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Flexural and Shear Strengthening of Concrete Structures with Prestressed GFRP Sheets

By

Syed Iftikhar Ahmad

A thesis submitted to the School of Graduate Studies and Research of the
University of Ottawa in partial fulfillment of the requirements
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ABSTRACT

Although Carbon Fibre Reinforced Polymer (CFRP) composite sheets have showed excellent results in laboratory for strengthening reinforced concrete structures, it could not get wide popularity in field application due to high material cost. To address this problem the present study was carried out to explore the possibility of using Glass FRP (GFRP) composite sheets which is relatively cheap material. GFRP has relatively low modulus of elasticity and therefore disregarded for strengthening in the past.

Active strengthening technique i.e. prestressing was used for maximum utilization of GFRP strength. Prestressing has not only helped in achieving this goal but also improved the behaviour of strengthened member under service load and delayed premature delamination. Long term losses associated with prestressing were also studied and it was found in acceptable range. The research work comprised of both experimental and analytical investigations.

The experimental program consisted of testing twenty three (eight flexure and fifteen shear), rectangular section, simply supported reinforced concrete beams. The variables in flexure beams were levels of prestressing, number of GFRP layers, and conventional reinforcement ratio. In shear beams variables were shear span to effective depth (a/d) ratio, number of prestressed GFRP layers, non-prestressed GFRP U-straps and U-jackets. Flexure beams strengthened with prestressed GFRP sheet showed higher increase in flexural capacity and better serviceability behaviour. Shear beams strengthened with prestressed GFRP sheets showed increase in shear capacity. Delay in sheet delamination was noted in prestressed GFRP sheet. Long term retention of prestressing force in GFRP sheets was noted in strengthened beams. An innovative and easy to apply prestressing system was developed in this study.

Recommendation was framed for modification in the prestressing system for field application.

The analytical tool i.e. Response 2000, used in this study accurately predicts the flexural beam behaviour whereas prediction of shear capacity was underestimated due to ignoring the effect of transverse clamping force on shear capacity. For the rest of shear beams, the model accurately predicted the shear capacity. Comparison of predicted and observed results of shear and flexure beams was carried out and it was concluded that prestressing of GFRP sheet is a useful and economical technique for strengthening of structural members.

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ABBREVIATIONS

ACI = American Concrete Institute

a/d = shear span to effective beam depth ratio

AFRP = Aramid Fibre Reinforced Polymer

ASCE = American Society of Civil Engineers

ASTM = American Standard for Testing Material

CFRP = Carbon Fibre Reinforced Polymer

CHBDC = Canadian Highway Bridge Design Code

CSA = Canadian Standard Association

CSCE = Canadian Society of Civil Engineer

EMPA = Swiss Federal Laboratories for Material Testing and Research

FRP = Fibre Reinforced Polymers

GFRP = Glass Fibre Reinforced Polymer

HKUST = Hong Kong University of Science and Technology

ISO = International Standard Organization

JCI = Japan Concrete Institute

LVDT = Linear Voltage Displacement Transducers

MCFT = Modified Compression Field Theory

NBCC = National Building Code of Canada

PCC = Plain Cement Concrete

PCI = Prestressed Concrete Institute

RC = Reinforced Concrete

RCC = Reinforced cement concrete

Symbols

A_{FRP} = cross-sectional area of FRP

A_s = cross-sectional area of non-prestressed longitudinal steel

a = maximum aggregate size used in concrete

b = width of the beam

E_f = Modulus of elasticity of FRP (Triantafillou and Antonopoulos 2000)

$E_{c,eff}$ = reduced modulus of elasticity (Collins and Mitchell, 1991)

E_{ci} = the instantaneous elastic modulus at time t_i ($E_{ci} = E_{ci}$)

E_{ci} = the initial tangent modulus of elasticity.

E_{FRP} = young modulus of GFRP sheet

E_s = modulus of elasticity of reinforcement

E_{vs} = young modulus of transverse steel

f_1 = principal tensile stress in concrete

f_2 = principal compressive stress in concrete

f_c = Concrete compressive strength (Triantafillou and Antonopoulos 2000)

f_c = Concrete web crushing strength (Spadea et al 2000)

f'_c = the 28 day compressive strength of concrete cylinder test

f_{ci} = Concrete stress applied at t_i days after casting

f_{cr} = stress in concrete at cracking

f_{sx} = average stress in x-reinforcement

f_{sxcr} = stress in x-reinforcement at crack location

f_{sy-cr} = stress in y-reinforcement at crack location

f_{syield} = yielding stress of reinforcement

f_{sy} = average stress in y-reinforcement

f_t = tensile strength of concrete

f_{ulFRP} = rupture stress of GFRP sheet

f_{vy} = yield stress of transverse reinforcement

f_x = stress applied to element in x-direction
 f_y = yield stress of longitudinal reinforcement
 f_y = stress applied to element in y-direction
 h = depth of the beam
 H = the relative humidity in %
 k_c = factor accounts for the influence of the volume-to-surface ratio of the member
 k_f = factor accounts for the influence of concrete strength
 k_h = factor accounts for the relative humidity
 k_s = factor accounts for the size of the member
 s_θ = spacing of cracks inclined at θ
 s_x = crack spacing in x-direction
 s_y = crack spacing in y-direction
 t = the time in days for which the concrete has been exposed to drying
 t_i = the time when stress is applied after casting
 t = the time when the strain is calculated
 y_c = position of layer relative to the top of beam
 y_s = position of longitudinal reinforcement to top of the beam
 y_{FRP} = position of GFRP Sheet to top of the beam
 v_c = reduction factor (Spadea et al 2000)
 $\varepsilon_{f,e}$ = Effective strain to axial rigidity of the externally bonded sheets (Triantafillou 1998)
 ρ_f = FRP ratio (Triantafillou and Antonopoulos 2000)
 $\mu\varepsilon$ = Micro strain
 θ = angle of inclination of principal strain to x-axis
 ρ_x = reinforcement ratio of x-reinforcement
 ρ_y = reinforcement ratio of y-reinforcement
 ν = shear applied to the element
 θ = angle of inclination of principal stress to x-axis
 ν_{ci} = shear on crack surface

ε_x = strain in x-direction

ε_y = strain in y-direction

ε_1 = principal tensile strain in concrete

ε_2 = principal compressive strain in concrete

θ = angle of inclination of principal strain to x-direction

γ_{xy} = shear strain relative to x and y axes

w = crack width

ε'_c = strain in concrete cylinder at peak stress f'_c

s_θ = crack spacing

ν_{ci} = maximum interface shear on a crack

ρ_v = amount of transverse reinforcement

ε'_c = Concrete strain at peak stress

$\Delta\varepsilon_{FRP}$ = initial pre-strain of FRP

$\phi(t, t_i)$ = the creep coefficient, and can be calculated using the following simplified expression (Collins and Mitchell, 1991):

CHAPTER 1

INTRODUCTION

1.1 Background

During the last century, concrete has played a vital role in the development of infrastructure around the world. The quality and properties of concrete have been improved during the last half century. A large number of reinforced concrete structures around the world, which are under service today, are inadequate to fulfill their intended purpose. The inadequacy of the concrete structures to fulfill its intended purpose mainly depends on the age of the structure and its exposure to the severe environment. The major causes of deterioration identified in deficient reinforced concrete structures include, corrosion of reinforcement, increase in design load, inferior quality of concrete, bad workmanship, vehicle collision, fire, blast pressure, earthquake loadings etc. The possible solution of the problem is to either rehabilitate the deficient concrete structure or to construct new structure. The decision of rehabilitation or replacement of a structure depends on its importance, the severity of damage, the availability of resources and the economic factors. The replacement of the structure is always an expansive and time consuming option. A rehabilitation technique, which extends the useful service life of the structure and consume less time, labour and economic resources, is always one of the alternatives to the replacement option.

Flexural and shear reinforcement is an important component of reinforced concrete structures. The deficiency or loss in the flexural and shear reinforcement reduces the strength and stiffness of a structure. To restore or enhance the stiffness

and strength of deficient concrete structures, additional flexural and shear reinforcements are often required. For additional flexural and shear reinforcement externally bonded steel plates have been used in most concrete structures during last three decades. This technique effectively restored the strength and stiffness of the strengthened structures. The major disadvantages of this technique are the heavy material weight and corrosion of steel plates at concrete and plate interface. The heavy material weight makes its application more laborious and increases the dead weight of the structure. Heavy machinery is often required for transportation and field application of steel plates. The high transportation and labour cost sometimes makes the process uneconomical. As heavy machinery and heavy weight materials are used in these techniques, in most cases the structure are required to be closed for operation during the rehabilitative process. The corrosion at the plate and concrete interface sometimes leads to premature delamination of externally bonded steel plates. To address the problems of steel plating, engineers have been in search of new materials for strengthening of structures. Fibre Reinforced Polymers (FRP), which are relatively new materials in civil engineering applications, have high strength to weight ratio and excellent corrosion resistance and are real competitors to steel plating.

Fibre Reinforced Polymers have been under research in laboratories and used in some civil engineering applications since the last two decades. FRP is used both in new construction as internal reinforcement and in rehabilitation/strengthening of structures as external reinforcement. FRP, being light weight, corrosion resistant and high strength material, is the best alternative to all other materials in rehabilitative methods. For flexure strengthening of concrete beams/girders, FRP sheet is normally bonded to the tensile face of beams/girders. U-straps or full jacket of FRP sheets bonded to vertical faces of beams are often used for shear strengthening of

beams/girders. FRP sheets consist of high strength fibres such as glass or carbon in an epoxy matrix. Most of Carbon Fibre Reinforced Polymer (CFRP) has high modulus of elasticity compatible to the modulus of elasticity of non-prestressed reinforcing steel, whereas Glass Fibre Reinforced Polymer (GFRP) has relatively low modulus of elasticity compatible to the normal strength concrete. CFRP has excellent chemical resistance as compared to GFRP. These properties make CFRP the 1st choice of the researcher. Lots of research and some field applications of CFRP have been carried out in the last two decades. Despite of the fact that CFRP sheet is the best material for rehabilitation work, it has not been widely used in field applications due to high material cost. GFRP is a relatively cheap material and could compete in the market with CFRP; however, it has its main defects of relatively low modulus of elasticity and potential strength decay in a high alkaline environment.

Although the GFRP sheet has a relatively low modulus, it is economical; therefore, the possibility of its use for rehabilitation work needs to be explored. The active strengthening technique i.e. pre-stressing of the GFRP sheet seems to be the best solution of the problem. In order to explore this application, first, a practical system of applying prestress to the FRP sheet is needed. Also an investigation on the improvement in flexural and shear behaviour of reinforced concrete beams, strengthened with prestressed GFRP sheets, under service and ultimate loads, is needed. The flexural and shear behaviour of beams strengthened with prestressed and non-prestressed GFRP sheets needs to be compared. The long term losses, in beam strengthened with prestressed GFRP sheet, due to creep and shrinkage etc. needs investigation. To investigate the behaviour of beams strengthened in flexure and shear with prestressed and non-prestressed GFRP sheet, an analysis method is required.

1.2 Objectives

The aim of the present study was to find means of using GFRP sheet, an economical material, in rehabilitation/strengthening of deteriorated concrete structures and to determine the long term losses due to creep and shrinkage. This goal was achieved by pursuing the following objectives;

1. to develop an innovative and practical method of FRP sheet prestressing and to compare it with methods of pre-stressing available in the literature;
2. to investigate experimentally the improvement in flexural capacity of beams strengthened with prestressed GFRP sheets and compare its results with experimental data of beams strengthened with non-prestressed GFRP sheets;
3. to investigate experimentally the improvement in shear capacity of beams with application of prestressed GFRP sheets on the tensile face of beams and compare its results with experimental data of beams strengthened with U-straps and full jackets in the shear span of the beams;
4. to investigate experimentally long term losses under sustained loads and to compare it with predicted results;
5. to conduct analytical study for estimation the flexure and shear behaviour of strengthened beams both with prestressed and non-prestressed GFRP sheets and compare its results with experimental data

1.3 Scope of Work

During the present study an innovative and practical method of pre-stressing was developed. The prestressing system consisted of four steel hollow sections as end anchors and prestressing bed. The GFRP sheet with end anchors was fixed in the prestressing bed and prestressing force was applied to get the desired level of prestressing. The tensile face of the beams was washed with high pressure water in the flexure beams and grinded with pneumatic grinder to remove the loss particles and

get a rough surface. The epoxy was applied to the beams surface and prestressed GFRP sheet was then glued to it. After curing of epoxy, the prestressed sheet was released to transfer the prestressing force to the strengthened beam.

This project included both experimental testing and analytical study. The experimental testing program was further subdivided into four major parts, 1) flexural strengthening; 2) shear strengthening; 3) long term losses study; and 4) ancillary material testing. For theoretical modeling of flexure beams, Response 2000 program developed by the University of Toronto and based on the Modified Compression Field Theory was used.

In flexure strengthening program, eight 2400mm long beams with rectangular cross section were constructed. The testing variables considered were ratio of flexure reinforcement, level of prestressing force in the GFRP sheet, and number of layers of GFRP sheets. Sufficient shear reinforcement was provided to ensure flexural failure of the testing beams. Two groups of beams i.e. group-1 and group-2, with different flexure reinforcement ratio were investigated. Group-1 consisted of six beams and group-2 consisted of two beams. Two beams, one from each group, were un-strengthened and acted as control beams. In group-1, two beams were strengthened with one and two layers of non-prestressed GFRP sheet, two beams each were strengthened with one layer of prestressed GFRP sheet with different prestressing levels, and one beam was strengthened with two layers of prestressed GFRP sheets. In group-2, one beam was strengthened with three layers of prestressed GFRP sheets. All beams were subjected to monotonic static load and were loaded to failure.

In the shear strengthening program, fifteen 3000mm long beams with rectangular cross section were constructed. The testing variables were shear span to depth ratio i.e. a/d ratio, type of shear reinforcement and layers of prestressed sheet. All tested

beams were divided into three groups on basis of a/d ratio. In each group one beam act as control, two beams were strengthened with one and two layers of prestressed GFRP sheets on the flexure face, one beam with U shape straps and, one beam with full jacket of GFRP sheet on two vertical and bottom faces of the beams. Sufficient flexure reinforcement was provided in all beams to ensure shear failure. All beams were subjected to monotonic, static load and were loaded to failure.

The long term losses due to shrinkage and creep were also studied. For obtaining the shrinkage of concrete data, two 500mm long beams with 100mm square cross section were constructed. The specimens were stored in a curing room and data were recorded as per American Standard for Testing Material (ASTM) standard. For creep, three concrete cylinders were constructed and were subjected to sustained load in a creep testing frame at 28 day age. The creep strain data was recorded as per ASTM standard. To investigate total losses, three beams were constructed and were strengthened with two layers of Prestressed GFRP sheet. Strain data was recorded as per ASTM standard.

In ancillary testing, six coupons of GFRP sheet were constructed and tested on 25kN tensile testing machine as per International Standard Organization (ISO) draft standard. Two 100mm wide strips of GFRP sheet were prestressed till failure to determine the approximate level of prestressing. Concrete cylinder and steel specimens, for both flexure and shear beams, were constructed and tested as per ASTM standard.

Response 2000 is a sectional analysis program and is based on the Modified Compression Field Theory. Response 2000 was tailored and used to predict the flexural and shear behaviour of un-strengthened and strengthened beams. The

program excellently predicted the behaviour of beams, both under service and ultimate loads.

1.4 Contents

The details of this investigation are presented in seven chapters. The literature review, which summarizes recent research in the following area: 1) strengthening systems, 2) flexure and shear strengthening with externally bonded steel plate and FRP laminates, 3) prestressing with steel and FRP tendons, 4) limited research on prestressed FRP sheet, and 5) long term losses due to creep and shrinkage, is presented in chapter 2. The experimental set up and instrumentation are discussed in chapter 3. The experimental results are presented in chapter 4. The results of analytical study are presented Chapter 5. A summary and thorough discussion on test results and comparison with analytical results are presented in Chapter 6. The conclusions of the study and recommendations for suggested future work are presented in chapter 7.

CHAPTER 2

LITERATURE REVIEW

2.1 General

Civil engineers have always been in search of new and economical techniques for strengthening/rehabilitation of structurally deficient and deteriorated reinforced concrete structures. The strength of concrete members may be inadequate due to increase in applied load or decrease in load carrying capacity due to corrosion of reinforcing steel or collision. Other reasons include fire, bad workmanship, blast and earthquake etc. The performance of reinforced concrete members largely depends on the quantity and condition of flexural and shear reinforcement. Additional shear and flexural reinforcement may be required to restore the strength and stiffness of deficient structural members.

There are several methods of repair or strengthening of a structure. Some of these methods use steel while others use FRP. Steel plate bonding has been used in field applications quite extensively around the world during the last three decades. Steel plating is an effective system of strengthening, but the transportation and placement efforts and durability of the system may limit its application in certain situation. External prestressing with steel tendons is another method used for strengthening of structures. Corrosion under severe environment is the main threat to the strengthening techniques using steel plates and tendons. FRP is a relatively new material and has high strength to weight ratio and excellent corrosion resistant properties. Several methods of flexure and shear strengthening with FRP, especially CFRP laminates and

sheets, have been investigated in the last decade. Techniques for prestressing of FRP tendons have also been investigated in laboratories. Limited application of CFRP sheets, plates and prestressed tendons has been reported so far. The CFRP could not gain popularity in the strengthening community due to higher material cost. In the present study the use of relatively cheap material i.e. GFRP for strengthening of deficient concrete members has been explored. Every strengthening method has some advantages and disadvantages. In the present study an innovative strengthening method has been developed which incorporated many of the benefits of previous methods and avoided significant shortcomings. Active strengthening technique i.e. prestressing has been used to fully utilize the strength of relatively low modulus GFRP. An Innovative method of prestressing the GFRP sheet has been developed. The durability and long term losses have also been investigated in the present study. The techniques presented in this review require careful attention as some of the aspects used in these techniques are applicable to the current study.

2.2 Flexure and Shear Strengthening of Reinforced Concrete Beams/Girders with Non-Prestressed Steel Plates and FRP Laminates.

2.2.1 General

A significant percentage of the bridges in North America were built after the Second World War. Most of them were originally designed for smaller vehicles, lighter loads, and a lower traffic volume than commonly experienced today. Only in the United States, over 50% of all bridges were built before 1940, and approximately 42% of these bridges are considered to be structurally deficient (Klaiber et al, 1987).

The flexure and shear strength of deficient concrete members can be restored with bonding steel plates to the concrete member. The bonded steel plate acts compositely with the concrete member. Steel is a dense and stiff material and can readily restore

the strength and stiffness of the concrete members (Swamy and Jones, 1992). However, significant efforts and heavy machinery are required for both transportation and the placement of plates due to heavy material weight and stiffness. If exposed to weathering, then steel is highly susceptible to corrosion. The situation gets worsen with the use of de-icing salt. The corrosion at the plate and concrete surface is the most dangerous situation (Meier, 1995). The corrosion of steel plate at concrete-plate interface can lead to de-bonding of the plate. Despite the short comings of this strengthening system, it has been successfully used in many field applications (Henwood and O'Connell, 1994). Disadvantages of this method include transporting, handling and installing heavy plates and corrosion of the plates (Garden and Hollaway, 1998)

In recent years, the repair of under strength or damaged reinforced concrete members by external bonding of FRP laminates has received considerable attention. FRP is light weight material which has high tensile strength and excellent corrosion resistance. The only drawback is that it is linearly elastic till failure. Flexure strengthening of concrete beams is accomplished by epoxy bonding the FRP plates to the tension face; for shear strengthening, the FRP plates are bonded to the beam web (Norris and Saadatmanesh, 1997). Experimental studies conducted by Shahway et al (1996), Arduini and Nanni (1997), Ross et al (1999), Rabinovitch and Frostig (2003) and Anania et al (2005) have demonstrated that epoxy bonding of FRP plates to the tension soffit of reinforced concrete beams can significantly increase the ultimate flexural capacity. The increased capacity can be as high as double the beam's original ultimate strength, depending on such factors as reinforcing steel ratio, concrete compressive strength, FRP ratio, FRP mechanical properties, and level of pre damage of the beam. The externally bonded composites laminate to concrete beams could

significantly improve the performance of this type of structures to resist impact loading. In addition, bonding laminates increase cracking and flexural strength, as well as residual stiffness of the beams. Furthermore, it reduces the number of cracks, crack widths, and the maximum deflection (Tang and Saadatmanesh, 2005).

FRP can be used for shear strengthening of reinforced concrete members. Many existing RC beams are deficient and in need of strengthening. The shear failure of a beam is clearly different from the flexural failure. In shear, the beam fails suddenly without sufficient warning and diagonal shear cracks are considerably wider than the flexural cracks. The need to prevent shear failure in old RC structures has been so significant that numerous studies have focused on shear strengthening (Norris et al., 1997; Taljsten, 2003; Khalifa and Nanni, 2002; Taljsten and Elfgren, 2000; Lin et al., 2000; Kachlakev and McCurry, 2000; Sato et al., 1996; Khalifa et al., 2000). Full scaled and scaled experimental studies have shown that proper quantities and placement of FRP for shear strengthening are needed to ensure adequate strength. The debonding of Shear FRP is a common problem which is due to non-availability of proper development length. This situation can be improved by providing proper anchorage. Higher ductility and in some cases change of mode of failure from shear to flexure can be achieved with properly designed externally bonded shear reinforcement.

2.2.2 Steel Plates

Externally bonded steel plates to the surface of concrete beams provide additional flexure or shear reinforcement to the strengthened beams. It increases the stiffness and strength of the member when loaded. The plates are typically mild steel. To achieve full composite action between concrete member and steel plate, they are bonded to the bottom face of beams, decks or slabs in bridges and buildings, with

epoxy and anchor bolts. The externally bonded steel plate strains under load and contributes to the strength of the member. This method of strengthening was pioneered in South Africa and France in 1960's (Shaw, 1993) and has been successfully applied in Europe, Japan and South Africa to strengthen bridges and buildings (Swamy and Jones, 1992). For more than three decades, structures in the United Kingdom were strengthened with this technique (Honwood and O'Connell, 1994).

Steel is a heavy weight material, therefore the size of the plate that can be used is limited. For long spans the plates must be spliced in some manner and then welded at site after application. Steel plates are difficult to shape and fix to irregular geometries. Swamy and Jones (1992) reviewed the structural behaviour of concrete beams strengthened with externally bonded steel plates and reported that full composite action of the beam and plate can be achieved, which improved the performance of the strengthened beams under service and ultimate loads. For certain situations it is simple and an efficient solution resulting in only minor increases in the member depth. This strengthening technique reduces the deflection and strain in the structural system. They noted superior crack control including a delay in crack initiation and retardation of crack growth in the plated beams. Increase in service and ultimate load has been reported.

The premature peeling failure of strengthened beams at relatively low load requires special consideration. To avoid this type of failure and achieve full composite action between concrete and steel plates the anchorage zone at the end of the plate must be appropriately designed. Corrosion of the steel plate by de-icing salt and exposure to the environment is another major threat to this technique of strengthening. The stresses and deflection induced by dead load of the structure are not relieved by

bonded plates. The steel plates are heavy and stiff, therefore, require great effort in transportation, handling and installation.

When beams are subjected to high shear forces, premature failure can occur by plate debonding. The plate debonding typically starts at the end of the plates and propagates through concrete cover to the depth of the lower flexure reinforcement. This sudden debonding results in substantial loss of beam strength. Such type of failure typically occurs in regions of high shear and rapidly changing bending moments (Swamy and Jones, 1992). Oehlers and Moran (1990) investigated the causes of premature failure of externally plated reinforced concrete beams and reported two distinct types of steel plate peeling. One type is shear peeling, caused by shear diagonal crack and the rapid peeling of plate due to high shear stresses created in concrete. The other type is flexure failure, caused by inducing a curvature in the plate which leads to high normal tensile stresses and ultimately fails the concrete cover.

With properly designed anchorage zone the peeling of plate can be avoided and hence the plated beam can maintain ductile failure beyond its ultimate strength. Anchor bolts resist the peeling forces. Staggering or tapering of the plate end reduce the shear stress concentration at the end of the plate and hence delay the peeling effect. The anchor bolts also prevent the buckling of the plate in the compression zone of the continuous beam. It is also used as backup system to prevent injury or damage in case of bond failure and plate separation from the beam (Swamy and Jones, 1992; Jones et al, 1988).

The externally bonded steel plates used for strengthening are exposed to the environment and are highly susceptible to corrosion. The chloride leakages from the joints and de-icing salt may cause corrosion at the steel epoxy interface which

ultimately leads to local debonding of the steel plate. To minimize the extent of this type of corrosion, epoxy having low moisture uptake must be used (Swamy and Jones, 1992; Shaw, 1993). The exposed steel plate may also be subjected to other hazards like fire etc.

The steel plates used for strengthening of existing structures only carry the additional imposed load and don't carry the original dead load. If the steel plates used for strengthening are prestressed then it can take the dead load of the structure. The up cambering of the structure with jack and then bonding steel plates before release of jacks can induce strain in the steel plate upon structure deformation. In this way a modest level of prestress is induced in the steel plate.

The steel is heavy and stiff material and need great effort in transportation, handling and fixing. Henwood and O'Connell (1994) reported the strengthening of Bolney Flyover, in which a total of 676, 6 m long, 360mm wide and 6mm thick steel plates were applied up to three layers to strengthen the reinforced concrete deck. The heavy weight of the plate was the main constrain in deciding the size of the plates for strengthening. The weight of each plate was 100 kg, and a total of approximately 70 tonnes of steel plates was used in this project. The steel is stiff material therefore special precautions were taken to ensure the flatness tolerances during preparation, transport and erection. Heavy and specialized machinery was used in this project. The project was completed in approximately 26 weeks.

Despite of the fact that steel plating can effectively restore the stiffness and strength of the structure, it still has some shortcomings. The significant short coming associated with steel plating identified in the literature includes: 1) heavy material weight, 2) high labour cost, 3) requirement of heavy machinery, 4) peeling failure,

and 4) corrosion of steel plating. These shortcomings of steel plating compile engineers to look for alternative materials, like FRP, for strengthening of structures.

2.2.3 Non-Prestressed FRP Laminates

2.2.3.1 General

The use of externally bonded FRP for flexure and shear strengthening of reinforced concrete structures is gaining popularity in the civil engineering community due to its high strength to weight ratio and excellent corrosion resistance property. FRP is light weight and easy to handle in installation, is the real alternative to steel plating in strengthening technology of existing reinforced concrete structures. Steiner (1996) has carried out the qualitative comparison of significant criteria of CFRP strips and steel plates which is shown in Table 2.1. The mechanical properties of FRP depend on the type of reinforcing fibre and fibre content. Carbon FRP (CFRP) is an expansive material but has higher modulus of elasticity and excellent chemical resistance under alkaline environment. Glass FRP (GFRP) is a cheap material but has relatively low modulus of elasticity and poor chemical resistance under alkaline environment. In the last decade, CFRP has gained much attention of researchers in both internal and external use due to its higher modulus of elasticity and ability of resistance in alkaline environment. CFRP sheets have been successfully applied for strengthening a number of concrete structures in Europe (Steiner 1996, Nanni 1997, Taljsten 2000) Japan (Ichimasu et al., 1993) and North America (Labossiere et al., 2000, Kachlakev et al., 2000). Despite of the fact that several field applications of CFRP sheets have been reported in the literature, it could not get vast popularity in field application probably due to high material cost.

For civil engineers the most important properties of FRP are its light weight and excellent corrosion resistance as compared to steel plates. FRP sheets and plates are

flexible and available in long lengths. Due to flexibility and light weight of FRP sheets and plates its installation is very easy and doesn't need any special equipments and techniques. The non-corrosive property of FRP makes it a more durable solution than steel plates for strengthening of reinforced concrete structures. The most significant disadvantage of FRP is its higher material cost. In certain situations, however, FRP is the most economical solution due to less labour cost (Meier and Erki, 1997).

The FRP is classified on the basis of the type of fibre used to reinforce sheet. The most frequently used fibres are carbon and glass fibres and less frequently used one is aramid fibres. The FRP used for external strengthening of structural members are either pre-peg sheets or pultruded plates. The FRP laminates are typically constructed of high strength continuous fibres in an epoxy matrix resin. The mechanical properties of the FRP laminates depend on the type of the fibres and its content. Stress-strain curves of various fibres are shown in Fig. 2.1. The stress-strain curves showed that all FRP laminates are linear elastic till failure. This property is significantly different from stress-strain behaviour of steel which has a yield plateau and a large failure strain. FRP is a brittle material and has no ductility beyond ultimate strength. At ultimate, the fibres rupture and have no residual stress. This is a significant disadvantage of FRP material. The FRP is light weight material which has density of approximately one quarter of steel which makes it easy to transport and install.

All three types of FRP i.e. Carbon, Glass and aramid, exhibit high tensile strength with varying modulus of elasticity. CFRP has a higher modulus of elasticity comparable to steel and has excellent chemical resistance in alkaline environment. These two properties have encouraged the civil engineers for field application of CFRP. CFRP also has excellent fatigue resistance. Brittle and sudden failure modes

and the reduction in ductility have been observed in the rehabilitated beams with CFRP composites (Rabinovitch and Frostig, 2003). The most significant disadvantage of CFRP is high material cost, which has limited its use in real applications. GFRP is less expensive material with relatively low modulus of elasticity comparable to modulus of elasticity of concrete. This allowed large deformations in rehabilitated members before failure.

2.2.3.2 Flexure Strengthening with Non-prestressed FRP Laminates

Extensive research has been conducted on the behaviour of beams reinforced with externally bonded FRP laminates in the last two decades. Lin et al (2000) investigated experimentally the behaviour of 28 beams strengthened in flexure with CFRP laminates and fabrics. The test variables were reinforcement ratio, CFRP thickness and anchorage methods. Three different flexural reinforcement ratios of 0.11%, 1.00% and 0.76% were used. The CFRP fabric tensile strength was 3550MPa and thickness was 0.11mm. Two types of CFRP laminate were used. One was Carbodur S1215 having tensile strength of 3050MPa and thickness of 1.2mm. The other type was Carbodur M1412 with tensile of 2900 MPa and thickness of 1.4mm. The test results are shown in Table 2.2. The experimental results showed greater increase in load carrying capacity of beams with low reinforcement ratio. The author concluded that strengthening with CFRP fabric or laminates was a highly effective technique which increased the strength by 20% ~60%. They observed three different types of failure mechanism i.e. debonding, rupture of CFRP and crushing of concrete. They also reported no difference in behaviour and results of beams strengthened with CFRP fabrics or laminates. Delay in crack initiation and reduction in crack width was reported in strengthened beams.

FRP sheets can significantly increase the ultimate capacity of beams in flexure. For lightly reinforced 2.4 m beam, the ultimate flexural strength of the strengthened beam is nearly double than that of un-strengthened beam as shown in Fig. 2.2. There is significant difference in the behaviour of the strengthened and un-strengthened beams. In the un-strengthened beam the ultimate capacity is controlled by yielding of reinforcing steel before crushing of concrete and hence showed large deflection before ultimate capacity of the beams. On the other hand the failure of strengthened beam was sudden and brittle at relatively low deflection. The increase in flexure capacity of large size beams with greater ratio of reinforcing steel was not so significant. For example, the ultimate strength of a 7 m beam with typical reinforcement was increased by 22% when the beam was strengthened with 1 mm thick CFRP sheet (Meier and Kaiser, 1991). The ultimate strength and stiffness of the FRP retrofitted beams increased with increase in number of layers of externally bonded FRP laminates. The increase in stiffness resulted in less deflection, lower ductility ratio and less energy dissipation (El-Deen et al 2000, Tang and Saadatmanesh, 2005).

The flexural strengthening with CFRP laminates controls both crack widths and overall deflections. Meier (1995) reported that crack width was dramatically reduced in the strengthened beams. The results of his research are shown in Fig. 2.3. The beam in this research program was loaded to 9.5 kN. The summation of the observed crack width at this load was 3.85 mm. The beam was unloaded and retrofitted with CFRP laminates. After retrofitting with CFRP laminates the beam was reloaded to 15 kN. It was noted that the summation of observed crack width was reduced to 2.58 mm at increased load. The deflection was reduced with retrofitting of the beam.

Several failure modes, controlling the ultimate strength of beams retrofitted with externally bonded FRP, have been observed by various researchers (Varastehpour and Hamelin, 1996; Triantafillou, 1998; Lin et al, 2000; Rabinovitch and Frostig, 2003; Pham and Al-Mahaidi, 2004). Four types of failure mechanisms for beams strengthened with FRP plates were identified by Varastepour (1996) and are shown in Fig. 2.4. The 1st is rupture of FRP plates, 2nd is crushing of concrete, 3rd is peeling of FRP plate due to debonding of plate at the epoxy/concrete interface and 4th is failure of concrete layer between the plate and steel. Pham and Al-Mahaidi (2004) discussed in detail the failure modes of strengthened beams. The control beam failed by typical steel yielding followed by concrete crushing. The S-series beam which were strengthened with two layers of CFRP sheet failed with combination of end and mid span de-bond. After steel yielded, flexure and shear cracks appeared clearly in the shear span. These cracks widened up as load was increased. Concrete was fractured simultaneously from the unclamped end of FRP and from the tip of wide flexure shear crack near middle of shear span. The failure of these beams is shown in Fig. 2.5. The E-series beams which were strengthened with 6 to 9 layers of CFRP sheet failed by typical end de-bond. Because of higher ratio of FRP reinforcement, the beam had higher stiffness after concrete cracked. Debonding happened either when tension steel had not yielded or just after it yielded. Flexure cracks appeared first in the pure bending region. On increase of load, visible flexure shear cracks appeared in the shear span. The portion of end shear crack in the concrete cover layer became more inclined and joined the adjacent shear crack. As the load was increased, since tension steel had not yet yielded, the tensile load in FRP was transferred to the intact portion and the crack propagated. The failure mode of this type of beams is shown in Fig. 2.6.

The debonding failure of strengthened beam with externally bonded FRP laminates can be categorized into three groups. The 1st group is peeling of sheet with thin layer of concrete at the sheet end points. The 2nd group is peeling of FRP sheet with thin layer of concrete at a point other than termination point of the sheet. The 3rd group is peeling of the sheet with large piece of concrete whose depth is from the lower tension reinforcement to the FRP sheet. These modes of failures were most frequently observed in the FRP strengthened beams and are confirmed by an extensive review of 64 specimens from ten studies which indicated that the most common mode of failure involved debonding of FRP sheet (Bonacci, 1996). The peeling of sheet along the length of the beam can be caused by initial unevenness or relative movement across the crack. The vertical displacement caused by the effects of the shear forces in the beam and the tensile force in the sheet may be sufficient large to peel the sheet. When the stresses created by these effects exceed the strength of the concrete, the FRP laminate will start peeling from the beam. This mechanism of failure is a function of concrete strength, the tensile force in the sheet, and the total shear resistance of the section including the dowel effect of the steel and FRP flexure reinforcement, which will effect the vertical movement across the crack. Varastehpour and Hamelin (1996) described the mechanism of the failure of full depth of concrete between the FRP and tension reinforcement from observation of beam failures. As shown in Fig. 2.7., the cover concrete below the flexure reinforcement tends to behave like a series of unreinforced cantilever beams. The tensile force in the FRP plate acts as a load at the tip of this cantilever beam. The concrete fails when the stresses at point A exceed the strength of concrete, and the entire section tear off from the beam.

To avoid peeling failures, several methods have been used such as changes in the geometry of the sheet in the anchorage zone, the use of mechanical anchorages, or use

of U straps of FRP sheet at the end of flexure FRP sheet or plate. These techniques have generally improved the performance of strengthened beams and delayed the premature debonding, but some of the local failures described above continue to occur. Bonacci (1996) observed that despite the specially designed anchorage zone, half of the beams failed with peeling of FRP sheets.

The ultimate flexural capacity of FRP strengthened reinforced concrete flexure members may be adequately evaluated by simple and direct analytical procedure derived from equilibrium equations and compatibility of strains (El-Mihilmy and Tedesco, 2000). This procedure is applicable to both singly and doubly reinforced rectangular sections, as well as flanged. To avoid a non-ductile failure, there is minimum and maximum FRP cross-sectional area requirement to exclude rupture of FRP or crushing of concrete, respectively. They established the limiting strain conditions in the cross section associated with FRP rupture and compression failure as shown in Fig. 2.8. In other words, they introduced upper and lower limit of cross-sectional area of FRP to ensure ductile behaviour of the strengthened beams. El-Mihilmy and Tedesco (2000) assumed that the beam is properly detailed with respect to FRP plate anchorage, shear reinforcement and epoxy bonding to preclude the anchorage, shear and debonding failures. Design monographs to facilitate implementation of the procedure were developed and are shown in Fig. 2.9.

Spadea et al (2000) modelled the behaviour at ultimate of a strengthened reinforced concrete beam by the truss analogy. They idealized a generic RC beam as plane truss as follows,

- a compressive stringer, carrying force C , and a tensile stringer, carrying force T . The distance between the stringers, d , equals the lever arm between the flexure tensile and compressive resultants;

- the beam shear strength is provided by internal vertical stirrups and concrete compressive chord;
- the concrete square yield locus, with zero tension cut-off is assumed as concrete yield condition. The assumed concrete web crushing strength is $f_c = v_c f'_c$, as the cylinder strength f'_c is reduced by an effective factor v_c ;
- perfect bond between concrete and external plate is assumed;
- the constant bond strength model with a zero tension cut-off is adopted as a yield condition for plate/concrete interface;
- the bond stress resultant U represents the global force flow of the plane stress field at the plate/concrete interface.

The assumed mechanism associated with the debonding failure mode is characterized by sliding of the plate, together with a diagonal crack along the compression trajectory ON , as shown in Fig. 2.10.

Based on the above assumptions, they derived the equation of the ultimate load for the bond failure mode. For bond failure not influenced by bond slip, four failure modes were defined as, a) failure mode related to crushing of concrete web and/or yielding of stirrups; 2) failure mode related to yielding of longitudinal and transverse reinforcement; 3) failure mode related to crushing of the concrete web and/or yielding of longitudinal reinforcement; and 4) failure mode related to FRP tensile rupture, yielding of longitudinal steel reinforcement and/or concrete crushing in compression. A comparison of experimental and theoretical strengths of the beam tests examined is given in Table 2.3. It can be observed that the ratio of experimental to theoretical strength ranges from 0.75 to 1.41, with a mean of 1.03, while the scatter of the predictions was relatively limited, with a coefficient of variation of 16%. The model tends to slightly underestimate the ultimate load, particularly in beams that failed in

debonding mode. The model was efficient in predicting the correct failure mode in many cases.

An et al. (1991) carried out a parametric study of reinforced concrete beams strengthened with FRP plates. The moment curvature response of rectangular and T-beam sections was carried out by plane section analysis. The variables in the study were steel reinforcement ratio, concrete compressive strength, and plate area and stiffness. They observed that ultimate moment of the beam increases with an increase in FRP area in case of failure mode other than debonding of FRP. They also reported that the ultimate strength of the strengthened beams increases linearly with increase in area of FRP to the point when the concrete in the upper fibre of the beam section fails in compression. The curve levels off at this point.

For modelling the complete member behaviour and to account for additional possible failure modes, researchers have used a number of section analyses or the finite element method. The analysis of adjacent sections provides a method of estimating shear stresses present through the depth of the beam as shown in Fig. 2.11. Some researchers feel it imperative to consider the tension stiffening behaviour of concrete in plane sections analysis (Arduni et al. 1996). When the tensile contribution of the concrete is considered, it may change the predicted mode of failure. Varastehpour and Hamelin (1996) introduced the slippage of the plate into their model. This consideration has some effect on the rigidity of the section. The peeling of the sheet may be predicted by estimating a shear stress at which this type of failure will occur. Peeling of the FRP is predicted to occur when the calculated shear stress exceeds the allowable shear stress determined from Mohr-Coulomb failure criteria. Estimation of the shear stress distribution at the sheet end point may be made by linear elastic and non linear equations. Finite element analysis has also been used to

predict shear stresses in the anchorage zone at the end of the FRP sheet (Malek et al., 1996).

2.2.3.3 Shear Strengthening with Non-prestressed FRP Laminates

The use of externally bonded FRP composite plates or sheets for the shear strengthening of reinforced concrete beams has become very popular in recent years. They can be used as flexible fabrics to wrap around corners and on convex surfaces, or they can be used as prefabricated laminates in various sizes and shapes. Externally bonded FRP fabric or laminates to the web of reinforced concrete beams and girders are effective shear reinforcement. The FRP fabric and laminates are typically bonded to the web of the beams with fibre aligned vertical to the longitudinal axis of the beam. With couple of exceptions, most researchers have idealized FRP shear reinforcement in an analogy with internal steel stirrups. In other words the FRP shear reinforcement behaves similar to the internal stirrups and becomes effective when the shear cracks open up. Extensive research has been done on shear strengthening of reinforced concrete beams in the last decade (Sato et al., 1996; Norris et al., 1997; Taljsten and Elfgren, 2000; Lin et al., 2000; Kachlakev and McCurry, 2000; Khalifa et al., 2000; Khalifa and Nanni, 2002; Taljsten, 2003).

The pre-treatment of the concrete surface is essential for good bond and strengthening effect. In most cases, sandblasting or grinding is used but also water blasting can be an alternative. The surface should be free from contaminants such as grease, oil, dust, etc., before adhesive is applied. For wrap system it is quite common to use a primer to enhance the bond for the adhesive (Taljsten, 2003). Khalifa et al., (2000) carried out an experimental investigation on the shear strengthening of RC beams with FRP composites. They studied the influence of concrete surface roughness on the shear strengthening effect of the FRP and concluded that this

variable have no significant effect. They recommended that surface preparation by regular water or sand blasting is sufficient.

The CFRP sheets bonded to the web or tension face of reinforced concrete beams increased the strength and stiffness of the beams. The magnitude of the increase and the mode of failure are related to the direction of the reinforcing fibres (Norris et al., 1997). They observed a larger increase in stiffness and strength when the CFRP fibres were placed perpendicular to the crack. Brittle failure occurred in these beams due to concrete rupture as a result of stress concentration near the end of CFRP. When the CFRP fibres were placed obliquely to the cracks in the beam, a smaller increase in strength and stiffness was observed; however, the mode of failure associated with this off-axis application of CFRP was more ductile and preceded by warning signs such as snapping sounds or peeling of the CFRP.

Taljsten (2003) also studied the behaviour of the shear strengthened beams with FRP fabrics. He tested seven beams where the variables were the angle and the weight of the fabric. All beams were tested in four points bending with no steel stirrups in the shear region. All the beams in the test are shown in Fig. 2.12, and the beam data are recorded in Table 2.4. The CFRP sheet was not wrapped around the beam. Two type of failure were noticed, first compressive failure in the concrete struts and second fibre failure in composite sheets. The angle of the fibre with respect to shear cracks has an effect on the strength and stiffness of the strengthened beams. It is clear from the load deformation curve shown in Fig. 2.13 that a minimal increase in strength was achieved in beam C4 in which the fibres were oriented parallel to the longitudinal axis of the beam. He reported that it is difficult to conclude that higher strengthening effect is obtained using thicker fabrics. Khalifa and Nanni (2002) also reported that increase in the amount of CFRP may not result in proportional increase in the shear strength.

They observed that an increase of 250% in CFRP amount in one strengthened beam in comparison to other strengthened beam resulted in only 10% increase in shear capacity. Higher CFRP ratio used for shear strengthening has little effect on the failure load and mode, but it can slow the cracks development and reduce cracks width (Lin et al., 2000). The shear span to beam depth (a/d) ratio also has an influence on the contribution of the externally bonded CFRP reinforcement to the shear capacity. Similarly great increase in shear capacity can be achieved in beam with no shear reinforcement as compared to beams with adequate shear reinforcement (Khalifa and Nanni, 2002). Lin et al. (2000) also observed that the gain in the ultimate strength was more significant in the beams with lower stirrups ratio.

Shear strengthening with CFRP composites bonded to the face of the concrete beam is a very effective method which can increase the strength of the beam by almost 300%. An increase of up to 100% in the strength of the completely fractured beams is possible (Taljsten and Elfgren, 2000). The use of GFRP for shear strengthening was reported sufficient to offset the lack of steel stirrups and allowed conventional RC beams to fail by yielding of the tension reinforcement. The change in the failure behaviour allowed the strengthened beam to fail at 200% higher deflection as compared to the pre-existing shear deficient beam (Kachlakev and McCurry, 2000).

The shear mode of failure is always very brittle; however in the beams strengthened with externally bonded shear reinforcement a number of failure modes have been observed. These include shear failure due to FRP rupture, shear failure without FRP rupture, and shear failure due to FRP debonding. Taljsten and Elfgren (2000) reported three kinds of failure modes in their experimental investigation, one is tensile rupture of the CFRP in the shear span, second is compressive failure of the

concrete, and third is bond failure between concrete and CFRP composite. Shear cracks normally appear near the support and propagate toward the loading point. In some cases a diagonal crack may appear abruptly. As the width of the diagonal crack increases, the maximum strain in the FRP eventually reaches its ultimate strain and FRP immediately tears at its most highly stressed location. The failure is brittle due to the brittleness of FRP and classified as shear failure due to the FRP rupture. The shear failure without FRP rupture is essentially the same as that of shear failure due to FRP rupture, except that FRP is not torn at failure in this mode of failure. This type of failure has been normally observed in beams strengthened with AFRP or GFRP strips. The ultimate strain of AFRP and GFRP is around 2.25%, which allows the failed beam to continue to carry a significant load. A shear strengthened RC beam may fail due to the debonding of the FRP from the beam. Debonding failure almost always occurs in the concrete at a small distance from the concrete/epoxy interface with some concrete attached to the bonded FRP. Concrete strength plays a key role in this failure mode. Once the FRP starts to peel off, the beam will fail quickly. The ductility of the beam failing in this mode is usually very limited.

Lin et al (2000) reported two types of failure in their experimental investigation, one is CFRP rupture and other is debonding of CFRP. They also reported that CFRP rupture usually appeared in the U-type strips near the supports because of longer anchorage length at this position and debonding failure appeared near the loading point if no anchorage strips is used here. The anchorage strip helps in delaying or preventing the debonding failure. Sato et al (2000) compared the behaviour of the beams strengthened with U type FRP strips and the FRP attached only to the vertical faces of the beams. They concluded that U type FRP strips were more effective in shear strengthening than the FRP strips attached to only two faces. They also

observed that the shear force carried by the FRP was greater than the shear force carried by steel stirrups.

Triantafillou (1998) reviewed the relevant literature up to 1997 and characterized the contribution of externally bonded FRP to the shear capacity of RC members based on both analysis and all experimental results available. He reported that the effectiveness of the external FRP shear reinforcement and its contribution to the shear capacity of RC members depends on the mode of failure, which may occur either by peeling of FRP from the concrete surface, or by FRP tensile fracture at a stress that may be lower than the FRP tensile strength. The mode of failure to occur first depends on the bond conditions, available anchorage length, type of attachment at the FRP curtailment, FRP thickness, FRP elastic modulus, concrete strength, and other factors. He derived a polynomial function that relates the strain in the FRP at shear failure of the member, defined as effective strain $\varepsilon_{f,e}$ to the axial rigidity of externally bonded strips or sheets. This polynomial was derived through curve fitting on about 40 test data published by various researchers and showed a clear tendency of effective strain reduction with increase in axial rigidity. The main drawback of this model was that concrete strength was not introduced as design variable.

Triantafillou and Antonopoulos (2000) refined the early model by Triantafillou (1998) and presented the FRP contribution in an analogy to conventional shear reinforcement, according to Eurocode, ACI, and JCI. The key element in the model is the calculation of an effective FRP strain, which is taken as the minimum of three values: maximum strain to control the crack opening, strain corresponding to premature shear failure due to FRP debonding, and strain corresponding to shear failure combined with or followed by FRP tensile fracture. The last two strains were shown to be functions of the quantity $E_f \rho_f / f_c^{2/3}$ (decreasing functions as this

quantity increases). For a given concrete strength it increase linearly with $E_f \rho_f$ until this product reaches a limiting value $(E_f \rho_f)_{lim}$ beyond which debonding controls and the gain in the in shear capacity is small unless the FRP is fully wrapped or properly anchored.

No distinction was made between side bonding and U jacketing in the model presented by Triantafillou and Antonopoulos (2000). The close examination of the data shows that the average ratio of the observed to the predicted FRP strain by this model is 1.07 with standard deviation of 0.4. This is statistically not satisfactory for direct practical use to produce safe design (Chen and Teng, 2003). The International Federation for Structural Concrete recommended the use of the Triantafillou and Antonopoulos (2000) effective strain with a reduction factor of 0.8.

Arduini and Nanni (1997) developed analytical technique for prediction of the behaviour of beams with bonded FRP generally termed as of discrete element analysis. This is rather simple model that account for the mechanical properties of the constituent materials and the characteristics of the concrete-to-FRP interface. The resultant tensile forces are found from the sectional moment-curvature analysis at the end of each discrete element. This approach is computationally attractive but has disadvantage that it does not consider the effects of diagonal tension cracking in the shear span of a beam (Wang and Chen, 2003). Wang and Chen (2003) refined the model of Arduini and Nanni (1997) by incorporating the effects of diagonal tension cracking resulting from the presence of shear forces in a beam. The main assumptions considered in the analytical model using the technique of discrete element analysis are

1. Plane section remain plane under bending;
2. the behaviour of concrete in tension is ignored in the flexure calculations but accounted for in the shear calculation;

3. perfect bond exists between composites plate and concrete beams; and
4. the presence of the adhesive layer between the composites plate and the reinforced concrete beam is ignored.

The Modified Compression Field Theory (MCFT) is implemented as a part of the refined analytical model for the prediction of the behaviour of FRP composite beam at any load level. The procedure for predicting the response using the MCFT of a beam with FRP plate applied to its soffit and loaded in shear and bending moment are described in the paper by Wang and Chen (2003). Upon convergence of the data, the sectional force can be computed. From the sectional forces the FRP bond stresses and the beam deformation can be calculated.

To improve the behaviour of the strengthened beam under service loading condition and to utilize the full strength of the FRP, some researchers have investigated the use of prestressed FRP tendons and laminates which are presented in the next section.

2.3 Prestressed Tendon and FRP Laminate Strengthening Systems

2.3.1 General

Concrete structures are prestressed in order to improve serviceability, i.e. to control deflection or to reduce or eliminate cracking. In conventional prestressed concrete structures, an initial compressive force is applied and sustained by highly tensioned steel reinforcement reacting on the concrete. Prestress applied to the concrete structure also helps in offsetting the effect of dead and live loads. Conventional steel tendons are commonly used to induce the prestress in concrete structure. In recent years, the deterioration of prestressed concrete structures by the corrosion of the prestressing steel, and the cost of rehabilitation have become a major concern. The use of FRP tendons for prestressing application is one of the most promising developments over the past two decades to overcome the problem of corrosion of

steel. The strengthening of reinforced concrete with prestressed FRP plates or sheet is relative new idea but due to its promising results, it is getting attention of researcher now a day.

2.3.2 External Prestressing with Steel Tendons

The principles of conventional prestressed concrete apply also to externally prestressed members, in which the prestressing force acts outside the concrete. The effect of the prestress is, to apply an axial load and a bending moment that opposes the self weight of the beam. The external prestressing of new concrete bridges with steel tendons or bars has been pioneered and used in Germany, Belgium, and France since 1930's (Virlogeux, 1990). This method involves the tensioning of high strength cables by reacting against the structures to be reinforced. An improvement in the range of elastic behaviour, ultimate capacity and fracture behaviour are benefits gained from this technique; a reduction in structural material weight and improved fatigue behaviour are also result (saadatmanesh and Albrecht, 1989). Post-tensioning has proven to be an effective and economical technique, and its use has expanded such that approximately 25 concrete bridges in Germany have strengthened with unbounded prestressed tendons (Falkner et al, 1995).

For flexure strengthening of existing concrete beams unbonded tendons made of high strength steel strand, wire or bar are used. These tendons are attached and tensioned against anchors which are built into the beam. The concentrated force of the prestress is transferred to the beam at the location of anchor. The deviation saddles are used to hold the tendons in place and transfer the components of the prestressing force into the beam. The deviation saddles may also used to harp the tendons used for strengthening. The fatigue behaviour of concrete members is improved because prestressing the tension face of a beam reduces the tensile component of the stress

cycles, thereby delaying or preventing fatigue crack initiation and growth in steel girders or reinforcing bars in concrete (saadatmanesh and Albrecht, 1989a). The external prestressing system is fully exposed and therefore little effort is required to install, monitor and replace if required. However several problems are also associated with this method, both at the time of construction and during the service life. These include the need to maintain lateral stability of girders during post-tensioning and the need to protect the cable against corrosion, collision, fire or vandalism. Due to high stress concentration at the anchorage zone it needs special design consideration. The installation of the anchors can be labour intensive. In the external prestressing, unbonded tendons are used which contribute much less to the ultimate strength of the beam as compared to the bonded tendons.

For corrosion protection, the tendons may be encased in a barrier. The most common barrier used is a high density polyethylene duct. The tendons are placed in these ducts and then stressed. After stressing the tendons the ducts are filled with cement grouts to protect the tendons against environmental attacks, corrosion, sabotage and fire. Grouting can be difficult to control, and it is possible that a section will be left unprotected (Bruggeling, 1990).

Klaiber et al (1985) outlined various prestressing schemes which have been used to strengthen reinforced concrete beams are shown in Fig. 2.14. Scheme A introduces a constant moment along the length of the beam to counteract applied loads. The other schemes introduce varying moment along the length of the beam. The tendons are anchored at each end and harped at desired points with the help of deviation saddles. The forces at the harp points are transferred to the beam through deviation saddles. Deviation saddles typically consist of steel tubes, brackets or preformed concrete elements with sufficient radius of curvature to prevent damage to the tendon.

In reinforced concrete beams the tension reinforcement is bonded to the concrete. When a reinforced concrete beam deforms, the bonded reinforcement that span a crack also deforms. The strain in the reinforcement creates a tensile force that contributes to the flexure resistance of the section. In the case of unbonded reinforcement, the phenomena do not occur. The strain induced in the unbonded tendons upon deformation and cracking of the section is less significant and hence the increase in the flexural capacity above the prestress is limited. To determine the additional contribution of the unbonded tendon to the ultimate flexural strength of the beam, it is necessary to conduct a complete member analysis, by computing moments and curvatures at a number of sections. Unbonded tendons are also less effective in controlling the crack growth; therefore, its effect on the concrete shear strength is limited. The vertical component of the prestressing force in the harped unbonded tendons resists the applied shear force. It is observed by Bonneau and Massicotte, (1994) that the shear strength of the beams strengthened by horizontal tendons has been significant increased. Their research on 5m long beam has shown an increase of 83% in the shear capacity of beam strengthened with horizontal prestressed tendons.

2.3.3 External Prestressing with FRP Tendons

The corrosion of the reinforcing and prestressing steel has become a major issue in the concrete structures. The situation become more dangerous in case of externally bonded prestressing steel tendons. One solution is the use of FRP as substitute to the steel reinforcement and prestressing tendons. FRP has high strength to weight ratio and excellent corrosion resistance which make it right candidate for substitution. The strengthening of existing structures with externally bonded FRP tendons is in the research stage. The prestressing system with FRP tendons is similar to that with steel tendons. The unbonded FRP tendons are attached to the beam by anchorage blocks.

Harp profiles, if required, are attained with the use of deviation saddles. The external prestressing system is an effective technique to strengthen the structure. Within specified limits and restriction the Canadian Highway Bridge Design Code (CHBDC) permit the use of FRP tendons as the main component of an external post tensioning strengthening system (CHBDC, 1996).

The prestressing with FRP tendons has all the advantages of the steel prestressing system with the added advantage of being non corrosive. Special concentration and efforts are required for proper design and installation of anchorage zone. A variety of anchorages are used to grip FRP tendons. These consist of anchors in which the FRP is encased in an epoxy or grout or held in a mechanical wedge type anchor. The anchorage of the FRP tendons presents difficulties. The tendons are much more susceptible to damage during the process of applying prestress (Erki and Rizkalla 1993). Carbon and glass fibres in particular have a low lateral compressive strength and, therefore, are susceptible to failure when highly stressed at the anchor. Jerrett et al, (1996) also reported the anchorages failure in two out of four test beams. This shows the difficulties in anchorage of the FRP tendons.

Unlike a steel strand which can be gripped to achieve its full tensile strength, FRP tendons will fail before their full tensile capacity. Al-Mayah et al, (2000) studied the behaviour of CFRP tendons wedge type anchorage system under monotonic and cyclic loading condition. The anchorage system was consisting of stainless steel barrel, four steel wedges and aluminium or copper sleeves. They tested the four wedge anchor-CFRP rod under monotonic and cyclic loading conditions. The variable in the static load test were loading steps, wedges usage history (new or re-used), and sleeve material (copper or aluminium). They concluded that as load increased the displacement of the rod and the sleeves decreased. No effect of anchor reuse was

noted. The anchor with copper sleeves performed poorly as compared to anchors with aluminium sleeves at small loads. The effect of cyclic loading on the slip behaviour of the rod was minor.

The Canadian Standard Association (CSA) S806-00 “Design and Construction of Building Components with Fibre Reinforced Polymers” limits the stresses at jacking as 40% of ultimate tensile strength of AFRP tendons and 70% of CFRP tendons for both pre and post tensioning. It also recommend the permissible stresses in AFRP and CFRP tendons as 70% and 85% of the ultimate strength respectively for both pre and post tensioned bonded tendons. In case of unbonded post tensioned tendons the limit is 65% and 80% for AFRP and GFRP respectively.

Strengthening of Concrete beams with the use of external prestressing tendons has been investigated in the laboratory during last decade (Jerrett et al., 1996; Gowripalan et al., 1996; Bakis et al., 2000; and Zou et al., 2000). Generally all the beams strengthened with externally bonded FRP tendons showed an increase in the flexure strength and reduction in the mid span displacement at ultimate. Like the Steel tendons, the contribution of the unbonded FRP tendons to the ultimate strength of the member is less significant than the bonded one. Dolan and Burke (1996) derived three equations for predicting nominal capacity of the FRP prestressed beams and calibrated against the existing literature data which validate their proposed equations.

Extensive laboratory testing and research have been conducted on the use of CFRP wires for prestressing or reinforcing of concrete elements. Few field applications have been reported, of which Herning Bridge Denmark has CFRP stay cables and CFRP reinforcement in its bridge deck. This is pedestrian bridge carrying moderate load only. This is the case with most of the bridges built with FRP reinforcement. Corte

and Bogaert (2005) found only eight bridges built so far around the world, using CFRP prestressing tendons and carrying road traffic.

2.4 Prestressing With FRP Laminates

2.4.1 General

The FRP laminates including sheets and plates can be prestressed before bonding to the concrete surface for flexure or shear strengthening. The prestressed laminates provide excellent control of cracks initiation and its width. It reduces the deflection at ultimate load and contributes significantly to the strength of the strengthened beams/girders. The behaviour of the strengthen member with prestressed FRP laminate improved considerably under service load condition. The reduction in the cracks width with the prestressed FRP laminates helps in delaying the corrosion of the main steel reinforcement of the strengthened member. As discussed previously in this literature review that the failure of the strengthened beams with non prestressed laminates is mostly due to the peeling of the laminate before utilizing the full tensile strength of the FRP. The prestressing of the FRP laminates helps in utilizing efficiently a greater portion of the tensile capacity of the FRP laminates. The contribution of the prestressed FRP laminates to the ultimate capacity of structural member is superior to the prestressed FRP tendons because it is bonded to the structure at its full length. Like prestressed tendons, the prestressed FRP laminates offset the effect of the dead and live load even before additional deformation occurs in the structure. Prestressed FRP laminate effectively restores the loss of internal prestressing of a prestressed structural member due to corrosion or other reasons.

The mode of failure of the prestressed FRP laminate is similar to the non prestressed FRP laminates, i.e., FRP rupture or FRP debonding. Due to severe stress concentration at the sheet end it requires special design consideration for end

anchorage zone. The FRP laminates are susceptible to failure during prestressing. It is also labour intensive to prestress the FRP before bonding. Despite of these challenges and extra effort requirement it is still feasible to use prestressed FRP laminates due to the extra benefit offered than the non prestressed FRP laminates.

The prestressing of the FRP laminates is relatively new subject and still under research in the laboratory. Currently three systems have been used experimentally to apply prestressing in the FRP laminates. One is cambering the structure, to be strengthened, before bonding the FRP laminates. The structure is released after setting of the bond and moderate level of prestress is induced in the bonded FRP. The second method is prestressing the FRP plate by applying direct tension through jacking against external reaction. The third is applying prestressing force to the FRP sheet against the strengthened beam. Small and full scale beam have been strengthened using these methods.

2.4.2 Prestressing of FRP plates with Cambered beam

In this method the prestressing force is not applied directly to the FRP laminates. The prestressing force is induced indirectly into FRP laminates by cambering the strengthened beam. Saadatmanesh and Ehsani (1991) conducted an experimental study of the strengthening of reinforced concrete beams using non-prestressed and prestressed GFRP plates. The beams were cambered upwards by the use of hydraulic jacks, so that compression was induced in the lower face of the beam and tension in the upper face. The jacking force of 36kN was applied at two locations on either side of the mid span. The GFRP plates were attached to the lower face of the beam. When the FRP was fully bonded to the concrete, the jacking force is removed. The beam deflected under its self weight and sustained dead loads and hence induced prestress in the bonded GFRP plate. One of the two prestressed beams contained a relatively

small amount of tensile steel reinforcement, and other contained larger bars and were pre-cracked prior to bonding of the plate. The plate prestress in the pre-cracked case closed some of the cracks, indicating the benefit of the prestressing from serviceability point of view.

Three other rectangular and a T beam were strengthened with non-prestressed GFRP plates. The beams had an overall depth of 455 mm, a width of 205 mm and a length of 4880 mm. The compressive strength of the concrete used was 35MPa, and average yielding strength of the steel was 456 MPa. The ultimate tensile strength of the GFRP plate was 400 MPa with modulus of elasticity of 37GPa. The tensile reinforcement was varied from 250 to 1000 mm^2 . All beams were strengthened with 6 mm thick, 152 mm wide, and 4260mm long GFRP plate. The beams were test under four point bending with a clear span of 4570 mm between the supports. The study indicated that the flexure capacity of the beams can be increased with bonded GFRP plates to the tensile face of the concrete beam. The researchers concluded that the introduction of prestress in the GFRP plates resulted in improved cracking behaviour of strengthened beams.

Char et al (1994) carried out analytical solution of the reinforced concrete girders strengthened with externally bonded prestressed FRP plates, which were investigated experimentally by saadatmanesh and Ehsani (1991). The analytical model developed was aimed to study the effect of concrete strength, material and geometric properties of FRP on the static strength of the girders. Plane section analysis with linear strain distribution and small deformation was used to calculate the flexure capacity. Creep, shrinkage or shear deformations were neglected. Strain compatibility was assumed, with no slip between concrete and steel or FRP reinforcement. Parabolic stress strain relationship with ultimate strain of 0.003 was used for concrete. A bilinear elastic

plastic stress strain relationship for steel reinforcement and linear elastic till failure stress strain relationship for FRP reinforcement was used. Analysis was carried out for theoretical beams strengthened with prestressed and non-prestressed CFRP and GFRP plates. The prestress was induced in the FRP plates by cambering of the beam. The analytical result of the beams with non-prestressed FRP plates are shown in Fig. 2.15, and that with Prestressed FRP plates in Fig. 2.16. The two curves correspond to beams reinforced with 2600 mm^2 and 5200 mm^2 area of CFRP plate respectively. The analysis did not show an increase in the moment capacity of the section with prestressing of the plate. However the cambering of the beam induced tensile and compressive stresses in the top and bottom fibres of the beam respectively. These stresses delay the crack initiation in the beam. The cambering of the beam also induced an initial compressive stress of 28 MPa in the longitudinal tension reinforcement of the beam which helped in delayed yielding of the steel. As perfect bonding was assumed between the concrete and FRP plate, therefore, the calculated ultimate moment of the beam with prestressed FRP plate was slightly less than non-prestressed beams. This can be attributed to the fact that prestressed FRP was highly stressed than non-prestressed FRP at the time of the application and failed in tension.

This method is simple and requires little effort for preparation of the beam before application of FRP plate or sheet. In this method the high tensile stresses created in the top fibres of the beam is of major concern and hence limit the level of prestressing of the applied FRP plates or sheets. A low level of prestressing as compare to ultimate strength of FRP is achievable in this method.

2.4.3 External Prestressing of FRP Laminates

The external prestressing of the concrete beams using FRP plates has been studied during last decade. Beam strengthening using non-prestressed bonded FRP plates has

been studied far more widely than the use of prestressed plate. External prestressing of the FRP plate was pioneered in Swiss Federal Laboratories for Material Testing and Research (EMPA). An experimental prestressing of inducing prestress into CFRP plate was developed at EMPA by means of bonded steel plate anchors and an external stressing system (Meier et al., 1992; and Triantafillou et al., 1992; Deuring, 1993).

The anchorage system developed includes bonding of the FRP plate to the two steel plates at each end. Tapered steel plates were used to minimize the effect of stress concentration. The FRP and steel plate's surfaces were roughened. The steel and FRP plates were then bonded together with high performance epoxy and cured in hydraulic jacks. The plate anchor is shown in Fig. 2.17. The steel plate anchors were then held in the steel clamps which were connected to a steel cable. The steel cable was then attached to a jacking unit at one end and a load cell at the other end on a prestressing frame. The prestressing force is then applied to the FRP plate. After post tensioning the FRP plate epoxy was applied to the lower face of the beam and upper face of the plate. The beam was then placed over the prestressed FRP plate. After the epoxy was fully cured the jacks were gradually released to transfer the prestressing force to the beam (Triantafillou et al., 1992). The plates were then cut at the ends. Lateral anchorage system was used to prevent the peeling failure due to the high shear stress present at the plate ends. The Laboratory set up for the external FRP prestressing system is shown in Fig 2.18 for medium scale beams and Fig 2.19 for large scale beams.

Deuring (1993) conducted four point bending tests on large scale T beams 6.7m long strengthened with 200x1mm CFRP plates with or without prestressing. The cross-sectional dimensions for the T beam were 260x340mm for the web and 160x900mm for the flange. The beam was reinforced with 2000mm² of tensile steel reinforcement. These beams were tested under six point bending at a clear span of

6000mm as shown in Fig. 2.20. Seven specimens were constructed, out of which four were strengthened with CFRP plate prestressed to 50% (1000 MPa) of its ultimate strength. One beam with prestressed CFRP plate was tested with static load till failure. The author concluded that addition of the non-prestressed CFRP plate increased cracking moment, yield moment, ultimate strength and stiffness of the beam. The experimental results showed that the cracking and yielding moment was increased with prestressed CFRP plates. The effect on ultimate strength was very little. The displacement at failure was much more less for beams with prestressed CFRP plates in comparison to the beams with non-prestressed beams as shown in Fig. 2.21. Deuring (1993) also studied the fatigue behaviour of the beams strengthened with non-prestressed FRP plates and founded this strengthening technique to be highly resistant to fatigue with failure occurring after breaking the internal steel reinforcement and subsequent yielding of the remaining overload test. In EMPA Meier et al. (1992) also carried out similar fatigue test on the beams strengthened with CFRP plates prestressed to 50% of its ultimate. Thirty million cycles were performed without any evidence of damage to either the concrete or the sheet.

Deuring (1993) also constructed and tested fourteen medium sized rectangular beams with cross-sectional dimension of 300x250mm and 2400mm length. These beams were reinforced with 800 mm² of tensile steel reinforcement and 200 mm² of CFRP plate as shown in Fig. 2.22. Eleven beams were strengthened with prestressed plates, two with non-prestressed plate and one without strengthening. The CFRP plates were prestressed from 50% to 75% of its ultimate strength. Eight of the prestressed beams were subjected to static loading and were tested under four-point bending over a clear span 2000mm. The test results of one control beam, one beam strengthened with non-prestressed CFRP plate and two beams strengthened with

prestressed CFRP plates are shown in Fig. 2.23. The control beam exhibit classical reinforced concrete behaviour. The cracking, steel yielding and ultimate loads for the beam strengthened with non-prestressed CFRP plate were much higher than Control beam. The beam strengthened with CFRP plate prestressed to 50% of its ultimate strength showed further improvement in the cracking, steel yielding and ultimate load capacity over the beam strengthened with non-prestressed CFRP plate. The beam strengthened with CFRP plate prestressed to 75% of its ultimate capacity, showed little improvement in the cracking load but the ultimate load was much less than the beam strengthened with plate prestressed with 50% of ultimate strength. The failure in both beams strengthened with prestressed CFRP plate occurred due to tensile rupture of the plate. The beam strengthened with non-prestressed CFRP plate failed with peeling of the plate at concrete/plate interface. The author concluded that prestressing can delay or even prevent this type of premature peeling failure and hence helped in utilizing the full strength of the CFRP plate.

Deuring (1993) studied the peeling behaviour of the plate and showed that normal stresses in the concrete in the horizontal direction decrease where the prestress is lost at the ends, but there is significant increase in the shear stresses. This high shear stress causes peeling failure. The distribution of the normal and shear stress are shown in the Fig. 2.24.

Prestress levels of at least 25% of the sheet strength may be necessary to achieve a significant improvement in term of the structural stiffness and load carrying capacity of the reinforced concrete member (Garden et al 1997). Meier et al (1992) suggested that a prestress level as high as 50% of the plate strength might be necessary to increase the ultimate strength by delaying the peeling failure of the FRP plate. As discussed above in the review of the test results presented by Deuring (1993), that

increasing the level of prestress in the CFRP plates from 50 to 75% reduced the ultimate strength of the beam because the highly prestressed laminates had little strain capacity remaining and the CFRP plate ruptured prematurely. Darby et al. (2000) demonstrated that two CFRP plate prestressed to 50% of the plate strength could increase the load carrying capacity of an 18m long girder by 46% compared to a 24% increase for three similar non-prestressed CFRP plates.

Garden and Hollaway (1998) study and compared the failure modes of the plated beams with and without prestress. For the purpose he tested seven 1.0 and seven 4.5m lengths reinforced concrete beams in four point bending. All beams were under reinforced to encourage flexure failure and the achievement of full bending capacity. Shear span to effective depth ratio of 4.05 and 7.44 was used for 1.0 and 4.5m beams respectively. Different level of prestressing ranging from 25 to 50% of the plate ultimate strength was induced into the bonded plate by using external prestressing system as developed by Deuring (1993). They used two types of anchorages one is continuation of the plate under the support and other is installation of steel bolts through the composites plate. The prestress level in the FRP plate, ultimate capacity and failure modes of the beams are given in Table 2.5. It was observed that the non-prestressed beams failed by separation of the plates from the beam, while most of the prestressed beams failed by plate fracture. The plate prestress prevented cracking of the adhesive layer, a phenomena associated with shear cracking in the concrete. The author also noted that for the beam with shear span to effective depth ratio of 3.40 plate separation was initiated by a shear displacement in the concrete.

Wu et al (1999) designed a system for prestressing CFRP sheets to strengthen reinforced concrete structures. They investigated several anchorages system to avoid debonding failure near the ends upon releasing the pre-tensioned force. The authors

also investigated the appropriate prestressing stress level of the CFRP sheet and structural optimization of reinforcement. The full details of prestressing system are shown in Fig. 2.25. For beams with prestressed CFRP sheets, three type of anchorage system were used. These anchorages system were; 1) U type extra bonded CFRP sheet; 2) extra bonded flat CFRP sheet; and 3) anchor bolts. The test specimens were reinforced concrete beams with cross-sectional dimensions of 150x200mm and 2100mm length. The CFRP sheet had a tensile strength of 3200MPa, modulus of elasticity of 230GPa, and thickness of 0.11mm. The prestress level of 25% of the tensile strength of the CFRP sheet was used in this investigation. All the beams were test over a clear span of 1800mm. In the 1st set of test three beams were tested. One beam was control, one beam strengthened with two layers of non-prestressed CFRP sheets, and one beam strengthened wit two layer of prestressed CFRP sheets. The CFRP sheet used was 150mm in width and 1700mm in length. The results showed that there was 100% increase in cracking load and 25% increase in ultimate load of the beam strengthened with two layers of prestressed CFRP sheets when compared with the beams strengthened with non-prestressed CFRP sheets. In the second set of test five beams were tested. One was control and other four were strengthen with two layers of 100mm wide and 1600mm long prestressed CFRP sheets. Different end anchorage systems were investigated and it was observed that the mode of failure for all the strengthened beams was by debonding and anchorage failure except the beams with large anchor area and 8 anchor bolts. The later beams failed by rupture of CFRP sheets. The increase in ultimate load carrying capacity of these beams was higher than the beams failed by debonding of the sheets.

Meier (2001) presented a new concept of strengthening with prestressed FRP sheets. The FRP is prestressed against external beam, epoxy is applied to the

prestressed sheet and the beams to be strengthened before brought into contact. After bonding the prestressed FRP sheet into the beam, heating elements are used to cure the central region of the prestressed sheet. At curing the central region the prestress level is lowered and the heating elements are moved to the next sections on the either sides of the central region. This process is repeated in several stages until the entire length of the FRP sheet is bonded and the prestress level at the ends of the sheet has been reduced to a low level. With this method maximum prestress in the middle and lower prestress toward the end of the sheet can be achieved. The author argued that with this method the shear stress at the end of the prestressed sheet will be reduced and hence no anchorage will be required.

CFRP has recently become popular in repair and rehabilitation of the deteriorated reinforced concrete structures due to its higher modulus of elasticity, but the difference in the CFRP and concrete properties is not favourable for transferring the prestress from the CFRP sheet to reinforced concrete member (Yue-lin et al 2005). An experimental study was conducted by Yue-lin et al (2005) on strengthening of reinforced concrete beams using prestressed GFRP sheet which has modulus of elasticity very close to the concrete. The ultimate loads and deflections of the beams strengthened with non-prestressed and prestressed GFRP sheet were studied. The GFRP sheet was prestressed to 50% of the ultimate strength of the sheet. The prestressed GFRP sheet caused camber in the strengthened beams with out cracks on the tensile face. It was concluded by the authors that load carrying capacity of the beam strengthened with prestressed GFRP sheet was enhanced by 100% as compare to 55% for the similar beam strengthened with non-prestressed GFRP sheet. The deflection under the same external load was higher for the beam strengthened with

non-prestressed GFRP sheet than the beam strengthened with prestressed GFRP sheet. The prestressed GFRP sheet reduces the deformability of the strengthened beam.

The research on the shear strengthening of the reinforced concrete beam with externally bonded non prestressed FRP laminate during last decade has been discussed previously in this literature review. Lees et al (2002) investigated the use of post tensioned; non laminated CFRP straps as external shear reinforcement for concrete. An experimental study was carried out to determine the influence of the post tensioned straps on the shear capacity of a concrete T-beam. An un-strengthened control beam and a beam strengthened with post tensioned FRP straps at 200mm centres were investigated. The CFRP tape used for strengthening was 0.16mm thick and 12mm wide with modulus of elasticity and ultimate rupture strain of 130GPa and 1% respectively. The straps were made of twenty layers of the tape. Based on the preliminary test on the strap, it was expected to fail at a load of 100kN. 30mm hole were drilled through the top flange of the concrete T beam to accommodate the strap. A specially designed interface steel pad was grouted on the bottom face of the beam. An equivalent interface pad was placed on the steel base plate on the top surface of the beam. The tape was wrapped around the pads until required number of layers was achieved. The straps were located at 200mm centres and were tensioned by lifting the steel base plate on which the top pad was supported. The average stress of 2MPa was induced through the concrete web. The authors conclude that non-laminated prestressed CFRP strap is an efficient and durable reinforcing element. The concrete beam strengthened with CFRP straps exhibited a significantly higher load capacity than an un-strengthened beam. The stiffness and angle of crack of the strengthened beam was increased with post tensioned CFRP straps.

2.4.4 Prestressing of FRP Laminates against the Strengthening Beam

Only a few studies of concrete girders strengthened with epoxy bonded prestressed FRP sheets jacked against the strengthened girder are reported in the literature (Izumo et al, 1997; Wight et al, 2001; and El-Hacha et al 2004).

Wight et al., (2001) examined the feasibility of post strengthening large scale reinforced and prestressed concrete beams with prestressed CFRP sheets. The beam specimens were of rectangular cross section 300x575mm and total length of 5000mm. The CFRP sheet used was 0.2mm thick and 300mm wide, having effective modulus of elasticity of 125GPa. The mechanical anchorage system consisted of steel roller anchors bonded to the sheets and steel anchor assemblies fixed to the beams. The roller anchors that gripped the sheet consisted of two stainless steel rollers bonded to each end of the sheet. The sheet was wrapped 2.5 times around the roller at least three days prior to prestressing operation. To prestress the sheets, the roller at one end of the FRP sheet was fixed to the beam (dead end) and the roller at the other end was moveable (jacking end). During prestressing, the moveable rollers were attached by steel prestressing strand to a hydraulic jack that reacted against the beam. The prestress was applied to the sheet, and the sliding roller was the attached, in its extended position, to a second permanent anchorage assembly. Subsequent layers were added to the beam, using the same technique, until all five layers were attached. The anchorage system is shown in Fig. 2.26. The authors reported that the prestressed FRP sheets can significantly reduced the crack width and delayed the onset of Cracking. The yielding of the steel reinforcement delayed in the strengthened beam with prestressed sheets. The ultimate strength of the concrete beam also improved with prestressed FRP sheet.

Izumo et al (1997) investigated experimentally and theoretically the efficiency of strengthening T-beams with prestressed FRP sheets. They studied the difference in strengthening effects between aramid and carbon fibre sheets, the prestressing levels and bond strength of the sheets. The cross sectional dimensions of the T beams were 150x300mm for the web and 100x500 for the flange, with a total length of 2760mm. the beams were tested under three-points loading over a clear span of 2500mm. The aramid sheet used had a tensile strength of 2493MPa and modulus of elasticity of 85.9GPa. The carbon sheet used had a tensile strength of 3879MPa and modulus of elasticity of 242GPa. Prestressing level of 28% and 23% of the ultimate tensile strength were attained in the aramid and carbon fibre sheets, respectively. The sheets were bonded and fastened to metal fittings at the end of one side. Steel frame was installed at the other end, and the other end of the sheet was wrapped around the roller. The roller was then attached to a hydraulic jack mounted on the steel frame and attached to the roller with steel bars. The tension was applied to the sheet and was bonded to the beam with epoxy. The sheet was kept under tension until the epoxy cured. Their experimental results showed that the strength of the strengthened beams with prestressed aramid and carbon fibre sheets was increased by 11% over the unstrengthened beams. The mode of failure of all beams was the strengthened beams were peeling shear failure. They found that the effective length of bonding FRP sheets did not depend on the sheet type and was about 100mm.

El-Hacha et al (2004) developed a new anchorage system to directly prestress the CFRP sheets by jacking and reacting against the strengthened concrete beam itself. The anchorage system consisted of a moveable flat plate anchor bonded to the sheets at the jacking side and a fixed anchor bonded to the sheet at the dead end. The beams permanent jacking and dead end anchors consisted of steel angle brackets mounted on

the sides of the beams web by three 20mm diameter threaded rods. The sheets were bonded at both ends to their anchors and allowed to cure for one week. Epoxy was applied to CFRP sheet and the concrete beam before prestressing. The CFRP sheets with their anchors at both ends were placed in position on the beam and then tensioned using a hydraulic jack that reacted on permanent anchors mounted on the web of the beam. They studied the feasibility and effectiveness of using bonded prestressed CFRP sheets to strengthened pre-cracked concrete beams at room (+22°C, +72°F) and low (-28°C, -20°F) temperatures. They observed significant increases in flexural stiffness and ultimate capacity as compared to the control un-strengthened beams. They noted that flexural behaviour of the strengthened beams was not adversely affected by short term exposure to reduced temperature (-28°C, -20°F).

2.5 Summary

An extensive literature review of strengthening reinforced concrete members in flexure and shear by using steel plate, non-prestressed FRP laminates, prestressed steel and FRP tendons and prestressed FRP laminate was conducted. The review shows that considerable efforts has been directed toward the use of FRP for structural strengthening and rehabilitation of building and bridges during last couple of decades. Most of the research has been focused on using CFRP which have excellent chemical resistance and mechanical properties.

Three method of prestressing FRP Laminate, i.e. cambering the strengthened beam, prestressing against the external reaction frame and prestressing against the strengthened beam have been developed and applied to various sized beams, small, moderate and large sized beams, rectangular and T-section beams. Prestressed levels up to 75% the ultimate strength of the FRP sheet was used in some tests. Only the flexure behaviour of the beams strengthened with prestressed FRP laminate has been

studied by various researchers. Results showed that the beams strengthened with FRP sheet were often stronger and the cracking and yielding loads were significantly higher than beams with non prestressed FRP sheets. Test results also showed that the prestressed FRP sheets controls the cracks width and delay the onset of cracking which is of significant importance to serviceability based design criteria. In many case the prestressing changed the mode of failure from peeling of the sheet to the tensile rupture of the FRP sheet. Strengthening with prestressed FRP sheet can help to utilize the full tensile capacity of the FRP which in turn reduces the quantity of FRP sheet. This will to achieve more economical design.

For shear strengthening various configuration of externally bonded FRP laminates, i.e. U straps, full sheet on three side of beams, wrapping of FRP straps around the beam and full wrapping with FRP sheet have used and studied. Post tensioned FRP strap has been used by one researcher for shear strengthening. All the test results showed increase in the shear capacity of the strengthened members.

For experimental program herein, the external prestressing methods using GFRP sheet with an innovative anchorage system will be used. Flexure and shear behaviour of the beam using prestressed FRP sheet will be studied. The effect of prestressed FRP sheet, bonded to the tension face of the beams, on the shear capacity will be studied. The long term losses in the prestressed beams will also be a part of the following investigation.

Table 2.1 Comparison of significant criteria of CFRP sheet and steel plate strengthening (Steiner, 1996)

Criteria	Strengthening with CFRP strips	Strengthening with steel plates
Own weight	Low	High
Tensile strength	Very high	High
Overall thickness	Very low	Low
Corrosion	None	Yes
Length of plate	Any	Limited
Handling	Easy, flexible	Difficult, rigid
Load bearing	Longitudinal direction only	In any direction
Lap joints	Easy	Complex
Fatigue behaviour	Outstanding	Adequate
Materials costs	High	Low
Installation costs	Low	High
Application	No equipment	With lifting equipment and clamping devices

Table 2.2 Experimental results of flexural strengthened beams (Lin et al., 2000)

Beam	Steel ratio	CFRP	Layer	Adhesive	Anchorage method	Fu (KN)	Fur (KN)	Fur/Fu	Mode
Fa1	0.11%	-	-	-	-	30	-	-	F
Fa2*	0.11%	FTS-C1-20	One	FR-E3P	-	-	73	2.43	R
Fa3*	0.11%	FTS-C1-20	Two	FR-E3P	-	-	100	3.33	S
Fa4*	0.11%	CarboDurM	One	Sikadur30	-	-	150	5.00	S
Fa5*	0.11%	Local	Three	Local	-	-	62	2.07	D
Fb1	1.00%	-	-	-	-	132	-	-	F
Fb2*	1.00%	FTS-C1-20	One	FR-E3P	-	-	165	1.25	C
Fc1	0.76%	-	-	-	-	125	-	-	F
Fc2	0.76%	-	-	-	-	124	-	-	F
Fc3	0.76%	FTS-C1-20	One	FR-E3P	-	-	165	1.33	D
Fc4	0.76%	FTS-C1-20	One	FR-E3P	-	-	165	1.33	R
Fc5	0.76%	FTS-C1-20	One	FR-E3P	-	-	166	1.34	R
Fc6*	0.76%	FTS-C1-20	One	FR-E3P	-	-	150	1.21	D
Fc7	0.76%	FTS-C1-20	One	Local	-	-	155	1.25	D
Fc8	0.76%	CarboDurS	One	Sikadur30	-	-	175	1.41	D
Fc9	0.76%	CarboDurM	One	Sikadur30	-	-	185	1.49	D
Fc10	0.76%	FTS-C1-20	One	FR-E3P	Beyond supports	-	178	1.44	R
Fc11	0.76%	FTS-C1-20	One	FR-E3P	U type CFRP strip	-	160	1.29	R
Fc12	0.76%	FTS-C1-20	One	FR-E3P	Expansion bolts	-	155	1.25	R
Fc13	0.76%	FTS-C1-20	One	FR-E3P	Expansion bolts	-	162	1.31	R
Fc14	0.76%	FTS-C1-20	One	Local	Expansion bolts	-	150	1.21	R
Fc15	0.76%	CarboDurS	Two	Sikadur30	-	-	180	1.46	D
Fc16	0.76%	Local	Two	Local	-	-	155	1.25	D
Fc17	0.76%	FTS-C1-20	Three	FR-E3P	-	-	185	1.49	D
Fc18	0.76%	FTS-C1-20	Three	FR-E3P	-	-	190	1.53	D
Fc19	0.76%	FTS-C1-20	Three	FR-E3P	-	-	200	1.61	D
Fc20	0.76%	FTS-C1-20	Three	FR-E3P	U type CFRP wide strip	-	210	1.69	D
Fc21	0.76%	FTS-C1-20	Three	Local	-	-	190	1.53	D

Note: 1. Beams with "*" means they had been loaded before repairing.

2. Fu represents the ultimate strength of unrepaired beam, Fur represents the ultimate strength of repaired beam.

3. "F", "R", "D", "C" represent flexural, rupture, debonding and crushing failure mode respectively.

Table 2.3 Test beam properties and comparison between experimental and analytical results (Spadea et al., 2000)

Reference	Beam	Geometric Data				Concrete		Plate			Long. reinf.		Transv. reinf.			Ultim. Strength		Failure Mode ^m	
		b (mm)	h (mm)	a (mm)	l _p (mm)	f _c (MPa)	A _p (mm ²)	b _p (mm)	f _{py} (MPa)	A _s (mm ²)	f _{sy} (MPa)	A _t (mm ²)	s (mm)	f _{tr} (MPa)	Exp. (kN)	An. (kN)	Exp.	An.	
Spades et al. (1997),(1998)	A1.1	140	300	1800	1750	30.0	96	80	2400	402	435	56.5	150	435	43.4	45.4	D	D	
	A3.1	140	300	1800	1750	30.0	96	80	2400	402	435	23.1	150	435	37.4	40.2	D	D	
David et al. (1997)	P2	150	300	900	800	40.0	450	150	55	308	500	56.5	130	500	50.5	47.8	F	F,S	
	P4	150	300	900	800	40.0	900	150	55	308	500	56.5	130	500	57.5	53.7	D	S	
	P7	150	300	900	800	40.0	120	100	2400	308	500	56.5	130	500	68.0	50.3	D	D	
Arduini et al. (1997)	A4	200	200	700	550	33.0	195	150	2506	308	540	56.5	150	540	55.0	38.9	S	D	
	A5	200	200	700	550	33.0	390	150	2906	308	540	56.5	150	540	45.0	38.9	D	D	
	B2	300	400	1100	1000	30.0	51	300	3000	398	340	100.5	100	340	85.0	84.8	F	F,S	
Hutchinson et al. (1996)	A.078	200	150	750	700	54.0	117	150	1532	157	575	56.5	150	575	31.6	38.9	S	F,S	
	A.115	200	150	750	700	54.0	172	150	1532	157	575	56.5	150	575	32.5	43.2	S	F	
	B.040	200	150	750	700	54.0	60	150	1532	157	575	56.5	75	575	27.0	31.3	F	F,S	
	B.115	200	150	750	700	54.0	172	150	1532	157	575	56.5	75	575	35.0	43.2	F	F	
	C.040	200	150	750	700	54.0	60	150	1532	402	575	56.5	75	575	38.0	47.8	F	F	
	C.115	200	150	750	700	54.0	172	150	1532	402	575	56.5	75	575	51.0	51.3	F	F,D	
Quattrill et al. (1995)	A1c	100	100	300	280	70.0	96	80	500 ⁽²⁾	84.8	350 ⁽²⁾	14.1	100	350 ⁽²⁾	22.0	19.0	D,S	S	
	A2b	100	100	300	280	42.0	96	80	500 ⁽²⁾	84.8	350 ⁽²⁾	14.1	100	350 ⁽²⁾	18.4	16.2	D,S	S	
	A2c	100	100	300	280	42.0	96	80	500 ⁽²⁾	84.8	350 ⁽²⁾	14.1	100	350 ⁽²⁾	18.7	16.2	D,S	S	
	B2	100	100	300	280	58.0	96	80	500 ⁽²⁾	84.8	350 ⁽²⁾	14.1	50	350 ⁽²⁾	17.0	14.1	D,S	D	
Fanning (1997)	3A	100	150	577	502	37.0	96	80	2400	100.5	250	56.5	135	250	18.7	18.2	S	D	
	4A	100	150	577	502	32.5	480	80	250	100.5	250	56.5	135	250	14.0	17.7	S	D	
	4B	100	150	577	502	32.5	96	80	2400	100.5	250	56.5	135	250	16.2	17.7	S	D	
Roat et al. (1999)	1B	200	200	914	914	54.8	90	200	2206	142	410	141.7	102	410	40.0	42.9	F	F	
	2B	200	200	914	914	54.8	90	200	2206	239	410	141.7	102	410	49.0	46.5	F	F	
	3B	200	200	914	914	54.8	90	200	2206	400	410	141.7	102	410	54.5	51.1	F	F	
	4B	200	200	914	914	54.8	90	200	2206	567	410	141.7	102	410	53.8	56.9	F	F	
	5B	200	200	914	914	54.8	90	200	2206	774	410	141.7	102	410	73.4	64.6	F	F	
	6B	200	200	914	914	54.8	90	200	2206	1019	410	141.7	102	410	84.5	74.0	F	F	

(1) : F=Flexure; S=Shear; D=Debonding ; (2) : estimated value

Table 2.4 Test beam properties and results of shear beams (Taljsten, 2003)

Beam	f _{cc} (MPa)	f _{ct} (MPa)	E _{cc} (MPa)	C _w (g/m ²)	γ (°)	P (kN)	δ (mm)	α (°)	Failure mode*
R1	67.4	4.4	45.1	—	—	248.1	7.8	27	B/C
RC1	67.4	4.4	45.1	200	45	612.1	21.4	35	B/C
C1	67.4	4.4	45.1	125	45	493.3	17.3	31	B/F
C2	71.4	4.8	46.5	200	45	514.4	17.2	32	B/C
C3	58.7	4.3	42.1	200	90	521.2	18.2	30	B/C
C4	58.7	4.3	42.1	200	0	308.1	8.9	25	D/C
C5	71.4	4.8	46.5	300	45	668.6	24.3	33	B/C

* B, brittle; D, ductile; C, failure in concrete; F, fibre failure.

Table 2.5 Prestress level, ultimate capacities and failure modes of the beam (Garden and Hollaway, 1998)

Beam	Anchor method	Nominal prestress(%)	Actual prestress(%)	Maximum load (kN)	Failure mode
1.0 m beams	—	—	—	—	—
Unplated	—	—	—	14.85	Steel yield followed by concrete crushing
1 U _{1.0 m}	None	0	0.0	34.00	Plate separation at base of shear crack (Figure 8)
2 U _{1.0 m}	Supports	0	0.0	49.65	Plate separation at base of shear crack (Figure 8)
3 U _{1.0 m}	Bolts	0	0.0	45.50	Plate separation at base of shear crack (Figure 8)
4 P _{1.0 m}	Supports	25	23.5	49.60	Plate separation at base of shear crack (Figure 8)
5 P _{1.0 m}	Bolts	25	23.7	43.65	Shear failure outside plated length at end
6 P _{1.0 m}	Supports	50	47.2	50.50	Tensile fracture of composite plate
4.5 m beams	—	—	—	—	—
Unplated, 2 bars	—	—	—	28.5	Steel yield followed by concrete crushing
Unplated, 3 bars	—	—	—	42.3	Steel yield followed by concrete crushing
1 U _{4.5 m, 2 bars}	None	0	0.0	60.0	Plate sep. from near load point (Figures 5 and 6)
2 P _{4.5 m, 2 bars}	Bolts	25	26.5	63.8	Tensile fracture of composite plate
3 P _{4.5 m, 3 bars}	Bolts	25	26.2	70.5	Conc. crush followed by plate tensile fracture
4 P _{4.5 m, 3 bars}	Bolts	40	33.6	72.8	Tensile fracture of composite plate
5 P _{4.5 m, 3 bars}	Bolts	50	46.6	76.5	Tensile fracture of composite plate

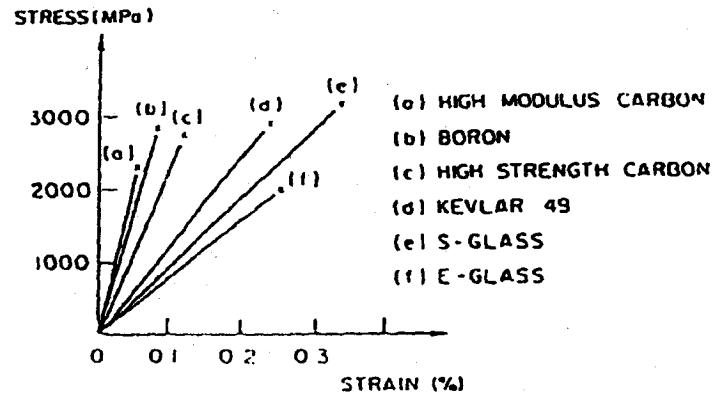


Fig. 2.1 Stress-strain curves for typical fibres (Neale and Labossiere, 1991)

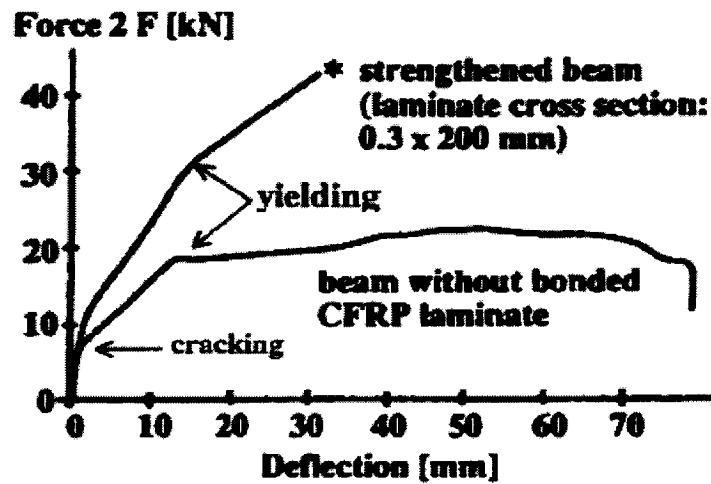


Fig. 2.2 Load-Deflection curves of 2.4m strengthened and un-strengthened concrete beams (Meier and Kaiser, 1991)

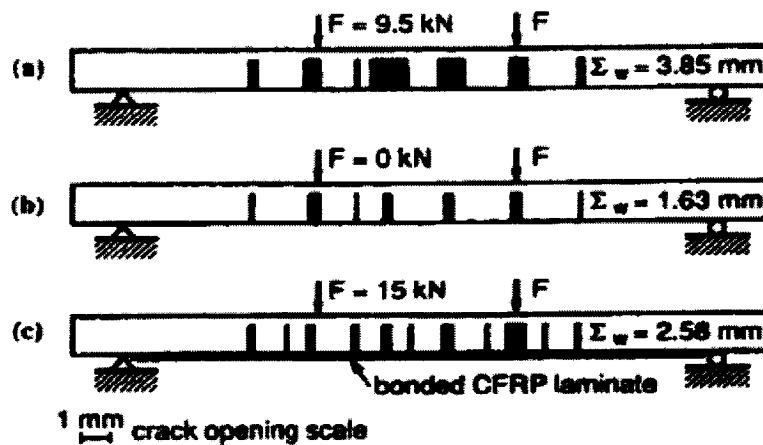


Fig. 2.3 Crack control in concrete beam strengthened with CFRP sheet (Meier, 1995)

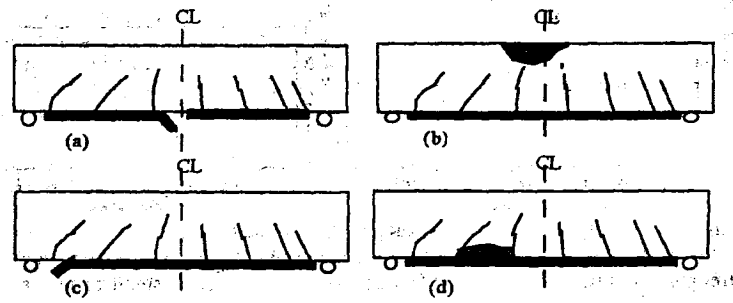


Fig. 2.4 Failure modes in a reinforced concrete beam strengthened with FRP sheets (Varastehpour and Hamelin, 1995)

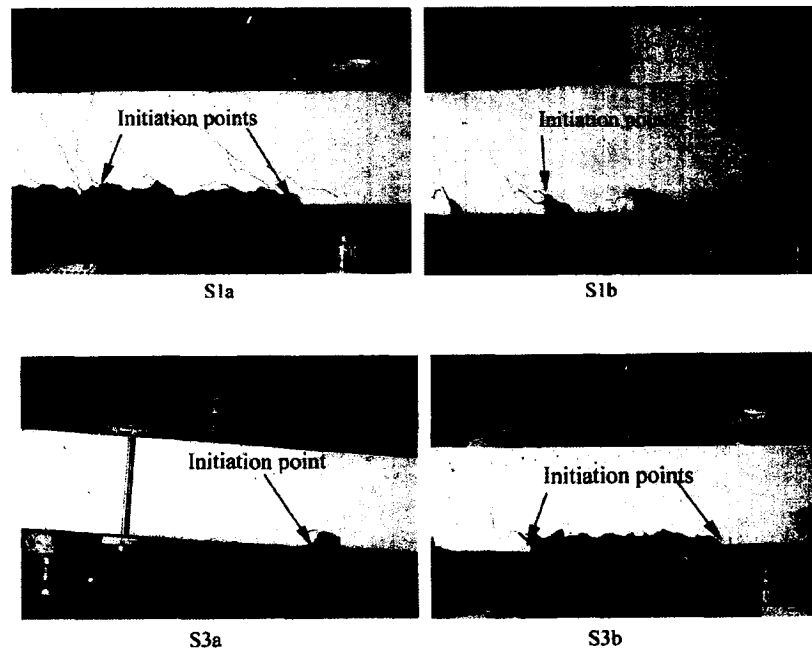


Fig. 2.5 Failure modes in a S-series beams strengthened with two layers of CFRP sheet (Pham and Al-Mahaidi, 2004)

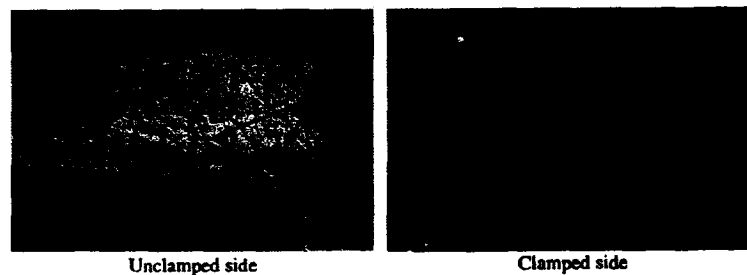


Fig. 2.6 Failure modes in a E-series beams strengthened with six to nine layers of CFRP sheet (Pham and Al-Mahaidi, 2004)

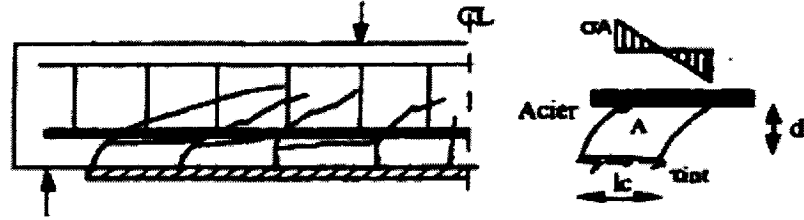


Fig. 2.7 Crack propagation and teeth behaviour in the concrete (Varastehpour and Hamelin, 1995)

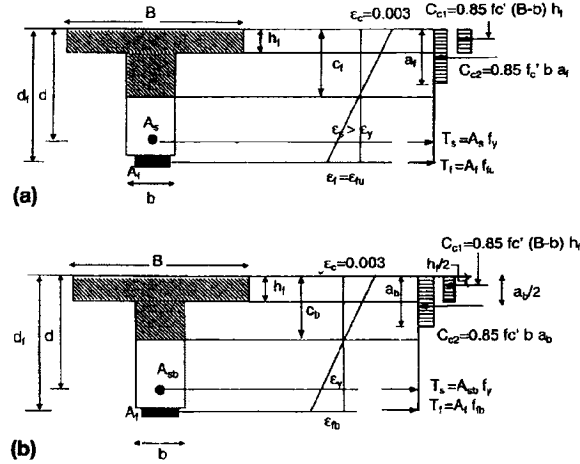


Fig. 2.8 Strain conditions corresponding to (a) Minimum and (b) Maximum FRP cross-sectional areas for T-sections (El-Mihilmy and Tedesco, 2000)

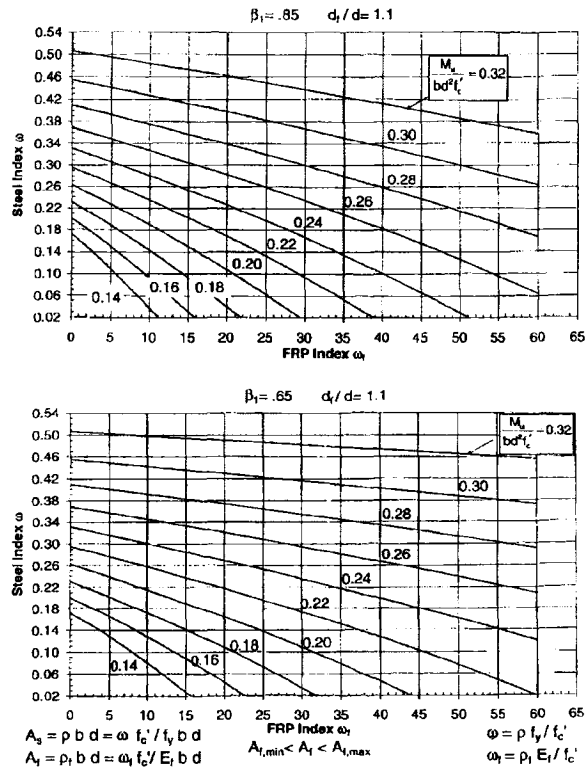


Fig. 2.9 Design monographs for FRP strengthened concrete beams (El-Mihilmy and Tedesco, 2000)

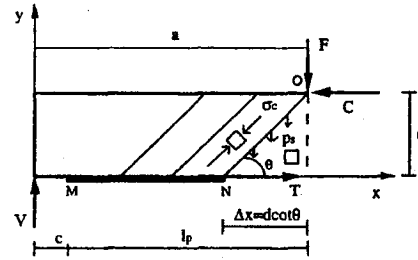


Fig. 2.10 Free body diagram for truss model concept (Spadea et al., 2000)

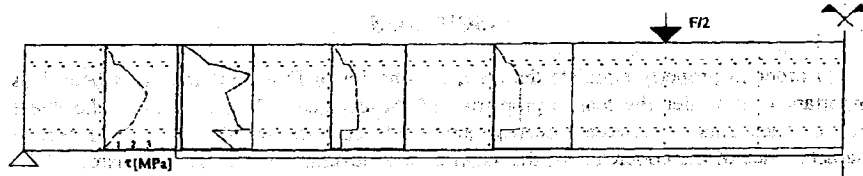


Fig. 2.11 Shear stress distribution in reinforced concrete beam strengthened with FRP sheets (Arduini et al., 2000)

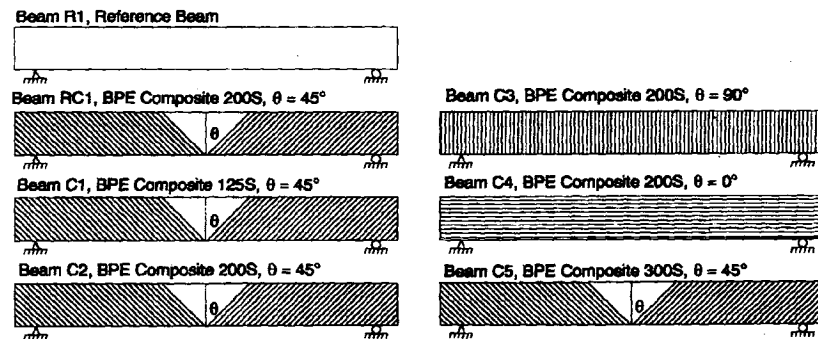


Fig. 2.12 Application of FRP sheet in the test beams for four point shear test (Taljsten, 2003)

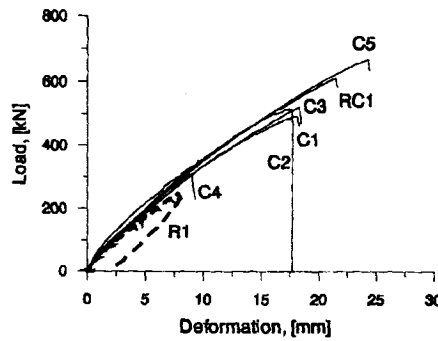


Fig. 2.13 Load deflection curves for concrete beams strengthened for shear (Taljsten, 2003)

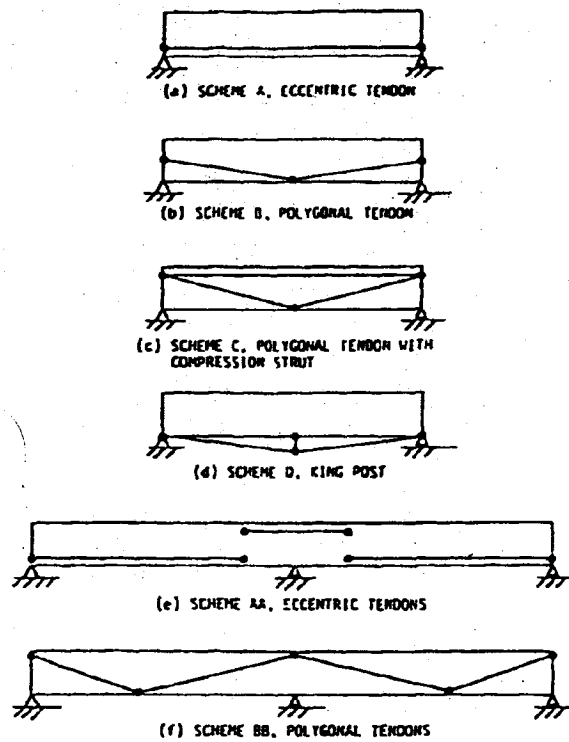


Fig. 2.14 External prestressing scheme for strengthening concrete girders (Klaiber et al., 1985)

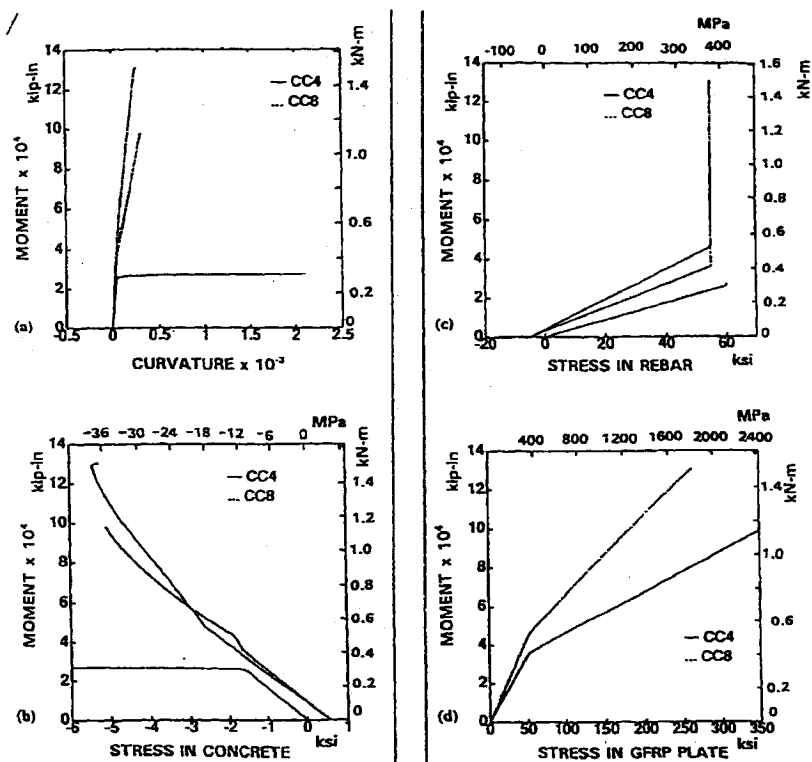


Fig. 2.15 Response of a T-Beam strengthened with non-prestressed CFRP sheets (Char et al., 1994)

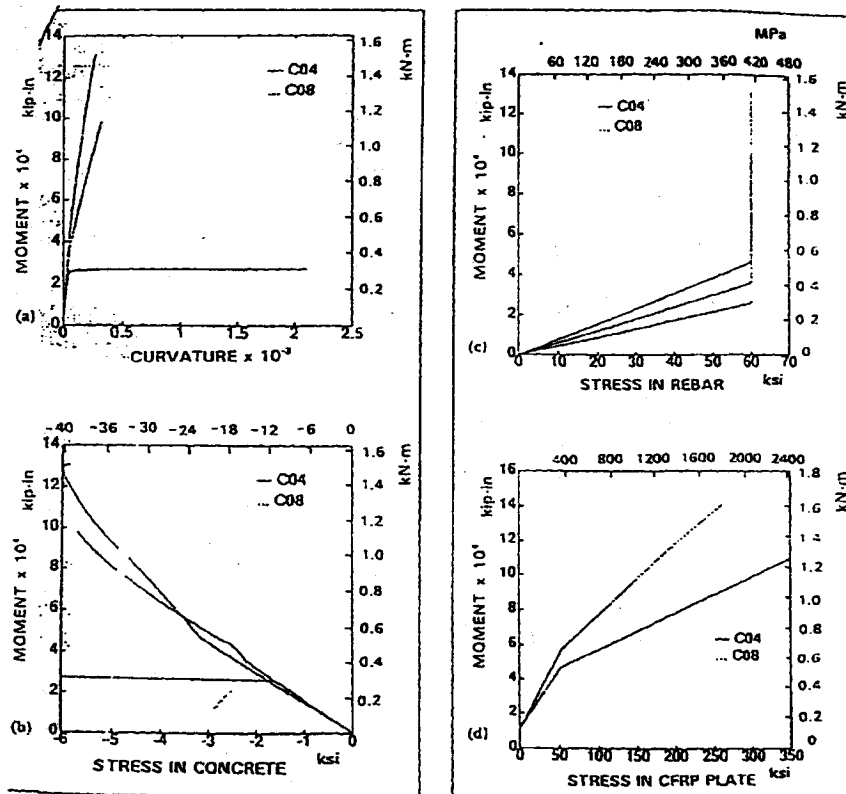


Fig. 2.16 Response of a T-Beam strengthened with prestressed CFRP sheets (Char et al., 1994)

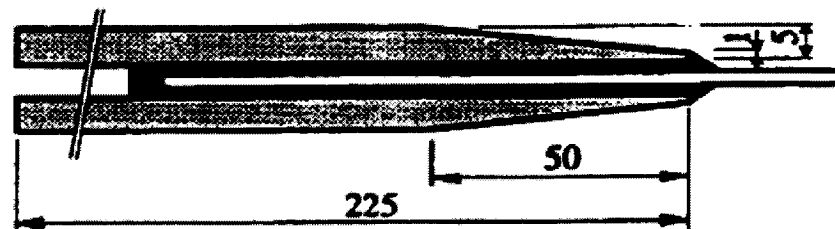


Fig. 2.17 Plate anchors for prestressing CFRP sheets (Deuring, 1993)

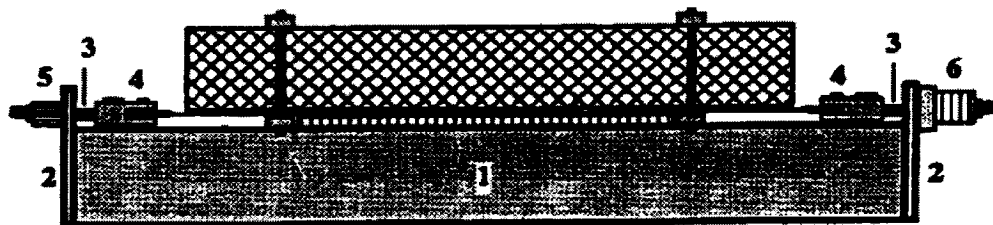


Fig. 2.18 External FRP prestressing system for medium scale concrete beams i.e. less than 2.4m (Deuring, 1993)

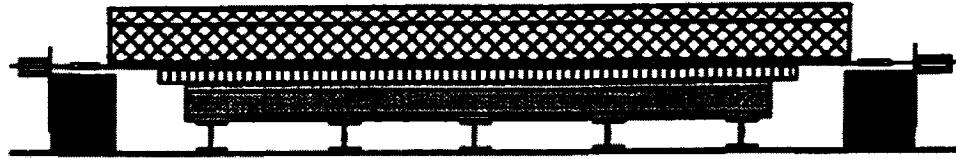


Fig. 2.19 External FRP prestressing system for large scale concrete beams i.e. less than 6.7m (Deuring, 1993)

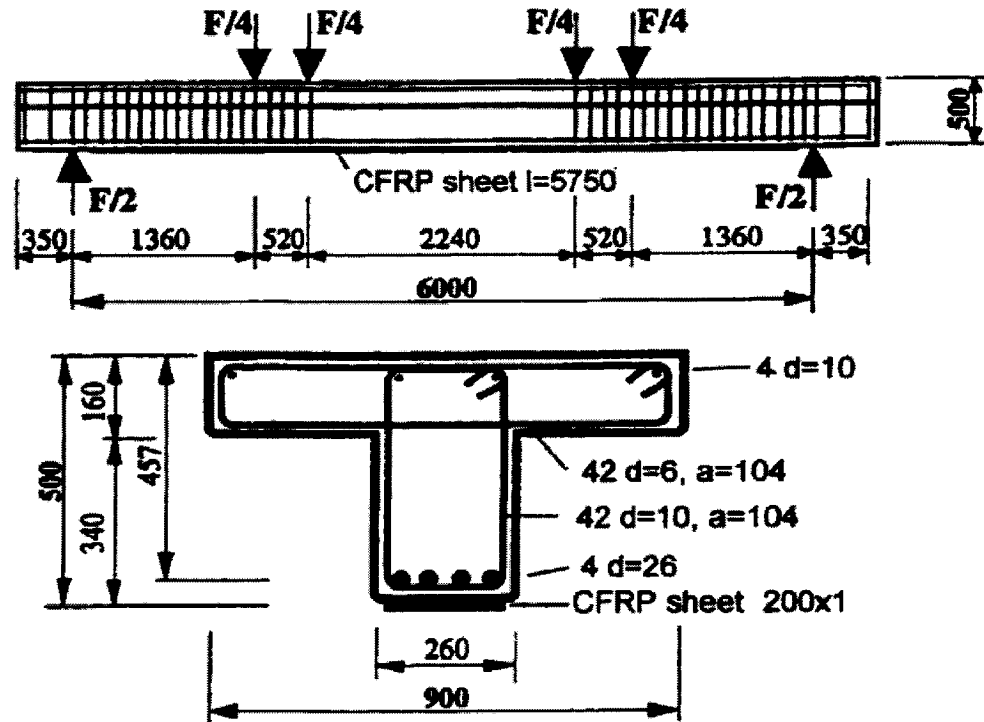


Fig. 2.20 Large size (6.7m) beam strengthened by prestressed CFRP (Deuring, 1993)

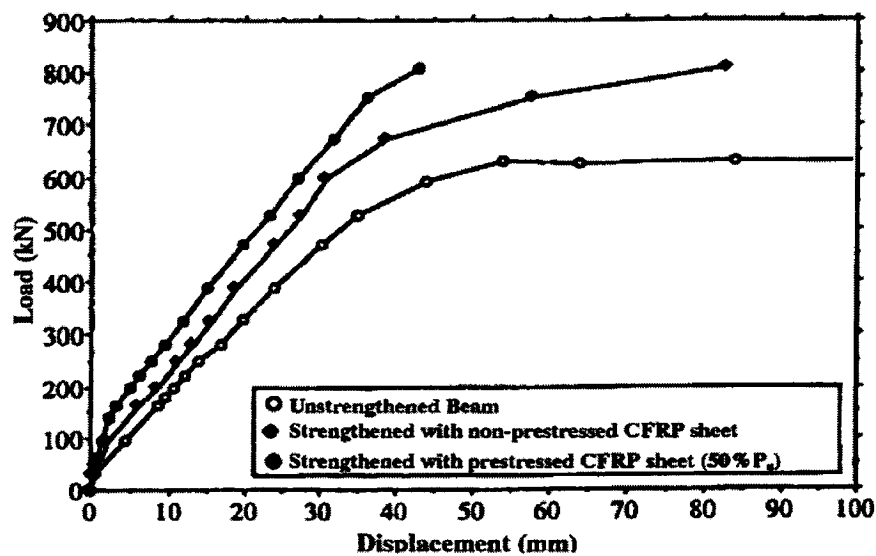


Fig. 2.21 Load displacement curves of large size (6.7m) beam strengthened by prestressed CFRP (Deuring, 1993)

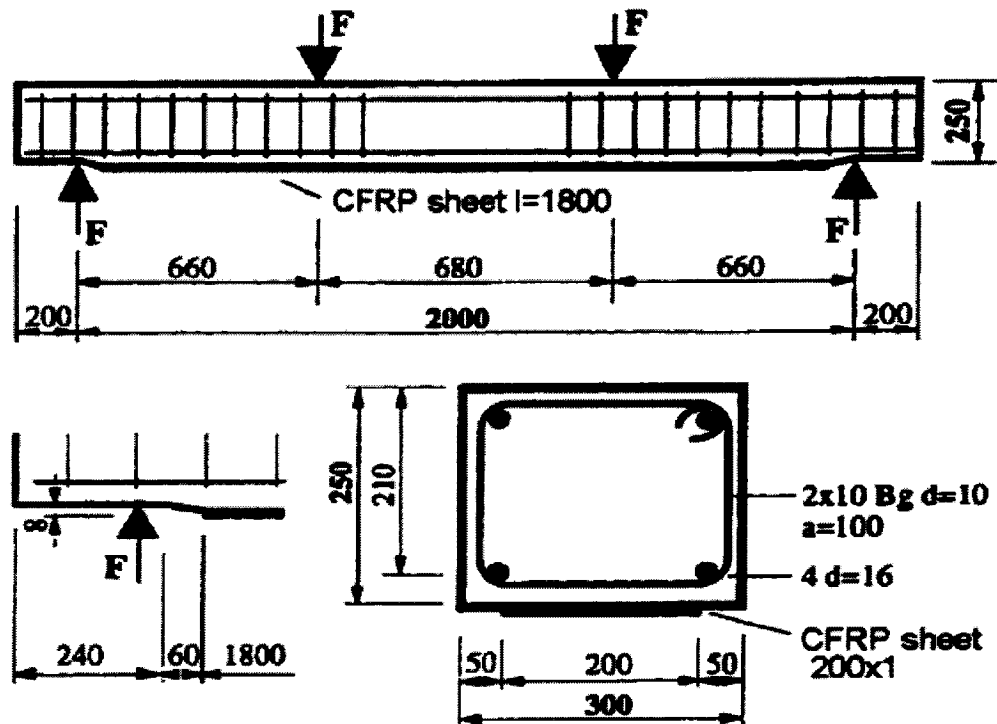


Fig. 2.22 Moderate size (2.4m) beam strengthened by prestressed CFRP (Deuring, 1993)

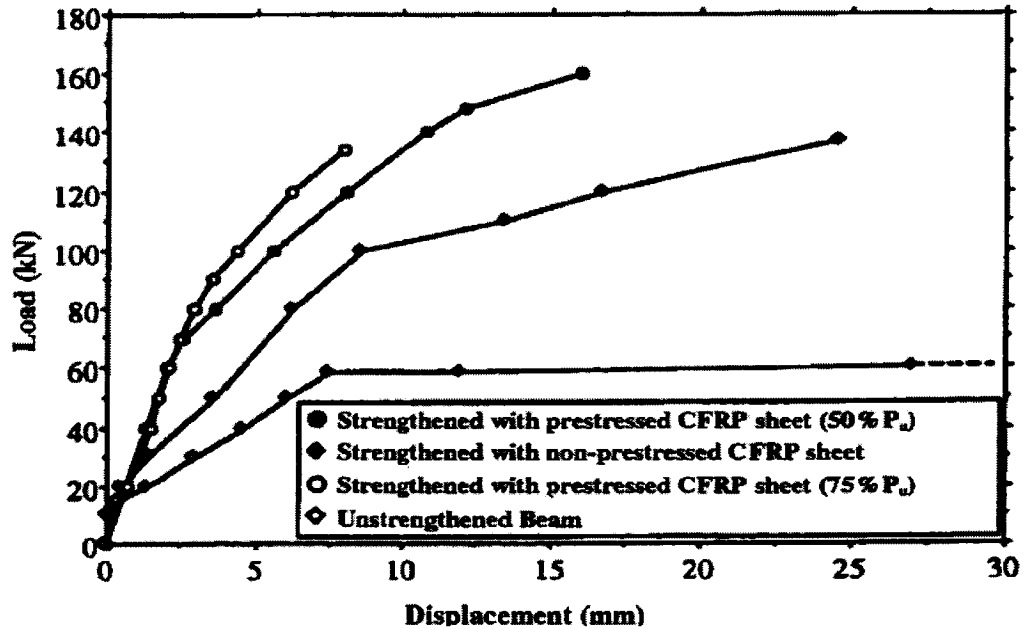


Fig. 2.23 Load displacement curves of moderate size (2.4m) beam strengthened by prestressed CFRP (Deuring, 1993)

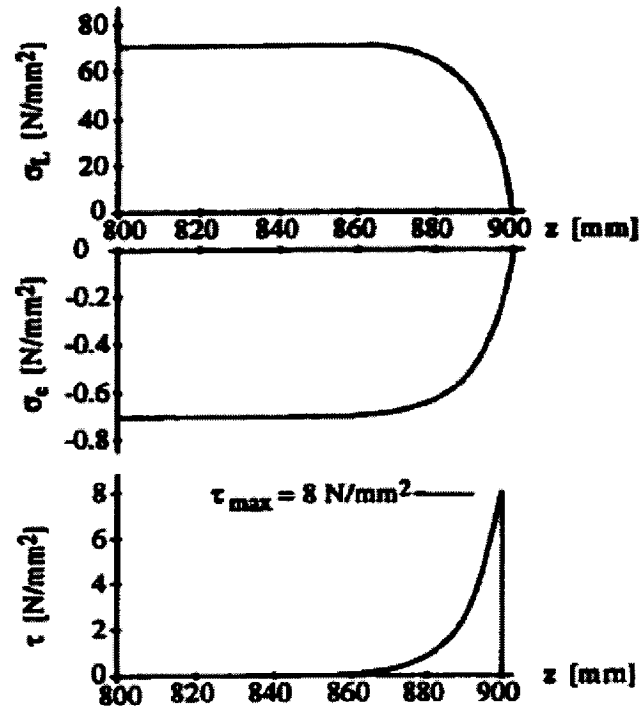
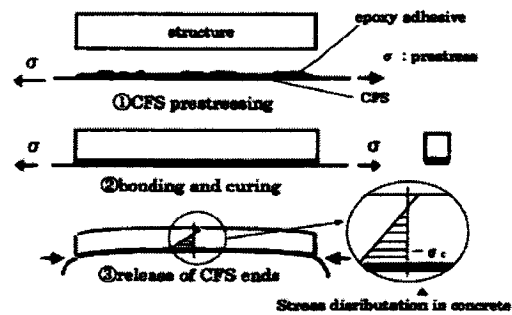
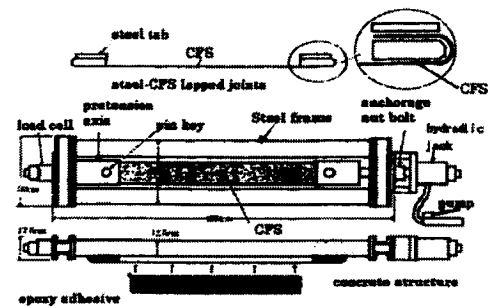


Fig. 2.24 Shear and normal stress distribution at the anchorage zone at transfer (Deuring, 1993)



(a) The concept of the strengthening method with CFS-prestressing



(b) Prestressing system

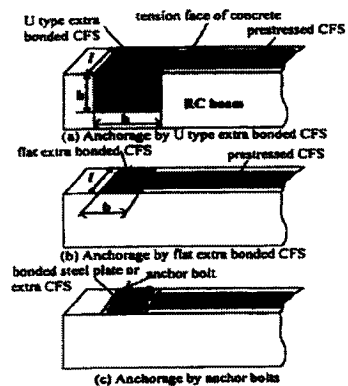
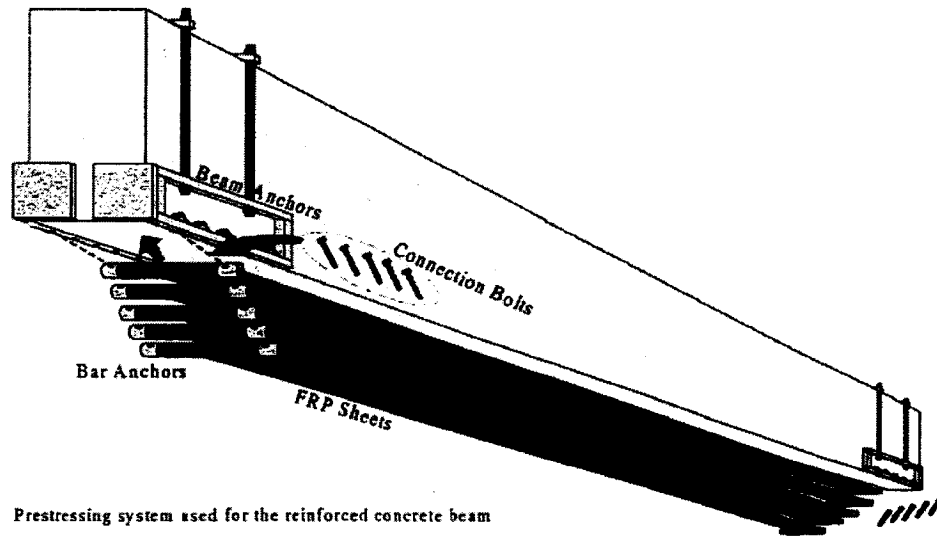
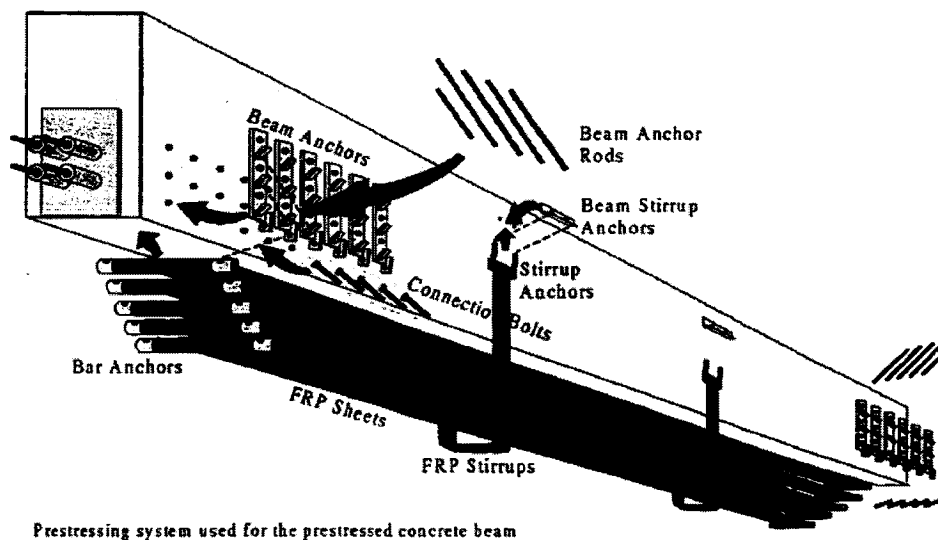


Fig. 2.25 The concept, prestressing system, and anchorage system used for prestressing CFRP sheet (Wu et al., 1999)



Prestressing system used for the reinforced concrete beam



Prestressing system used for the prestressed concrete beam

Fig. 2.26 Prestressing of CFRP sheet against the beams used for reinforced concrete and prestressed concrete beam (Wight et al., 2001)

CHAPTER 3

EXPERIMENTAL PROGRAM

3.1 General

This chapter describes experimental program used for investigation of flexural and shear behaviour of un-strengthened and strengthened reinforced concrete beams. Strengthening was done with either non-prestressed or prestressed GFRP sheets. The experimental procedure used for studying long term losses due to creep and shrinkage etc., associated with prestressing of GFRP sheet, is also described herein. The GFRP sheet can't be prestressed with conventional prestressing procedures. The sheet need end anchorage system for external prestressing. An innovative end anchorage system was developed and applied to prestress the GFRP sheet. The details of prestressing procedure are presented in this chapter. The ancillary testing procedures required for studying the constituent material properties is also part of this chapter. The objective of this experimental investigation was to examine the feasibility and effectiveness of the prestressed GFRP sheet to increase the flexural and shear capacity, and simultaneously improve the serviceability of the strengthened reinforced concrete beams. The effect of prestressed GFRP sheet on flexural and shear capacity of beam was also compared to the experimental results of beam strengthened with non-prestressed GFRP Sheet. In addition, the long term losses of prestressed GFRP sheet were also examined. All beams were subjected to a static monotonic load and were tested until failure. The properties of the constituent materials used in both flexural and shear deficient beams were determined from the ancillary testing.

3.2 Beams for Flexure and Shear Testing

Eight 2440mm long beams deficient in flexural reinforcement and fifteen 3000mm long beams deficient in shear reinforcement were constructed. Flexural reinforcement deficient beams were designed with two different tension steel ratios and were divided in two groups i.e. Group-1 and group-2. Sufficient shear reinforcement was provided to ensure flexural mode of failure. Shear reinforcement deficient beams were designed with sufficient tension steel reinforcement and minimum shear reinforcement to ensure shear failure. Beams deficient in flexural reinforcement will be called flexural beams and beams deficient in shear reinforcement will be called shear beam in subsequent sections of this chapter.

The aim of using two different tension steel ratios in flexural beams was to simulate the loss of reinforcement due to corrosion etc. Group-1 comprised of six beams with 3#15 bars, and Group-2 of two beams with 2#15 and 1#10 bars as tension reinforcement. Two level of prestress called P1 and P2 was induced in the GFRP sheet. P1 was equal to 32% and P2 was equal to 47% of the ultimate tensile strength of the GFRP sheet. In Group-1, one beam was kept un-strengthened which acted as control-1, one beam was strengthened with one layer of non-prestressed GFRP sheet (B1U), one with two layers of non-prestressed GFRP sheet (B2U), one with one layer of GFRP sheet prestressed to level P1 (B1P1), one with two layers of GFRP sheet prestressed to level P1 (B2P1), and one with one layer of GFRP sheet prestressed to level P2 (B1P2). In Group-2, one beam was kept un-strengthened which acted as control-2 and second beam was strengthened with three layers of GFRP sheet prestressed to level P1 (B3P1). All flexure beams were tested under four point bending over a clear span of 2200mm. The flexural beams were subjected to static monotonic load till failure and their behaviour was monitored.

Shear beams were subdivided into three groups on the basis of the shear span to effective depth (a/d) ratio used for testing of beams. Each group comprises of five beams. Three values of shear span to effective depth ratio, i.e. 2.5, 3.0, and 3.5 were used. In each group, one beam was kept un-strengthened which acted as control beam, one was strengthened with 100mm wide U Straps of GFRP sheet on three sides of the beam at 200mm centre to centre, one was strengthened with full jacket of GFRP sheet wrapped on three sides of the beams in the shear span only, one was strengthened with one layer of prestressed GFRP sheet, and one was strengthened with two layers of prestressed GFRP sheet bonded to the tension face of the beam and prestressed to 40% of the ultimate tensile strength of the GFRP sheet. In group of beams tested on shear span to effective depth ratio (a/d) of 2.5, two and three layers of prestressed GFRP sheets was used instead of one and two layers. One un-strengthened beam and one beam strengthened with three layers of prestressed GFRP sheet with a/d ratio of 2.5 were tested over clear span of 2800mm. The rest thirteen shear beams were tested under four point bending over a clear span of 2600mm. The shear beams were subjected to the static monotonic load till failure and their behaviour was monitored.

3.2.1 Flexure Beam Description

Eight rectangular reinforced concrete beams with cross sectional dimensions of 150x300mm and total length of 2440mm were constructed. The test variables were tension reinforcement ratio, number of layers of GFRP sheet, and the level of prestress applied into GFRP sheet. As mentioned before, two different ratio of tension reinforcement was provided to simulate the loss of tension reinforcement due to corrosion etc. A loss of 16% in the tension reinforcement i.e. reduction from 600 mm² to 500 mm² was adopted in this study. In one set of beams (Group-1) tension

reinforcement of 600mm^2 was provided by 3 No. 15 bars, and in the other set (Group-2) tension reinforcement of 500mm^2 was provided by 2 No. 15 plus 1 No. 10 bars. Compression reinforcement of 200mm^2 was provided by two No. 10 bars in all beams. Shear reinforcement consisted of No. 6 bars double leg stirrups at the centre to centre spacing of 120mm. The shear reinforcement was sufficient to ensure flexure failure. Grade 400 steel was used for tension, compression, and shear reinforcement. Clear cover of 40mm was provided to bottom and top reinforcement whereas clear cover of 25mm was provided to vertical or web reinforcement. The cross sections of Group-1 and Group-2 beams are shown in the Fig 3.1 and 3.2, respectively.

The reinforcement cage was placed in the forms constructed of 10mm ply wood and 50x100mm rectangular wooden planks. Photographs of the reinforcement of the flexure critical beams are shown in Fig. 3.3. Concrete for the beams was delivered by a ready mix truck. The concrete had a specified minimum strength of 35 MPa and a maximum aggregate size of 20mm. The slump of the concrete was 75mm and the air content was 3%. The concrete was consolidated by internal vibration and hand finished by hand screed. The concrete was covered by plastic sheets and moist cured for seven days after which the beams were taken out of the form.

3.2.2 Shear Beam Description

Fifteen rectangular reinforced concrete beams with cross sectional dimensions of 250x400mm and total length of 3000mm were constructed. The test variables were shear span to effective depth ratio, number of layers of prestressed GFRP sheet, type of externally bonded GFRP sheet shear reinforcement i.e. U straps or full sheet on three sides of the beam in shear span. The tension reinforcement of 2000mm^2 was provided was provided by 4 No. 25 bars. Compression reinforcement of 200mm^2 was provided by two No. 10 bars in all beams. Shear reinforcement consisted of No. 6

bars double leg stirrups at the centre to centre spacing of 175mm. Grade 400 steel was used for tension, compression, and Grade 250 was used for shear reinforcement. Clear cover of 40mm was provided to all reinforcement in the beams. The cross section of the typical shear beam is shown in the Fig 3.4.

The reinforcement cage was placed in the forms constructed of 16mm ply wood. Photographs of the reinforcement of the shear critical beams are shown in Fig. 3.5. Concrete for the beams was delivered by a ready mix truck. The concrete had a specified minimum strength of 35 MPa and a maximum aggregate size of 20mm. The slump of the concrete was 75mm and the air content was 3%. The concrete was consolidated by internal vibration and hand finished by hand screed. The concrete was covered by plastic sheets and moist cured for seven days after which the beams were taken out of the form.

3.2.3 Application of Non-Prestressed GFRP Sheet to the Beam

The strengthening system with externally bonded non-prestressed GFRP sheet is a two stage process, one is preparation of the specimen and other is application of the sheet. During the preparation stage, the concrete surface was cleaned with water in flexure specimens, whereas in shear specimens the concrete surface was grinded with compression air grinder and was cleaned with pressure water before the primer is applied. No primer was used in the flexure beams. During the application stage, alternating layers of epoxy and sheets was applied to the beam. All beams were inverted for the preparation of the concrete surface and application of GFRP sheet. In other words the bottom face of the beam was the top face during preparation and sheet application stages. The beams were returned to its normal orientation for testing.

In flexure specimens, all dust and laitance was removed from the surface of the beam by washing and air brushing. No primer coating was applied in the flexure

beams. In shear specimens, the surface of the strengthened beams was grinded with air pressure grinder so that the thin layer of the cement paste was removed and the coarse aggregate was exposed. It was then cleaned with water and air brushing. After beam surface get dried, the primer coating was applied. The purpose of the primer coating was to strengthen the concrete surface, provide a surface suitable for the binding of the fibre and fill the pores in the concrete. It also reduced the epoxy absorption of the concrete. The primer was allowed to cure before the first layer of the sheet was applied. The photo of the flexure beams strengthened with non-prestressed GFRP sheets are shown in the Fig. 3.6. Appropriate safety measures was taken through out the experimental procedure for the application of the primer and epoxy, including the use of face mask, protective gloves, goggles, and fan assisted ventilation.

The GFRP sheet used in this experimental program for the strengthening of the reinforced concrete beams was SEH 51 distributed by Fyfe Co. LLC “The Fibre Wrap Company”. The 1500mm wide sheet was consisted of unidirectional Glass fibres. The GFRP sheet had a manufacturer’s estimated elastic modulus of 26.1GPa, and an ultimate tensile strength of 575MPa with ultimate elongation of 2.2%. The properties of the GFRP sheet provided by the manufacturer are shown in Table 3.1. The GFRP sheets were bonded to the beam using a two part epoxy i.e. component A & B, supplied by the fibre manufacturer. The epoxy was mixed as per manufacturer recommended ratio and procedure just before application. The application rate of the epoxy mixture was approximately 1.20 Litres/m² for flexure critical beams and approximately 1.00 Litres/m² for shear critical beams. Primer coating was applied in the shear specimens only.

The epoxy resin was applied to the beam and spread with hand. The GFRP sheet was cut into required length and width with knife cutter and scissor. The GFRP sheet was applied to the beam in alternating layers of the epoxy resins and GFRP sheets. Hand pressure was applied to the sheet until it was smoothened and saturated with the epoxy. Entrapped air bubbles between the sheet and the concrete were removed by hand pressure in case of non prestressed sheet. In case of prestressed sheet air bubble were removed by injecting the epoxy with 60ml injection and needle. The process of application of GRRP sheet layers was repeated until the desire number of layers was attained. In the flexure beams, non-prestressed GFRP sheet was applied to the tension face of the beam, and in the shear beams, small non-prestressed U straps or full sheet U jacket in the shear span was applied. Photos of the beams strengthened with U-straps and U-jacket are shown in Fig. 3.7 and Fig. 3.8, respectively. The application of prestressed GFRP sheet to the tension face of the reinforced concrete beam is outlined in next section.

3.2.4 GFRP Sheet Prestressing System

All sheets were prestressed using an external prestressing technique. The external prestressing technique developed in this study has two stages: 1) anchor preparation; and 2) pre-stressing on pre-stress bed. The GFRP sheet was wrapped and bonded to a square hollow section as shown in Fig. 3.9. The sheet was prestressed on a prestressing bed with 15mm treaded steel bars in case of flexure specimens and with hydraulic jack in case of shear specimens. The photo in Fig. 3.10 and 3.11 showed the prestressing of flexure and shear specimens.

The sheet anchor which was used for prestressing the GFRP sheet in both flexure and shear beams consisted of square hollow steel section. The square hollow section was 50x50x5mm and 500mm long. The 1st stage of anchor preparation involved

cutting of sheet in required size and its bonding to the sheet anchors. Plastic sheets were wrapped around sheet anchor before bonding the GFRP sheet, to avoid bonding of the GFRP sheet to the steel sections. The GFRP sheet was then impregnated with two part epoxy resin and bonded to the square hollow section at each end of the sheet. The sheet was wrapped 2 times around the square hollow section and cured for 48 hours at room temperature. After curing of epoxy, another similar square hollow steel section was attached to the bottom face with two 10mm steel bolts. Attachment of the other hollow section helped to apply the concentric axial prestressing load to the GFRP sheet and prevented sheet unwinding during prestressing. In 2nd stage, the CFRP sheet was tension on the prestressing bed. Two types of pre-stressing beds were used for flexure and shear beams. The basic mechanism of post tensioning was the same but in flexure beams the prestressing force was applied manually through steel threaded bars and in shear beams through hydraulic jacks.

The pre-stressing bed used for flexure beams consisted of two end supports mounted on a heavy steel I-section. At one support the sheet anchors was fixed with four number 15mm steel bolts, whereas at the other end the sheet anchors was attached to a load cell and supports with the help of four number 15mm threaded steel bars. A small Initial force was applied to the GFRP sheet by tightening the steel bars in such a way to get uniform strain along the width of the sheet. The sharp edges of the beams were grinded to get smooth and rounded edges. The tension face of the beam was cleaned with compressed air and water. The beam was placed on the prestressing bed in such a way that its tension face was at the top. The level was adjusted such that the sheet just touched the beam surface. The epoxy was applied to tension face of the beam and GFRP sheet before application of the prestressing force. Post-tensioning is then carried out manually by tightening the screws on the threaded

bars. The prestressed load was monitored with load cell and strain in the sheet was monitored with strain gauges mounted on the GFRP sheet.

The pre-stressing bed used for shear beams consisted of two end supports mounted on strong floor of the structure laboratory. At one support, sheet anchor was fixed with four number 15mm steel bolts, whereas at the other end the sheet anchor was attached to a load cell, hydraulic jack and support with the help of four number 15mm steel threaded bars. A small Initial force was applied to the GFRP sheet by tightening the steel bars in such a way to get uniform strain along the width of the sheet. The tension face and sharp edges of the beams were grinded with steel bar air compressor to remove the loose particles and thin layer of cement paste. It was then cleaned with compressed air and water. The epoxy primer was applied to the beams in order to strengthen the concrete surface, provide a surface suitable for the binding of the fibre and fill the pores in the concrete. The beam was placed on the prestressing bed in such a way that its tension face was at the top. The level of the beam was adjusted such that the sheet just touched the beam surface. Before application of the prestressing force, epoxy was applied to tension face of the beam and GFRP sheet. Post-tensioning was then carried out through hydraulic jack. The screws on the threaded bars were tightened at the end of the post tensioning to retain the prestressing force in the sheet. The prestressed load was monitored with load cell attached to end of hydraulic jack.

After application of prestressing force to GFRP sheets, it was separated from the beam surface at few locations. This was caused due to imperfections in the tension face of the strengthened beam. To bring GFRP sheet into contact with the beam surface and remove the entrapped air more epoxy was injected into the strengthened beam. In few beams vertical load perpendicular to the beam surface was applied to get perfect bond between sheet and beam.

3.2.5 Flexural Beams Strengthening Details

Eight reinforced concrete beams were constructed for flexure strengthening study. Two types of beams based on reinforcement ratios were constructed to simulate the loss of flexural reinforcement due to corrosion etc. Sufficient shear reinforcement was provided in all beams to ensure the flexural mode of failure. One reinforced concrete beam from each group was not strengthened and was used as controls. All other beams were strengthened with either prestressed or non-prestressed GFRP sheets bonded to the flexure face of the beam. Two levels of prestress were used in this study. The beams were strengthened with one, two, or three layers of the GFRP sheets. Details of the strengthening system applied to various beams are shown in Table 3.2. The study of sheet delamination and sheet anchorage system was not part of this investigation. The anchorage adopted in this study was the continuation of the GFRP sheet under the support which is shown in Fig. 3.12. This method of anchorage was used for all beams strengthened in flexure with prestressed and non-prestressed GFRP sheets. In this anchorage system, a normal force equal to applied shear force was generated against the GFRP sheet which provided the greatest possible anchorage force without the use of an externally fitted anchorage system.

3.2.6 Shear Beams Strengthening Details

Fifteen reinforced concrete beams were constructed for shear strengthening study. Minimum shear reinforcement with sufficient flexure steel was provided in all beams to ensure shear mode of failure. The beams were divided and tested in three groups on the basis of shear span to effective depth ratios. Each group contain five beams. One reinforced concrete beam from each group was not strengthened and was used as controls. Two beams in each group were strengthened with prestressed GFRP sheet bonded to the flexure face of the beam. One beam from each group was strengthened

with U-strap and one with full sheet U-jacket bonded to two vertical faces and bottom face of the beam. The influence of prestressed sheet bonded to the flexure face of the beam, U strap, and U jacket, on the shear capacity of the strengthened beams were experimentally investigated in this study. One level of prestress was used in this study. The beams were strengthened with one, two, or three layers of prestressed GFRP sheets or single layer U-straps or single layer U-jackets. Details of the strengthening system applied to various beams are shown in Table 3.3. No anchorage system was used in beams strengthened with U straps or U jackets. The anchorage used for the beams strengthened with prestressed GFRP sheet was the continuation of the GFRP sheet under the support. In this anchorage system, a normal force equal to applied shear force was generated against the GFRP sheet which provided the greatest possible anchorage force without the use of an externally fitted anchorage system.

3.2.7 Instrumentation

In flexure beams, displacement at five points along the length of the beam was monitored by 10 linear voltage displacement transducers (LVDTs), as shown in Fig. 3.13. The LVDTs were mounted on steel rods on both side of the beam. Five copper rods having 1mm diameter were passed through every two LVDTs on either side of the beam. The rubber bands were used to keep the copper rods attached to the bottom face of the beam. LVDTs were placed on both side of the beam at mid span of the beam, 550mm from the supports, and at two load points i.e. 850mm from supports.

Strains in the beam were monitored by strain gauges mounted on reinforcement in the beams as shown in the Fig. 3.14. In all flexure beams strain gauges were fixed to the tensile reinforcement at mid span and at 850mm from the supports. On two control beams strain gauges were also mounted on compression reinforcement at mid span. The wires from all the internal strain gauges exited at one end of the beam.

Strains in the GFRP sheets during the test were monitored by strain gauges. Strain gauges were mounted GFRP on sheet at mid span and at 250mm on either side from mid span as outlined in Fig. 3.15.

In shear beams, displacement at nine points along the length of the beam was monitored by 9 linear voltage displacement transducers (LVDTs), as shown in Fig. 3.16. The LVDTs were mounted on angle iron section under the beam at 0.1L spacing. The displacements at the centre line on the bottom face of the beam were recorded by 9 LVDTs.

Strains in the beam were monitored by strain gauges mounted on steel reinforcement in the shear beams as shown in the Fig. 3.17. In all shear beams strain gauges were fixed to the tensile reinforcement at mid span and at five stirrups on one side of beam. The wires from all the internal strain gauges exited at one end of the beam. In beams strengthened with prestressed GFRP sheet, strains in the GFRP sheets were monitored by strain gauges as outlined in Fig. 3.18. Twenty two strain gauges were mounted on GFRP sheet along the length of the beam to monitor the distribution of the strain. In beams strengthened with U-straps and U-jackets of GFRP sheet, strains in the GFRP sheet were monitored by strain gauges as out lined in Fig. 3.19 and Fig. 3.20 respectively.

3.2.8 Testing

The flexure beams were tested in the structural laboratory of Carleton University, Ottawa. The beams were subjected to four points loading as shown in Fig. 3.21. The beams were tested over a clear span of 2200mm with shear span of 850mm and 500mm constant moment region. The beams were mounted on two roller supports. At one end, the support was free to roll, and, at the other end, the support was restrained to provide stability to the system. The load was applied to the beam by a loading

actuator controlled through a MTS controller. The total load was transferred to two equal loading points through a stiffened steel beam, two rollers and bearing plates. A photograph of flexural beam testing assembly is shown in Fig. 3.22. The actuator, LVDTs and strain gauges were attached to data acquisition system. All channels were monitored every 10 seconds by the data acquisition system. The beams were loaded to failure. The cracks were marked and measured with the help of microscope at different loading stages.

The shear beams were tested in the structural laboratory of Hong Kong University of Science and Technology (HKUST), Hong Kong. The beams were subjected to four points loading as shown in Fig. 3.23. The control beam and beam strengthened with three layers of prestressed GFRP sheet with shear span to effective depth ratio of 2.5 were tested over a clear span of 2800mm. All other beams were tested over a clear span of 2600mm with various shear spans to effective depth ratio i.e. 2.5, 3.0, and 3.5. The shear span of 875mm, 1050mm, and 1225mm was used in three groups of the shear beams. The beams were mounted on two roller supports. At one end, the support was free to roll, and, at the other end, the support was restrained to provide stability to the system. The load was applied to the beam by a loading actuator controlled through a MTS controller. The total load was transferred to two equal loading points through a stiffened steel beam, two rollers and bearing plates. A photograph of shear beam testing assembly is shown in Fig. 3.24. The actuator, LVDTs and strain gauges were attached to data acquisition system and monitored every 10 seconds by data acquisition system. The beams were loaded till failure.

3.3 Long Term Losses Specimens

In the present study, long term losses due to creep and shrinkage under sustained loads were investigated experimentally. Losses due to other factors like relaxation of

fibre, elastic shortening of GFRP sheet due to multiple layers application etc. was lumped together and investigated experimentally. For investigating the long term losses due to shrinkage and creep, test as per ASTM standard were carried out. The shrinkage tests were carried out on Plain Cement Concrete (PCC) beams and the creep test were conducted on concrete cylinders. In addition to shrinkage and creep test specimens, three reinforced concrete beams were constructed to investigate the total losses due to application of the prestressed GFRP sheet.

3.3.1 Description of Shrinkage Beams

The beams were PPC with cross sectional dimensions of 100x100mm and 500mm length. Demarcation gauges were mounted on the opposite faces of the shrinkage beams. The shrinkage specimens were placed in the laboratory at control temperature and humidity conditions. The shrinkage beam and position of demarcation gauges are shown in Fig. 3.25. The strain data was recorded as per ASTM standard.

3.3.2 Description of Creep Test Cylinders

To investigate the creep losses, three concrete cylinders having 150mm diameter and 300mm height were constructed. After wet curing for seven days the concrete cylinders were kept at room temperature for 28 days. After 28 days of casting, the concrete cylinders were placed in the creep testing frame under a sustain load of 10kN. The load level in the creep testing frame resembles the axial load induced into reinforced concrete beams by prestressed GFRP sheets. Demarcation gauges were mounted and strain reading were recorded as per ASTM C 512-87 (Re-approved 1994). The photo of the creep testing assembly is shown in Fig. 3.26.

3.3.3 Description of Total Long Term Losses Beams

To investigate the total long term losses due to externally bonded prestressed GFRP sheet, three beams, with cross-sectional dimension of 100x100mm and 500mm

length, reinforced with one 15mm diameter bars were constructed. The beams were strengthened by bonding two layers of prestressed GFRP sheets to the tensile face of the beams. The GFRP sheet was prestressed to 45% of the ultimate tensile strength of the GFRP sheet. After curing of the epoxy, the prestressing was released and transferred to the beam. The strain gauges and demarcation gauges were mounted on the bonded GFRP sheet. The strains data, from the strain gauges and demarcation gauges, were recorded at the same time when strain data were recorded in the creep and shrinkage specimens. The photo of the total long term losses specimens is shown in Fig. 3.27.

3.4 Ancillary Tests

Ancillary tests were conducted on all constituent materials used in this experimental investigation i.e. concrete, steel reinforcement, and the GFRP sheets and laminates, in order to determine the constituent material properties. Standard concrete cylinders were tested to determine the compressive strength of the concrete. Tensile tests were performed on the all steel reinforcement to determine the stress-strain behaviour of the steel. Tensile tests were also performed on the GFRP sheets and laminate to determine the mechanical properties and stress-strain behaviour of the sheet and laminate.

3.4.1 Concrete Tests

During casting of the concrete flexure and shear beams, standard cylinders 150mm in diameter and 300mm in height, were cast to determine the uniaxial compressive strength and the splitting tensile strength of the concrete. The cylinders were cured for seven days, and then removed from the forms, concurrent with the flexure and shear beams. All concrete cylinders were at least 28 days old at the time of testing. For each

type of test, a minimum of three cylinders were tested and the average value of all three test specimens was calculated.

During casting, the slump of the concrete was determined in accordance with CSA Standard A23.2-5C (CSA, 1994) “Slump of Concrete”. The concrete cylinders were consolidated by blows of steel rod in accordance with CSA standard A23.2-3C (CSA, 1994) “Making and Curing Concrete Compression and Flexure Test Specimens”. Uniaxial compression tests were conducted in accordance with the CSA Standard A23.2-9C (CSA, 1994) “Compression Strength of Cylindrical Concrete Specimens”. The compressive strength of the concrete was determined at the time of the flexure and shear beams testing.

3.4.2 Steel Reinforcement Tensile Tests

Five different types of reinforcing steels were used in the construction of flexure and shear beams in this study. Uniaxial tension tests were performed on each type of reinforcing steel to determine stress-strain behaviour. Specimens for each type were cut with a length equal to 600mm. The bars were sanded and smoothed at mid length so as to have a reduced cross section to localize the zone of yielding and fracture and to provide smooth surface for strain gauges mounting. Tests were conducted in accordance with the ASTM A370-96 “Standard Test Methods for Mechanical Testing of Steel Products”. Two strain gauges were attached to either side of the reduced cross section and were aligned parallel to the direction of the applied force. The steel specimens were tested using a 250kN capacity MTS universal tension testing machine. Readings of strains and applied tension loads were collected using a data acquisition system connected to a personal computer. The tests were conducted under displacement control mode at a constant loading rate of 1mm/min, and the readings

were scanned every 5 seconds. The applied load, the strain gauge and the stroke displacement data were monitored during the test.

3.4.3 GFRP Tension Tests

The tension tests on GFRP sheets and laminates were conducted to investigate the ultimate tensile strength, rupture strain and modulus of elasticity. The tension test on the GFRP sheet was conducted with the aim to find out the approximate maximum prestressing stress. The GFRP sheet specimens were continuous strips, approximately 1500mm long and 100mm wide. The GFRP sheet was wrapped around the square hollow sections at the sheet ends to provide anchors. Additional square hollow section were attached at each end to prevent unwinding of sheet during test and helped in applying concentric loading. A strip of epoxy was placed at mid length where strain gauges were attached. The strain gauges were placed at third length across the width of the sheet. The test specimen was tested in a 100kN MTS universal testing machine. Readings of strains and applied tension loads were collected using a data acquisition system connected to a personal computer. The tests were conducted under displacement control mode at a constant loading rate of 1mm/min, and the readings were scanned every 5 seconds. The applied load, the strain gauge and the stroke displacement data were monitored during the test.

The modulus of elasticity and an approximate maximum tensile strength of GFRP laminate were estimated from tensile tests on coupons of GFRP laminate. The coupons of GFRP laminate were prepared and tested in tension in accordance with draft ISO Standard. To prepare the coupons of GFRP laminate, the GFRP sheet was cut in 300mm length and 300mm width. The two part epoxy, which was used in flexure and shear beams, was mixed. The weight of the epoxy resin used for preparation of test coupons was equal to weight of the GFRP sheet. Plastic separation

film was laid on the glass surface and the GFRP sheet was put on it. The epoxy resin was applied by hand and efforts were made to get bubble free surface. The sheet was covered by another plastic separation film and was pressed by edge of steel ruler to get uniform thickness and remove the excess epoxy. The specimen was cured for 72 hours at room temperature before cutting in 10 mm width and 200 mm length strips. A diamond cutter was used for cutting the test coupons. Aluminium anchors 10 mm wide and 40 mm length were glued to the end of each specimens. Strain gauge was attached to the middle of the specimen. The testing coupons of GFRP laminates are shown in Fig 3.28. The specimen was tested with a MTS testing machine with 25kN load capacity. Readings of strains and applied tension loads were collected using a data acquisition system connected to a personal computer. The tests were conducted under displacement control mode at a constant loading rate of 1mm/min, and the readings were scanned every 5 seconds. The applied load, the strain gauge and the stroke displacement data were monitored during the test. The test set up of tensile test of GFRP laminates are shown in Fig 3.29.

Table 3.1 GFRP sheet properties (Tyfo SHE-51 Composite)

Property	ASTM test method	Magnitude
Fibre Tensile Strength		3.24 GPa
Fibre Tensile Modulus		72.4 GPa
Fibre Ultimate Elongation		4.5 %
Fibre Density		2.55 g/cm ²
Fibre Weight per Sq. meter		915 g/m ²
Fibre Thickness		0.36 mm
Composite Laminate Tensile strength	D-3039	575 MPa
Composite Laminate Elongation at break	D-3039	2.2 %
Composite Laminate Tensile Modulus	D-3039	26.1 GPa
Composite Laminate Thickness		1.3 mm
Epoxy Resin Tensile Strength	D-638 (Type 1)	72.4 MPa
Epoxy Resin Tensile Modulus	D-638 (Type 1)	3.18 GPa
Epoxy Resin Elongation Percentage	D-638 (Type 1)	5 %

Table 3.2 Flexure beams strengthening details

Specimen name	Tension Rebar	FRP sheet details	Level of prestress
Control 1	3 Nos. 15	None	None
B1U	3 Nos. 15	1 Layer of sheet	Non-prestressed
B2U	3 Nos. 15	2 Layers of sheet	Non-prestressed
B1P1	3 Nos. 15	1 Layer of Sheet	Prestressed to level P1*
B2P1	3 Nos. 15	2 Layers of Sheet	Prestressed to level P1
B1P2	3 Nos. 15	1 Layer of Sheet	Prestressed to level P2**
Control 2	2 Nos.15 + 1 No 10	None	None
B3P1	2 Nos.15 + 1 No 10	3 Layers of sheet	Prestressed to level P1

* Prestressed to 30% of the ultimate strength of the FRP sheet

** Prestressed to 45% of the ultimate strength of the FRP sheet

Table 3.3 Shear beams strengthening details

Specimen name	Strengthening system	FRP sheet detail attached to tension face of the beam
<u>a/d = 2.5, Shear span = 875mm</u>		
Control2.5	None	None
BU2.5	U-straps in shear spans *	None
BJ2.5	U-jacket in shear spans	None
B2P2.5	Prestressed sheet **	2 Layers of prestressed Sheets
B3P2.5	Prestressed sheet **	3 Layers of prestressed Sheets
<u>a/d = 3.0, Shear span = 1050mm</u>		
Control3.0	None	None
BU3.0	U-straps in shear spans *	None
BJ3.0	U-jacket in shear spans	None
B1P3.0	Prestressed sheet **	1 Layer of prestressed Sheet
B2P3.0	Prestressed sheet **	2 Layers of prestressed Sheets
<u>a/d = 3.5, Shear span = 1225mm</u>		
Control3.5	None	None
BU3.5	U-straps in shear spans *	None
BJ3.5	U-jacket in shear spans	None
B1P3.5	Prestressed sheet **	1 Layer of prestressed Sheet
B2P3.5	Prestressed sheet **	2 Layers of prestressed Sheets

* 100mm wide U-strap attached to two vertical and bottom faces of the beam at 200mm c/c

** Prestressed to 45% of the ultimate strength of the FRP sheet

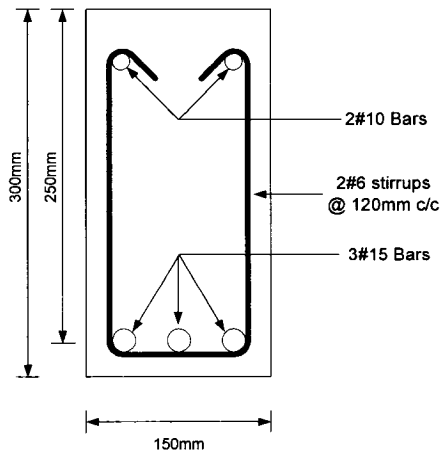


Fig. 3.1 Cross-section of the flexure Beam (Group-1)

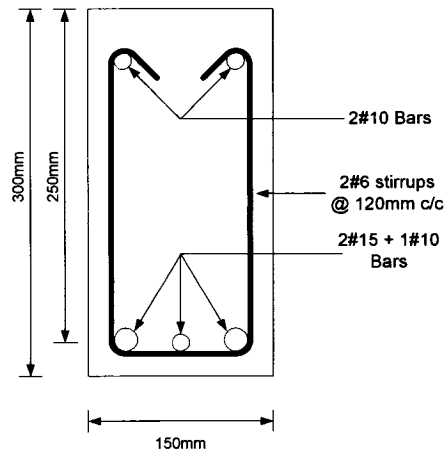


Fig. 3.2 Cross-section of the flexure Beam (Group-2)



Fig. 3.3 Flexure beam reinforcement

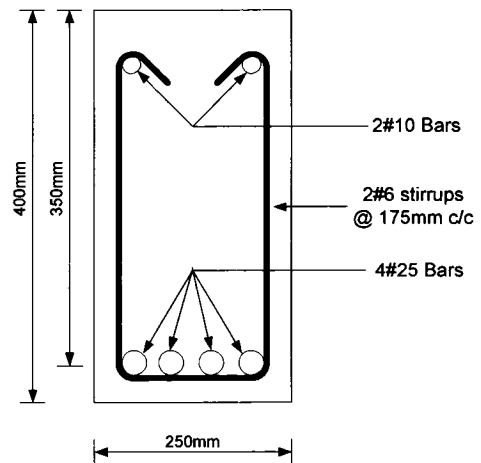


Fig. 3.4 Cross-section of the shear beam

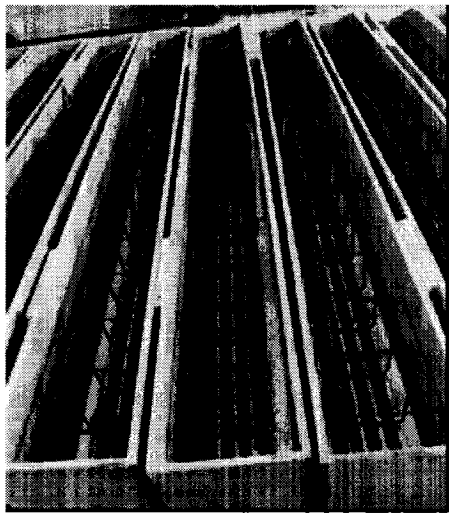


Fig. 3.5 Shear beam reinforcement



Fig. 3.6 Flexure beam strengthened with one and two layers of non-prestressed sheet



Fig. 3.7 Shear beam strengthened with U-straps



Fig. 3.8 Shear beam strengthened with U-Jackets

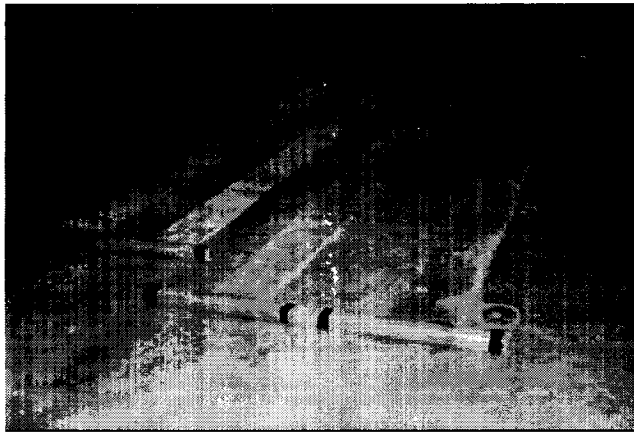
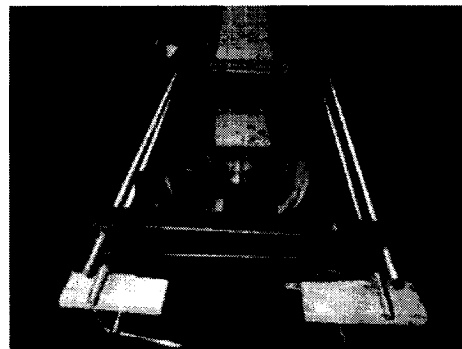


Fig. 3.9 Sheet anchor wrapped around square hollow steel section



a) Application of prestressed sheet

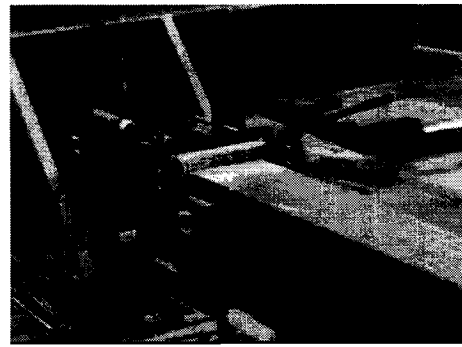


b) Prestressing with threaded bars

Fig. 3.10 Prestressing technique of GFRP sheet used for flexure beam



a) Application of prestressed sheet



b) Prestressing with hydraulic jack

Fig. 3.11 Prestressing technique of GFRP sheet used for shear beam

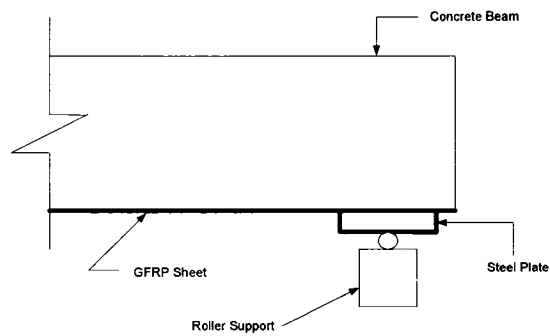


Fig. 3.12 Anchorage detail i.e. continuation of GFRP sheet over the support

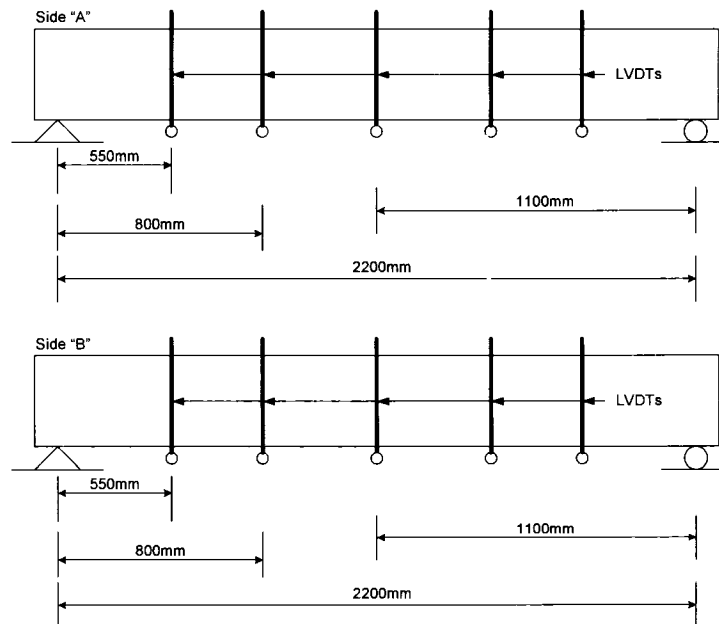


Fig. 3.13 Detail of LVDTs positions in flexure beams

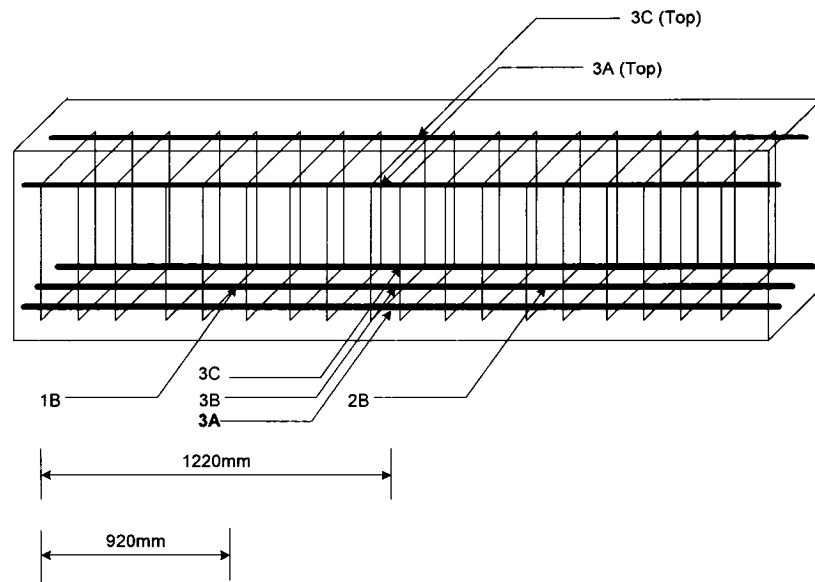


Fig. 3.14 Detail of strain gauges mounted on steel reinforcement in flexure beams

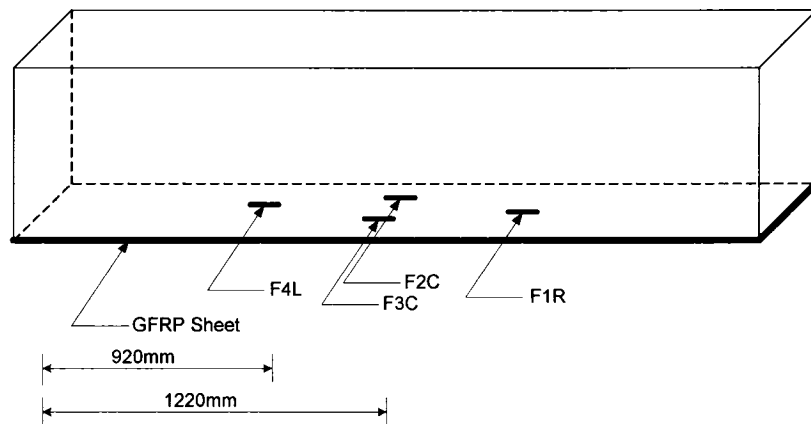


Fig. 3.15 Details of strain gauges mounted on GFRP sheet in flexure beam

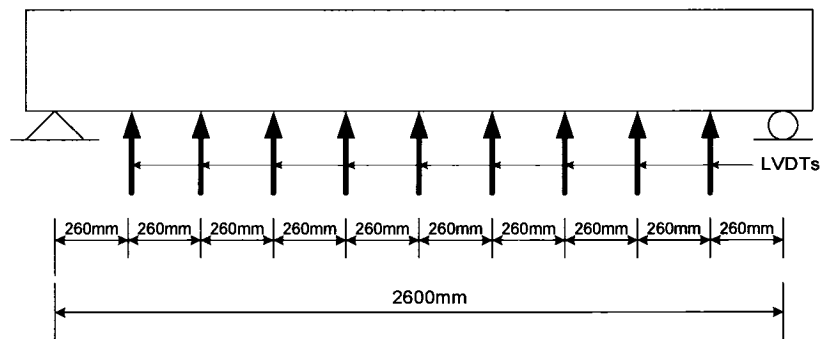


Fig. 3.16 Details of LVDTs positions in shear beam

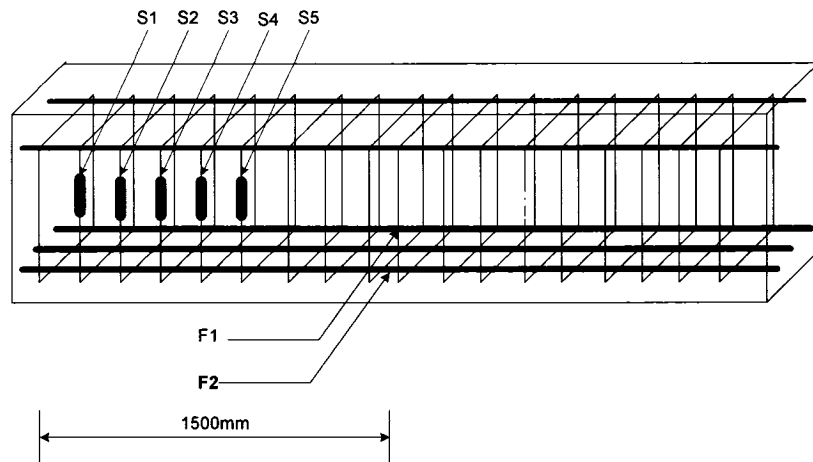


Fig. 3.17 Details of strain gauges mounted on steel reinforcement in shear beam

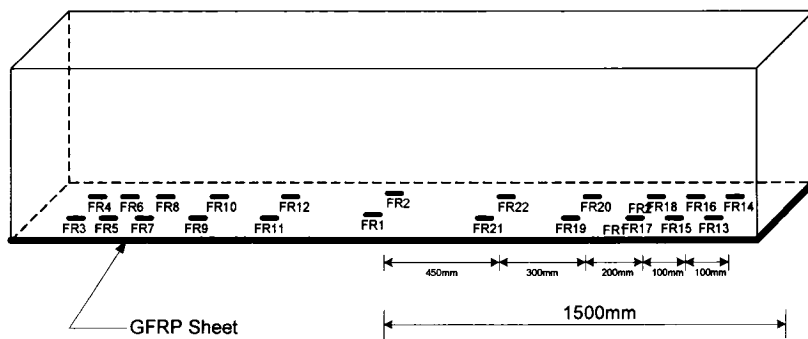


Fig. 3.18 Details of strain gauges mounted on Prestressed GFRP sheet in shear beam

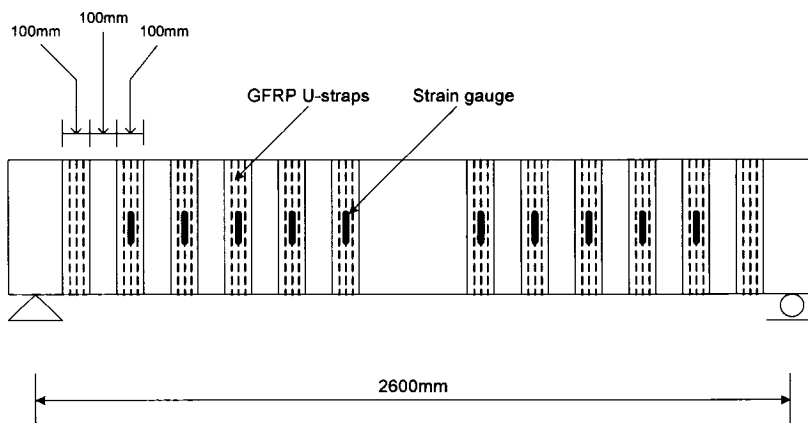


Fig. 3.19 Details of strain gauges mounted on U-straps of GFRP sheet in shear beam

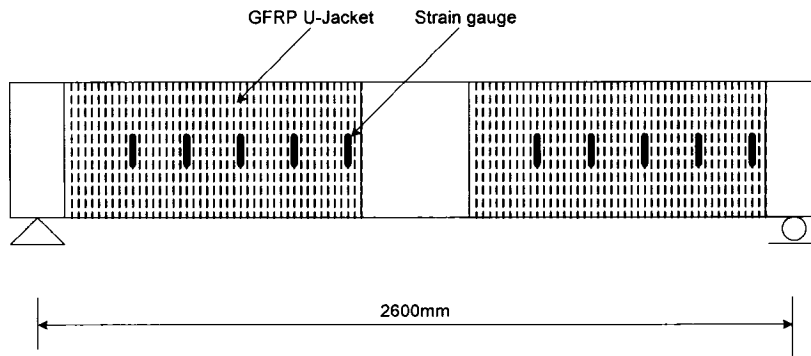


Fig. 3.20 Details of strain gauges mounted on U-jacket of GFRP sheet in shear beam

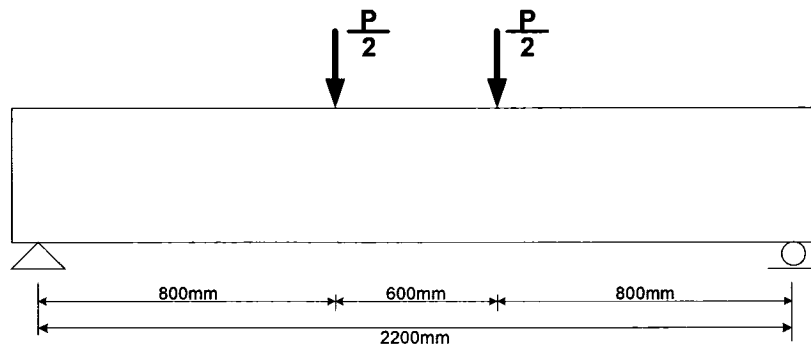


Fig. 3.21 Flexure beam testing plan



Fig. 3.22 Flexure beam test set up

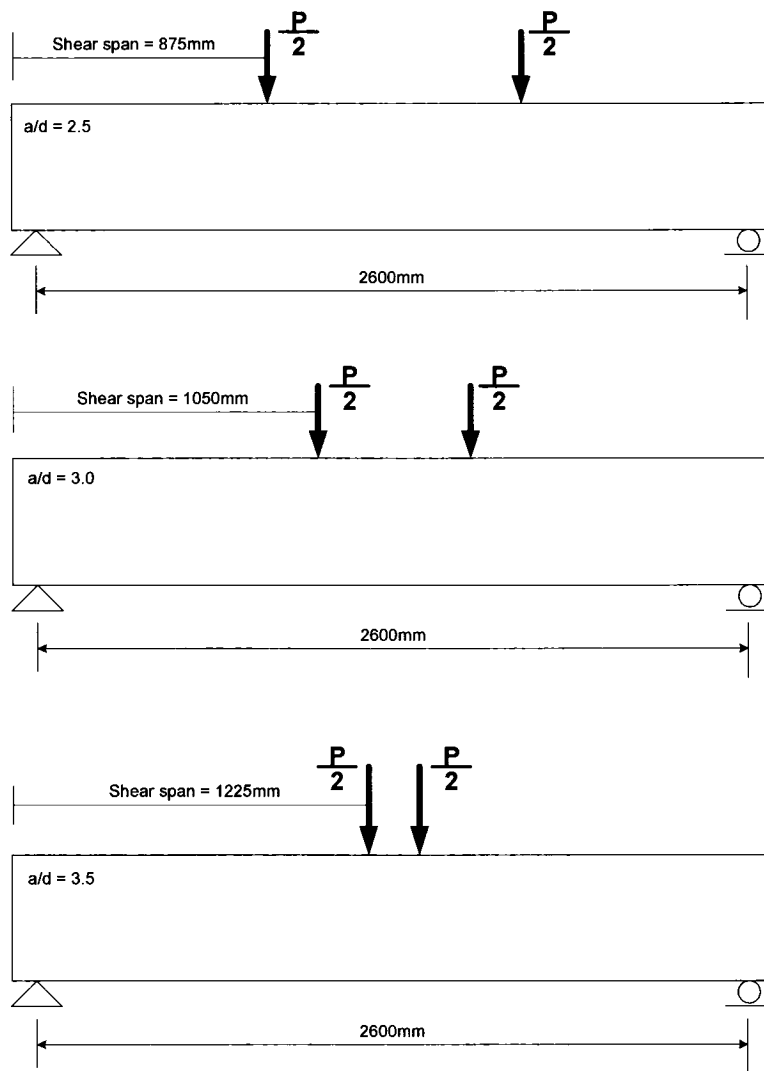


Fig. 3.23 Shear beams testing plan

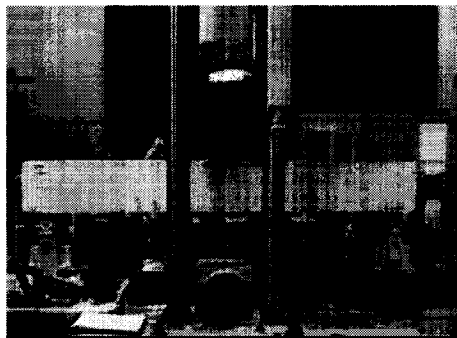


Fig. 3.24 Shear beams test set up

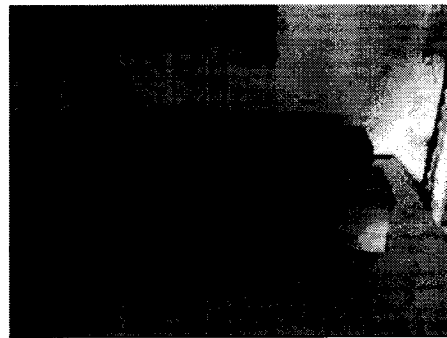
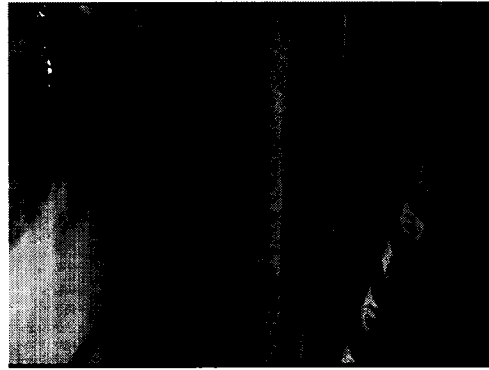


Fig. 3.25 Shrinkage beams test set up

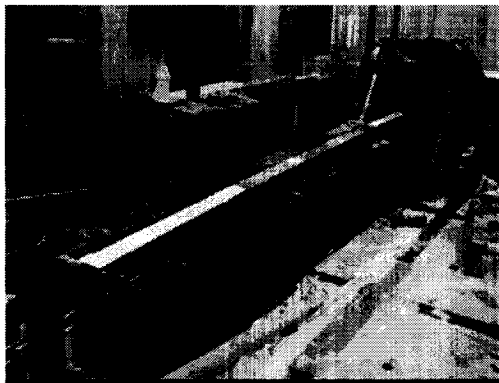


a) Concrete Cylinders in Creep test frame

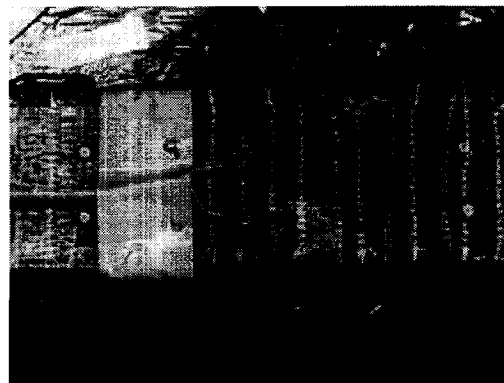


b) Demarcation gauges on creep cylinders

Fig. 3.26 Creep beams test set up



a) Prestressing technique used for Construction of total loss beams.



b) Demarcation gauges and strain gauges mounted on total loss beam

Fig. 3.27 RC beams with prestressed GFRP sheet - Long Term Losses Specimens.



Fig. 3.28 GFRP laminates test coupons



Fig. 3.29 GFRP laminate tensile test

CHAPTER 4

EXPERIMENTAL RESULTS

4.1 General

In previous chapter experimental program, to investigate the flexure and shear behaviour of the reinforced concrete beam strengthened with externally bonded prestressed and non-prestressed GFRP sheet, was described. The experimental results are presented in this chapter. Mechanical properties of the constituent materials used in construction of flexure and shear beams were obtained from the ancillary testing and are presented first in this chapter. The compressive strength of the concrete was estimated by averaging the results of tests on concrete cylinders. The stress-strain curves of steel reinforcement were obtained from the tension test on steel coupons. These curves provided the yield strength and modulus of elasticity of the reinforcing steel. The tension tests on GFRP sheet and laminates provided the stress-strain curves of these materials. From the stress-strain curves of the GFRP sheet the approximate maximum prestressing stress and modulus of elasticity were obtained. The stress-strain curve of GFRP laminate provided its modulus of elasticity and ultimate rupture strength.

The experimental results of flexure beams strengthened with externally bonded prestressed and non-prestressed GFRP sheet are presented and compared with un-strengthened control beams. The effect of prestressed GFRP sheet on crack initiation, crack width control and yielding strength of the beam are also discussed. The results of shear beams, having different shear span to effective depth ratios, strengthened

with U-straps and U-jackets are presented and compared with un-strengthened control beams. The increase in shear capacity by bonding the prestressed GFRP sheet on the flexure face of the beams were investigated experimentally and the results are presented. The experimental investigation on the total long term losses including losses due to creep and shrinkage is presented herein.

4.2 Ancillary Tests

The tests results of ancillary tests i.e. compression test on concrete cylinders, tension test on steel reinforcement, and tension test on GFRP sheets and laminates, are presented in the following sections. Test data that were considered to be suspect due to inaccuracy of a measuring device or to probable unexpected failure are indicated as struck out.

4.2.1 Concrete Tests

The compressive strength of the concrete cylinders is listed in Table 4.1. The average compressive strengths of the concrete used for the construction of flexure and shear beams are 39.0MPa and 38.6MPa, respectively. Cone failure mode was observed in most of the tested cylinders. For some of the cylinders, the mode of failure was due to the formation of large diagonal crack through the specimen. This was probable caused by unlevel caps on the end of the specimen which upon loading resulted in uneven distribution of load at the end of the specimen and caused wedge type of failure. The specimens failed in this manner yielded low compressive strength and has been strike through in the Table 4.1. These values were not used in calculation of average compressive strength of the concrete. Little variation was noted between sample strength tested at various ages. Specimens were tested throughout a four week testing period for flexure beams and six week period for shear beams. As small variations was noted between the specimens test results, therefore, single average compressive

strength value was chosen to represent the strength of the concrete mixes. The concrete modulus of elasticity value was calculated as per CSA 23.3A-94 equation for secant modulus, E_{cs} , for normal density concrete with compressive strength between 20 and 40MPa. The equation is given as;

$$E_{cs} = 4500 \sqrt{f'_c} \quad (\text{MPa Units})$$

The calculated value of modulus of elasticity of 29.1 and 28.9GPa, for flexure and shear beams, respectively, were used in theoretical determination of load-deflection and moment-curvature curves. The value of 2350Kg/m³ for the density of concrete was used in the analytical study.

4.2.2 Steel Reinforcement Tension Tests

The tension test results of the steel reinforcement bars used in construction of flexure and shear beams are shown in Table 4.2 and 4.3, respectively. In flexural beams No. 6 and No. 15 reinforcing bars were used as stirrups and main flexure reinforcement, respectively. Tension tests were conducted for No.6 and No.15 bars, whereas the mechanical properties of No.10 bars provided by supplier are presented in Table 5.2. Yield strengths of the No. 6, No. 10, and No. 15 reinforcing steel bars used in construction of flexure beams are 475MPa, 400MPa and 462.5MPa, respectively. The modulus of elasticity of the No. 6, No. 10, and No. 15 bars are 215.9GPa, 200GPa, and 193.2GPa, respectively. Stress-strain curves for No. 6 bar and No. 15 bar used in flexure critical beams are shown in Fig. 4.1 and Fig 4.2.

In shear beams, low modulus steel was used for stirrups and compression steel reinforcement. In shear beams, No. 6 bars were used as stirrups, No. 10 bars were used as compression reinforcement, and No. 25 bars were used as main flexure reinforcement. Yield strengths of the No. 6, No. 10, and No. 25 reinforcing steel bars used in construction of shear critical beams are 283MPa, 383.1MPa and 520MPa,

respectively. The modulus of elasticity of the No. 6, No. 10, and No. 25 bars are 162GPa, 191.5GPa, and 195.2GPa, respectively. The uniaxial stress-strain relationship of all the reinforcing steel, i.e. No. 6 bars, No. 10 bars and No. 25 bar, used in construction of shear critical beams are shown in Fig. 4.3 to Fig 4.5. The No. 6 and No.10 reinforcing steel test specimens exhibited the expected typical stress-strain behaviour as shown in Fig 4.3 and Fig. 4.4. The No. 25 reinforcing steel test specimen did not exhibit a distinct yield plateau as shown in Fig. 4.5. The yield strength was determined at 2% offset strain and the modulus of elasticity was determined from the slope of the stress-strain curve.

4.2.3 GFRP sheet and Laminate Tension Tests

Results of GFRP sheet and laminate tension tests are listed in Table 4.4 and Table 4.5, respectively. The GFRP sheets were tested to find out the approximate maximum prestressing load during the prestressing of the sheet. The GFRP sheets were anchored and tested in the manner it was prestressed during application to the beams. Two 100mm wide specimens of the GFRP sheet were tested and its results are presented in Table 4.4. The stress-strain curves for the specimens shown in Fig 4.6. The stress-strain curves are generally linear, and any deviation from the linear are thought to be caused by the testing method. It was observed that all the fibres across the width of the sheet were not under same stress during the test. The stress concentration resulted in breakage of highly stressed fibres, and redistribution of load caused a sudden increase in strain in the remaining intact fibres. The loss of small section of the sheet also resulted in decrease in the effective stiffness of the sheet. Premature loss of the fibres in the specimens was partially due to uneven loading across the width of the GFRP sheet. The modulus of elasticity of 100mm wide sheet is 61.2 and 66.0GPa. The difference in the modulus of elasticity is due to unequal distribution of stresses

across the width of the sheet. The efficient control of even stress distribution across the width of the sheet can yield higher modulus of elasticity. The ultimate strengths of the sheets are 528MPa and 548MPa. Generally, failures initiate at one side and propagate to the far side of the specimen width. Fibre broke and snapped individually until the entire width failed. The ultimate tensile strength of the sheet i.e. 538MPa was less as compared to the fibre tensile strength of the single fibre i.e. 3.24GPa (provided by manufacturer and listed in Table 4.1). This difference in the ultimate strength is due to the fact that all fibres were not stressed equally. During tension test of GFRP sheet, some of the fibres reached its ultimate capacity and ruptured and hence redistribute the load which resulted in low ultimate strength for the GFRP sheet in comparison to the GFRP fibre. This testing provided a fair idea about calculation of approximate maximum prestressing level without rupture of the sheet. An average value of 63.3GPa and 538MPa for modulus of elasticity and ultimate strength was used in calculation of prestressing force in the subsequent tests.

The results of the tension tests of GFRP laminate are listed in Table 4.5. The GFRP laminates coupons were tested in accordance to ISO draft standard. The stress-strain curves for the specimens are shown in Fig. 4.7. The curves are almost linear till failure. Two specimens exhibit smaller modulus of elasticity and ultimate strength values and were strike out from average modulus of elasticity and ultimate strength calculation. These specimens failed in the grip of testing machine due to high grip pressure. The grip pressure was adjusted in subsequent tests. The remaining test specimens were ruptured in the gauge length. The average ultimate strength of the GFRP laminate is 625MPa as compared to 538MPa, the average ultimate strength of GFRP sheet. The difference is thought to be caused by the epoxy used in fabrication of GFRP laminate. The epoxy resin provided a media which helped in even

distribution of load across the width of the sheet. The low modulus of GFRP laminate i.e. 27.4GPa, as compared to the GFRP sheet i.e. 63.6GPa, is thought to be due to low modulus of the epoxy. The manufacturer provided the modulus of elasticity of the epoxy resin as 3.18GPa as listed Table 4.1. The test results show high values of average tensile strength and modulus of elasticity as compared to the values provided by the manufacturer. This difference is thought to be caused by different test methods used by manufacturer and different fibre volume fraction.

4.3 Flexure and Shear Beams Test Results

The results of the prestressing operations performed on the GFRP sheet are presented initially. The results of the flexure beams followed by the critical beams, tested under four point bending, are presented subsequently. The flexure beams include results for the control beams, beams strengthened with non-prestressed GFRP sheet, and beams strengthened with prestressed GFRP sheet. The shear beams include results for the control beams, beams strengthened with U-straps, beams strengthened with U-Jackets, and beams strengthened with prestressed GFRP sheet bonded to flexure face of the beams. The behaviour of the beams was monitored through many stages of loading, i.e. from the initial cracking of the concrete, to yielding of the flexure or shear reinforcement, and failure of the beams at ultimate strength conditions.

4.3.1 Prestressing Operation on GFRP Sheet

The GFRP sheets were tensioned individually as outlined in section 3.2.4. In the cases of four flexure beams strengthened with prestressed GFRP sheets, the final estimated prestressing force and stress in the sheets is shown in Table 4.6. A total of 7 sheets were prestressed. The numbers of sheets on each strengthened beams are outlined in Table 3.2. The sheets were prestressed initially by tightening nuts on the

threaded bars. The level of the jacking force and stress was estimated from load cell and strain gauges data. The prestressed sheets were then bonded to the reinforced concrete beam with epoxy and were let to cure. The level of the prestress at transfer was estimated again from the load cell and strain gauges data. Three strain gauges were applied to each sheet during prestressing. Most of the strain gauges damaged during prestressing process. The jacking force was estimated from the output of external load cell mounted on prestressing bed and are presented in Table 4.6. The jacking force applied to the sheet was approximately 9 to 10kN for the prestress level 1 and 13.60kN for prestress level 2. Average jacking stress in the sheets varies from 169 to 182MPa, i.e. approximately 30% of the ultimate strength of the GFRP sheet, for prestress level 1. In prestress level 2, the average jacking stress was 252MPa, i.e. approximately 45% of the ultimate strength of the GFRP sheet. The force in the sheets at transfer varied from 8.5 to 9kN for prestress level 1 and was 12.46kN for prestress level 2. The losses of 5 to 9% in the jacking force were noted at the time of transfer. It is believed that these losses were due to the deformation of threaded bars and supports on the external prestressing bed. Prestressing force was transferred to the beam at curing of the epoxy. A separate experimental investigation was conducted on long term losses of prestressing force and its results are presented later.

The final estimate of prestressing force and stress in the sheets of the six shear beams strengthened with prestressed GFRP sheet are shown in Table. 4.7. A total of 11 sheets were prestressed. The numbers of sheets on the strengthened shear beams are outlined in Table 3.3. The sheets were prestressed initially by hydraulic jack and the level of the jacking force and stress was estimated from load cell data. The prestressed sheets were then bonded to the reinforced concrete beam with epoxy and were let to cure. The level of the prestress at transfer was estimated again from the

load cell. No strain gauges were applied to the GFRP sheet during prestressing for strengthening the shear beams due to poor performance of the strain gauges during strengthening of flexure beams. The jacking force on all beams was estimated from the output of external load cell mounted on prestressing bed. The jacking force applied to the sheet was approximately 18 to 21kN with average jacking stress in the sheets varies from 203 to 224MPa, i.e. approximately 40% of the ultimate strength of the GFRP sheet. Force in the sheets at transfer varied from 17 to 19.8kN with the exception of one layer applied to B2P2.5 in which the force at the transfer was 12.9kN. This significant loss was due to the rupture of some fibres during prestressing which left around 70% width of the sheet to carry load. The losses of 5 to 9% in the jacking force were noted at the time of transfer. It is believed that these losses were due to the deformation of threaded bars, supports on the external prestressing bed and release of hydraulic pressure in the jack under sustain loading.

4.3.2 Flexure Beams Test Results

The significant values describing the behaviour of both un-strengthened and strengthened reinforced concrete beams are given in Table 4.8. Control beams displayed typical under-reinforced concrete beams behaviour. The addition of non-prestressed sheets increased the ultimate load carrying capacity of the strengthened beams. Non-prestressed GFRP sheets moderately improved the behaviour of the reinforced concrete beams under service loading condition by controlling crack widths and limiting deflections in the beams. Its influence on the cracking load is not significant. The non-prestressed GFRP sheets provided additional tension reinforcement and hence the ultimate load carrying of the strengthened beams increased. The application of prestressed sheets dramatically improved the serviceability of the beams. The cracking load was increased and the crack width was

reduced with application of prestressed GFRP sheet. The addition of prestressed GFRP sheet resulted in further increases in the beams ultimate strength as compared to the beams strengthen with non-prestressed GFRP sheets.

Moment-curvature and load-mid-span displacement curves of beam strengthened with one, two and three layers of GFRP sheet shown in Fig. 4.8 through Fig. 4.13. Control-2 and beam strengthened with three layers of GFRP sheet has 16% less flexure reinforcement than rest of beams. This variable was considered in experimental program to simulate the loss of steel reinforcement due to corrosion etc. The moment-curvature and load deflection curves of control-1, control-2, B2P1 and B3P1 are compared Fig. 4.14 and Fig15, respectively. All beams were tested under four point bending with a shear span of 800mm. Control-1 and control-2 exhibited the typical behaviour of under reinforced concrete beams, as shown in Fig. 4.8 through Fig. 4.13. Initially the beams displayed linear elastic behaviour. In control- 1 beam the 1st crack was observed at relatively low load i.e. 20kN which is 13% of the ultimate load. The onset of the cracking resulted in a sudden increase in displacement and a subsequent loss of stiffness. The beams strengthened with one layer of prestressed GFRP sheets tensioned to two different prestressing levels i.e. B1P1 and B1P2 cracked at higher load of 40kN and 45kN, respectively. The cracking load of B1P1 and B1P2 is 100% and 125% higher than the cracking load of the control-1 beam. The cracking load of beams prestressed with two layers of prestressed GFRP sheet i.e. B2P1 was not observed due to use of oil paint. The oil paint bridges the cracks, due to which the crack onset and width could not be observed in the initial stage of loading. In control-2 beam the 1st crack was observed at relatively low load i.e. 18kN which is 14% of the ultimate load. The onset of the cracking also resulted in a sudden increase in displacement and a subsequent loss of stiffness in control-2 beam. The beams

strengthened with three layers of prestressed GFRP sheets cracked at higher load of 50kN. The cracking load of B3P1 was 175% higher than the cracking load of the control-2 beam. The summation of cracks versus load for beam strengthened with one and three layers of GFRP sheet are shown in Table 4.11 and 4.12, respectively. These results are graphically shown in Fig. 4.16 and 4.17, respectively. The curves show that application of GFRP sheet has tremendously reduces the crack widths at any particular load. The improvement was more pronounced in service load condition. The increase in cracking load and reduction of crack width has not only retained stiffness of structure up to higher loads but is also beneficial for durability of structure under severe environmental condition. Yielding of flexure reinforcement in both control beams resulted in an excessive increase in displacement and further loss of stiffness. As the control beams deformed, the ultimate load was attained. At ultimate loads the concrete in the compression zone reached its ultimate strength and started crushing. The effective area of compression concrete decreases with crushing and the hence resulted in drop of load carrying capacity. The ultimate loads were observed at 22.4 and 15.7mm mid-span deflections in control-1 and control-2 beams, respectively.

The failure in both control beams was dominated by flexure effects, since the beams were designed to be sufficiently strong in shear. Cracking pattern of control beams in both groups of reinforced concrete beams can be seen in Fig. 4.22 and Fig. 4.28. Vertical flexure cracks formed along the sides of the reinforced concrete control beams. In the shear span of the beam, where shear was active and stresses were sufficiently large to initiate shear cracking in the concrete, the flexure cracks propagate into diagonal shear cracks. The pure flexure vertical cracks were observed between two loading points where no shear stresses were present. The widest crack width of 0.8mm was observed in control 1 beam at load of 150kN. The largest crack

width of 0.9mm at a load of 127kN was observed in control 2 beam. In the control beams, the crack spacing in the constant moment region varies from 75mm to 200mm.

The performance of the beams strengthened with GFRP sheet was similar to the control beams before the 1st crack formation. The stiffness of the un-cracked beams strengthened with non-prestressed and prestressed GFRP sheet were the same as the control beams as observed in the moment-curvature and load-deflection curves of Fig. 4.8 and Fig. 4.13. The addition of GFRP sheets had little effect on the stiffness of the un-cracked concrete section since the area of the sheets was relatively small and, therefore, changes in the transformed properties of the un-cracked section was also small. The addition of non prestressed GFRP sheets did not affect the first crack load; however, the prestressed sheets caused a dramatic increase in the first crack load of both groups of the strengthened beams. The 1st crack was marked from visual inspection of cracks and the corresponding 1st crack load was noted in all test beams. The cracks were marked and it widths were measured in all tested beams at specified loading intervals. The initial cracks were measured at 50kN and then at 10kN interval till ultimate load. The application of prestressed sheets to reinforced concrete beams in both groups caused a significant increase in the cracking load of the strengthened beams. The application of one and three layers of prestressed GFRP sheets resulted in an increase of 100% and 175%, respectively, in the cracking load of the strengthened beams over the corresponding control beams.

The behaviour of both groups of beams strengthened with non-prestressed and prestressed GFRP sheets changed after the formation of first crack, as evident from moment-curvature and load-mid span deflection curves shown in Fig. 4.8 and Fig. 4.13. The GFRP sheets contributed to the load bearing capacity of the beams, and

tensile stresses carried by steel reinforcement were redistributed to the GFRP sheets. This resulted in an increase in the yielding load of the strengthened beams as shown in Table 4.8. In reinforced concrete beams strengthened with one and two layers of non-prestressed GFRP sheets, the yielding of the steel was delayed until an average load 11% higher than control beam was attained. By application of prestressed GFRP sheets, the tension steel reinforcement was relieved of tensile stresses due to dead load and was placed slightly into compression. Because of prestress, a great portion of the tensile stresses carried by tensile reinforcement in the beam were transferred to the GFRP sheets. When prestressed sheets were bonded to the concrete beams, yielding occurred at 23%, 27%, and 30% higher loads than the corresponding control beams for the beams strengthened with one, two and three layers of prestressed GFRP sheet, respectively. The yielding of the principal tensile reinforcement for group 1 and group 2 beams can be seen in Fig. 4.18 and Fig. 4.19.

The application of GFRP sheets provides an additional tensile reinforcement. The additional tensile reinforcement activated a larger compressive section in the compression zone of the concrete beam and hence increased the ultimate load carrying capacity after yielding of steel reinforcement. Significant increases in the ultimate strength of the concrete beams were observed with addition of GFRP reinforcement. The increase in the ultimate load with addition of non-prestressed and prestressed GFRP sheets is shown in Table 4.9. An increase of 18% and 26% in the ultimate load over the control beam was observed in the beams strengthened with one and two layers of non-prestressed GFRP sheets, respectively. The increase in the ultimate load carrying capacity was slightly higher when prestressed GFRP sheet was applied to the beam. An increase of 23% and 39% in the ultimate load over the control beam was observed in the beams strengthened with one and two layers of

prestressed GFRP sheets, respectively. The increase in ultimate load of the beam strengthened with one layer of GFRP sheet with higher prestressed force was 22%. A significant increase of 63% in the ultimate load carrying capacity over control beam was recorded for the beam strengthened with three layers of the GFRP sheet.

The ultimate strength of the control beams was limited by yielding of the flexure steel reinforcement followed by crushing of the concrete in the compression zone of beam. With application of the GFRP sheets to the tensile face of the concrete beam, the ultimate strength of the strengthened beams were governed by tensile strength of GFRP sheet, concrete tensile or shear strength. At ultimate state conditions, either tensile rupture of the GFRP sheet, or debonding of the GFRP sheet from the concrete surface over a short, or complete peeling of the sheet from the concrete surface, has occurred. The strengthened beams failed immediately when the GFRP sheets were no longer active in resisting the applied moment. After failure, the residual strengths of the strengthened beams were comparable to the control beams. Tensile failure of the sheet was generally initiated at the section of the greatest moment, i.e. central portion of the tested beams. This type of failure was observed in beam B1P2 which was strengthened with one layer of prestressed GFRP sheet having high initial strain. It is thought that the tensile failure of the GFRP sheet happened because of smaller residual capacity of the sheet for resisting ultimate loads due to high initial stress induced in the sheet before application. At ultimate loading, high strains in GFRP sheets were observed in the beams strengthened with prestressed GFRP sheets as compared to the beams strengthened with non-prestressed GFRP sheet. This shows that the contribution of the GFRP sheet to the ultimate strength of the beam increased with prestressing of the sheet. In other word that prestressing delayed the debonding or peeling off of the sheet which resulted in utilizing the maximum capacity of the

GFRP sheet. The strains in the GFRP sheet at the mid span of the beam versus load are shown in Fig. 4.20.

The failure in all strengthened beam occurred due to debonding of GFRP sheet except the beam strengthened with on layer of GFRP sheet tensioned to high level of prestress. In these beams, the concrete cover between the internal steel reinforcement and the GFRP sheet failed. The GFRP sheets were either de-bonded with thin layer of concrete attached to the de-bonded sheet or with the big piece of cover concrete. The peeling of the sheet with thin layer of concrete normally occurred in the region of combined moment and shear, i.e. the ends of the sheets where high shear stresses were present in the sheet. The de-bonding of sheet with big piece of concrete occurred in the region of maximum moment i.e. mid span of beam. Reaching to ultimate loading stage, the flexure cracks in the constant moment region became wider. The concrete between the cracks acted as cantilever starting from the main flexure reinforcement. The tension in the sheet act as applied force at the tip of this cantilever. The cover concrete between the cracks failed when the stresses exceed the flexure strength of the concrete and resulted in de-bonding of the sheet with big piece of cover concrete attached to the sheet. This type of debonding was observed in beam, B1U. The de-bonded sheet with big piece of cover concrete attached at mid span is shown in Fig. 4.21. As a result of sheet debonding, the tensile stresses in the sheet were dissipated and the total load carrying capacities of the beams were subsequently decreased to that of the un-strengthened control beams. The failure of the un-strengthened control beams, beams strengthened with non-strengthened GFRP sheets, and beams strengthened with prestressed GFRP sheets are shown in Fig. 4.22 to Fig 4.29. Application of GFRP sheets efficiently control crack onset and crack widths which resulted in decreased displacement at any load. It was noted from test results,

that displacement were further decreased when the beams were strengthened with prestressed GFRP sheets. This was because of initial camber induced in the beam and increase in cracking load of the beam by prestressed sheets. The increase in cracking load ensured that the initial stiffness was active until higher loads were attained. The decreased deflection associated with any load is evident from the load deflection curve in Fig. 4.9, 4.11, and Fig. 4.13. The steel yielded at 130kN with associated deflection of 7.78mm in un-strengthened control-1 beam. The deflections of 7.44mm and 6.90mm at 130kN load, yielding load of control-1 beam, were noted in the beams strengthened with one and two layers of un-strengthened GFRP sheets, respectively. This was 5% and 11% less than the control beam deflection. It is evident from the test results that reduction in deflection increased with increase in number of sheets. Application of prestressed GFRP sheet further reduced the deflection. One layer of prestressed GFRP sheet on two beams with two different levels of initial prestressing forces resulted in 9% reduction in deflection as compared to deflection of control beam at 130kN. Two layers of prestressed sheets resulted in 19% reduction in deflection at this load. The deflection of 8.41mm was noted at 120kN, the yielding load of control-2 beam. The corresponding deflection at this load, in beam strengthened with three layers of prestressed sheet, was 6.32mm which is 25% less than the control-2 beam deflection.

As mentioned before, control-2 and beam B3P1 have 16% less flexural reinforcement than control-1 and beam B2P1. The test results of these four beams were compared to investigate the restoration of stiffness and strength of beam with flexural reinforcement loss. The moment-curvature and load-deflection curves in Fig. 4.14 and Fig. 4.15, shows that addition of one extra layer of prestressed GFRP sheet has effectively restored the stiffness and strength of beam at all loading stages. Beam

B2P1 and B3P1 behaved exactly in same fashion on all loading stages. The comparison of control beams shows that control-2 have less stiffness and strength than control-1 beam. It is evident from test results that mass loss of steel reinforcement due to corrosion etc. can be effectively restored with application of externally bonded prestressed GFRP sheet. In other words, 150 mm^2 of prestressed GFRP sheet has compensated for 100 mm^2 of conventional steel reinforcement.

The GFRP sheet provided extensive control of the cracking pattern, as shown in Fig. 4.23 to Fig. 4.27 and Fig 4.29. In control beam, fewer, relatively wide cracks were noted in the constant moment region. In shear span of the beam, diagonal cracks extended to the top compression zone of the beam were note. The cracks in the beam strengthened with non prestressed GFRP sheet were closer in the constant moment region. The diagonal cracks in the shear span of these beams were almost identical to the control beam. The beam strengthened with prestressed GFRP sheet showed superior crack control. Flexure cracks were noted in the constant moment region. Fewer diagonal shear cracks with smaller crack widths extended to half depth of the beams were noted. The influence of the prestressed GFRP sheet over the width and height of the shear cracks shows its influence on the shear capacity of the strengthened beams. The shear cracks were appeared at relatively high loads in the beams strengthened with prestressed GFRP sheets. The change in the shear behaviour of the beam promoted the idea of investigating the effect on the shear capacity of the beams strengthened with prestressed GFRP sheet. For this purpose six beams, having shear reinforcement deficiency and strengthened with prestressed GFRP sheets attached to tension face of the beam, were tested on three different shear span to effective depth ratios. The results of this investigation are presented in the next section.

4.3.3 Shear Beams Test Results

The significant values describing the behaviour of beams deficient in shear reinforcement and strengthened with U-straps, U-jackets and prestressed GFRP sheet attached to the face of the beams is shown in Table 4.10. Three groups of beams were tested with three different shear-spans to effective depth ratios, i.e. 2.5, 3.0 and 3.5, to investigate the shear behaviour of both deep and slender beams. Each group include one control beam, two beams either strengthened with U-straps or U-jacket in shear span, and two beams strengthened with prestressed GFRP sheet. As shown in Table 4.10, the beams strengthened with U-straps and U-jackets of GFRP sheets sustained substantially more loads than the control beams. External shear reinforcement was absent in the beams strengthened with prestressed GFRP sheets bonded to tension face of the beam. In classic sense, the addition of prestressed GFRP sheet to the flexure face of the beam should not be expected to add shear strength, however, the experimental result shows a significant increase in the shear capacity of the strengthened beams. The test results shown in Table 4.10 reveal that increase in the shear strength of the strengthened beams was more significant in the beams tested with shear span to effective depth (a/d) ratio of 2.5 than the beams tested with a/d ratio of 3.0 and 3.5. This can be attributed to the difference in shear behaviour of deep and slender beams. The beams tested with a/d ratio of 2.5 behaved as deep beam where prestressed sheet acted as tension tie in strengthened beam and hence the increase was more significant. The beam tested with a/d ratio of 3.0 and 3.5 was more slender and its shear behaviour was different. The increase in shear capacities of these beams was thought to be due to efficient crack control due to application of prestressed GFRP sheet. Prestressed GFRP sheets bridges the cracks, due to which the crack opening occurred at relatively higher load. The concrete contribution to the

shear capacity of the strengthened beams, especially concrete interlock, remained active until higher applied loads. Details of shear beams test results tested over different a/d ratios are presented in subsequent sections.

4.3.3.1 Results of Shear Beams Tested on a/d Ratio of 2.5

Five beams, one control, one strengthened with U-straps, one with U-jackets, and two strengthened with prestressed GFRP sheets were tested over a/d ratio of 2.5. The shear force-displacement curves of shear beams tested over this a/d ratio are shown in Fig. 4.30. All beams were tested in a four point bending mode with displacement control at a loading rate of 0.5mm/minute. The test span of control beam and beam strengthened with three layers of prestressed GFRP sheets, B3P2.5, was 2800mm. In both control and beam B3P2.5, crushing of concrete took place over roller supports. It was thought that high reaction loads acted as point load which resulted in stress concentration exceeding the crushing strength of concrete and resulted in local failure over supports. To avoid this problem in rest of testing program, test span was reduced to 2600mm. 150mm wide and 250mm long stiff steel plate was placed over roller supports to distribute the reaction at wider area. This arrangement prevents crushing of concrete over the support.

Control beam failed in shear, as is evident by a lack of distinct yield point on the shear force-displacement curves as shown in Fig. 4.30. Flexure cracks occurred at a single jack load of 121kN, and the flexure shear cracks appeared at approximately 260kN. The cracks were marked on the face of the beam. Test was stopped at a single jack load of 200kN, and the crack widths were measured. The summation of the all crack widths at this load was 0.78mm. Test was started again and cracks were marked until the loading reached to 250kN. In the loading range from 200 to 250kN, most of the old cracks opened up and few new cracks appeared. The summation of crack

widths at this loading was 1.22mm. Cracks continued to form until a load of approximately 276kN, when a single major diagonal shear crack starting from left support appeared and propagate to loading point. This shear crack opened up and lead to abrupt shear failure. The analysis of stirrups strain data showed that stirrups which crossed the wide shear crack yielded. As expected, the average strain in main flexure reinforcement at this load was $1075 \mu\epsilon$, which is well below yielding strain. The failure of the control beam is shown in Fig. 4.33.

Beam strengthened with external U-straps, BU2.5, showed an increase of 64% in the shear capacity over the control beam. The externally bonded GFRP U-straps provided additional shear reinforcement which helped in resisting higher shear force. The shear stress was distributed between the external GFRP U-straps and internal steel reinforcement. Flexure cracks occurred at a single jack load of 120kN, and the flexure shear cracks appeared at approximately 300kN. The cracks were marked on the face of the beam. Test was stopped at a single jack load of 200kN, and the crack widths were measured. The summation of crack widths at this load was 0.42mm. Cracks continued to form until a load of approximately 452.5kN, when a single major diagonal shear crack, starting from right support and propagate to loading point, opened up and lead to sudden shear failure. Crack widths were measured at every 50kN load increment till failure. The summation of crack width at 250kN, 300kN, 350kN and 400kN was recorded as 0.76mm, 1.04mm, 1.54mm and 2.46mm, respectively. The analysis of stirrups strain data showed that stirrups, which crossed the wide shear crack, yielded. As expected, the average strain in main flexure reinforcement at this load was $1650 \mu\epsilon$, which is well below yielding strain. Maximum strain of $12947 \mu\epsilon$ was recorded in the GFRP U-straps before failure. At ultimate load, GFRP U-straps in the right shear span de-bonded from the vertical face

of the beam followed by yielding of internal steel stirrups. The failure of beam, BU2.5, is shown in Fig.4.34.

Beam strengthened with external U-jacket, BJ2.5, showed an increase of 160% in the shear capacity over the control beam as shown in shear force-displacement curve in Fig. 4.30. The area of the additional shear reinforcement provided by externally bonded GFRP U-jacket was twice as compared to the beam, BU2.5. The behaviour of the beam, BJ2.5, was similar to beam BU2.5 except that it resist higher load before failure. Due to externally bonded GFRP sheets in the shear span, only flexure cracks were noted in the constant moment region. The flexure crack occurred at a single jack load of 123kN. The ultimate load of 717.7kN was achieved in this beam. The tension reinforcement yielded as it approached the ultimate loads. The average strain of $3195 \mu\epsilon$ was recorded in the tension reinforcement at ultimate load. The debonding of GFRP sheet occurred in the right shear span followed by yielding of internal shear reinforcement which resulted in sudden shear failure. The failure of beam, BJ2.5, is shown in Fig. 4.35.

Beam strengthened with two layers of prestressed GFRP sheets attached to the tension face of beam, B2P2.5, showed an increase of 125% in the shear capacity over the control beam as shown in shear force-displacement curve in Fig. 4.30. In conventional calculations, horizontal structural component are not allowed to be used to resist the diagonal tension cracks, however, the experimental investigation provided different results. It is believed that application of prestressed GFRP sheet induced compressive stresses in the bottom fibre of the beam and provided efficient crack control in the strengthened beam which resulted in an increase in shear capacity. This beam behaved like deep beam and the GFRP acted as tension tie. The concrete in the compression strut were crushed at ultimate. 1st flexure crack occurred at a single jack

load of 176kN which is 43% higher than cracking load of control. Flexure shear cracks appeared at approximately 300kN. The cracks were marked on the face of the beam. Test was stopped at a single jack load of 200kN, and the crack widths were measured. The summation of crack widths at this load was 0.16mm. Cracks continued to form until a load of approximately 623.5kN. Two major diagonal shear cracks, starting from both supports started and propagate to loading point at a load of 500kN. The concrete between the diagonal cracks crushed at ultimate load. Crack widths were measured at every 50kN load increment until loading of 500kN. The summation of crack width at 250kN, 300kN, 350kN, 400kN, 450kN, and 500kN was recorded as 0.44mm, 0.92mm, 1.78mm, 2.92mm, 4.32mm and 5.08mm, respectively. The maximum average strain of $2075 \mu\epsilon$ was recorded in the tension steel, which is below yielding strain of the tension reinforcement. The stirrups crossing the cracks yielded. The failure of the beam occurred due to concrete failure on the compression struts. The failure of the beam is shown in Fig. 4.36.

Beam strengthened with three layers of prestressed GFRP sheets attached to the tension face of beam, B3P2.5, showed an increase of 90% in the shear capacity over the control beam as shown in shear force-displacement curve in Fig. 4.30. As mentioned early in this chapter, the failure of this beam occurred due to crushing of concrete over the support. It is thought that beam did not reach its ultimate shear capacity before failure, that why the increase in shear capacity is less as compared to Beam, B2P2.5. The increase in shear capacity was thought due to the reason explained in beam B2P2.5. 1st flexure crack occurred at a single jack load of 200kN which is 63% higher than cracking load of control. Flexure shear cracks appeared at approximately 400kN. The cracks were marked on the face of the beam. Test was stopped at a single jack load of 200kN, and the crack widths were measured. Only one

crack having width of 0.04mm was noted at this load. Cracks continued to form until a load of approximately 525.3kN. Two major diagonal shear cracks, starting from both supports started and propagate to loading point at a load of 400kN. Crack widths were measured until loading of 500kN. The summation of crack width at 250kN, 300kN, 350kN, 400kN, and 500kN was recorded as 0.06mm, 0.32mm, 1.50mm, 2.14mm, and 4.46mm, respectively. The maximum average strain of $1750 \mu\epsilon$ was recorded in the tension steel, which is less than beam, B2P2.5, and below yielding strain of the tension reinforcement. The stirrups crossing the cracks yielded. The failure of the beam occurred due to local concrete failure over the support. The failure of the beam is shown in Fig. 4.37.

4.3.3.2 Results of Shear Beams Tested on a/d Ratio of 3.0

Five beams, one control, one each strengthened with U-straps and U-jackets, and two strengthened with prestressed GFRP sheets were tested over a/d ratio of 3.0. The shear force-displacement curves of shear beams tested over at this a/d ratio are shown in Fig. 4.38. All beams were tested in a four point bending mode over a clear span of 2600mm with displacement control at a loading rate of 0.5mm/minute.

Control beam failed in typical shear failure, as is evident by a lack of distinct yield point on the shear force-displacement curves as shown in Fig. 4.38. Flexure cracks occurred at a single jack load of 120kN, and the flexure shear cracks appeared at approximately 218kN. The cracks were marked on the face of the beam and its width were measured at a load of 200kN. The summation of crack widths at this load was 0.78mm. Test was started again and cracks were marked until failure of the beam at a load of 251.1kN. In the loading range from 200 to 251.1kN, most of the old cracks opened up and few new cracks appeared. Cracks continued to form until a load of approximately 218kN, when a single major diagonal shear crack starting from right

support appeared and propagate to loading point. This shear crack opened up and lead to abrupt shear failure. The analysis of stirrups strain data showed that stirrups which crossed the wide shear crack yielded. As expected, the average strain in main flexure reinforcement at this load was $1240 \mu\epsilon$, which was higher than control2.5 due to higher a/d ratio but still lower than yielding strain of the beam. The failure of the control beam is shown in Fig. 4.39.

Beam strengthened with external U-straps, BU3.0, showed 100% increase in the shear capacity over the control beam. Like beam BU2.5, the externally bonded GFRP U-straps provided additional shear reinforcement which helped in resisting higher shear force. The beam behaved like BU2.5 until failure. Flexure cracks occurred at a single jack load of 112kN, and the flexure shear cracks appeared at approximately 300kN. The cracks were marked on the face of the beam. Test was stopped at a single jack load of 200kN, and the crack widths were measured. The summation of crack widths at this load was 0.74mm. Crack widths were measured at every 50kN load increment till failure. The summation of crack width at 250kN, 300kN, 350kN, 400kN, and 450kN was recorded as 1.04mm, 1.98mm, 2.46mm, 3.66mm, and 4.80mm, respectively. The analysis of stirrups strain data showed that stirrups, which crossed the wide shear crack, yielded. As expected, the average strain in main flexure reinforcement at this load was $2400 \mu\epsilon$. Maximum strain of $6320 \mu\epsilon$ was recorded in the GFRP U-straps before failure as compared to $12947 \mu\epsilon$ in beam, BU2.5. The big difference is thought due to position of the strain gauges on the U-strap. At ultimate load, GFRP U-straps in the right shear span rupture and de-bonded at lower corner of the rectangular beam followed by yielding of internal steel stirrups. The failure of beam, BU2.5, is shown in Fig. 4.40.

Beam strengthened with external U-jacket, BJ3.0, showed an increase of 150% in the shear capacity over the control beam as shown in shear force-displacement curve in Fig. 4.38. The area of the additional shear reinforcement provided by externally bonded GFRP U-jacket was twice as compared to the beam, BU3.0. The behaviour of the beam, BJ3.0, was similar to beam BJ2.5. The ultimate load of 626.1 was achieved in this beam. The tension reinforcement yielded as it approached the ultimate loads. The average strain of $3600 \mu\epsilon$ was recorded in the tension reinforcement at ultimate load. The maximum strain of $3600 \mu\epsilon$ was recorded in GFRP sheet before debonding. The debonding of GFRP sheet occurred in the right shear span followed by yielding of internal shear reinforcement which resulted in sudden shear failure. The failure of beam, BJ2.5, is shown in Fig. 4.41.

Beam strengthened with one layer of prestressed GFRP sheet attached to the tension face of beam, B1P3.0, showed an increase of 26% in the shear capacity over the control beam as shown in shear force-displacement curve in Fig. 4.38. This beam behaved like slender beam. The prestressed GFRP sheet induced compressive stresses in the bottom fibre of the beam which resulted in reduction of diagonal tension stresses responsible for onset of flexure shear crack. The GFRP sheet also bridges the cracks at the bottom face of the beam which helped in keeping intact the concrete interlock forces until higher applied loads. The dual effect of prestressed GFRP sheet resulted in an increase in shear capacity. 1st flexure crack occurred at a single jack load of 163kN which is 45% higher than cracking load of control. Flexure shear cracks appeared at approximately 250kN. The cracks were marked on the face of the beam and its widths were measured. A major diagonal shear crack started from right support and propagates to loading point at a load of 300kN. The summation of crack width at 200kN, 250kN, and 300kN was recorded as 0.14mm, 0.68mm, and 3.94mm,

respectively. The maximum average strain of $1250 \mu\epsilon$ was recorded in the tension steel, which is well below yielding strain of the tension reinforcement. The diagonal shear crack opened up at once and the internal steel stirrups crossing the cracks yielded. The sudden shear failure of the beam occurred at 315.4kN. The failure of the beam is shown in Fig. 4.42.

Beam strengthened with two layers of prestressed GFRP sheets attached to the tension face of beam, B2P2.5, showed an increase of 58% in the shear capacity over the control beam as shown in shear force-displacement curve in Fig. 5.38. The increase in shear capacity was thought due to the reasons explained for beam B1P2.5. 1st flexure crack occurred at a single jack load of 150kN which is 34% higher than cracking load of control. Flexure shear cracks appeared at approximately 300kN. The cracks were marked on the face of the beam. Test was stopped at a single jack load of 200kN, and the crack widths were measured. The summation of cracks widths at this load was 0.6mm. Cracks continued to form until a load of approximately 350kN. Two major diagonal shear cracks, starting from both supports started and propagate to loading point at a load of 350kN. The diagonal shear crack in the right shear span opened up and a sudden shear failure occurred. The summation of crack width at 250kN, 300kN, and 350kN was recorded as 0.9mm, 2.16mm, and 6.32mm, respectively. The maximum average strain of $1600 \mu\epsilon$ was recorded in the tension steel, which is less than beam, B1P3.0, and below yielding strain of the tension reinforcement. The stirrups crossing the cracks yielded. The failure of the beam is shown in Fig. 4.43.

4.3.3.3 Results of Shear Beams Tested on a/d Ratio of 3.5

Five beams, one control, one each strengthened with U-straps and U-jackets, and two strengthened with prestressed GFRP sheets were tested over a/d ratio of 3.5. The

shear force-displacement curves of shear beams tested over at this a/d ratio are shown in Fig. 4.44. All beams were tested in a four point bending mode over a clear span of 2600mm with displacement control at a loading rate of 0.5mm/minute.

Control beam failed in typical shear failure, as is evident by a lack of distinct yield point on the shear force-displacement curves as shown in Fig. 4.44. Flexure cracks occurred at a single jack load of 99kN, and the flexure shear cracks appeared at approximate load of 210kN. A big shear crack appeared in right hand side shear span at a load of 242kN. The applied load dropped to 216kN. Due to shear cracking, the loss in beam stiffness is clear from the shear force-deflection curves shown in Fig. 4.44. The steel stirrups strain data shows no strain in the stirrups before onset of big shear crack at load of 242kN. The steel stirrups started to take the shear load and yielded at ultimate load of 267.7kN. Cracks were marked on the face of the beam and its width were measured at a load of 200kN. The summation of the all crack widths at this load was 0.84mm. The shear cracks continued to open up which led to abrupt shear failure. The analysis of stirrups strain data showed that stirrups which crossed the wide shear crack yielded. As expected, the average strain in main flexure reinforcement at ultimate load was $1400 \mu\epsilon$, which was higher than control2.5 and control3.5, due to higher a/d ratio but still lower than yielding strain of the beam. The failure of the control beam is shown in Fig. 4.45.

Beam strengthened with external U-straps, BU3.5, showed 88% increase in the shear capacity over the control beam. Like beam BU2.5 and BU3.0, the externally bonded GFRP U-straps provided additional shear reinforcement which helped in resisting higher shear force. The beam behaved like BU2.5 and BU3.0 until failure. Flexure cracks occurred at a single jack load of 113kN, and the flexure shear cracks appeared at approximately 300kN. The cracks were marked on the face of the beam.

Test was stopped at a single jack load of 200kN, and the crack widths were measured. The summation of crack widths at this load was 0.36mm. Crack widths were measured at every 50kN load increment till failure. The summation of crack width at 250kN, 300kN, 350kN, 400kN, and 450kN was recorded as 0.88mm, 2.14mm, 3.14mm, 4.00mm, and 5.54mm, respectively. The analysis of stirrups strain data showed that stirrups, which crossed the wide shear crack, yielded. As expected, the main flexure reinforcement yielded at this load and average strain in at this load was recorded as $3600 \mu\epsilon$. Maximum strain of $7990 \mu\epsilon$ was recorded in the GFRP U-straps before failure as compared to $12947 \mu\epsilon$ in beam, BU2.5. The big difference is thought due to position of the strain gauges on the U-strap. At ultimate load, two GFRP U-straps in the right shear span was ruptured and de-bonded at lower corner of the rectangular beam and one U-strap was de-bonded from the vertical face, followed by yielding of internal steel stirrups. The failure of beam, BU2.5, is shown in Fig. 4.46.

Beam strengthened with external U-jacket, BJ3.5, showed an increase of 117% in the shear capacity over the control beam as shown in shear force-displacement curve in Fig. 4.44. The area of the additional shear reinforcement provided by externally bonded GFRP U-jacket was twice as compared to the beam, BU3.5. The behaviour of the beam, BJ3.5, was similar to beam BJ3.0, except that the tension steel yielded before debonding of U-jacket occurred. The ultimate load of 581.8kN was achieved in this beam. The maximum strain of $8700 \mu\epsilon$ was recorded in the tension reinforcement. The maximum strain of $6750 \mu\epsilon$ was recorded in GFRP sheet before debonding. The debonding of GFRP sheet occurred in the right shear span followed by yielding of internal shear reinforcement which resulted in sudden shear failure. The failure of beam, BJ2.5, is shown in Fig. 4.47.

Beam strengthened with one layer of prestressed GFRP sheet attached to the tension face of beam, B1P3.5, showed an increase of 33% in the shear capacity over the control beam as shown in shear force-displacement curve in Fig. 4.44. This beam behaved like slender beam. Like beam, B1P3.0, the compressive stresses in the bottom fibre of the beam and crack control provided by prestressed GFRP sheet resulted in an increase in shear capacity. 1st flexure crack occurred at a single jack load of 110kN which is 11% higher than cracking load of control3.5. Flexure shear cracks appeared at approximately 250kN. The cracks were marked on the face of the beam and its widths were measured. A major diagonal shear crack started from right support and propagated to loading point at a load of 300kN. The summation of crack width at 200kN, 250kN, 300kN and 350kN was recorded as 0.52mm, 1.08mm, 4.82mm and 7.2mm, respectively. The maximum average strain of $1740 \mu\epsilon$ was recorded in the tension steel, which is well below yielding strain of the tension reinforcement. The diagonal shear crack opened up at once and the internal steel stirrups crossing the cracks yielded. The sudden shear failure of the beam occurred at 355.2kN. The failure of the beam is shown in Fig. 4.48.

Beam strengthened with two layers of prestressed GFRP sheets attached to the tension face of beam, B2P3.5, showed an increase of 40% in the shear capacity over the control beam as shown in shear force displacement curve in Fig. 4.44. The increase in shear capacity was thought due to the reasons explained for beam B1P3.5. 1st flexure crack occurred at a single jack load of 170kN which is 72% higher than cracking load of control3.5. Flexure shear cracks appeared at approximately 300kN. The cracks were marked on the face of the beam. Test was stopped at a single jack load of 200kN, and the crack widths were measured. The summation of cracks widths at this load was 0.04mm. Cracks continued to form until a load of approximately

350kN. Two major diagonal shear cracks, starting from both supports started and propagate to loading point at a load of 350kN. The diagonal shear crack in the left shear span opened up and a sudden shear failure occurred. The summation of crack width at 250kN, 300kN, and 350kN was recorded as 0.42mm, 2.54mm, and 5.84mm, respectively. The maximum average strain of $1705 \mu\epsilon$ was recorded in the tension steel. The stirrups crossing the cracks yielded. The failure of the beam is shown in Fig. 4.49.

4.4 Results Long Term Losses Test

The experimental investigation on long term losses due to the prestressed GFRP sheet is presented in this section. Application of prestressed GFRP sheet induced creep strain in the beam which resulted in long term prestress losses. Similarly the epoxy at the sheet and concrete interface are subjected to high shear stresses which may cause some shear deformations in the epoxy layer and hence reduce the prestress forces in the GFRP sheet. The shrinkage strains may also cause some long term prestress losses. To evaluate the effect of creep and shrinkage on long term prestress losses, an experimental program was carried out. The observed results of creep and shrinkage loss are presented in Table. 4.13. Similarly, the observed results of shrinkage and creep strains are plotted in Fig. 4.50 and 4.51, respectively. The test was carried out for six months. It is clear from both curves that concrete shows high creep and shrinkage strains in the initial period and the gain in strain reduces with time. The maximum creep strain of $52 \mu\epsilon$ was observed in six months. The small creep value is due to the fact that stress level in concrete was very low due to low prestressing force in concrete. Like creep strain, the increase in shrinkage strain was high in early days of concrete age and reduces with time. The shrinkage strain curve shows rapid increase in shrinkage strain during early age of concrete which tends to

reduce with increase in concrete age. The maximum shrinkage strain of $170 \mu\epsilon$ was observed at age of six months. The creep and shrinkage strain reduce the prestress in the GFRP sheets. The total prestress loss of $222 \mu\epsilon$ was observed due to these two factors. To get total losses in the beam prestressed with GFRP sheet, the strain losses on beam strengthened with two layers of GFRP sheet was recorded. The strain due to creep and shrinkage were subtracted from total losses and the reminder was considered as loss due shear strain in epoxy layer, fibre relaxation, elastic shortening due to multi layer application and other unknown factors. The results of total losses and total strain due to creep and shrinkage are compared in Table 4.14. The strain data of the prestressed beam show similar trend. The total loss of $319 \mu\epsilon$ was recorded at the age of 6 months. The loss due to other factors than creep and shrinkage was calculated as $97 \mu\epsilon$ at the age of six months.

In summary it can be say that shrinkage strain is main contributor of prestress losses. Low creep strain was recorded due to low prestressing force. Despite all these factor total strain of $319 \mu\epsilon$ have very little effect on prestress losses because GFRP sheet is relatively low modulus material. The modulus of elasticity of the GFRP sheet is 27400MPa. The total prestress loss 8.7MPa is recorded due to total strain of $319 \mu\epsilon$. The prestress in this sheet at the jacking time was 208MPa. This means that a total loss of approximately 4% occurred in the beam prestressed with GFRP sheet. This is very low losses as compared to losses in beam prestressed with conventional prestressing wires where long term losses amounts to approximately 25%. The lower loss figure is due to relatively low prestressing force and lower modulus of elasticity of the GFRP sheet as compare to prestressing wires.

Table 4.1 Concrete compression tests

Flexure Beams Compressive strength (MPa)	Shear Beams Compressive strength (MPa)
40.2	40.4
41.3	39.5
37.4	40.9
32.1	39.5
39.4	35.9
38.7	39.5
37.5	36.3
38.5	37.0
33.2	34.9
Average: 39.0	38.6

Table 4.2 Tension tests results of flexure beams steel reinforcements

Specimen Name	Elasticity Modulus(GPa)	Yield Stress(MPa)	Ultimate Stress (MPa)
# 6 bar	215.9	475.0	550.6
#10 bar	200.0	400.0	460.0
#15 bar	193.2	462.5	599.4

Table 4.3 Tension tests results of shear beams steel reinforcements

Specimen Name	Elasticity Modulus(GPa)	Yield Stress(MPa)	Ultimate Stress (MPa)
# 6 bar	158.5	283.0	286.0
	158.5	283.0	311.0
	Average 158.5	283.0	294.0
#10 bar	157.3	384.1	468.0
	148.1	382.1	473.2
	Average 152.7	383.1	470.6
#25 bar	195.2	520.0	677.8

Table 4.4 GFRP sheet properties

Specimen Name	Modulus of Elasticity (GPa)	Ultimate Stress (MPa)
100mm strip 1	61.0	548
100mm strip 2	66.2	528
Average	63.6	538

Table 4.5 GFRP laminate properties

Specimen Name	Modulus of Elasticity (GPa)	Ultimate Stress (MPa)
10a	27.0	640
10b	28.0	654
10c	27.2	600
10d	24.7	496
10e	25.2	475
10f	27.5	605
Average	27.4	625

Table 4.6 Forces and stresses applied to GFRP sheet (Flexure beams)

Specimen Name	Jacking		Transfer	
	Force (kN)	Stress (MPa)	Force (kN)	Stress (MPa)
B1P1 Layer 1	9.19	170	8.50	157
B1P2 Layer 1	13.60	252	12.46	231
B2P1 Layer 1	8.95	166	8.33	154
Layer 2	9.30	172	8.90	165
Average	9.12	169	8.66	160
B3P1 Layer 1	11.23	208	10.86	201
Layer 2	9.29	172	8.56	159
Layer 3	9.01	167	7.66	142
Average	9.84	182	9.03	167

Table 4.7 Forces and stresses applied to GFRP sheet (Shear beams)

Specimen Name	Jacking		Transfer	
	Force (kN)	Stress (MPa)	Force (kN)	Stress (MPa)
B2P2.5 Layer 1	20.7	230	19.0	211
Layer 2	19.5	217	18.7	208
Average	20.1	224	18.8	210
B3P2.5 Layer 1	18.9	210	12.9	143
Layer 2	21.0	233	19.8	220
Layer 3	20.7	230	19.7	219
Average	20.2	224	17.5	194
B1P3.0 Layer 1	19.3	215	18.0	200
B2P3.0 Layer 1	20.2	224	18.4	204
Layer 2	18.1	201	16.4	182
Average	19.2	213	17.4	193
B1P3.5 Layer 1	18.9	210	17.6	196
B2P3.5 Layer 1	18.3	203	17.0	189
Layer 2	20.1	223	18.7	208
Average	19.2	213	17.9	199

Table 4.8 Significant observed loads for flexure beams

Beam	Crack Load (kN)	Yielding Load (kN)	Ultimate Load (kN)
Control1	18.05	138.36	152.17
B1U	18.44	160.38	180.28
B1P1	29.85	167.38	185.65
B1P2	****	169.67	185.03
B2U	***	165.02	188.41
B2P1	***	183.41	210.84
Control2	15.35	107.10	127.74
B3P1	35.65	177.80	208.98

**** Pre-Cracked Beam

*** Oil painted Beams

Table 4.9 Increase in ultimate load carrying capacity with addition of GFRP sheets

Beam Name	Ultimate Load (kN)	%age increase over control
Group 1		
Control 1	149.5	-----
B1U	176	17.72%
B2U	188	25.75%
B1P1	184	23.08%
B2P1	208	39.13%
BIP2	182	21.74%
Group 2		
Control	127	-----
B3P1	206	62.20%

Table 4.10 Test results of shear critical beams

Specimen Name	Ultimate Shear Force (kN)		
	a/d = 2.5	a/d = 3.0	a/d = 3.5
Un-strengthened Control Beam	136.4	125.6	133.9
Beam strengthened with U-Straps	226.3	251.1	251.9
Beam strengthened with U-Jackets	358.9	313.1	290.9
Beam with one layer of prestressed sheet	-----	157.7	177.6
Beam with two layers of prestressed sheet	311.8	198.4	186.5
Beam with three layers of prestressed sheet	262.7	-----	-----

Table 4.11 Summation of crack widths at various loading for beam strengthened with one layer of GFRP sheet

Load (kN)	Summation of Cracks Widths (mm)		
	Control1	B1U	B1P1
0	0.00	0.00	0.00
50	0.20	0.14	0.02
60	0.46	0.28	0.04
70	0.56	0.44	0.08
80	0.72	0.44	0.16
90	0.80	0.62	0.34
108	0.98	0.82	0.54
125	1.58	1.16	0.78
142	2.04	1.68	1.16
152	4.76	1.84	1.34
176		5.1	2.36
184			4.06

Table 4.12 Summation of crack widths at various loading for beam strengthened with three layers of GFRP sheets

Load (kN)	Summation of Cracks Widths (mm)	
	Control2	B3P1
0	0.00	0.00
50	0.28	0.04
60	0.40	0.10
70	0.64	0.18
80	0.70	0.30
90	0.86	0.36
100	1.30	0.54
110	1.60	0.90
120	2.04	1.36
125	3.30	1.50
140		1.60
175		2.72
190		3.40
206		4.74

Table 4.13 Observed creep and shrinkage strains

Duration (Days)	Creep Strain ($\mu\epsilon$)	Shrinkage Strain ($\mu\epsilon$)	Total Strain ($\mu\epsilon$) (Creep + Shrinkage)
1	24	4	28
2	25	8	33
3	27	11	38
4	28	15	43
5	29	17	46
6	30	20	50
7	31	22	53
14	35	42	77
21	37	58	95
28	39	73	112
35	41	85	126
63	45	116	161
81	47	130	177
111	49	150	199
141	50	162	212
171	52	170	222

Table 4.14 Table showing total observed strain minus creep and shrinkage strain

Duration (Days)	Total strain Observed On Prestressed beam ($\mu\epsilon$)	Observed strain (Creep + shrinkage) ($\mu\epsilon$)	Strain due to Other reasons ($\mu\epsilon$)
1	46	28	18
2	66	33	33
3	76	38	38
4	87	43	44
5	92	46	46
6	100	50	50
7	106	53	53
14	146	77	69
21	169	95	74
28	200	112	88
35	216	126	90
63	257	161	96
81	275	177	98
111	293	199	94
141	310	212	98
171	319	222	97

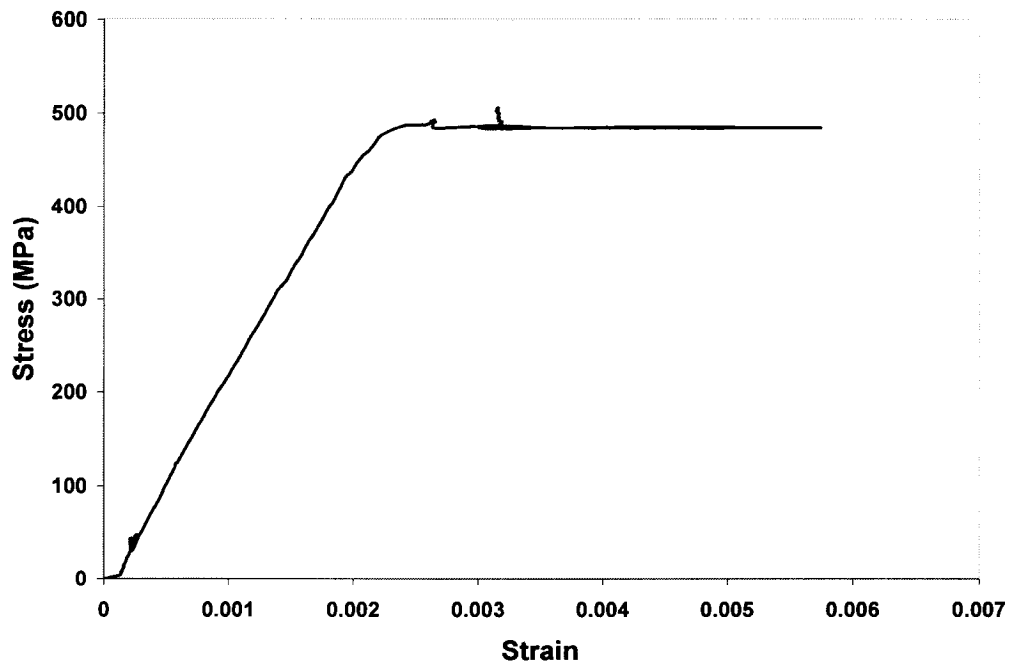


Fig. 4.1 Tensile stress-strain curve of No. 6 bar used in flexure beams

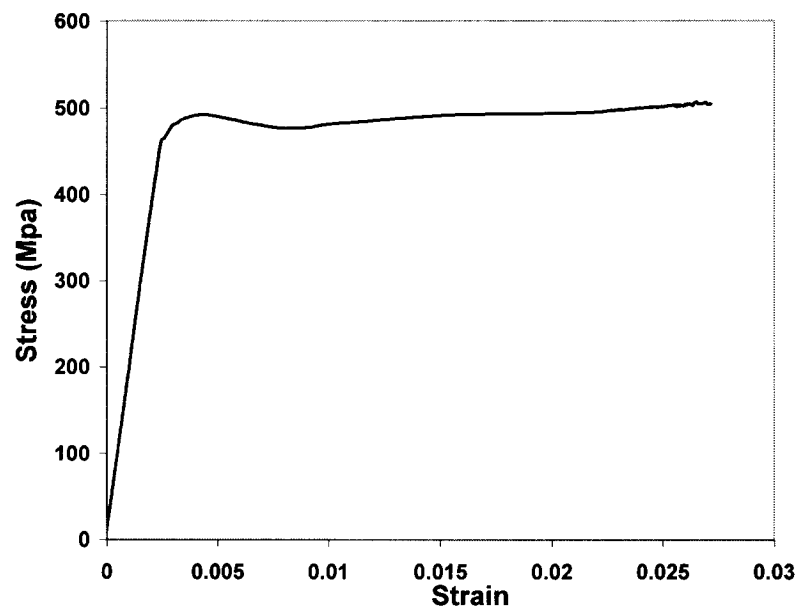


Fig. 4.2 Tensile stress-strain curve of No. 15 bar used in flexure beams

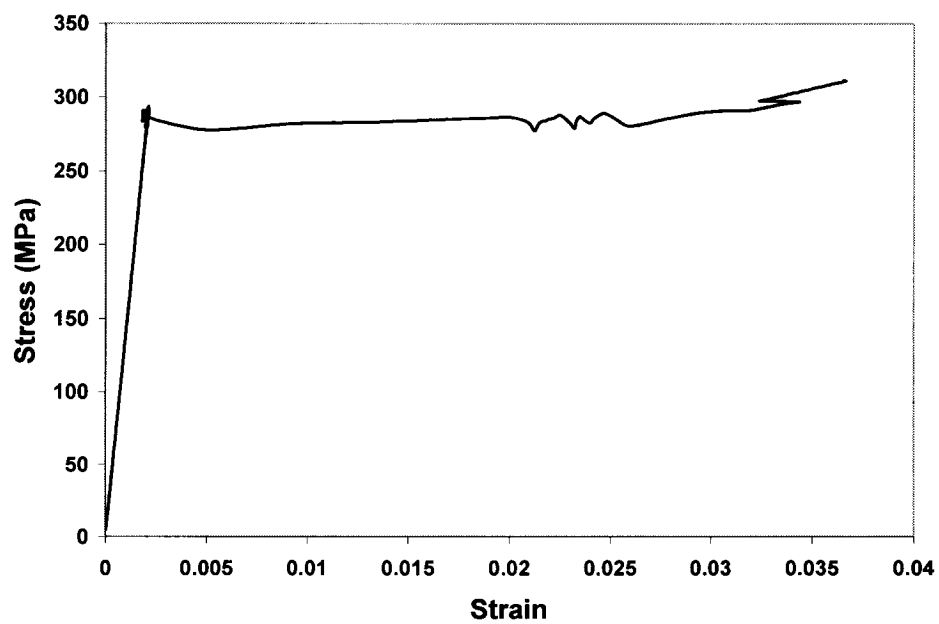


Fig. 4.3 Tensile stress-strain curves of No. 6 bars used in shear beams

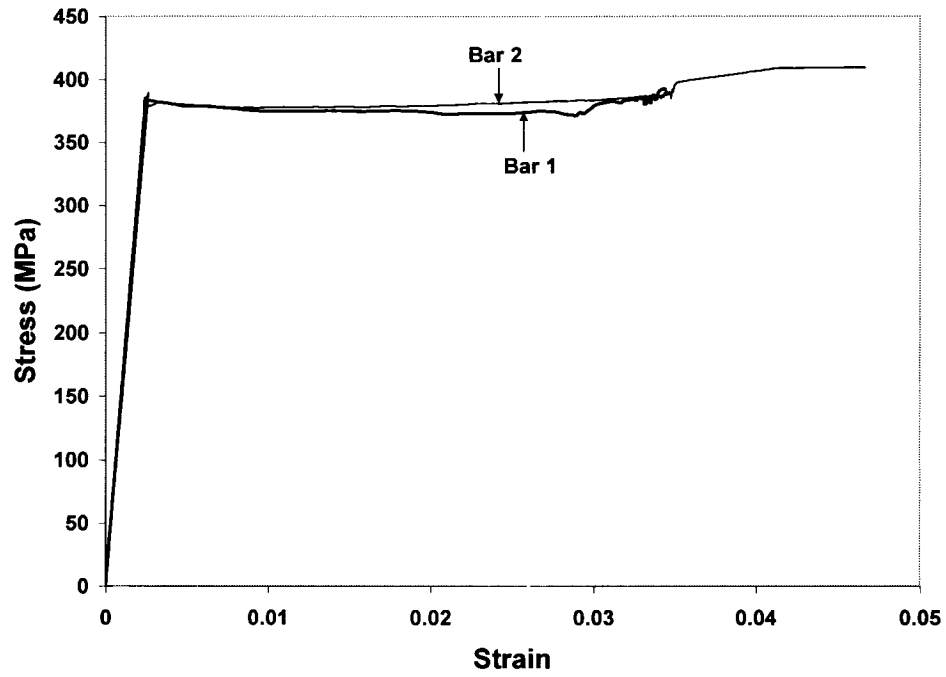


Fig. 4.4 Tensile stress-strain curves of No.10 bars used in shear beams

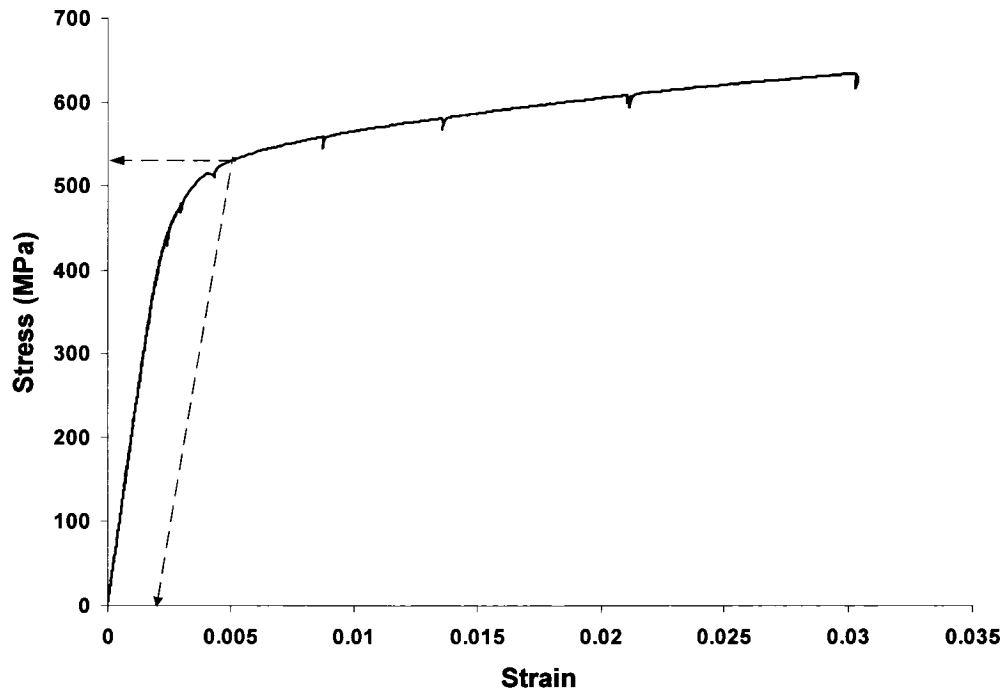


Fig. 4.5 Tensile stress-strain curve of No.25 bars used in shear beams

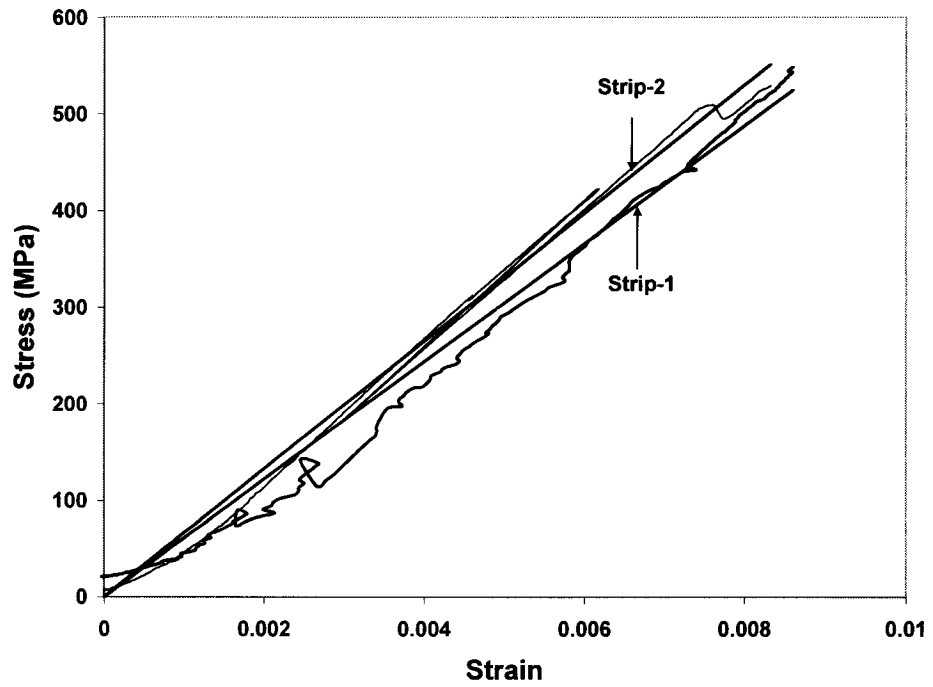


Fig. 4.6 Tensile stress-strain curves of GFRP sheets

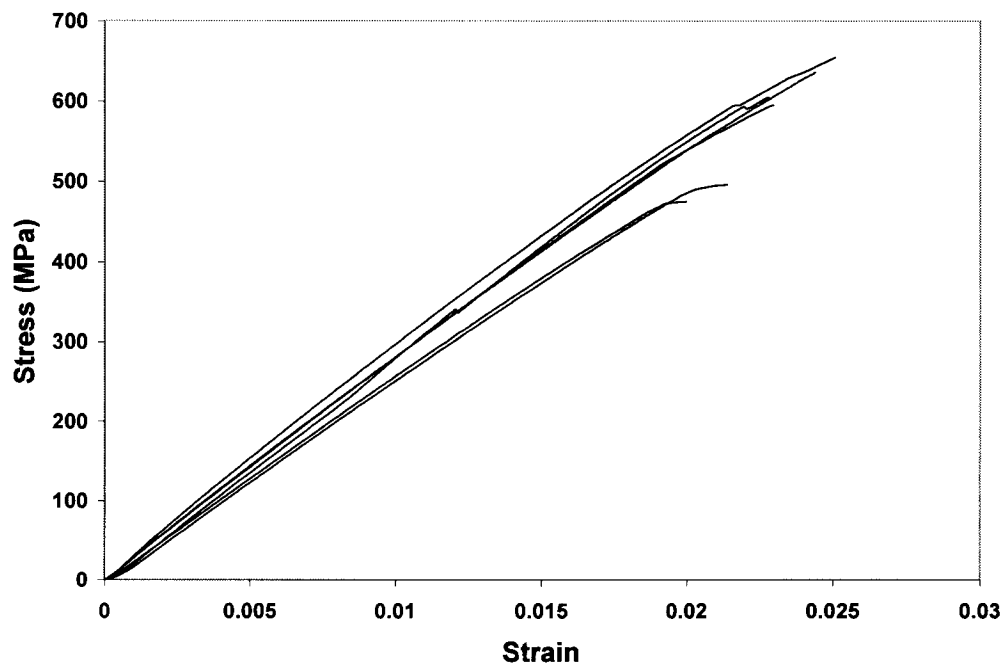


Fig. 4.7 Tensile stress-strain curves of GFRP laminates

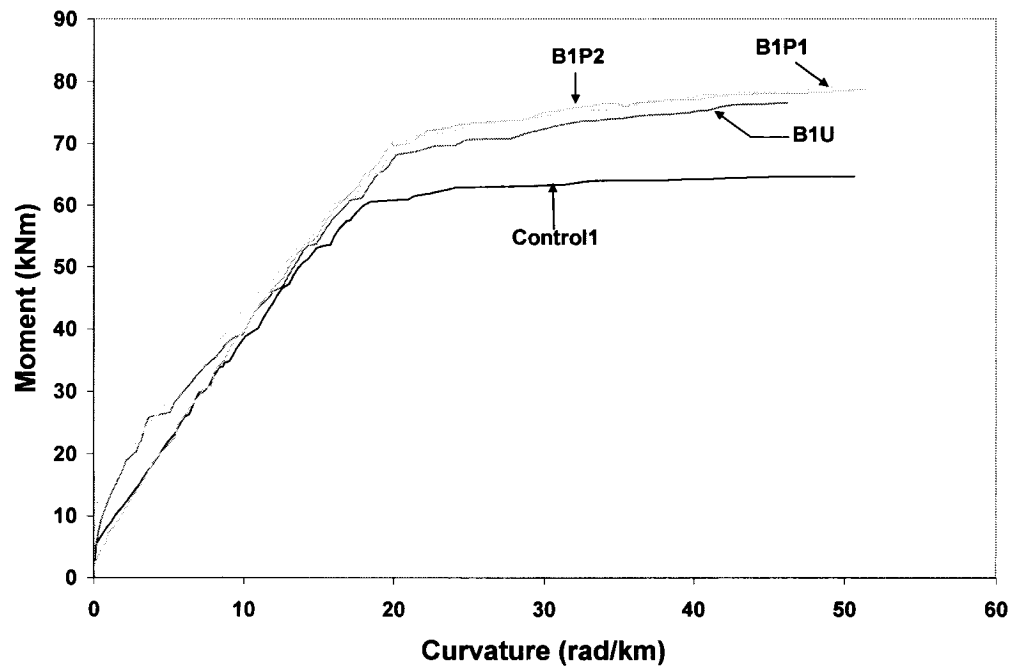


Fig. 4.8 Moment- curvature curves of control beam and beams strengthened with one layer of GFRP sheet.

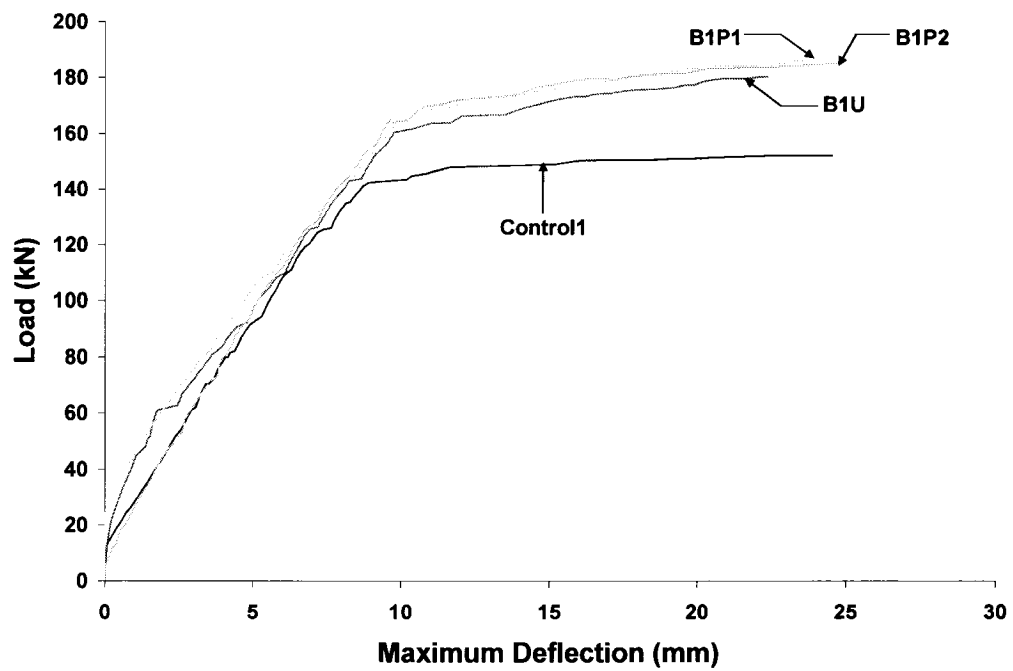


Fig. 4.9 Load – mid span deflection curves of control beam and beams strengthened with one layer of GFRP sheet.

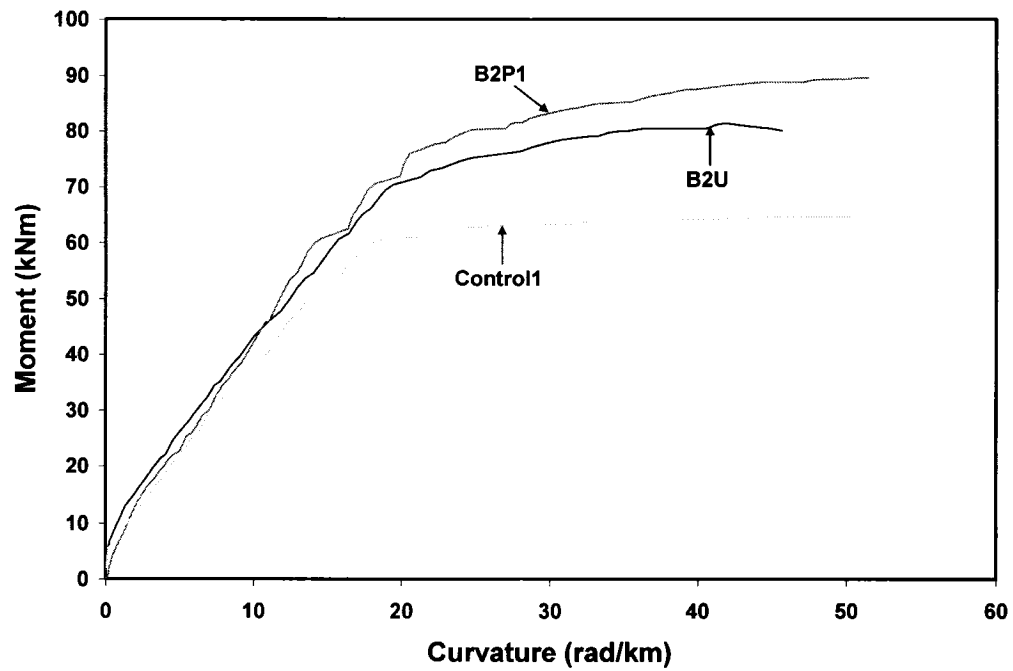


Fig. 4.10 Moment- curvature curves of control beam and beams strengthened with two layers of GFRP sheet.

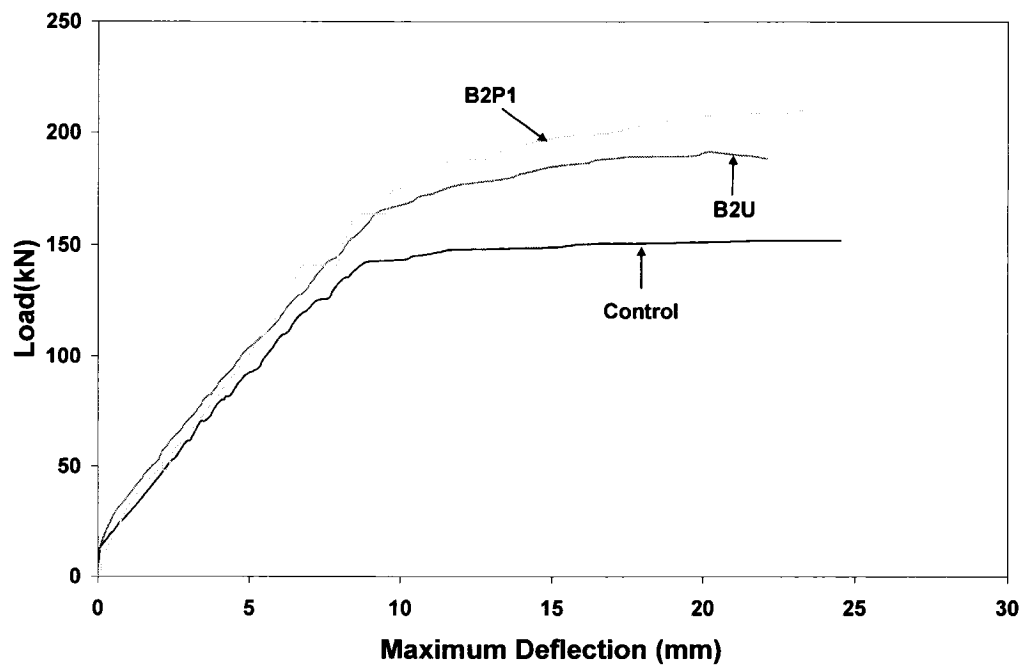


Fig. 4.11 Load – mid span deflection curves of control beam and beams strengthened with two layers of GFRP sheets.

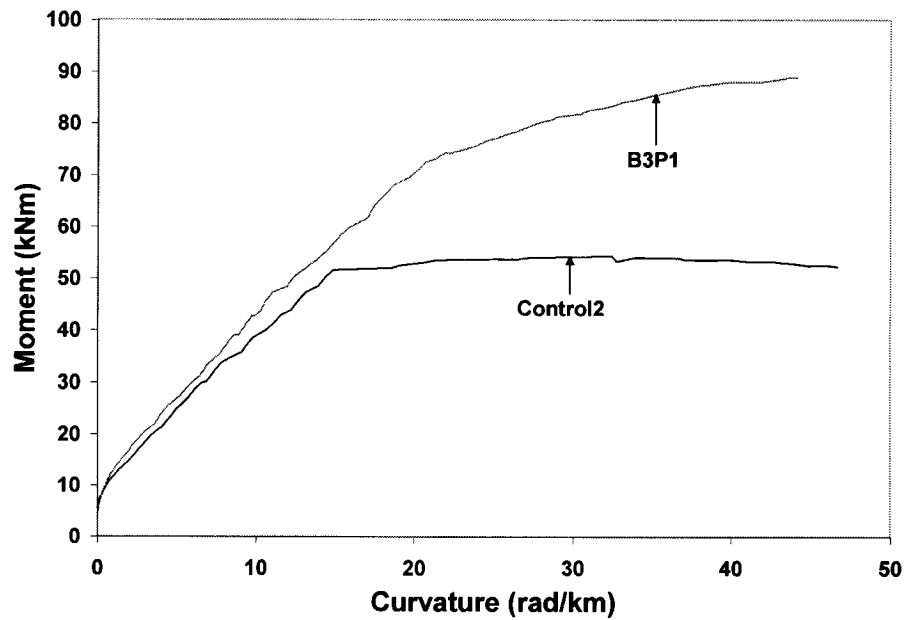


Fig. 4.12 Moment- curvature curves of control beam and beam strengthened with three layers of GFRP sheets.

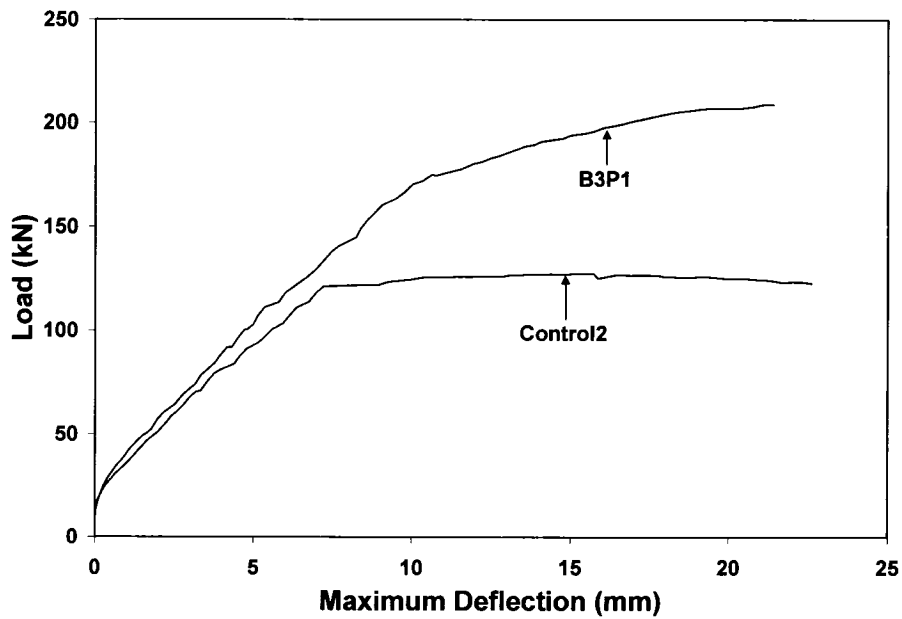


Fig. 4.13 Load – mid span deflection curves of control beam and beam strengthened with three layers of GFRP sheet.

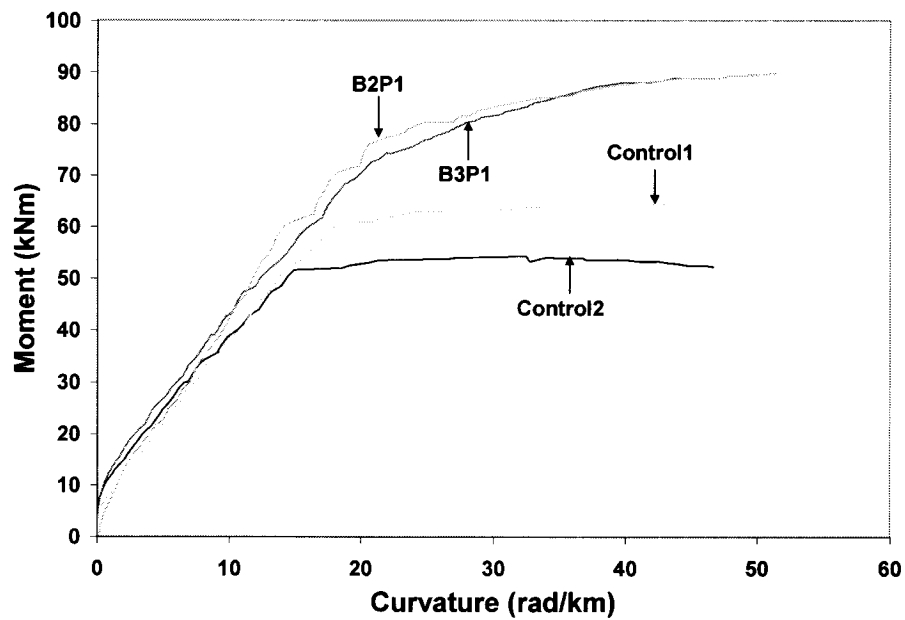


Fig. 4.14 Moment- curvature curves of control beams and beams strengthened with two and three layers of GFRP sheet.

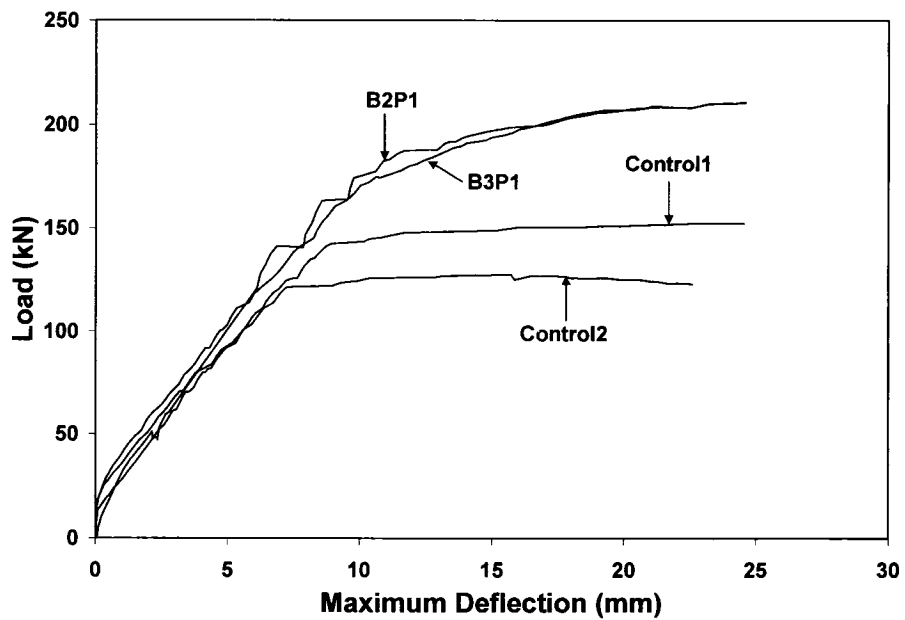


Fig. 4.15 Load – mid span deflection curves of control beams and beams strengthened with two and three layers of GFRP sheets.

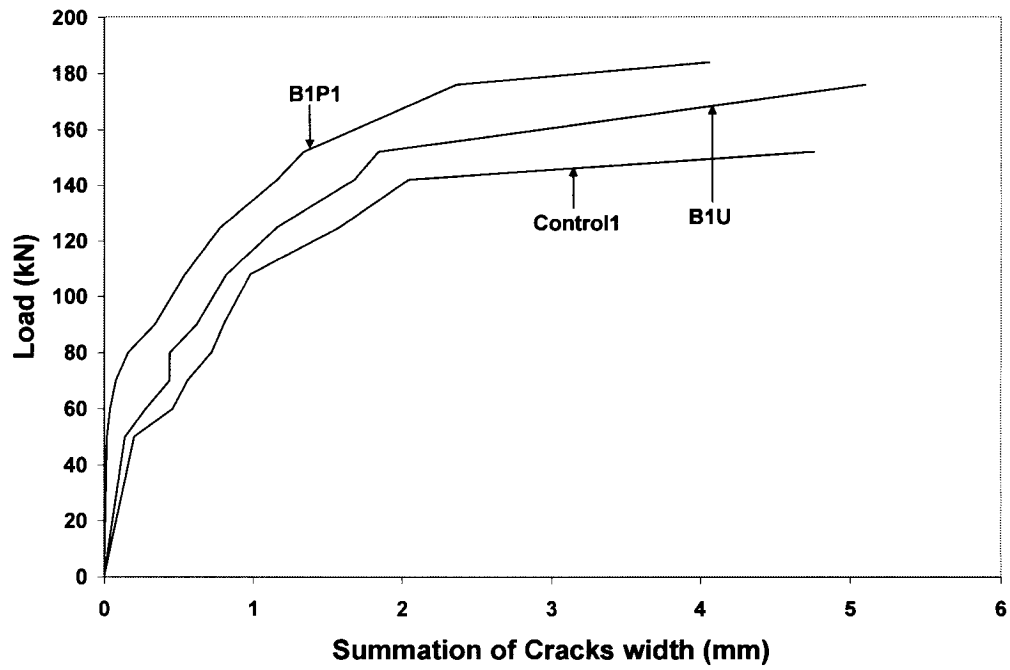


Fig. 4.16 Load – summation of cracks widths curves of control beam and beams strengthened with one layer of GFRP sheet.

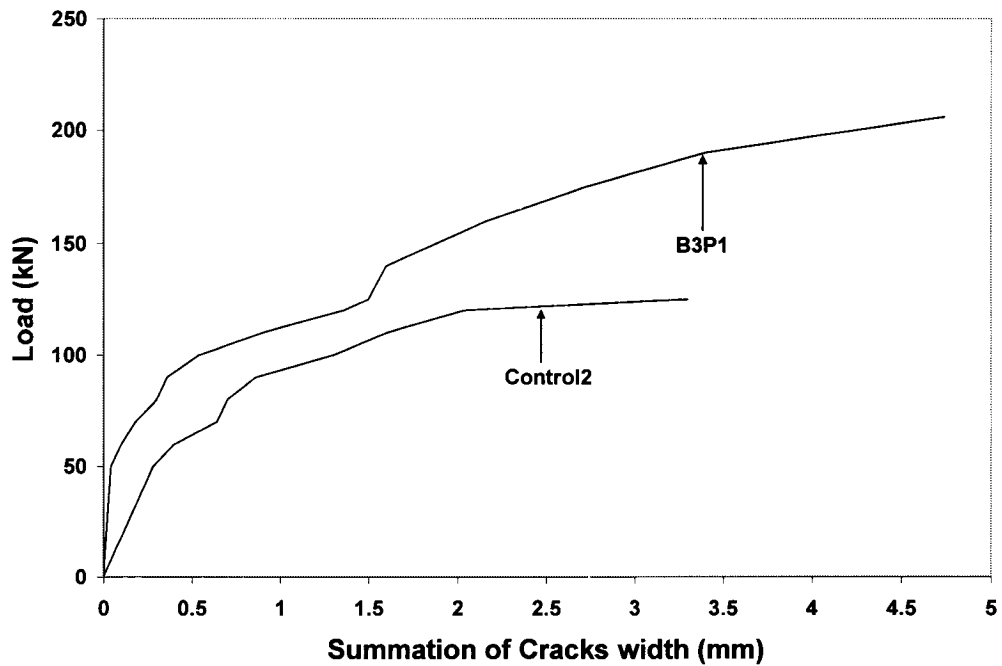


Fig. 4.17 Load – summation of crack widths curves of control beam and beam strengthened with three layer of GFRP sheet.

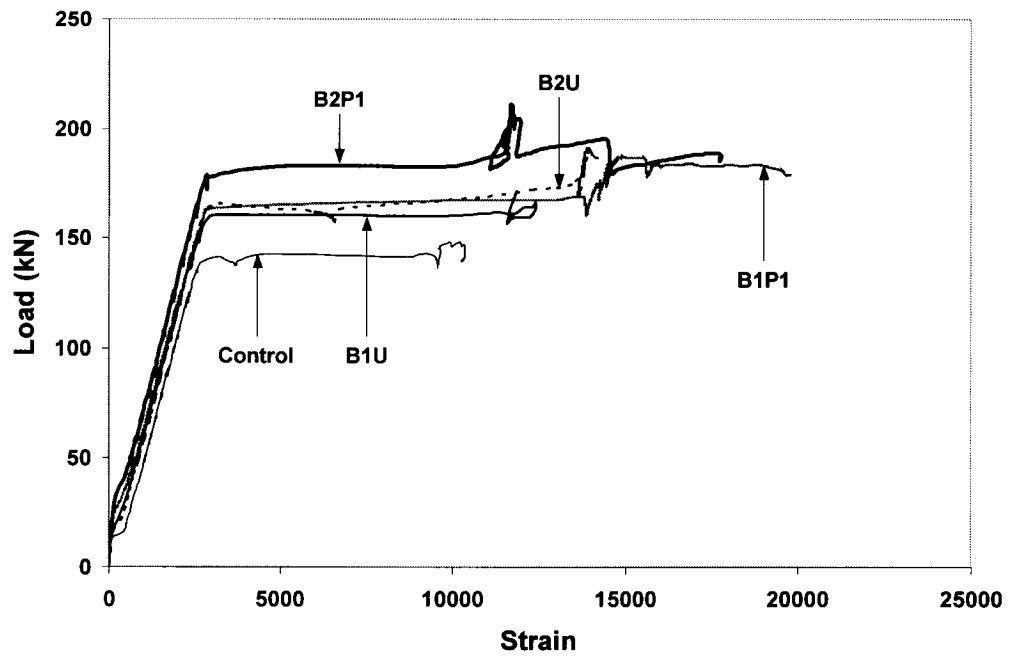


Fig. 4.18 Tension steel strains at mid span vs. load curves of Group-1 beams

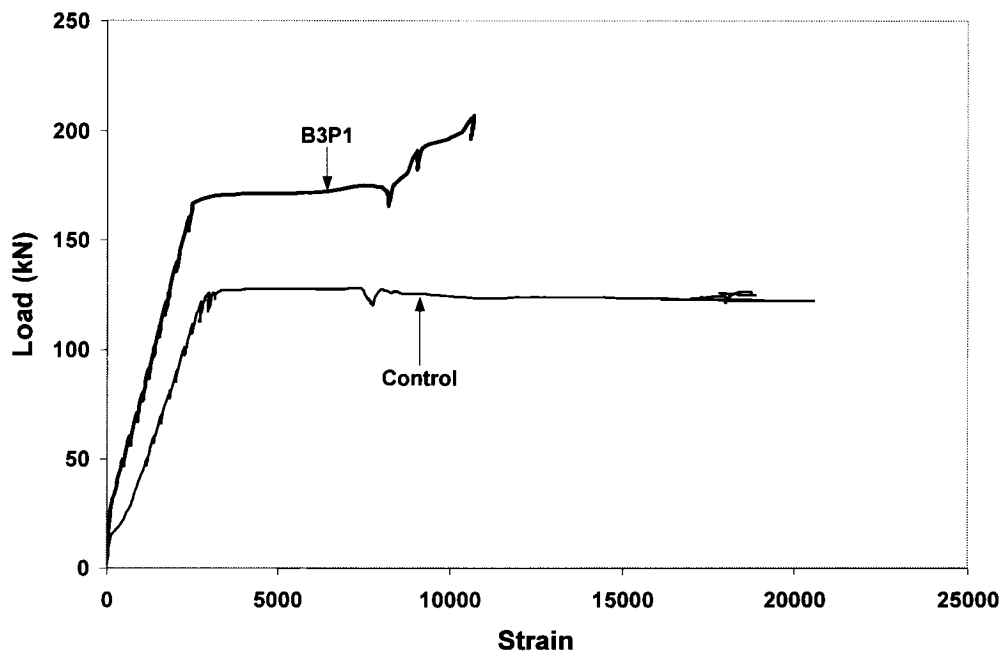


Fig. 4.19 Tension steel strains at mid span vs. load curves of Group-2 beams

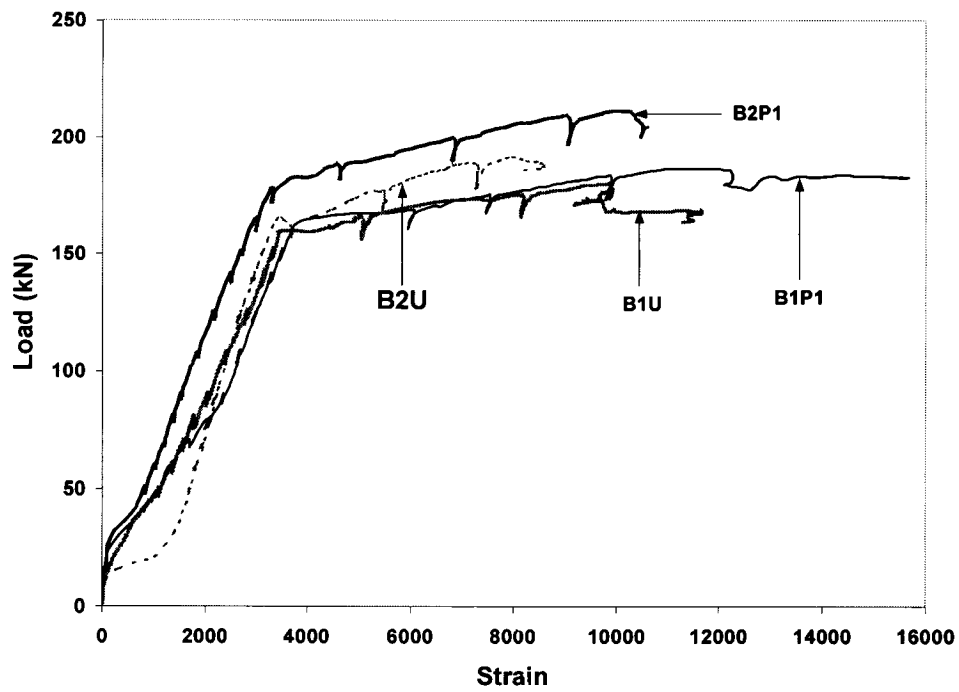


Fig. 4.20 GFRP sheet strains at mid span vs. load curves

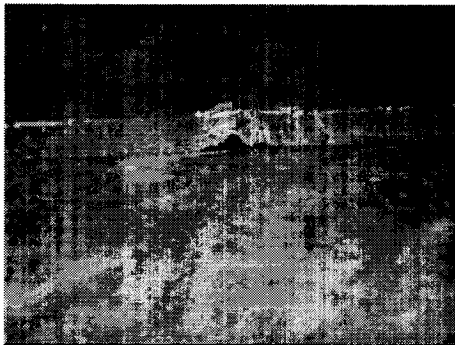


Fig. 4.21 Be-bonded GFRP sheet

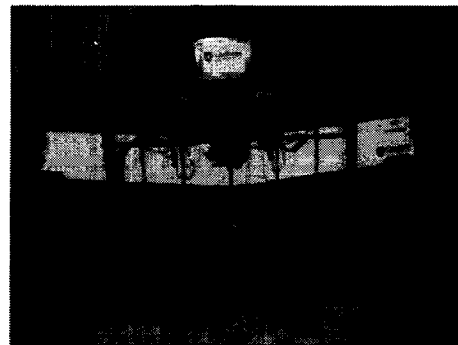


Fig. 4.22 Failure of Control-1 Beam

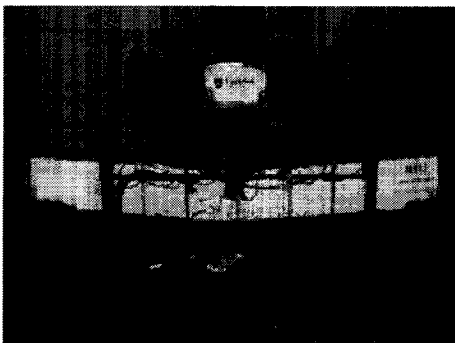


Fig. 4.23 Failure of Beam B1U

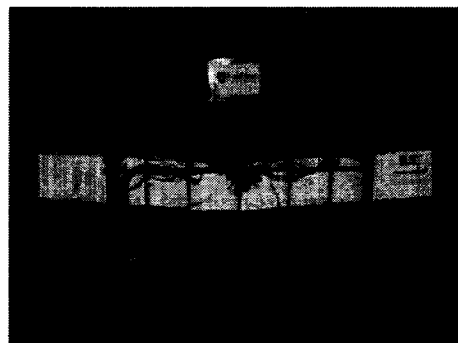


Fig. 4.24 Failure of Beam B2U

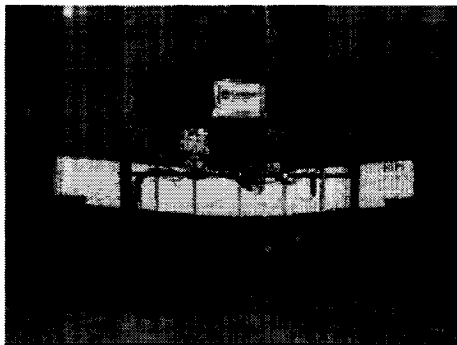


Fig. 4.25 Failure of Beam B1P1

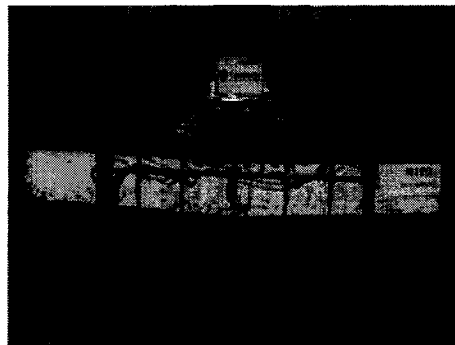


Fig. 4.26 Failure of Beam B1P2

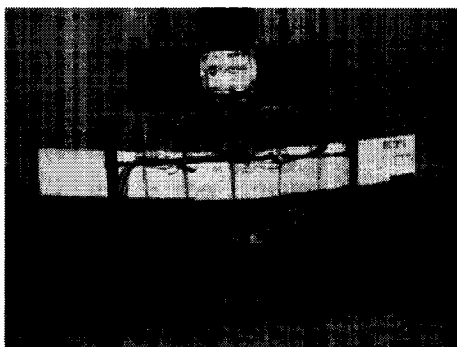


Fig. 4.27 Failure of Beam B2P1

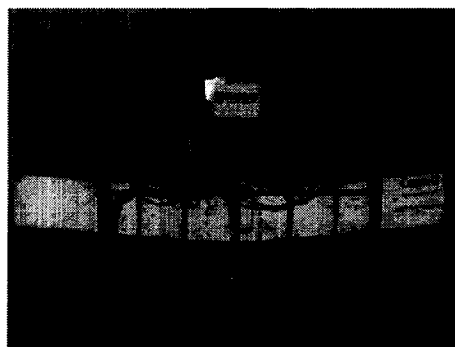


Fig. 4.28 Failure of Control-2Beam

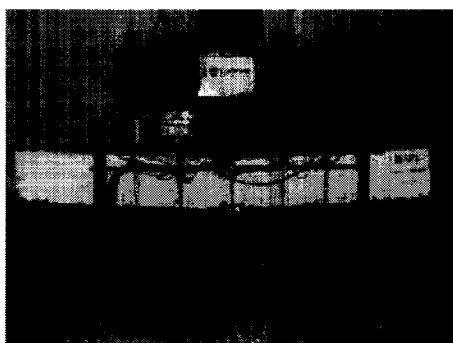


Fig. 4.29 Failure of Beam B3P1

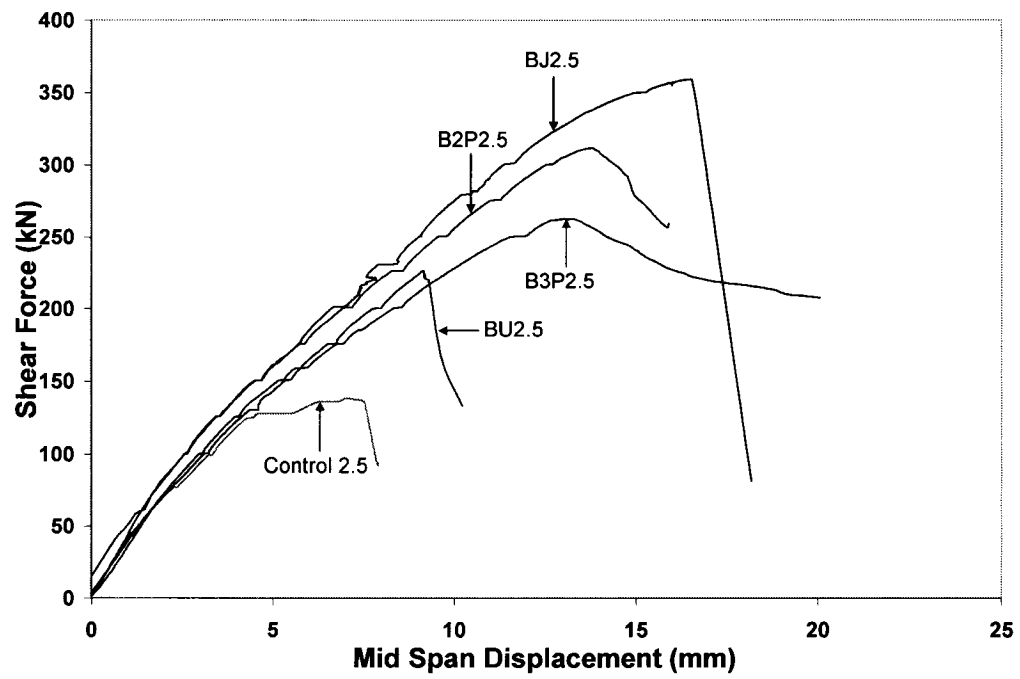


Fig. 4.30 Load - mid span deflection curves of beam test over a/d ratio of 2.5

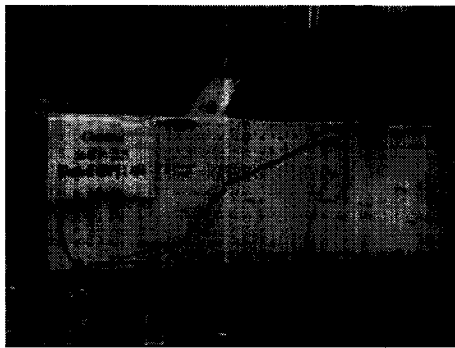


Fig. 4.31 Concrete failure over support in beam Control-2.5

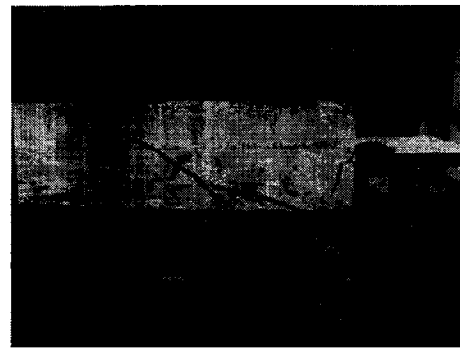


Fig. 4.32 Concrete failure over support in beam B3P2.5



Fig. 4.33 Failure of Beam Control2.5



Fig. 4.34 Failure of Beam BU2.5

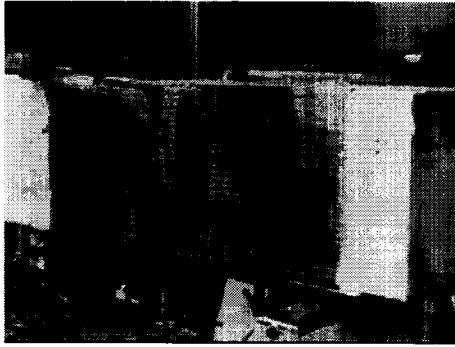


Fig. 4.35 Failure of Beam BJ2.5

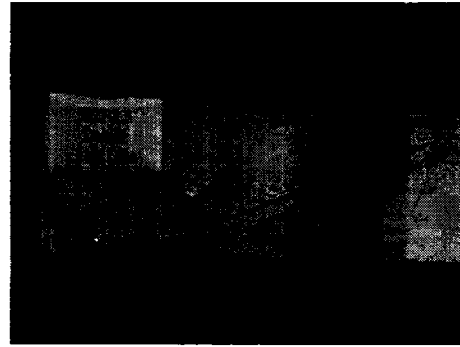


Fig. 4.36 Failure of Beam B2P2.5

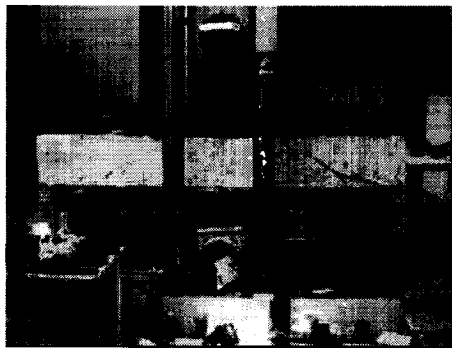


Fig. 4.37 Failure of Beam B3P2.5

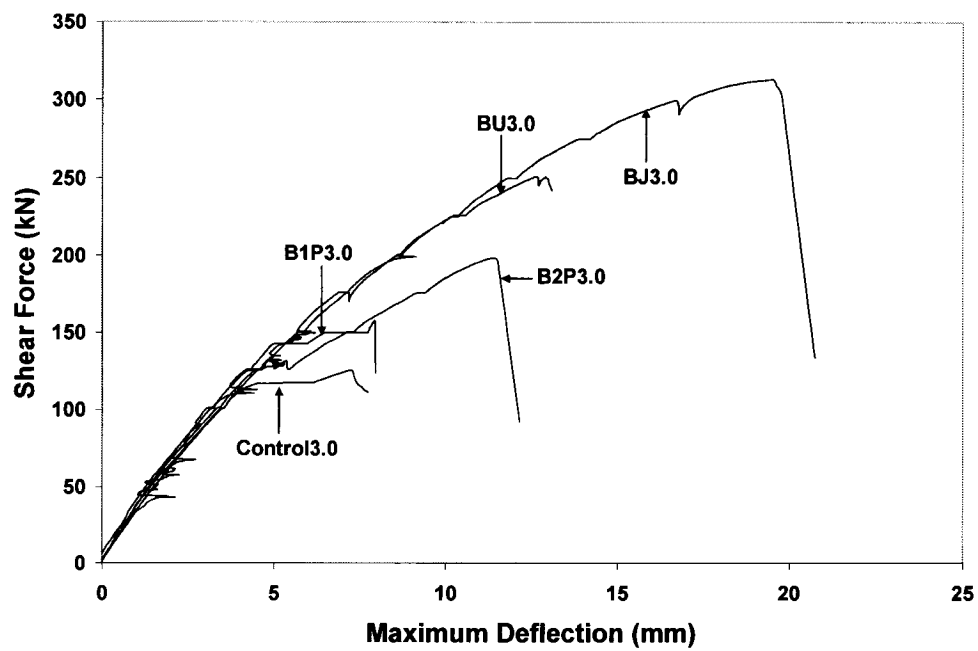


Fig. 4.38 Load - mid span deflection curves of beam test over a/d ratio of 3.0



Fig. 4.39 Failure of Beam Control3.0



Fig. 4.40 Failure of Beam BU3.0



Fig. 4.41 Failure of Beam BJ3.0

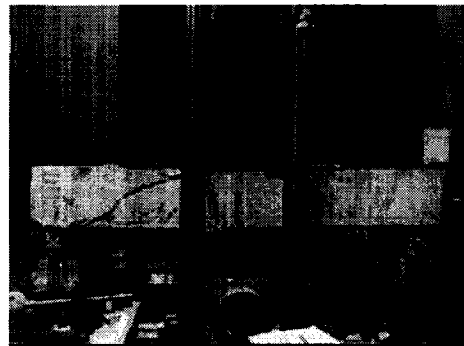


Fig. 4.42 Failure of Beam B1P3.0



Fig. 4.43 Failure of Beam B2P3.0

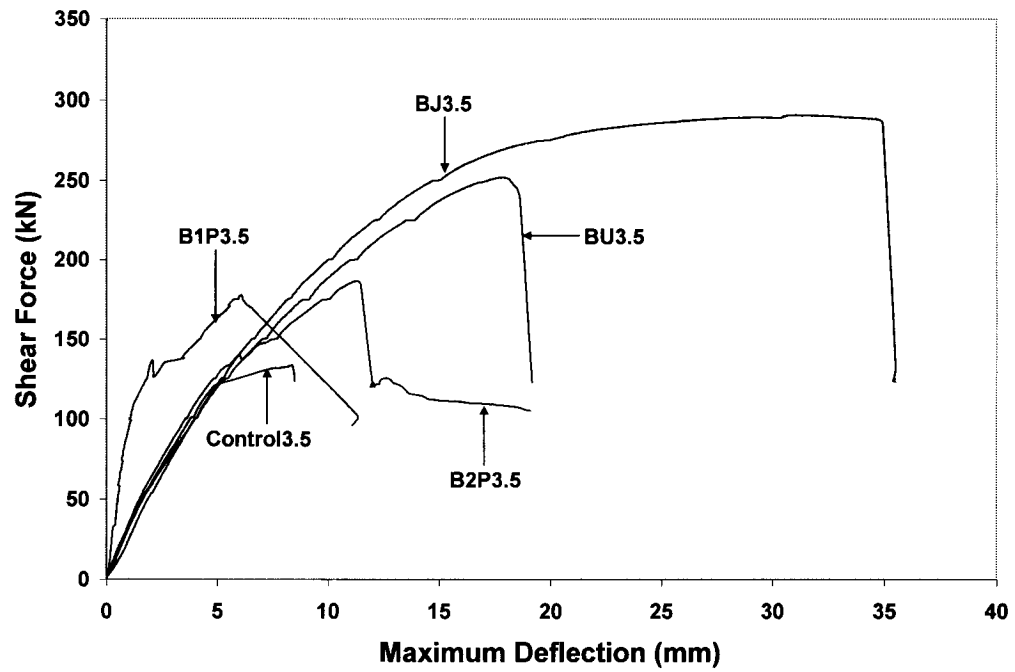


Fig. 4.44 Load - mid span deflection curves of beam test over a/d ratio of 3.5

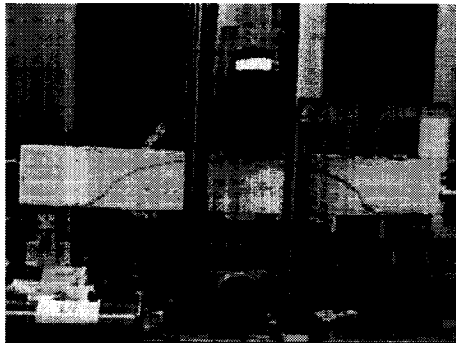


Fig. 4.45 Failure of Beam Control3.5

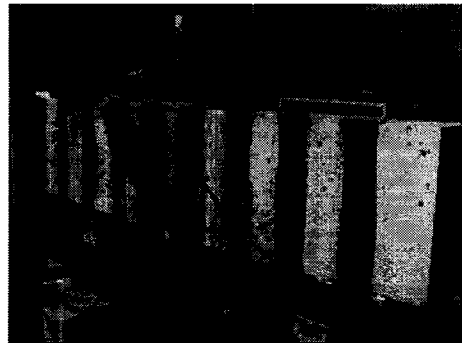


Fig. 4.46 Failure of Beam BU3.5



Fig. 4.47 Failure of Beam BJ3.5

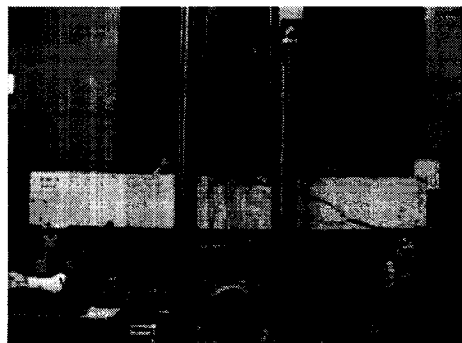


Fig. 4.48 Failure of Beam B1P3.5

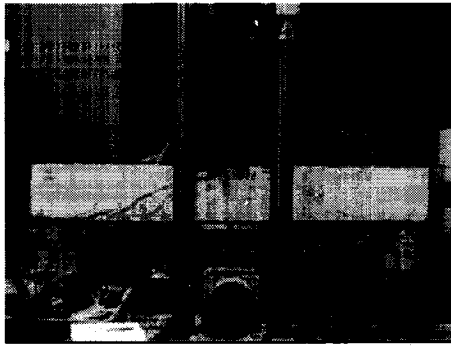


Fig. 4.49 Failure of Beam B2P3.5

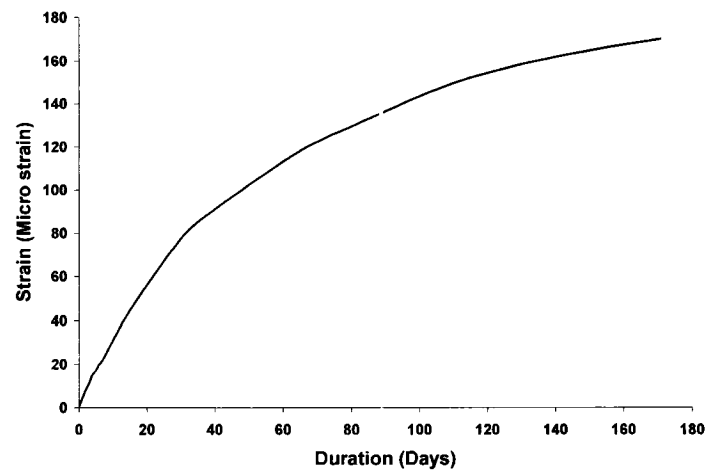


Fig. 4.50 Time-shrinkage strain curve

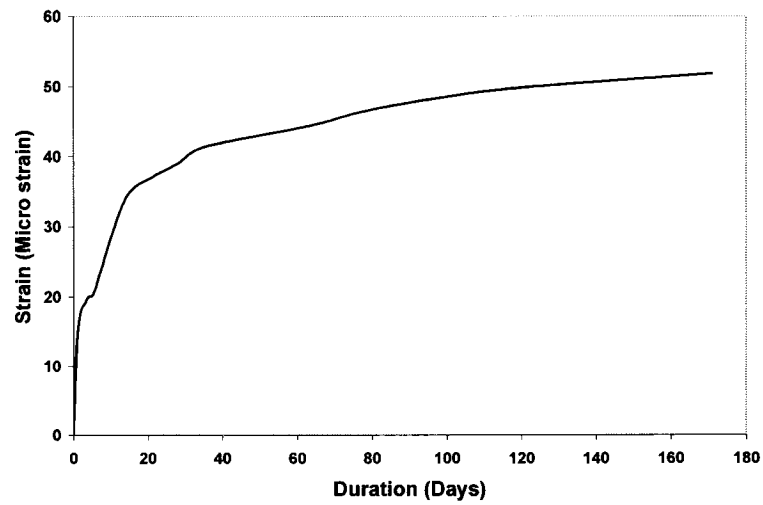


Fig. 4.51 Time-Creep strain curve

CHAPTER 5

THEORETICAL MODEL

5.1 General

To investigate the flexural and shear behaviour of the strengthened reinforced concrete beam, an experimental program was outlined in chapter 3 and its results were presented in chapter 4. This chapter presents the details of analytical research on predicting the flexural and shear behaviour of reinforced concrete beams under static loading. The reinforced concrete beams used for studying the flexural behaviour were either un-strengthened or strengthened with prestressed or non-prestressed GFRP sheet, and those used for studying the shear behaviour were either un-strengthened or strengthened with prestressed GFRP sheets attached to the flexural face of the beam or wrapped with GFRP straps/jackets in the shear span of the beam. The work in this chapter can be divided into four main topics:

- Description of the computer software and models used for the analytical study
- Modelling and static inelastic analysis of all eight flexural test specimens
- Modelling and static inelastic analysis of all fifteen shear test specimens
- Description of model for prediction of long term losses

All analyses were conducted using the computer software “Response-2000” (Bentz et al. 2000) developed at the University of Toronto. Response-2000 is a sectional analysis program which uses the Modified Compression Field Theory (MCFT) to model the behaviour of cracked reinforced concrete elements subjected to shear.

5.2 Computer Program Response 2000

Response 2000, developed by Bentz et al at the University of Toronto, is a useful computer program which performs non-linear sectional analysis. Response 2000 is the successor of program “Response” developed by Felber in 1990 and Program “Sma1” developed by Ho in 1994 at the University of Toronto. Response 2000 can calculate strengths and deformations of beams and columns subjected to arbitrary combinations of axial load, moment and shear. The task of determining the response of a reinforced concrete beam by using sectional analysis approach can be divided into two steps. First, the sectional forces at desired locations in the structure induced by the applied loads are determined by assuming that the structure remains linear elastic. In the second step, the response of the local section to these sectional forces is determined by considering the non-linear characteristic of the cracked reinforced concrete. The two basic assumptions in Response 2000 are outlined as below:

- The first modelling assumption in the implementation of sectional analysis including shear is that engineering beam theory is valid i.e. the strain distribution is linear through the depth of the section. This mean that the longitudinal strain in concrete, reinforcing steel and GFRP sheet at various depths across the section is assumed directly proportional to the distance from the neutral axis provided perfect bond exists between concrete, reinforcing steel and GFRP sheet.
- The second assumption is that there is no significant net stress in the transverse direction. This means that the concrete and transverse steel forces must balance at each point trough the depth of the element.

Both these assumptions are good ones when the analysis is being performed a distance apart from the support and the load points. Close to the load and to the

reaction, however, there will be a transverse clamping stress from application of the load itself. This clamping stress will tend to locally increase the shear strength. This is one reason that short beams are noticeably stronger in shear than long beams with the same cross-section and transverse reinforcement. Similarly, in case of prestressed beams the clamping stresses are more pronounced in the middle region of the shear span i.e. between the load point and support, than the reinforced concrete beams. This implies that a sectional model of a prestressed beam would be more conservative in predicting the shear strength than a reinforced concrete beam as the analysis would ignore the beneficial effect of this small compressive clamping stress. Due to the aforementioned reason the response 2000 program needs that the structural element be reasonably long for adequately modelling the behaviour of a real element. For example, the shear span of a beam must be at least 2 times the depth of the beam in order for shear analysis based on sectional model to be accurate. In case of shorter beams and prestressed beams, this model will under estimate the shear strength.

Response 2000 can perform analysis without shear i.e. flexural analysis and analysis with shear. To adequately model the full member response, the program includes the method to integrate the sectional behaviour for simple prismatic beam segments. The analysis will calculate an entire moment-shear interaction diagram and determine the load deflection properties and crack diagram for the entire half span of the beam. Response 2000 uses the Modified Compression Field Theory (MCFT) to model the behaviour of cracked reinforced concrete elements subjected to shear. This is perhaps the most important point which differentiates Response 2000 from other sectional analysis programs. The MCFT is described in more detail in the next section.

5.3 Modified Compression Field Theory (MCFT)

The MCFT is a general model for the load-deformation behaviour of two-dimensional cracked reinforced concrete subjected to shear. It models concrete considering concrete stresses in principal directions summed with reinforcing stresses assumed to be only axial. The concrete stress-strain relationship in compression and tension, used in this model, was originally derived from Vecchio's tests and has since been confirmed with about 250 test experiments performed on two large special purpose testing machines at the University of Toronto (Bentz, 2000).

The basic assumption of MCFT modelled is that the cracked concrete in reinforced concrete structural element can be model as a new material. The stress-strain relationship of this new material can be defined empirically which can differ from traditional stress-strain curve of a concrete cylinder. This is possible because the strains and stresses used in this model are average strains and stresses. The average strain used in this model lumps together the effects of local strains at cracks, strains between cracks, bond slip and crack slip. Similarly the average stresses include the combined effect of stresses between the cracks, stresses at the cracks, interface shear at the crack and dowel action. To use the assumption of average stresses and strains it is necessary that the distance between two sections should be reasonably long to include few cracks in between them. Response 2000 satisfies this assumption by modeling the elements which are at least a couple of sections depth long. The use of average stress-strain relationship warrants an explicit crack check which ensures that average stresses are compatible to the actual cracked condition of the concrete. This crack check is a critical part of MCFT, which involves limitation of average principal tensile stress in concrete to a maximum allowable value. This maximum allowable

value is determined by considering the steel stress at a crack and the ability of the cracked surface to resist shear stresses.

The MCFT does not require explicit calculation of stress components in cracked reinforced concrete element like dowel action forces, shear stresses on cracks, reinforcing stresses at a crack, crack slip stresses, and bond stresses but solely depends on average the stresses condition. If required, these parameters may be calculated from condition of equilibrium. Consideration of average stress-strain relationship makes MCFT method very simple by ignoring the more complex phenomena in cracked concrete. The MCFT also assumes that the angle of inclination of principal concrete stress to the x-axis can be taken as equal to the angle of inclination of principal strain to the x-axis (θ).

Cracked reinforced concrete element subjected to externally applied normal and shear stresses is shown in Fig. 5.1. The following equations of average stresses can be derived simply from Mohr's circle of stress by considering the condition of equilibrium in case of MCFT for two-dimensional case.

Average Stresses:

$$\rho_x f_{sx} = f_x + \nu \cot \theta - f_1 \quad \text{Eq. 5.1}$$

$$\rho_y f_{sy} = f_y + \nu \tan \theta - f_1 \quad \text{Eq. 5.2}$$

$$f_2 = \nu(\tan \theta + \cot \theta) - f_1 \quad \text{Eq. 5.3}$$

The stresses at the cracks can be calculated from the following equations:

$$\rho_x f_{sxcr} = f_x + \nu \cot \theta + \nu_{ci} \cot \theta \quad \text{Eq. 5.4}$$

$$\rho_y f_{sy cr} = f_y + \nu \tan \theta - \nu_{ci} \tan \theta \quad \text{Eq. 5.5}$$

Where:

ρ_x = reinforcement ratio of x-reinforcement

ρ_y = reinforcement ratio of y-reinforcement

f_{sx} = average stress in x-reinforcement

f_{sy} = average stress in y-reinforcement

f_x = stress applied to element in x-direction

f_y = stress applied to element in y-direction

ν = shear applied to the element

θ = angle of inclination of principal stress to x-axis

f_1 = principal tensile stress in concrete

f_2 = principal compressive stress in concrete

f_{sxcr} = stress in x-reinforcement at crack location

f_{sy-cr} = stress in y-reinforcement at crack location

ν_{ci} = shear on crack surface

Fig. 5.2 shows the strain condition of a cracked reinforced concrete element. The following equations of average normal and shear strains can be derived from Mohr's circle of strain by considering compatibility condition in the two-dimensional case.

Average strains:

$$\varepsilon_x = (\varepsilon_1 \tan^2 \theta + \varepsilon_2) / (1 + \tan^2 \theta) \quad \text{Eq. 5.6}$$

$$\varepsilon_y = (\varepsilon_1 + \varepsilon_2 \tan^2 \theta) / (1 + \tan^2 \theta) \quad \text{Eq. 5.7}$$

$$\gamma_{xy} = 2(\varepsilon_x - \varepsilon_2) / \tan \theta \quad \text{Eq. 5.8}$$

$$\tan^2 \theta = (\varepsilon_x - \varepsilon_2) / (\varepsilon_y - \varepsilon_2) \quad \text{Eq. 5.9}$$

The crack width can be calculated from the following equation;

$$w = s_\theta \varepsilon_1 \quad \text{where;} \quad \text{Eq. 5.10}$$

$$s_{\theta} = 1 / \left(\frac{\sin \theta}{s_x} + \frac{\cos \theta}{s_y} \right) \quad \text{Eq. 5.11}$$

Where

ε_x = strain in x-direction

ε_y = strain in y-direction

ε_1 = principal tensile strain in concrete

ε_2 = principal compressive strain in concrete

θ = angle of inclination of principal strain to x-direction

γ_{xy} = shear strain relative to x and y axes

w = crack width

s_{θ} = spacing of cracks inclined at θ

s_x = crack spacing in x-direction

s_y = crack spacing in y-direction

Some of the above term are most important and has been discussed in detail in the following sections;

5.3.1 Average stresses in reinforcement(f_{sx}, f_{sy})

The MCFT assumes average behaviour of reinforcement. This behaviour can be approximated by the bare-bar response. The average stress-average strain relationships of reinforcement used in MCFT is given by the following equation;

$$f_{sx} = E_s \varepsilon_x \leq f_{syield} \quad \text{Eq. 5.12}$$

$$f_{sy} = E_s \varepsilon_y \leq f_{syield} \quad \text{Eq. 5.13}$$

where;

E_s = modulus of elasticity of reinforcement

f_{yield} = yielding stress of reinforcement

5.3.2 Concrete tensile stress response(f_1)

Concrete in tension is assumed to act linearly elastic until first cracking with stiffness equal to the initial compression tangent stiffness. Response 2000 uses the following equation for cracking strength of concrete (MPa units):

$$f_t = 0.45(f_c')^{0.4} \quad \text{Eq. 5.14}$$

where;

f_t = tensile strength of concrete

f_c' = maximum compressive stress observed in concrete cylinder test

After cracking, tensile stresses in the un-cracked concrete between the cracks will continue to stiffen the concrete, and in some cases will increase the strength. To model the post cracked and pre-reinforcement yielding behaviour of tension stiffening, Response 2000 uses the following simple equation for concrete tensile stress response formulated by Collins and Mitchell in 1987.

$$f_1 = \frac{f_{cr}}{1 + \sqrt{500\varepsilon_1}} \quad \text{Eq. 5.15}$$

where;

f_{cr} = stress in concrete at cracking

5.3.3 Concrete Compressive stress response(f_2)

It is assumed in this model that the un-cracked concrete in compression will follow the cylinder stress strain curve. The following equation has been used in MCFT to model the concrete compressive response;

$$f_2 = \frac{f_c'}{0.8 + 170\varepsilon_1} \left[2 \frac{\varepsilon_2}{\varepsilon_c'} - \left(\frac{\varepsilon_2}{\varepsilon_c'} \right)^2 \right] \quad \text{Eq. 5.16}$$

where;

ε'_c = strain in concrete cylinder at peak stress f'_c

It is clear from the equation that concrete compressive stress response is a function of the principal compressive strain (ε_2) and the principal tensile strain (ε_1). The tensile strain component models the decrease in concrete compressive strength in case of transverse cracking.

5.3.4 Crack width(w)

When a reinforced concrete element is subjected to shear, new cracks may form; the old cracks may close or become inactive. To simplify this complex load-history behaviour, only a single set of parallel cracks forming at the average angle of principal compressive strain or stress is assumed in this model. The crack spacing (s_θ) can be calculated by eq. 5.11 which converts the calculated crack spacing in two orthogonal directions into an estimated diagonal spacing. Crack widths are assumed to be simply the product of the principal tensile strain and the crack spacing. This model ignores the elastic strains in the un-cracked concrete between the cracks.

5.3.5 Shear on the Crack(v_{ci})

This model assumes a limit on the maximum interface shear on a crack (v_{ci}) that can be transferred before the crack begins to slip. The following equation is used in this model for this purpose;

$$v_{ci} \leq \frac{0.18\sqrt{f'_c}}{0.31 + \frac{24w}{a+16}} \quad \text{Eq. 5.17}$$

Where;

a = maximum aggregate size used in concrete

The shear on the crack interface will be higher for stronger concrete or larger aggregate whereas it will be lesser for higher crack widths. Further detail on MCFT and computer software Response 2000 can be found elsewhere (Bentz 2000).

5.4 Modelling of Beams

Response 2000 uses the MCFT formulation to analyze a reinforced or prestressed concrete beam. It considers that a beam is composed of a series of concrete layers, longitudinal steel elements, and longitudinal GFRP sheet elements. This layer by layer approach is used by Collins and Mitchell (1987) for predicting the response of reinforced concrete beam subjected to shear using the MCFT. In this approach each layer of concrete is defined by its width (b), depth (h), amount of transverse reinforcement (ρ_v), position of layer relative to the top of beam (y_c). The properties of constituent material for each layer include concrete cylinder strength (f'_c), concrete strain at peak stress (ϵ'_c), maximum size of aggregate used in concrete (a), yield stress of transverse reinforcement (f_{vy}), and young modulus of transverse steel (E_{vs}). The non-prestressed longitudinal steel elements are defined by their cross-sectional area (A_s), and position of longitudinal reinforcement to top of the beam (y_s), yield stress of longitudinal reinforcement (f_y), and young modulus of longitudinal reinforcement (E_s). The GFRP sheet elements are defined by their cross-sectional area (A_{FRP}), position of GFRP sheet to top of the beam (y_{FRP}), initial pre-strain ($\Delta\epsilon_{FRP}$), rupture stress of GFRP sheet (f_{uFRP}), and young modulus of GFRP sheet (E_{FRP}). The GFRP sheet is modeled in the same manner as a longitudinal reinforcement element except it is defined by the stress-strain relationship of GFRP sheet, which is linear elastic till failure. In response2000 the stress strain curve of the

prestressing tendons was modified in such a manner that it reflects the stress strain curve of GFRP a sheet.

Response2000 analyzes the concrete layers, longitudinal steel elements, and GFRP sheet elements individually such that conditions of compatibility and equilibrium are satisfied for the section as a whole. The only sectional compatibility requirement used is that plane sections remain plane. Thus, the longitudinal strain in each of the concrete layers and reinforcing bar elements are determined by defining the top and bottom fibre strains in the section. Response2000 satisfies the equilibrium requirement by balancing shear, moment, and axial load acting on the section. It also satisfies horizontal shear equilibrium on the element. In this program condition of equilibrium and compatibility in concrete layers are dictated by the modified compression field theory.

Response2000 estimates the longitudinal strain distribution and shear stress distribution across the section. The stress in each reinforcing bar element and GFRP sheet element can be determined from the corresponding stress strain curves of the respective materials. The longitudinal stresses in a concrete layer are determined from the given longitudinal strain and normal shear stress acting on that particular concrete layer with the help of the MCFT formulation. The program satisfies that the resultant of these stresses must balance the applied sectional forces i.e. axial load N , moment M , and shear V . In case it does not balance then the process is repeated by readjustment of the assumed longitudinal strain gradient until equilibrium is achieved. The correct shear stress distribution is determined by analyzing two adjacent sections a small distance apart. Both sections are analyzed for the same shear stress distribution, satisfying section equilibrium in each case. The assumed shear stresses are then checked by examining the static equilibrium of each layer. If they don't

match then the assumed shear flow distribution is revised and the analysis is repeated until a corrected shear flow is obtained.

5.5 Analysis Results of Flexural Test Specimens

Static inelastic analyses were carried out using the computer software Response 2000. The analyses included all eight flexural test specimens, un-strengthened or strengthened with prestressed or non-prestressed GFRP sheets. Two different quantities of flexural reinforcement were used in these test specimens. The whole lot of test specimens was broadly divided in two sub groups i.e. (Group-1 & Group-2) based on quantity of flexural reinforcement. The group-1 included six beams and group-2 included two beams. The basic feature of the analyzed test specimens are as follows;

Group-1:

- 1) The un-retrofitted beam act as control beam (Control-1)
- 2) Beam retrofitted with one layer of non-prestressed GFRP sheet (B1U)
- 3) Beam retrofitted with two layers of non-prestressed GFRP sheets (B2U)
- 4) Beam retrofitted with one layer of GFRP sheet prestressed to level-1 (B1P1)
- 5) Beam retrofitted with two layers of GFRP sheets prestressed to level-1 (B2P1)
- 6) Beam retrofitted with one layer of GFRP sheet prestressed to level-2 (B1P2)

Group-2:

- 7) The un-retrofitted beam act as control beam (Control-2)
- 8) Beam retrofitted with three layers of GFRP sheets prestressed to level-1 (B3P1)

The beams analysed in this analytical model have cross-section of 150x300mm. The clear span of the beam was 2200mm. The cross-sections of Group-1 & 2 beams are shown in Fig. 5.4 & 5.5, respectively. The stress-strain relationship of constituent

materials used in this analytical study i.e. concrete, steel reinforcement and GFRP sheet are shown in Fig. 5.6 through Fig. 5.10. The results of the static inelastic analysis for eight flexural beams are shown in Fig. 5.11 through Fig. 5.16 in the form of moment curvature and load deflection curves.

Fig 5.11 shows the moment curvature curves of control and beam strengthened with one layer of GFRP sheet prestressed to 0%, 30%, and 45% of ultimate strength of GFRP sheet and were named B1U, B1P1, and B1P2, respectively.

The moment curvature curves shows that addition of one layer of un-prestressed GFRP does not have any effect on the cracking moment of the beam. Both control and B1U beams cracked at 10.89kNm. Beams strengthened with one layer of prestressed GFRP i.e. B1P1 and B1P2, resulted in an increase in cracking moment from 10.89kNm to 13.85 and 14.74kNm, respectively. The cracking moment was increased by 27% and 35% in case of beam B1P1 and B1P2, respectively.

The yielding moment of 54.75kNm was predicted for control beam. An increase of approximately 2kNm i.e. 4% of yielding moment of control beam was predicted with addition of one layer of non-prestressed GFRP sheet, which is believed due to increase in flexural reinforcement. The moment-curvature curve, B1U, shows that at the yield of reinforcement the sheet start taking load till a moment of 67.18kNm. The model predicted that with application of prestressed GFRP sheet the behaviour of beam under service load was improved. The yielding of flexural reinforcement was delayed till a moment of 69.49kNm in beam B1P1, which is 27% higher than the yielding moment of the control beam. At yielding of flexural reinforcement, the prestressed GFRP sheet further took loads till a moment of 73.94kNm. The application of GFRP sheet prestressed to a higher level i.e. 45% of ultimate strength (B1P2), further improved the condition of the strengthened beam under service load.

Comparatively smaller curvature values were predicted at same moment values in beam B1P2 than beam B1P1. The increase in yielding moment of the beam strengthened with prestressed GFRP sheet shows the beneficial effect of prestressing on the behaviour of the beam under service load conditions. A negligible increase in yielding moment was predicted in beam B1P2 as compared to beam B1P1. It is clear from Fig. 5.11 that active strengthening technique i.e. prestressing of GFRP sheet is an effective method of strengthening, which insured utilization of maximum strength of low modulus GFRP sheet without excessive deflection of the strengthened beam.

The ultimate moment capacity of 66.22kNm and 76.32kNm, at a curvature of 47.85rad/km, was predicted for control1 and beam B1U, respectively. An increase of 15% in ultimate moment capacity was predicted with addition of one layer of non-prestressed GFRP sheet without any change in curvature. It is believed that increase in ultimate moment capacity of beam (B1U) as compared to control1 beam was mainly contributed by increase in flexural reinforcement. On the other hand, addition of one layer of prestressed GFRP sheet further increased the ultimate moment capacity of the beam to 79.69kNm and 81.69kNm, but reduced the curvature to 35.46rad/km and 31.54rad/km in case of beam B1P1 and beam B1P2, respectively.

Figure 5.12 show Load-deflection curves of control1 beam and beams strengthened with one layer of GFRP sheet. The predicted results are given in Table 5.1. The curves shows that the cracking load of both control1 and B1U is the same i.e. 19.43kN, which means that addition of one layer of un-prestressed GFRP sheet does not have any effect on cracking load. On the other hand, the cracking loads of beams B1P1 and B1P2 have been increased to 28.45kN and 29.15kN, respectively. The load-deflection curves in Fig. 5.12 demonstrate that beams strengthened with prestressed GFRP sheet carries higher load at any deflection value as compared to the control1

beam. Control beam has a yielding load of 140.20kN with mid span deflection of 7.68mm. The yielding load of beam B1U has been increased to 162.94kN with mid span deflection of 9.15mm. It is believed that this increase in yielding load was due to increase in flexural reinforcement. The prestressing of GFRP sheet in beams B1P1 and B1P2 resulted in further increase of yielding load as compared to beam B1U. The yielding of flexural reinforcement in beam B1P1 and B1P2 starts at 170.69kN with mid span deflection of 8.68mm and 174.92kN with mid span deflection of 8.66mm, respectively. The increase in yielding load of beam B1P1 and B1P2 over beam B1U was due to the prestressing of the GFRP sheet. The model predicted the ultimate loading capacities of 155.78kN and 181.05kN with mid span deflections of 17.40mm and 18.65mm in the case of control and B1U, respectively. The higher ultimate loading capacity of beam B1U is believed due to higher quantity of flexural reinforcement. Beams B1P1 and B1P2 have same flexural reinforcement as beam B1U. The model predicted ultimate loads of 189.66kN and 194.35kN with ultimate deflection of 14.91mm and 13.80mm for beams B1P1 and B1P2, respectively. The increase in ultimate load and reduction in ultimate deflection of beams B1P1 and B1P2 in comparison to B1U is believed due to prestressing of GFRP sheet. From the above discussion, it can be concluded that prestressing of GFRP sheet is a useful technique of strengthening, which improves the behaviour of the strengthened beam under service loading as well as under ultimate loading conditions.

The study was further extended to investigate the behaviour of beams strengthened with two layers of GFRP sheets. Two beams strengthened with two layers of GFRP sheets prestressed to 0% and 30% of its ultimate strength named as B2U and B2P1, respectively, were investigated with the present model and their results were compared with control beam. Fig 5.13 shows the moment curvature

curves of control1 and beams B2U and B2P1. The moment curvature curves show that addition of two layer of un-prestressed GFRP does not have any effect on the cracking moment of the beam. This is a similar prediction as the beam strengthened with one layer. Both control1 and B2U beams cracked at 10.89kNm. The beam strengthened with two layers of prestressed GFRP i.e. B2P1, resulted in an increase in cracking moment from 10.89kNm to 15.89kNm, which is 46% higher than control1 and B2U beams.

The yielding moment of 54.75kNm was predicted by this model for control1 beam. An increase of approximately 4.23kNm, i.e. 8% of the yielding moment of the control beam, was predicted for beam B2U. This increase in yielding moment of B2U is believed due to increase in flexural reinforcement. The moment-curvature curve, B2U, shows that at the yield of reinforcement the sheet start taking load till a moment of 73.75kNm. The model predicted that with application of prestressed GFRP sheet the behaviour of beam under service load was improved. The yielding of flexural reinforcement was delayed till a moment of 80.39kNm in beam B2P1 which is 47% higher than the yielding moment of the control1 beam. The increase in yielding moment of the beam strengthened with prestressed GFRP sheet shows the beneficial effect of prestressing on the behaviour of the beam under service load conditions. It is clear from Fig. 5.13 that active strengthening technique i.e. prestressing of GFRP sheet, is an effective method of strengthening, which insured utilization of maximum strength of low modulus GFRP sheet without excessive deflection of the strengthened beam.

The ultimate moment capacity of 66.22kNm and 85.44kNm, at a curvature of 47.85rad/km and 52.64rad/km, was predicted for control1 and beam B2U, respectively. It is believed that increase in ultimate moment capacity of beam (B2U)

as compared to control1 beam was mainly contributed by increase in flexural reinforcement. On the other hand, addition of two layers of prestressed GFRP sheets further increased the ultimate moment capacity of the beam to 92.57kNm but reduced the curvature to 37.17rad/km. The increase in ultimate moment capacity of beam B2P1 as compared to beam B2U is due to prestressing of GFRP sheet.

Figure 5.14 shows load-deflection curves of control1 beam and beams strengthened with two layers of GFRP sheet. The predicted results are given in Table 5.2. The curves show that the cracking load of both control1 and B2U is the same i.e. 19.43kN, which means that addition of two layers of un-prestressed GFRP sheet does not have any effect on the cracking load. On the other hand, the cracking load of beam B2P1 has been increased to 43.74kN. Control1 beam has a yielding load of 140.20kN with mid span deflection of 7.68mm. The yielding load of beam B2U has been increased to 171.35kN with mid span deflection of 9.88mm. It is believed that this increase in yielding load was due to increase in flexural reinforcement. The prestressing of GFRP sheet in beams B2P1 resulted in further increase of yielding load as compared to beam B2U. The yielding of flexural reinforcement in beam B2P1 starts at 196.82kN with mid span deflection of 8.68mm. The increase in yielding load of beam B2P1 over beam B2U was due to prestressing of GFRP sheet. The model predicted the ultimate loading capacities of 155.78kN and 190.39kN with mid span deflections of 17.40mm and 16.08mm in the case of control1 and B2U, respectively. The higher ultimate loading capacity of beam B2U is believed due to higher quantity of flexural reinforcement. Beams B2P1 has same flexural reinforcement as beam B2U. The model predicted ultimate loads of 218.69kN with ultimate deflection of 13.89mm for beam B2P1. The increase in ultimate load and reduction in ultimate

deflection of beam B2P1 in comparison to B2U is believed due to prestressing of GFRP sheet.

Analytical investigation was also conducted on the behaviour of beams which lose some percentage of flexural reinforcement due to corrosion etc. For this purpose, two beams, one control and one strengthened with three layers of prestressed GFRP sheet, were assumed. Both beams have similar cross section and shear reinforcement as control1 beam, except 16% less conventional flexural reinforcement. Control beam was called control2 and strengthened beam was called B3P1 beam. The beam strengthened with three layers of GFRP sheets prestressed to 30% of its ultimate strength (B3P1) was investigated with the present model and its result was compared with control2 beam. Fig 5.15 shows the moment curvature curves of control2 and beam B3P1. It is obvious from the moment curvature curves that the beam strengthened with three layers of prestressed GFRP i.e. B3P1, resulted in an increase of cracking moment as compared to control2 beam. The model predicted cracking moments of 9.22kNm and 18.04 for control2 and B3P1 beams, respectively. This means that application of prestressed GFRP sheet resulted in 96% increase in cracking moment as compared to control2 beam.

The yielding moment of 39.33kNm at a curvature of 12.60rad/km was predicted by this model for control2 beam. The moment curvature curve in Fig. 5.15 shows that the yielding of flexural reinforcement was delayed till a moment of 80.78kNm at a curvature of 16.19rad/km in beam B3P1, which is 105% higher than the yielding moment of control1 beam. The increase in yielding moment of the beam strengthened with prestressed GFRP sheet shows the beneficial effect of prestressing on the behaviour of the beam under service load conditions and demonstrates that active

strengthening technique i.e. prestressing of GFRP sheet, is an effective method of strengthening.

The ultimate moment capacity of 55.55kNm at a curvature of 57.91rad/km was predicted for control2 beam. Addition of three layers of prestressed GFRP sheets in beam B3P1 resulted in an increase of ultimate moment capacity of beam to 96.77kNm but reduced the curvature to 38.18rad/km. The 74% increase in ultimate moment capacity of beam B3P1 as compared to control2 beam is believed due to increase in flexural reinforcement quantity as well as prestressing of GFRP sheet.

Figure 5.16 show load-deflection curves of control3 and B3P1 beams. The predicted results are given in Table 5.3. The cracking load of 21.34kN and 53.35kN at deflection of 0.483mm and 1.17mm were predicted for control2 and B3P1 beams, respectively. Control2 beam has a yielding load of 111.10kN with mid span deflection of 6.57mm. The yielding of flexural reinforcement in beam B3P1 starts at 192.07kN with mid span deflection of 9.10mm. The increase in yielding load of beam B3P1 over beam control2 is believed due to both increase in flexural reinforcement quantity and prestressing of GFRP sheet. The model predicted the ultimate load of 130.70kN with mid span deflection of 21.02mm for control2 beam and ultimate load of 213.41kN with ultimate deflection of 13.27mm for beam B2P1.

5.6 Analysis Results of Shear Test Specimens

Static inelastic analyses were carried out using the computer software Response 2000. The analyses included all fifteen shear test specimens, un-strengthened or strengthened with prestressed or non-prestressed GFRP sheets. Three different shear spans to effective depth (a/d) ratios were used in testing the shear specimens. The whole lot of test specimens was broadly divided into three sub groups on the basis of a/d ratio. Each group included five beams. The basic features and properties of the

constituent material in all analyzed test specimens are similar. In the names of the beams the last part i.e. 2.5, 3.0, and 3.5 indicate the a/d ratio on which the beam is tested. The detail description of all specimens is given below;

Beams tested with a/d ratio = 2.5

- 1) The un-retrofitted beam act as control beam (Control2.5)
- 2) Beam retrofitted with two layers of prestressed GFRP sheet (B2P2.5)
- 3) Beam retrofitted with three layers of prestressed GFRP sheets (B3P2.5)
- 4) Beam retrofitted with 100mm wide U-straps of non-prestressed GFRP sheet @ 200mm c/c in shear span only. The U-straps are applied to two vertical and bottom faces of beam (BU2.5).
- 5) Beam retrofitted with full jacket of non-prestressed GFRP sheet in shear span only. The jacket is applied to two vertical and bottom faces of beam (BJ2.5).

Beams tested with a/d ratio = 3.0

- 6) The un-retrofitted beam act as control beam (Control3.0)
- 7) Beam retrofitted with one layer of prestressed GFRP sheet (B1P3.0)
- 8) Beam retrofitted with two layers of prestressed GFRP sheets (B2P3.0)
- 9) Beam retrofitted with 100mm wide U-straps of non-prestressed GFRP sheet @ 200mm c/c in shear span only. The U-straps are applied to two vertical and bottom faces of beam (BU3.0).
- 10) Beam retrofitted with full jacket of non-prestressed GFRP sheet in shear span only. The jacket is applied to two vertical and bottom faces of beam (BJ3.0).

Beams tested with a/d ratio = 3.5

- 11) The un-retrofitted beam act as control beam (Control3.5)
- 12) Beam retrofitted with one layer of prestressed GFRP sheet (B1P3.5)
- 13) Beam retrofitted with two layers of prestressed GFRP sheets (B2P3.5)

14) Beam retrofitted with 100mm wide U-straps of non-prestressed GFRP sheet @ 200mm c/c in shear span only. The U-straps are applied to two vertical and bottom faces of beam (BU3.5).

15) Beam retrofitted with full jacket of non-prestressed GFRP sheet in shear span only. The jacket is applied to two vertical and bottom faces of beam (BJ3.5).

The beams analysed in this analytical model have cross-sections of 250x400mm. The clear span was kept 2600mm for all test specimens except Control2.5 and B2P2.5 which were tested on a 2800mm clear span. The cross-section of the shear beam is shown in Fig. 5.17. The stress-strain relationship of constituent materials used in this analytical study i.e. concrete, steel reinforcement and GFRP sheet are shown in Fig. 5.18 through Fig. 5.22. The results of the static inelastic analysis for fifteen beams tested in three groups on a/d ratios of 2.5, 3.0, and 3.5 are shown in Fig. 5.23 through Fig. 5.25 in form of shear force vs. maximum deflection curves. The predicted results of the analysis are given in Table 5.4. To study the effect of a/d ratio on the shear capacity of the beams, results of five different beams is compared in three a/d ratios and plotted in Fig. 5.26 through 5.30 in the form of shear force vs. maximum deflection curves.

Figure 5.23 shows the shear force vs. maximum deflection curves of five beams tested with an a/d ratio of 2.5 i.e. control2.5, B2P2.5, B3P2.5, BU2.5, and BJ2.5. The model predicted shear capacity of 131.85kN with maximum deflection of 5.68mm for control2.5. The curves show that addition of two and three layers of prestressed GFRP resulted in reduction of deflection at any given shear force as compared to control2.5. An increase of 4.32kN and 13.69kN, i.e. 3% and 10%, in shear capacities of the beams B2P2.5 and B3P2.5, respectively, is predicted as compared to control2.5. It is worth mentioning here that this model ignores the beneficial effect of small

compressive clamping stress due to prestressed GFRP sheet and hence more conservative in predicting the shear strength of beams strengthened with prestressed GFRP sheets. As discussed earlier in this chapter, the effect of clamping stress will be more pronounced in beams with smaller a/d ratios as compared to large a/d ratios. The model assumes full bond between GFRP sheet and concrete till failure and ignores any premature delamination. The application of U-straps in beam BU2.5 increased the ultimate shear capacity of the beam to 291.60kN with a maximum deflection of 12.50mm. This is a 121% increase in shear capacity as compared to control2.5 and is believed due to increase in transverse reinforcement of the beam. The increase in transverse reinforcement was doubled in beam BJ2.5 which resulted in further increase of ultimate shear capacity of the beam. The model predicted ultimate shear capacity of 330.36kN with a maximum deflection of 15.43mm for beam BJ2.5. This is a 151% increase in shear capacity as compared to control 2.5. The increase in shear capacity of BJ2.5 is not proportionate to the increase in quantity of transverse reinforcement as compared to beam BU2.5. It is believed that the maximum shear capacity of beam BJ2.5 was achieved, and flexural behaviour started to dominate.

Fig 5.24 shows shear force vs. maximum deflection curves of five beams tested with an a/d ratio of 3.0, i.e. control3.0, B1P3.0, B2P3.0, BU3.0, and BJ3.0. The model predicted a shear capacity of 123.65kN with maximum deflection of 4.90mm for control3.0. The curves show that the addition of one and two layers of prestressed GFRP resulted in reduction of deflection at any given shear force as compared to control3.0. An increase of 4.72kN and 7.18kN, i.e. 4% and 6%, in shear capacities of the beams B1P3.0 and B2P3.0, respectively, is predicted as compared to control3.0. The reason for predicting this marginal increase in shear capacity is ignoring the beneficial effect of small compressive clamping stress by this model, which is

discussed in detail above. The application of U-straps in beam BU3.0 increased the ultimate shear capacity of the beam to 275.30kN with a maximum deflection of 14.89mm. This is 123% increase in shear capacity as compared to control3.0 and is believed to be due to increase in transverse reinforcement of the beam. The increase in transverse reinforcement was doubled in beam BJ3.0, which resulted in no further increase of ultimate shear capacity of beam. The model predicted ultimate shear capacity of 275.30kN with maximum deflection of 13.66mm for beam BJ2.5. The maximum shear capacity predicted for beam BJ3.0 is the same as BU3.0, whereas a small decrease in ultimate deflection has occurred. It is believed that the maximum shear capacity of the beam was achieved in beam BU3.0 and hence did not get any further increase in shear capacity of beam BJ3.0 with increase in quantity of transverse reinforcement.

Figure 5.25 shows shear force vs. maximum deflection curves of five beams tested with an a/d ratio of 3.5, i.e. control3.5, B1P3.5, B2P3.5, BU3.5, and BJ3.5. The model predicted shear capacity of 120.86kN with maximum deflection of 5.52mm for control3.5. The curves show that addition of one and two layers of prestressed GFRP resulted in reduction of deflection at any given shear force as compared to control3.5. An increase of 4.46kN and 9.02kN, i.e. 4% and 7%, in shear capacities of the beams B1P3.5 and B2P3.5, respectively, was predicted as compared to control3.5. The application of U-straps in beam BU3.5 increased the ultimate shear capacity of the beam to 235.97kN with maximum deflection of 13.41mm. This is a 95% increase in shear capacity as compared to control3.5 and is believed to be due to increase in transverse reinforcement of the beam. The increase in transverse reinforcement was doubled in beam BJ2.5 which resulted in no further increase of the ultimate shear capacity of the beam. The model predicted ultimate shear capacity of 235.30kN with

maximum deflection of 12.30mm for beam BJ2.5. The maximum shear capacity predicted for beam BJ3.5 is the same as BU3.5 whereas a small decrease in ultimate deflection has occurred. It is believed that the maximum shear capacity of the beam was achieved in beam BU3.5 and hence we did not get any further increase in shear capacity with increase in quantity of shear reinforcement in beam BJ3.5.

The analysis results of the beams were further studied to predict the effect of a/d ratio on the shear capacity of the beam. Fifteen shear beams were divided into five sub groups. Each group consists of three exactly similar beams i.e. control, B1P, B2P, BU, and BJ, which were tested on three different a/d ratios.

Figure 5.26 shows the shear force vs. maximum deflection curves of three un-strengthened control beams. The curves show decrease in shear capacity of the beam with increase in a/d ratio. This is due to the fact that with increase in a/d ratio the quantity of applied moment increases and hence decreases the shear capacity. The curves shows higher deflection value for higher a/d ratio at any given shear force value. This is due to the fact that with increase in a/d ratio the behaviour of the beam starts to change from shear to flexural and hence resulted in higher deflection values. The model predicts ultimate shear capacities of 131.86kN, 125.44kN, and 120.86kN with ultimate deflection of 5.00mm, 4.93mm, and 5.52mm, for beam control2.5, control3.0, and control3.5, respectively.

Figure 5.27 shows the shear force vs. maximum deflection curves of three beams strengthened with two layers of prestressed GFRP sheet. These curves also show decrease in shear capacity of the beam with increase in a/d ratio. However the difference is negligible in beam B2P2.5 and B2P3.0. The reason is the same that with increase in a/d ratio the quantity of applied moment increases and hence decreases the shear capacity. The curves shows higher deflection value for higher a/d ratio at any

given shear force value. The model predicts ultimate shear capacities of 136.17kN, 132.62kN, and 129.88 with ultimate deflection of 3.30mm, 3.45mm, and 3.95mm, for beam B2P2.5, B2P3.0, and B2P3.5, respectively.

Figure 5.28 shows the shear force vs. maximum deflection curves of two beams strengthened with one layer of prestressed GFRP sheet i.e. B1P3.0 and B1P3.5. The beam tested with a/d ratio of 2.5 was strengthened with three layers of GFRP sheets and hence it is not possible to compare its results here. These curves also show decrease in shear capacity of the beam with increase in a/d ratio. The reason is the same that with increase in a/d ratio the quantity of applied moment increases and hence decreases the shear capacity. The curves shows higher deflection value for higher a/d ratio at any given shear force value. The model predicts ultimate shear capacities of 130.16kN, and 125.32kN, with ultimate deflection of 4.46mm, and 4.65mm, for beam B1P3.0, and B1P3.5, respectively.

Figure 5.29 shows the shear force vs. maximum deflection curves of three beams strengthened with U-straps of non-prestressed GFRP sheet i.e. BU2.5, BU3.0 and BU3.5. These curves also show decrease in shear capacity of the beam with increase in a/d ratio. The reason is the same that with increase in a/d ratio the quantity of applied moment increases and hence decreases the shear capacity. The curves shows higher deflection value for higher a/d ratio at any given shear force value. The model predicts ultimate shear capacities of 291.60kN, 275.30kN and 235.97kN, with ultimate deflection of 12.50mm, 14.89mm and 13.97mm, for beam BU2.5, BU3.0, and BU3.5, respectively.

Figure 5.30 shows the shear force vs. maximum deflection curves of three beams strengthened with full jacket of non-prestressed GFRP sheet i.e. BJ2.5, BJ3.0 and BJ3.5. These curves also show decrease in shear capacity of the beam with increase in

a/d ratio. The reason is the same that with increase in a/d ratio the quantity of applied moment increases and hence decreases the shear capacity. The curves shows higher deflection value for higher a/d ratio at any given shear force value. The model predicts ultimate shear capacities of 330.35kN, 275.30kN and 235.97kN, with ultimate deflection of 14.69mm, 13.66mm and 12.30mm, for beam BJ2.5, BJ3.0, and BJ3.5, respectively.

5.7 Computation of Long Term Losses

In this model, computations of short-term deflections of reinforced concrete beams are made with the assumption that the concrete is an elastic and homogeneous material. This assumption is only approximately correct, because the elastic modulus for concrete is not a constant for all stress levels. The elastic modulus depends on the age of the concrete, mix design, loading history, and the environment. Concrete gains strength with age and its modulus of elasticity increases with time. It also undergoes the-dependent volumetric changes as it creeps under sustained loads and shrinks upon drying. The creep and shrinkage resulted in reduction of prestressing force in the GFRP sheet and hence compression in the concrete. In addition, in a simply supported member, the prestressing force produces an upward deflection. Creep increases this camber to grow with time while shrinkage reduce the prestressing force and hence the camber. Dead loads produce downward deflection and creep, and shrinkage increase this deflection with time.

The short-term response does not include the effect of shrinkage and creep of the concrete. The long-term response includes these effects. The knowledge of creep and shrinkage properties of concrete will allow the prediction of prestress losses in the GFRP sheet and strains in the section which affect long-term deformations of the

concrete structures. In this section, the concrete behaviour is reviewed and analytical expressions for its prediction and its variation with time and temperature are given.

5.7.1 Creep of Concrete

The stress-strain response of concrete depends on the loading history. Concrete under constant stress undergoes a gradual increase of strain with time because of creep deformations of the concrete. With time, the ratio of the stress to total strain decreases due to creep strain. This effect can be represented by using a reduced modulus of elasticity, $E_{c,eff}$, as follows (Collins and Mitchell, 1991):

$$E_{c,eff} = \frac{E_{ci}}{1 + \phi(t, t_i)} \quad \text{Eq: 5.18}$$

Where

E_{ci} = the instantaneous elastic modulus at time t_i ($E_{ci} = E_{ct}$)

E_{ct} = the initial tangent modulus of elasticity.

t_i = the time when stress is applied after casting

t = the time when the strain is calculated

$\phi(t, t_i)$ = the creep coefficient, and can be calculated using the following simplified expression (Collins and Mitchell, 1991):

$$\phi(t, t_i) = 3.5 k_c k_f \left[1.58 - \frac{H}{120} \right] t_i^{-0.118} \frac{(t - t_i)^{0.6}}{10 + (t - t_i)^{0.6}} \quad \text{Eq: 5.19}$$

Where;

H = the relative humidity in %

k_c = a factor that accounts for the influence of the volume-to-surface ratio of the member. Values of k_c , are given in Figure 5.31.

k_f = a factor that accounts for the influence of concrete strength. This factor accounts for the low creep of high strength concrete and is given as follows:

$$k_f = \frac{1}{0.67 + \left(\frac{f'_c}{62}\right)} \text{ MPa} \quad \text{Eq: 5.20}$$

Where

f'_c = the 28 day compressive strength of concrete.

Thus, to calculate the strain at t days after casting caused by a stress of f_{ci} applied at t_i days after casting and then held constant, the following linear approximation is used:

$$\varepsilon_{cf}(t, t_i) = \frac{f_{ci}}{E_{c,eff}} \quad \text{Eq: 5.21}$$

5.7.2 Shrinkage of Concrete

Shrinkage occurs when the concrete loses moisture with time and decreases in volume. Typically, shrinkage strain in the order of -800×10^{-6} or even -1000×10^{-6} may occur within the first 6 to 12 months after casting (Neville, 1995). The following expression can be used to estimate the shrinkage strain in the concrete for moist cured concrete (Collins and Mitchell, 1991):

$$\varepsilon_{sh} = -k_s k_h \left(\frac{t}{35 + t} \right) 0.51 \times 10^{-3}$$

Where t = the time in days for which the concrete has been exposed to drying. The two factors k_s and k_h account for the size and relative humidity and can be determined from Fig. 5.32.

In the analysis of total losses, creep and shrinkage strains are added to calculate the loss of prestress in the GFRP sheet due to these two phenomena. Using the above

equations, creep and shrinkage strains were predicted. The predicted results are shown in Table 5.5. The shrinkage and creep strains were plotted versus time and are shown in Fig. 5.33 and Fig. 5.34, respectively. Like observed results, the increase in both strains is high at the initial age of concrete and reduces with time. Both curves get approximately flat at the age of six months.

Table 5.1 Predicted results of flexural critical beams strengthened with one layer of GFRP Sheet (Group-1)

Loads and Displacement at various stages	<u>Specimen Name</u>			
	Control1	B1U	B1P1	B1P2
Cracking Load (kN)	18.43	18.86	28.45	29.15
Deflection at 1 st Crack (mm)	0.39	0.39	0.61	0.59
Yielding Load (kN)	140.20	162.94	170.69	174.92
Deflection at Yielding (mm)	7.68	9.15	8.68	8.66
Ultimate Load (kN)	155.78	181.05	189.66	194.35
Deflection at Ultimate (mm)	17.40	18.65	14.81	13.80

Table 5.2 Predicted results of flexural critical beams strengthened with two layers of GFRP Sheets (Group-1)

Loads and Displacement at various stages	<u>Specimen Name</u>		
	Control1	B2U	B2P1
Cracking Load (kN)	18.43	19.43	43.74
Deflection at 1 st Crack (mm)	0.39	0.45	0.82
Yielding Load (kN)	140.20	171.35	196.82
Deflection at Yielding (mm)	7.68	9.88	8.68
Ultimate Load (kN)	155.78	190.39	218.69
Deflection at Ultimate (mm)	17.40	16.08	13.89

Table 5.3 Predicted results of flexural critical beams strengthened with three layers of GFRP Sheets (Group-2)

Loads and Displacement at various stages	<u>Specimen Name</u>	
	Control2	B3P1
Cracking Load (kN)	16.34	37.43
Deflection at 1 st Crack (mm)	0.48	1.17
Yielding Load (kN)	111.10	188.40
Deflection at Yielding (mm)	6.57	9.10
Ultimate Load (kN)	130.70	213.40
Deflection at Ultimate (mm)	21.02	13.27

Table 5.4 Predicted results of ultimate shear capacity of shear critical beams

<u>Specimen Name</u>	<u>Shear Capacity (kN)</u>		
	a/d = 2.5	a/d = 3.0	a/d = 3.5
Un-strengthened Control Beam	131.85	123.65	120.86
Beam strengthened with U-Straps	291.60	275.30	235.30
Beam strengthened with U-Jackets	330.36	275.30	235.30
Beam with one layer of prestressed sheet	-----	128.37	125.32
Beam with two layers of prestressed sheet	136.17	130.83	129.88
Beam with three layers of prestressed sheet	145.54	-----	-----

Table 5.5 Predicted creep and shrinkage strains

Duration (Days)	Creep Strain ($\mu\varepsilon$)	Shrinkage Strain ($\mu\varepsilon$)	Total Strain (Creep + Shrinkage)
1	14	4	18
2	18	7	25
3	19	11	30
4	20	14	34
5	20	18	38
6	21	21	42
7	23	24	47
14	34	44	78
21	37	60	97
28	39	74	113
35	41	86	127
63	44	120	165
81	47	135	182
111	49	154	203
141	51	166	217
171	52	176	228

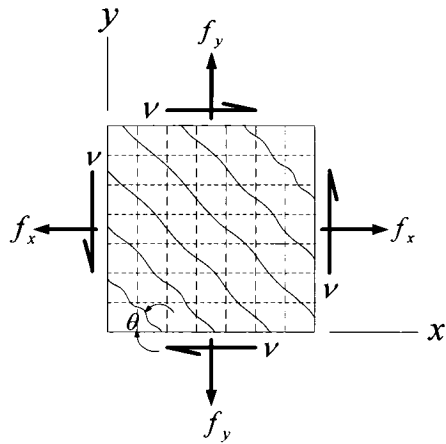


Fig. 5.1 Two dimensional concrete element under normal and shear stresses

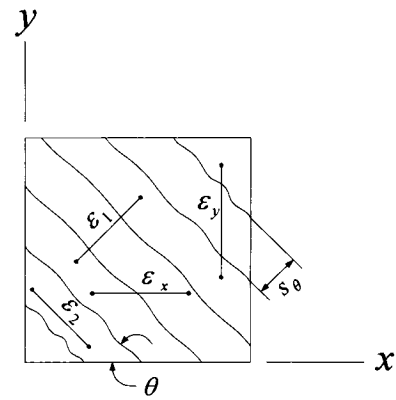


Fig. 5.2 Strain diagram of two dimensional concrete element

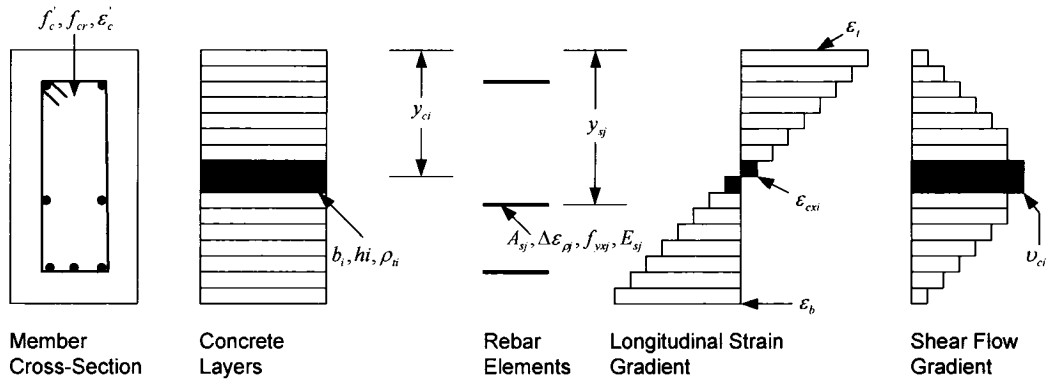


Fig. 5.3 Estimation of longitudinal strain and shear flow distributions using layer by layer model (Collins and Mitchell - 1987)

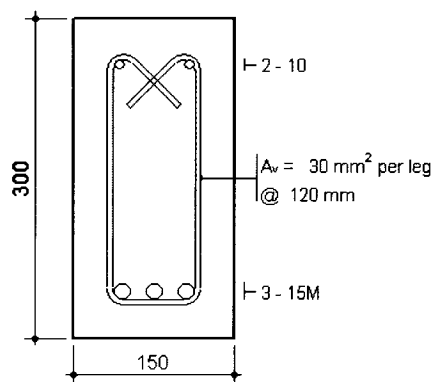


Fig. 5.4 Flexural Beam Cross-section (Group-1)

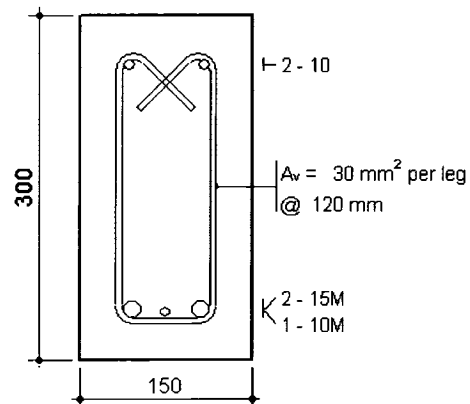


Fig 5.5 Flexural Beam Cross-section (Group-2)

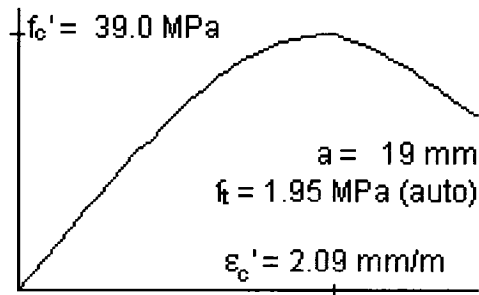


Fig. 5.6 Concrete Stress-Strain Curve
Used in Flexural beams

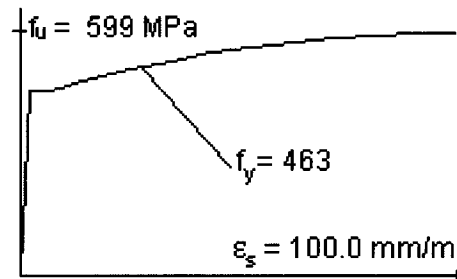


Fig. 5.7 Stress-Strain curve of #15 bar
Used in Flexural beams

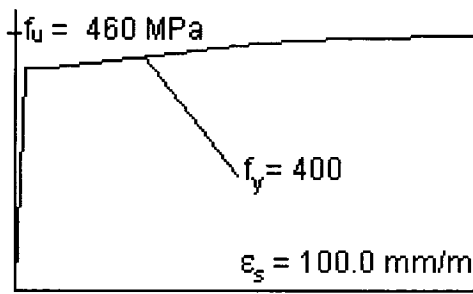


Fig. 5.8 Stress-Strain Curve of #10 bar
Used in Flexural beams

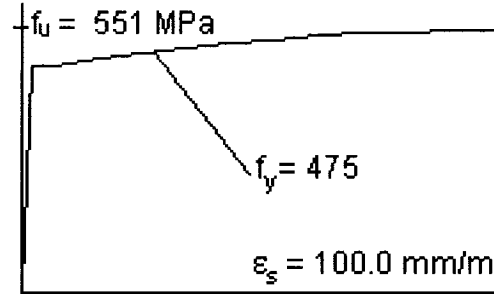


Fig. 5.9 Stress-Strain curve of #6 bar
Used in Flexural beams

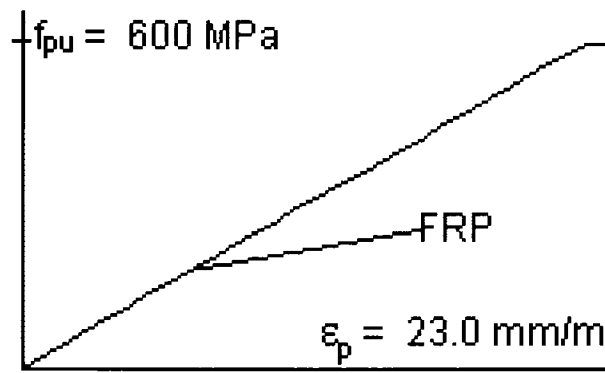


Fig. 5.10 Stress-Strain Curve FRP sheet Used in Flexural beams

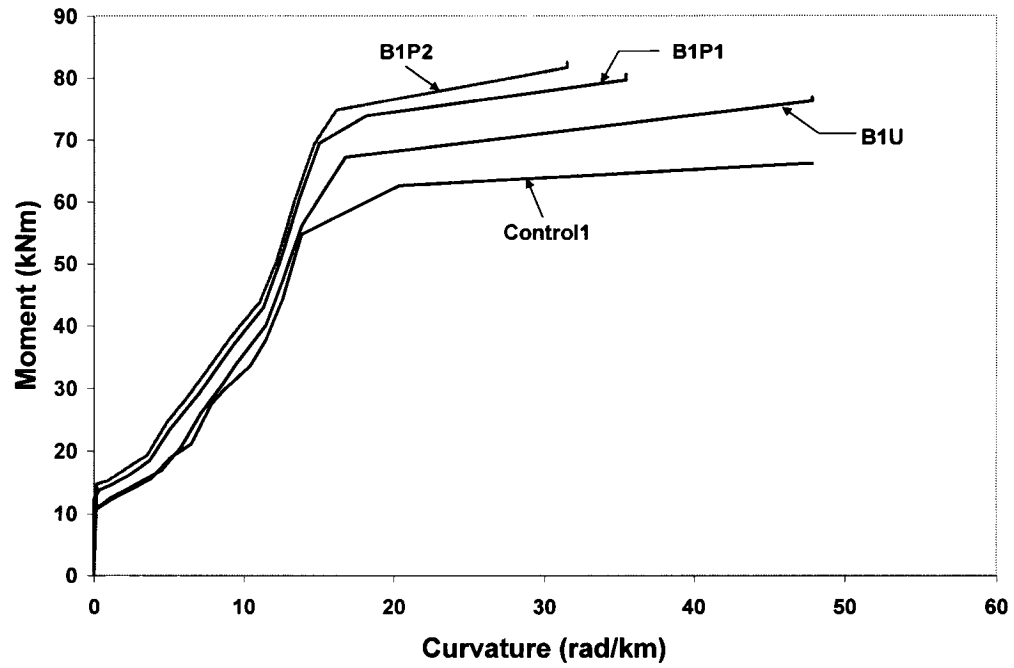


Fig. 5.11 Moment-Curvature Curves of control1 and beams strengthened with one layer of GFRP sheet

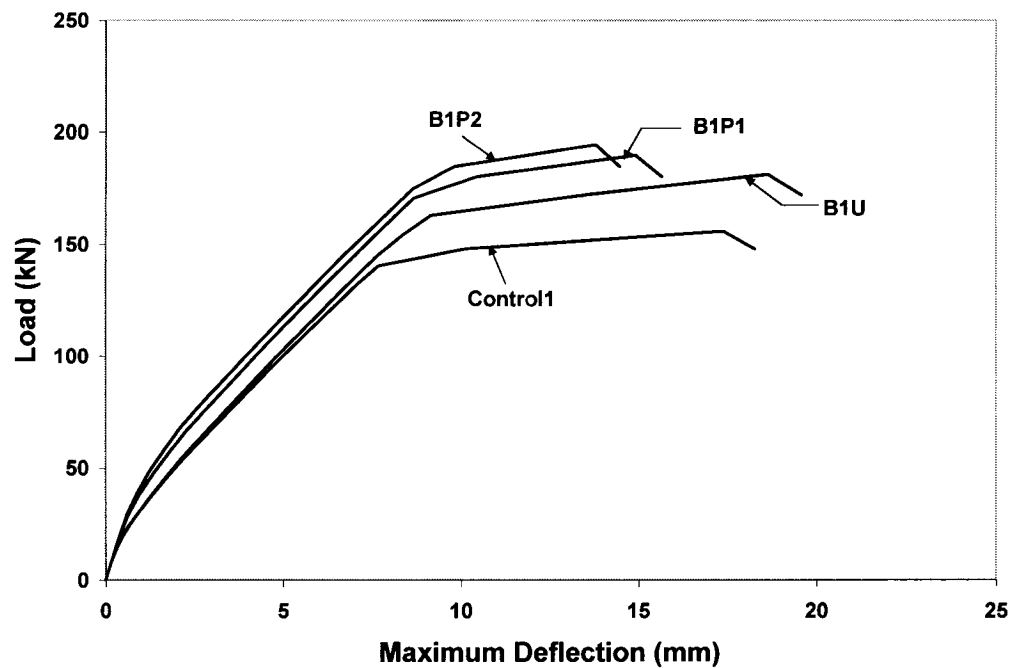


Fig. 5.12 Load – Maximum deflection Curves of control1 and beams strengthened with one layer of GFRP sheet

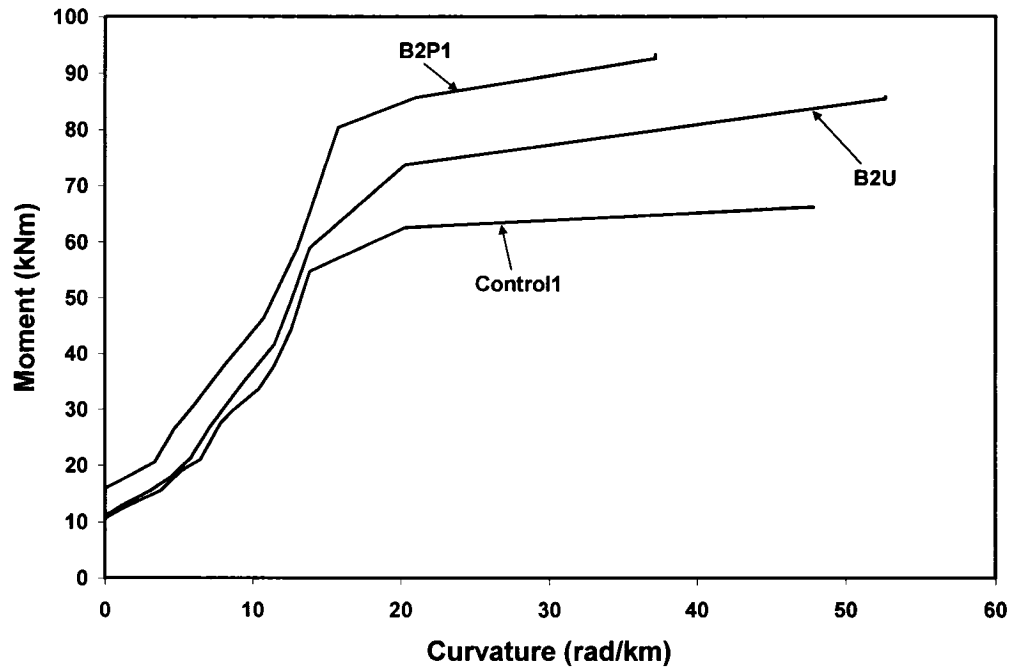


Fig. 5.13 Moment-Curvature Curves of control1 and beams strengthened with two layers of GFRP sheets

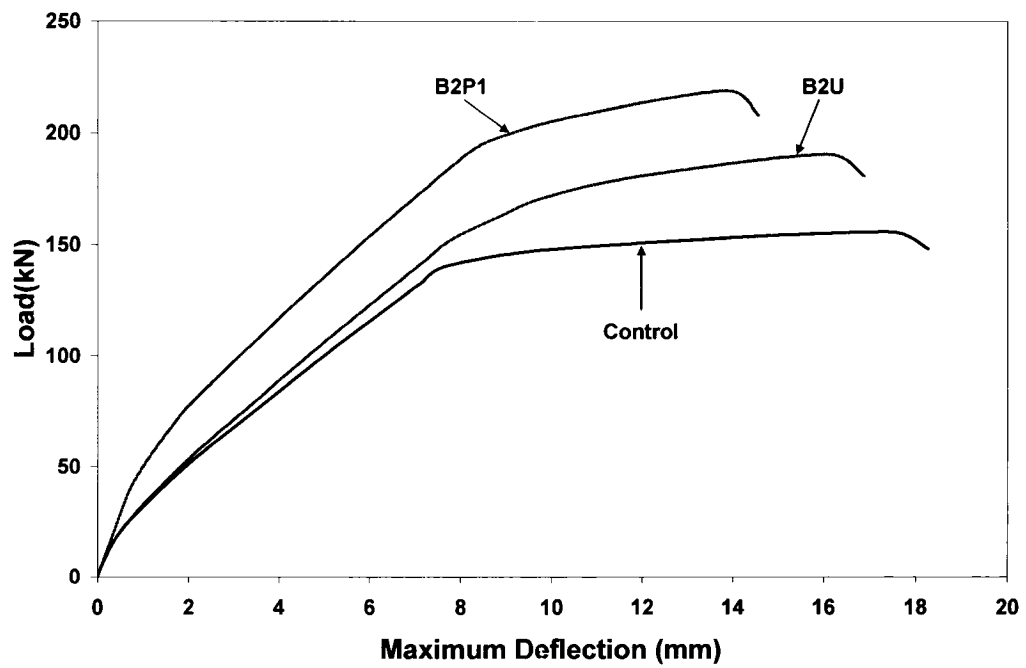


Fig. 5.14 Load – Maximum deflection Curves of control1 and beams strengthened with two layers of GFRP sheet

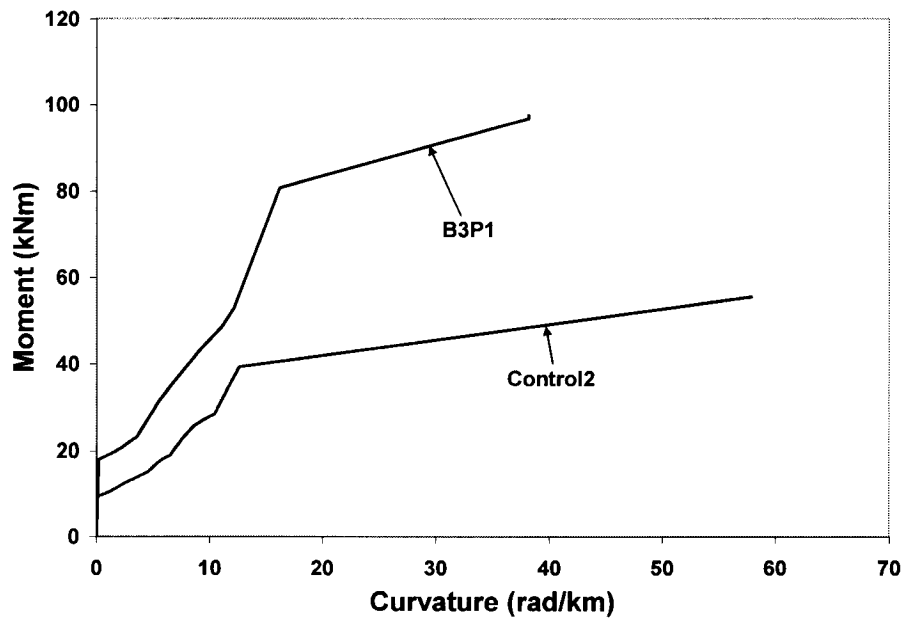


Fig. 5.15 Moment-Curvature Curves of control2 and beams strengthened with three layers of GFRP sheets

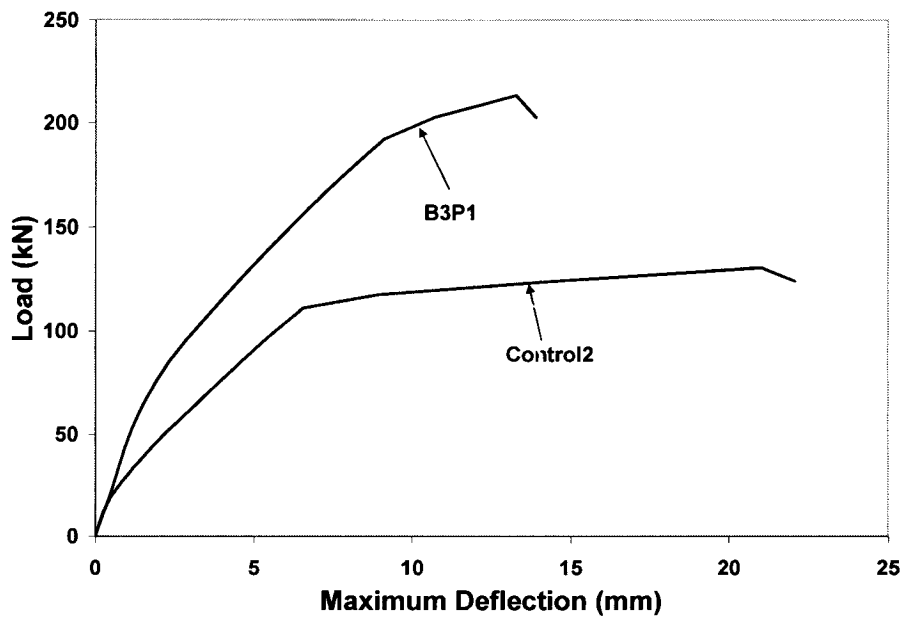


Fig. 5.16 Load - Maximum deflection Curves of control2 and beams strengthened with three layers of GFRP sheet

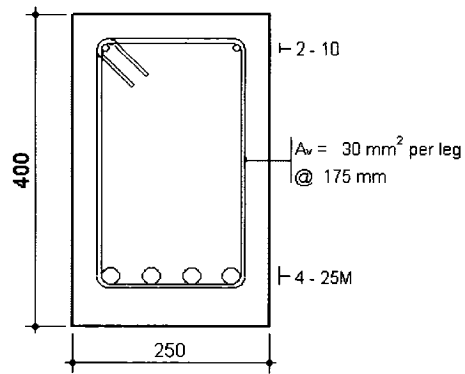


Fig. 5.17 Shear Beam Cross-section

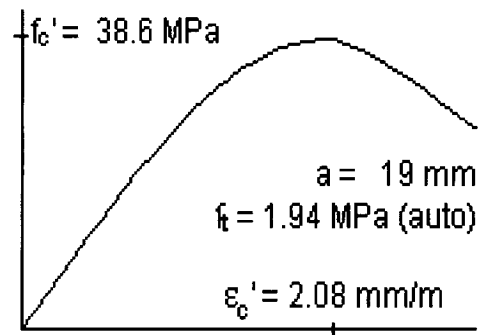


Fig. 5.18 Concrete Stress-Strain Curve
Used in Shear beams

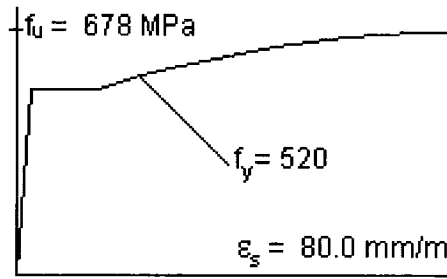


Fig. 5.19 Stress-Strain curve of #25 bar
Used in Shear beams

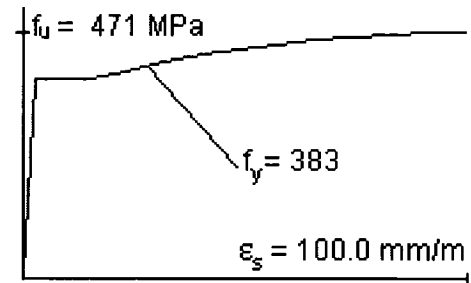


Fig. 5.20 Stress-Strain Curve of #10 bar
Used in Shear beams

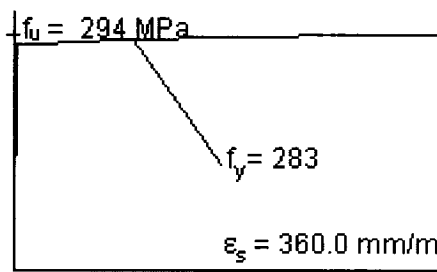


Fig. 5.21 Stress-Strain curve of #6 bar
Used in Shear beams

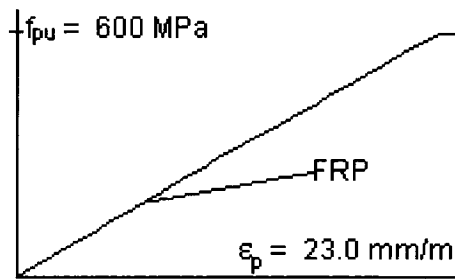


Fig. 5.22 Stress-Strain Curve FRP sheet
Used in Shear beams

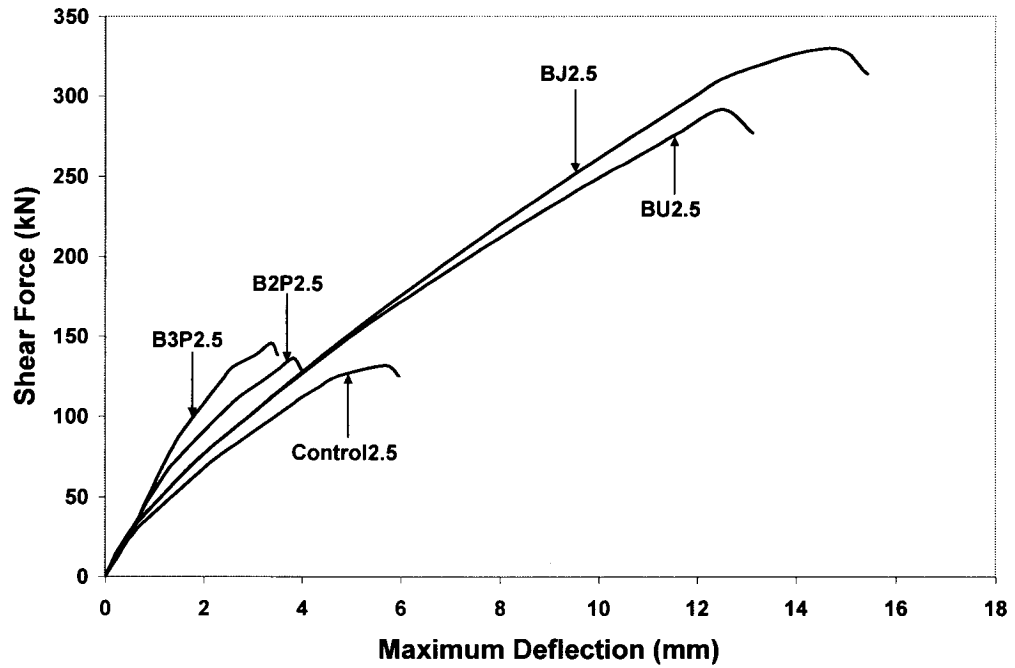


Fig. 5.23 Shear Force – Maximum Deflection curves of Shear beams having shear span to effective depth (a/d) ratio of 2.5

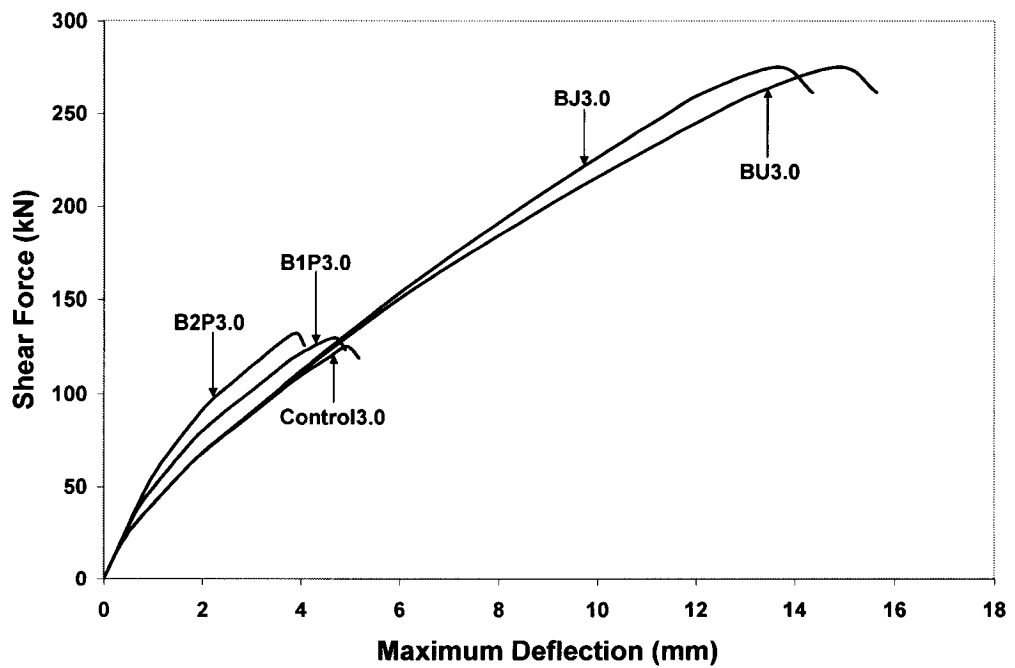


Fig. 5.24 Shear Force – Maximum Deflection curves of Shear beams having shear span to effective depth (a/d) ratio of 3.0

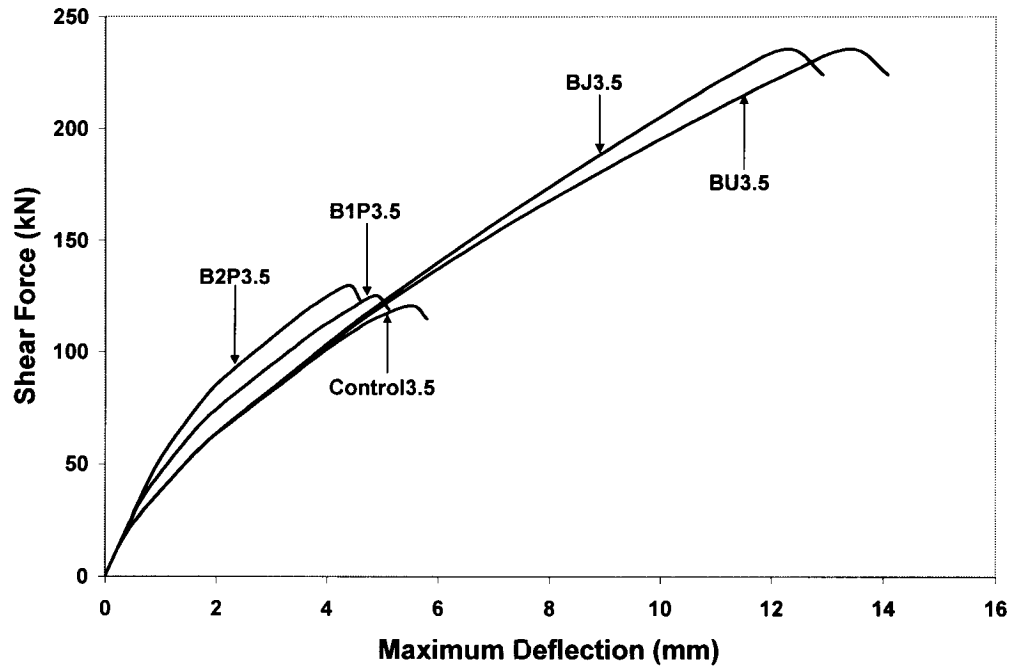


Fig. 5.25 Shear Force – Maximum Deflection curves of Shear beams having shear span to effective depth (a/d) ratio of 3.5

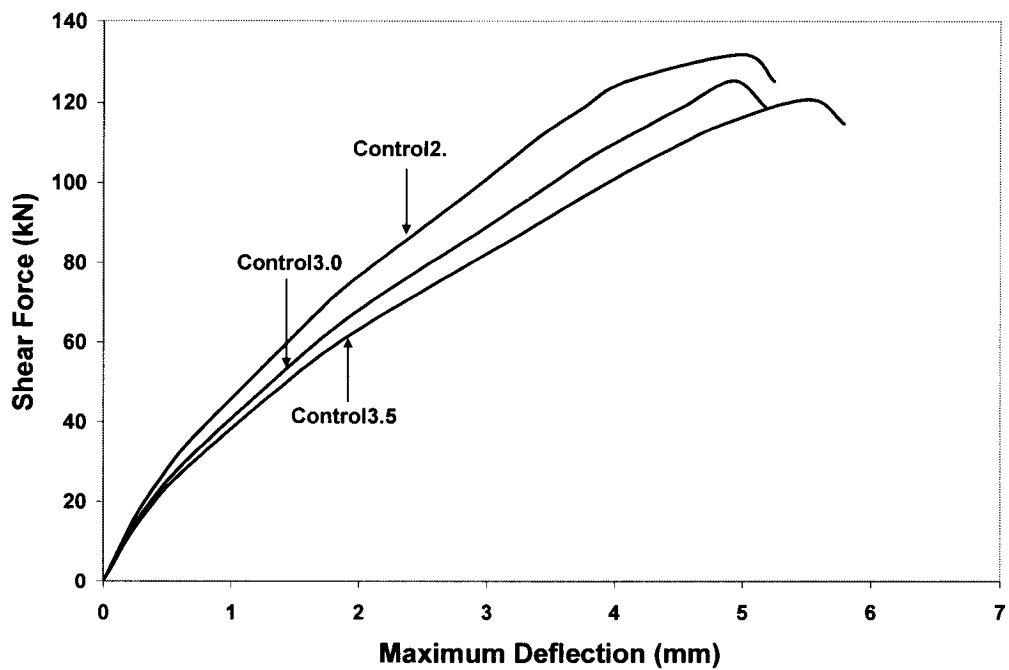


Fig. 5.26 Shear Force – Maximum Deflection curves of Control beams having various shear spans to effective depth (a/d) ratio i.e. 2.5, 3.0, and 3.5.

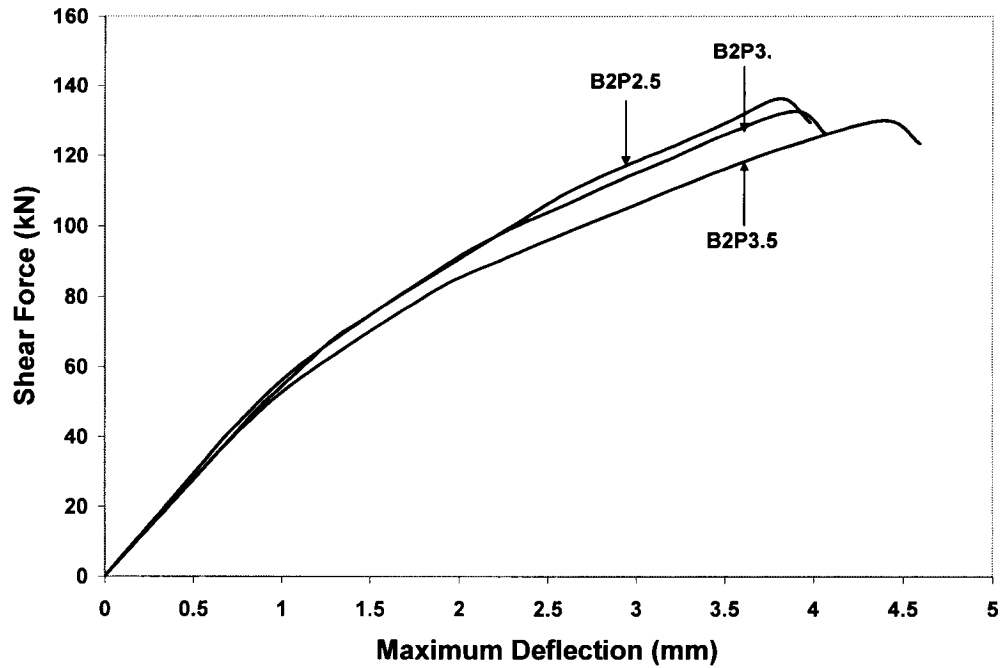


Fig. 5.27 Shear Force – Maximum Deflection curves of beams strengthened in flexural with two layers of Prestressed GFRP sheet having various shear spans to effective depth (a/d) ratio i.e. 2.5, 3.0, and 3.5.

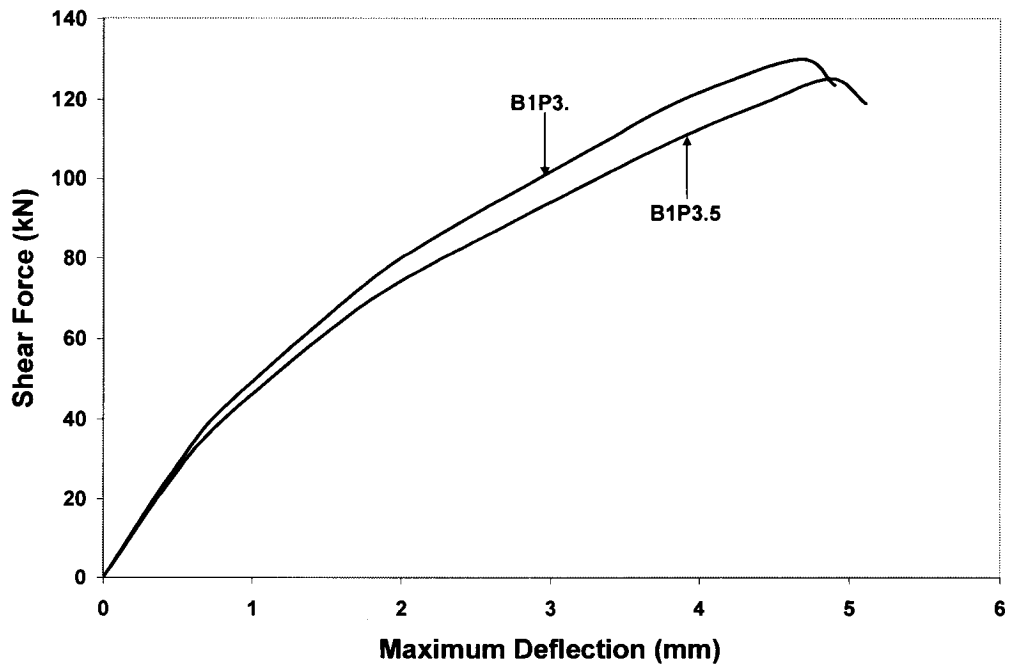


Fig. 5.28 Shear Force – Maximum Deflection curves of beams strengthened in flexural with one layer of Prestressed GFRP sheet having various shear spans to effective depth (a/d) ratio i.e. 2.5, 3.0, and 3.5.

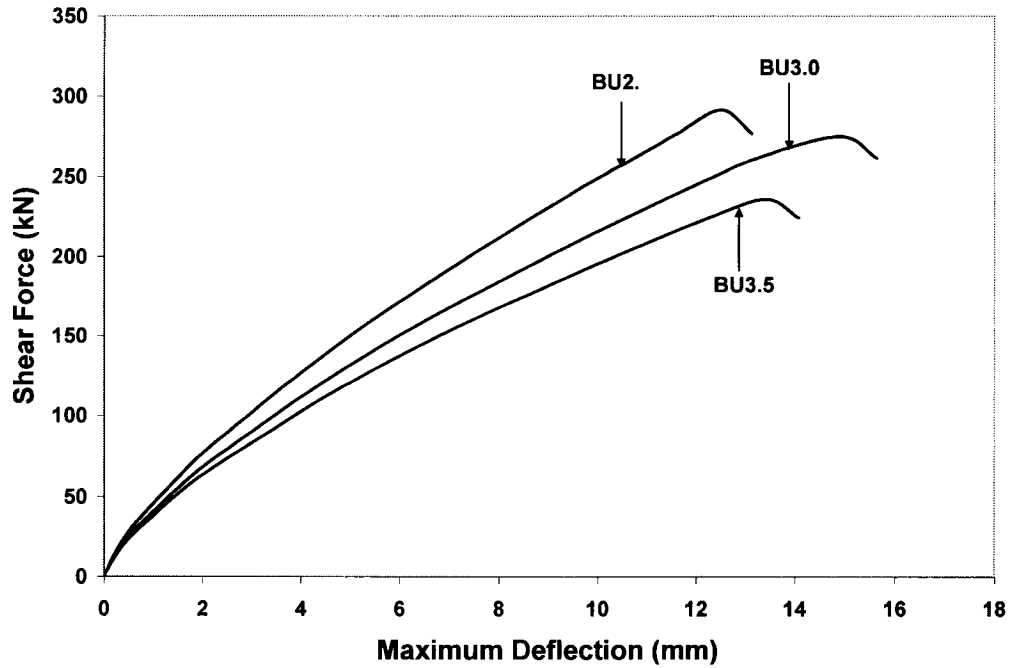


Fig. 5.29 Shear Force – Maximum Deflection curves of beams strengthened with U- straps Non-Prestressed GFRP sheet having various shear spans to effective depth (a/d) ratio i.e. 2.5, 3.0, and 3.5.

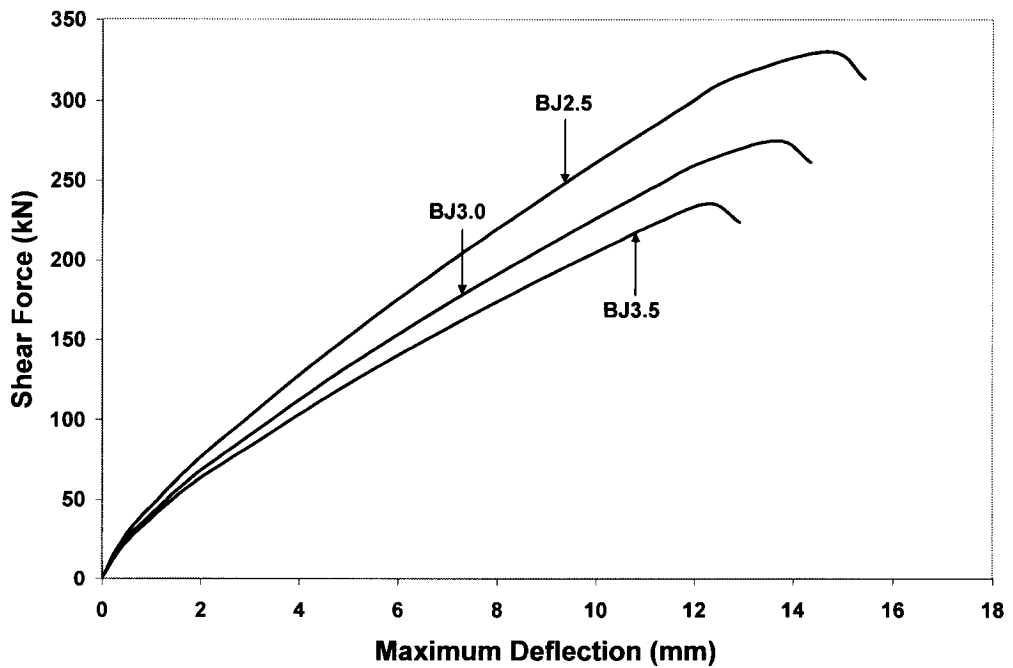


Fig. 5.30 Shear Force – Maximum Deflection curves of beams, strengthened with full Jacket of Non-Prestressed GFRP sheet in shear span, having various shear spans to effective depth (a/d) ratio i.e. 2.5, 3.0, and 3.5.

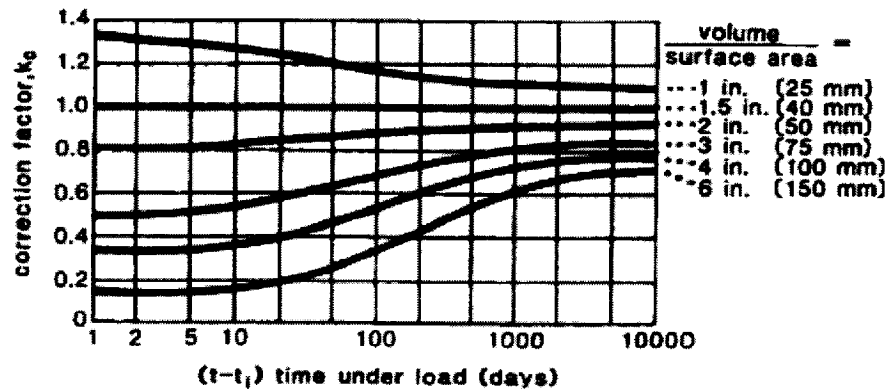


Fig. 5.31 Correction factor for Volume/surface Ratio (Collins and Mitchell, 1991)

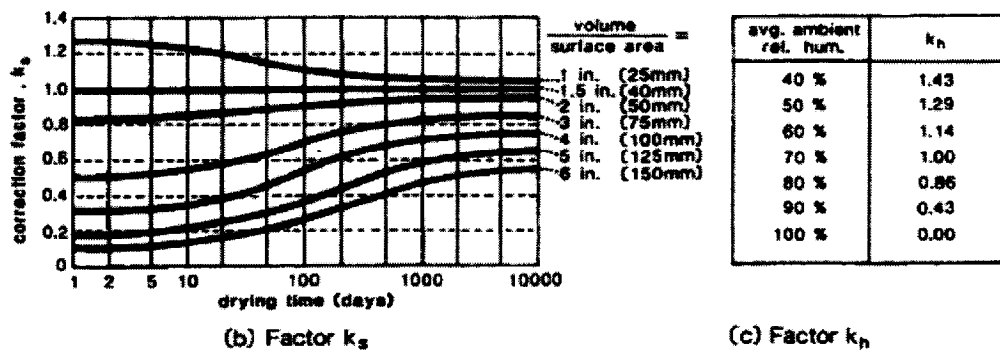


Fig. 5.32 Correction factor for relative Humidity (Collins and Mitchell, 1991)

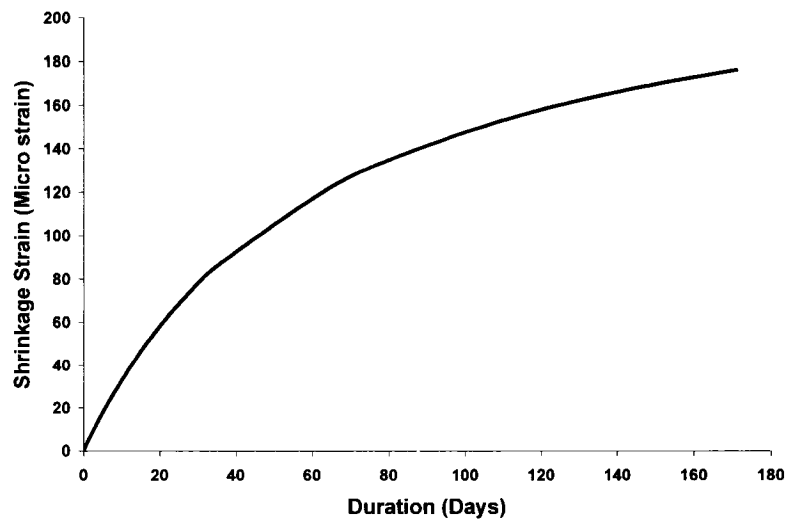


Fig. 5.33 Time versus shrinkage strain curve

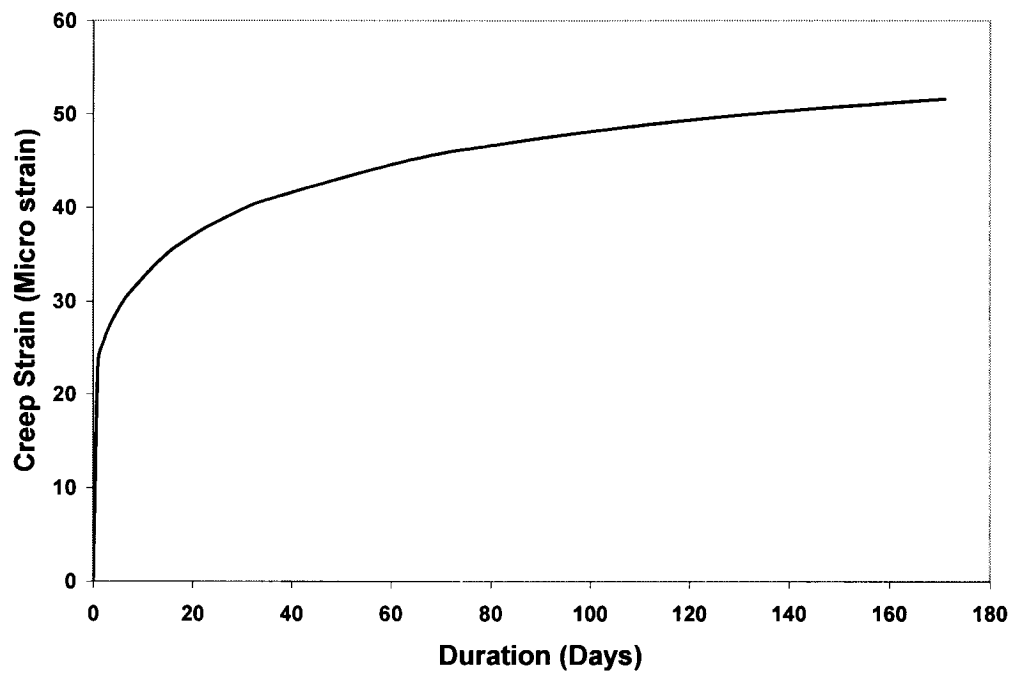


Fig. 5.34 Time versus Creep strain curve

CHAPTER 6

DISCUSSION

6.1 General

In this study, GFRP sheets were effectively used for flexural and shear strengthening of reinforced concrete beams. A new prestressing technique using square hollow section sheet anchor was developed. This chapter presents and discuss the GFRP prestressing operation and the prestressing system, as well as the flexural and shear behaviour of the un-strengthened and strengthened concrete beams and their comparison with the prediction of the theoretical model. The effectiveness of the prestressing system as well as the difficulties encountered are also outlined. The effect of the prestressed and non-prestressed GFRP sheets is highlighted and behaviour of the strengthened beams is outlined. Recommendations are given to modify the prestressing operation of GFRP sheet for field applications. The developed system is also compared with the prestressing techniques proposed in the literature.

6.2 Detail of Prestressing Technique and operation

The prestressing operation developed in this study was successfully performed on both flexural and shear deficient specimens of reinforced concrete beams. GFRP sheet was anchored to square hollow section and was prestressed on an external prestressing bed as described in chapter 3. Results indicated that the GFRP sheets were effectively prestressed using the developed anchorage system and that, by prestressing the sheets, the material was used more efficiently as compared to non-prestressed sheets. In this

section, the effectiveness of the prestressing system and the levels of prestress achieved are discussed. The prestressing operation conducted in the laboratory used inverted beams and is not directly applicable in the field. This section also includes recommendation on modification in the prestressing operation for field application.

6.2.1 GFRP Prestressing operation

Prestressing operations consisted of cutting GFRP sheet in required length and width, preliminary anchorage into square hollow section, final jacking and final gluing of prestressed sheet to the beam. Throughout the experimental program, four flexural deficient and six shear deficient concrete beams were strengthened by applying the prestressed GFRP sheet into the flexural face of the beam. The prestressing procedure was easy to apply. No difficulties were encountered when completing these steps in the laboratory where the beams were inverted. All steps can be easily performed in field application with slight modification. The light weight and flexibility of GFRP sheets make them easy to handle and place during prestressing operation. Only two persons with some experience in prestressing will be required to complete the prestressing operation in field application.

The anchorage system used in this experimental program consists of two square hollow sections and its details are given in Chapter 3. In the laboratory, one end of the GFRP sheet was kept fixed and a prestressing force was applied on the other end. In field application it will be difficult to apply prestressing force at one end because it will overturn the prestressing bed. Application of prestressing force at both ends of the sheet is recommended for field application. More detail on prestressing operation for field application is given in subsequent sections. The GFRP sheet was successfully prestressed to 30% and 45% of its ultimate strength. It was observed during testing that prestressing of GFRP sheet beyond 85% of its ultimate strength resulted in sheet

failure. Unidirectional GFRP sheet was used in this study. During anchor preparation, efforts were made that all fibres across the width of the sheet had the same length. Despite careful anchor preparation the length of the fibres across the width of the sheet was not the same. At average strength, approximately 85% of ultimate strength of the sheet, failure of some fibres started due to higher strain. The failure of some fibres across the width of the sheet resulted in overall reduction of cross sectional area and hence increased the stress level in the remaining fibres. This was a common cause of progressive failure of the GFRP sheet during prestressing.

The material characteristics and composition of the sheets minimize the risk of injury for the person conducting prestressing operations in the event of prestressing material failure during prestressing. During the prestressing of GFRP sheets, failure was by breaking and fracturing of the fibre individually across the sheet. As a result, the energy stored in the sheet was released gradually. Because of the light weight of the sheets, the kinetic energy associated with the sheet on release was low, and because of the large surface area to volume ratio of the sheet, this caused the velocity of the sheet at failure to dissipate quickly. Also, because of the wet epoxy on the sheet during prestressing, the energy was absorbed as the fibres must overcome the resistance of the epoxy as they broke away from the remainder of the sheet.

6.2.2 GFRP Anchorage System

During this study, various trials were made to develop an appropriate anchor system for prestressing of GFRP sheet. In the first trial pipe with circular cross section was used for end anchor of the sheet as shown in Fig. 6.1. GFRP sheets cut in appropriate width and length were wrapped and glued with epoxy on circular pipe at both ends. After curing of epoxy the pipe anchors were placed and hold with threaded steel bars in the prestressing bed. A prestressing force was applied to the sheet by

tightening nuts on threaded steel bars. With increase in prestressing force the sheet starts to slip on the circular section rather than prestressing. This method was abandoned due to failure of sheet anchor. In the second trial, a square hollow section was used to avoid slippage and opening up of sheet at anchor. The sheet was wrapped and glued with epoxy around the square hollow section at both ends as shown in Fig. 6.2. The square hollow sections were placed and held with threaded bars in the prestressing bed. Two threaded bars were passed through the centre of each hollow section and were attached to the vertical supports of prestressing bed. The GFRP sheet, which was wrapped around the square hollow section, was passed through the bottom face of the section. This arrangement resulted in an eccentricity equal to half of the depth of the square hollow section between the applied force in threaded bars and the prestressing force in the GFRP sheet. In this trial the unwinding of GFRP sheet at anchor did not happen as was the case in the circular section. This time the threaded bars started bending due to the moment created by the eccentric force applied and prestressing force. The bending of threaded bars limited the amount of applied force. This method was further refined in a third and final trial. To avoid eccentricity between applied force and prestressing force, another hollow section of the same dimension and cross sectional area was fastened to the sheet anchor at each end with the help of vertical nut and bolts. The additional square hollow section was also attached to the vertical support of the prestressing bed with the help of two threaded bars. The final anchorage system is shown in Fig. 6.3. The application of prestressing force at one end of the sheet anchor was kept fixed supported in prestressing bed and force was applied at other end by tightening nuts on threaded bars. In this arrangement the prestressing force was applied by the four threaded bars at one end. The application of force by four threaded bars helped in controlling the

eccentricity between applied and prestressing force in GFRP sheet. This type of anchorage system resulted in the application of force till failure of the sheet without any unwinding of sheet and bending of threaded bars. This arrangement was successfully used for prestressing of GFRP sheet in strengthening of four flexural deficient beams where force was applied manually by tightening nuts on threaded bars. Epoxy was applied to GFRP sheet and beam surface. The prestressed sheet was then glued to the beam with the help of epoxy resin. The prestressing force was transferred to the strengthened beams after curing of epoxy. Minimal loss of approximately 2% was recorded between the jacking force and transferred force. The difference between jacking and transferred force can be attributed to the deformations of the square hollow sections and threaded bars under sustained loading. The prestressing system was further refined by adding a hydraulic jack into the system which is shown in Fig. 6.4. In the six shear deficient beams, the prestressing force was applied by hydraulic jacks. The rest of operation was similar to the one used in strengthening of flexural deficient beams. In this case a loss of approximately 3% was recorded between jacking and transfer force. The increase in losses can be attributed to relation of hydraulic jack under sustained load. In comparison to other methods of FRP sheet prestressing available in the literature, the method developed in this study is the best one which has no constrain on the limit of prestressing force and minimum loss of prestressing force between jacking and transfer.

6.2.3 Distribution and Level of prestress in GFRP sheet

For the analysis method outlined in Chapter 5, the level of prestress across the width of the sheet was assumed to be constant. During the experimental study, local variations over the width and length of the sheet were noted. The reasons of these variations are discussed herein. Generally, however, the level of prestress in the FRP

sheet at a particular section along the length of the beam has been represented by an equivalent average fibre stress. The initial jacking prestress applied per sheet was approximately 32% of the ultimate tensile strength of the sheet in prestress level-1 and 47% in prestress level-2 of flexural deficient beams. The initial jacking prestress applied per sheet was approximately 41% of the ultimate strength of the sheet in shear deficient beams. These percentages were selected based on the extensive testing of GFRP sheets using different anchors to examine the effects of anchor details and stress concentration on the achievable level of prestress. It was difficult to accurately estimate the prestress levels in the sheet, since there was significant variation in the strain gauge measurements at different locations of the sheet. These local variations can be attributed to a number of effects, including difference in the lengths of individual fibre across the width of the sheet, variation in stress due to bending of the sheet anchor, localized slipping of the fibres and lateral straining of the strain gauges. The stress is, therefore, not only calculated from strain gauge data but also from load cell data at the time of jacking and transfer of the prestressing force. During prestressing of the GFRP sheets, the initial applied prestress per sheet varied slightly from beam to beam. After prestressing the sheets and transferring the force from the jacking apparatus to the concrete beam, a prestress loss was recorded between jacking and transfer prestressing levels. On average, the recorded losses were approximately 2.0% of the ultimate strength of the sheet in flexural deficient beams and 3% in shear deficient beams. At the time of testing the average stresses in the sheets were estimated to be approximately 30% of the sheet's ultimate tensile stress for beams B1P1, B2P1, and B3P1, and approximately 43% for beam B1P2. Similarly, at the time of testing, average stresses in the sheet were estimated to be approximately 38%

of the sheet's ultimate tensile stress for all shear deficient beams strengthened with prestressed GFRP sheet.

Multiple layers i.e. up to three layers of prestressed GFRP sheets were used in both flexural and shear deficient beams. Multiple layer prestressing provides a mean of customizing the strengthening and application of prestress over the length of the member. For example most prestress is required at mid span for flexural strengthening of a simply supported beam, therefore, more layers can be applied in the middle and fewer near the supports. The important point of consideration in multiple layer prestressing is the distribution of shear stress in the concrete cover at the bottom of the beam where the prestressed sheets are bonded. The effect of these shear stresses is significant in beams with more layers of prestressed GFRP sheet with high level of prestressing force. In beam B3P2.5, a delamination of the sheet extending to approximately 10cm at one end of the beam was recorded and is shown in Fig. 6.5. This portion of the delaminated GFRP sheet was outside the testing span of the beam and therefore no corrective measures were taken. No delamination was recorded in beams prestressed with one or two layers of GFRP sheets. This type of delamination alter that the high shear stresses at concrete and GFRP interface can initiate the failure of the concrete which limits the number of GFRP layers and level of prestress in sheet. The reason of such type of delamination can be attributed to the fact that at release of the prestressing force the strengthen beam has a tendency to shorten which depends on the axial rigidity of the beam. Similarly prestress is not reduced along the entire length of the beam because the adhesive restrains the shortening of the sheet. The deformation of adhesive layer causes the shear stress in adhesive and at the lower face of the concrete. The delamination at the ends of the beam happens due the fact that at termination point of prestress GFRP sheet, it tends to deform so that axial force

at the free end get zero. This large deformation results in high shear stresses concentration at the concrete to FRP interface. The delamination commences when such shear stresses exceed the shear strength of concrete. However it is reported in the literature that these high shear stresses exist over a very short region and the adhesive transfers the full prestress of the sheet over a very short length (Wight 1998). As long as the behaviour of the system remains elastic, the shear stress at the interface will be significantly higher when all the prestressed sheets terminate at the same point than when the cut-off points are distributed over a short length of the beam. The distribution of shear stresses is function of the prestressing force applied to each sheet, spacing of termination points, and stiffness of sheet and adhesive. If termination points are spaced sufficiently, the shear stress distribution of one termination point will be independent of the effects of adjacent one and hence will eliminate the danger of sheet delamination. It is observed in the experimental program that the failure/delamination of prestressed GFRP sheet happens at higher strain values as compared to the non-prestressed beams. Maximum strains of $12415 \mu\epsilon$ and $18021 \mu\epsilon$ were recorded at mid span of the sheet in beam strengthened with one layer of non-prestressed GFRP sheet (B1U) and beam strengthened with one layer of prestressed GFRP sheet (B1P1), respectively, before delamination of the sheet occurred. Similarly maximum strains of $14024 \mu\epsilon$ and $17611 \mu\epsilon$ were recorded at mid span of the sheet before delamination in beam B2U and B2P1, respectively. The high mid span strain value in prestressed sheet as compared to non-prestressed sheet shows the beneficial effect of prestressing on delaying delamination and utilization of maximum strength of this relatively low modulus material. The delay in delamination of prestress sheet is attributed to the presence of shear stress at concrete and adhesive interface induced by prestressing force. The applied load on the beam induces tensile stresses in the GFRP

sheet. The tensile force in the GFRP sheet results in shear stresses at the concrete and adhesive interface. When the induced shear stresses increased the concrete shear strength then delamination starts. On the other hand, prestressing stresses induce compressive force in the sheet. The shear stresses induced by this compressive force at concrete and adhesive interface is opposite in direction to the shear stresses induced by tensile force in the sheet, therefore, the delamination start at higher strain in prestress sheet. The delamination of FRP sheets is a major problem which limits the utilization of full strength of strengthening material. The delay in delamination of prestressed sheets is of significant value and one of the great contribution.

6.2.4 Modification in Prestressing Technique For Field Applications

The method of GFRP sheet prestressing developed in this study has been successfully used in the laboratory. The prestressing process includes anchor preparation and prestressing of sheet against vertical supports of the prestressing bed as shown in Fig 6.6. The beams were inverted and prestress sheets were applied to the bottom face of the beam. For flexural strengthening of beams and girders in the field, the developed prestressing system can not be used directly and hence need some modification. The present section includes recommendations on change in prestressing bed for field application. This section also includes recommendations on the entire strengthening procedure for field applications.

The anchor preparation using square hollow sections is the same as used in the laboratory and discussed in section 6.2.2. The GFRP sheet of required length and width is attached to end anchors with epoxy. After curing of epoxy, another similar hollow section is attached with anchor bolts to the bottom face such that sheet passes in between the two hollow sections. This arrangement helps in preventing the unwinding of the sheet and ensures the axial application of pre-stressing force.

In the laboratory two vertical supports with base steel plates were attached to strong floor of structural lab by nuts and bolts. The sheet was then prestressed against these vertical supports. This arrangement is not possible for field application. Therefore a modified system is recommended for field application which includes prestressing bed, vertical supports with prestressing bed, hydraulic bed etc. The flexural rigidity of the prestressing bed is most important to minimize the prestressing losses. I-section steel girder with steel plate stiffeners welded to the girder web is proposed for prestressing web. To get the required length of prestressing bed in the field two or more lengths of the I-section girders can be jointed with thick steel plates to the webs of two girders with the help of nuts and bolts. Vertical supports of appropriate height with frictionless rollers on top are then attached to the end of prestressing bed. Under each frictionless roller vertical hydraulic jacks are attached to load cells, prestressing bed, and two square hollow sections. The GFRP sheets, with square hollow section anchors at each end, are then placed on frictionless rollers and attached to the hydraulic jacks by threaded bars. Initial prestressing force is applied with threaded bars to ensure approximately uniform tensile stress across the width of the sheet. The sheet can then be prestressed to desired level with the help of hydraulic jacks. After prestressing the sheet to the desired level epoxy resin will be applied to the sheet and concrete surface. The whole prestressing assembly can then be lifted to the bottom of the beam by other two hydraulic jacks placed between scaffolding and prestressing bed. To remove the air bubbles between GFRP sheet and the concrete surface, epoxy can be injected through syringe and needle. After curing of epoxy the sheet jack is released and pre-stressing force will be transferred to the beam. The sheet is then cut at roller level. The over hang non prestressed sheet are then glued to

the beam or adjacent column with epoxy resin. A conceptual line diagram of the proposed process for field application is shown in Fig. 6.7.

For field application the entire process can be divided into six progressive steps, with number of activities occurring concurrently during each step. The initial step is reconnaissance of the structure. During this stage, details of the structure are observed and recorded. Visual survey is performed to assess the overall condition of the structure. Measurements are made and quality of concrete is assessed by non destructive testing. The location and quantity of the reinforcement is determined and marked with magnetic detection. Second step involves the restoration of the structural member and the design of the strengthening system. Loose and spalling concrete cover must be removed from the face of the member to which GFRP sheet will be attached. It must be replaced with new concrete cover. The surface must be repaired smooth and level finish. If shear reinforcement is to be added then the corner of the member should be rounded. Repair should be done with rapid set grout. The strength of the old and new material and the quality of bond between the two are critical to the effectiveness of the repair. Concurrently with repair operation, the strengthening system is designed. During the third step, scaffolding is erected under the structure. Prestressing bed is fabricated over the scaffolding. Concurrently the GFRP is cut in required length and width. The sheet is then attached to end anchor with the help of epoxy and left to cure. During fourth step, the GFRP sheet is stretched over the end rollers of the prestressing bed and attached to the hydraulic jack with the help of threaded bars. An initial force is applied manually by threaded bars to ensure uniform stress across the width of the sheet. The loading cell readings are then set to zero and prestressing force is applied by two hydraulic jacks till desire level of prestressing. In the fifth step, epoxy is applied to the FRP sheet and bottom surface of the structural

member. The whole assembly is then raised to the level such that GFRP sheet gets attached to the concrete surface. The sheet is pressed by hand to remove the entrapped air. If some spots have air bubbles then epoxy is injected into it by syringe and needle. The system is held in place till curing of epoxy. If required, transverse GFRP sheet straps are applied at sheet termination point which increases the shear strength and help in avoiding the peeling failure. In the sixth and final step, the prestressing force is gradually released from the hydraulic jack and sheet is cut at the anchor. This results in transfer of prestress from sheet and prestressing frame to the composite section. Compressive stresses are induced in the lower surface of the concrete girder and tensile stresses are induced on the upper portion of the member.

6.3 Flexural Behaviour of Strengthened Reinforced Concrete Beams

This section includes discussion on improvement in flexural behaviour of reinforced concrete beams strengthened with both prestressed and non-prestressed GFRP sheets. Comparison of experimental results to theoretical model predicted results is also included herein. The effectiveness of the prestressed GFRP sheets in comparison to non-prestressed GFRP sheet on the flexural behaviour of the reinforced concrete beam is also highlighted.

It is obvious from both experimental and predicted results that prestressed sheets improve the behaviour of beam under service loads. An increase in cracking load and decrease in crack widths and deflections has been noted in beams strengthened with prestressed GFRP sheets. Application of prestressed GFRP sheet also improves the ultimate strength of the strengthened concrete member as compared to beams strengthened with non prestressed GFRP sheets. Generally, when a sheet is prestressed, more efficient use of material is made, since the sheet can attain their full

strength. In case of non-prestressed GFRP sheet the failure tends to occur by peeling of the sheet much before it reaches the full tensile strength.

Results of both the predicted and observed behaviour of the experimental beams, when subjected to external loading are shown in Fig. 6.8 to Fig. 6.15. Differences between the experimental and predicted results are due to the damage of the reinforcement where strain gauges were attached, rupture or peeling of part of FRP sheet at LVDTs locations, diagonal shear cracks and bond slip. Damage of the reinforcement at the strain gauge locations results in yielding moments and stiffness lower than the predicted values at the section where strain gauges are located. In addition to above reasons, Response 2000 also includes the effect of tension stiffening which tends to overestimate the stiffness and strength of the beams. Similar discrepancies between predicted and experimental behaviour has been noted in the literature (Saadatmenesh and Ehsani, 1991, Wight, 1998, and El-Hacha 2000). Although the strength and stiffness of the section have been overestimated by the model, it can be seen that similar trends occur in both predicted and experimental curves. The effect of the addition of prestressed and non-prestressed sheet is similar in both the predicted and experimental values. The comparison of the experimental and predicted results has been made on the basis of various variables used in this study. Fig. 6.8 and Fig 6.9 show the comparison of experimental and predicted results, in shape of moment-curvature and load-deflection curves, for the beams strengthened with one layer of non-prestressed and prestressed GFRP sheet. Fig. 6.10 and Fig 6.11 show the comparison of experimental and predicted results, in shape of moment-curvature and load-deflection curves, for the beams strengthened with two layers of non-prestressed and prestressed GFRP sheet. Fig. 6.12 and Fig 6.13 show the comparison of experimental and predicted results, in shape of moment-curvature and

load-deflection curves, for the beams strengthened with three layers of prestressed GFRP sheet. Fig. 6.14 and Fig 6.15 show the comparison of experimental and predicted results, in shape of moment-curvature and load-deflection curves, for the beams strengthened with two and three layers of prestressed GFRP sheet to emphasize the restoration of flexural strength of beams with flexural reinforcement losses. The effect of the sheets considering serviceability and ultimate conditions are discussed in sections 6.3.1 and 6.3.2, respectively. The predicted and observed values of significant load are shown in Table 6.3.

6.3.1 Cracking and Serviceability

The addition of prestressed GFRP sheet improved the cracking behaviour of the beams and reduced the deflections in all strengthened beams. As predicted, increase in cracking load was observed with increase in number of prestressed GFRP sheet and level of prestressing in the sheets.

As predicted, addition of non-prestressed GFRP sheets shows negligible effect on the behaviour of the beam before formation of first crack and increase in cracking strength of the beam. An increase of 3% was predicted in the cracking strength of the beam strengthened with one layer of non-prestressed GFRP sheet. This observation was confirmed by experimental results. In the experimental results, the addition of one prestressed GFRP sheet results in an increase of 60% in the cracking strength of the beam. The model predicts an increase of 55% in cracking strength of the beam strengthened with one layer of prestressed GFRP sheet. Similarly, addition of three prestressed GFRP sheets result in an increase of 129% in the cracking strength of the experimental beam as compared to 132% increase predicted by model for the same beam. The difference between the predicted and observed cracking strength may be due to difference in the prestress assumed in the model and the actual prestress level

in the GFRP sheet, imperfection in beams construction and variability of constituent material properties. The model used 157MPa prestress. The predicted and observed values confirmed that the addition of prestressed sheet significantly increased the cracking strength of the concrete beam, whereas the addition of non-prestressed sheet has negligible effect on the cracking strength. Cracks were marked and their widths were measured in experimental beams. Table 6.1 and Table 6.2 shows the observed values of summation of crack widths at various loading stages for beams which were un-strengthened, strengthened with one layer of un-prestressed sheet, strengthened with one layer of prestressed sheet, and strengthened with three layers of prestressed sheets. The careful examination of results shows that addition of prestressed sheet tremendously reduces the crack widths, especially under service load conditions, as compared to un-strengthened beam and beam strengthened with non-prestressed GFRP sheet. At loading of 70kN which is approximately 50% of the ultimate load of control1 beam, the summation of crack widths was 0.56mm in control1 beam, 0.44mm in beam B1U, and 0.08mm in beam B1P1. In other words, at load of 70kN which is considered as upper level of service load, application of un-prestressed GFRP sheet resulted in a decrease of 21% in crack widths as compared to 85% by application of prestressed GFRP sheet. At loading of 70kN which is approximately 56% of the ultimate load of control2 beam, the summation of crack widths was 0.64mm in control2 beam and 0.18mm in beam B3P1. This shows a decrease of 72% in cracks widths at this loading stage by application of three layers of prestressed GFRP sheet. In this study, one variable was the amount of flexural reinforcement. Control2 and B3P1 beams were reinforced with 500 mm^2 of ordinary steel reinforcement and Control1 and B2P1 beams with 600 mm^2 of ordinary steel reinforcement. The aim was to simulate the loss of flexural reinforcement due to

corrosion or other reason and study the effect of an extra layer of prestressed GFRP layer in deteriorated beams on restoring their strength and stiffness. A mass loss of 16% in main flexural reinforcement was selected for this study. The crack width could not be observed due to use of oil painting therefore comparison of crack control is not possible. However, the predicted values show that an extra layer of prestressed GFRP sheet in beam B3P1 increased the cracking strength of the beam, which is comparable to beam B2P1. This shows that addition of one layer of prestressed GFRP sheet effectively restores the cracking strength loss due 16% mass loss of main flexural reinforcement.

The summations of observed crack widths were plotted versus externally applied loads in Fig. 6.16 and Fig. 6.17. The curves show that cracks width has been effectively reduced by application of prestressed GFRP sheet as compared to non-prestressed sheet. It is worth to mention that reduction in crack width by application of prestressed GFRP sheet is more pronounced under service load conditions. Strengthening with prestressed GFRP sheet is an active strengthening technique which induced compressive stresses in the bottom fibres and tensile stresses in the top fibres of the beam. The external loads need first to neutralize these compressive stresses and then induce tensile stresses in the bottom fibre of the beam in excess to tensile strength of the concrete to develop cracks in the concrete beam. This is the main reason of delaying crack propagation, reduction in crack widths at any specific load, and increasing cracking strength of the beam strengthened with prestressed GFRP sheets. From both experimental and predicted moment curvature and load-displacement curves it is clear that bonding of prestressed GFRP sheets to the tensile face of the concrete beam provides extensive crack control. During testing many cracks of small width were observed in beams strengthened with prestressed GFRP

sheets while fewer cracks of comparatively larger width were observed in un-strengthened beams. On the moment curvature curves, this effect is shown since at any load the curvature is reduced with the addition of the sheets. The reason for this effect is, as section deforms, the GFRP sheets generate a tensile force in the strengthened beam which adds to the tensile force in the flexural steel reinforcement and equilibrate the concrete compressive force. The balancing of internal forces occurs at comparatively smaller deflections in strengthened beams as compared to un-strengthened beams because of the tensile force of the sheet. Hence smaller deflections at all loading stages results in comparatively smaller curvatures in the strengthened beam. Further decrease in displacements and curvatures in the beams strengthened with prestressed GFRP sheets is due to the larger tensile forces carried by the sheets and the delay in the initial cracking.

Bonding of prestressed GFRP sheets to the tensile face of a concrete beam not only provides extensive crack control and limits deflection in the concrete beam, but also improves the durability of the concrete structure, since deterioration is directly related to the width of the cracks and its growth with time. As the cracks become wider, the reinforcement gets much exposed to oxygen and water-borne chlorides that accelerate corrosion of the steel reinforcement. In cases when the structure is exposed to severe weather conditions such as successive freeze-thaw cycles, these cracks provide a path for the deposition of water. In situations when de-icing salts are used, the structure may deteriorate at an early age. Closing of these cracks cannot be achieved by using non-prestressed sheets. In field applications, where the structural member may already cracked, strengthening with prestressed GFRP sheets combined with epoxy injection techniques will be helpful in closing cracks in the structure. The closing of cracks will not only restore the flexural stiffness of the structural member but will also reduce the

ingress of de-icing salt and water and hence will improve the durability. Hence beams strengthened with prestressed GFRP sheets will be less susceptible to corrosion than beams strengthened with non-prestressed FRP sheets.

6.3.2 Beam Behaviour under Yielding and Ultimate Loading

In the experimental program, all beams were tested under monotonic loading till failure. The beams strengthened with non-prestressed and prestressed GFRP sheets showed increase in yielding and ultimate strength in comparison to un-strengthened control beams. The prestressed sheets were more effective at strengthening concrete beams than the non-prestressed sheets. Prestressing of GFRP sheets helped in utilizing the maximum strength of the material and hence was more effective in increasing the yielding and ultimate strength of the strengthened beams.

Fig 6.8 to 6.15 show that application of GFRP sheet to the bottom face of beams results in delay of steel yielding until significantly higher loads are attained. This is confirmed from both predicted and observed results. It is observed that the beam strengthened with one layer of non-prestressed GFRP sheet, the steel yields at 16% higher load than un-strengthened beam i.e. Control1. An increase of 16% in yielding load with comparison to Control1 beam has also predicted by the model. In the beam strengthened with one layer of GFRP sheet prestressed to 32% of its ultimate strength, the yielding of steel happened at 21% higher load than control1. The model predicted 22% increase in yielding load for this beam. When the beam was strengthened with one layer of GFRP sheet prestressed to 47% of its ultimate strength the yielding occurred at 23% higher load than Control1 beam. The model predicted 25% increase in yielding loads for this beam. The reason for increase in yielding load of the beams is that application of GFRP sheet increased the quantity of flexural reinforcement and hence some portion of tensile stresses were carried by the GFRP sheet which reduced

the stresses in the steel reinforcement at a given load. In case of beam strengthened with prestressed GFRP sheet, a further increase in yielding load happened due to transfer of compressive stresses in bottom fibres of the beam by prestressing force in prestressed sheet. It is clear from above discussion that approximately 37% further increase in yielding load was observed and predicted in beam strengthened with prestressed sheet than beam strengthened with non-prestressed sheet.

The observed data shows that in the beam strengthened with two layers of non-prestressed GFRP sheet, the steel yields at 22% higher load than un-strengthened beam i.e. Control1. An increase of 19% in yielding load with comparison to Control1 beam has also predicted by the model. It is interesting to note doubling the quantity of GFRP sheets did not result in doubling the increase in yielding load of the beam. This may be due to the fact that distribution of tensile forces between the steel and GFRP sheet is not linear but depends on the deformation at specific load. In the beam strengthened with two layers of GFRP sheet prestressed to 32% of its ultimate strength, the yielding of steel happened at 33% higher load than control1. The model predicted 36% increase in yielding load for this beam. The reason for increase in yielding load of the beams is same as discussed above except this time an approximately 64% further increase in observed and predicted yielding load of beam strengthened with prestressed sheet than beam strengthened with non-prestressed sheet. This is due to the fact that two layers of prestressed GFRP sheet doubled the magnitude of prestressing force, and hence higher compressive stresses were induced in the bottom fibres of the beam, which resulted in further increase in yielding load of the beam. In summary, it can be stated that increase in quantity of prestressed GFRP sheet is more effective in delaying the yielding of steel than non-prestressed GFRP sheet.

Beam representing 16% mass loss of reinforcement due to environmental conditions was strengthened with three layers of prestressed GFRP sheet. An increase of 66% in yielding load was observed in strengthened beam as compared to the companion un-strengthened beam i.e. Control2. The model predicted 70% increase in yielding load of this beam. The reason for increase in yielding strength is the same as discussed above. Comparing the observed results of beam, B3P1, with Control1 shows that yielding of the steel reinforcement occurred at 37% higher load than control1. This is confirmed by the predicted result which shows 40% increase in yielding load of the beam in comparison to the control1 beam. The predicted and observed yielding loads of beams B2P1 and B3P1 are approximately comparable. This finding give us the opportunity to argue that one layer of the prestressed GFRP sheet having cross-sectional area of 150 mm^2 has compensated the 16% mass loss of the steel reinforcement i.e. 100 mm^2 .

Fig 6.8 to 6.15 show that applications of GFRP sheet to the bottom face of beams also resulted in increasing the ultimate strength of the strengthened beams. This is confirmed from both predicted and observed results. It is observed that the ultimate strength of beam strengthened with one layer of non-prestressed GFRP sheet was increased by 18% over the un-strengthened beam i.e. Control1. An increase of 16% in ultimate strength was predicted by the model for the same beam. Beam strengthened with one layer of GFRP sheet prestressed to 32% of its ultimate strength shows an increase of 22% in ultimate strength as compared to ultimate strength of un-strengthened beam i.e. Control1. The model also predicted 22% increase in ultimate strength of this beam. When the beam was strengthened with one layer of GFRP sheet prestressed to 47% of its ultimate strength, 22% increase in ultimate strength was observed as compared to Control1. The model predicted 25% increase in ultimate

strength for this beam. The reason for increase in ultimate strength of the beams is that application of GFRP sheet increased the quantity of flexural reinforcement which helped in utilizing more concrete compressive strength at ultimate stage. In case of beam strengthened with prestressed GFRP sheet, a further increase in ultimate strength delayed in delamination of sheet as discussed in section 6.2.3. It is clear from above discussion that approximately 22% further increase in ultimate strength was observed and predicted in beam strengthened with prestressed sheet than beam strengthened with non-prestressed sheet.

The observed data shows an increase of 24% in ultimate strength of the beam strengthened with two layers of non-prestressed GFRP sheet than un-strengthened beam i.e. Control1. An increase of 26% in ultimate strength was predicted by the model for the same beam. In beam strengthened with two layers of GFRP sheet prestressed to 32% of its ultimate strength, the ultimate strength was increased by 39% as compared to control1. The model predicted 40% increase in ultimate strength for this beam. The reason for increase in ultimate strength of the beams is same as discussed above except this time an approximate 63% further increase in observed and predicted ultimate strength of beam strengthened with prestressed sheets than the beam strengthened with non-prestressed sheets. This is due to the fact that two layers of prestressed GFRP sheet doubled the magnitude of prestressing force and hence increased the magnitude of shear stress at adhesive and concrete interface, which results further delay in delamination of sheet and hence increased the ultimate strength. In summary it can be stated that increase in quantity of prestressed GFRP sheet is more effective for ultimate strength of the beam than non-prestressed GFRP sheet.

Beam representing 16% mass loss of steel reinforcement and strengthened with three layers of prestressed GFRP sheet shows an increase of 64% in ultimate strength as compared to the companion un-strengthened beam i.e. Control2. The model predicted 63% increase in yielding load of this beam. The reason for increase in yielding strength is the same as discussed above. Comparing the observed results of beam, B3P1, with Control1 shows an increase of 37% in ultimate strength than control1. This is confirmed by predicted result which shows 40% increase in ultimate strength in comparison to the control1 beam. The predicted and observed ultimate loads of beams B2P1 and B3P1 are approximately comparable. This further strengthened our argument that one layer of the prestressed GFRP sheet having cross-sectional area of 150 mm^2 has compensated the 16% mass loss of the steel reinforcement i.e. 100 mm^2 .

The mode of failure in all beams was delamination of sheet except beam B1P2 in which rupture of the sheet occurred. The delamination either initiated on sheet ends or at centre of the sheet. As discussed in section 6.2.3, the sheet termination points experience peak in shear stress at adhesive and concrete interface. If these shear stresses are higher than concrete shear strength a thin layer of concrete and the sheet will peel off the concrete beam. The design of end zone with staggered termination of sheet ends, as proposed in section 6.2.3, along with U-strap of sheet in end zone will probably delay this mode of failure.

The other failure occurred in concrete cover at the level of the internal reinforcement. The reason for this failure can be explained by considering cover concrete between adjacent cracks. The presence of flexural steel reinforcement provides weak section for failure. Concrete between two adjacent cracks act as cantilever with load acting at tip of the cantilever. The length of this cantilever is

equal to depth of concrete cover. The unbalanced horizontal force in the sheet at two adjacent cracks level act as load. When the tensile stresses at the end of this cantilever exceed the tensile strength of the concrete then delamination of sheet along with cover concrete occurred. This type of failure was observed in beam B2U. As observed in this study the induction of prestress in sheet can prevent the premature delamination.

6.4 Shear Behaviour of Strengthened Reinforced Concrete Beams

This section includes discussion on improvement in shear capacity of reinforced concrete beams strengthened with prestressed GFRP sheets attached to the bottom face of the beam, U-straps, and U-jackets in the shear span of the beam. Comparison of experimental results to theoretical model predicted results is also included herein. The effect of the transverse clamping stress, induce by prestressed GFRP sheets attached to the flexural face of the strengthened beam, on the shear capacity of the reinforced concrete beam is also highlighted.

It is obvious from experimental results that prestressed sheets improved the shear behaviour of the beam. The model, which is based on the modified compression field theory, ignores the effect of transverse clamping stresses, predicted nominal improvement in the shear capacity of the strengthened beam. The effect of transverse clamping stresses on the shear capacity was more pronounced in the beams with smaller shear span to effective depth (a/d) ratios. The model predicted same increase in shear capacity of beam strengthened with U-straps or U-jackets for the beam tested at a/d ratio of 3.0 and 3.5. However, the observed results show that delamination of U-straps limited the increase in shear capacity as compared to U-jackets. In case of U-straps strengthening technique, the failure tends to occur by peeling of the sheet much before it reaches the full tensile strength.

Results of both the predicted and observed behaviour of the experimental beams, when subjected to external loading are shown in Fig. 6.18 to Fig. 6.20. The model used in this study used the modified compression field theory and ignores the beneficial effect of transverse clamping stresses on the shear capacity which tends to underestimate the shear capacity of the beam strengthened with prestressed GFRP sheets. The magnitude of these clamping stresses depends on the a/d which is confirmed from the observed results. The increase in shear capacity of the beam by prestressed GFRP sheet decreases with increases in a/d ratio. Anyhow, this is main reason of differences between the experimental and predicted results of the beams strengthened with prestressed GFRP sheets. The difference in the predicted and observed results of the beams strengthened with U-straps is due to premature delamination of GFRP sheet. The model assumes full bond between the concrete and sheet till failure and hence overestimate the capacity of such beams. The premature delamination was not observed in beams strengthened with U-jackets. Although the shear strength of the beam strengthened with U-jackets has been overestimated by the model, it can be seen that similar trends occur in both predicted and experimental curves. The comparison of the experimental and predicted results has been made on the basis of various variables used in this study. Fig. 6.18 and Fig 6.20 shows the comparison of experimental and predicted results, in shape of shear force-mid span displacement, for the beams tested on a/d ratios of 2.5, 3.0, and 3.5. Application of U-straps and U-jackets in shear span was used with the aim to study the effect of quantity of shear reinforcement on shear strengthening of the beams. As quantity of GFRP sheet in shear span was increased, the magnitude of induced shear stresses in the sheet was reduced and hence premature delamination was avoided. The predicted and observed values of significant load for are shown in Table 6.1.

6.4.1 Shear Behaviour of Beams Tested on a/d Ratio = 2.5

Five beams were tested on a/d ratio of 2.5. This a/d ratio was selected on the basis that it is lower limit of a slender beam. One beam was un-strengthened acted as control, one was strengthened with two layers and one with three layers of prestressed GFRP sheet bonded to flexure face of the beam, one was strengthened with 100mm wide U-straps of GFRP sheet bonded @ 200mm c/c in the shear span of the beam, and one was strengthened with U-Jacket of GFRP sheet in the shear span of the beam. Analytical study was done for all above configuration of strengthening by model used in this study. The observed and predicted results are presented in Fig. 6.18 in the shape of shear force-mid span displacement curves. The predicted and observed shear strengths of the beams are presented in Table 6.4.

The Control beam was tested on clear span of 2800mm. The observed shear strength of the control beam was 138.4kN as compared to predicted shear strength of 131.85kN. Although the ultimate shear strength of the control beam has been underestimated by the model, it can be seen in Fig. 6.18 that similar trends occur in both predicted and experimental curves. The difference may be due to variability in constituent material properties, aggregate size etc. The beam strengthened with three layers of prestress GFRP sheet was also tested on clear span of 2800mm. The beam was supported on roller support at both ends. We get local failure in beam over right support before shear failure of the beam. The support failure was observed at approximately 250kN applied shear force followed by a big diagonal crack started at right support and propagates toward the load point. It can be seen in Fig. 6.21. The overall failure of the beam is believed not a shear failure but local failure of concrete due to concentrated line load of reaction force. The maximum shear force of 262.7kN was observed in this beam. The model predicted 145.34kN shear strength for this

beam. Although the observed shear capacity is 90% higher than the control and 81% higher than predicted shear capacity but still it is believed that this is much less than the actual capacity of the beam. To avoid such failure, clear span was reduced to 2600mm and steel plate of size 300x200x10mm was used over each support in the rest of the testing. The number of layers of prestressed GFRP sheet was reduced from 3 to 2 & 2 to 1 layers in remaining beams. These arrangements helped in avoiding the occurrence of local support failure. The beam strengthened with two layer of prestressed GFRP sheet failed at shear load of 311.8kN. The observed data shows strain of approximately 0.21% in flexure reinforcement which is yielding strain of the flexural reinforcement. The shear capacity of the beam was increased to the extent that it was at verge of transformation from shear failure to flexural failure. The model predicted 136.2kN shear capacity of the beam. Thus the observed capacity was 129% more than the predicted results. This tremendous increase of 129% in shear capacity confirm our argument for the previous beam B3P2.5, that it was not reach to its ultimate shear capacity at the time of failure. The big difference in predicted and observed results is believed due to ignorance of transverse clamping stresses induced by load and prestressed GFRP sheets. As discussed in chapter 5 of this thesis the a/d ratio has vital role on shear capacity of the beam. The reason is that close to the load and to the reaction, there exist transverse clamping stresses from application of the load itself. In addition, prestressed GFRP sheet also induced clamping stresses in the middle region of shear span i.e. between the load point and support. These clamping stresses tend to locally increase the shear strength. The sectional model used in this study is more conservative in predicting the shear strength of the beam strengthened with prestressed GFRP sheet as analysis ignore the beneficial effect of small compressive clamping stress induced by load and prestressed sheets. This is reason of

big difference in predicted and observed shear capacity of beam strengthened with prestress GFRP sheet.

The ultimate shear capacity of 226.4kN was observed in the beam strengthened with U-straps of GFRP sheet. The model predicted shear capacity of 291.6kN for this beam. An increase of 121% and 64% in shear capacity as compared to control beam was predicted and observed, respectively, for beam BU2.5. The model overestimate the shear capacity of the beam as it ignore the possibility of premature delamination.

It is observed in testing that failure occurred due to premature delamination of the GFRP U-straps. The model assumed full bond between the sheet and beam till failure of GFRP sheet. The main reason of premature delamination is that at onset of shear crack, tensile stresses were induced in to GFRP straps. These tensile stresses acted as shear stress on epoxy and concrete interface. Once the induced shear stresses at concrete and epoxy interface exceed the shear strength of concrete, a sudden delamination of GFRP straps occurred. In the other words the shear strength of concrete limits the utility of GFRP sheet. This argument was confirmed analytically by limiting the rupture strain of the sheet to 60% of its ultimate strength. The prediction of the model was plotted against the observed results and was found in good agreement results. This comparison curves is shown in Fig. 6.22.

The ultimate shear capacity of 358.9kN was observed in the beam strengthened with U-jacket of GFRP sheet. The model predicted shear capacity of 330.36kN for this beam. An increase of 151% and 159% in shear capacity as compared to control beam was predicted and observed, respectively, for beam BJ2.5. It is observed that flexural reinforcement reached to strain of 0.32% which is range of strain hardening. This means that mode of failure has been changed from shear to flexural. The failure occurred by delamination of the GFRP U-jacket. The reason of delamination is same

as discussed above, but this time the surface area was double as compared to the U-straps, which reduces the magnitude of shear stresses at epoxy concrete interface at any specific external load. The beam reaches to its maximum shear capacity as predicted by the model. The observed result is higher than predicted because the failure mode was changed in this case. The comparison of theoretical and observe results in Fig. 6.18 are in good agreement. The increase in GFRP quantity for shear reinforcement avoided the premature delamination problem.

6.4.2 Shear Behaviour of Beams Tested on a/d Ratio = 3.0

Five beams were tested on a/d ratio of 3.0. This a/d ratio was increased to study the effect of slenderness of the beam on the shear capacity. One beam was un-strengthened acted as control, one was strengthened with one layer and one with two layers of prestressed GFRP sheet bonded to flexure face of the beam, one was strengthened with 100mm wide U-straps of GFRP sheet bonded @ 200mm c/c in the shear span of the beam, and one was strengthened with U-Jacket of GFRP sheet in the shear span of the beam. Analytical study was done for all above configuration of strengthening by model used in this study. The observed and predicted results are presented in Fig. 6.19 in the shape of shear force-mid span displacement curves. The predicted and observed shear strengths of the beams are presented in Table 6.4.

All five beams were tested on clear span of 2600mm. The Ultimate shear strength of 125.6kN was observed in the testing of control beam. The model predicted shear strength of 125.4kN for control beam. In addition to close resemblance in observed and predicted ultimate shear strength, it can be seen in Fig. 6.19 that similar trends occur in both predicted and experimental curves. In the testing of beam strengthened with one layer of prestress GFRP sheet, ultimate shear strength of 157.7kN was recorded. The model predicted 130.16kN ultimate shear strength for this beam. The

addition of one layer of prestressed GFRP sheet to the flexural face of the increased the ultimate shear capacity of the beam by 26% than the control and 21% than predicted shear capacity. The beam strengthened with two layers of prestressed GFRP sheet failed at shear load of 198.4kN. The model predicted 132.6kN shear capacity of the beam. This time the observed capacity was 58% higher than control beam and 50% higher than the predicted results. The failure of both beams occurred after delamination of GFRP sheet. It was observed that big diagonal shear crack was initiated at relatively higher load than control beam. The shear deformation at the crack pushed the sheet downward and a sudden delamination of the sheet was observed. The difference in predicted and observed results is believed due to ignorance of transverse clamping stresses. In comparison to beam tested at a/d ratio of 2.5 the increase in shear capacity was relatively low. It is believed that with increase in a/d ratio the effect of transverse clamping stresses due to load and reaction reduced. In these beam the clamping stresses induced by prestressed GFRP sheet, in the middle region of shear span i.e. between the load and reaction points, were active. In other words the magnitude traverse clamping stresses was reduced and hence the increase in shear capacity was reduced. This argument supports the idea that these transverse stresses are function of a/d ratio. As said before, the sectional model used in this study is more conservative in predicting the shear strength of the beam strengthened with prestressed GFRP sheet. It ignore the beneficial effect of small compressive clamping stress induced by load and prestressed sheets.

The ultimate shear capacity of 251.30kN was observed in the beam strengthened with U-straps of GFRP sheet. The model predicted shear capacity of 275.3kN for this beam. An increase of 120% and 100% in shear capacity as compared to control beam was predicted and observed, respectively, for beam BU3.0. The model overestimate

the shear capacity of the beam as it ignore the possibility of premature delamination. It is observed in testing that failure occurred due to premature delamination of the GFRP U-straps whereas model assumed full bond between the sheet and beam till failure of GFRP sheet. As describe before, the main reason of premature delamination is excess of shear stress on epoxy and concrete interface than shear strength of concrete which initiate sudden delamination of GFRP straps. In the other words the shear strength of concrete limits the utility of GFRP sheet. Again, the rupture strain of the sheet to limited 60% of its ultimate strength in analytical study and results were compared to experimental data. This time the model predicted 221.4kN ultimate shear strength at this rupture strain which is less than observed results. The prediction of the model was plotted against the observed results in Fig. 6.23. Although, the predicted ultimate strength is less than observed but the curve in Fig. 6.23 shows same trend.

The ultimate shear capacity of 313.1kN was observed in the beam strengthened with U-jacket of GFRP sheet. The model predicted shear capacity of 275.3kN for this beam. An increase of 119% and 149% in shear capacity as compared to control beam was predicted and observed, respectively, for beam BJ3.0. Like BJ2.5, it is observed that flexural reinforcement reached to strain of 0.37% which is range of strain hardening. This means that mode of failure has been changed from shear to flexural. The failure occurred by delamination of the GFRP U-jacket. The reason of delamination is same as discussed above, but this time the surface area was double as compared to the U-straps, which reduces the magnitude of shear stresses at epoxy concrete interface at any specific external load. The beam reaches to its maximum shear capacity as predicted by the model. The observed result is higher than predicted because the failure mode was changed in this case. The comparison of theoretical and

observed results in Fig. 6.18 is in good agreement. It is observed again that increase in GFRP quantity for shear reinforcement avoided the premature delamination problem.

6.4.3 Shear Behaviour of Beams Tested on a/d Ratio = 3.5

Five beams were tested on a/d ratio of 3.5. This a/d ratio was further increased to study the effect of slenderness of the beam on the shear capacity. One beam was un-strengthened acted as control, one was strengthened with one layer and one with two layers of prestressed GFRP sheet bonded to flexure face of the beam, one was strengthened with 100mm wide U-straps of GFRP sheet bonded @ 200mm c/c in the shear span of the beam, and one was strengthened with U-Jacket of GFRP sheet in the shear span of the beam. Analytical study was done for all above configuration of strengthening by model used in this study. The observed and predicted results are presented in Fig. 6.20 in the shape of shear force-mid span displacement curves. The predicted and observed shear strengths of the beams are presented in Table 6.4.

All five beams were tested on clear span of 2600mm. The Ultimate shear strength of 133.9kN was observed in the testing of control beam. The model predicted shear strength of 120.9kN for control beam. Although the ultimate shear strength of the control beam has been underestimated by the model, it can be seen in Fig. 6.20 that similar trends occur in both predicted and experimental curves. The difference may be due to variability in constituent material properties, aggregate size etc. In the testing of beam strengthened with one layer of prestress GFRP sheet, ultimate shear strength of 177.6kN was recorded. The model predicted 125.316kN ultimate shear strength for this beam. The addition of one layer of prestressed GFRP sheet to the flexural face of the beam increased the ultimate shear capacity by 32% than the control and 42% than predicted shear capacity. The beam strengthened with two layers of prestressed GFRP sheet failed at shear load of 186.5kN. The model predicted 129.88kN shear capacity

of the beam. This time the observed capacity was 40% higher than control beam and 44% higher than the predicted results. The failure of both beams occurred after delamination of GFRP sheet. It was observed that big diagonal shear crack was initiated at relatively higher load than control beam. The shear deformation at the crack pushed the sheet downward and a sudden delamination of the sheet was observed. As discuss before, the difference in predicted and observed results is believed due to ignorance of transverse clamping stresses. In comparison to beam tested at a/d ratio of 3.0 the increase in shear capacity by prestressed sheet is almost similar. This is contradictory to the comparison of beam results tested on a/d ratio of 2.5 and 3.0. This strengthen our argument that with increase of a/d ratio the effect of load and reaction on traverse clamping stresses diminishes and only the clamping stresses due to prestress sheet in the centre of the shear span remain active. This is main reason that big difference in increase of shear capacity was noted when we a/d ratio was increased from 2.5 to 3.0 but negligible difference was noted when a/d ratio was increased from 3.0 to 3.5. As said before, the sectional model used in this study is more conservative in predicting the shear strength of the beam strengthened with prestressed GFRP sheet. It ignore the beneficial effect of small compressive clamping stress induced by load and prestressed sheets.

The ultimate shear capacity of 251.90kN was observed in the beam strengthened with U-straps of GFRP sheet. The model predicted shear capacity of 235.97kN for this beam. An increase of 95% and 88% in shear capacity as compared to control beam was predicted and observed, respectively, for beam BU3.5. The strain of 0.35% was noted in the flexural reinforcement at ultimate load. As the a/d ratio increased, the beam acted as slender beam and flexural behaviour dominate the shear behaviour. In

contrast to beam tested on a/d ratio of 2.5 and 3.0, delamination of U-straps was not the limiting criterion in this case.

The ultimate shear capacity of 290.9kN was observed in the beam strengthened with U-jacket of GFRP sheet. The model predicted shear capacity of 235.97kN for this beam. An increase of 95% and 117% in shear capacity as compared to control beam was predicted and observed, respectively, for beam BJ3.5. Like BJ2.5 and BJ3.0, it is observed that flexural reinforcement reached to strain of 0.87% which is range of strain hardening. This means that mode of failure has been changed from shear to flexural. The failure occurred by delamination of the GFRP U-jacket. The beam reaches to its maximum shear capacity as predicted by the model. The observed result is higher than predicted because the failure mode was changed in this case. The comparison of theoretical and observed results in Fig. 6.18 is in good agreement. It is observed again that increase in GFRP quantity for shear reinforcement avoided the premature delamination problem.

6.5 Long Term Loss

In the beam strengthened with prestressed GFRP sheet, concrete is subjected to creep strains which reduce sheet prestress in the long term. Similarly the epoxy at the sheet and concrete interface is subjected to high shear stresses which may cause some shear deformations in the epoxy layer and hence reduces the prestress in the beam. The shrinkage strains also causes some long term prestress losses. Relaxation of fibres under sustain load, elastic shortening of GFRP layer due to application of subsequent layers may be other causes of prestress losses.

In deflections calculations of beams it is normally assumed that the concrete section acts as an elastic and homogeneous material. This assumption is only approximately correct, because the elastic modulus for concrete is not a constant

value for all stress levels. The elastic modulus varies with the age of the concrete and is influenced by other factors such as the mix design, the loading history and the environment. Concrete gain strength with time and its modulus of elasticity increases with time. It also undergoes time-dependent volumetric changes as it creeps under sustained loads and shrinks upon drying. The result of creep and shrinkage is reduction of tension in the prestressing sheet and compression in the concrete. The short-term response implies no shrinkage or creep of the concrete. The long-term response implies all of the creep and shrinkage have taken place. The knowledge of creep and shrinkage properties of concrete will allow the prediction with time of prestress losses in the sheet. To evaluate the effect of creep and shrinkage on long term prestress losses an experimental program was carried out. The losses due to these factors were predicted by using the analytical tool as presented in chapter 5. The observed and predicted results of creep and shrinkage loss are presented in Table. 6.6. Similarly, the observed and predicted results of creep and shrinkage strains are plotted in Fig. 6.24 and 6.25, respectively. The test was carried out for six months. It is clear from both curves that concrete show higher rate of increase of creep and shrinkage strains in initial period and the gain in strain reduces with time. The predicted and observed results are in good agreement and have similar trend. The maximum creep strain of $52 \mu\epsilon$ was observed due to creep after six months. The small creep value is due to the fact that stress level in concrete was very low due to low prestressing force in concrete. The model also predicted creep strain of $52 \mu\epsilon$ for same period. The predicted and observed curves of shrinkage strain are in good agreement and have similar trend. Like creep strain, the rate of increase in shrinkage strain was high in early days of concrete age and reduces with time. The maximum shrinkage strain of $170 \mu\epsilon$ was observed at age of six months whereas model predicted $176 \mu\epsilon$ of

shrinkage strain at same age. The creep and shrinkage strain reduce the prestress in the GFRP sheets. The total prestress loss of $222 \mu\epsilon$ was observed due to these two factors.

To observe the total prestress losses due to creep, shrinkage, shear strain of epoxy, fibre relaxation, elastic shortening of GFRP layers etc., in the beam prestressed with GFRP sheet, the strain losses on strengthened beam were recorded. The strain due to creep and shrinkage were subtracted from the total losses and the reminder was considered as loss due shear strain in epoxy layer, fibre relaxation, elastic shortening of GFRP layers due to application of subsequent layers and any other unknown factors. The results of total losses and total strain due to creep and shrinkage are compared in Table 6.7. The strain data of the prestressed beam show similar trend. The total loss of $319 \mu\epsilon$ was recorded at the age of 6 months. The loss due to other factors than creep and shrinkage was calculated as $97 \mu\epsilon$ at the age of six months. It is worth mentioning here that total prestress loss was measured under no load condition. The presence of load on beam might affect these figures.

In summary it can be stated that shrinkage strain is main contributor of prestress losses. Low creep strain was recorded due to low prestressing force. Despite all these factor, a total strain of $319 \mu\epsilon$ have very little effect on prestress losses due to the fact that GFRP sheet is a relatively low modulus material. The modulus of elasticity of the GFRP sheet is 27400MPa. The total prestress loss of 8.7MPa is recorded due to total strain of $319 \mu\epsilon$. The prestress in sheet at jacking time was 208MPa. This means that a total loss of approximately 4% occurred in the beam prestress with GFRP sheet. This is very low losses as compared to losses in beam prestressed with conventional prestressing wires where long term losses amounts to approximately 25%. The reason of low prestress loss can be contribute to relatively low modulus of elasticity of the

GFRP sheet and small prestressing force as compare to conventional prestressing wires. For design purpose it is necessary to develop prestress loss equations. It is obvious from comparison of observed and predicted results of creep and shrinkage that equations used for these losses predicted pretty good results. However, more investigation is needed for losses due to other factors like fibres relaxation, shear strain at epoxy interface, elastic shortening due to multiple layer application etc. The presented results are first step in this area and can be refined with further research.

Table 6.1 Summation of crack widths at various loading for beam strengthened with one layer of GFRP sheet

Load (kN)	<u>Summation of Cracks Widths (mm)</u>		
	Control1	B1U	B1P1
0	0.00	0.00	0.00
50	0.20	0.14	0.02
60	0.46	0.28	0.04
70	0.56	0.44	0.08
80	0.72	0.44	0.16
90	0.80	0.62	0.34
108	0.98	0.82	0.54
125	1.58	1.16	0.78
142	2.04	1.68	1.16
152	4.76	1.84	1.34
176		5.1	2.36
184			4.06

Table 6.2 Summation of crack widths at various loading for beam strengthened with three layers of GFRP sheets

Load (kN)	<u>Summation of Cracks Widths (mm)</u>	
	Control2	B3P1
0	0.00	0.00
50	0.28	0.04
60	0.40	0.10
70	0.64	0.18
80	0.70	0.30
90	0.86	0.36
100	1.30	0.54
110	1.60	0.90
120	2.04	1.36
125	3.30	1.50
140		1.60
175		2.72
190		3.40
206		4.74

Table 6.3 Comparison of significant predicted and observed loads for flexural deficient beams

Beam	Crack Load (kN)		Yielding Load (kN)		Ultimate Load (kN)	
	Predicted	Observed	Predicted	Observed	Predicted	Observed
Control1	18.35	18.05	140.20	138.36	155.78	152.17
B1U	18.86	18.44	162.94	160.38	181.05	180.28
B1P1	28.45	29.85	170.69	167.38	189.66	185.65
B1P2	29.15	****	174.92	169.67	194.35	185.03
B2U	19.43	***	171.35	165.02	196.82	188.41
B2P1	43.74	***	190.39	183.41	218.69	210.84
Control2	16.34	15.35	111.10	107.10	130.70	127.74
B3P1	37.43	35.65	188.40	177.80	213.40	208.98

**** Pre-Cracked Beam

*** Oil painted Beams

Table 6.4 Comparison of predicted and observed shear forces for shear beams

Specimen	Ultimate Shear Force (kN)					
	a/d = 2.5		a/d = 3.0		a/d = 3.5	
	Predicted	Observed	Predicted	Observed	Predicted	Observed
Control	131.8	138.4	125.4	125.6	120.9	133.9
BU	291.6	226.3	275.3	251.1	236.0	251.9
BJ	330.4	358.9	275.3	313.1	236.0	290.9
B1P	-----	-----	130.2	157.7	125.3	177.6
B2P	136.2	311.8	132.6	198.4	129.9	186.5
B3P	145.3	262.7	-----	-----	-----	-----

Table 6.5 Increase in predicted and observed shear capacity of beams by addition of prestressed GFRP sheets to bottom face of the beams

Specimen	Ultimate Shear Force (kN)					
	a/d = 2.5		a/d = 3.0		a/d = 3.5	
	Predicted	Observed	Predicted	Observed	Predicted	Observed
Control	131.8	138.4	125.4	125.6	120.9	133.9
B1P	-----	-----	130.2 (4%)	157.7 (26%)	125.3 (4%)	177.6 (32%)
B2P	136.2 (3%)	311.8 (125%)	132.6 (6%)	198.4 (58%)	129.9 (7%)	186.5 (40%)
B3P	145.3 (10%)	262.7 (90%)	-----	-----	-----	-----

Table 6.6 Comparison of observed and predicted creep and shrinkage strains

Duration (Days)	Creep Strain ($\mu\epsilon$)		Shrinkage Strain ($\mu\epsilon$)		Total Strain (Creep + Shrinkage)	
	Observed	Predicted	Observed	Predicted	Observed	Predicted
1	24	14	4	4	28	17
2	25	18	8	7	33	25
3	27	19	11	11	38	30
4	28	20	15	14	43	34
5	29	20	17	18	46	38
6	30	21	20	21	50	42
7	31	23	22	24	53	47
14	35	34	42	44	77	78
21	37	37	58	60	95	97
28	39	39	73	74	112	113
35	41	41	85	86	126	127
63	45	44	116	120	161	165
81	47	47	130	135	177	182
111	49	49	150	154	199	203
141	50	51	162	166	212	217
171	52	52	170	176	222	228

Table 6.7 Table showing total observed strain minus creep and shrinkage strain

Duration (Days)	Total strain Observed On Prestressed beam ($\mu\epsilon$)	Observed strain (Creep + shrinkage) ($\mu\epsilon$)	Strain due to Other reasons ($\mu\epsilon$)
1	46	28	18
2	66	33	33
3	76	38	38
4	87	43	44
5	92	46	46
6	100	50	50
7	106	53	53
14	146	77	69
21	169	95	74
28	200	112	88
35	216	126	90
63	257	161	96
81	275	177	98
111	293	199	94
141	310	212	98
171	319	222	97

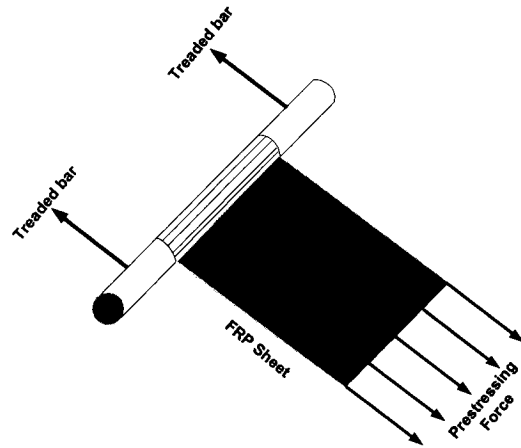


Fig. 6.1 Prestressing anchor using circular section (Trial-1)

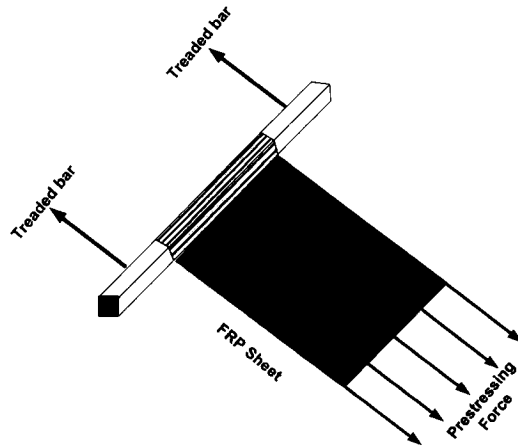
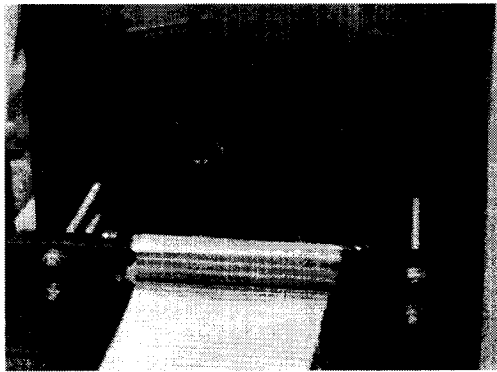
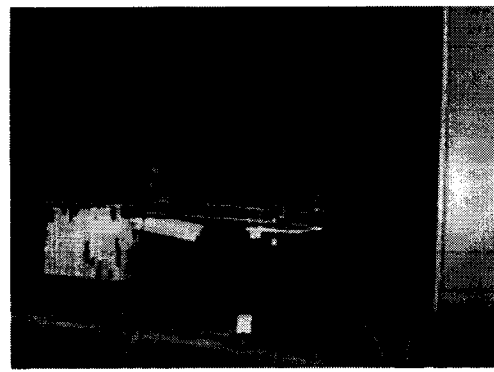


Fig. 6.2 Prestressing anchor using single square hallow section (Trial-2)



a) Anchor detail at fixed end



b) Prestressing with threaded bars

Fig. 6.3 Prestressing anchor using two square hollow sections and threaded bars

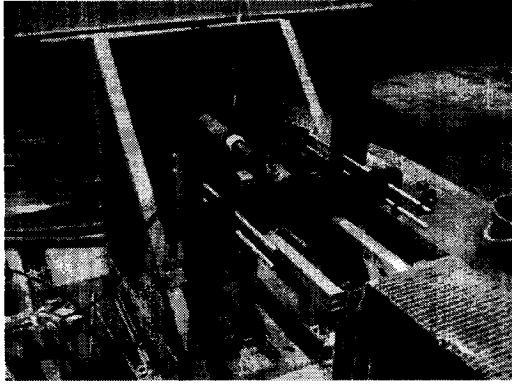


Fig. 6.4 Prestressing anchor using two square hollow sections and hydraulic jack

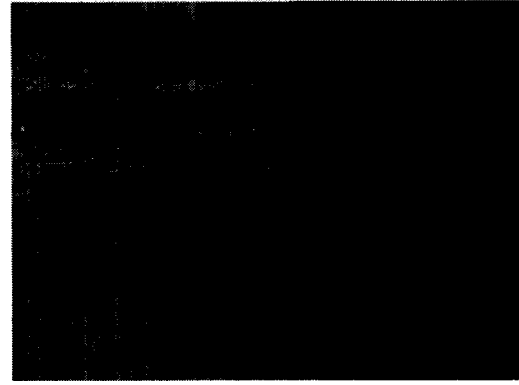


Fig 6.5 Delamination of Sheets at beam end after transfer of prestressing force



a) Preparation of sheet anchor



b) Prestressing of GFRP sheet

Fig 6.6 Prestressing of GFRP sheet process

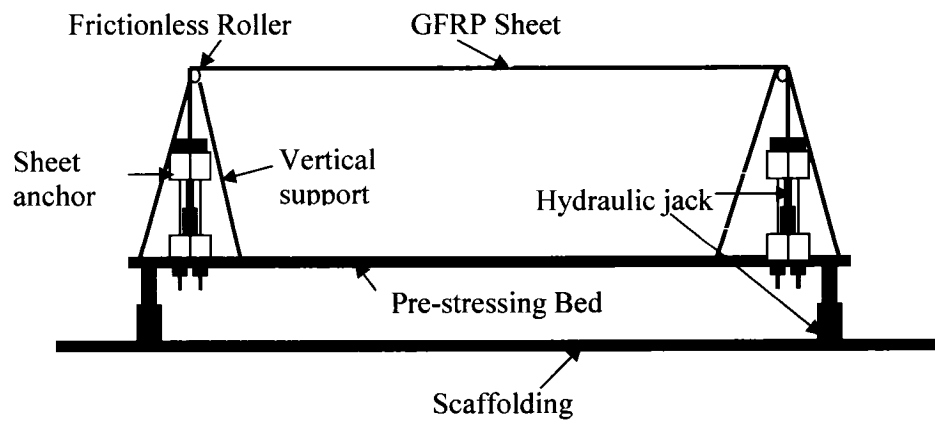


Fig 6.7 Modification in prestressing process for field application

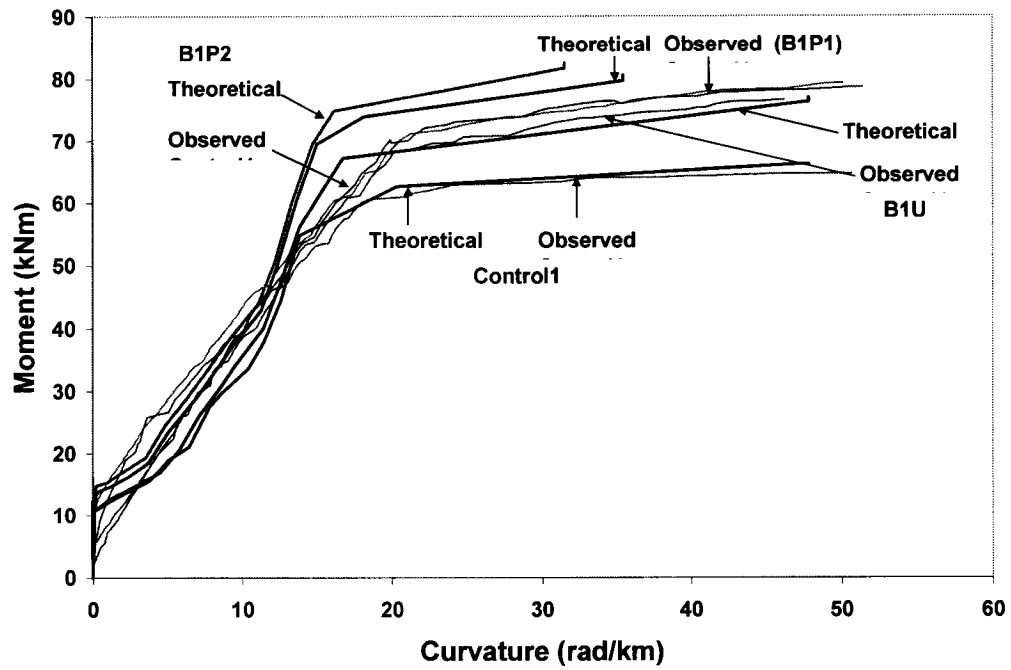


Fig. 6.8 Theoretical & observed moment-curvature curves for beam having 600 mm^2 of flexural reinforcement and strengthened with one layer of GFRP sheet

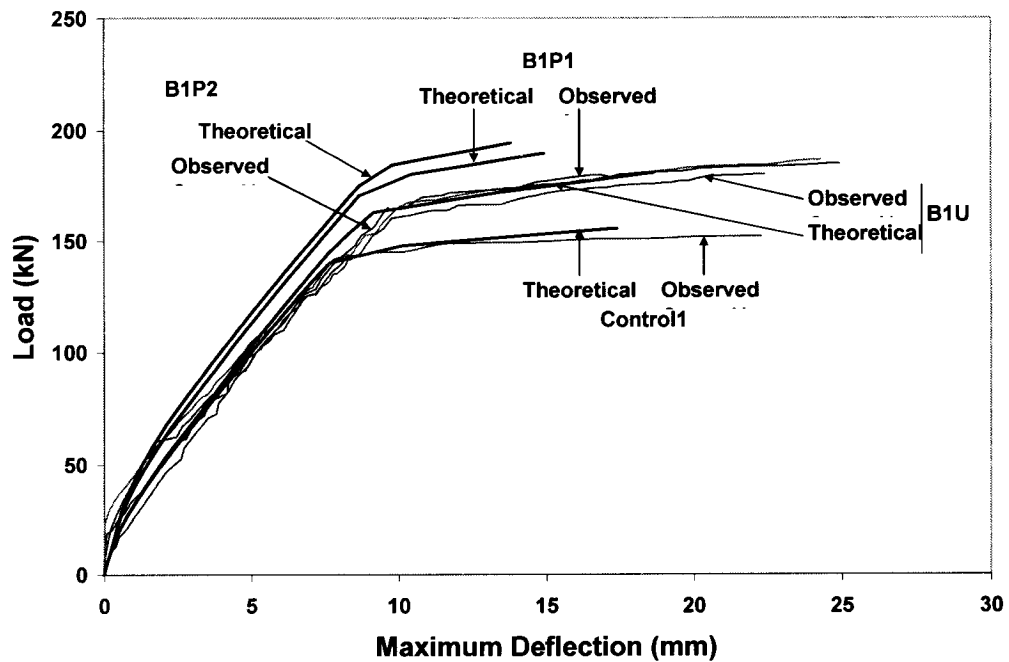


Fig. 6.9 Theoretical & observed load-displacement curves for beam having 600 mm^2 of flexural reinforcement and strengthened with one layer of GFRP sheet

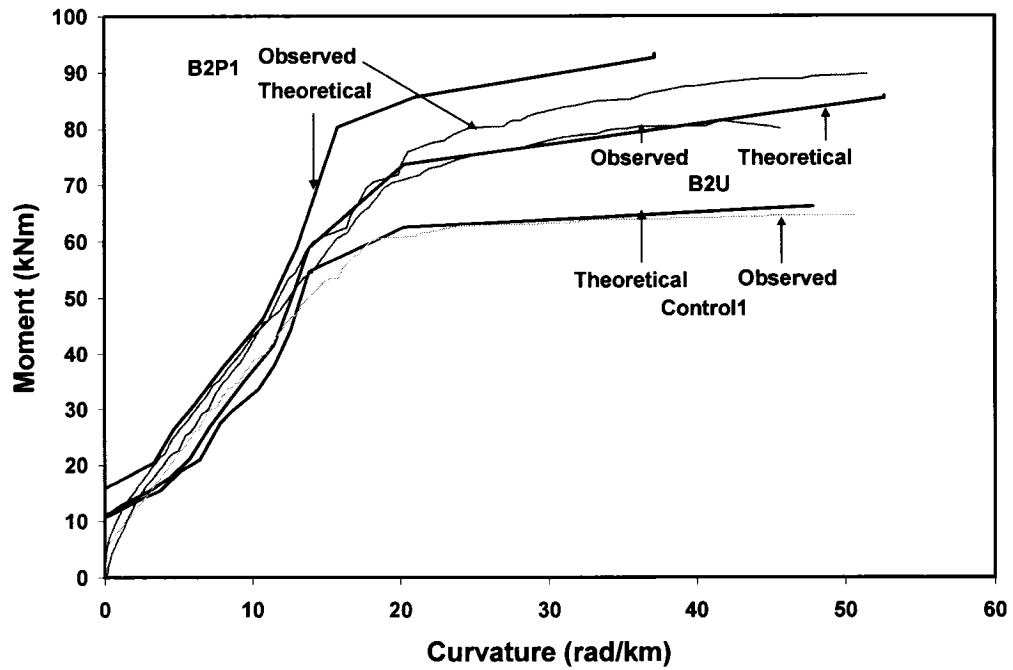


Fig. 6.10 Theoretical & observed moment-curvature curves for beam having 600 mm^2 of flexural reinforcement and strengthened with two layers of GFRP sheets

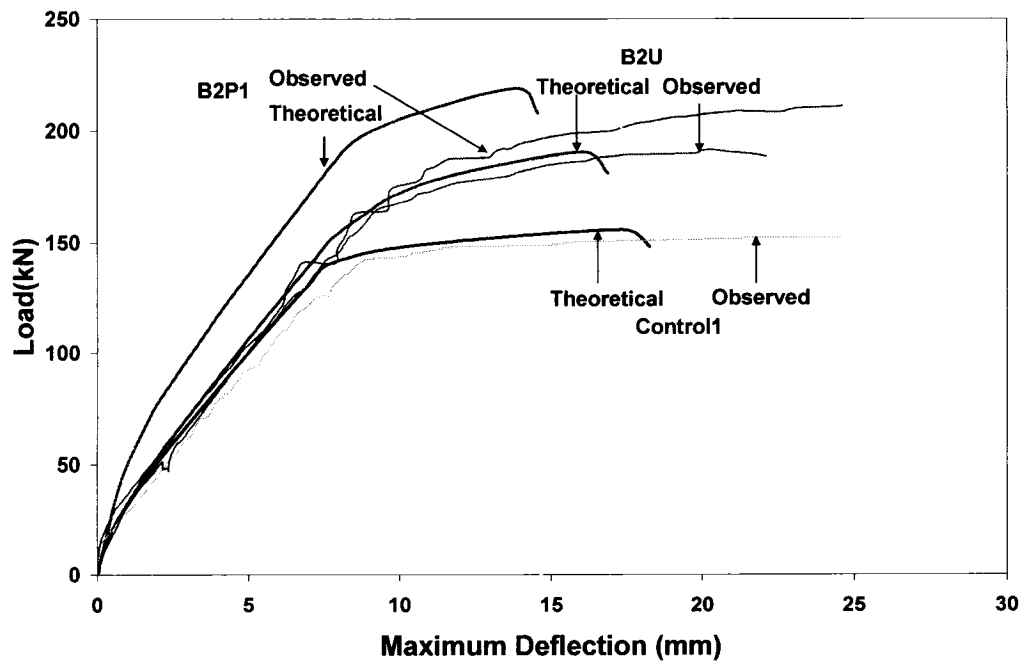


Fig. 6.11 Theoretical & observed load-displacement curves for beam having 600 mm^2 of flexural reinforcement and strengthened with one layer of GFRP sheet

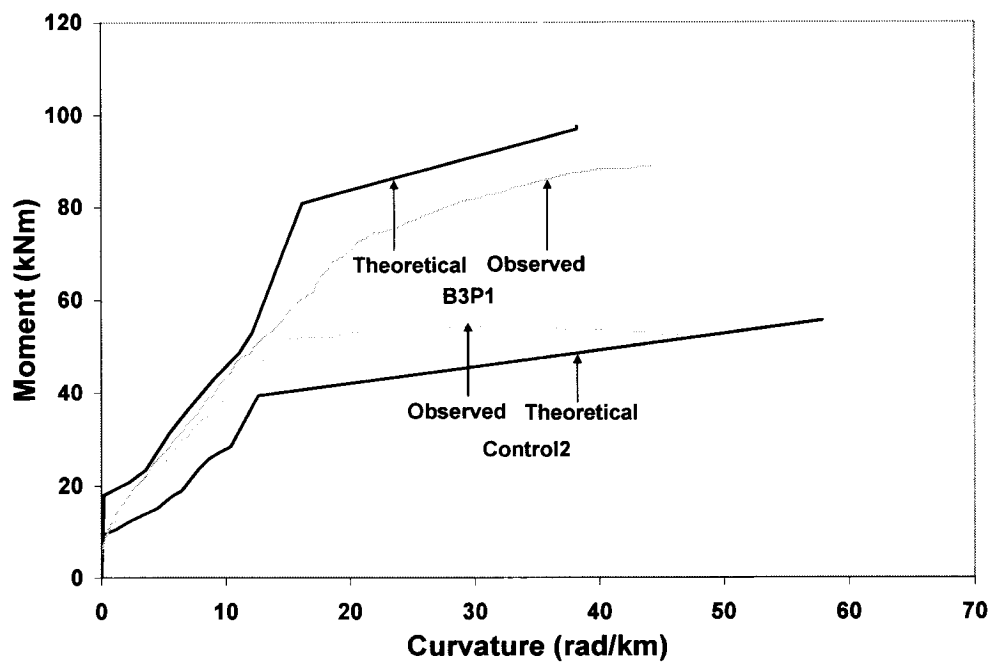


Fig. 6.12 Theoretical & observed moment-curvature curves for beam having 500 mm^2 of flexural reinforcement and strengthened with three layers of GFRP sheets

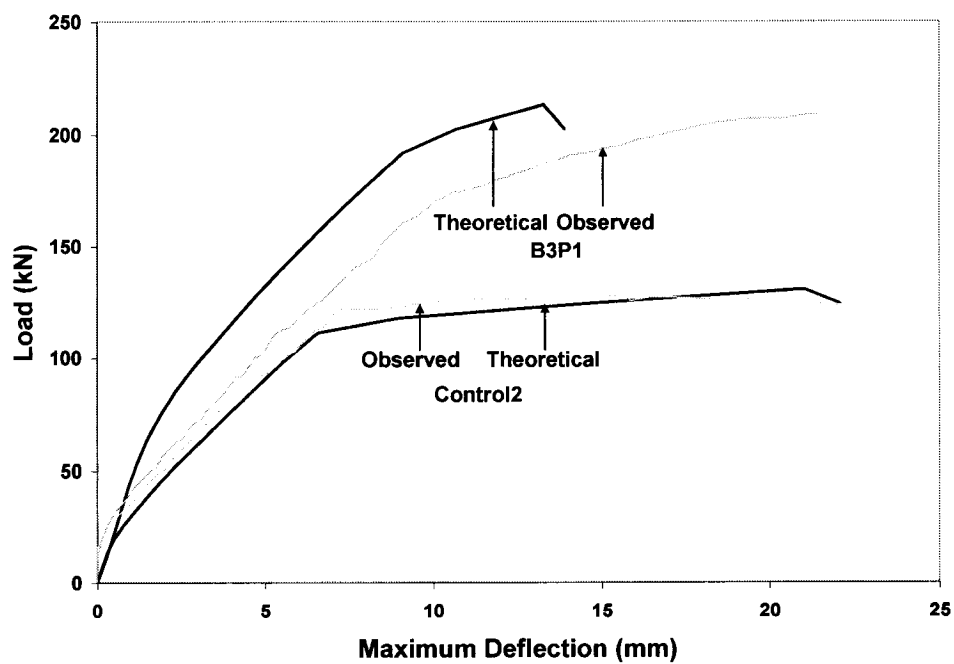


Fig. 6.13 Theoretical & observed load-displacement curves for beam having 500 mm^2 of flexural reinforcement and strengthened with three layers of GFRP sheets

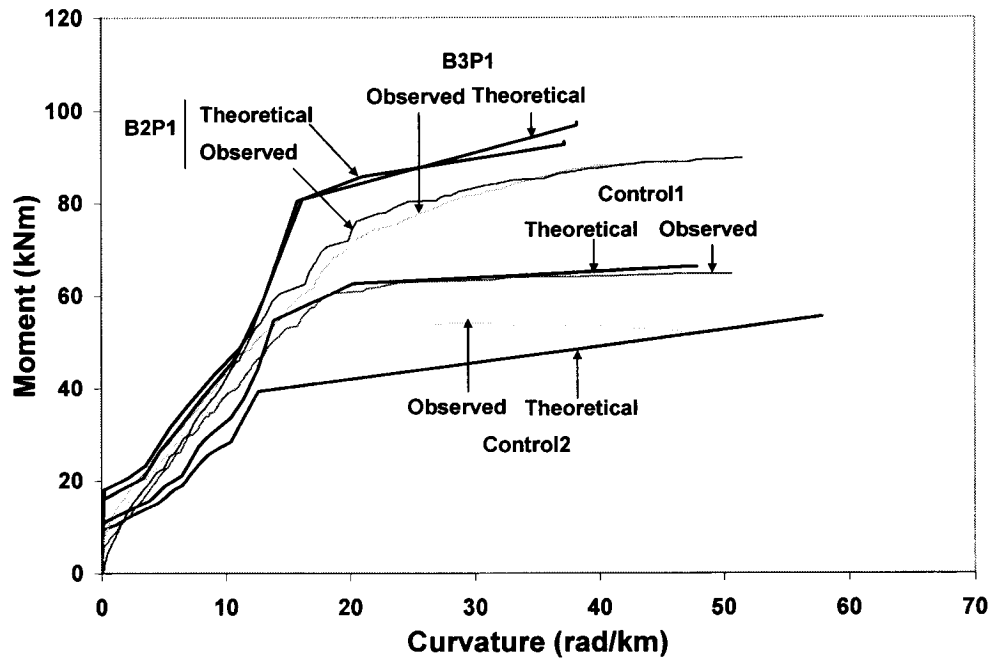


Fig. 6.14 Theoretical & observed moment-curvature curves for beam having 500 mm^2 & 600 mm^2 of flexural reinforcement and strengthened with three & two layers of GFRP sheets, respectively

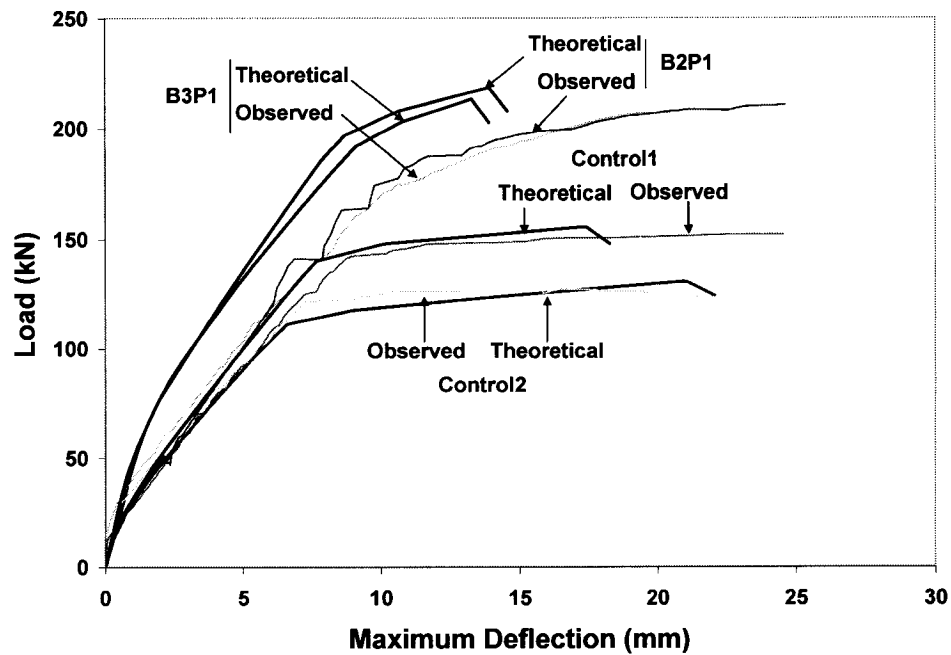


Fig. 6.15 Theoretical & observed load-displacement curves for beam having 500 mm^2 & 600 mm^2 of flexural reinforcement and strengthened with three & two layers of GFRP sheets, respectively

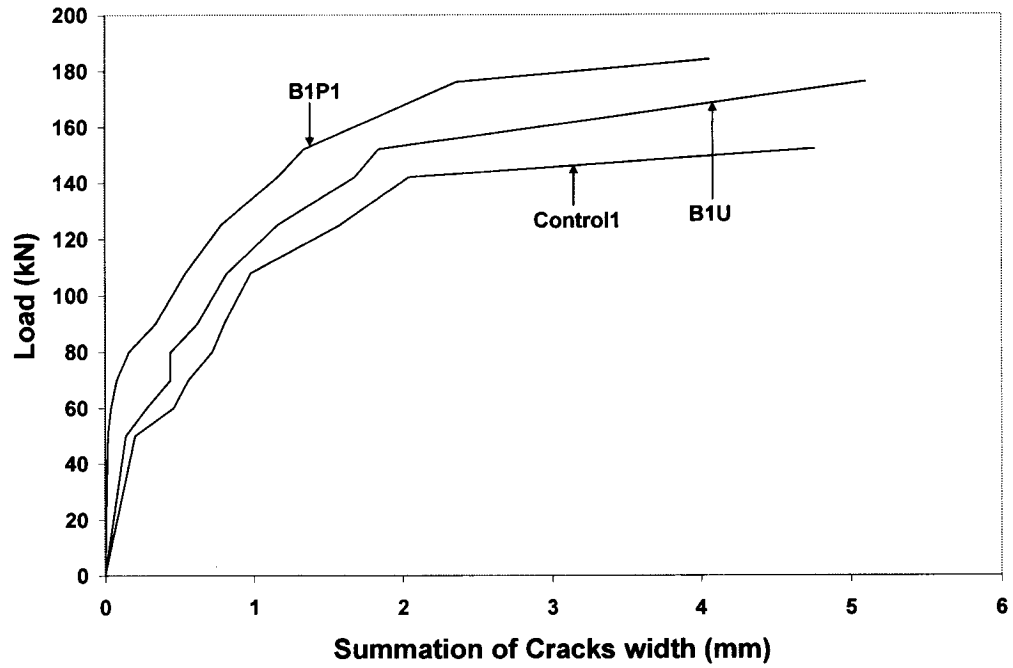


Fig. 6.16 Observed load-summations of crack widths curves for beam strengthened with one layer of GFRP sheet

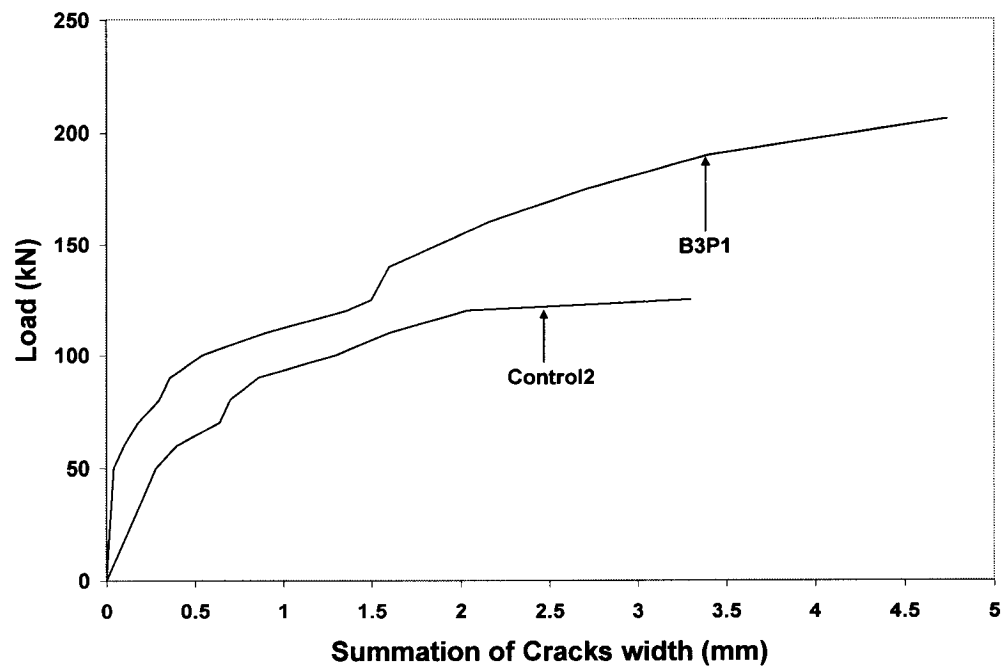


Fig. 6.17 Observed load-summations of crack widths curves for beam strengthened with three layers of GFRP sheets

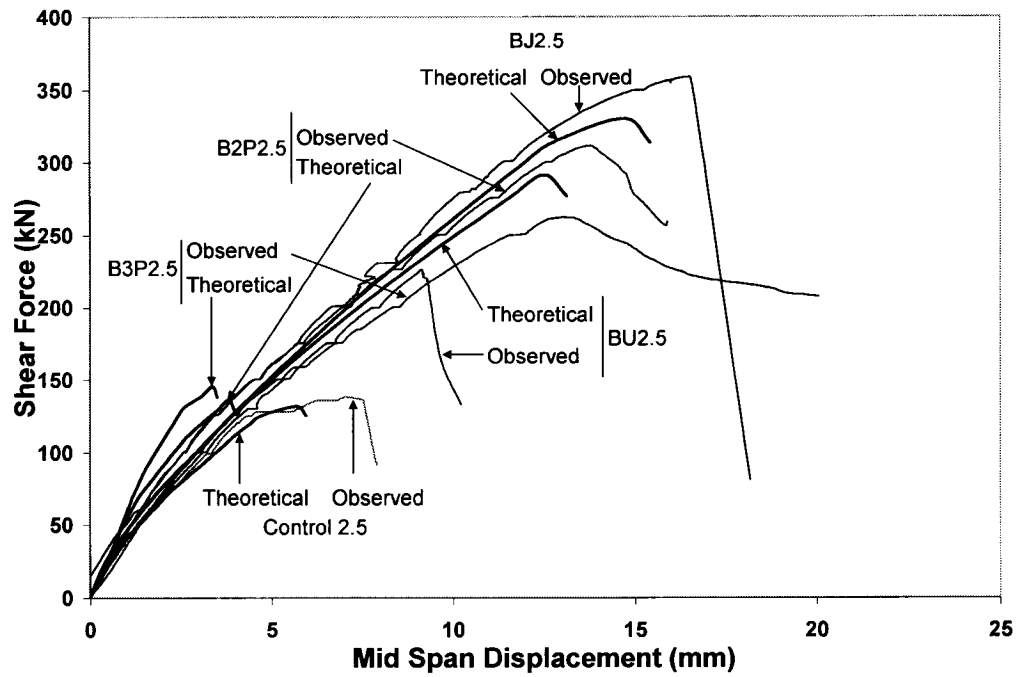


Fig. 6.18 Observed and predicted shear force – mid span displacement curves for beams tested on a/d ratio = 2.5

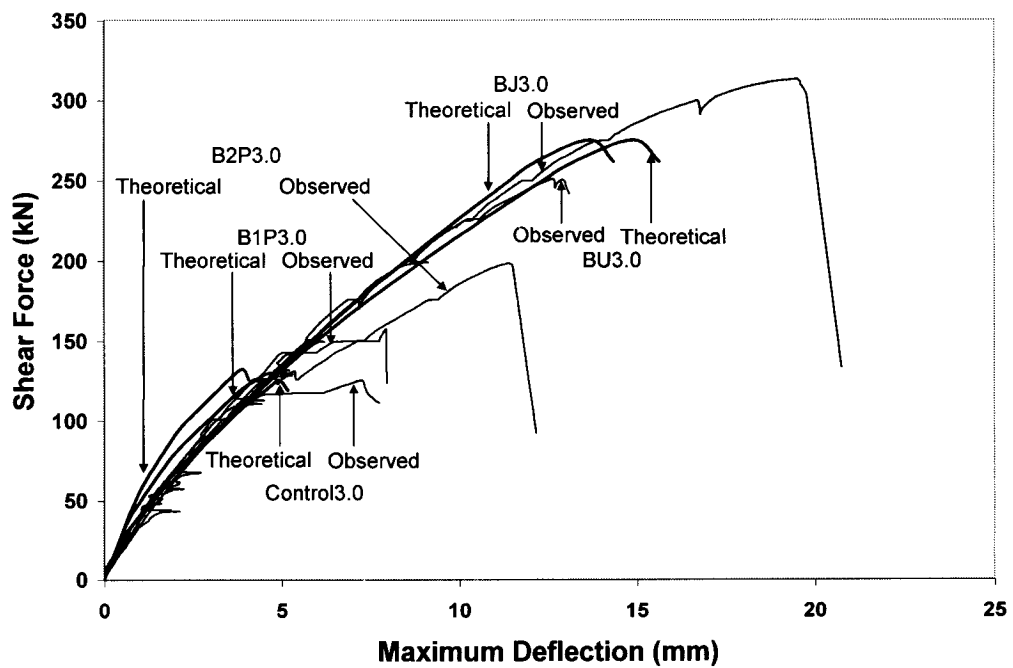


Fig. 6.19 Observed and predicted shear force – mid span displacement curves for beams tested on a/d ratio = 3.0

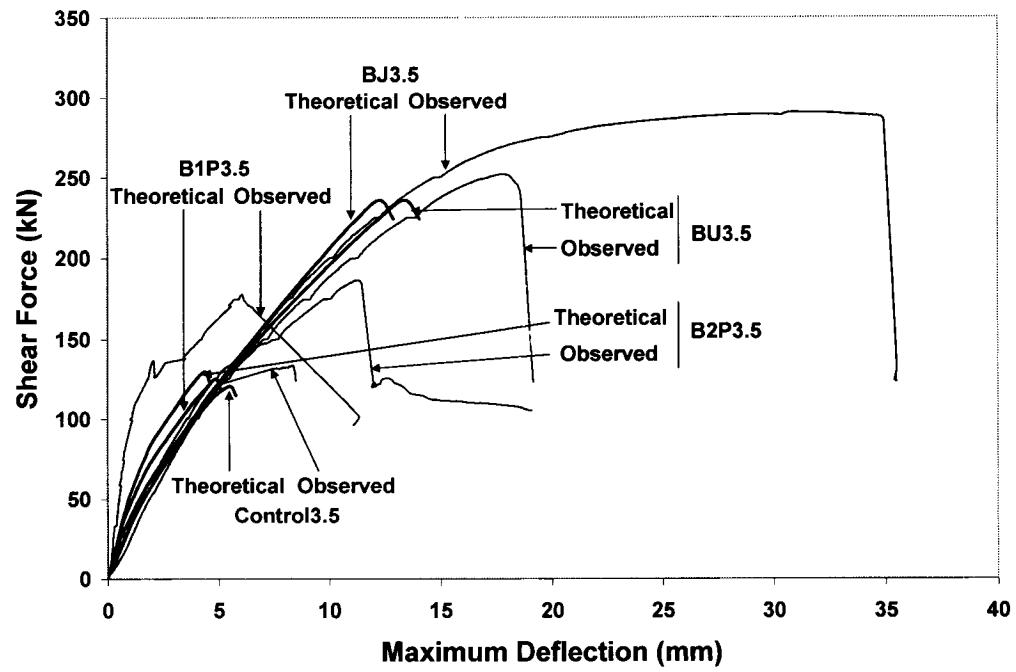


Fig. 6.20 Observed and predicted shear force – mid span displacement curves for beams tested on a/d ratio = 3.5



a) Support Failure



b) Full beam view

Fig. 6.21 Failure of beam B3P2.5 over support

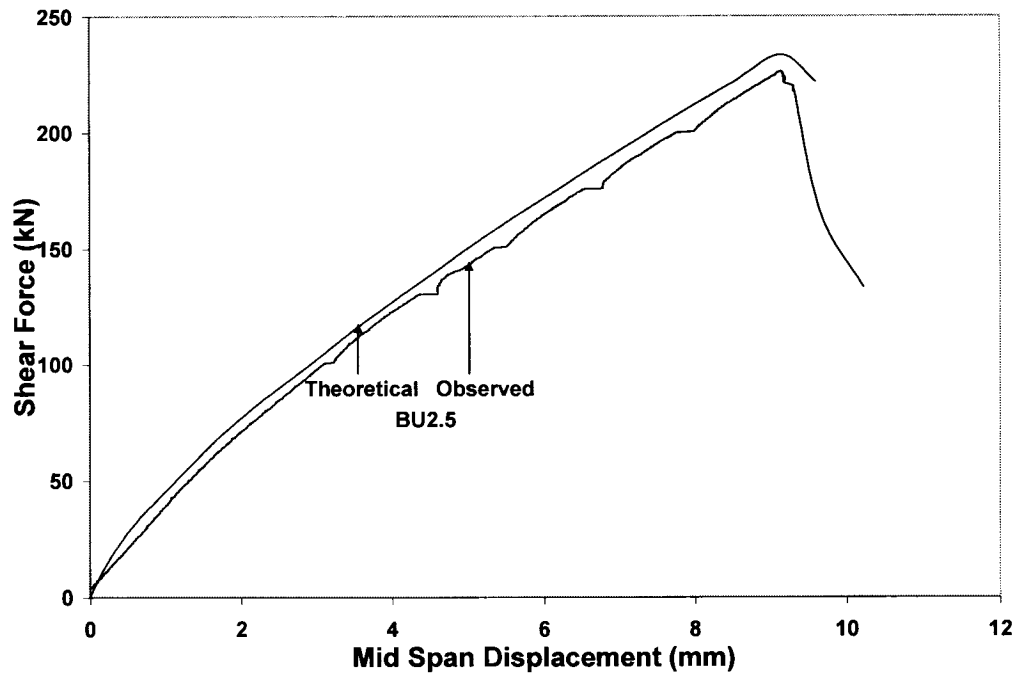


Fig. 6.22 Comparison of observed and theoretical shear force-mid span displacement curves for the beam BU2.5, where rupture strain is limited to 60% of ultimate strength

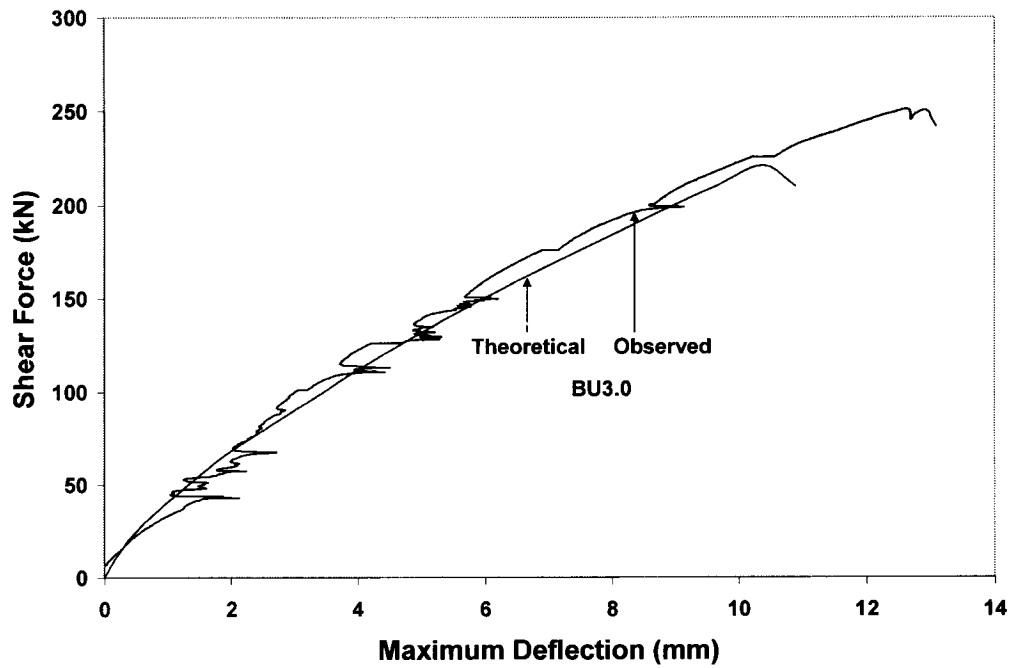


Fig. 6.23 Comparison of observed and theoretical shear force-mid span displacement curves for the beam BU3.0, where rupture strain is limited to 60% of ultimate strength

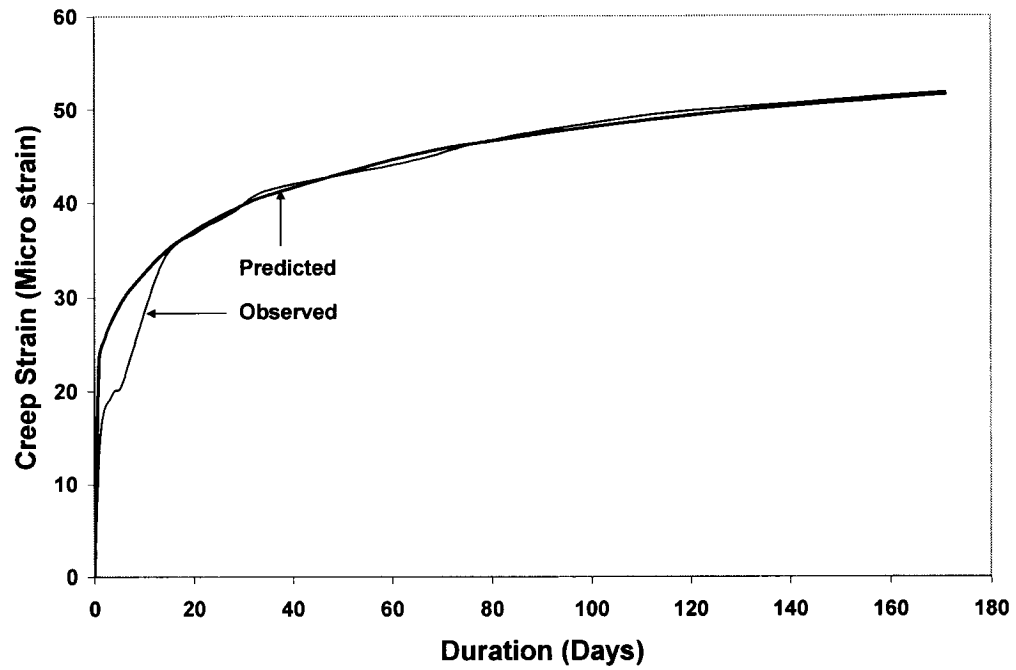


Fig. 6.24 Comparison of observed and predicted creep strains

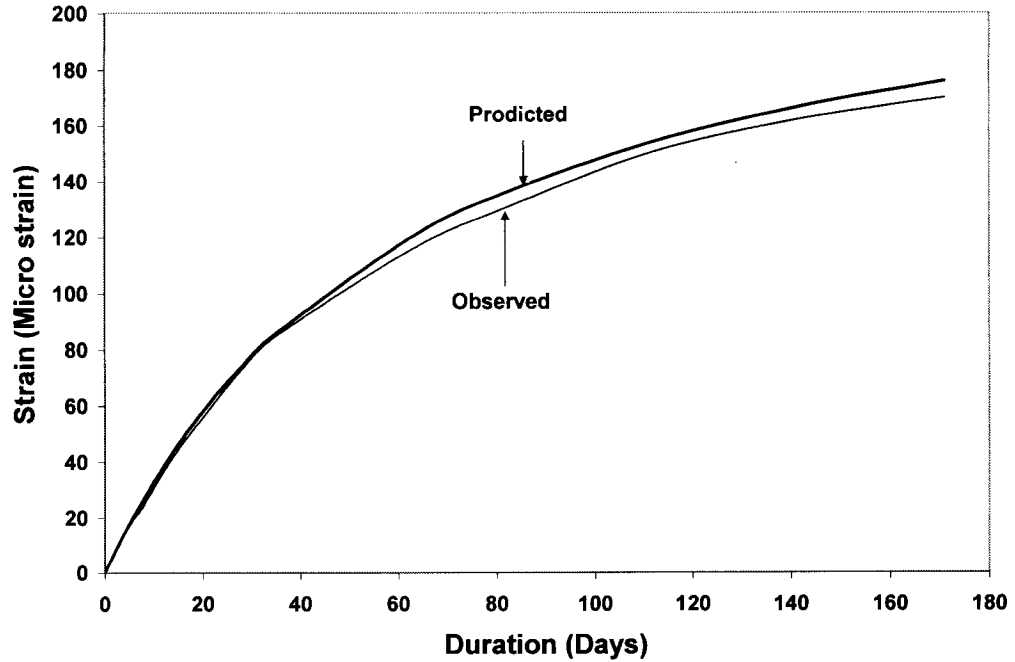


Fig. 6.25 Comparison of observed and predicted shrinkage strains

CHAPTER 7

CONCLUSIONS AND RECOMMENDATIONS

7.1 General

A lot of research work has been done on carbon fibre reinforced polymer (CFRP) in the last decade but it could not be commercialized due to high material cost. GFRP being a cheaper material was disregarded by researchers due to its relatively low modulus of elasticity. The aim of this experimental study was to investigate a practical solution of using relatively cheap and low modulus GFRP for flexural and shear strengthening of reinforced concrete members. Active strengthening technique i.e. prestressing of GFRP sheet, was used for full utilization of its strength. A new and innovative technique of external prestressing was developed in this studied. Recommendation was framed to modify the laboratory prestressing procedure for field application. The prestressed sheets were bonded on the tension face of the concrete beams and its contribution to flexural and shear strength was studied. The results of beams strengthened with prestressed sheets were compared to beams strengthened with non-prestressed sheet and reference beams. The effect of prestressing on sheet delamination and beam behaviour under service and ultimate load was also studied. The study of long term losses associated with prestressing was also part of this study. The use of prestressed GFRP sheet technique combines the advantage of using a non-corrosive and lightweight advanced composite material with the high efficiency offered by external prestressing.

In addition to the experimental investigation, exiting analytical models were used to predict the flexural and shear behaviour of un-strengthened concrete beams, beams strengthened with prestressed sheets and beams strengthened in flexural and shear with non-prestressed sheets.

7.2 Conclusions

The following conclusions were made on the basis of the experimental tests and analytical study carried out in this thesis:

1. The prestressing process developed in this study was easy, effective and practical to apply. It can be used for field application with slight modification.
2. The long term losses due to creep and shrinkage of concrete and shear strain of epoxy etc. was only 4% of initial prestress of GFRP sheets. This is a very low loss as compared to other prestressing techniques available in the literature. The relatively low modulus of GFRP sheet helped in keeping the long term loss very low as compared to conventional prestressing.
3. The largest component of long term losses was attributed to concrete shrinkage. The increase in shrinkage strain decreases with age of concrete. It is believed that the actual prestress losses will be lot less when strengthening of old concrete is carried out.
4. Prestressed GFRP sheets were successful in improving the serviceability of the strengthened beams by controlling crack widths and limiting deflections in the beams. Prestressed GFRP sheets have effectively increased the initial cracking load and reduced the crack widths at any load. Non-prestressed sheets were less effective in crack control and increasing the initial cracking load.
5. Prestressed GFRP sheets are slightly more effective at strengthening than unstressed sheets because prestressing prevents premature peeling failure.

6. Non-prestressed sheets have little effect on yielding load of strengthened beams. Prestressed GFRP sheet dramatically decreased the strain in the tension reinforcement of the beam and hence increased the yielding load.
7. Durability of the strengthened beam is likely to be improved by use of prestressed GFRP sheet as it delay crack initiation and effectively control crack width. Crack width control may help in minimizing the ingress of de-icing salt and water and hence will help in improving durability of the structure.
8. An increase in the ultimate strength of the strengthened concrete beams was observed. It is noted that prestressed GFRP sheet was more effective in increasing the ultimate strength of concrete member than non-prestressed GFRP sheet. The reason was delaying delamination of the sheet.
9. Higher strains were observed in prestressed GFRP sheet than non-prestressed GFRP sheets before delamination. This means that prestressing of GFRP sheet delayed the delamination of sheet and helped in maximum utilization of material strength.
10. Predictions of theoretical model for flexural beams were in good agreement with the experimental results. The differences in ultimate load prediction were approximately 1 ~ 5 %, and were thus within the range of experimental error. Experimental errors were associated with constituent material properties, testing setup, and errors in electronic measuring devices etc.
11. Bonding of prestressed GFRP sheets to the tension face of the beam have increased the ultimate shear capacity of strengthened beam. The increase was more pronounced in beams with smaller a/d ratio. This is due to the presence of small clamping stresses due to load points and prestressed GFRP sheet.

With increase in a/d ratio, the effect of clamping stresses from load points decreases and hence the increase in ultimate shear strength was decreased. An increase of 125%, 58% and 40% in ultimate shear capacity was observed in beam strengthened with two layers of GFRP sheets which were tested with an a/d ratio of 2.5, 3.0 and 3.5, respectively.

12. Theoretical model predicted ultimate shear strength less than experimental results for beams strengthened with prestressed GFRP sheets bonded to its tension face. The difference in ultimate shear strength was due to disregard of the beneficial effect of the small clamping stresses. These clamping stresses were induced in the strengthened beams by load points and the prestressed sheet.
13. The model predicted higher shear strength for beam strengthened with U-straps of GFRP sheets than the experimental results. The reason was premature delamination of GFRP sheet. The model assumes full bond between concrete and sheet till failure. In case of U-straps, the area of external shear reinforcement provided by GFRP U-straps was less and hence higher strains were induced in it. This led to premature delamination.
14. Strain in GFRP U-straps was a limiting condition for increase in shear capacity of strengthened beams. Limiting the strain in GFRP U-straps to 60% of its ultimate rupture strain in the analytical model helped in the accurate prediction of shear capacity of strengthened beams.
15. The use of full jacket of GFRP sheet as external shear reinforcement increased tremendously the shear capacity of strengthened beam. The failure behaviour of strengthened beams changed from shear to flexural. Theoretical model predictions were on the conservative side compared to observed results. The

reason was that behaviour of strengthened beam was changed from shear to flexural.

16. The area of GFRP sheet in beam strengthened with full jacket of GFRP sheet was almost doubled than beam strengthened with U-straps. The strain induced in the external reinforcement was less and hence no premature delamination was observed.
17. The mass loss of 16% i.e. 100 mm^2 in conventional flexural reinforcement was simulated in two groups of flexural beams. The use of one extra layer of GFRP sheet i.e. 150 mm^2 in the beam with 16% less reinforcement has effectively restored the strength and stiffness of the beam.
18. The experimental results of flexural tests done on beam strengthened with two layers and beams strengthened with three layers but less steel reinforcement are in well agreement. The use of an extra layer of GFRP sheet has effectively compensated the loss of steel reinforcement.

7.3 Recommendations

The present study was the first to work on prestressed GFRP sheets. Although the GFRP sheet was effectively prestressed and used for flexural and shear strengthening of reinforced concrete beams in the laboratory, it still needs more performance data and further research for field application. The present investigation was limited to some important parameters that affect the behaviour of concrete members strengthened with externally bonded prestressed GFRP sheets and long term losses associated with prestressing of the GFRP sheet. Further research is needed to in this area. This will provide a comprehensive understanding of the behaviour of structural concrete members under this type of strengthening technique and durability of GFRP

sheet under severe environmental conditions. Some of the investigations needed in this area are recommended below:

1. Premature delamination of FRP sheet is the main problem with this type of strengthening technique. It was observed in the present study that delamination was delayed in beams strengthened with prestressed GFRP sheets. To establish a clear and accurate relation between levels of prestressing and delamination of FRP sheets, more tests are needed. This will provide a clear picture about the effect of prestressing on premature delamination.
2. Although in this strengthening technique, GFRP sheet was used as external reinforcement on old concrete, which is less susceptible to alkaline attack, this parameter still needs attention. We know that GFRP deteriorates in an alkaline environment, therefore, a durability study of GFRP under an alkaline environment is needed. Presence of an alkaline solution in old concrete needs to be studied first, and then an experimental program should be devised to simulate the actual field condition in the laboratory.
3. Out of ten beams strengthened with prestressed GFRP sheets, one beam strengthened with three layers of sheets showed end delamination at transfer of prestressing force. This sort of failure can limit the maximum number of prestressed sheets for strengthening of beams. More tests are needed to study this behaviour. Staggering of sheet termination points should be studied as it will help in reducing the concentration of shear stresses in the sheet ends.

4. Tests need to be carried out to study the enhancement of serviceability and ultimate strength under fatigue and cyclic loading of concrete members strengthened with prestressed GFRP sheets.
5. The long-term performance of the bond at the interface of the concrete and FRP sheets, including the effect of fatigue and unfavourable environmental conditions, needs to be investigated.
6. The effects of harsh environmental conditions, such as wet/dry cycles using salt water, on the performance of FRP bonded sheets to concrete beams and on the interface bond between the sheets and the concrete need to be investigated in order to establish the durability of the system.
7. The prestressing system recommended for field application needs to be used in field experimentation for investigation of system feasibility.
8. Fire resistance is another problem associated with external prestressing systems. The fire resistance of the system needs investigation.
9. It is observed in this experimental program that improvement in shear capacity by prestressed GFRP sheet bonded to the tension face of the beam was dramatically reduced with increase in a/d ratio. More tests are needed on such beams with varying a/d ratios.
10. As observed that transverse clamping stress in the shear span helped in increasing the shear capacity of the beam, it is recommended that prestressed GFRP sheet attached to vertical faces of the beam should be tested in laboratory for investigating the shear capacity increase.

Experimental work is currently in progress in Hong Kong University of Science and Technology (HKUST), Hong Kong on the effect of a/d ratio on shear strengthening of beam with prestressed GFRP sheet.

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