom GFRP and were thus able to take full advantage of...
the increased GFRP strength.

From an energy absorption standpoint, only the Scheme C specimens were not significantly improved by the change in FRP properties, probably due to a change in failure mode. Though these beams failed at higher load level, they also exhibited an increased stiffness, resulting in a leftward shift of the load-deflection curve and a smaller AUC.

Looking at the typical load-deflection curves for each retrofit scheme from the higher strength FRP specimens (Figure 6.23), it is evident that the Scheme D retrofit is far and away the most effective approach. Also evident is the increase in stiffness (described previously) between the Scheme C/D specimens and the control/Scheme A members.

Figure 6.23: Shear strengthening - Phase 3 typical load-deflection curves.
Adding the load-deflection curves from the initial study specimens indicates the large improvements produced by the superior FRP strength used in this part of the study (Figure 6.24). The increased stiffness induced by the higher strength FRP is also visible in this figure.

![Load-Deflection Curves](image)

**Figure 6.24:** Shear strengthening - comparison of typical load-deflection curves for different FRP strengths.

Figures 6.25 and 6.26 illustrate the large differences in fracture energy absorption between the control specimens and the retrofitted beams for the specimens retrofitted with the higher strength FRP. Figure 6.25 includes the entire range of results up to the conclusion of testing for the Scheme D specimens while Figure 6.26 is a magnification of the initial portions of this graph.
Figure 6.25: Shear strengthening - Phase 3 typical energy absorption curves (104 MPa FRP).

Figure 6.26: Shear strengthening - Phase 3 typical energy absorption curves (104 MPa FRP, magnified).
Here we can see that the Scheme C specimen curve is again identical to that of the Scheme D specimens until it reaches its ultimate load. The deviation from the Scheme D curve occurs during the post-peak performance region and its continuation and shape is a direct result of the concrete crushing failure mode. This is a relatively ductile failure mode and (as evident in the figure) has the ability to absorb significant amounts of fracture energy even after reaching its ultimate load.
6.2.5.4 - Effect of FRP Thickness

As described in Chapter 4, the sprayed GFRP application process is fairly straightforward with the exception of the operator's ability to judge the thickness of the applied material. Not only is it difficult to determine the amount of material that has been placed on the surface, the additional step of roller compacting the material complicates the issue further. The amount of compaction achieved by a given amount of rolling at a specified pressure varies with fiber length.

The shorter fiber lengths tend to be sensitive to excessive pressure on the roller and thus need more passes with less applied force. The longer fibers, on the other hand, can be rolled out very quickly using more roller pressure. Eventually, all of the fiber lengths can be compacted to the same degree (as indicated in the fiber volume fraction measurements in Section 5.4). The question becomes; how can the thickness of the material be judged during the spraying operation.

During initial experimentation with the spraying equipment it was discovered that the amount of material deposited before rolling should be limited. Too much built up material makes rolling very difficult as the fibers themselves tend to move around during rolling. Applying the GFRP in stages, rolling out each stage immediately after it is applied, makes the process of building up thicker coatings more feasible.

In this phase of the study, an attempt was made to quantify the rate at which the material was
deposited and to examine the effect of variations in the applied thickness. Several different GFRP thicknesses were examined with those thicknesses determined by the application process itself. A series of 17 beams was cast and divided into 9 groups, as indicated in Table 6.13. Only retrofit Schemes C and D were used in this phase of the project and all GFRP contained 32 mm fibers.

<table>
<thead>
<tr>
<th>Retrofit Scheme</th>
<th>GFRP Thickness (mm)</th>
<th>Number of Specimens</th>
</tr>
</thead>
<tbody>
<tr>
<td>None</td>
<td>N/A</td>
<td>3</td>
</tr>
<tr>
<td>C</td>
<td>3.5</td>
<td>2</td>
</tr>
<tr>
<td>C</td>
<td>4.5</td>
<td>2</td>
</tr>
<tr>
<td>C</td>
<td>6.0</td>
<td>2</td>
</tr>
<tr>
<td>C</td>
<td>8.0</td>
<td>2</td>
</tr>
<tr>
<td>D</td>
<td>3.5</td>
<td>2</td>
</tr>
<tr>
<td>D</td>
<td>4.5</td>
<td>1</td>
</tr>
<tr>
<td>D</td>
<td>6.0</td>
<td>2</td>
</tr>
<tr>
<td>D</td>
<td>8.0</td>
<td>1</td>
</tr>
</tbody>
</table>

Table 6.13 indicates the nominal thickness in millimeters for each group of specimens. However, these values were measured after GFRP application was complete and are thus a function of the process, not defined values. In the spraying operation itself, there are two variables which affect the final thickness of the material, the number of passes and the number of layers. While spraying the GFRP, the operator moves the gun in a back and forth motion while at the same time moving from one end of the specimen to the other. When the
other end of the specimen is reached, it is considered one full pass. Basically, this means the entire surface has been covered by one layer of the spray with minimal overlap at the edges.

As mentioned previously, there is a limit to the amount of material that can be built up before the rolling process starts to becomes difficult. This upper limit appears to be about four passes, after which the GFRP must be is rolled out. This constitutes one layer. Thus, the spraying operation consists of a number of passes, followed by the rolling operation, more passes, another rolling operation, etc. In other words, each layer is made up of several passes, in this case either two, three or four. To create the different thicknesses needed for the study, four combinations of passes and layers were used. These combinations are detailed in Table 6.14.

<table>
<thead>
<tr>
<th>Number of Layers</th>
<th>Number of Passes</th>
<th>Equivalent Thickness (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>3</td>
<td>3.5</td>
</tr>
<tr>
<td>2</td>
<td>2</td>
<td>4.5</td>
</tr>
<tr>
<td>2</td>
<td>3</td>
<td>6.0</td>
</tr>
<tr>
<td>2</td>
<td>4</td>
<td>8.0</td>
</tr>
</tbody>
</table>

The smallest thickness of 3.5 mm was chosen from experience as the minimum thickness that could be applied consistently. Anything less than this made it very difficult to maintain thickness consistency due to the small amount of material involved and the variability in
material deposition inherent in the spraying process. The rolling process was capable of only a limited amount of material redistribution since the fibers resisted such movement.

The other thicknesses were determined simply by incrementally increasing the total number of passes (i.e. number of layers x number of passes) from the original 3 to 4, 6 and finally 8. Thicknesses beyond 8 mm were not considered due to their overly large size relative to the width of the specimen. Measurements indicated that the variation in material thickness was typically within ± 0.5 mm. Test results for the specimens in this phase of the project are shown below in Table 6.15. Further details are included in Appendix F.

<table>
<thead>
<tr>
<th>Retrofit Scheme</th>
<th>GFRP Thickness</th>
<th>Stiffness</th>
<th>Strength</th>
<th>Absorbed Energy</th>
<th>Failure Mode*</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Initial (kN/mm)</td>
<td>Change (%)</td>
<td>Peak (kN)</td>
<td>Change (%)</td>
</tr>
<tr>
<td>None</td>
<td>N/A</td>
<td>22.0</td>
<td>-</td>
<td>46</td>
<td>-</td>
</tr>
<tr>
<td>C</td>
<td>3.5</td>
<td>24.0</td>
<td>9</td>
<td>125</td>
<td>171</td>
</tr>
<tr>
<td></td>
<td>4.5</td>
<td>24.5</td>
<td>11</td>
<td>131</td>
<td>184</td>
</tr>
<tr>
<td></td>
<td>6.0</td>
<td>33.7</td>
<td>53</td>
<td>145</td>
<td>215</td>
</tr>
<tr>
<td></td>
<td>8.0</td>
<td>31.9</td>
<td>45</td>
<td>155</td>
<td>237</td>
</tr>
<tr>
<td>D</td>
<td>3.5</td>
<td>26.5</td>
<td>20</td>
<td>163</td>
<td>254</td>
</tr>
<tr>
<td></td>
<td>4.5</td>
<td>28.6</td>
<td>30</td>
<td>182</td>
<td>296</td>
</tr>
<tr>
<td></td>
<td>6.0</td>
<td>34.2</td>
<td>55</td>
<td>228</td>
<td>395</td>
</tr>
<tr>
<td></td>
<td>8.0</td>
<td>37.9</td>
<td>72</td>
<td>277</td>
<td>502</td>
</tr>
</tbody>
</table>

* - Failure modes: Y = Rebar Yield, S = Shear, C = Concrete Crushing, B = GFRP Bond, P = GFRP Plate
The initial member stiffness results show increasing stiffness as the GFRP thickness increases, with the sole aberration being the 6.0 mm thickness in the Scheme C retrofit. This result appears to be excessively high and does not seem to fit into the trend exhibited by the other Scheme C specimens. Otherwise, the remaining Scheme C specimens, and the Scheme D members follow a definitive upward trend. This type of consistent trend is also apparent in the peak load results. In this case, both retrofit schemes exhibited consistent trends, though the rate of increase was much higher in the Scheme D specimens. All of the specimens showed very large improvements over the control beams.

The increases in energy absorption were very large for all specimens, when compared to the control beams. The variation between thicknesses was quite low in the Scheme C specimens relative to the Scheme D members.

Typical load-deflection curves for the various combinations of retrofit scheme and GFRP thickness are depicted in Figure 6.27. Both retrofit schemes are included to indicate the significant difference between them. To provide further clarity, Figure 6.28 depicts only beams from the Scheme C retrofit while Figure 6.29 shows only the Scheme D beams. In both cases, there is a definite trend of increasing performance with increasing GFRP thickness. The control curve is also included in both figures for comparison purposes.
**Figure 6.27:** Shear strengthening - Phase 4 typical load-deflection curves by FRP thickness.

**Figure 6.28:** Shear strengthening - Phase 4 typical load-deflection curves by FRP thickness (Scheme C).
Figure 6.29: Shear strengthening - Phase 4 typical load-deflection curves by FRP thickness (Scheme D).

Figure 6.30 shows the initial portion of the fracture energy curves for the various specimen configurations. There is a significant improvement in absorption rate over the control specimens for all beams. Initially, all of the retrofitted beams absorb energy at the same rate, though in general the Scheme D retrofit curves appear to exhibit steeper curves than their Scheme C counterparts. There is also a consistently increasing trend corresponding to the increases in GFRP thickness, which becomes more accentuated in the post-peak performance of the beams.
The failure modes of this set of specimens tended to be much more consistent than previous sets. Of the 14 retrofitted beams tested, only one failed due to debonding of the GFRP. Otherwise, all of the Scheme C specimens failed due to crushing of the concrete in the compression zone while the Scheme D specimens failed due to fracture of the GFRP in the tension zone.
6.2.5.5 - Effect of Fiber Length

The material properties of the sprayed GFRP material are directly related to the length of the fibers, as indicated in Chapter 5. Thus, it is desirable to produce the longest fibers possible. The initial shear strengthening beams discussed in Section 6.2.5.2 were sprayed by GU Manhole Liners Ltd. using 8 mm fibers. Up to this point in the project, all the beams retrofitted at the University of British Columbia were done so using a fiber length of 32 mm. It was decided to see if the fiber length could be further increased and what effects the different fiber lengths would have on test results.

As previously mentioned in Section 4.2, fiber length is determined within the chopper unit of the spraying equipment. The chopper rotor has a series of blades embedded around its circumference and the length of chopped fiber is entirely dependent upon the number and spacing of these blades. The existing rotor that came installed in the chopper gun possessed three blade locations (32 mm apart) and was thus limited to producing 32 mm or 96 mm fibers, assuming a constant fiber length was desired.

First, an attempt was made to increase the fiber length by removing two of the blades and spraying the maximum 96 mm fiber length. This experiment was entirely unsuccessful as the fiber would not exit the chopper housing properly, simply building up inside it and clogging the exit. A second attempt was made with two of the blades in place. This should result in a mixture of 32 mm and 64 mm fibers. Again, the longer fibers could not be sprayed due to clogging of the chopper housing.
During consultation with the spraying equipment manufacturer, it was discovered that alternative cutting rotors designs were available. A multi-length rotor was obtained which had embedment positions for a maximum of 12 blades. This would allow production of several different constant fiber lengths, ranging from 8 mm to 96 mm, depending upon the number of blades present. These combinations are shown in Table 6.16.

<table>
<thead>
<tr>
<th>Number of Blades</th>
<th>Fiber Length (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>12</td>
<td>8</td>
</tr>
<tr>
<td>6</td>
<td>16</td>
</tr>
<tr>
<td>3</td>
<td>32</td>
</tr>
<tr>
<td>2</td>
<td>48</td>
</tr>
<tr>
<td>1</td>
<td>96</td>
</tr>
</tbody>
</table>

As previously mentioned, use of the 96 mm length was not feasible due to clogging problems so the next step was to try the next longest constant length of 48 mm. This fiber length did indeed work and was thus added to this phase of the testing program.

There was a total of 15 beams cast for use in this phase, all cast from normal strength (45 MPa) concrete. These specimens were divided into 8 groups to allow investigation of four different fiber lengths (8, 16, 24 and 48 mm). The 32 mm fiber length was not included since it was the standard length used previously on the Phase 3 specimens and did not need to be
repeated. Additionally, only the Scheme C and D retrofits were implemented in this phase.

The 8 specimen groups resulting from these variables are detailed below in Table 6.17.

<table>
<thead>
<tr>
<th>Retrofit Scheme</th>
<th>Fiber Length (mm)</th>
<th>Number of Specimens</th>
</tr>
</thead>
<tbody>
<tr>
<td>None</td>
<td>N/A</td>
<td>3</td>
</tr>
<tr>
<td>C</td>
<td>8</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>16</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>24</td>
<td>2</td>
</tr>
<tr>
<td>D</td>
<td>8</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>16</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>24</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>48</td>
<td>1</td>
</tr>
</tbody>
</table>

Again, the total number of beams was limited due to mixer size. Results from the testing of these specimens are provided in Table 6.18 while full details for these specimens have been included in Appendix G. Material properties of the GFRP for the various fiber lengths has been previously discussed in Chapter 5.
<table>
<thead>
<tr>
<th>Retrofit Scheme</th>
<th>Fiber Length</th>
<th>Stiffness</th>
<th>Strength</th>
<th>Absorbed Energy</th>
<th>Failure Mode*</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Initial (kN/mm)</td>
<td>Change (%)</td>
<td>Peak (kN)</td>
<td>Change (%)</td>
</tr>
<tr>
<td>None</td>
<td>N/A</td>
<td>19.6</td>
<td>-</td>
<td>49</td>
<td>-</td>
</tr>
<tr>
<td>C</td>
<td>8</td>
<td>26.4</td>
<td>35</td>
<td>132</td>
<td>170</td>
</tr>
<tr>
<td></td>
<td>16</td>
<td>28.5</td>
<td>46</td>
<td>136</td>
<td>179</td>
</tr>
<tr>
<td></td>
<td>24</td>
<td>30.8</td>
<td>57</td>
<td>149</td>
<td>205</td>
</tr>
<tr>
<td>D</td>
<td>8</td>
<td>31.3</td>
<td>60</td>
<td>205</td>
<td>320</td>
</tr>
<tr>
<td></td>
<td>16</td>
<td>33.5</td>
<td>71</td>
<td>227</td>
<td>366</td>
</tr>
<tr>
<td></td>
<td>24</td>
<td>34.1</td>
<td>74</td>
<td>258</td>
<td>429</td>
</tr>
<tr>
<td></td>
<td>48</td>
<td>35.3</td>
<td>80</td>
<td>269</td>
<td>452</td>
</tr>
</tbody>
</table>

* - Failure modes: Y = Rebar Yield, S = Shear, C = Concrete Crushing, B = GFRP Bond, P = GFRP Plate

All of the retrofits produce a significant increase in member stiffness over the control specimens and increases in fiber length resulted in a corresponding increase in beam stiffness. Looking back at the results from Phase 3 (Table 6.10), we find that the 32 mm fibers produced an increase of 64% for Scheme C but only 61% for Scheme D. The first figure fits well into the trend shown above in Table 6.18 but the Scheme D value is very low, falling between the 8 mm and 16 mm fiber lengths. This value is probably an anomaly since it was low compared to the other Phase 3 results as well (i.e. lower than the Scheme C retrofit). Though there appears to be no overlap between the Scheme C and the Scheme D specimens, it should be noted that there was no 48 mm fiber length used in the Scheme C retrofit. Following the trend indicated in Table 6.18, this length should produce a higher
stiffness than that exhibited by the 8 mm Scheme D specimens.

Examining the peak load results, we find that once again a consistent trend appears and that there are increasingly larger improvements in peak load as the fiber length is increased. Looking back, the Phase 3 results (Table 6.10) reveal that the 32 mm fibers produced an increase of 207% for Scheme C and 507% for Scheme D. Again, the Scheme C figure fits well into the trend shown in Table 6.18 but this time the Scheme D value is very high, exceeding even the result for the 48 mm fiber length.

Going on to the energy absorption results, we once again see the same trend, with the exception of one anomaly in the Scheme C specimens where the 16 mm fiber length appears to be too low. The 32 mm results from Phase 3 indicate that the Scheme C members improved by 719% and Scheme D by 6525%. Now, the Scheme D result appears to fit well into the trend while the Scheme C result is low.

Typical load-deflection curves for each combination of retrofit scheme and fiber length are shown in Figure 6.31. Once again, the trend of improved performance with increased fiber length is evident, as is the large difference between the Scheme C and Scheme D approaches. Also evident is the increase in initial member stiffness over the control specimens, though the difference between retrofit schemes appears to be much less significant than this initial improvement.
Examining the cumulative energy absorbed by the specimens at equivalent deflection levels, as shown in Figure 6.32 indicates that the initial rate of absorption is very similar, regardless of retrofit scheme, though this rate is much higher than the control specimens. The post-peak performance of the specimens indicates that each retrofit scheme has a unique, though consistent trend. All of the Scheme C specimens remain parallel, as do the Scheme D members, though they are significantly different from each other. The higher slope of the Scheme D specimens during this stage indicates that they are capable of absorbing larger amounts of fracture energy during specimen failure.
The failure modes of the beams in this phase also exhibited specific trends. For the Scheme C specimens, the 8 and 16 mm fiber lengths failed due to debonding of the GFRP while the 24 mm specimens failed due to concrete crushing. Also consistent with this trend was the 32 mm Scheme C specimens from Phase 3, which also failed due to concrete crushing. It appears that an increase in fiber length improves the GFRP-concrete bond, likely due to an improved stress transfer mechanism. The Scheme D specimens, including the 32 mm fiber length from Phase 3, all failed due to tensile fracture of the GFRP applied to the bottom face of the beam.
6.2.5.6 - Comparison of Fabric Wraps & Sprayed FRP

6.2.5.6.1 - Fabric Wrapped Specimens

As discussed in Chapter 2, the majority of research into the use of fiber reinforced polymers for the rehabilitation of existing concrete structures has revolved around continuous fiber fabrics. In the case of beam strengthening, these fabrics are either bonded to the soffit of the member to provide flexural reinforcement or applied to the side (usually at a 45° angle) to act as shear strengthening.

To allow comparison of the sprayed GFRP system with such wrapping techniques, a series of 12 beams was cast and strengthened with continuous glass fiber fabrics. These beams are detailed below in Table 6.19. Since the number of beams cast required they be cast in two separate mixes, one control specimen was taken from each mix to ensure uniformity. Each of the remaining specimens was retrofitted with a different combination of fabric fiber orientation and number of layers of fabric. These combinations are indicated in Table 6.12 as well.

The fabrics used in this project phase are discussed in Section 3.3.5. These fabrics were applied bonded with the same polyester resin used in the spraying operation. The vinyl ester coupling agent discussed in Section 3.3.3 was also applied. The application procedure commenced with the application of the vinyl ester coupling agent as described in Section 6.2.5.3. Once it reached the tacky stage, the surface was sprayed with a light coating of the
polyester resin using the normal spraying apparatus. The fabric was placed on the surface and rolled out to ensure maximum wetting of the material. Another coating of the polyester resin was sprayed on top of the fabric and it was again rolled out to remove any entrapped air and to force the fabric fully against the surface of the concrete. Once curing was complete, any material projecting beyond the edge of the beam was ground off.

<table>
<thead>
<tr>
<th>Retrofit Scheme</th>
<th>Fabric Fiber Orientation</th>
<th>Number of Layers</th>
<th>Number of Specimens</th>
</tr>
</thead>
<tbody>
<tr>
<td>None</td>
<td>N/A</td>
<td>N/A</td>
<td>2</td>
</tr>
<tr>
<td>C</td>
<td>±45°</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>0/90°</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>1</td>
</tr>
<tr>
<td>D</td>
<td>±45°</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>0/90°</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>1</td>
</tr>
</tbody>
</table>

It should be noted here that these fabrics were not capable of wrapping around the corners of the beams. Applying a 45° bevel, as was done with the sprayed material (refer to Section 6.1.3.), was not an option due to the much thinner profile of the fabrics. A single layer of fabric created a layer approximately 1.5 mm thick. Even beveled, the corner was too sharp.
for the fabric to wrap around. As a result, adjacent sides were not overlapped, simply butted together.

The test results for each of the fabric wrapped beams are presented in Table 6.20. Again, the control values represent the average of two specimens while the remaining results represent one beam only. Further details for these specimens can be found in Appendix H.

Examining the stiffness values indicates that the initial member stiffness increases to some extent, though the addition of successive layers does not produce a further increase of the

Table 6.20: Shear strengthening - Phase 6 test results (fabric wrapping).

<table>
<thead>
<tr>
<th>Retrofit Scheme</th>
<th>Fabric Fiber Orient</th>
<th>Number of Layers</th>
<th>Stiffness</th>
<th>Strength</th>
<th>Absorbed Energy</th>
<th>Failure Mode*</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Initial (kN/mm)</td>
<td>Change (%)</td>
<td>Peak (kN)</td>
<td>Change (%)</td>
</tr>
<tr>
<td>None</td>
<td>N/A</td>
<td>N/A</td>
<td>19.3</td>
<td>-</td>
<td>42</td>
<td>-</td>
</tr>
<tr>
<td>C</td>
<td>±45</td>
<td>1</td>
<td>21.3</td>
<td>10</td>
<td>97</td>
<td>131</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>21.8</td>
<td>13</td>
<td>117</td>
<td>179</td>
</tr>
<tr>
<td></td>
<td>0/90</td>
<td>1</td>
<td>21.3</td>
<td>10</td>
<td>96</td>
<td>129</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>21.8</td>
<td>13</td>
<td>123</td>
<td>193</td>
</tr>
<tr>
<td>D</td>
<td>±45</td>
<td>1</td>
<td>19.2</td>
<td>0</td>
<td>91</td>
<td>117</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>22.5</td>
<td>17</td>
<td>110</td>
<td>162</td>
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<td></td>
<td></td>
<td>3</td>
<td>23.4</td>
<td>21</td>
<td>117</td>
<td>179</td>
</tr>
<tr>
<td></td>
<td>0/90</td>
<td>1</td>
<td>21.5</td>
<td>12</td>
<td>100</td>
<td>138</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>23.6</td>
<td>22</td>
<td>117</td>
<td>179</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>27.3</td>
<td>42</td>
<td>115</td>
<td>174</td>
</tr>
</tbody>
</table>

* - Failure modes: Y = Rebar Yield, S = Shear, C = Concrete Crushing, B = GFRP Bond, P = GFRP Plate
same magnitude, except for the Scheme D specimens with 3 layers of 0/90 fabric. The remaining values are all either equivalent to or greater than the control specimens but this improvement is marginal at best and certainly cannot be considered significant.

The peak load values, on the other hand, did show a marked increase due to the retrofitting. All of the strengthened beams showed large increases in ultimate load, with that improvement getting larger as the number of layers increased. There did not, however, seem to be any significant difference between the Scheme C and Scheme D members. This is due to the failure process exhibited by the beams. In most cases, the bonded fabrics separated at the corners and thus the resulting pieces were forced to act independently of each other. This was a direct result of the inability of these fabrics to wrap around the 90° corners at the edges of the beams.

Examining the energy absorbed by the specimens up to the peak load, the same trend becomes evident. All of the specimens exhibit large increases, with the addition of extra layers improving performance even further. Of particular interest here is the superior performance of the Scheme C specimens retrofitted with three layers of fabric. These specimens even outperformed the Scheme beams with a similar number of layers.

Another unexpected result is the similarity in performance between the fabrics with fibers oriented at ±45° and those oriented at 0/90°. As depicted in Figure 6.33 for the Scheme C members and in Figure 6.34 for Scheme D, there is very little difference between the two orientations. This point is particularly applicable to the Scheme C specimens, which are
virtually identical save for the post-peak performance of the members retrofitted with three layers of fabric.

![Figure 6.33: Shear strengthening - Phase 6 load-deflection curves for Scheme C beams (fabric wrapping).](image)

This discrepancy can be accounted for by the difference in failure mode exhibited by these two specimens. The $\pm 45^\circ$ retrofit failed due to crushing of the concrete in the compression zone of the beam (prior to rebar yielding) while the $0/90^\circ$ retrofit failed due to debonding of the GFRP. The crushing failure mode is more ductile and thus extends the load-deflection curve after the ultimate load is reached. Both of the single layer retrofits failed in shear. The Scheme D specimens exhibited very similar load-deflection curves regardless of the number of layers added.
Looking at the energy absorbed by the Scheme C specimens throughout their testing cycle (Figure 6.35), we can see that all of the specimens absorbed energy at a nearly identical rate up until their individual points of failure. The change in rate at the point of failure was also the same for each member, with the exception of the beam with three layers of ±45° fabric. Again, this difference in post-peak performance was due to the change in failure mode to crushing.
Examining a similar set of curves for the Scheme D specimens (Figure 6.36) reveals that the absorption rate among these specimens is not consistent. In fact, it continually increases firstly as the fabric fiber orientation changes from $\pm 45^\circ$ to $0/90^\circ$ and secondly as the number of layers increases. The sole exception to this pattern is the specimen retrofitted with three layers of the $0/90^\circ$ fabric. The energy absorption curves for this beam starts out at a higher slope than the rest but appears reach its ultimate load prematurely and then exhibits a much lower post-peak slope than the other retrofits.
Figures 6.37 and 6.38 show similar energy absorption curves, but this time divided into specimens retrofitted with one layer or three layers of fabric, respectively. These figures show that there is very little difference in energy absorption rate between the two different fabrics or between the two different retrofit Schemes.

The failure mode exhibited by these specimens was very consistent. With a single exception (the Scheme C specimen retrofitted with three layers of ±45° fabric) all of the retrofitted beams failed due to debonding of the GFRP fabric. This failure mode is very common with continuous fiber fabrics and was indeed the prevalent failure mode reported by other researchers, as mentioned previously in Chapter 2.
Figure 6.37: Shear strengthening - Phase 6 energy absorption curves for 1 layer (fabric wrapping).

Figure 6.38: Shear strengthening - Phase 6 energy absorption curves for 3 layers (fabric wrapping).
6.2.5.6.2 - Sprayed Specimens

In order to allow a meaningful comparison between the fabric wrapped specimens from the previous section and beams retrofitted with the sprayed FRP, it was deemed necessary to prepare a new set of sprayed beams which would represent the optimal performance from the preceding work. Thus, these specimens were retrofitted with a 3.5 mm thick sprayed FRP coating (minimum practical thickness from Section 6.2.5.4) which contained 48 mm fibers (maximum fiber length from Section 6.2.5.5). This represents the first set of specimens containing this combination.

Though only retrofit Schemes C and D were used in the fabric wrapping portion of the project, Scheme A was also included here. A series of 12 beams was cast with normal strength (45 MPa) concrete and divided into 4 groups, as detailed below in Table 6.21.

<table>
<thead>
<tr>
<th>Retrofit Scheme</th>
<th>Number of Specimens</th>
</tr>
</thead>
<tbody>
<tr>
<td>None</td>
<td>2</td>
</tr>
<tr>
<td>A</td>
<td>3</td>
</tr>
<tr>
<td>C</td>
<td>4</td>
</tr>
<tr>
<td>D</td>
<td>3</td>
</tr>
</tbody>
</table>

Table 6.21: Shear strengthening - Phase 6 specimen groups (sprayed).
The smallest practical application thickness of 3.5 mm was chosen for this portion of the study in order to minimize the ratio of GFRP thickness to beam width. The maximum available fibre length of 48 mm was chosen due to its superior performance in previous testing of both retrofitted beams and GFRP tensile coupons. The retrofit schemes were limited to the chosen three since Scheme E is practical only for flexural strengthening and Scheme B is not feasible in real-life applications. The results from this phase of testing are presented in Table 6.22 while more complete details have been included in Appendix H.

<table>
<thead>
<tr>
<th>Retrofit Scheme</th>
<th>Stiffness</th>
<th>Strength</th>
<th>Absorbed Energy</th>
<th>Failure Mode*</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Initial (kN/mm)</td>
<td>Change (%)</td>
<td>Peak (kN)</td>
<td>Change (%)</td>
</tr>
<tr>
<td>None</td>
<td>20.2</td>
<td>-</td>
<td>48</td>
<td>-</td>
</tr>
<tr>
<td>A</td>
<td>21.3</td>
<td>6</td>
<td>96</td>
<td>103</td>
</tr>
<tr>
<td>C</td>
<td>24.1</td>
<td>20</td>
<td>127</td>
<td>167</td>
</tr>
<tr>
<td></td>
<td>24.4</td>
<td>21</td>
<td>137</td>
<td>188</td>
</tr>
<tr>
<td>D</td>
<td>25.2</td>
<td>25</td>
<td>199</td>
<td>318</td>
</tr>
</tbody>
</table>

* Failure modes: Y = Rebar Yield, S = Shear, C = Concrete Crushing, B = GFRP Bond, P = GFRP Plate

The results indicated in Table 6.22 for the Scheme C specimens were divided into two groups, with two beams in each group. This division was made due to a difference in failure mode between the two groups, which resulted in a significant difference in member properties between the groups. The first group failed due to debonding of the GFRP while the second group failed due to crushing of the concrete in the compression zone. It is the
latter failure mode which is of the most interest since it allows the GFRP to reach its maximum potential effect. Within each group, the two beams behaved very similarly. In the following figures, the values shown for the Scheme C retrofit represent the testing results from the latter group (crushing failure) unless otherwise noted.

Test results for the sprayed beams indicate, once again, that the Scheme A retrofit was unable to produce a significant improvement since only the sides were sprayed. Applying the GFRP to the bottom face, as in the Scheme C and D arrangements, did induce a significant increase in stiffness. Peak load results showed were large improvements for all three retrofit schemes while the improvements in energy absorption up to peak load were even more dramatic, with extremely large gains delivered by the Scheme D arrangement (more than 5000% improvement over the control specimen).

Typical load-deflection curves for the different retrofit schemes are shown in Figure 6.39. Note that the two groups of Scheme C specimens are both included in this diagram in order to show the large difference in performance between them. Their behavior up to ultimate load is very similar, with the pair exhibiting crushing failures simply being able to carry higher loads. The other pair failed due to debonding before that ultimate load could be attained.
Comparing the Scheme C and Scheme D curves, it becomes apparent that the very large increase in absorbed energy observed in the Scheme D members is due to their ability to undergo large amounts of deformation beyond the point where the Scheme C specimens begin to fail.

The energy absorption curves are shown in Figure 6.40 while Figure 6.41 gives a magnified view of the initial portion of these curves. All of the retrofitted beams behaved similarly up to their individual failure points, though the control specimens absorbed energy at a significantly lower rate.
Figure 6.40: Shear strengthening - Phase 6 typical energy absorption curves (sprayed).

Figure 6.41: Shear strengthening - Phase 6 typical energy absorption curves (sprayed, magnified).
The typical failure modes exhibited by the specimens in this project phase were indicated previously in Table 6.22. Photographs showing examples of these failure modes are provided in the following four figures (6.42 - 6.45). Figure 6.42 indicates the typical shear failure mode exhibited by all of the control specimens, not only in this phase, but throughout the shear strengthening portion of this project. The Scheme A members failed due to debonding of the GFRP applied to the sides of the specimen. An example of this debonding failure is shown in Figure 6.43.

![Figure 6.42: Shear strengthening - typical shear failure of control specimens.](image)
Figure 6.43: Shear strengthening - typical debonding failure (Scheme A).

Figure 6.44 consists of a photograph depicting the crushing failure exhibited by two of the Scheme C retrofitted beams. These are the two beams whose results were presented throughout this section. This photo was taken well after the point of failure and the concrete crushing has progressed down to the reinforcement.

Finally, the GFRP tensile fracture exhibited by the Scheme D specimens is shown in Figure 6.45. This photo was also taken well after failure and buckling of the top GFRP coating is visible.
Figure 6.44: Shear strengthening - typical crushing failure (Scheme C).

Figure 6.45: Shear strengthening - typical FRP tensile failure (Scheme D).
6.2.5.6.3 - Comparison

Before comparing the actual test results from the fabric wrapped specimens and the sprayed specimens, it is first necessary to discuss the fiber volume fraction relative to each retrofit technique. There were two different fabrics used in the wrapping portion of the project; a 0/90° combination with a designated material weight of 23.0 oz/yd² (0.78 kg/m²) and a ±45° configuration with at 24.0 oz/yd² (0.81 kg/m²). A single layer of either material resulted in a final measured thickness of approximately 1.5 mm.

Assuming that the rolling out process was capable of providing full compaction (i.e. zero void volume fraction), the theoretical fiber volume fraction would be 19.8% for the 0/90° fabric and 20.7% for the ±45° fabric. Since these values are very close to that obtained with the spraying operation using the 48 mm fiber length (V_f = 19.0%), comparisons between the two techniques can be directly based on FRP thickness (i.e. both techniques will provide the same total fiber at the same thickness).

As for the assumption of achieving complete compaction for the fabrics; this is probably a valid assumption since a great deal of pressure could be applied to the roller throughout application the process. An interesting phenomenon was noticed during application of the first layer of fabric. After rolling, the fabric and resin combination became translucent, making the underlying concrete surface easily visible. This indicates that the air spaces between fibers were virtually eliminated. Successive layers made viewing of the concrete surface more difficult simply because of the greater thickness of material.
The results for the Scheme C retrofitted specimens, both fabric wrapped and sprayed, have been tabulated in Table 6.23 below.

<table>
<thead>
<tr>
<th>Retrofit Type</th>
<th>Fiber Orient</th>
<th>FRP Thickness (mm)</th>
<th>Stiffness</th>
<th>Strength</th>
<th>Energy Absorbed</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Initial (kN/mm)</td>
<td>Change (%)</td>
<td>Peak (kN)</td>
</tr>
<tr>
<td>Fabric Wrap</td>
<td>±45</td>
<td>1.5</td>
<td>21.3</td>
<td>10</td>
<td>97</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4.5</td>
<td>21.8</td>
<td>13</td>
<td>117</td>
</tr>
<tr>
<td></td>
<td>0/90</td>
<td>1.5</td>
<td>21.3</td>
<td>10</td>
<td>96</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4.5</td>
<td>21.8</td>
<td>13</td>
<td>123</td>
</tr>
<tr>
<td>Spray</td>
<td>Random</td>
<td>3.5</td>
<td>24.4</td>
<td>21</td>
<td>137</td>
</tr>
</tbody>
</table>

As evident from this data, the sprayed technique outperformed the fabric wrapping approach in virtually every category. The sprayed specimens reported the highest actual values for stiffness, peak load and energy absorbed of all the tested members. Looking at the percentage improvement figures, however, seems to indicate that wrapping with three layers (4.5 mm) produced a large improvement than the sprayed technique with respect to the energy absorbed up to peak load. There are two things that must be considered before such a conclusion can be made.

First, the control specimen from the fabric wrapping project phase exhibited significantly lower test results than its counterparts from the sprayed FRP phase. Though the stiffness and strength values were very similar between the two (19.3 vs. 20.2 kN/mm for stiffness and 42
vs. 48 kN for strength), the energy absorption values showed a dramatic difference (45 vs. 70 N·m). It was this low energy absorption value that resulted in the high percentage improvements for the wrapped specimens in Table 6.23. The second thing to keep in mind is the FRP thickness, which was 3.5 mm for the sprayed FRP technique and 4.5 mm for three layers of fabric.

For the Scheme D specimens, no such ambiguity exists. As shown in Table 6.24, the sprayed FRP retrofit simply outperformed the fabric wrapping in virtually every instance.

<table>
<thead>
<tr>
<th>Retrofit Type</th>
<th>Fiber Orient</th>
<th>FRP Thickness (mm)</th>
<th>Stiffness</th>
<th>Strength</th>
<th>Energy Absorbed</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Initial (kN/mm)</td>
<td>Change (%)</td>
<td>Peak (kN)</td>
</tr>
<tr>
<td>Fabric Wrap</td>
<td>±45</td>
<td>1.5</td>
<td>19.2</td>
<td>0</td>
<td>91</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3.0</td>
<td>22.5</td>
<td>17</td>
<td>110</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4.5</td>
<td>23.4</td>
<td>21</td>
<td>117</td>
</tr>
<tr>
<td></td>
<td>0/90</td>
<td>1.5</td>
<td>21.5</td>
<td>12</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3.0</td>
<td>23.6</td>
<td>22</td>
<td>117</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4.5</td>
<td>27.3</td>
<td>42</td>
<td>115</td>
</tr>
<tr>
<td>Spray</td>
<td>Random</td>
<td>3.5</td>
<td>25.2</td>
<td>25</td>
<td>199</td>
</tr>
</tbody>
</table>

In the stiffness results, the sprayed approach performed better than all of the wrapping configurations except three layers at 0/90°. For the other two properties (ultimate strength and energy absorption) the sprayed technique outperformed the fabric wrapping technique by
a very large margin.

From the actual test data, it appears that the sprayed FRP retrofit procedure can actually produce better results than a fabric wrapping approach using the same materials. To further support this observation, the load-deflection curves for the Scheme C specimens have been plotted in Figure 6.46 while those associated with the Scheme D specimens are shown in Figure 6.47.

Figure 6.46: Shear strengthening - Phase 6 typical load-deflection curves (wrap/spray comparison, Scheme C).
These figures indicate that the sprayed FRP technique produce better member performance in both the Scheme C and Scheme D retrofits. Another interesting result that became apparent during this comparison is the effect of the two techniques on the failure mode of the specimens. The majority of the fabric wrapped specimens failed due to debonding of the FRP from the concrete surface while this failure mode was virtually eliminated in the sprayed FRP beams. Even though the latter specimens were subjected to higher loads, and thus higher stresses at the FRP-concrete interface, debonding was not the prevalent failure mode.

Again, this effect is believed to be related to the difference in stress transfer mechanisms between the continuous fiber fabrics and the discontinuous, randomly oriented sprayed fibers. When debonding initiates in the former, it progresses very quickly along the bond line
since this is the direction in which all of the stresses are acting in the FRP. For the randomly oriented fibers, the stresses are transferred in all directions, effectively acting to arrest growth of the delamination crack and thus prevent debonding.
6.2.6 - Comparison with Published Results

As previously discussed in Chapter 2, a great deal of research has been conducted around the world related to the rehabilitation of reinforced concrete structural members with fiber reinforced polymers. Several different approaches have been examined, including the use of FRP plates, fabrics and strips. A number of fiber types have also been investigated, such as glass, carbon and aramid. The following section compares the results obtained in this study using the sprayed GFRP technique with results previously published by some of these other researchers.

6.2.6.1 - Flexural Strengthening

A survey of published literature related to the strengthening effectiveness of various FRP rehabilitation techniques on conventionally reinforced concrete beams was previously performed by J. Bonacci\textsuperscript{85}. Summarizing the results of 10 different studies including 64 total specimens ranging in length from 950 mm to 4580 mm, it showed that the improvement in ultimate load carrying ability varied with failure mode. Table 6.25 provides the results from this survey along with the corresponding results from the flexural strengthening portion of this study. The survey results are composed of three values which represent the minimum, mean and maximum value for each range of results.

Unfortunately, there were a couple of problems associated with the values reported in the literature survey. For one, it was not specified which retrofit schemes were included, though
it is likely that the majority of the results, if not all, referred to the Scheme E approach in which only the bottom surface of the beam is strengthened. This is by far the most common approach used in flexural strengthening research. Second, the value corresponding to shear failure has no counterpart in this research project. All of the Scheme E specimens failed due to debonding of the GFRP while the higher confinement schemes produced either crushing or GFRP tensile fractures.

<table>
<thead>
<tr>
<th>Source</th>
<th>Shear</th>
<th>Compression</th>
<th>Fracture</th>
<th>Debonding</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>min</td>
<td>mean</td>
<td>max</td>
<td>min</td>
</tr>
<tr>
<td>Literature Survey</td>
<td>18</td>
<td>29</td>
<td>38</td>
<td>31</td>
</tr>
<tr>
<td>Sprayed GFRP</td>
<td>N/A</td>
<td>116</td>
<td></td>
<td>265</td>
</tr>
</tbody>
</table>

For specimens exhibiting concrete compression failure, the sprayed GFRP technique produced far larger improvements (116%) than the average, or even the maximum, values reported in the survey (41% and 55%, respectively). The same can be said for beams failing due to tensile fracture of the FRP (265% vs. 41% and 101%). In the debonding category, on the other hand, the sprayed GFRP results (37%) fell near the bottom end of the survey range (0% to 335%).

One very interesting observation made by the author concerned the prevalence of each of the four failure modes. Shear failure occurred in only 8% of all beams included in the survey,
concrete compression failure in 6% and tensile fracture in 22%. By far the most common failure mode, FRP debonding, occurred in 64% of all test specimens.

Due to the unknowns associated with this survey, it is not possible to make a proper comparison in between the results presented by the author and those obtained in this project using the sprayed GFRP technique. A more direct approach is needed in which individual studies could be compared.

Table 6.26 provides just such a comparison for conventionally designed reinforced concrete beams which were retrofitted to improve flexural strength characteristics. The comparative values shown represent the percentage increase in strength and fracture energy for each of the retrofit techniques relative to the respective control specimens.

Though the increases in peak load were calculated directly from the ultimate load values presented by the authors, in most cases the corresponding fracture energy values were not provided. These values were determined from the provided load-deflection curves, whenever possible. Cases where the fracture energy increase is shown as N/A refer to publications in which neither fracture energy values nor load-deflection curves were provided.
This comparison includes results for plate and strip bonding techniques, as well as the fabric wrapping approach. In the table, fiber types are indicated by G for glass, C for carbon and A for aramid while the fiber orientation is shown in parenthesis following the fiber type. Fiber orientation is specified relative to the longitudinal direction of the member, which is
designated as $0^\circ$. A single value represents a unidirectional fibre composite while dual values represent a woven or bi-directional fibre combination. A material type designation of $S$ represents the steel plate bonding technique, which was included for comparison purposes.

Looking at the Scheme E results, which again is the most common rehabilitation approach, it is apparent that the different retrofit techniques tend to produce a very large variation in results. Improvements in peak load range from a low of 6% to a high of 87%, with the sprayed GFRP technique falling near the middle of this range at 37%. The fracture energy results exhibit an even larger variation, ranging from a 68% drop to a 112% increase. Again, the result for the sprayed GFRP technique falls within this range with a 22% drop. Apparently, the sprayed GFRP technique is capable of producing increases in load carrying ability and fracture energy absorption similar to those exhibited by some of the other rehabilitation techniques.

As for the Scheme C and D retrofit approaches, it was much more difficult to find comparative values from previously published research. A couple of examples were discovered for the Scheme C arrangement, but none were found for Scheme D. For the Scheme C results, the sprayed GFRP technique produced much larger increases in peak load (116%) than either of the other two processes (55% and 64%), though the effect on fracture energy was very similar (a 7% increase vs. a 7% drop). The Scheme D approach produced even larger improvements, this time in both peak load as well as absorbed fracture energy though, again, no comparative values could be found.
This comparison shows that the sprayed FRP technique can produce results similar to many of the other retrofitting procedures when used for pure flexural strengthening (Scheme E), though it is obviously inferior to some of the methods shown. Once the degree of confinement is increased by coating the sides and top of the beam, the sprayed system becomes favorable. This is probably due to its superior ability to wrap around the corners of the beam, thus allowing all three (or four) of the sprayed sides to act as a single unit, something the bonded FRP fabrics and plates cannot do.

It is interesting to note that a large proportion of the rehabilitation techniques presented in Table 6.26 actually resulted in a drop in fracture energy. As discussed previously in Section 6.1.4., this phenomenon is a direct result of the stiffening effect of the applied FRP materials.

6.2.6.2 - Shear Strengthening

A similar comparison was compiled for the effects of the various FRP retrofit techniques on the rehabilitation of shear strength deficient beams, shown in Table 6.27. Again, the improvements in load carrying ability and fracture energy absorption (expressed as percent increases) are compared to results from previously published literature.
For this case, a large number of comparable research projects were found for the Scheme A and the Scheme C retrofit approaches, while only one was uncovered in which the full confinement of Scheme D was implemented. In all instances, the sprayed GFRP produced a
significantly larger increase in both peak load and fracture energy than any of the other retrofit techniques. The Scheme A retrofit produced a 103% increase in peak load, as compared to the published data which ranged from 20% to 82%. Published results for the Scheme C approach ranged from a low of 33% to a high of 122%, much lower than the 188% increase exhibited by the sprayed GFRP technique. The Scheme D specimens were improved 318% by the sprayed GFRP, compared to a 101% increase attributed to the carbon fiber fabric wrapping procedure found in the literature.

The fracture energy results again showed a much higher variation, with the published Scheme A results ranging from the minimum of a 9% drop all the way to a maximum 273% increase, still inferior to the sprayed technique’s improvement of 342%.

Spraying the beams with the Scheme C arrangement produced a 655% improvement, also significantly higher than published results which ranged from a drop of 32% to an increase of 479%. By far the biggest difference, however, appeared in the Scheme D comparison where the sole published value of 549% was far lower than the sprayed technique’s 5350% increase.

This comparison indicates that the sprayed FRP approach is far more effective than any of the other rehabilitation techniques in providing shear strengthening. The primary reason for this increased effectiveness is related to the 2-D isotropic nature of the sprayed FRP material and will be discussed further in Chapter 7.
6.2.7 - Result Variability

An investigation was conducted into the variability inherent in the testing and retrofitting procedures used throughout this project. The variation in results was different for the three properties evaluated. The ultimate load values were the most consistent results, with an average coefficient of variation of 3.6% for groups of similar specimens. The maximum value for any one group was 9.8% but the majority of values were less than 5%.

Initial member stiffness values were slightly less consistent, with an average coefficient of variation of 6.0% and a maximum of 13.5%. Most of these coefficients fell within the 4 - 10% range. Finally, the energy absorbed by the beams up to peak load produced the least consistent results. Though the average coefficient of variation was still only 10.5%, there was a much wider range of coefficients for this property, with a maximum value of 27.7% and a typical range of 8 - 13%. No significant difference in variability could be found between retrofit schemes.
6.3 - Full-Scale Specimens

In order to evaluate the potential of the sprayed FRP technique for use on real structures in the field, it was first necessary to attempt the retrofit of much larger scale specimens in the laboratory. Toward this end, the Ministry of Transportation and Highways of British Columbia graciously supplied three full scale bridge channel beams. These beams came from an existing but badly deteriorated bridge which required complete replacement after approximately 50 years of service. During destruction, the Ministry salvaged three of the stringers for our use.

6.3.1 - Beam Description

As mentioned previously, the members tested in this phase were channel beams, as depicted schematically in Figure 6.48. These beams were cast from a structural lightweight aggregate concrete which exhibited a core compressive strength of 35 MPa.

The steel reinforcement contained in these sections was also sampled and tested in tension. It was discovered that all of the reinforcing steel present was in imperial bar sizes with tensile strengths ranging from 304 MPa to 408 MPa. The typical reinforcement details for these beams are shown in Figure 6.49, again with all sizes being indicated using imperial bar sizes.
The shear stirrups were spaced at varying intervals along the length of the beam, ranging from a minimum spacing of 125 mm at the ends of the beam to a maximum spacing of 760 mm at midspan.

One point that should be made regarding the condition of these channel beams concerns the
significant variation in damage between specimens. All of the members exhibited significant
damage in the form of cracking, concrete cover loss and reinforcement corrosion.
Photographic examples of this damage have been included in Appendix I. The severity of
damage, relative quantity of each type of damage and the location of the damage varied from
beam to beam. As a result, comparison of properties between beams can only be performed
on a very general basis.

6.3.2 - Specimen Preparation

Of the three channel beams supplied, one was to be rehabilitated with the sprayed GFRP
technique while a second was to be retrofitted using a commercially available continuous
glass fiber fabric system. This latter approach utilized the MBrace® system produced by
Master Builder Technologies. The third specimen was to be tested in its original unretrofitted
state to serve as a control specimen.

The surfaces of all of the channel beams were sandblasted by the Ministry of Transportation
and Highways before being delivered to UBC. Due to the amount of deterioration present,
specifically the large sections of missing cover concrete, it was deemed necessary to repair
the more severely damaged regions. Such repair was done by applying a fast setting grout to
these areas to replace the missing material and build up the original profile of the beams, thus
creating a continuous flat surface to which the retrofit materials could be applied. This repair
technique required the fabrication of forms to create the desired profile, placement of the
grout itself and curing of the grout following placement.
Such repair is particularly important for the continuous fiber fabric systems, which would dramatically suffer in efficiency should the material not be applied to a flat surface. Actually, applying these materials over a region of damage such as this would result in the longitudinal stresses in the continuous fibers acting to pull the fabric away from the concrete and thus initiating bond failure.

The sprayed technique, on the other hand, would be essentially unaffected by these regions of missing concrete. Due to the nature of the material (i.e. built up to the required thickness during the spraying operation), it need not be applied in a consistent thickness. The GFRP itself could be used to build up the areas of missing concrete. Though the GFRP material is more expensive than the grout used in the repair approach described above, the elimination of this extra step, not to mention the time and effort required for forming the curing time associated with the grout, would easily be worth the ability to perform the entire retrofit in one step.

For the purpose of this project, it was decided that all of the beams would be repaired using the grouting technique. Doing so meant that comparisons between the two retrofit procedures would be more significant since both were applied to the same type of surface. Following placement of the grout, it was covered with damp burlap followed by plastic and allowed to cure for 24 hours. The curing materials were then removed and the surface of the grout sandblasted. At this point, it was observed that the original sandblasting performed by MOTH personnel was not particularly consistent so some non-repaired areas were redone.
In should be noted that in order to simplify the work being done on these specimens, the beams were turned upside down so that all work could be performed on either upper horizontal surfaces or side vertical surfaces. This includes grouting, sandblasting and application of the GFRP materials (both sprayed and wrapped). Such an approach would obviously not be possible in the field on an existing structure, where all work would have to be performed on the bottom or sides of the members.

Field application would present its own problems for both systems. The fabric bonding technique would require either more personnel or a method for holding the fabric in place while the bonding agent is applied. The GFRP spraying equipment, on the other hand, is not capable of spraying directly overhead in its present form. Modifications to the equipment would be necessary in order to permit such operation. Since neither technique was attempted under field conditions, comparisons between the two application procedures are restricted to the conditions faced in the laboratory.

For the channel beam retrofitted with the sprayed technique, the procedure involved first coating the concrete surfaces to be sprayed with the Derakane® 8084 vinyl ester resin to act as a coupling agent. This coupling agent was allowed to cure for approximately 15 minutes. Then, the sprayed GFRP material was applied to all bottom surfaces of the member (slab and leg) as well as to the insides of the legs, as depicted in Figure 6.50. This material was applied in three layers, with each layer rolled out before application of the successive layer. The nominal GFRP thickness was 8 mm and contained fibers 48 mm in length ($\sigma_{\text{ult}} = 108$ MPa, $E = 11.8$ GPa, $\Delta_r = 1.32\%$, $\rho = 1394$ kg/m$^3$, $V_f = 19\%$).
The beam retrofitted with the MBrace® fabric system was also precoated with a coupling agent or primer. After allowing the primer to cure for 24 hours, the fabric was applied in two different orientations, as indicated in Figure 6.51. Fiber orientation is specified with respect to the longitudinal axis of the member, which is designated as 0°. Since the fabric used in this case contained only unidirectional continuous fibers, each orientation consists of only a single value as opposed to the dual values mentioned earlier for woven fiber fabrics.

First, a single layer fabric in which the fibers were oriented at 0° was applied to the lower surface of the deck portion of the member and to the bottoms of the two vertical legs. Next, the same fabric was applied to the inside of the vertical legs, though this time the fabric was
turned 90° so that the fibers ran vertically. This application was lapped over the 0° fabric at both the top and bottom of the channel legs. At the top, it lapped the 0° fabric by only a couple of inches while at the bottom it was continued across the entire width of the stem soffit.

Application of the fabric was performed simply by brushing a thin layer of resin onto the surface, applying the fabric itself and then brushing on another layer of resin. A roller was then used to ensure full impregnation of the fabric with the epoxy resin. All of the products used in this retrofitting technique (primer, fabric and epoxy resin) were part of the MBrace® fabric system. Both the sprayed and wrapped systems were allowed to cure for several days before testing.

6.3.3 - Testing Program

All three of the beams were tested under third-point loading using four large hydraulic jacks as load actuators. Figure 6.52 consists of a schematic diagram of the testing equipment used for this portion of the project. LVDTs were used to record the deflection of the beam while the hydraulic pressures in the jacks were monitored to provide load information. Figure 6.53 shows a photograph of the test setup during loading of the control specimen.
Figure 6.52: Schematic of MOTH channel beam test setup.

Figure 6.53: Photograph of MOTH channel beam test setup.
Before applying the retrofitting materials, each of the stringers was first loaded in the elastic range in its unretrofitted state to provide a basis for comparison for the strengthening techniques. Though ultimate strengths were not reached in this preloading stage, it did allow us to determine the stiffness of each beam before retrofitting. Upon completion of the test, we could thus determine the increase in stiffness induced by the retrofitting process relative to the control specimen and relative to the same beam before rehabilitation.

The beams were then retrofitted with their respective systems (with the exception of the control specimen) and again loaded in the third-point configuration until failure.

6.3.4 - Results & Discussion

Test results for initial member stiffness, ultimate load carrying capability and fracture energy absorbed up to peak load are shown in Table 6.28, along with the failure mode exhibited by the beams. Values shown for the percent change in these results are calculated relative to the control specimen in all cases.

<table>
<thead>
<tr>
<th>Retrofit Type</th>
<th>Stiffness</th>
<th>Strength</th>
<th>Absorbed Energy</th>
<th>Failure Mode*</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Initial (kN/mm)</td>
<td>Change (%)</td>
<td>Peak (kN)</td>
<td>Change (%)</td>
</tr>
<tr>
<td>None</td>
<td>6.69</td>
<td>-</td>
<td>214</td>
<td>-</td>
</tr>
<tr>
<td>Wrap</td>
<td>7.67</td>
<td>15</td>
<td>284</td>
<td>33</td>
</tr>
<tr>
<td>Spray</td>
<td>9.00</td>
<td>35</td>
<td>419</td>
<td>96</td>
</tr>
</tbody>
</table>

* - Failure modes: Y = Rebar Yield, S = Shear, C = Concrete Crushing, B = GFRP Bond, P = GFRP Plate
As mentioned previously in Section 6.3.2.1., there was a significant amount of damage evident on all three of these specimens and this damage was not consistent among the specimens. Thus, comparisons of the test results from the retrofitted specimens with those obtained from the control specimen, or with each other, can only be done in a very general sense.

The primary exception to this is the initial stiffness values. As mentioned in Section 6.3.3, initial stiffness values were determined for each specimen before and after retrofitting. Table 6.29 indicates the results of this testing for the wrapped and sprayed specimens. The percent change in this table represents the improvement in stiffness of each member due to retrofitting.

<table>
<thead>
<tr>
<th>Retrofit Type</th>
<th>Initial Stiffness Before Retrofit (kN/mm)</th>
<th>Initial Stiffness After Retrofit (kN/mm)</th>
<th>Change (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wrap</td>
<td>6.90</td>
<td>7.67</td>
<td>11</td>
</tr>
<tr>
<td>Spray</td>
<td>7.78</td>
<td>9.00</td>
<td>16</td>
</tr>
</tbody>
</table>

These values are depicted graphically in Figure 6.54 for the two retrofitted members as well as the control beam. Here we can see that both retrofit techniques tend to increase member stiffness. Also evident is the fact that the sprayed technique produced greater results, both with respect to the control specimen and relative to the same member before retrofitting.
The increase in stiffness due to the MBrace® fabric wrapping system was 15% over the control specimen and 11% over the unretrofitted value. The sprayed GFRP, on the other hand, produced an initial member stiffness 35% higher than the control specimen and 16% better than the unretrofitted value from the same beam.

![Figure 6.54: MOTH channel beams - initial stiffness results.](image)

The peak load results are indicated graphically in Figure 6.55. In this case, the fabric wrapping system produced a 33% increase over the control specimen while the improvement due to the sprayed GFRP technique nearly tripled that increase at 96%. Though these figures may be distorted somewhat due to the variability in damage among the specimens, the very large difference between the two techniques is indicative that the sprayed technique is indeed capable of producing superior results.
Figure 6.55: MOTH channel beams - peak load results.

Figure 6.56: MOTH channel beams - energy absorption results.
The energy absorption results (Figure 6.56) indicate that both retrofitting approaches are capable of producing very large improvements in the energy absorption characteristics of the channel beams.

The MBrace® fabric increased the energy absorbed up to peak load by 174% while the sprayed GFRP technique resulted in an increase of 195%. Both of these improvements are significant regardless of the variable damage factor, though the variation between the two techniques may not be significant.

Actual load-deflection curves of the three beams are shown in Figure 6.57.

Figure 6.57: MOTH channel beams - load-deflection curves.
The increases in initial member stiffness and peak load are very evident in this graph, as are the increases in energy absorption. Also evident is the reason why the energy absorption for the wrapped and sprayed specimens is so similar, even though the difference in peak load is much larger. The wrapped specimen reached a higher deflection value before failure, thus extending the load-deflection curve and increasing the absorbed energy (area under the curve).

The rate of fracture energy absorption was also increased by both retrofit techniques, as shown in Figure 6.58. Again, the sprayed approach produced a larger improvement than the wrapping technique.

Figure 6.58: MOTH channel beams - energy absorption curves.
As previously noted in Table 6.27, the control specimen failed due to yielding of the flexural reinforcement. For both of the retrofitted beams, the event corresponding to the failure point was tensile fracture of the GFRP in the tension zone at the bottom of the member. The photographs in Figures 6.59 and 6.60 depict the failure modes exhibited by the fabric wrapped beam and by the GFRP sprayed beam, respectively.

Though, again, exact comparisons cannot be made due to the variation in damage among the specimens, it is evident from the test results that both the MBrace® fabric wrapping system and the sprayed GFRP technique greatly enhanced the structural performance of the channel beams. It is also apparent that the sprayed technique produced larger improvements than the fabric wrapping procedure.

One thing that should be noted is the difference in thickness between the two materials. The sprayed GFRP material was applied in an 8 mm thickness while the fabric in the MBrace® system was only about 1.5 mm thick per layer. The fabric thickness thus varied between areas where only one layer was applied and those areas where a second layer was added. Even though the sprayed technique consisted of a larger volume of material, the actual cost of this technique was still less than the fabric wrapping system. Further details concerning the cost comparison between the two systems can be found in Chapter 8.
Figure 6.59: Failure of MOTH channel beam retrofitted with MBrace® system.

Figure 6.60: Failure of MOTH channel beam retrofitted with sprayed GFRP.
Theoretical Analysis of Sprayed FRP Beam Retrofits

7.1 - Introduction

In order for the sprayed GFRP to be acceptable as a feasible alternative for rehabilitation of reinforced concrete beams, it is essential that the upgraded properties of these beams be predictable. This section is divided into two parts. In the first part, the ultimate load carrying capacity of the specimens are predicted while the second part discusses prediction of the prepeak load-deflection curve for each beam configuration.

In order to predict the load carrying ability for each beam, two possible failure modes must be examined; flexure and shear. The failure load will be determined for each mode and the lower of the two will be taken as the ultimate load for the specimen. A third failure mode is also potentially possible, a splitting bond failure can occur should the development length of the longitudinal tension reinforcement be insufficient. For the beam designs used in this project, however, this third failure mode will not be considered since it did not occur during
Furthermore, the CSA A23.3-94 Design Code recommends development lengths of 242 mm for 10M reinforcing bars and 343 mm for 15M bars. The beam design used here provides 325 mm of development length, which is sufficient for the 10M bars used in the flexural strengthening beams but appears to be insufficient for the 15M bars used for the shear strengthening specimens. Since none of the laboratory specimens actually failed due to splitting bond failure, it was assumed that the CSA recommended development length of 343 mm was simply a conservative design value and that the 325 mm provided was sufficient to prevent this failure type.

7.2 - Ultimate Load Prediction

7.2.1 - Material Characteristics

Before analysis of the retrofitted beams is possible, the stress-strain characteristics of the individual materials must be defined. The concrete was assumed to remain linearly elastic in tension until failure, while the equivalent rectangular stress block approach described in the CSA A23.3-94 Design Code was implemented for the compressive behavior. The compressive strength used varies depending upon which specimens are involved, with the value used being an average of the mixes from which the specimens in question were cast.

For design purposes, the stress-strain relationship for reinforcing steel is normally assumed
to be bilinear in nature, as indicated in Figure 7.1. This relationship assumes that the steel’s loading response is perfectly elastic up to the yield point, and then becomes perfectly plastic until ultimate failure. As a result, the yield and ultimate strengths are considered to be identical in magnitude. For the purpose of this analysis, a reinforcing steel yield strength $f_y$ of 500 MPa was assumed along with an elastic modulus $E_s$ of 200 GPa.

Figure 7.1: Stress-strain relationship - reinforcing steel.

These values correspond to a 400 Grade steel as specified by CSA A23.3 for elastic modulus and CSA G30.18 for strength. In beam design, the yield strength for this grade of steel is normally taken to be 400 MPa. However, this value actually represents the minimum allowable strength for the 400 Grade steel and is thus very conservative. For analysis purposes, a less conservative value must be chosen.
The previously mentioned value of 500 MPa was chosen for a couple of reasons. First, CSA G30.18 specifies the minimum and maximum yield strengths of 400 Grade reinforcement as 400 MPa and 525 MPa, respectively, resulting in an average yield strength of 462 MPa. This value was raised even further to account for the higher load carrying ability of the steel beyond the yield strength. Though the design guidelines ignore this extra capacity, it was necessary to include it in the following analyses for accuracy reasons. Since it is the ultimate load of the beams that is of interest, the steel at this point is well beyond its yield point when failure occurs though it has not yet reached its ultimate strain. Since CSA G30.18 specifies the ultimate tensile strength to be not less than 1.15 times the yield strength, it was decided that a value of approximately one half of this extra strength would be used, and thus 500 MPa was chosen.

The stress-strain properties of the sprayed fiber reinforced polymer material were taken directly from the GFRP materials testing portion of this project (Section 5.2). Two different relationships were needed since the two types of beams to be analysed were repaired using different fiber lengths. The flexural strengthening phase (Section 6.1.4) was performed using a 32 mm fiber length while the final shear strengthening results were obtained using a 48 mm fiber length. Unfortunately, the stress-strain behaviors of these two materials (depicted in Figure 7.2) were not linear and thus a relationship was needed to allow determination of the stress applied to the material at different strain levels.

Such relationships were determined from the experimental stress-strain curves using a statistical curve fitting program. The resulting relationships for the two fiber lengths in
question, along with those corresponding to all of the other fiber lengths tested, have been included in Appendix J.

Figure 7.2: Stress-strain relationship - sprayed GFRP.

Table 7.1: GFRP stress-strain relationship by fiber length.

\[ f_f = a + b \cdot \varepsilon_f + c \cdot \varepsilon_f^2 + d \cdot \varepsilon_f^{2.5} + e \cdot \varepsilon_f^3 \]

<table>
<thead>
<tr>
<th>Fiber Length (mm)</th>
<th>a</th>
<th>b</th>
<th>c</th>
<th>d</th>
<th>e</th>
</tr>
</thead>
<tbody>
<tr>
<td>GU 8</td>
<td>-0.05388</td>
<td>6365</td>
<td>0</td>
<td>-6271234</td>
<td>52090408</td>
</tr>
<tr>
<td>8</td>
<td>1.30266</td>
<td>8627</td>
<td>0</td>
<td>-6523681</td>
<td>30629216</td>
</tr>
<tr>
<td>16</td>
<td>0.69810</td>
<td>10864</td>
<td>-567983</td>
<td>2033607</td>
<td>0</td>
</tr>
<tr>
<td>24</td>
<td>0.33314</td>
<td>10893</td>
<td>0</td>
<td>-5418455</td>
<td>24495788</td>
</tr>
<tr>
<td>32</td>
<td>0.75181</td>
<td>12147</td>
<td>-633311</td>
<td>2283722</td>
<td>0</td>
</tr>
<tr>
<td>48</td>
<td>-0.94811</td>
<td>15445</td>
<td>-2478366</td>
<td>32184530</td>
<td>-130460450</td>
</tr>
</tbody>
</table>
These relationships are of the form:

\[ y = a + bx + cx^2 + dx^{2.5} + ex^3 \]

where \( y \) represents the stress applied to the GFRP \((f_j)\) and \( x \) is the resulting strain in the GFRP \((e_j)\). The variables \( a, b, c, d \) and \( e \) are constants that vary depending upon the fiber length in question. Table 7.1 provides the actual values of these constants for each fiber length.

7.2.2 - Flexural Strength

Probably the most important structural property that must be predicted is the ultimate load carrying ability or moment resistance of the members. The actual beam design method described in the CSA Standard A23.3 can be used as a predictive tool for these beams once adjustments are made for the GFRP material. Figure 7.3 depicts the CSA approach for flexural design of conventionally reinforced concrete beams.

![Figure 7.3: Flexural analysis of conventionally reinforced concrete sections.](image)
One significant difference between the CSA design approach and the analysis method described here is the treatment of material strength. In design, resistance factors are applied to material strength values to account for inherent variations in strength and dimensions. These factors ($\phi_s$ for steel and $\phi_c$ for concrete) contribute to the conservatism which is necessary in structural design but lead to large inaccuracies if used in the type of structural analysis found here. Consequently, all resistance factors have been omitted in the following analysis.

To simplify the design process, a rectangular compressive stress block is assumed in place of the typical parabolic distribution normally associated with concrete. The intensity and depth of this stress block are indicated in Figure 7.3 and are dependent upon the expressions for $\alpha_i$ and $\beta_i$, respectively. These two terms are defined with respect to the compressive strength of the concrete and limited to a minimum value of 0.67 as shown below.

$$\alpha_i = 0.85 - 0.0015f'_c \geq 0.67$$
$$\beta_i = 0.97 - 0.0025f'_c \geq 0.67$$

Flexural design of reinforced concrete beams is an iterative process based on the principle of strain compatibility. The concrete strain at the uppermost edge of the compression block $\varepsilon_c$ is assumed to be equal to the maximum allowable compressive strain for concrete $\varepsilon_{cu}$.

This upper limit is designated as 0.0035 in the CSA design standard but for the purpose of
analysis a less conservative figure is required. Paulay and Priestley $^{95}$ suggest an ultimate strain of 0.0040 for unconfined concrete subjected to compressive stresses. It is this latter figure which will be used throughout the structural analysis presented in this work. It should be noted here, however, that the expressions for $\alpha$ and $\beta$ provided above refer to an ultimate strain of 0.0035 and have not been modified for the 0.0040 value used in the analysis.

Once the strain at the top of the compression block is set to the ultimate concrete compressive strain, the resulting strain in the reinforcing steel can, using the similar triangle approach, easily be determined as

$$\varepsilon_s = \frac{\varepsilon_c (d - c)}{c}$$

At failure, the reinforcing steel is assumed to have already yielded, thus making the actual strain in the steel irrelevant since the steel has already reached its maximum load, which is defined as

$$T_s = A_s \phi_s f_y$$

Due to its bilinear stress-strain relationship, the steel will continue to carry this maximum load regardless of any further increase in strain (assuming it does not reach its failure strain before the beam fails, which is extremely unlikely). A neutral axis depth $c$ is assumed and the resulting compression force in the concrete $C_c$ is calculated in accordance with the following equation.
where the term $a$ represents the depth of the compression block and is equivalent to $\beta c$. These two forces, $C_c$ and $T_s$, must be equal in magnitude and opposite in direction in order for static equilibrium to exist. Thus,

$$C_c = T_s$$

Since the value of $C_c$ is dependent upon $a$, which is in turn dependent upon the neutral axis $c$, the determination of $C_c$ is an iterative process as the assumed value of $c$ is iteratively modified until the two forces reach equilibrium. The moment resistance $M_r$ of the beam can then be computed as the force in the steel reinforcement multiplied by the internal moment arm, the distance between the forces $T_s$ and $C_c$.

$$M_r = T_s \cdot \left( d - \frac{a}{2} \right)$$

The ultimate load carrying capacity of the beam $P_{ult}$ can then be determined from the moment resistance, depending upon the loading configuration. As described earlier in Section 6.1.2., a third-point loading arrangement was used throughout this project. For this configuration the maximum moment occurring in the beam is related to the applied load by:

$$M_{\text{max}} = \frac{PL}{6}$$
Rearranging this equation for the applied load gives:

$$ P_{ult} = \frac{6M_r}{L} $$

To account for the retrofit material applied to the bottom surface of the beam, the FRP is simply considered as a second plane of flexural reinforcement. Figure 7.4 illustrates the modifications that must be made to the previously discussed CSA approach for flexural design of conventionally reinforced concrete beams.

![Cross-Section, Strains, Forces](image)

**Figure 7.4:** Flexural analysis of RC sections retrofitted with FRP - Scheme E.

Basically, the original design approach is amended by adding another force ($T_{fb}$) to account for this extra reinforcement. At this point, the next step in the analysis depends upon the failure mode exhibited by the beam, concrete crushing in the compression zone at the top of the beam or tensile fracture of the applied FRP.
First, assuming the failure will occur due to concrete crushing, the concrete strain $\varepsilon_c$ is once again set to the ultimate concrete strain and the strain in the reinforcing steel is determined as before. The strain in the FRP ($\varepsilon_{fb}$) can be determined, due to the strain compatibility assumption, using the similar triangle approach. This results in the equation

$$\varepsilon_{fb} = \frac{\varepsilon_c \cdot (d_{fb} - c)}{c}$$

The stress applied to the FRP is then calculated from this strain using the FRP stress-strain relationship shown in Table 7.1. Since we now know the applied stress and the material area, the actual force carried by the FRP ($T_{fb}$) can also be computed as

$$T_{fb} = f_{fb} \cdot A_{fb}$$

If, on the other hand, the failure mode is assumed to be tensile fracture of the FRP, the approach is slightly different. In this case, the strain at the lowermost edge of the FRP is set to the ultimate strain of this material and the strains in the steel and concrete are calculated relative to this limiting value. Now, the steel and concrete strains are represented by

$$\varepsilon_s = \frac{\varepsilon_{fb} \cdot (d - c)}{h + t - c} \quad \varepsilon_c = \frac{\varepsilon_{fb} \cdot c}{h + t - c}$$
The force carried by the FRP is also slightly modified as

\[ T_{fb} = f'_{fb} \cdot A_{fb} \]

Regardless of failure mode, the force balance equation is modified to include this force carried by the FRP and thus becomes

\[ C_c = T_s + T_{fb} \]

The expression for moment resistance of the member must also be modified to include the effect of both tensile forces

\[ M_r = T_s \cdot \left( d - \frac{a}{2} \right) + T_{fb} \cdot \left( d_{fb} - \frac{a}{2} \right) \]

These modifications take into account only the FRP applied to the bottom face of the beam. Though the FRP applied to the sides of the beam were expected to contribute to the shear strength of the member, examination of the results from the flexural strengthening specimens reveals that it contributes to flexural strength as well. When comparing specimens retrofitted with the Scheme E approach (bottom face only) to those retrofitted with Scheme C (bottom and sides coated), it becomes evident that the addition of the FRP to the sides significantly increases the flexural strength of these beams.
This conclusion is further supported by the cracking pattern exhibited by the sprayed GFRP material in the Scheme D retrofit. Following the tensile fracture of the GFRP applied to the bottom face, cracks would propagate up either side of the specimen until they reached the neutral axis. Here, they would split, with each branch progressing horizontally. The GFRP above this line would eventually fail in compression due to buckling. This indicates that the GFRP applied to the side of the beam in the region below the neutral axis is subjected to an applied tensile stress while the FRP above the neutral axis carries a compressive load. The photograph in Figure 7.5 depicts this cracking pattern.

![Cracking pattern exhibited by sprayed GFRP on sides of beams.](image)

**Figure 7.5:** Cracking pattern exhibited by sprayed GFRP on sides of beams.

In large scale structures, the FRP material that is actually in compression need not be considered since its contribution to overall member strength would be insignificant. In the
small scale specimens being analysed here, however, this contribution cannot be overlooked. The difference between the two cases lies in the ratio of GFRP thickness to overall beam size.

As mentioned previously in Section 6.1.5.6., the minimum practical application thickness of the sprayed GFRP material is 3.5 mm, as used in the shear strength deficient specimens discussed later. Even at this minimum thickness, the retrofit material makes up almost 7% of the total beam width and over 5% of its total depth (for Scheme D). This represents a substantial portion of the cross-sectional area of the specimen and results in a significant amount of GFRP material being subjected to compression. As long as the GFRP-concrete bond remains intact, the GFRP will resist buckling and act as compression reinforcement. In large scale structures, the ratio of GFRP thickness to overall member size would be much smaller and thus the effect of the GFRP in compression would become minimal, not to mention that fact that ignoring its contribution would simply add to the conservatism of the design approach anyway.

There are two modifications of the analysis procedure necessary in order to adapt it to the Scheme C retrofit arrangement. First, the FRP applied to the sides of the beam below the neutral axis must be included as flexural reinforcement, as shown in Figure 7.6.
As mentioned previously, this would normally be the only modification necessary for large scale specimens and for designing FRP retrofits. For analysis of the small scale specimens in this project, however, a further modification is required to account for the side GFRP above the neutral axis. This material is included as compression reinforcement and the necessary changes are shown in Figure 7.7.

Figure 7.6: Flexural analysis of RC sections retrofitted with FRP - Scheme C (FRP in tension only).

Figure 7.7: Flexural analysis of RC sections retrofitted with FRP - Scheme C (includes FRP in compression).
The biggest complication related to the inclusion of the two components of the side FRP material involves the fact that their dimensions are relative to the location of the neutral axis. For the sake of simplicity in the analysis, the side FRP will be considered to include only that FRP actually in contact with the surface of the beam. The extra material beyond the bottom face of the concrete (or above in the Scheme D specimens) will be accounted for by increasing the width of the bottom and/or top FRP by twice the side thickness.

Again, the analysis will depend upon the expected failure mode of the beam. For a concrete crushing failure, the concrete strain $\varepsilon_c$ is set to its ultimate value $\varepsilon_{cu}$ and the strains in the steel and the bottom FRP are determined as before. The strains at the mid-heights of the side GFRP, which are designated $\varepsilon_{fst}$ for the portion above and $\varepsilon_{fbb}$ for the portion below the neutral axis, are

$$
\varepsilon_{fst} = \frac{\varepsilon_c \cdot c}{2c} = \frac{\varepsilon_c}{2} \\
\varepsilon_{fbb} = \frac{\varepsilon_c \cdot (h - c)}{2c}
$$

Note that the value of $\varepsilon_{fst}$ will always be exactly one half the upper surface concrete strain since its location is midway between the upper surface and the neutral axis. The only variable relative to $\varepsilon_{fst}$ will be its location since that depends upon the neutral axis location. These strains are then converted to stresses, again using the relationship from Table 7.1, and finally to forces using

$$
T_{fst} = f_{fst} \cdot A_{fst} \\
T_{fbb} = f_{fbb} \cdot A_{fbb}
$$

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The force equilibrium equation now becomes

\[ C_c + C_{fst} = T_s + T_{fb} + T_{fsb} \]

Subsequently modifying the moment resistance equation gives

\[ M_r = T_s \cdot \left( d - \frac{a}{2} \right) + T_{fb} \cdot \left( d_{fb} - \frac{a}{2} \right) + T_{fsb} \cdot \left( \frac{h + c - a}{2} \right) - C_{fa} \cdot \left( \frac{c - a}{2} \right) \]

Care must be taken to ensure that a consistent sign convention is followed. Since the compression force \( C_{fa} \) acts below the concrete compression force \( C_c \) (the point around which the moment resistance is calculated), it will act opposite to the moment created by the other three terms. This analysis assumes that the absolute value (always positive) of each force is used in the expressions for moment resistance.

The remaining contribution of the retrofit material that must be considered is that associated with the FRP applied to the top surface of the beam. This material will also act as compression reinforcement as long as it is prevented from buckling. The modifications to the beam analysis procedure that are necessary to include the effect of the top surface FRP are indicated in Figure 7.8.
Unfortunately, compression testing of the sprayed GFRP material could not be properly carried out with the available equipment. An extensive literature search of composite material properties revealed that a compressive strength value of 200 MPa is typical for a short, randomly distributed glass/polyester composite containing 15-25% glass. Once this value was confirmed by a second reference, it was accepted for use in the analysis portion of the project. Though the compressive strength is not actually used in the calculations, it does represent the upper limit of stress than can be carried by the FRP in compression. In no case did the calculated compressive stress actually approach this limiting value.

Once again, the strain calculations will depend upon the expected failure mode. The first case to be considered will again be concrete crushing at the upper surface of the member. Since the upper FRP can only act as compression reinforcement as long as it is prevented from buckling, and its compressive strength is significantly higher than that of the concrete, the FRP should not fail until after the concrete has (assuming no bond failure). Thus, the
previous failure criterion based on ultimate concrete strain still applies and $\varepsilon_c$ is set to its ultimate value $\varepsilon_{cu}$. The strain in the top FRP plate can then be represented as

$$\varepsilon_{f1} = \frac{\varepsilon_c \cdot (c + \frac{t}{2})}{c}$$

while the remaining strain calculations remain the same as before. This strain is first converted to a stress $(f_{fl})$ using the relationship from Table 7.1 and then to the force $C_{fl}$ where

$$C_{fl} = f_{fl} \cdot A_{fl}$$

Adding this force to the force equilibrium expression gives

$$C_c + C_{fpl} + C_{fl} = T_s + T_{fb} + T_{f3b}$$

Subsequently modifying the moment resistance equation results in

$$M_r = T_s \cdot \left( d - \frac{a}{2} \right) + T_{fb} \cdot \left( d_{fb} - \frac{a}{2} \right) + T_{f3b} \cdot \left( \frac{h + c - a}{2} \right) - C_{fa} \cdot \left( \frac{c - a}{2} \right) + C_f \cdot \left( \frac{a + t}{2} \right)$$

Because the force $C_{fl}$ acts above the line of action of $C_c$, its effect on moment resistance will be additive with the tension forces, unlike $C_{fpl}$. 205
7.2.3 - Effect of Confinement

When the previously described analysis techniques were applied to the small scale specimens cast and tested in this project, an interesting trend was revealed. The accuracy of the predictions became significantly worse as the number of coated sides increased, with the predicted strengths getting successively lower (further described in Section 6.3.5.). One concept that was identified as a possible explanation for this trend was the effect of confinement on the concrete in the compression zone. In an attempt to offset this effect, it was decided that the confining effect should be included in the analysis techniques. The approach used to accomplish this feat was first described by Mander et al.\(^9^8\) and further detailed by Paulay & Priestley\(^9^5\). Their relationship was actually designed for use in columns. Its implementation into this analysis is simply an attempt to take the confining effect into account.

The technique first involves determination of the ratio of transverse reinforcement area providing the confinement (the FRP coating in this case) to concrete area in each direction. These two ratios are labelled \(\rho_x\) and \(\rho_y\) and were calculated as

\[
\rho_x = \frac{2t_f}{h} \quad \rho_y = \frac{2t_f}{b}
\]

For the Scheme D retrofit, both directions are applicable since the beam is completely confined due to the GFRP coating on all four sides. The Scheme C retrofit arrangement, on
the other hand, does not provide complete confinement since only three sides are coated. For this case, $\rho_x$ was assumed to be zero and $\rho_y$ was calculated using the above formula. No confinement effect was considered in any of the other retrofit schemes.

Next, the effective confining stress ratio that could be developed by the FRP was determined in each direction using the formulae

\[
\frac{f_{lx}}{f'_c} = \frac{K_e \rho_x f'_f}{f'_c} \quad \frac{f_{ly}}{f'_c} = \frac{K_e \rho_y f'_f}{f'_c}
\]

where the term $K_e$ is a confinement effectiveness coefficient. The value of $K_e$ for rectangular sections is specified as 0.75. Once the effective confining stress ratios are known, the confined strength ratio $K$ is determined from Figure 7.9.

---

*Figure 7.9: Effective compressive strength determination for confined concrete.*

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To use Figure 7.9, the larger of the two effective confining stress ratios described above is first located on the y-axis. Then, move across horizontally until reaching the curve corresponding to the smaller effective confining stress ratio. From there, move vertically upward to the top of the figure and read off the confined strength ratio $K$. The confined concrete compressive strength $f'_{cc}$ is then taken to be

$$f'_{cc} = K \cdot f'_c$$

Confining stresses not only increase the effective compressive strength of concrete, but also affect the ultimate concrete compression strain, according to the following formula

$$\varepsilon_{cc} = 0.004 + \frac{1.4 \rho_s f'_f \varepsilon_{fu}}{f'_{cc}}$$

where $\rho_s = \rho_s + \rho_y$ for rectangular sections. These two confined concrete values, $f'_{cc}$ and $\varepsilon_{cc}$, are then used in place of $f'_c$ and $\varepsilon_{cu}$ in the flexural analysis technique previously described for the retrofit schemes where confinement is provided by the FRP.

### 7.2.4 - Shear Strength

Determining the shear resistance of reinforced concrete beams can be even more straightforward. According to CSA Standard A23.3, the shear resistance of a conventionally
reinforced concrete beam is simply the shear capacity of the concrete plus the added capacity of the shear reinforcement (typically in the form of steel stirrups). The CSA Standard presents two different approaches for determining these components, a Simplified Method (Clause 11.3) and a General Method (Clause 11.4). First, the Simplified Method described in Clause 11.3 will be adapted to include the contribution of the sprayed GFRP to shear resistance. Then, a modified version of the General Method will follow in Section 7.2.4.2.

7.2.4.1 - Simplified Method

To begin with, Figure 7.10 depicts a typical reinforced concrete beam which will be used to illustrate the CSA approach for shear design of conventionally reinforced concrete beams.

![Cross-Section](image)

**Figure 7.10:** Shear design of conventionally reinforced concrete sections.

The shear resistance contribution of the concrete is calculated as

\[ V_c = 0.2 \lambda \phi_c \sqrt{f'_c b_w d} \]
where $b_w$ represents the beam width and $\lambda$ is a correction factor used to account for low density concrete (otherwise $\lambda = 1.0$). The contribution of the shear reinforcement is determined from the equation

$$V_s = \frac{\phi_s A_v f_y d}{s}$$

where the term $A_v$ represents the area of shear reinforcement perpendicular to the axis of the beam within the stirrup spacing distance $s$.

Thus, the total shear resistance of the member is simply the total of these two components and is represented by

$$V_r = V_c + V_s$$

Modifying this approach to account for the sprayed GFRP retrofit material involves inclusion of the FRP applied to the sides of the beam into the shear strength equation. The contributions from the FRP applied to the top and bottom of the beam are considered to be insignificant compared to the total shear resistance and are thus ignored.

Figure 7.11 indicates the dimensions and areas applicable to the shear analysis of FRP retrofitted beams.
Inclusion of the FRP applied to the sides of the member is accomplished simply by considering the FRP to be continuous stirrups. Thus, the contribution of the FRP to the overall shear resistance of the member can be represented by

\[ V_{frp} = \phi_f \cdot 2t_f \cdot f_f \cdot h_f \]

where \( t_f \) is the thickness of the FRP, which is equivalent to the horizontal area of FRP per unit length in the longitudinal direction. The term \( h_f \) refers to the overall height of the FRP. Adding this term to the expression for total shear resistance gives

\[ V_r = V_c + V_s + V_{frp} \]

Finally, the ultimate load carrying ability of the beam for the third-point loading arrangement can be calculated as

\[ P_{ult} = 2V_r \]
Though the FRP applied to the top and bottom surfaces of the beam does not directly contribute to shear resistance, it does improve the anchorage of the FRP and help prevent debonding. Thus, this extra material does indirectly affect the shear strength of concrete beams.

7.2.4.2 - General Method

The CSA General Method also defines the total shear resistance of a reinforced concrete member as the shear resistance provided by the concrete plus the contribution of the shear reinforcement. However, the approach used to calculate these components is a bit more complex due to the inclusion of a residual tensile stress factor $\beta$ to account for the shear resistance of cracked concrete and the effect of the angle of inclination of the diagonal compressive stresses $\theta$. The actual values of $\beta$ and $\theta$ are determined using modified compression field theory and tabulated in the CSA code. For the General Method, the shear capacity of the concrete is calculated as

$$V_{cg} = 1.3\lambda \phi_c \beta \sqrt{f_c'} \cdot b_w d_v$$

The contribution of the shear stirrups situated perpendicular to the longitudinal axis of the member is determined using
For beams in which the shear stirrups are inclined at an angle \( \alpha \) to the longitudinal axis of the member, the shear contribution is

\[
V_{ sg} = \frac{\phi_s A_v f_y d_v \cot \theta}{s} (\cot \theta + \cot \alpha) \sin \alpha
\]

Thus, the total shear resistance of a conventionally reinforced concrete beam would be

\[
V_{rg} = V_{eg} + V_{sg}
\]

Modifying this approach to include the contribution of the sprayed GFRP applied to the sides of the beam would consist of adding a term to represent this additional shear resistance. This term should be of the same form as for the contribution of shear stirrups located at an angle \( \alpha \) to the longitudinal axis. This equation would thus become

\[
V_{sg} = 2 \cdot t_f f_f h \cdot (\cot \theta + \cot \alpha) \sin \alpha
\]

Furthermore, the value of \( \alpha \) should be taken as 45° at all times. The reason for these assumptions lies in the fact that the sprayed GFRP material is two dimensionally isotropic, resulting in a shear stirrup that acts as an inclined stirrup that is always aligned in the best
possible orientation. In the equation shown above for shear contribution of inclined stirrups, this would be a 45° angle.

This isotropic nature is the primary reason behind the superior shear strengthening performance of the sprayed FRP material when compared to unidirectional fiber systems such as fabric wraps or laminated plates. The sprayed material has the same strength in any direction. Continuous fiber FRPs, on the other hand, quickly lose strength as the direction of applied stress deviates from the fiber direction.\textsuperscript{99,100} This effect is depicted in Figure 7.12 for a carbon fiber/epoxy composite.

\begin{figure}[h]
\centering
\includegraphics[width=0.5\textwidth]{frp_strength.png}
\caption{Tensile and compressive strength as a function of fiber orientation for a unidirectional fiber composite.\textsuperscript{99}}
\end{figure}

Including the contribution of the FRP in the overall shear resistance equation results in the following expression
Finally, the ultimate load carrying ability of the beam for the third point loading arrangement can be calculated as

\[ P_{ul} = 2V_{rg} \]

### 7.2.5 - Comparison with Experimental Results

In order to check the accuracy of these analysis methods, ultimate load predictions were made for each configuration of the small scale beams tested in the project. Since there were two beam designs, the flexural strengthening beams discussed in Section 6.2.4 and the shear strengthening beams used throughout the remaining phases of the project, both designs were included in the analysis. For each of these beam designs, an analysis was performed on the unretrofitted control specimen as well on each retrofit scheme used.

For the flexural strengthening specimens, this resulted in an analysis of the unretrofitted control as well as retrofit Schemes E, C and D. As far as the shear strengthening beams are concerned, the final set of beams from Phase G were examined, resulting in four combinations as well, the unstrengthened control along with Schemes A, C and D. In each of these eight different configurations, both the ultimate flexural and shear strengths were predicted.

For some configurations this resulted in a number of flexural analysis attempts, depending
upon the arrangement of the FRP and the failure mode expected in the specimen. In schemes containing FRP subjected to compression (A, C and D), each beam was first analysed without any contribution from the FRP in compression. This was done in order to show the contribution of the compression FRP to overall member strength. Then, the compression FRP was added and the analysis repeated.

In addition, the initial assumption for all retrofitted beams was that failure would occur due to concrete crushing. If the stress in the FRP applied to the bottom face of the beam exceeded the ultimate strength of the FRP material, the analysis was redone using the assumption that the failure mode would instead be tensile fracture of the bottom FRP. This latter condition occurred in both Scheme D retrofitted beams and also in the Scheme C beam from the flexural strengthening series. As a result of this approach, different beam/retrofit configurations required varying analysis sequences to arrive at their predicted ultimate strength value.

7.2.5.1 - Flexural Strengthening Specimens

The sequences used for the flexural strengthening specimens are detailed in Table 7.2. Peak load values indicated as N/A represent an incidence where the concrete crushing failure mode was assumed but the stress in the bottom GFRP surpassed the ultimate GFRP material strength. Thus, the peak load value produced by this analysis was invalid. All of the flexural strength analyses were performed using a custom designed spreadsheet. Printouts of the results of each step in the sequences are included in Appendix L.
As shown in the table, the contribution from the compression FRP is indeed significant. In both schemes containing such FRP (C and D), inclusion of the compression FRP resulted in a change in failure mode from concrete crushing to tensile fracture of the FRP, which was accompanied by a significant increase in the predicted ultimate load.

<table>
<thead>
<tr>
<th>Retrofit Scheme</th>
<th>Specimen Condition</th>
<th>Peak Load (kN)</th>
<th>Failure Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>Control</td>
<td>Concrete Crushing Assumed</td>
<td>56</td>
<td>Concrete Crushing</td>
</tr>
<tr>
<td>E</td>
<td>Concrete Crushing Assumed</td>
<td>92</td>
<td>Concrete Crushing</td>
</tr>
<tr>
<td>C</td>
<td>No Compression FRP Concrete Crushing Assumed</td>
<td>134</td>
<td>Concrete Crushing</td>
</tr>
<tr>
<td></td>
<td>Compression FRP Added Concrete Crushing Assumed</td>
<td>N/A</td>
<td>FRP Fracture</td>
</tr>
<tr>
<td></td>
<td>Compression FRP Added FRP Fracture Assumed</td>
<td>147</td>
<td>FRP Fracture</td>
</tr>
<tr>
<td>D</td>
<td>No Compression FRP Concrete Crushing Assumed</td>
<td>153</td>
<td>Concrete Crushing</td>
</tr>
<tr>
<td></td>
<td>Compression FRP Added Concrete Crushing Assumed</td>
<td>N/A</td>
<td>FRP Fracture</td>
</tr>
<tr>
<td></td>
<td>Compression FRP Added FRP Fracture Assumed</td>
<td>176</td>
<td>FRP Fracture</td>
</tr>
</tbody>
</table>

The predicted values reported in Table 7.2 only consider the flexural strength analysis of the beams. A shear strength analysis was also performed on each of these beams to determine whether their ultimate shear strength was sufficiently high to avoid a shear failure. Results from this shear strength analysis are shown in Table 7.3.

Comparing these shear strength predictions with the flexural strength predictions in Table 7.2...
7.2 reveals one interesting anomaly. Shear strength analysis indicates that the Scheme E retrofit specimens should fail in shear under a load of 92 kN. Flexural strength analysis predicted a concrete crushing failure, also at 92 kN.

<table>
<thead>
<tr>
<th>Retrofit Scheme</th>
<th>Individual Shear Components</th>
<th>Total Shear Strength $V_{tg}$ (kN)</th>
<th>Ultimate Load $P_{ult}$ (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Control</td>
<td>Concrete $V_{cg}$ (kN)</td>
<td>Stirrups $V_{sg}$ (kN)</td>
<td>GFRP $V_{gr}$ (kN)</td>
</tr>
<tr>
<td></td>
<td>12.8</td>
<td>33.1</td>
<td>N/A</td>
</tr>
<tr>
<td>E</td>
<td>12.8</td>
<td>33.1</td>
<td>46</td>
</tr>
<tr>
<td>C</td>
<td>12.8</td>
<td>33.1</td>
<td>316.2</td>
</tr>
<tr>
<td>D</td>
<td>12.8</td>
<td>33.1</td>
<td>316.2</td>
</tr>
</tbody>
</table>

Even though the control beam had significantly more shear strength than necessary to avoid a shear failure, the Scheme E specimens are borderline and may very well fail in shear. This situation is not really surprising. By adding the GFRP to the bottom of the beam, we are effectively increasing its flexural strength without affecting its shear strength. If this trend were to be continued, eventually a shear failure mode would occur.

Looking at the Scheme C and D specimens, it becomes apparent that the shear strength contribution from the sprayed GFRP applied to the sides of the beam is extremely large. Both of these specimen types have more than enough shear reinforcement to avoid a shear failure and thus they should fail in accordance with their predicted flexural failure modes. By taking the lowest strength value from either the flexural analysis or the shear analysis of each
retrofit scheme, a comparison can be made between these predicted values and the results actually measured in the lab during testing of these beams. These values have been compiled and are shown in Table 7.4.

<table>
<thead>
<tr>
<th>Retrofit Scheme</th>
<th>Predicted Result</th>
<th>Experimental Result</th>
<th>Difference (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Peak Load (kN)</td>
<td>Peak Load (kN)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Failure Mode</td>
<td>Failure Mode</td>
<td></td>
</tr>
<tr>
<td>Control</td>
<td>56</td>
<td>63</td>
<td>-11</td>
</tr>
<tr>
<td></td>
<td>Concrete Crushing</td>
<td>Concrete Crushing</td>
<td></td>
</tr>
<tr>
<td>E</td>
<td>92</td>
<td>86</td>
<td>7</td>
</tr>
<tr>
<td></td>
<td>Concrete Crushing/Shear</td>
<td>GFRP Debonding</td>
<td></td>
</tr>
<tr>
<td>C</td>
<td>147</td>
<td>136</td>
<td>8</td>
</tr>
<tr>
<td></td>
<td>GFRP Tensile Fracture</td>
<td>Concrete Crushing</td>
<td></td>
</tr>
<tr>
<td>D</td>
<td>176</td>
<td>230</td>
<td>-23</td>
</tr>
<tr>
<td></td>
<td>GFRP Tensile Fracture</td>
<td>GFRP Tensile Fracture</td>
<td></td>
</tr>
</tbody>
</table>

Examining the comparisons presented in this table, it appears that there are a couple of significant discrepancies. Even though the Scheme E specimens were predicted to fail due to either concrete crushing or shear, neither actually occurred in the lab. These specimens all failed due to debonding of the GFRP from the concrete surface. The ultimate load reached by these specimens, however, came very close to that predicted by the two analysis techniques. Apparently, the GFRP-concrete bond failed just before the beams reached their ultimate load.

The Scheme C specimens tended to fail due to concrete crushing even though the flexural analysis predicted a GFRP tensile fracture. This result is actually very close to the prediction. In the second step of the analysis sequence described in Table 7.2, the assumption of a crushing failure resulted in the stress in the bottom GFRP surpassing its ultimate strength.
and led to the next step of assuming a GFRP tensile fracture. However, the calculated stress in the GFRP at the initiation of concrete crushing in this second step was only 106 MPa, only 2% higher than the ultimate strength of the GFRP which was 104 MPa. Looking back at the GFRP tensile testing results from Section 5.1, the coefficient of variation within the sprayed GFRP material is typically about 15%. This means that the discrepancy between the predicted and experimental results for the Scheme C retrofit falls well within the GFRP material variability.

The Scheme D prediction was further off, being 23% low of the experimental value. Again, a great deal of this discrepancy may be due to the variation within the GFRP material itself. Increasing the GFRP strength by 15% (one standard deviation) in the flexural strength analysis results in the predicted strength jumping from the 176 kN shown in Table 7.4 to 204 kN, much closer to the experimental value of 230 kN. This large jump is due not only to the increased load carrying ability of the GFRP itself but also to the increase in concrete confinement it provides.

7.2.5.2 - Shear Strengthening Specimens

A similar approach was taken with the final set of shear strength deficient beams from Phase G of the project. First, a flexural strength analysis was performed on the control specimen and on each retrofit scheme used in this phase (Schemes A, C and D). The analysis sequence for each of these specimen types is shown in Table 7.5.
All three of the retrofit schemes were first analysed without considering the contribution from the compression FRP and under the assumption that a concrete crushing failure would occur. Next, the analysis was redone for the three retrofit schemes with the compression FRP included but under the same failure mode assumption. This assumption turned out to be correct for the Scheme A and C retrofits, though the Scheme D arrangement produced a failure stress in the bottom FRP. Analysis of this scheme was then performed once again under the assumption that an FRP tensile fracture would ensue.

<table>
<thead>
<tr>
<th>Retrofit Scheme</th>
<th>Specimen Condition</th>
<th>Peak Load (kN)</th>
<th>Failure Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>Control</td>
<td>Concrete Crushing Assumed</td>
<td>100</td>
<td>Concrete Crushing</td>
</tr>
<tr>
<td>A</td>
<td>No Compression FRP Concrete Crushing Assumed</td>
<td>103</td>
<td>Concrete Crushing</td>
</tr>
<tr>
<td></td>
<td>Compression FRP Added Concrete Crushing Assumed</td>
<td>105</td>
<td>Concrete Crushing</td>
</tr>
<tr>
<td>C</td>
<td>No Compression FRP Concrete Crushing Assumed</td>
<td>124</td>
<td>Concrete Crushing</td>
</tr>
<tr>
<td></td>
<td>Compression FRP Added Concrete Crushing Assumed</td>
<td>129</td>
<td>Concrete Crushing</td>
</tr>
<tr>
<td>D</td>
<td>No Compression FRP Concrete Crushing Assumed</td>
<td>139</td>
<td>Concrete Crushing</td>
</tr>
<tr>
<td></td>
<td>Compression FRP Added Concrete Crushing Assumed</td>
<td>N/A</td>
<td>FRP Fracture</td>
</tr>
<tr>
<td></td>
<td>Compression FRP Added FRP Fracture Assumed</td>
<td>165</td>
<td>FRP Fracture</td>
</tr>
</tbody>
</table>

A shear strength analysis was also performed on each of these specimen configurations, with the results compiled in Table 7.6. As expected, the control specimens had limited shear
resistance compared to other three schemes, which received very large contributions from the sprayed GFRP applied to their sides.

Table 7.6: Shear strength analysis results - shear strengthening specimens.

<table>
<thead>
<tr>
<th>Retrofit Scheme</th>
<th>Individual Shear Components</th>
<th>Total Shear Strength $V_s$ (kN)</th>
<th>Ultimate Load $P_{ult}$ (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Control</td>
<td>$V_{cg}$ (kN)</td>
<td>N/A</td>
<td>22</td>
</tr>
<tr>
<td>A</td>
<td>21.8</td>
<td>0</td>
<td>196</td>
</tr>
<tr>
<td>C</td>
<td>21.8</td>
<td>173.7</td>
<td>196</td>
</tr>
<tr>
<td>D</td>
<td>21.8</td>
<td>173.7</td>
<td>196</td>
</tr>
</tbody>
</table>

Again, the lowest predicted value for each specimen (from the two analyses performed, flexural and shear) is shown in Table 7.7, along with the average result obtained in the laboratory for the same specimen configuration.

Table 7.7: Comparison of theoretical vs. experimental results - shear strengthening specimens.

<table>
<thead>
<tr>
<th>Retrofit Scheme</th>
<th>Predicted Result</th>
<th>Experimental Result</th>
<th>Difference (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Control</td>
<td>Peak Load (kN)</td>
<td>Peak Load (kN)</td>
<td>-8</td>
</tr>
<tr>
<td>A</td>
<td>44</td>
<td>48</td>
<td>9</td>
</tr>
<tr>
<td>C</td>
<td>105</td>
<td>96</td>
<td>9</td>
</tr>
<tr>
<td>D</td>
<td>129</td>
<td>137</td>
<td>-6</td>
</tr>
<tr>
<td>D</td>
<td>165</td>
<td>199</td>
<td>-17</td>
</tr>
</tbody>
</table>

The results for this series of beams are even more accurate than the flexural strengthening
specimens. This time, however, it was the Scheme A retrofit specimens that failed due to debonding of the FRP, as opposed to the predicted failure mode of concrete crushing. Again, the beams tested in the laboratory came very close to their predicted failure load before the bond failure occurred. They failed at a load of 96 kN, only 9% below the predicted failure load of 105 kN. All of the other predicted failure modes were identical to the laboratory results, with the control beams failing in shear, the Scheme C specimens due to concrete crushing and the Scheme D members by tensile fracture of the FRP.

The control specimens were predicted to fail in shear at a peak load of 44 kN, only 8% lower than the actual failure load found in the lab. The Scheme C prediction of 129 kN was only 6% low and the Scheme D prediction (165 kN) was 17% low. All of these variations can be explained by the variability inherent in the materials themselves, predominantly the GFRP.

It appears that the flexural and shear analysis techniques described in this chapter are sufficiently accurate for predicting both the ultimate load carrying ability and the failure mode of reinforced concrete beams retrofitted with sprayed GFRP. Analysis of the full scale bridge channel beams was performed by S. Ross in another work and has not been included here.
7.3 - Load-Deflection Prediction

To predict the load-deflection response of the beams tested in this project, a similar analysis technique to that described in Section 7.2.2 was implemented. However, in this case the various strains were all calculated with respect to the strain in the reinforcing steel. The steel strain value was increased incrementally (in steps of 0.00025) while the strains in the other materials (concrete and GFRP) were monitored to determine when, and how, failure occurred. Failure was deemed to occur once the strain in any particular material reached its ultimate value.

The applied load and deflection were also calculated at each steel strain increment. Load values were calculated as per the procedure outlined in Section 7.2.2 while the deflection at midspan was determined from the beam curvature. For this purpose, it was assumed that the curvature of the beam was perfectly circular in shape and that the length of the arc created by the neutral axis of the beam remained constant at 900 mm. Specimen curvature $\phi$ was calculated as

$$\phi = \frac{\varepsilon_{ct}}{c}$$

where $\varepsilon_{ct}$ is the strain at the extreme upper surface of the concrete. The radius of curvature $R_c$ could then be determined as the inverse of the curvature ($1/\phi$). From these values, the specimen's midspan deflection $\delta$ was calculated as
\[ \delta = R_c \cdot \left( 1 - \cos \left( \frac{s}{2R_c} \right) \right) \]

Though the exact curvature of the beam could have been determined and implemented in place of the circular assumption mentioned above, the circular approximation was used for simplicity.

Thus, for each increment in steel strain, the iterative analysis approach described in Section 7.2.2 was performed until the neutral axis depth corresponding to static equilibrium was found. Material strains were checked to determine whether failure had occurred and the values for applied load and deflection were recorded. All calculations were carried out with the use of a spreadsheet to simplify the iteration process. The data were then plotted to show the load-deflection characteristics of that particular beam arrangement.

This entire procedure was then repeated for the other specimen configurations. In all, 8 different beam combinations were examined, four each from the flexural and shear strengthening portions of the project. This allowed for the inclusion of retrofit Schemes E, C and D from the flexural specimens and Schemes A, C and D from the shear specimens (along with their respective control specimens).

7.3.1 - Material Characteristics

In the previous section, the ultimate load carrying abilities of the various reinforced concrete
beams used throughout this project were predicted. To simplify this procedure, the stress-strain relationship for reinforcing steel was assumed to be bilinear, as is the typical practice in reinforced concrete design, while an equivalent rectangular stress block was used for concrete. Since only one point was of interest (the peak load), this approach was satisfactory. In order to accurately predict the entire load-deflection curve prior to this peak load, however, a more definitive stress-strain relationship is required for each material.

As mentioned in Section 7.2.1, CSA G30.18 specifies the minimum and maximum yield strengths of 400 Grade reinforcement as 400 MPa and 525 MPa, respectively, resulting in an average yield strength of 462 MPa. The load carrying ability of the steel increases beyond this yield point until reaching its ultimate load level and failing. For analysis purposes, this extra load carrying ability must be included into the calculations to improve accuracy. Toward this end, a stress-strain relationship was developed for the range of steel strains encountered in the loading history of these beams. This relationship is depicted in Figure 7.13.

The elastic modulus was again assumed to be 200 GPa while the yield strength was lowered to 460 MPa in order to nominally represent the average value specified by the CSA standard. Beyond the yield point, the stress-strain behaviour was assumed to follow the trend reported by Nilson\textsuperscript{102} and Mander.\textsuperscript{103} An expression was derived to allow calculation of the load carried by the steel directly from the strain.
The stress-strain relationship can thus be described as

\[ f_s = \varepsilon_s \cdot E_s \]

for strains in the elastic range \((\varepsilon_s < 0.00230)\). Beyond this yield point, the steel stress can be determined using the equation

\[ f_s = 417 + 9301\varepsilon_s - 171900\varepsilon_s^2 + \frac{0.5507}{\varepsilon_s} - \frac{0.001122}{\varepsilon_s^2} \]
Developing a similar expression for the stress-strain relationship of concrete was a little more complex since this relationship would be affected by the two different levels of confinement provided in the Scheme C and D retrofit arrangements. To further compound the problem, the concrete strength varied between mixes. A general formula was needed to allow modifications to these variables as needed. The research by Mander et al\textsuperscript{98} into confinement of concrete again provided just such an expression.

A graphical representation of the Mander model for confined concrete is shown in Figure 7.14.

![Figure 7.14: Mander model for confined concrete\textsuperscript{98}.](image)

This model allows calculation of the concrete stress directly from the strain using the expression

\[
f_c = \frac{f'_c x r}{r - 1 + x^r}
\]
where $f_{ce}$ is the compressive strength of the confined concrete (defined in Section 7.2.3) and $x$ is determined by

$$x = \frac{\varepsilon_c}{\varepsilon_{cc}}$$

where $\varepsilon_c$ is the actual compressive strain applied to the concrete and $\varepsilon_{cc}$ is calculated from the expression

$$\varepsilon_{cc} = \varepsilon_{co} \left[ 1 + 5 \left( \frac{f'_{ce}}{f'_{co}} - 1 \right) \right]$$

where $f'_{co}$ represents the concrete's unconfined compressive strength and $\varepsilon_{co}$ the strain corresponding to this strength. Mander et al suggest that $\varepsilon_{co}$ can be assumed equal to 0.002 (one half of the ultimate strain for unconfined concrete). The variable $r$ is defined as

$$r = \frac{E_c}{E_c - E_{sec}}$$

where $E_c$ is the tangent modulus of elasticity, which can be approximated as

$$E_c = 5000 \sqrt{f_{co}'}$$
The secant modulus of elasticity $E_{\text{sec}}$ is further defined as

$$E_{\text{sec}} = \frac{f'_{\text{cc}}}{\varepsilon'_{\text{cc}}}$$

The Mander model can also be used to create the stress-strain relationship for unconfined concrete simply by setting the confined compressive strength $f'_{\text{cc}}$ equal to the unconfined strength $f'_c$. Applying this model to the two concretes used in the analysis section, for the flexural strengthening and shear strengthening beams, results in the stress-strain curves depicted in Figures 7.15 and 7.16, respectively.

![Stress-strain relationship curve](image)

**Figure 7.15:** Stress-strain relationship - concrete (flexural beams).
These figures show the curves for unconfined concrete (applicable to the control specimens as well as the Scheme A and E retrofits) as well as for the two retrofit schemes providing confinement.

Unfortunately, such a relationship results in a non-linear stress distribution in the concrete. Typically, the concrete stress distribution would be divided into a number of regions, depending upon the shape of the stress-strain curve, and a force would be determined for each of these regions. Calculation of this force would require integrating the stress-strain relationship over the depth of the region corresponding to that force. In this case, such an approach was not feasible due to the very large number of calculations required so a simplified method was devised.
The concrete compression zone was divided into four regions, with the stress at the midpoint of each region taken to be the average stress over that region. Inherently, this assumes that the stress distribution within each region is linear, with the midpoint stress being the average value. By breaking the stress distribution into these smaller segments and determining the internal concrete forces using the midpoint stresses, the calculations could be more easily adapted for implementation in the spreadsheet.

These stress-strain relationships (reinforcing steel and concrete) were used throughout the load-deflection behavior prediction process. For the sprayed GFRP material, the existing stress-strain behavior described in Section 7.2.1 was again implemented.

7.3.2 - Flexural Strengthening Specimens

The predicted load-deflection curves for each flexural strengthening beam configuration will be presented in this section. For each case, the actual test results from the flexural strengthening portion of the project have been plotted along with the predicted behaviour for comparison purposes. The actual test result curves are continued a short distance beyond their peak load points while the predicted curves are terminated once failure occurs.

Though the concrete stress-strain relationships depicted in the previous section imply that some further load can be carried by the concrete beyond its ultimate load, in practice this would require closed loop testing equipment in order to avoid catastrophic failure and to allow accurate measurement of member deflection. Such a system was not used in the
laboratory testing, meaning comparison of the curves beyond the peak load point would not be valid. The other failure mode (GFRP tensile fracture) is catastrophic by nature and thus no post-peak performance is relevant. To begin with, the control specimen results are depicted in Figure 7.17.

Figure 7.17: Load-deflection curves - flexural beams (controls).

As shown in this figure, the predicted load-deflection curve for these control specimens is very close to those corresponding to the laboratory specimens. The primary deviation occurs at failure, where the predicted failure deflection is slightly lower than that measured in the lab. Figure 7.18 depicts the predicted and actual curves for the flexural specimens retrofitted with Scheme E.
Here we can see that initially the predicted behaviour closely follows that of the laboratory specimens but soon deviates from them and eventually reaches a higher failure load. This discrepancy is due to the failure mode exhibited by the specimens tested in the lab. These beams failed due to debonding of the GFRP coating from the concrete surface, as opposed to the predictive model which reached failure due to crushing of the concrete in the compression zone at the upper surface of the beam. In this case, this debonding occurred slowly, as is typical for the sprayed GFRP material.

Going on to the Scheme C retrofit, the relevant load-deflection curves are presented in Figure 7.19.
Here, the predicted curve falls just below the test results, though it is well within the variation inherent in the materials being used (specifically the GFRP). Otherwise, the predicted curve corresponds very well with respect to the reinforcement yield point (evidenced in the figure by the slope change at the end of the initial linear portion of the curve) and the ultimate load carried by the test beams. On the other hand, the predicted deflection at failure is slightly higher than those found in the lab.

Figure 7.20 shows the curves related to the Scheme D retrofit configuration.
Again, the predicted curve falls slightly below those from the laboratory results. The predicted reinforcement yield point corresponds well but the peak load is significantly lower than what was found in the test results. Again, this variation could be at least partially due to variations in GFRP strength and thickness. These beams came from an early phase of the project (actually they were the first ones sprayed at U.B.C. after the equipment was obtained) when control and thus consistency of the material thickness was not yet very good.

To illustrate the effect of GFRP strength variations on the load-deflection results, the prediction procedure for this specimen configuration was repeated with the ultimate GFRP strength increased by 15% (one standard deviation). The results are shown in Figure 7.21.
Though the predicted curve still falls below the lab results, this exercise does show that variations in material properties (such as GFRP strength in this case) can have a very large influence on member performance. This revised prediction only takes into account a 15% increase in GFRP strength to allow for the 15% coefficient of variation found for this material. However, as the strength increases, so does the stiffness. The coefficient of variation within the initial elastic modulus results was found to be as high as 10% as well. Combined, these two variations can have a significant effect on member properties, a fact that must be taken into serious consideration before this material can be considered for commercial application.
Revising the predicted peak load/failure mode table from Section 7.1.5.1 (Table 7.4) to reflect the results obtained in this section gives Table 7.8.

<table>
<thead>
<tr>
<th>Retrofit Scheme</th>
<th>Predicted Result</th>
<th>Experimental Result</th>
<th>Difference (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Peak Load (kN)</td>
<td>Failure Mode</td>
<td>Peak Load (kN)</td>
</tr>
<tr>
<td>Control</td>
<td>60</td>
<td>Concrete Crushing</td>
<td>63</td>
</tr>
<tr>
<td>E</td>
<td>95</td>
<td>Concrete Crushing</td>
<td>86</td>
</tr>
<tr>
<td>C</td>
<td>133</td>
<td>Concrete Crushing</td>
<td>136</td>
</tr>
<tr>
<td>D</td>
<td>181</td>
<td>GFRP Tensile Fracture</td>
<td>230</td>
</tr>
</tbody>
</table>

Comparing these results with the previous values in Table 7.4 indicates that the analysis approach used in this section produced results even closer to those obtained in the lab specimens. It was also capable of producing the correct failure mode for the Scheme C retrofit, which was previously predicted as GFRP tensile fracture. These improvements are due to the more representative stress-strain relationships that were used for steel and concrete in this section.

7.3.3 - Shear Strengthening Specimens

Since the shear strengthening specimens were fundamentally different in their actual design than the flexural strengthening specimens, load-deflection predictions for the three retrofit schemes used with these beams (A, C and D) were also performed. The results from the control specimens are depicted in Figure 7.22 below.
This comparison really doesn’t show much since the control specimens failed in shear while the analysis procedure used to predict their behaviour examined only their flexural performance. As a result, the prediction curved in Figure 7.22 represents what should occur if these beams had sufficient shear capacity to reach the load levels indicated. Since these beams contained no shear reinforcement whatsoever, they failed at much lower loads. The Scheme A retrofit (coating both sides of the beam) was designed to provide this extra shear capacity. Unfortunately, these beams tended to fail due to debonding of the GFRP from the concrete surface. Results and predicted behaviour are shown in Figure 7.23.
As with the debonding evidenced in the Scheme E flexural specimens, the load-deflection curves here also gradually deviate from the predicted result and failure typically occurs at lower load levels. As previously noted in Section 7.2.5.2, these beams appear to have come very close to achieving their predicted ultimate loads before the debonding occurred. One of the laboratory tested beams did manage to reach the predicted load level, though it still failed due to debonding.

The predicted and lab testing results from Scheme C are depicted in Figure 7.24. In this case, the correlation between predicted and actual results is very good. In fact, they are virtually identical until just before the failure load is reached. Though some deviation is evident at this point, the predicted failure load is within 4% of the average result reported in the lab.
Finally, the load-deflection curves for the Scheme D specimens are shown in Figure 7.25. Again, the correlation between the predicted and laboratory results is extremely close up to the predicted failure load. As with the flexural specimens, this predicted value falls below those exhibited by the test specimens. In this case, however, it appears that the variation may be due to GFRP strength variations alone since the predicted stiffness is much better.
Revising the predicted peak load/failure mode table from Section 7.2.5.2 (Table 7.7) to reflect the results obtained in this section gives the values shown below in Table 7.9.

<table>
<thead>
<tr>
<th>Retrofit Scheme</th>
<th>Predicted Result</th>
<th>Experimental Result</th>
<th>Difference (%)</th>
</tr>
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<tbody>
<tr>
<td></td>
<td>Peak Load (kN)</td>
<td>Failure Mode</td>
<td></td>
</tr>
<tr>
<td>Control</td>
<td>105</td>
<td>Concrete Crushing</td>
<td>48 Shear</td>
</tr>
<tr>
<td>A</td>
<td>107</td>
<td>Concrete Crushing</td>
<td>96 GFRP Debonding</td>
</tr>
<tr>
<td>C</td>
<td>132</td>
<td>Concrete Crushing</td>
<td>137 Concrete Crushing</td>
</tr>
<tr>
<td>D</td>
<td>170</td>
<td>GFRP Tensile Fracture</td>
<td>199 GFRP Tensile Fracture</td>
</tr>
</tbody>
</table>
The results for the control specimens cannot be compared since shear strength was not considered in the load-deflection prediction performed in this section. The other predicted values have improved slightly and are within tolerances set by material variability.

7.4 - Discussion

7.4.1 - Beam Failure Mechanism

Upon examination of the results from the structural analysis section, a number of observations can be made concerning the behavior of the beams during testing. As the applied load is increased, the midspan deflection of the member also increases in a linear manner up to the point where the strain in the reinforcing steel reaches its ultimate value. At this time, the steel begins to yield, resulting in a much larger rate of deformation (i.e. increase in deflection). Identification of the reinforcement yielding point on the load-deflection diagrams is typically very easy since it is represented by a significant change in slope, as can be seen quite clearly for the flexural specimens in Figure 6.9 and for the shear strengthening specimens in Figure 6.39. For those members with sprayed FRP applied to their bottom face, the strain in this bottom FRP is below 0.0040 at this time.

As loading continues, the strains in all materials continue to increase until the next limiting value is reached. If the beam has no FRP applied to the upper surface (i.e. Schemes C and E), then the next material to reach its ultimate strain will be the concrete in the compression zone at the upper surface of the specimen and a crushing failure will ensue, as indicated in Figure 6.11. For the specimens in which FRP was applied to the top surface as well (i.e. Scheme D),
the next limiting value will be the ultimate strain of the bottom surface FRP, resulting in
tensile fracture of this FRP, as depicted in Figure 6.12.

In order for this failure mode to occur, the strain in the concrete at its upper surface must be
able to reach levels well above its ultimate unconfined value. Thus, tensile fracture of the
FRP can only occur in retrofit schemes providing sufficient confining stress to increase the
effective ultimate strain of the concrete to the necessary levels.

There are also two other failure modes that can supersede the sequence described above.
First, should the shear capacity of the member be exceeded, then a shear failure will occur at
that point. Second, failure of the FRP - concrete bond may also occur before either concrete
crushing or FRP tensile fracture takes place. Once debonding occurs, any contribution from
the debonded FRP is immediately lost and beam behavior reverts to that of an unretrofitted
specimen.

7.4.2 - Bond Performance

The primary failure mode reported by other researchers using plate bonding and fabric
wrapping techniques is debonding of the FRP retrofit material. Though the bond itself has
not been studied in the research project presented here, a brief description of the issue
follows.

The actual strength of the bond is dependent on two properties; the adhesion capacity of the
bonding agent itself and the mechanical interlock produced by the roughened concrete surface. The debonding issue appears to be less prevalent in the sprayed FRP technique, likely due to the use of a rougher substrate. Plate bonding and fabric wrapping techniques require a fairly smooth surface in order to avoid discontinuities which can induce out of plane stresses and thus lead to localized debonding. This approach is required due to the continuous nature of the fibers and the constant thickness of the composite.

The sprayed FRP, on the other hand, is much more adaptable to surface defects and is, in fact, applied to a sandblasted surface. Not only does this rougher surface improve the mechanical interlock between the FRP resin and concrete, but it also permits adhesive bonding directly to the exposed aggregates. Additionally, the sprayed technique normally produces a single interface or bond line, with the matrix resin acting as the bonding agent as well. This direct linkage between the matrix and the concrete substrate improves the efficiency of the stress transfer across the bond.

For the plate bonding or fabric wrapping techniques, there are actually two interfaces that need to be considered, one between the concrete surface and the epoxy adhesive and a second between the epoxy adhesive and the applied plate or fabric. Optimizing both of these interfaces with the same adhesive can be very difficult.

In the later portions of the research, a vinyl ester coupling agent was applied to the concrete surface prior to spraying in an attempt to improve the FRP-concrete bond. Since the polyester matrix resin was applied before setting of the vinyl ester, there was chemical
interaction between the two resins, resulting in an excellent chemical bond. Moreover, the vinyl ester produces a superior adhesive bond with the concrete than the polyester, resulting in a better FRP-concrete bond.

There are a number of factors influencing the required strength of the bond. Two possible bond failure modes were identified during the testing program. The most prevalent of these was the Type II or shear failure which was evident in the Scheme A and E retrofitted members. The Scheme C specimens tended to exhibit Mode I delamination as the FRP applied to the sides of the beam buckled under high member deformations, resulting in the FRP pulling directly away from the concrete surface. This Mode I delamination only occurred after the specimen had reached its ultimate load and is thus much less important than the shear failure mode.

Examining the bond itself, it is essentially subjected to a shearing stress as the material to one side (FRP) is subjected to a higher strain than the material on the other side (concrete). The magnitude of this shearing stress is not constant over the entire bond, and actually increases in the regions adjacent to the ends of the FRP. This increase is induced by the transition from a plane stress state near the middle of the FRP to a three-dimensional stress state at and near the free edge represented by the end of the FRP material. The variation in this stress is shown in Figure 7.26.

It is this phenomenon that causes the debonding to initiate at the ends of the applied FRP. The length of the region over which this interlaminar shear stress acts is directly proportional
to the thickness of the FRP, resulting in an increase in shearing stress as FRP thickness increases. This relationship is actually a drawback for the sprayed FRP technique since the material thickness is significantly higher with this technique. Adding to the problem is the fact that a thicker FRP plate is also going to be stiffer and thus less willing to conform to the deformed shape of the beam during loading, resulting in the introduction of a peeling stress as well. Counteracting these effects, to some extent, is the simple fact that a thicker material shifts the location of the FRP-concrete bond further away from the extreme fiber and thus lowers the strains to which the bond is subjected.

Figure 7.26: Stress distribution in a laminate at a free edge.109
Once delamination has initiated, the sprayed FRP requires higher fracture energies to maintain crack propagation than continuous fiber systems. Thus, once debonding has begun, it proceeds very quickly in the continuous fiber systems while the sprayed material debonds much more gradually.

Another difference between the two materials is their reaction to cracking of the concrete. As mentioned previously, the sprayed material is more adaptable to the out of plane dislocations that can occur over a crack. It is also better at handling the large stress concentrations created when a new crack forms, resulting in very large localized strains. The higher strain capabilities of the sprayed material allow it to alleviate these localized stress concentrations quickly and leads to multi-cracking, which in turn, distributes the additional strains due to cracking over a much wider area.

Though the preceding discussion covers the general concepts related to bond failure of FRP retrofit materials, it is by no means a definitive work on the subject. A great deal of investigation into such bond performance, especially for the sprayed FRP technique, is necessary before such a system could be safely implemented in the construction industry.
8.1 - Introduction

One other advantage that the sprayed GFRP technique has over the commercially available products is its economy. Specifically, the fiber used in the spraying system is in a roving format. The fabrics used in the wrapping procedures are typically constructed from these same rovings, whether they are woven or stitched together. This extra mechanical step tends to have a significant impact on the final cost of the material.

This section will present a material cost comparison between the sprayed GFRP approach and the commercially available MBrace® system. As described in Section 6.2., these were the two retrofit techniques used in the rehabilitation of the large scale bridge channel beams.
8.2 - Material Cost - Sprayed GFRP

First, the relevant information (retail price, density and coverage) for the sprayed GFRP approach has been compiled in Table 8.1.

<table>
<thead>
<tr>
<th>Component</th>
<th>Retail Price ($/kg)</th>
<th>Density (kg/m³)</th>
<th>Coverage (m²/L)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Resin: Derakane 8084 Vinyl Ester</td>
<td>7.93 × 10⁴</td>
<td>1020 × 10³</td>
<td>6.5 × 10⁴</td>
</tr>
<tr>
<td>Resin: K1907 Polyester</td>
<td>3.38 × 10⁴</td>
<td>1080 × 10⁴</td>
<td>-</td>
</tr>
<tr>
<td>Catalyst: Lupersol DDM9 MEKP</td>
<td>7.80 × 10⁴</td>
<td>1000 × 10⁴</td>
<td>-</td>
</tr>
<tr>
<td>Glass Fiber: Advantex 360RR</td>
<td>3.55 × 10⁴</td>
<td>2620 × 10⁶</td>
<td>-</td>
</tr>
</tbody>
</table>

The retail prices shown in Table 8.1 came directly from the manufacturer/supplier, Ashland Canada Ltd., as did the density values for their products. These prices were obtained in October of 1999. The cost per square metre of concrete surface to be sprayed will be calculated for each of these components.

First, the cost of the coupling agent (Derakane 8084 Vinyl Ester Resin) can be determined directly since the quantity required is dependent entirely upon the surface area to be covered and is unaffected by the final thickness of the sprayed GFRP composite.

\[
\text{Cost} = \frac{\rho \cdot \text{Retail Price}}{1000 \cdot \text{Coverage}} = \frac{1020 \times 10³ \cdot 7.93}{1000 \cdot 6.5} = 1.24 \text{ per m}^²
\]
As for the composite itself, there are three components that must be included, the polyester resin, the MEKP catalyst and the glass fiber. As previously discussed in Chapter 5, the volume fraction of fiber $V_F$ is approximately 19%, leaving a matrix volume fraction $V_M$ of 81%. Since the catalyst content in the matrix was set to 3%, the latter value can be further subdivided into volume fractions for resin $V_R$ and catalyst $V_C$, which are 78.6% and 2.4%, respectively.

The final cost of each of these components will depend upon the applied thickness of the sprayed GFRP material. Consequently, each of these costs will be determined in terms of $t$, the composite thickness.

\[
\text{Composite Resin - K1907 Polyester Resin}
\]

\[
\text{Cost} = \frac{V_R \cdot \rho \cdot \text{Retail Price} \cdot t}{1000} = \frac{0.79 \cdot 1080 \cdot 3.38 \cdot t}{1000} = \$2.88t \text{ per m}^2
\]

\[
\text{Composite Catalyst - Lupersol DDM9 MEKP}
\]

\[
\text{Cost} = \frac{V_C \cdot \rho \cdot \text{Retail Price} \cdot t}{1000} = \frac{0.02 \cdot 1000 \cdot 7.80 \cdot t}{1000} = \$0.15t \text{ per m}^2
\]

\[
\text{Glass Fiber - Advantex 360RR}
\]

\[
\text{Cost} = \frac{V_F \cdot \rho \cdot \text{Retail Price} \cdot t}{1000} = \frac{0.19 \cdot 2620 \cdot 3.55 \cdot t}{1000} = \$1.77t \text{ per m}^2
\]
The total cost of the sprayed GFRP laminate would thus be

\[
\text{Composite Cost} = (2.88 + 0.15 + 1.77)t = 4.80t \text{ per } \text{m}^2
\]

Adding the cost of the vinyl ester coupling agent results in a total material cost of

\[
\text{Total Material Cost} = (1.24 + 4.80t) \text{ per } \text{m}^2
\]

where \(t\) is the thickness of the sprayed GFRP coating in millimetres.

8.3 - Material Cost - MBrace\textsuperscript{®} System

The MBrace\textsuperscript{®} composite strengthening system is made up of a number of components. Those components used in the rehabilitation of the large scale bridge channel beams in this project are shown, along with their retail prices and coverage ratings, in Table 8.2.

<table>
<thead>
<tr>
<th>Component</th>
<th>Retail Price (^{107})</th>
<th>Coverage (^{108}) (m(^2)/L)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Primer</td>
<td>$20.67 per L</td>
<td>5.50</td>
</tr>
<tr>
<td>Saturant</td>
<td>$16.62 per L</td>
<td>0.73</td>
</tr>
<tr>
<td>EG 900 E-Glass Reinforcement</td>
<td>$16.20 per m(^2)</td>
<td>-</td>
</tr>
</tbody>
</table>

In this case, the retail pricing again comes directly from the manufacturer, Master Builders Technologies, Inc., and were obtained in September of 1999.
The cost of each component has again been calculated, though with this system the costs are not dependent upon the composite thickness but on the number of layers of material applied. Again, referring to the large scale bridge channel beams from Section 6.2., the following prices are for a single layer of EG 900 E-glass reinforcement.

<table>
<thead>
<tr>
<th>Component</th>
<th>Retail Price</th>
<th>Cost Calculation</th>
<th>Cost per m²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Primer</td>
<td>20.67</td>
<td>Cost = \frac{\text{Retail Price}}{\text{Coverage}} = \frac{20.67}{5.5} = $3.76 per m²</td>
<td></td>
</tr>
<tr>
<td>Saturant</td>
<td>16.62</td>
<td>Cost = \frac{\text{Retail Price}}{\text{Coverage}} = \frac{16.62}{0.73} = $22.77 per m²</td>
<td></td>
</tr>
</tbody>
</table>

The total cost of the materials for the MBrace® system as used in this project would thus be

\[ \text{Total Cost} = 16.20 + 3.76 + 22.77 = $42.73 \text{ per m}^2 \]

It should be noted that this cost does not include a number of items described in the actual MBrace® manual as being part of the system. After the primer coat is applied, a special putty is suggested in order to fill any holes in the concrete surface and to level uneven surfaces. The manual also mentions the possibility of a second coat of primer and a protective top coat which is applied over the finished product. Any of these additional items will increase the cost of the system even further, though pricing was not obtained since they
are somewhat optional. The following comparison will not take these extra items into account.

**8.4 - Comparison**

As mentioned the total material cost of the MBrace® system with one layer of EG 900 E-glass reinforcement is $42.73 per m$^2$ of concrete surface while the cost of the sprayed GFRP material depends upon the thickness of the applied GFRP. Table 8.3 provides a comparison of the material costs related to these two systems for a range of sprayed GFRP thicknesses.

The material cost of the MBrace® system is equivalent to about 8.6 mm of the sprayed GFRP. In the section dealing with the rehabilitation of large scale bridge channel beams, a structural comparison was done between a single layer of the MBrace® system and an 8 mm coating of the sprayed GFRP material. In that case, the sprayed material performed better. Now we can see that it is also more economical from a material cost perspective.

| Table 8.3: Material cost comparison - MBrace® system vs. sprayed GFRP. |
|---------------------|---------------------|---------------------|---------------------|
| MBrace® Cost ($/m^2$) | Thickness (mm) | Sprayed GFRP Cost ($/m^2$) | Difference (%) |
| 42.73 | 3.5 | 18.04 | 137 |
|        | 4.0 | 20.44 | 109 |
|        | 6.0 | 30.04 | 42 |
|        | 8.0 | 39.64 | 8 |
|        | 8.65 | 42.71 | - |
There are a number of other cost issues that could eventually be added to this comparison. The sprayed system should be much easier and less time intensive to implement in the field, resulting in a further cost savings from a reduction in required labour and a shorter downtime for the structure being retrofitted. Such a comparison could not be performed at this time since the spraying equipment cannot currently be used for field application where overhead spraying is required. Modifications to this equipment are necessary to allow such operation.

Another issue is the concrete surface itself. Continuous fiber fabrics, like the MBrace® system, must be applied to a flat surface. As previously mentioned, the manufacturer suggests the application of a special putty to achieve this condition\textsuperscript{108}, which would entail additional labour and downtime costs. Since the sprayed GFRP is a built-up material, it can be applied directly onto the original rough surface immediately after sandblasting (which both techniques require).
9 - Conclusions & Recommendations

9.1 - Conclusions

In general, the results obtained in this research project support the hypothesis that sprayed glass fiber reinforced polymers do have the potential to become a feasible alternative to existing rehabilitation techniques for reinforced concrete beams.

The spraying technique implemented in this project is capable of producing a consistent fiber reinforced polymer which exhibits two dimensionally isotropic strength properties. With the actual equipment used for this research, a glass fiber reinforced polyester matrix was produced. This material exhibited a maximum composite strength of 110 MPa and an elastic modulus of 12 GPa at a fiber volume fraction of 19%. Strength properties are highly dependent upon fiber length, with the values shown above referring to a material containing 48 mm fibers, the maximum possible with the equipment used.
It appears to be possible to accurately predict the strength properties of the sprayed FRP material from the properties of the individual components, based upon existing composite material theory. Modifications must be made to account for the random orientation and the discontinuous nature of the fibers. A model was developed that worked very well for the combination of materials used in this project.

The application process (spraying) is very straightforward and requires a minimal amount of operator training. Material thickness is directly controlled by the operator, resulting in more flexibility in the design process when compared to the application of continuous fiber fabrics (requiring multiple layers) or FRP plates (specified thicknesses).

One benefit of the discontinuous fibers is their ability to bend around much sharper corners than continuous fiber fabrics. This can be extremely important in ensuring that the FRP applied to adjacent retrofitted sides of the member act together instead of behaving as individual units. Such a response is especially beneficial in preventing debonding of the FRP on the tension side of the member when the adjacent sides are retrofitted as well.

The discontinuous and random orientation of the fibers also appears to enhance resistance to debonding of the sprayed FRP material. Delamination tends to more gradual than with continuous fiber systems, probably due to differences in the fiber-to-fiber stress transfer mechanism.
The sprayed FRP material was shown to be capable of significantly increasing both the strength and stiffness of existing concrete beams, while at the same time dramatically improving their energy absorption characteristics. In flexural strengthening situations, these increases appear to be in the same range, or slightly lower, than the improvements typically produced by FRP fabric and plate bonding techniques. For shear strengthening purposes, however, the sprayed GFRP material is capable of producing far better results than any of the other techniques. Direct comparison of the sprayed FRP technique with a commercially available fabric wrapping system for the rehabilitation of large scale members confirmed that this technique is capable of producing superior results.

Prediction of the load carrying ability and load-deflection behavior of retrofitted specimens is possible and accurate within the bounds induced by material variability. Subsequently, design parameters for this rehabilitation technique could be devised to make sprayed FRP rehabilitation projects simple, efficient and safe.

From an economic perspective, the sprayed FRP technique represents a significant reduction in material cost over the commercially available fabric wrapping system studied. This cost savings could potentially become much larger once labor and structure downtime costs associated with the application technique are considered.
9.2 - Recommendations for Future Research

Probably the most obvious suggestion for continued research into the sprayed FRP technique is to investigate the feasibility of implementing this process in the field. The design of the apparatus used in this process appears to be well adapted for such use since it is completely portable and requires minimal support equipment. However, in its current configuration on-site use would be very difficult since the spraying of overhead surfaces is virtually impossible without modifications to equipment design. Such modifications should be pursued with the aid of one or more of the manufacturers of this type of apparatus.

The individual components of the sprayed FRP material must be optimized to provide the best possible properties. This would include such issues as determining the best fiber and resin types for both structural and durability performance. The existing combination of glass fiber and polyester resin is not necessarily the best possible alternative. Unfortunately, changing fiber types would require further equipment modification to eliminate the problems associated with the other fibers available, including static buildup with carbon fiber and cutting difficulties with aramid.

Optimization of the resin may involve a couple of steps. First, the durability of the composite resin must be investigated to ensure sufficient performance in the field. There is a vast array of alternative resins available for different exposure conditions. Second, the choice of coupling agent must be examined in order to maximize the FRP to concrete bond. Again, a large selection of potential products are available, though performance will depend upon the
choice of composite resin. Finally, the possibility of combining the two resins should also be considered. A composite resin with good bonding characteristics would certainly be beneficial from an application perspective. Unfortunately, high bond strength resins tend to be significantly more expensive as well. This was the original reason why a separate coupling agent was originally chosen instead of simply using it as the composite resin.

One of the primary failure modes encountered in FRP retrofit of concrete structures is debonding of the FRP. Though this problem is less common with the sprayed FRP technique, it does still exist. There is a definite need to investigate this interface to develop a better understanding of the mechanisms involved in the debonding process and to identify methods or materials that will assist in avoiding this issue altogether.

In this project, it was found that the sprayed FRP technique is not quite as effective as some of the continuous fiber fabric or plate bonding techniques for flexural strengthening of reinforced concrete beams. On the other hand, the sprayed material was found to be much more effective in shear strengthening. In order to take advantage of the optimum performance of both techniques, a hybrid system using continuous fiber materials and sprayed FRP at the same time would be feasible. An economic evaluation would be necessary to determine whether such a hybrid system should be considered or if it would simply be better to apply a thicker coating of the sprayed FRP.

The final step in deciding whether this is a useful rehabilitation technique is to determine its ability to withstand the elements and other detrimental influences out there in the field. The
durability characteristics of the material must be fully investigated. Of particular concern is the FRP-concrete interface. One specific problem associated with the FRP-concrete interface is the potential for moisture buildup along this interface due to the vast difference in permeabilities between the two materials. In a freezing environment this could result in the formation of an ice lense and lead to exfoliation of the FRP reinforcement.
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Material Property Testing Data
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Table A6: Sprayed FRP material property testing - fiber content results.

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<th>Specimen ID</th>
<th>Fiber Length (mm)</th>
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<th>Volume Fraction (%)</th>
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Calculation of Theoretical Tensile Strength of Sprayed FRP Composites

Mechanical Properties of Components

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<th>Glass Fiber</th>
<th>Polyester Resin</th>
<th>FRP Composite</th>
</tr>
</thead>
<tbody>
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<td>$\sigma_u$ (Tensile Strength)</td>
<td>$\sigma_{mu}$ (Tensile Strength)</td>
<td>$\rho_{up}$ Ultimate Density</td>
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<td>3450 MPa</td>
<td>45 MPa</td>
<td>1395 kg/m³</td>
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<tr>
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<td>3.7 GPa</td>
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<td>48 MPa</td>
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<td>$G_r$ (Shear Modulus)</td>
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<td>$A_f$ (Fiber Area)</td>
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Determination of Expected Tensile Strength

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<th>$L_f$ Fiber Length (mm)</th>
<th>$\rho_{up}$ FRP Fiber Density (kg/m³)</th>
<th>$V_f$ Fiber Fraction (%)</th>
<th>$V_v$ Void Fraction (%)</th>
<th>$C_v$ Voids Correction Separation ($\mu$m)</th>
<th>$\beta$ Fiber diffusion coeff. (m²/s)</th>
<th>$\eta_L$ Efficiency Factor</th>
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Correction for Fiber Length

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<th>$F_{2l}$ Transverse Strength (MPa)</th>
<th>$F_{6l}$ Shear Strength (MPa)</th>
<th>$\alpha_1$</th>
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<th>$F_{frp}$ Theoretical Strength (MPa)</th>
<th>$F_{frp}$ Measured Strength (MPa)</th>
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Calculation of Theoretical Elastic Modulus of Sprayed FRP Composites

Mechanical Properties of Components

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Determination of Expected Elastic Modulus

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Correction for Fiber Length

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Calculation of Theoretical Elastic Modulus of Sprayed FRP Composites

Mechanical Properties of Components

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<th>Polyester Resin</th>
<th>FRP Composite</th>
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<td>3450 MPa</td>
<td>(\sigma_m) (Tensile Strength)</td>
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<td>(E_r) (Elastic Modulus)</td>
<td>80.5 GPa</td>
<td>(E_m) (Elastic Modulus)</td>
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<td>(G_m) (Shear Modulus)</td>
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<tr>
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<td>(\eta)</td>
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Determination of Expected Elastic Modulus

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<th>(\rho_{frp}) (Fiber Fraction) (%)</th>
<th>(\rho_{frp}) (Void Fraction) (%)</th>
<th>(\rho_{frp}) (Matrix Fraction) (%)</th>
<th>(\rho_{frp}) (Separation Coeff.) (µm)</th>
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Correction for Fiber Length

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Flexural Strengthening
### External Sprayed GFRP Reinforcement

**Testing Data Summary - Phase 1**

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* - Failure modes: Y = Rebar Yielding, S = Shear, C = Concrete Crushing, B = GFRP Bond, P = GFRP Plate
Deflection (mm) vs. Load (kN)

- All Beams
- Scheme D
- Scheme C
- Control
- Scheme E

Load (kN) vs. Deflection (mm)

- Damaged Beam
- Undamaged Beam

Scheme E
Shear Strengthening - Initial Study
(Normal Strength Concrete)
## External Sprayed GFRP Reinforcement
### Testing Data Summary - Phase 2

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* Failure modes: Y = Rebar Yielding, S = Shear, C = Concrete Crushing, B = GFRP Bond, P = GFRP Plate
Scheme A

Deflection (mm)

Load (kN)

Damaged Beam
Undamaged Beam

Scheme B

Deflection (mm)

Load (kN)

Damaged Beam
Undamaged Beam
Shear Strengthening - Initial Study
(High Strength Concrete)
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*Failure modes: Y = Rebar Yielding, S = Shear, C = Concrete Crushing, B = GFRP Bond, P = GFRP Plate*
Scheme D

Deflection (mm)

Load (kN)

Individual Beam
Average

Deflection (mm)
Shear Strengthening - Effect of FRP Strength
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* Failure modes: Y = Rebar Yielding, S = Shear, C = Concrete Crushing, B = GFRP Bond, P = GFRP Plate

Notes:
1. A coupling agent (vinyl ester) was used to improve FRP - concrete bond.
2. FRP was applied in 3 layers, each consisting of 3 passes.
Shear Strengthening - Effect of FRP Thickness
### External Sprayed GFRP Reinforcement

**Testing Data Summary - Phase 4**

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<tr>
<td>Y2</td>
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<td>679</td>
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<td>C</td>
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<tr>
<td>Y3</td>
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<td>551</td>
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<td>C</td>
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<tr>
<td>Y4</td>
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<td>6.0</td>
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<td>P</td>
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<td>-</td>
<td>-</td>
<td>S</td>
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<td>B</td>
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<tr>
<td>Z3</td>
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<td>659</td>
<td>C</td>
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<td>4.5</td>
<td>C</td>
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<tr>
<td>Z4</td>
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<tr>
<td>Z6</td>
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<td>2210</td>
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<td>1 x 3</td>
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<td>P</td>
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</tbody>
</table>

* - Failure modes: Y = Rebar Yielding, S = Shear, C = Concrete Crushing, B = GFRP Bond, P = GFRP Plate

**Notes:**

1. A coupling agent (vinyl ester) was used to improve FRP - concrete bond.
2. Previous standard application = 3 passes x 3 layers.
Shear Strengthening - Effect of Fiber Length
## External Sprayed GFRP Reinforcement
### Testing Data Summary - Phase 5

<table>
<thead>
<tr>
<th>Beam ID</th>
<th>Peak Load (kN)</th>
<th>Initial Stiffness (kN/mm)</th>
<th>Energy Absorbed to Peak Load (N-m)</th>
<th>Retrofit Scheme</th>
<th>Fibre Length (mm)</th>
<th>Failure Mode</th>
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</thead>
<tbody>
<tr>
<td>U1</td>
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<td>P</td>
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<td>B</td>
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<td>C</td>
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<tr>
<td>V1</td>
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<td>B</td>
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<td>V4</td>
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<tr>
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<td>4791</td>
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<td>48</td>
<td>P</td>
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</tbody>
</table>

* - Failure modes: Y = Rebar Yielding, S = Shear, C = Concrete Crushing, B = GFRP Bond, P = GFRP Plate

**Notes:**

1. A coupling agent (vinyl ester) was used to improve FRP - concrete bond.
2. FRP was applied in 2 layers, each consisting of 3 passes.

<table>
<thead>
<tr>
<th># Rotor Blades</th>
<th>Fibre Length</th>
<th>Notes:</th>
</tr>
</thead>
<tbody>
<tr>
<td>12</td>
<td>8 mm</td>
<td>1. A coupling agent (vinyl ester) was used to improve FRP - concrete bond.</td>
</tr>
<tr>
<td>6</td>
<td>16 mm</td>
<td>2. FRP was applied in 2 layers, each consisting of 3 passes.</td>
</tr>
<tr>
<td>4</td>
<td>24 mm</td>
<td>Previous standard application</td>
</tr>
<tr>
<td>3</td>
<td>32 mm</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>48 mm</td>
<td></td>
</tr>
</tbody>
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Shear Strengthening - Wrap/Spray Comparison
**Fabric Wrap GFRP Reinforcement**  
**Testing Data Summary - Phase 6**

<table>
<thead>
<tr>
<th>Beam ID</th>
<th>Peak Load (kN)</th>
<th>Initial Stiffness (kN/mm)</th>
<th>Energy Absorbed to Peak Load (N·m)</th>
<th>Fabric Fiber Orientation</th>
<th>Retrofit Scheme</th>
<th>Number of Layers</th>
<th>Failure Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1</td>
<td>48</td>
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<td>52</td>
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<td>None</td>
<td>None</td>
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<td>B</td>
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<tr>
<td>S3</td>
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<td>B</td>
</tr>
<tr>
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<td>97</td>
<td>21.3</td>
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<td>B</td>
</tr>
<tr>
<td>S5</td>
<td>110</td>
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<td>358</td>
<td>±45</td>
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<td>B</td>
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<tr>
<td>S6</td>
<td>115</td>
<td>27.3</td>
<td>402</td>
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<td>3</td>
<td>B</td>
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<tr>
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<td>1</td>
<td>B</td>
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<td>T3</td>
<td>123</td>
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<td>0/90</td>
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<td>3</td>
<td>B</td>
</tr>
<tr>
<td>T4</td>
<td>91</td>
<td>19.2</td>
<td>269</td>
<td>±45</td>
<td>D</td>
<td>1</td>
<td>B</td>
</tr>
<tr>
<td>T5</td>
<td>117</td>
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<td>469</td>
<td>±45</td>
<td>C</td>
<td>3</td>
<td>C</td>
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<tr>
<td>T6</td>
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<td>23.4</td>
<td>398</td>
<td>±45</td>
<td>D</td>
<td>3</td>
<td>B</td>
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</tbody>
</table>

* - Failure modes: Y = Rebar Yielding, S = Shear, C = Concrete Crushing, B = GFRP Bond, P = GFRP Plate

**Notes:**

1. A coupling agent (vinyl ester) was used to improve FRP - concrete bond.
<table>
<thead>
<tr>
<th>Beam ID</th>
<th>Peak Load (kN)</th>
<th>Initial Stiffness (kN/mm)</th>
<th>Energy Absorbed to Peak Load (N-m)</th>
<th>Retrofit Scheme</th>
<th>Failure Mode</th>
</tr>
</thead>
<tbody>
<tr>
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<tr>
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<td>C</td>
</tr>
<tr>
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<td>C</td>
<td>C</td>
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<td>B</td>
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<tr>
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<td>B</td>
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<td>B</td>
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<td>21.1</td>
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<td>None</td>
<td>S</td>
</tr>
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<td>AB2</td>
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<td>24.8</td>
<td>3903</td>
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<td>P</td>
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<td>P</td>
</tr>
<tr>
<td>AB5</td>
<td>107</td>
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<td>399</td>
<td>A</td>
<td>B</td>
</tr>
<tr>
<td>AB6</td>
<td>93</td>
<td>21.6</td>
<td>291</td>
<td>A</td>
<td>B</td>
</tr>
</tbody>
</table>

* - Failure modes: Y = Rebar Yielding, S = Shear, C = Concrete Crushing, B = GFRP Bond, P = GFRP Plate

Notes:
1. A coupling agent (vinyl ester) was used to improve FRP - concrete bond.
2. GFRP thickness = 3.5 mm for all beams.
3. Fibre Length = 48 mm.
Rehabilitation of Full Scale Bridge Channel Beams
Figure 11: MOTH bridge channel beams - damage example 1.

Figure 12: MOTH bridge channel beams - damage example 2.
Figure 13: MOTH bridge channel beams - damage example 3.

Figure 14: MOTH bridge channel beams - damage example 4.
Theoretical Analysis Data
GU 8 mm Fiber Length

Eqn 2071 $y = a + bx + cx^2 + dx^3$

$r^2 = 0.999605908$ DF Adj $r^2 = 0.999585122$ FitStdErr = 0.215939543 Fstat = 15222.362

$a = -0.053375512$ $b = 6365.1457$ $c = -6271233.6$ $d = 32050408$

8 mm Fiber Length

Eqn 2071 $y = a + bx + cx^2 + dx^3$

$r^2 = 0.999387322$ DF Adj $r^2 = 0.99931058$ FitStdErr = 0.402211699 Fstat = 18486.6971

$a = 1.3026633$ $b = 8626.56$ $c = -6523681.2$ $d = 30629216$

334
16 mm Fiber Length

Eqn 2039 \( y = a + bx + cx^2 + dx^{2.5} \)

\( r^2 = 0.99996084 \)  DF Adj \( r^2 = 0.999956489 \)  FitStdErr = 0.151852724  Fstat = 314934.779

\( a = 0.69809509 \) \( b = 10863.554 \) \( c = -567983.47 \) \( d = 2033606.8 \)

24 mm Fiber Length

Eqn 2071 \( y = a + bx + cx^{2.5} + dx^3 \)

\( r^2 = 0.999926306 \)  DF Adj \( r^2 = 0.999918548 \)  FitStdErr = 0.234557474  Fstat = 176391.12

\( a = 0.33313942 \) \( b = 10892.662 \) \( c = -5418454.7 \) \( d = 24495788 \)
32 mm Fiber Length
Eqn 2039 \( y = a + bx + cx^2 + dx^{2.5} \)
\( r^2 = 0.999940605 \) DF Adj \( r^2 = 0.99993544 \)
FitStdErr=0.233151875 Fstat=263754.973
\( a = 0.75181198 \) b=12147.288
c=-633310.76 d=2283722.4

48 mm Fiber Length
Eqn 4341 \( y = a + bx + cx^2 + dx^{2.5} + ex^3 \)
\( r^2 = 0.999927889 \) DF Adj \( r^2 = 0.999918401 \)
FitStdErr=0.299343364 Fstat=135198.938
\( a = -0.9481139 \) b=15445.11 c=-2478366.1
d=32184530 e=-1.3046045e+08
## Flexural Strengthening Beams - Control

**Flexural Analysis**
Rebar Yielding Failure Assumed

<table>
<thead>
<tr>
<th></th>
<th>Strains</th>
<th>Stresses</th>
<th>Forces</th>
</tr>
</thead>
<tbody>
<tr>
<td>Comp</td>
<td>Concrete</td>
<td>0.0040</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Tension Steel</td>
<td>0.0087</td>
<td>500.000 MPa</td>
</tr>
</tbody>
</table>

\[
\text{SUM} = 0.0000 \text{ kN} \quad \text{SE} = 0.0000 \text{ kN} \quad \text{SE} = 0.0000 \text{ kN}
\]

(Equilibrium = 0)

Neutral Axis \(c\) = 30.12343 mm

**Stresses**

- \(\beta_1\) = 0.836
- \(\alpha_1\) = 0.769
- Area of Steel (As) = 200 mm²
- \(\alpha\) = 25.2 mm
- Steel Strength (fy) = 500 MPa

**Beam Dimensions**

- \(b\) = 96 mm
- \(d\) = 96 mm
- \(h\) = 125 mm
- Concrete Strength (f’c) = 53.8 MPa

**Shear Analysis**

- \(\theta\) = 41
- \(\beta\) = 0.162
- \(\alpha_s\) = 90
- \(s\) = 75 mm
- \(dv\) = 86.4 mm

- Area of Stirrups (Av) = 50 mm²
- Concrete Strength (f’c) = 53.8 MPa

\[
\begin{align*}
V_{cg} & = 12.8 \text{ kN} \\
V_{sg} & = 33.1 \text{ kN} \\
V_{fg} & = 0.0 \text{ kN} \\
V_{rg} & = 45.9 \text{ kN} \\
P_{ult} & = 91.9 \text{ kN}
\end{align*}
\]

Legend: | Formula | Entered Value |
Flexural Strengthening Beams - Scheme E Retrofit

Flexural Analysis
Concrete Crushing Failure Assumed

<table>
<thead>
<tr>
<th>Comp</th>
<th>Concrete</th>
<th>Strains</th>
<th>Stresses</th>
<th>Forces</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tension Steel</td>
<td>0.0041</td>
<td>500.000 MPa</td>
<td>100.000 kN</td>
<td></td>
</tr>
<tr>
<td>Tension Bot frp</td>
<td>0.0069</td>
<td>63.547 MPa</td>
<td>56.938 kN</td>
<td></td>
</tr>
</tbody>
</table>

Strains
- Concrete: 0.0040
- Tension Steel: 0.0041
- Bot frp: 0.0069

Stresses
- Concrete: 0.0040
- Tension Steel: 500.000 MPa
- Bot frp: 63.547 MPa

Forces
- 156.938 kN
- 100.000 kN
- 56.938 kN

Neutral Axis: 47.2751 mm

Mr: 13.85 kN\cdot m
P_max: 92.30 kN

Beam Dimensions
- b: 96 mm
- d: 96 mm
- h: 125 mm
- d_{bf}: 129.0 mm

Area of Steel (A_s): 200 mm²
Steel Strength (f_y): 500 MPa
Steel Modulus (E_s): 200 GPa
Concrete Strength (f_c): 53.8 MPa
FRP Tensile Strength (f’f): 104 MPa
Thickness frp (t_f): 8 mm
Area of Bottom frp (A_{fb}): 896 mm²
FRP Stress-Strain Relationship Constants
- a: 0.751812
- b: 12147.29
- c: -633311
- d: 228372
- e: 0

Shear Analysis

theta: 41
\beta: 0.162
alphas: 90
s: 75 mm
dv: 86.4 mm

Area of Stirrups (A_v): 50 mm²
Concrete Strength (f’c): 53.8 MPa

V_{cg}: 12.8 kN
V_{sg}: 33.1 kN
V_{fg}: 0.0 kN
V_{rg}: 45.9 kN
P_{ult}: 91.9 kN

Legend:
- Formula
- Entered Value
Flexural Strengthening Beams - Scheme C Retrofit

Flexural Analysis
Compression FRP Not Included - Concrete Crushing Failure Assumed

<table>
<thead>
<tr>
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<th>Stresses</th>
<th>Forces</th>
</tr>
</thead>
<tbody>
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<tr>
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<tr>
<td>Tension</td>
<td>Steel</td>
<td>0.0066</td>
<td>500.000 MPa</td>
</tr>
<tr>
<td></td>
<td>Bot frp</td>
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<td>94.962 MPa</td>
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<td>243.474 kN</td>
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<td>58.387 kN</td>
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<td>100.000 kN</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>85.086 kN</td>
</tr>
<tr>
<td>SUM</td>
<td></td>
<td></td>
<td>0.0000 kN</td>
</tr>
</tbody>
</table>

(Equil = 0)

Neutral Axis: 61.51134 mm

$\beta_1 = 0.797$

$\alpha_1 = 0.746$

Area of Steel (As) = 200 mm$^2$

Steel Strength (fy) = 500 MPa

Steel Modulus (Es) = 200 GPa

Concrete Strength (f'c) = 69.4 MPa

FRP Tensile Strength (f'tf) = 104 MPa

Beam Dimensions:

$b = 96$ mm

$d = 96$ mm

$h = 125$ mm

$db = 129.0$ mm

Thickness frp (tf) = 8 mm

Area of Side frp T (Afsb) = 1.016 mm$^2$

Area of Bottom frp (Afb) = 0.896 mm$^2$

Shear Analysis

$\theta = 41$

$\beta = 0.162$

$\alpha_\theta = 90$

$\alpha_{\beta} = 45$

$s = 75$ mm

$dv = 86.4$ mm

Area of Stirrups (Av) = 50 mm$^2$

Concrete Strength (f'c) = 53.8 MPa

Vcg = 12.8 kN

Vsg = 33.1 kN

Vfg = 316.3 kN

Vrg = 362.2 kN

Vult = 724.4 kN

Legend:

Formula

Entered Value

339
### Flexural Strengthening Beams - Scheme C Retrofit

#### Flexural Analysis
Compression FRP Included - Concrete Crushing Failure Assumed

<table>
<thead>
<tr>
<th>Comp</th>
<th>Strains</th>
<th>Stresses</th>
<th>Forces</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete</td>
<td>0.0117</td>
<td>61.425 MPa</td>
<td>216.981 kN</td>
</tr>
<tr>
<td>Side frp</td>
<td>0.0059</td>
<td>67.291 MPa</td>
<td>53.875 kN</td>
</tr>
<tr>
<td>Side frp</td>
<td>0.0075</td>
<td>500.000 MPa</td>
<td>75.562 kN</td>
</tr>
<tr>
<td>Tension</td>
<td>0.0088</td>
<td>106.355 MPa</td>
<td>100.000 kN</td>
</tr>
<tr>
<td>Bot frp</td>
<td>0.0158</td>
<td></td>
<td>95.294 kN</td>
</tr>
</tbody>
</table>

**SUM**

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>(Equil = 0)</td>
<td>0.0000 kN</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Strains</th>
<th>Stresses</th>
<th>Forces</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0117</td>
<td>61.425 MPa</td>
<td>216.981 kN</td>
</tr>
<tr>
<td>0.0059</td>
<td>67.291 MPa</td>
<td>53.875 kN</td>
</tr>
<tr>
<td>0.0075</td>
<td>500.000 MPa</td>
<td>75.562 kN</td>
</tr>
<tr>
<td>0.0088</td>
<td>106.355 MPa</td>
<td>100.000 kN</td>
</tr>
</tbody>
</table>

**Neutral Axis**

<table>
<thead>
<tr>
<th>c</th>
</tr>
</thead>
<tbody>
<tr>
<td>54.81813 mm</td>
</tr>
</tbody>
</table>

**Mr**

| 22.47 kN•m |

**Pmax**

| 149.8 kN   |

<table>
<thead>
<tr>
<th>β1</th>
<th>0.797</th>
</tr>
</thead>
<tbody>
<tr>
<td>alpha1</td>
<td>0.746</td>
</tr>
<tr>
<td>a</td>
<td>43.7 mm</td>
</tr>
</tbody>
</table>

| Area of Steel (As) | 200 mm² |
| Steel Strength (fy) | 500 MPa |

<table>
<thead>
<tr>
<th>Beam Dimensions</th>
<th>Steel Modulus (Es)</th>
<th>200 GPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>b</td>
<td>96 mm</td>
<td>69.4 MPa</td>
</tr>
<tr>
<td>d</td>
<td>96 mm</td>
<td></td>
</tr>
<tr>
<td>h</td>
<td>125 mm</td>
<td></td>
</tr>
<tr>
<td>dfb</td>
<td>129.0 mm</td>
<td></td>
</tr>
<tr>
<td></td>
<td>FRP Tensile Strength (f’f)</td>
<td>104 MPa</td>
</tr>
<tr>
<td></td>
<td>FRP Modulus (Ef)</td>
<td>10.5 GPa</td>
</tr>
</tbody>
</table>

| Thickness frp (tf) | 8 mm |
| Area of Side frp T (Afsb) | 1123 mm² |
| Area of Bottom frp (Afb) | 896 mm² |
| Area of Side frp C (Afst) | 877 mm² |

**FRP Stress-Strain Relationship Constants**

| a | 0.751812 |
| b | 12147.29 |
| c | -633311  |
| d | 2283722  |
| e | 0        |

#### Shear Analysis

<table>
<thead>
<tr>
<th>theta</th>
<th>41</th>
</tr>
</thead>
<tbody>
<tr>
<td>β</td>
<td>0.162</td>
</tr>
<tr>
<td>alphas</td>
<td>90</td>
</tr>
<tr>
<td>alphaf</td>
<td>45</td>
</tr>
<tr>
<td>s</td>
<td>75 mm</td>
</tr>
<tr>
<td>dv</td>
<td>86.4 mm</td>
</tr>
</tbody>
</table>

| Area of Stirrups (Av) | 50 mm² |
| Concrete Strength (f’c) | 53.8 MPa |

| Vcg | 12.8 kN |
| Vsg | 33.1 kN  |
| Vfg | 316.3 kN |

| Vrg | 362.2 kN |
| Pult | 724.4 kN |

**Legend:**

<table>
<thead>
<tr>
<th>Formula</th>
<th>Entered Value</th>
</tr>
</thead>
</table>

340
## Flexural Strengthening Beams - Scheme C Retrofit

### Flexural Analysis
Compression FRP Included - GFRP Tension Failure Assumed

<table>
<thead>
<tr>
<th>Comp</th>
<th>Concrete</th>
<th>Side frp</th>
<th>Side frp</th>
<th>Tension</th>
<th>Steel</th>
<th>Bot frp</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strains</td>
<td>0.0105</td>
<td>0.0053</td>
<td>0.0068</td>
<td>0.0080</td>
<td>0.0152</td>
<td></td>
</tr>
<tr>
<td>Stresses</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Forces</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>215.802 kN</td>
<td></td>
<td>48.252 kN</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>70.870 kN</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>100.000 kN</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>93.184 kN</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SUM</td>
<td>0.0000 kN</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

(Equil = 0)

**Neutral Axis**

- $c = 54.5202$ mm

**Beam Dimensions**

- $b = 96$ mm
- $d = 96$ mm
- $h = 125$ mm
- $dfb = 129.0$ mm

**Area of Steel (As)**

- 200 mm$^2$

**Steel Strength (fy)**

- 500 MPa

**Steel Modulus (Es)**

- 200 GPa

**Concrete Strength (fc)**

- 69.4 MPa

**FRP Tensile Strength (f'f)**

- 104 MPa

**FRP Modulus (Ef)**

- 10.5 GPa

**FRP Stress-Strain Relationship Constants**

- $a = 0.751812$
- $b = 12147.29$
- $c = -633311$
- $d = 2283722$
- $e = 0$

### Shear Analysis

- $\theta = 41$
- $\beta = 0.162$
- $\alpha_s = 90$
- $\alpha_h = 45$
- $s = 75$ mm
- $dv = 86.4$ mm

**Vcg**

- 12.8 kN

**Vsg**

- 33.1 kN

**Vfg**

- 316.3 kN

**Vrg**

- 362.2 kN

**Pult**

- 724.4 kN

**Area of Stirrups (Av)**

- 50 mm$^2$

**Concrete Strength (f'c)**

- 53.8 MPa

**Legend:**

- Formula
- Entered Value
Flexural Strengthening Beams - Scheme D Retrofit

Flexural Analysis
Compression FRP Not Included - Concrete Crushing Failure Assumed

<table>
<thead>
<tr>
<th></th>
<th>Strains</th>
<th>Stresses</th>
<th>Forces</th>
</tr>
</thead>
<tbody>
<tr>
<td>Comp</td>
<td>Concrete</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tension</td>
<td>Side frp</td>
<td>0.0069</td>
<td>63.272 MPa</td>
</tr>
<tr>
<td></td>
<td>Steel</td>
<td>0.0082</td>
<td>500.000 MPa</td>
</tr>
<tr>
<td></td>
<td>Bot frp</td>
<td>0.0145</td>
<td>101.591 MPa</td>
</tr>
<tr>
<td>SUM</td>
<td>c</td>
<td>0.0101</td>
<td></td>
</tr>
</tbody>
</table>

(Equil = 0)

Neutral Axis 52.94105 mm

Mr 22.93 kN\cdot m
Pmax 152.9 kN

Area of Steel (As)
200 mm²

Steel Strength (fy)
500 MPa

Steel Modulus (Es)
200 GPa

Concrete Strength (f’c)
107.6 MPa

FRP Tensile Strength (f’f)
104 MPa

FRP Stress-Strain Relationship Constants
a 0.751812
b 12147.29
c -633311
d 2283722
e 0

Shear Analysis

theta 41

Vcg 12.8 kN

Vsg 33.1 kN

Vfg 316.3 kN

Vrg 362.2 kN

Pult 724.4 kN

Area of Stirrups (Av)
50 mm²

Concrete Strength (f’c)
53.8 MPa

Legend:

Formula
Entered Value

342
## Flexural Strengthening Beams - Scheme D Retrofit

### Flexural Analysis

**Compression FRP Included - Concrete Crushing Failure Assumed**

<table>
<thead>
<tr>
<th>Strains</th>
<th>Stresses</th>
<th>Forces</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top frp</td>
<td>0.0111</td>
<td>116.841 MPa</td>
</tr>
<tr>
<td>Concrete</td>
<td>0.0101</td>
<td>53.025 MPa</td>
</tr>
<tr>
<td>Side frp</td>
<td>0.0051</td>
<td>86.761 MPa</td>
</tr>
<tr>
<td>Side frp</td>
<td>0.0110</td>
<td>500.000 MPa</td>
</tr>
<tr>
<td>Steel</td>
<td>0.0146</td>
<td>128.461 MPa</td>
</tr>
<tr>
<td>Bot frp</td>
<td>0.0230</td>
<td>0.0000 MPa</td>
</tr>
</tbody>
</table>

**Neutral Axis**

| c | 39.31095 mm |

**Stresses**

- **Top frp**: 116.841 MPa
- **Concrete**: 53.025 MPa
- **Side frp**: 86.761 MPa
- **Steel**: 500.000 MPa
- **Bot frp**: 128.461 MPa

**SUM (Equil = 0)**

<table>
<thead>
<tr>
<th>Forces</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0000 kN</td>
</tr>
</tbody>
</table>

**Beam Dimensions**

- **b**: 96 mm
- **d**: 96 mm
- **h**: 125 mm
- **df**: 128.0 mm

**Area of Steel (As)**

| Area  | 200 mm² |

**Steel Strength (fy)**

| Steel Strength | 500 MPa |

**Steel Modulus (Es)**

| Steel Modulus | 200 GPa |

**Concrete Strength (f’c)**

| Concrete Strength | 107.6 MPa |

**FRP Tensile Strength (f’f)**

| FRP Tensile Strength | 104 MPa |

**FRP Modulus (Ef)**

| FRP Modulus | 10.5 GPa |

**Thickness frp (tf)**

| Thickness frp | 8 mm |

**Area of Side frp T (Afsb)**

| Area of Side frp | 1371 mm² |

**Area of Bottom frp (Afb)**

| Area of Bottom frp | 896 mm² |

**Area of Top frp (Aft)**

| Area of Top frp | 896 mm² |

**Area of Side frp C (Afst)**

| Area of Side frp | 629 mm² |

**FRP Stress-Strain Relationship Constants**

| a       | 0.751812 |
| b       | 12147.29 |
| c       | -633311  |
| d       | 2283722  |
| e       | 0        |

### Shear Analysis

| theta  | 41 |
| s      | 75 mm |
| dv     | 86.4 mm |

**Area of Stirrups (Av)**

| Area of Stirrups | 50 mm² |

**Concrete Strength (f’c)**

| Concrete Strength | 53.8 MPa |

**Formula**

- **Vcg**: 12.8 kN
- **Vsg**: 33.1 kN
- **Vfg**: 316.3 kN
- **Vrg**: 362.2 kN
- **Pult**: 724.4 kN

**Legend:**

| Formula | Entered Value |
Flexural Strengthening Beams - Scheme D Retrofit

Flexural Analysis
Compression FRP Included - GFRP Tension Failure Assumed

<table>
<thead>
<tr>
<th>Strains</th>
<th>Stresses</th>
<th>Forces</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top frp</td>
<td>0.0069</td>
<td>72.974 MPa</td>
</tr>
<tr>
<td>Comp</td>
<td>0.0063</td>
<td>33.097 MPa</td>
</tr>
<tr>
<td>Concrete</td>
<td>0.0063</td>
<td></td>
</tr>
<tr>
<td>Side frp</td>
<td>0.0032</td>
<td>63.698 MPa</td>
</tr>
<tr>
<td>Side frp</td>
<td>0.0069</td>
<td>500.000 MPa</td>
</tr>
<tr>
<td>Tension</td>
<td>0.0092</td>
<td>104.000 MPa</td>
</tr>
<tr>
<td>Bot frp</td>
<td>0.0152</td>
<td></td>
</tr>
</tbody>
</table>

Neutral Axis 39.05118 mm

\[ \beta_1 = 0.701 \]
\[ \alpha_1 = 0.689 \]
\[ a = 27.4 \text{ mm} \]
\[ b = 96 \text{ mm} \]
\[ d = 96 \text{ mm} \]
\[ h = 125 \text{ mm} \]
\[ d_{bf} = 129.0 \text{ mm} \]

Area of Steel (As) 200 mm²
Steel Strength (fy) 500 MPa
Steel Modulus (Es) 200 GPa
Concrete Strength (f’c) 107.6 MPa
FRP Tensile Strength (f’f) 104 MPa
FRP Modulus (Ef) 10.5 GPa

Thickness frp (t) 8 mm
Area of Side frp T (Afsb) 1375 mm²
Area of Bottom frp (Afb) 896 mm²
Area of Top frp (Aft) 896 mm²
Area of Side frp C (Afst) 625 mm²

Shear Analysis

\[ \theta = 41 \]
\[ \beta = 0.162 \]
\[ \alpha = 90 \]
\[ \alpha_f = 45 \]
\[ s = 75 \text{ mm} \]
\[ d_v = 86.4 \text{ mm} \]

Area of Stirrups (Av) 50 mm²
Concrete Strength (f’c) 53.8 MPa

\[ V_{cg} = 12.8 \text{ kN} \]
\[ V_{fg} = 33.1 \text{ kN} \]
\[ V_{rg} = 362.2 \text{ kN} \]
\[ P_{ult} = 724.4 \text{ kN} \]

Legend:

<table>
<thead>
<tr>
<th>Formula</th>
<th>Entered Value</th>
</tr>
</thead>
</table>

344
# Shear Strengthening Beams - Control

## Flexural Analysis

Rebar Yielding Failure Assumed

<table>
<thead>
<tr>
<th>Comp</th>
<th>Concrete Stains</th>
<th>0.0040</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel</td>
<td>Stresses</td>
<td>500.000 MPa</td>
</tr>
</tbody>
</table>

- **Forces**
  - 200.000 kN

- **SUM**
  - 0.0000 kN

**Neutral Axis**
- 59.67933 mm

| \( \beta_1 \) | 0.834 |
| \( \alpha_1 \) | 0.768 |
| \( a \) | 49.8 mm |
| Beam Dimensions |  |
| \( b \) | 96 mm |
| \( d \) | 100 mm |
| \( h \) | 125 mm |

**Stresses**
- Area of Steel (\( A_s \))
  - 400 mm²

**Forces**
- \( P_{max} = 100.2 \) kN

**Shear Analysis**

| Theta | 32 |
| Beta | 0.263 |
| Alphas | 90 |
| \( s \) | 75 mm |
| \( d_v \) | 90 mm |

- Area of Stirrups (\( A_v \))
  - 0 mm²

- Concrete Strength (\( f'_c \))
  - 54.5 MPa

| Vcg | 21.8 kN |
| Vsg | 0.0 kN |
| Vfg | 0.0 kN |
| Vrg | 21.8 kN |
| \( P_{ult} \) | 43.6 kN |

**Legend:**
- Formula
- Entered Value

---

345
Shear Strengthening Beams - Scheme A Retrofit

**Flexural Analysis**
Compression FRP Not Included - Concrete Crushing Failure Assumed

<table>
<thead>
<tr>
<th></th>
<th>Strains</th>
<th>Stresses</th>
<th>Forces</th>
</tr>
</thead>
<tbody>
<tr>
<td>Comp Concrete</td>
<td>0.0040</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tension Side frp</td>
<td>0.0020</td>
<td>24.488 MPa</td>
<td></td>
</tr>
<tr>
<td>Tension Steel</td>
<td>0.0024</td>
<td>500.000 MPa</td>
<td>10.652 kN</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>200.000 kN</td>
</tr>
<tr>
<td>SUM (Equil = 0)</td>
<td></td>
<td></td>
<td>0.0000 kN</td>
</tr>
</tbody>
</table>

Neutral Axis 62.85786 mm

<p>| | | | |</p>
<table>
<thead>
<tr>
<th></th>
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<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>$\beta_1$</td>
<td>0.834</td>
<td></td>
<td></td>
</tr>
<tr>
<td>alpha1</td>
<td>0.768</td>
<td></td>
<td></td>
</tr>
<tr>
<td>a</td>
<td>52.4 mm</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Beam Dimensions</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>b</td>
<td>96 mm</td>
<td></td>
<td></td>
</tr>
<tr>
<td>d</td>
<td>100 mm</td>
<td></td>
<td></td>
</tr>
<tr>
<td>h</td>
<td>125 mm</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mr</td>
<td>15.48 kN.m</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Pmax</td>
<td>103.2 kN</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

$\alpha$ 0.834

<p>| | | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Area of Steel (As)</td>
<td>400 mm$^2$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Steel Strength (fy)</td>
<td>500 MPa</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Steel Modulus (Es)</td>
<td>200 GPa</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Concrete Strength (f'c)</td>
<td>54.5 MPa</td>
<td></td>
<td></td>
</tr>
<tr>
<td>FRP Tensile Strength (f'f)</td>
<td>108 MPa</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Thickness frp (tf)</td>
<td>3.5 mm</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Area of Side frp (Afsb)</td>
<td>435 mm$^2$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>FRP Stress-Strain Relationship Constants</td>
<td>a -0.9481139</td>
<td>b 15445.11</td>
<td>c -2478366.1</td>
</tr>
</tbody>
</table>

**Shear Analysis**

<p>| | | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>theta</td>
<td>32</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\beta$</td>
<td>0.263</td>
<td></td>
<td></td>
</tr>
<tr>
<td>alpha</td>
<td>90</td>
<td></td>
<td></td>
</tr>
<tr>
<td>alphaf</td>
<td>45</td>
<td></td>
<td></td>
</tr>
<tr>
<td>s</td>
<td>75 mm</td>
<td></td>
<td></td>
</tr>
<tr>
<td>dv</td>
<td>90 mm</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Area of Stirrups (Av)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Concrete Strength (f'c)</td>
<td>54.5 MPa</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Vcg</td>
<td>21.8 kN</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Vsg</td>
<td>0.0 kN</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Vfg</td>
<td>173.8 kN</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Vrg</td>
<td>195.6 kN</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Pult</td>
<td>391.1 kN</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Legend: | Formula | Entered Value |
Shear Strengthening Beams - Scheme A Retrofit

### Flexural Analysis

Compression FRP Included - Concrete Crushing Failure Assumed

<table>
<thead>
<tr>
<th>Strains</th>
<th>Stresses</th>
<th>Forces</th>
</tr>
</thead>
<tbody>
<tr>
<td>Comp Concrete</td>
<td>0.0040</td>
<td>23.600 MPa</td>
</tr>
<tr>
<td>Side frp</td>
<td>0.0020</td>
<td>26.379 MPa</td>
</tr>
<tr>
<td>Tension Side frp</td>
<td>0.0021</td>
<td>500.000 MPa</td>
</tr>
<tr>
<td>Steel</td>
<td>0.0026</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Neutral Axis</th>
<th>c</th>
</tr>
</thead>
<tbody>
<tr>
<td>60.27445 mm</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Strain</th>
<th>Force</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0040</td>
<td>201.994 kN</td>
</tr>
<tr>
<td>0.0020</td>
<td>11.952 kN</td>
</tr>
<tr>
<td>0.0021</td>
<td>200.000 kN</td>
</tr>
<tr>
<td>0.0026</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>β1</th>
<th>0.834</th>
</tr>
</thead>
<tbody>
<tr>
<td>alpha1</td>
<td>0.768</td>
</tr>
<tr>
<td>a</td>
<td>50.3 mm</td>
</tr>
<tr>
<td>b</td>
<td>96 mm</td>
</tr>
<tr>
<td>d</td>
<td>100 mm</td>
</tr>
<tr>
<td>h</td>
<td>125 mm</td>
</tr>
</tbody>
</table>

| Area of Steel (As) | 400 mm² |
| Steel Strength (fy) | 500 MPa |
| Steel Modulus (Es) | 200 GPa |
| Concrete Strength (f’c) | 54.5 MPa |

<table>
<thead>
<tr>
<th>FRP Stress-Strain Relationship Constants</th>
</tr>
</thead>
<tbody>
<tr>
<td>a</td>
</tr>
<tr>
<td>b</td>
</tr>
<tr>
<td>c</td>
</tr>
<tr>
<td>d</td>
</tr>
<tr>
<td>e</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Thickness frp (tf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.5 mm</td>
</tr>
</tbody>
</table>

### Shear Analysis

| theta | 32 |
| β | 0.263 |
| alphas | 90 |
| alphaf | 45 |
| s | 75 mm |
| dv | 90 mm |

| Vcg | 21.8 kN |
| Vsg | 0.0 kN |
| Vfg | 173.8 kN |
| Vrg | 195.6 kN |
| Pult | 391.1 kN |

Legend:
- Formula
- Entered Value

347
Shear Strengthening Beams - Scheme C Retrofit

**Flexural Analysis**

Compression FRP Not Included - Concrete Crushing Failure Assumed

<table>
<thead>
<tr>
<th>Comp</th>
<th>Concrete</th>
<th>Strains</th>
<th>Stresses</th>
<th>Forces</th>
</tr>
</thead>
<tbody>
<tr>
<td>Side frp</td>
<td>0.0034</td>
<td>0.0071</td>
<td>39.465 MPa</td>
<td>16.889 kN</td>
</tr>
<tr>
<td>Tension</td>
<td>Steel</td>
<td>0.0040</td>
<td>500.000 MPa</td>
<td>200.000 kN</td>
</tr>
<tr>
<td></td>
<td>Bot frp</td>
<td>0.0070</td>
<td>72.847 MPa</td>
<td>26.261 kN</td>
</tr>
</tbody>
</table>

SUM (Equil = 0)

<table>
<thead>
<tr>
<th>Neutral Axis</th>
<th>63.86476 mm</th>
<th>Mr</th>
<th>18.66 kN·m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pmax</td>
<td>124.4 kN</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

\[ \beta_1 = 0.807 \]
\[ \alpha_1 = 0.752 \]
\[ a = 51.5 \text{ mm} \]

**Beam Dimensions**

\[ b = 96 \text{ mm} \]
\[ d = 100 \text{ mm} \]
\[ h = 125 \text{ mm} \]
\[ dfb = 126.8 \text{ mm} \]

**Area of Steel (As)**

\[ 400 \text{ mm}^2 \]

**Steel Strength (fy)**

\[ 500 \text{ MPa} \]

**Steel Modulus (Es)**

\[ 200 \text{ GPa} \]

**Concrete Strength (f'c)**

\[ 65.4 \text{ MPa} \]

**FRP Tensile Strength (f'f)**

\[ 108 \text{ MPa} \]

**Area of Side frp (T)**

\[ 428 \text{ mm}^2 \]

**Area of Bottom frp (F)**

\[ 361 \text{ mm}^2 \]

**Thickness frp (tf)**

\[ 3.5 \text{ mm} \]

**FRP Stress-Strain Relationship Constants**

\[ a = -0.9481139 \]
\[ b = 15445.11 \]
\[ c = 2476366.1 \]
\[ d = 32184530 \]
\[ e = -1.30E+08 \]

**Shear Analysis**

\[ \theta = 32 \]
\[ \beta = 0.263 \]
\[ \alpha = 90 \]
\[ \alpha_f = 45 \]
\[ s = 75 \text{ mm} \]
\[ dv = 90 \text{ mm} \]

**Area of Stirrups (Av)**

\[ 0 \text{ mm}^2 \]

**Concrete Strength (f'c)**

\[ 54.5 \text{ MPa} \]

Legend:

<table>
<thead>
<tr>
<th>Formula</th>
<th>Entered Value</th>
</tr>
</thead>
</table>

348
Shear Strengthening Beams - Scheme C Retrofit

Flexural Analysis
Compression FRP Included - Concrete Crushing Failure Assumed

<table>
<thead>
<tr>
<th>Strains</th>
<th>Stresses</th>
<th>Forces</th>
</tr>
</thead>
<tbody>
<tr>
<td>Comp Concrete 0.0071</td>
<td>41.890 MPa</td>
<td>230.438 kN</td>
</tr>
<tr>
<td>Side frp 0.0036</td>
<td>43.265 MPa</td>
<td>17.748 kN</td>
</tr>
<tr>
<td>Tension Side frp 0.0038</td>
<td>43.265 MPa</td>
<td>19.526 kN</td>
</tr>
<tr>
<td>Steel 0.0046</td>
<td>500.000 MPa</td>
<td>200.000 kN</td>
</tr>
<tr>
<td>Bot frp 0.0078</td>
<td>79.500 MPa</td>
<td>28.660 kN</td>
</tr>
</tbody>
</table>

SUM (Equil = 0) 0.0000 kN

Neutral Axis 60.52584 mm

Mr 19.28 kN•m
Pmax 128.6 kN

\[ \beta_1 = 0.807 \]
\[ \alpha_1 = 0.752 \]
\[ a = 48.8 \text{ mm} \]

Beam Dimensions
\[ b = 96 \text{ mm} \]
\[ d = 100 \text{ mm} \]
\[ h = 125 \text{ mm} \]
\[ dfb = 126.8 \text{ mm} \]

Area of Steel (As)
\[ 400 \text{ mm}^2 \]

Steel Strength (fy)
\[ 500 \text{ MPa} \]

Steel Modulus (Es)
\[ 200 \text{ GPa} \]

Concrete Strength (fc)
\[ 65.4 \text{ MPa} \]

\[ \text{Area of Side frp T (Afsb)} = 451 \text{ mm}^2 \]
\[ \text{Area of Bottom frp (Afb)} = 361 \text{ mm}^2 \]
\[ \text{Area of Side frp C (Afst)} = 424 \text{ mm}^2 \]

FRP Tensile Strength (f')
\[ 108 \text{ MPa} \]

FRP Modulus (Efrp)
\[ 11.8 \text{ GPa} \]

FRP Stress-Strain Relationship Constants
\[ a = -0.9481139 \]
\[ b = 15445.11 \]
\[ c = -2478366.1 \]
\[ d = 32184530 \]
\[ e = -1.30E+08 \]

Shear Analysis

\[ \theta = 32 \]
\[ \beta = 0.263 \]
\[ \alpha = 90 \]
\[ \alpha_{hf} = 45 \]
\[ s = 75 \text{ mm} \]
\[ dv = 90 \text{ mm} \]

Area of Stirrups (Av)
\[ 0 \text{ mm}^2 \]

Concrete Strength (fc)
\[ 54.5 \text{ MPa} \]

Vcg 21.8 kN
Vsg 0.0 kN
Vfg 173.8 kN
Vrg 195.6 kN

Pult 391.1 kN

Legend:

<table>
<thead>
<tr>
<th>Formula</th>
<th>Entered Value</th>
</tr>
</thead>
</table>
Shear Strengthening Beams - Scheme D Retrofit

**Flexural Analysis**
Compression FRP Not Included - Concrete Crushing Failure Assumed

<table>
<thead>
<tr>
<th>Comp</th>
<th>Concrete</th>
<th>Strains</th>
<th>Stresses</th>
<th>Forces</th>
</tr>
</thead>
<tbody>
<tr>
<td>Side frp</td>
<td>0.0048</td>
<td>0.0083</td>
<td>52.699 MPa</td>
<td>24.645 kN</td>
</tr>
<tr>
<td>Tension Steel</td>
<td>0.0060</td>
<td>500.000 MPa</td>
<td>34.405 kN</td>
<td></td>
</tr>
<tr>
<td>Bot frp</td>
<td>0.0088</td>
<td>95.438 MPa</td>
<td>200.000 kN</td>
<td></td>
</tr>
</tbody>
</table>

SUM (Equil = 0)
0.0000 kN

Neutral Axis 58.19268 mm

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>b</td>
<td>96 mm</td>
</tr>
<tr>
<td>d</td>
<td>100 mm</td>
</tr>
<tr>
<td>h</td>
<td>125 mm</td>
</tr>
<tr>
<td>dfb</td>
<td>126.8 mm</td>
</tr>
<tr>
<td>Area of Steel (As)</td>
<td>400 mm²</td>
</tr>
<tr>
<td>Steel Strength (fy)</td>
<td>500 MPa</td>
</tr>
<tr>
<td>Steel Modulus (Es)</td>
<td>200 GPa</td>
</tr>
<tr>
<td>Concrete Strength (f’c)</td>
<td>84.5 MPa</td>
</tr>
<tr>
<td>FRP Tensile Strength (f’f)</td>
<td>108 MPa</td>
</tr>
</tbody>
</table>

**Shear Analysis**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>theta</td>
<td>32</td>
</tr>
<tr>
<td>b</td>
<td>0.263</td>
</tr>
<tr>
<td>alphas</td>
<td>90</td>
</tr>
<tr>
<td>alphaf</td>
<td>45</td>
</tr>
<tr>
<td>s</td>
<td>75 mm</td>
</tr>
<tr>
<td>dv</td>
<td>90 mm</td>
</tr>
<tr>
<td>Area of Stirrups (Av)</td>
<td>0 mm²</td>
</tr>
<tr>
<td>Concrete Strength (f’c)</td>
<td>54.5 MPa</td>
</tr>
</tbody>
</table>

Legend: Formula Entered Value

Pult 391.1 kN
Shear Strengthening Beams - Scheme D Retrofit

Flexural Analysis
Compression FRP Included - Concrete Crushing Failure Assumed

<table>
<thead>
<tr>
<th>Strains</th>
<th>Stresses</th>
<th>Forces</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top frp</td>
<td>0.0086</td>
<td>101.375 MPa</td>
</tr>
<tr>
<td>Comp Concrete</td>
<td>0.0083</td>
<td>48.970 MPa</td>
</tr>
<tr>
<td>Side frp</td>
<td>0.0042</td>
<td>66.272 MPa</td>
</tr>
<tr>
<td>Side frp</td>
<td>0.0062</td>
<td>500.000 MPa</td>
</tr>
<tr>
<td>Tension Steel</td>
<td>0.0083</td>
<td>113.604 MPa</td>
</tr>
<tr>
<td>Bot frp</td>
<td>0.0128</td>
<td>113.604 MPa</td>
</tr>
<tr>
<td></td>
<td>SUM</td>
<td>0.0000 kN</td>
</tr>
</tbody>
</table>

Neutral Axis: c = 49.90161 mm

- \( \beta_1 = 0.759 \)
- \( \alpha_1 = 0.723 \)
- Area of Steel (As) = 400 mm²
- Steel Strength (fy) = 500 MPa
- Steel Modulus (Es) = 200 GPa
- Concrete Strength (f’c) = 84.5 MPa
- FRP Tensile Strength (f') = 108 MPa
- FRP Modulus (Ef) = 11.8 GPa
- Thickness frp (tf) = 3.5 mm
- Area of Side frp T (Afsb) = 526 mm²
- Area of Bottom frp (Afb) = 361 mm²
- Area of Top frp (Art) = 361 mm²
- Area of Side frp C (Afst) = 349 mm²

Shear Analysis

<table>
<thead>
<tr>
<th>theta</th>
<th>32</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \beta )</td>
<td>0.263</td>
</tr>
<tr>
<td>( \alpha_s )</td>
<td>90</td>
</tr>
<tr>
<td>( \alpha_f )</td>
<td>45</td>
</tr>
<tr>
<td>s</td>
<td>75 mm</td>
</tr>
<tr>
<td>dv</td>
<td>90 mm</td>
</tr>
<tr>
<td>Area of Stirrups (Av)</td>
<td>0 mm²</td>
</tr>
<tr>
<td>Concrete Strength (f’c)</td>
<td>54.5 MPa</td>
</tr>
<tr>
<td>Vcg</td>
<td>21.8 kN</td>
</tr>
<tr>
<td>Vsg</td>
<td>0.0 kN</td>
</tr>
<tr>
<td>Vfg</td>
<td>173.8 kN</td>
</tr>
<tr>
<td>Vrg</td>
<td>195.6 kN</td>
</tr>
<tr>
<td>Pult</td>
<td>391.1 kN</td>
</tr>
</tbody>
</table>

Legend:
- Formula
- Entered Value
Shear Strengthening Beams - Scheme D Retrofit

**Flexural Analysis**
Compression FRP Included - GFRP Tension Failure Assumed

<table>
<thead>
<tr>
<th>Strains</th>
<th>Stresses</th>
<th>Forces</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top frp</td>
<td>0.0116 136.845 MPa</td>
<td>49.333 kN</td>
</tr>
<tr>
<td>Comp</td>
<td>0.0112 66.029 MPa</td>
<td>214.912 kN</td>
</tr>
<tr>
<td>Side frp</td>
<td>0.0086 88.677 MPa</td>
<td>22.314 kN</td>
</tr>
<tr>
<td>Tension</td>
<td>0.0089 500.000 MPa</td>
<td>47.625 kN</td>
</tr>
<tr>
<td>Steel</td>
<td>0.0120</td>
<td>200.000 kN</td>
</tr>
<tr>
<td>Bot frp</td>
<td>0.0186 108.000 MPa</td>
<td>38.934 kN</td>
</tr>
</tbody>
</table>

**SUM**
(\text{Equil} = 0)

Neutral Axis: \(c = 48.27756 \text{ mm}\)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>(\beta_1)</td>
<td>0.759</td>
</tr>
<tr>
<td>alpha1</td>
<td>0.723</td>
</tr>
<tr>
<td>a</td>
<td>36.6 mm</td>
</tr>
<tr>
<td>(\text{Beam Dimensions})</td>
<td></td>
</tr>
<tr>
<td>b</td>
<td>96 mm</td>
</tr>
<tr>
<td>d</td>
<td>100 mm</td>
</tr>
<tr>
<td>h</td>
<td>125 mm</td>
</tr>
<tr>
<td>dfb</td>
<td>126.8 mm</td>
</tr>
<tr>
<td>(\text{Area of Steel (As)})</td>
<td>400 \text{ mm}^2</td>
</tr>
<tr>
<td>(\text{Steel Strength (fy)})</td>
<td>500 MPa</td>
</tr>
<tr>
<td>(\text{Steel Modulus (Es)})</td>
<td>200 GPa</td>
</tr>
<tr>
<td>(\text{Concrete Strength (f'c)})</td>
<td>84.5 MPa</td>
</tr>
<tr>
<td>(\text{FRP Tensile Strength (f'f)})</td>
<td>108 MPa</td>
</tr>
<tr>
<td>(\text{FRP Modulus (Ef)})</td>
<td>11.8 GPa</td>
</tr>
<tr>
<td>(\text{Thickness frp (tf)})</td>
<td>3.5 mm</td>
</tr>
<tr>
<td>(\text{Area of Side frp T (Afsb)})</td>
<td>537 mm^2</td>
</tr>
<tr>
<td>(\text{Area of Bottom frp (Afb)})</td>
<td>361 mm^2</td>
</tr>
<tr>
<td>(\text{Area of Top frp (Aft)})</td>
<td>361 mm^2</td>
</tr>
<tr>
<td>(\text{Area of Side frp C (Afst)})</td>
<td>338 mm^2</td>
</tr>
</tbody>
</table>

**Shear Analysis**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>(\theta)</td>
<td>32</td>
</tr>
<tr>
<td>(\beta)</td>
<td>0.263</td>
</tr>
<tr>
<td>alphas</td>
<td>90</td>
</tr>
<tr>
<td>alphaf</td>
<td>45</td>
</tr>
<tr>
<td>s</td>
<td>75 mm</td>
</tr>
<tr>
<td>dv</td>
<td>90 mm</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>(\text{Area of Stirrups (Av)})</td>
<td>0 \text{ mm}^2</td>
</tr>
<tr>
<td>(\text{Concrete Strength (f'c)})</td>
<td>54.5 MPa</td>
</tr>
</tbody>
</table>

\(\text{Vcg} = 21.8 \text{ kN}\)
\(\text{Vsg} = 0.0 \text{ kN}\)
\(\text{Vfg} = 173.8 \text{ kN}\)
\(\text{Vrg} = 195.6 \text{ kN}\)
\(\text{Pult} = 391.1 \text{ kN}\)

Legend:

<table>
<thead>
<tr>
<th>Formula</th>
<th>Entered Value</th>
</tr>
</thead>
</table>