"EFFECT OF INITIAL STRESS LEVELS ON STRENGTH OF BEAMS RETROFITTED USING PRE-STRESSED FIBRE COMPOSITES"

A thesis report submitted in the partial fulfillment

of the requirement for the award of the degree

of

MASTERS OF ENGINEERING

IN

CIVIL (STRUCTURES)

Submitted by:-

Ankur Mehta

Roll No.800922001

Under the supervision of

Dr. Prem Pal Bansal

Assistant Prof., CED

Thapar University
Patiala

DEPARTMENT OF CIVIL ENGINEERING

THAPAR UNIVERSITY, PATIALA-147004, INDIA

JULY-2011

CERTIFICATE

This is to certify that thesis entitled **"Effect of Different Wrapping Techniques on Retrofitting of RCC Beam Column Joints Using Ferrocement"**, being submitted by Mr. Manzoor Ahmad Dar (Roll No. 800922006), in partial fulfillment for award of the degree of Master of Engineering in Civil Engineering (Structures) at Thapar University, Patiala is a bonafide work carried out by him under our guidance and supervision and that no part of this thesis has been submitted for the award of any other degree.

(Dr. Maneek Kumar)
Professor & Head
Department of Civil Engineering
TU Patiala (PUNJAB)

(Dr. Prem Pal Bansal)
Assistant Professor
Department of Civil Engineering
TU Patiala (PUNJAB).

Countersigned by:

(Chairman, Board of Studies)
Department of Civil Engineering
TU Patiala (PUNJAB)

(Dr. S.K. Mohapatra)
Dean Academic Affairs
TU Patiala (PUNJAB)

i

ACKNOWLEDGEMENT

A dissertation cannot be completed without the help of many people who contribute directly or indirectly through there constructive criticism in the evolution and preparation of this work. It would not be fair on my part, if I don't say a word of thanks to all those whose sincere advice made this period a real educative, enlightening, pleasurable and memorable one.

First of all, a special debt of gratitude is owned to my supervisor **Dr. Prem Pal Bansal** for his gracious efforts and keen pursuits, which has remained as a valuable asset for the successful completion of research work.

I also like to offer my sincere thanks to all faculty members, teaching and non-teaching staff of Civil Engineering Department (CED), and staff of central library, TU, Patiala for their assistance. I am extremely thankful to my friends, Mr. Sumeet Sharma, Mr. Parveen Jangra and all other manpower for helping me carry out experimental work.

ANKUR MEHTA

M.E Civil (Structures)

Roll No. 800922001

ABSTRACT

Many existing structures now-a-days are unable to give their service effectively. Due to various reasons, they get deteriorated or get damaged much before the time for which they are designed for. The evaluation of the damage can be measured by various non-destrctive tests, pulse velocity tests etc. In view of this, the word comes “Retrofitting” which simply means to repair the structures or to increase the strength of the structures to increase their service life. Various structures like bridges, dams, for which rehabilitation is very difficult, in such cases retrofitting can be used as an effective measure. There are various methods for this like jacketing, shear wall, infill wall etc., apart from these conventional methods Fibre composites can be used very effectively. A lot of research is being carried on all across the globe for the use of Fibre composites for retrofitting.

On the lights of the above discussion, a research is carried on to study the effect of initial stress levels on retrofitting of beams. In other words the effect of the extent of damage in the beams is studied on the retrofitting using pre-stressed carbon fibre sheets. For this the beams of size 600mm×300mm are used which are stressed to three different initial stress levels taken respectively from elastic zone, elasto-plastic zone and plastic zone from the load-deflection plot of the beam taken as control specimen. These initially stressed beams are then retrofitted using pre-stressed fibre composite mechanism. The effect of these initial stress levels on retrofitting are then studied.

Through this study it is observed that load carrying capacity of the retrofitted beams is remarkably improved compared to that of fresh control beam. The control specimen is failed at 94.35kN and the retrofitted beams are seen to be failed in the range of 125kN to 136kN. Subsequent decrease in the deflection is also seen as compared to that of the fresh control beam at all the stress levels. The 53% stressed beam showed the maximum deflection whereas the 100% stressed beam showed the minimum deflection with overall decrease of around 65% of that of control beam. Stiffness has also increased after retrofitting at all stressed levels as compared to the control beam. The 53% and 89% stressed beams showed almost same stiffness but the 98% stressed beam showed the maximum stiffness with increase of around 25% of that of control specimen.

LIST OF FIGURES

LIST OF TABLES

TABLE OF CONTENTS

\

CHAPTER 1 INTRODUCTION

1.1 General

Concrete construction is generally expected to give trouble free service throughout its intended design life. However, these expectations are not realised in many constructions because of structural deficiency, material deterioration, unanticipated over loadings or physical damage. Premature material deterioration can arise from a number of causes, the most common being when the construction specifications are violated or when the facility is exposed to harsher service environment than those expected during the planning and design stages. Physical damage can also arise from fire explosion-as well as from restraints, both external and internal, against structural movement. Except in extreme cases, most of the structures require restoration to meet its functional requirements by appropriate repair techniques.

1.2 Historical Background

Since 1950s, the construction activity in India has been increasing geometrically without matching increase in the availability of quality inputs, in terms of materials and skilled workmen. The gap between the quality planned and the quality achieved continues to become wider, the factor contributing to damages/distresses in buildings have, thus, become intrinsic right from the construction stage. Often these are concealed under external renderings and the defect takes time to manifest itself. Construction documents contain adequate specifications and instructions required to execute quality works. However, they remain as written document without achieving the desired level of results, because of lack of understanding of their significance by the field engineers. Standard cube size results are taken as a measure of quality in the construction. Whereas the factor such as method of placing, compaction and curing of concrete, which have significant influence on the quality achieved in the hardened concrete, are given scant attention. Many a times, the quality of concrete as placed and hardened in position has no co-relation to the cube test results, which are used for quality control measures. Procedures, mandatory or otherwise, for periodic inspection of buildings and structures and documenting defects, like cracks, excessive deflections, corrosion of reinforcements, etc., in logical manner and recording of structural repairs already carried out, are generally not followed or maintained. In some buildings, only visual inspections is carried out for preparing maintenance budget estimates and this exercise is often led to the engineers who have no experience in such problems. Concrete constructions require proper care in form of regular maintenance. Buildings remain for several years without getting due attentions. Water stagnations, paint peeling, plaster break-off, fungus growth, cracking of external rendering and cover concrete are common and wide spread. Penetration of moisture into RC concrete components promotes corrosion process and further damages the concrete cover.

FRP composites have been used on a limited basis in structural engineering for almost 50 years for both new construction and for repair rehabilitation of existing structures. Structural and civil engineers have been affixing their professional stamps to design for building and bridges for many years, even though these materials have been not recognised by official building codes, and no code-approved design procedures has existed until very recently These forward-thinking engineers have tended to be people who has specialise expertise in the use of FRP composite in structural engineering or who were in-house structural engineers directly affiliated with a manufacturer of FRP components. In addition to being registered structural engineers involved in engineering practices, many of these engineers have had fundamental knowledge of materials, manufacturing methods, and fabrication and installation methods for FRP composite for civil engineering structures. Many have worked on teams to develop new FRP components for civil engineering structures. Since the mid 1990's, however, other structural engineers and architects have begin to design with FRP composites on fairly routine basis. In general these engineers and architects have not had specialised training or exposure to FRP composites as construction materials. These designs are completed with the aid of published design procedures or by proof testing, and often, with the aid of an in-house engineer from an FRP product manufacturer, who advises on the details of design and provides sample specification for FRP material for contract documents. Many structural engineers of this type have also affixed their professional seals to these designs over the past decade.

The historical review provides selected examples of where FRP composites have been used in buildings and bridges in the past half century. More attention is paid to the applications from the 1990's, which were designed in the routine fashion by structural engineers, as opposed to those from before 1990's, which were generally designed by engineers with specialised knowledge of composites. The review describes application s that used FRP components and products available at the time of their manufacture. Many FRP products used in these projects are no longer produced or have been replaced by improved parts or products.

The state of the art of the early work, from 1980 to 1990 in the area of FRP composites for reinforcement of concrete structures in the US, Japan and Europe is detailed in the collection papers and reports edited by Iyer, Sen and Nanni. In 1993, a series of biannual international symposis devoted to FRP reinforcement of concrete structures was initiated .Art about this time, international research interest in use of FRP in concrete increased dramatically. Collections of papers on the use of FRP profile section in structures have been published by the American society of engineers (ASCE) since the early 1890's by the now disbanded Structural Plastics Research Council (SPRC) and in proceedings of the ASCE materials congresses In 1997, American Society of civil engineers founded the Journal of Composites for Construction and development in the field of FRP materials for AEC industry. In 2003 the International institute of FRP construction was established in Honk Kong. To date, thousands of research studies and structural engineering projects using FRP material have worldwide.

1.3 Need for Retrofitting

The need for retrofitting refers to the damaged structures that are being damaged due to various causes. So it is very important to know how a structure, which is to be retrofitted, has damaged. Concrete is a composite material that consists essentially of a binding medium, within which, are embedded particles or fragments of aggregates. However in cement concrete, it is relevant to RCC structures, the binding medium is the mixture of hydraulic cement and water. All concrete in service will be subject to chemical and physical changes. A durable concrete is one in which these changes occur at a rate, which does not detrimentally affect its performance within its intended life. Leave it to concrete alone, the material remains by and large durable, but concrete alone cannot be utilised extensively for structural applications. It is the RCC, a composite structural material which is utilised for variety of structural uses. But, it has been observed that RCC has not proved to be durable due to large number of factors, including variations in production, loading conditions in service life and subsequent attack by the environmental factors.

1.3.1 Signs of deterioration of concrete:

The various symptoms of deterioration of concrete are:

- Cracking occurs over time in virtually all concrete. Cracks vary in depth, width, direction, pattern, location, and cause. Cracks can be either active or dormant (inactive). Active cracks widen, deepen, or migrate through the concrete. Dormant cracks remain unchanged. Some dormant cracks, such as those caused by shrinkage during the curing process, pose no danger, but if left unrepaired, they can provide convenient channels for moisture penetration, which normally causes further damage.
- Structural cracks can result from temporary or continued overloads, uneven foundation settling, or original design inadequacies. Structural cracks are active if the overload is continued or if settlement is ongoing; they are dormant if the temporary overloads have been removed, or if differential settlement has stabilized. Thermally induced cracks result from stresses produced by temperature changes. They frequently occur at the ends or corners of older concrete structures built without expansion joints capable of relieving such stresses. Random surface cracks (also called "map" cracks due to their resemblance to the lines on a road map) that deepen over time and exude a white gel that hardens on the surface are caused by an adverse reaction between the alkalis in cement and some aggregates. Since superficial repairs that do not eliminate underlying causes will only tend to aggravate problems, professional consultation is recommended in almost every instance where noticeable cracking occurs.
- Spalling is the loss of surface material in patches of varying size. It occurs when reinforcing bars corrode, thus creating high stresses within the concrete. As a result, chunks of concrete pop off from the surface. Similar damage can occur when water absorbed by porous aggregates freezes. Vapour proof paints or sealants, which trap moisture beneath the surface of the impermeable barrier, also can cause spalling. Spalling may also result from the improper consolidation of concrete during construction. In this case, water rich cement paste rises to the surface (a condition known as laitance). The surface weakness encourages scaling, which is spalling in thin layers.
- Deflection is the bending or sagging of concrete beams, columns, joists, or slabs, and can seriously affect both the strength and structural soundness of concrete. It can be produced by overloading, by corrosion, by inadequate construction techniques (use of low-strength concrete or undersized reinforcing bars, for example), or by concrete creep (long term shrinkage). Corrosion may cause deflection by weakening and ultimately destroying the bond between the rebar and the concrete, and finally by destroying the reinforcing bars them. Deflection of this type is preceded by significant

cracking at the bottom of the beams or at column supports. Deflection in a structure without widespread cracking, spalling, or corrosion is frequently due to concrete creep.

- Stains can be produced by alkali aggregate reaction, which forms a white gel exuding through cracks and hardening as a white stain on the surface. Efflorescence is a white, powdery stain produced by the leaching of lime from Portland cement, or by the pre-World War II practice of adding lime to whiten the concrete. Discoloration can also result from metals inserted into the concrete, or from corrosion products dripping onto the surface.
- Erosion is the weathering of the concrete surface by wind, rain, snow, and salt air or spray. Erosion can also be caused by the mechanical action of water channelled over concrete, by the lack of drip grooves in belt courses and sills, and by inadequate drainage.
- Corrosion, the rusting of reinforcing bars in concrete, can be a most serious problem. Normally, embedded reinforcing bars are protected against corrosion by being buried within the mass of the concrete and by the high alkalinity of the concrete itself. This protection, however, can be destroyed in two ways. First, by carbonation, which occurs when carbon dioxide in the air reacts chemically with cement paste at the surface and reduces the alkalinity of the concrete, second, chloride ions from salts combine with moisture to produce an electrolyte that effectively corrodes the reinforcing bars. Chlorides may come from seawater additives in the original mix, or from prolonged contact with salt spray or deicing salts. Regardless of the cause, corrosion of reinforcing bars produces rust, which occupies significantly more space than the original metal, and causes expansive forces within the concrete. Cracking and spalling are frequent results. In addition, the load-carrying capacity of the structure can be diminished by the loss of concrete, by the loss of bond between reinforcing bars and concrete, and by the decrease in thickness of the reinforcing bars themselves. Rust stains on the surface of the concrete are an indication that internal corrosion is taking place.

1.3.2 Causes of deterioration of concrete:

It is imperative to establish the cause of the concrete degradation or steel reinforcement corrosion before repairs are carried out. Poor understanding of the cause of the problem can result in inappropriate repairs. Concrete is a relatively durable and robust building material, but it can be severely weakened by poor manufacture or a very aggressive environment. Concrete degradation can be a cause for concern on its own, or in reinforced structures it may lead to decreased protection to the steel. This in turn encourages corrosion of the steel, often followed by cracking and spalling of the concrete.

Deterioration of concrete is due to either:

- chemical degradation of the cementitious matrix,
- corrosion of the reinforcement steel,

- physical damage (impact, abrasive and fire damage).

The most important causes of concrete deterioration are described below:

1.3.2.1 Design problems

There are a number of design and specification problems that can result in reinforced-concrete deterioration.

Causes of the problem	Results
Poor reinforcement details, for example congested or inadequate reinforcement, inadequate cover to reinforcement	Leads to cracking, poor compaction, loss of alkaline environment, and voids around the steel
Poor detailing of fixings, window frames, handrails, supports, and expansion joints	Water penetration, localised cracking, and balcony weakness
Long, slender components	Excessive flexing may lead to cracking
Inadequate design for creep	Deflection due to strain under continued stress that can result in cracking
Decorative finishes, such as acid etching, bush hammering, and fluting	Results in varying depth of cover around the steel and localised corrosion
Poor drainage	Leads to water ponding and localised corrosion/degradation
Incorrect concrete grade for purpose	Can produce concrete that is too weak/too strong
Mixes that result in high drying shrinkage	Can result in cracking
Mixes that are permeable to chloride ions	Chloride induced reinforcement corrosion

Table 1.1: Deterioration of concrete due to Design problems

1.3.2.2 Workmanship

Care and attention during construction is crucial to the long-term durability of reinforced concrete. Current building codes, including AS 3600, give guidance appropriate to the exposure conditions, and on mix design (particularly the water/cement ratio and amount of cement binder), additives, compaction, detail, and thickness of cover. The type of cement binder is also now well defined. However, this has not always been the case and durability problems of earlier concrete are summarised in the following table:

Causes of the Problem	Result
Poor mixing	Leads to inhomogeneous concrete, localised weakness, and reinforcement corrosion
Incorrect water/cement ratios	Can lead to variable strength, inadequate durability, increased drying shrinkage, excessive permeability
Poor compaction/vibration	Results in honeycombing, voids, excessively permeable concrete, localised reinforcement corrosion
Varying and inadequate cover depths around the steel	Leads to localised reinforcement corrosion, penetration of damaging substances
Poor curing techniques	Results in shrinkage cracks, increased permeability, poor durability
Premature stripping of formwork	Can result in cracking

Table 1.2: Deterioration of concrete due to workmanship problems

1.3.2.3 Materials

Lack of knowledge about the importance of careful selection and specification of materials and the use of additives has created a number of durability problems for historic concrete structures. They can be the result of the use of the following:

Causes of the Problem	Result
Too low cement content	Results in weakened and poor durability concrete
Too high cement content	Can result in excessive shrinkage/poor workability and cracking
Additives, such as calcium chloride	Chloride ions destroy the protective passive-oxide layer on the steel
High alumina cement	Weakening concrete in wet environments
Too finely-ground cements	Causing excessive shrinkage and cracking that compromises the permeability of the concrete
Poor quality aggregates	Resulting in alkali-aggregate reaction, poor workability of the concrete, poor compaction, high drying shrinkage and weak concrete
Contaminated aggregates	Resulting in corrosion of steel and degradation of concrete in extreme cases
Poorly shaped and badly graded aggregates	Results in poor workability often necessitating extra water or vibration during forming which can lead to segregation, bleeding etc
Incorrect water/cement ratios	Giving rise to weak concrete, loss of durability, increased permeability to gases and chloride ions

Table 1.3: Deterioration of concrete due to material problems

1.3.2.4 Environmental influences

These play an important part in reinforced-concrete deterioration and include the following:

Environmental condition	Result
Carbon dioxide and acidic gases	Lower pH around the steel that enables corrosion to progress
Water	Can introduce depassivating chloride ions into concrete
Freeze thaw in colder zones	Breakdown of surface, progressive cracking, water penetration to reinforcement that enables corrosion to progress
Salt ingress	Marine salt introduces depassivating chloride ions into the surface of the concrete
Chemical attack	Chemical attack by chlorides can cause corrosion of steel or sulphates that can cause degradation of the cementitious matrix
Vibration	Causes cracking, spalling, and delamination
Impact damage	Causes physical weakening of structural components, exposure of steel reinforcement, cracking etc

Table 1.4: Deterioration of concrete due to environmental problems

1.3.2.5 Physical damage

Concrete structures can also be affected by physical damage, such as:

• excessive loads

• impact damage

• abrasion

• fire damage.

1.3.2.6 Lack of maintenance and poor repair

When concrete first began to be used as a major structural building material, it was promoted as a material that needed little or no maintenance. Maintenance is an attempt to avoid failures, unnecessary production loss and safety violations. To increase the service life of a structure, it should be ensured that proper maintenance is there at regular time. However lack of maintenance is a major contributor to reinforced-concrete deterioration. The problem related to improper maintenance and repair is explained in Table 1.5.

Sources of deterioration	Materials	Design and workmanship	Environmental	Physical damage
Potential causes of deterioration	Cement type/quality Mix design	Poor detailing	Presence of CO_2 and acid gases	Inadequate design
	Poor aggregate selection / reactivity	Insufficient cover to reinforcement	Freeze thaw	Impact
	Incorrect water:cement ratio	Poor drainage	Salt and chemical attack (de-icing salts, ground water)	Vibration
	Mechanical strength of aggregate	Inadequate design for creep	Biological growth	Settlement
	Additives or contaminants	Poor mixing	Weathering	Seismic
		Poor vibration and compaction	Thermal movement	Change of use increased floor loadings
		Bleeding and segregation	Fire damage	Wind
		Poor construction joints	Inadequate maintenance	
		Problematic finishes		
		Poor repairs		
		Inadequate maintenance		

Table 1.5: Deterioration of concrete due to lack of maintenance.

1.3.2.7 Corrosion

The major cause of deterioration in reinforced concrete structures is corrosion of the reinforcing steel. Corrosion requires air and water to take place. Corrosion of steel in concrete is an electrochemical reaction in which the major constituent of steel (iron) goes into solution as iron ions with a flow of electrons (electrical flow). The most common causes of steel corrosion are the loss of alkalinity around the steel as a result of carbonation and the presence of chlorides at the steel surface. Although both processes produce different forms of corrosion, carbonation and the presence of chlorides result in the loss of the protective passive layer around the steel and the commencement of the electrochemical reaction explained above. Both are also directly related to the durability of the concrete and its ability to prevent acidic gases destroying the alkalinity of the concrete, or to prevent chloride ions from migrating to the steel. Harmful corrosion can occur in the presence of dissimilar metals that are linked by an electrolyte in the concrete, for example when zinc-coated steel is linked to uncoated steel or where there are copper pipes or aluminium fittings in the concrete.

- *Carbonation* occurs when atmospheric carbon dioxide reacts with the pore water in the concrete creating carbonic acid, which reacts with the lime in solution ($Ca(OH)_2$) reducing the alkalinity of the concrete from 12.5 to less than 9.0. Carbonation is not harmful to concrete but when the carbonation front reaches the level of the steel reinforcement, corrosion will commence.
- *Chlorides* may have been added to the concrete at the time of construction as a calcium chloride accelerator (used in cold weather, or to speed up turnaround of moulds and formwork), or cast in from poorly-washed marine aggregates, or may have ingressed from sea water, wind-blown marine spray or saline groundwater. Chlorides attack the passive oxide layer of the steel but do not affect the alkalinity of the concrete.

Most reinforced concrete deterioration is related to reinforcement corrosion either directly or indirectly as the result of other degradation. Reinforcement corrosion will usually result in the development of expansive rust products (hydrated iron oxides) that put the concrete into tension around the steel. This leads to cracking, and eventually spalling of the concrete. Degradation of the concrete matrix can result in loss of protection to the reinforcement steel and corrosion. The two mechanisms – chemical degradation of the cementitious matrix and corrosion of the steel reinforcement – may be initiated independently but become inter-related. Cement paste is usually highly alkaline, containing mobile hydroxyl ions within the pore water that is always present. These hydroxyl ions $(OH)^-$ in the pore water coat the surface of the embedded steel reinforcement producing a strong passive-oxide layer. It is when this oxide layer is broken down, by the ingress of aggressive ions such as chlorides $(Cl)^-$, or not maintained by the high alkalinity (due to atmospheric carbon dioxide or attack by acidic gases), that corrosion can occur. The effectiveness of the passive layer in providing long-term corrosion protection to the steel is dependent on:

• The constituents of the concrete,

• The thickness and quality of the concrete forming the cover,

• The durability and permeability of the concrete forming the cover,

• The homogeneity of the concrete forming the cover,

• Environmental conditions.

1.4 Retrofitting Using Fibre Reinforced Polymers (FRP)

1.4.1 Introduction

Besides the conventional methods used for retrofitting in the world like adding shear walls. Providing additional foundations, beam jacketing, column jacketing, providing additional columns, etc., as shown in Fig, there is another method which has wide range of scope that overcomes the difficulties and drawbacks of these conventional methods. The most innovative method used for such purpose is the use of Fibre Reinforced polymers. It is basically a composite material made of a polymer matrix reinforced with fibers. The fibers are usually fiber glass, carbon, or aramid, while the polymer is usually an epoxy, vinyl ester or polyester thermosetting plastic. FRPs are commonly used in the aerospace, automotive, marine, and construction industries. Fiber reinforced plastics are a category of composite plastics that specifically use fibrous materials to mechanically enhance the strength and elasticity of plastics. The original plastic material without fiber reinforcement is known as the matrix. The matrix is a tough but relatively weak plastic that is reinforced by stronger stiffer reinforcing filaments or fibers. The extent that strength and elasticity are enhanced in a fiber reinforced plastic depends on the mechanical properties of both, the fiber and matrix, their volume relative to one another, and the fiber length and orientation within the matrix. Reinforcement of the matrix occurs by definition when the FRP material exhibits increased strength or elasticity relative to the strength and elasticity of the matrix alone.

Various researches are being carried out on it all over the world. It has wide range of scope of retrofitting of big dams, bridges, etc.. These FRP`s can be used in its two forms:

1. Fibre wraps or sheets.
2. Fibre laminates.

1.4.2 History

Global polymer production on the scale present today began in the mid 20th century, when low material and productions costs, new production technologies and new product categories combined to make polymer production economical. The industry finally matured in the late 1970s when world polymer production surpassed that of Steel, making polymers the ubiquitous material that it is today. Fibre reinforced plastics have been a significant aspect of this industry from the beginning. There are three important categories of fibre used in FRP, These are:

1. Carbon Fibre Reinforced Polymer.
2. Glass Fibre Polymer.
3. Aramid Fibre Polymer.

Glass fibre reinforcement was tested in military applications at the end of World War II, Carbon fibre production began in the late 1950s and was used, though not widely, in British industry beginning in the early 1960s, Aramid fibres were being produced around this time also, appearing first under the trade name Nomex by DuPont. Today each of these fibres is used widely in industry for any applications that require plastics with specific strength or elastic qualities. Glass fibres are the most common across all industries, although carbon fibre and carbon fibre aramid composites are widely found in aerospace, automotive and sporting good applications (Sen et al 2000).

1.4.3 Manufacturing Process

FRP involves two distinct processes, the first is the process whereby the fibrous material is manufactured and formed, and the second is the process whereby fibrous materials are bonded with the matrix during the moulding process.

1.4.3.1 Fibre Process

Reinforced Fibre is manufactured in both two dimensional and three dimensional orientations:

- Two Dimensional Fibre Reinforced Polymer: are characterized by a laminated structure in which the fibres are only aligned along the plane in x-direction and y-direction of the material. This means that no fibres are aligned in the through thickness or the z-direction, this lack of alignment in the through thickness can create a disadvantage in cost and processing. Costs and labour increase because conventional processing techniques used to fabricate composites, such as wet hand lay-up, autoclave and resin transfer moulding, require a high amount of skilled labour to cut, stack and consolidate into a preformed component.
- Three-dimensional Fibre Reinforced Polymer composites are materials with three dimensional fibre structures that incorporate fibres in the x-direction, y-direction and z-direction. The development of three-dimensional orientations arose from industry's need to reduce fabrication costs, to increase through-thickness mechanical properties, and to improve impact damage tolerance, all were problems associated with two dimensional fibre reinforced polymers.

Fibre preforms are often manufactured in sheets, continuous mats, or as continuous filaments for spray applications. The four major ways to manufacture the fibre preform is through the textile processing techniques of Weaving, knitting, braiding and stitching.

- *Weaving*

Weaving can be done in a conventional manner to produce two-dimensional fibres as well in a multilayer weaving that can create three-dimensional fibres. However, multilayer weaving is required to have multiple layers of warp yarns to create fibres in the z- direction creating a few disadvantages in manufacturing, namely the time to set up all the warp yarns on the loom. Therefore most multilayer weaving is currently used to produce relatively narrow width products or high value products where the cost of the preform production is acceptable. Another one of the main problems facing the use of multilayer woven fabrics is the difficulty in producing a fabric that contains fibres oriented with angles other than 0" and 90" to each other respectively.

- *Braiding*

Braiding is suited to the manufacture of narrow width flat or tubular fabric and is not as capable as weaving in the production of large volumes of wide fabrics. Braiding is done over top of mandrels that vary in cross-sectional shape or dimension along their length. Braiding is limited to objects about a brick in size. Unlike the standard weaving process, braiding can produce fabric that contains fibres at 45 degrees angles to one another. Braiding three-dimensional fibres can be done using four step, two-step or Multilayer Interlock Braiding. Four step or row and column braiding utilizes a flat bed containing rows and columns of yarn carriers that form the shape of the desired preform. Additional carriers are added to the outside of the array, the precise location and quantity of which depends upon the exact preform shape and structure required. There are four separate sequences of row and column motion, which act to interlock the yarns and produce the braided preform. The yarns are mechanically forced into the structure between each step to consolidate the structure in a similar process to the use of a reed in weaving. Two-step braiding is unlike the four step process because the two-step includes a large number of yarns fixed in the axial direction and a fewer number of braiding yarns. The process consists of two steps in which the braiding carriers move completely through the structure between the axial carriers. This relatively simple sequence of motions is capable of forming preforms of essentially any shape, including circular and hollow shapes. Unlike the four steps process the two steps process does not require mechanical compaction the motions involved in the process allows the braid to be pulled tight by yarn tension alone. The last type of braiding is multi-layer interlocking braiding that consists of a number of standard circular braiders being joined together to form a cylindrical braiding frame. This frame has a number of parallel braiding tracks around the circumference of the cylinder but the mechanism allows the transfer of yarn carriers between adjacent tracks forming a multilayer braided fabric with yarns interlocking to adjacent layers. The multilayer interlock braid differs from both the four step and two-step braids in that the interlocking yarns are primarily in the plane of the structure and thus do not significantly reduce the in-plane properties of the preform. The four step and two step processes produce a greater degree of interlinking as the braiding yarns travel through the thickness of the preform, but therefore contribute less to the in-plane performance of the preform. A disadvantage of the multilayer interlock equipment is that due to the conventional sinusoidal movement of the yarn carriers to form the preform, the equipment is not able to have the density of yarn carriers that is possible with the two step and four step machines.

- *Knitting*

Knitting fibre preforms can be done with the traditional methods of Warp and [Weft] Knitting, and the fabric produced is often regarded by many as two-dimensional fabric, but machines with two or more needle beds are capable of producing multilayer fabrics with yams that traverse between the layers. Developments in electronic controls for needle selection and knit loop transfer and in the sophisticated mechanisms that allow specific areas of the fabric to be held and their movement controlled. This has allowed the fabric to form itself into the required three-dimensional preform shape with a minimum of material wastage.

- *Stiching*

Stitching is arguably the simplest of the four main textile manufacturing techniques and one that can be performed with the smallest investment in specialized machinery. Basically the stitching process consists of inserting a needle, carrying the stitch thread, through a stack of fabric layers to form a 3D structure. The advantages of stitching are that it is possible to stitch both dry and prepreg fabric, although the tackiness of the prepreg makes the process difficult and generally creates more damage within the prepreg material than in the dry fabric. Stitching also utilizes the standard two-dimensional fabrics that are commonly in use within the composite industry therefore there is a sense of familiarity concerning the material systems. The use of standard fabric also allows a greater degree of flexibility in the fabric lay-up of the component than is possible with the other textile processes, which have restrictions on the fibre orientations that can be produced.

1.4.3.2 Moulding Process

There are two distinct categories of moulding processes using FRP plastics, this includes composite moulding and wet moulding. Composite moulding uses Prepreg FRP, meaning the plastics are fibre reinforced before being put through further moulding processes. Sheets of Prepreg FRP are heated or compressed in different ways to create geometric shapes. Wet moulding combines fibre reinforcement and the matrix or resist during the moulding process (Erhard & Gunter 2006). The different forms of composite and wet moulding are listed below.

- Bladder moulding

Individual sheets of prepreg material are laid -up and placed in a female-style mould along with a balloon-like bladder. The mould is closed and placed in a heated press. Finally, the bladder is pressurized forcing the layers of material against the mould walls. The part is cured and removed from the hot mould. Bladder moulding is a closed moulding process with a relatively short cure cycle between 15 and 60 minutes making it ideal for making complex hollow geometric shapes at competitive costs.

- Compression moulding

A "preform" or "charge", of SMC, BMC or sometimes prepreg fabric, is placed into mould cavity. The mould is closed and the material is compacted & cured inside by pressure and heat. Compression moulding offers excellent detailing for geometric shapes ranging from pattern and relief detailing to complex curves and creative forms, to precision engineering all within a maximum curing time of 20 minutes.

- Autoclave / vacuum bag

Individual sheets of prepreg material are laid-up and placed in an open mold. The material is covered with release film, bleeder/breather material and a vacuum bag. A vacuum is pulled on part and the entire mould is placed into an autoclave (heated pressure vessel). The part is cured with a continuous vacuum to extract entrapped gasses from laminate. This is a very common process in the aerospace industry because it affords precise control over the moulding process due to a long slow cure cycle that is anywhere from one to two hours. This precise control creates the exact laminate geometric forms needed to ensure strength and safety in the aerospace industry, but it is also slow and labour intensive, meaning costs often confine it to the aerospace industry.

- Mandrel wrapping

Sheets of prepreg material are wrapped around a steel or aluminium mandrel. The prepreg material is compacted by nylon or polypropylene cello tape. Parts are typically batch cured by hanging in an oven. After cure the cello and mandrel are removed leaving a hollow carbon tube. This process creates strong and robust hollow carbon tubes.

- Wet layup

Fibre reinforcing fabric is placed in an open mould and then saturated with a wet [resin] by pouring it over the fabric and working it into the fabric and mould. The mould is then left so that the resin will cure, usually at room temperature, though heat is sometimes used to ensure a proper curing process. Glass fibres are most commonly used for this process, the result are widely known as fibreglass, and is used to make common products like skis, canoes, kayaks and surf boards.

- Chopper gun

Continuous strand of fibre glass are pushed through a hand-held gun that both chops the strands and combines them with a catalyzed resin such as polyester. The impregnated chopped glass is shot onto the mould surface in whatever thickness the design and human operator think is appropriate. This process is good for large production runs at economical cost, but produces geometric shapes with less strength than other moulding processes and has poor dimensional tolerance.

- Filament winding

Machines pull fibre bundles through a wet bath of resin and wound over a rotating steel mandrel in specific orientations Parts are cured either room temperature or elevated temperatures. Mandrel is extracted, leaving a final geometric shape but can be left in some cases.

- Pultrusion

Fibre bundles and slit fabrics are pulled through a wet bath of resin and formed into the rough part shape. Saturated material is extruded from a heated closed die curing while being continuously pulled through die. Some of the end products of the pultrusion process are structural shapes, i.e. I beam, angle, channel and flat sheet. These materials can be used to create all sorts of fibreglass structures such as ladders, platforms, handrail systems tank, pipe and pump supports.

- RTM & VARTM

Fabrics are placed into a mould which wet resin is then injected into. Resin is typically pressurized and forced into a cavity which is under vacuum in the RTM (Resin Transfer Moulding) process. Resin is entirely pulled into cavity under vacuum in the VARTM (Vacuum Assisted Resin Transfer Moulding) process. This moulding process allows precise tolerances and detailed shaping but can sometimes fail to fully saturate the fabric leading to weak spots in the final shape.

1.4.4 Fibre Materials

The matrix must also meet certain requirements in order to first be suitable for the FRP process and ensure a successful reinforcement of itself. The matrix must be able to properly saturate, and bond with the fibres within a suitable curing period. The matrix should preferably bond chemically with the fibre reinforcement for maximum adhesion. The matrix must also completely envelope the fibres to protect them from cuts and notches that would reduce their strength, and to transfer forces to the fibres. The fibres must also be kept separate from each other so that if failure occurs it is localized as much as possible, and if failure occurs the matrix must also deboned from the fibre for similar reasons. Finally the matrix should be of a plastic that remains chemically and physically stable during and after reinforcement and moulding processes. To be suitable for reinforcement material fibre additives must increase the tensile strength and modulus of elasticity of the matrix and meet the following conditions, fibres must exceed critical fibre content, the strength and rigidity of fibres itself must exceed the strength and rigidity of the matrix alone, and there must be optimum bonding between fibres and matrix.

1.4.4.1 Glass fibre material

FRP plastics use textile glass fibres; textile fibres are different from other forms of glass fibres used for insulating applications. Textile glass fibres begin as varying combinations of SiO_2, Al_2O_3, B_2O_3, CaO, or MgO in powder form. These mixtures are then heated through a direct melt process to temperatures around 1300 degrees Celsius, after which dies are used to extrude filaments of glass fibre in diameter ranging from 9 to 17 µm. These filaments are then wound into larger threads and spun onto bobbins for transportation and further processing. Glass fibre is by far the most popular means to reinforce plastic and thus enjoys a wealth of production processes, some of which are applicable to aramid and carbon fibres as well owing to their shared fibrous qualities.

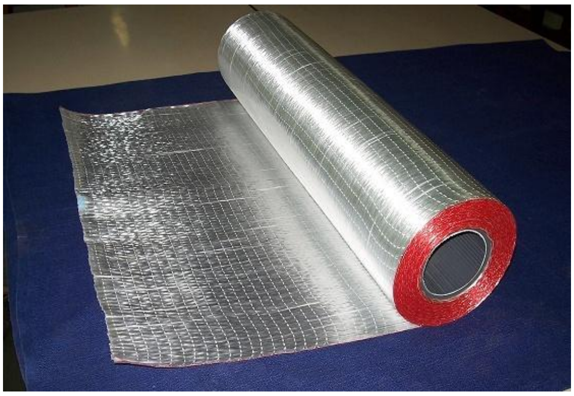


Fig 1.1: Glass fibre material.

1.4.4.2 Carbon fibre material

Carbon fibres are created when polyacrylonitrile fibres (PAN), Pitch resins, or Rayon are carbonized (through oxidation and thermal pyrolysis) at high temperatures. Through further processes of graphitizing or stretching the fibres strength or elasticity can be enhanced respectively. Carbon fibres are manufactured in diameters analogous to glass fibres with diameters ranging from 9 to 17 µm. These fibres wound into larger threads for transportation and further production processes (Erhard & Gunter 2006). Further production processes include weaving or braiding into carbon fabrics, cloths and mats analogous to those described for glass that can then be used in actual reinforcement processes.

Fig 1.2: Carbon fibre material.

1.4.4.3 Aramid Fibre Material

Aramid fibres are most commonly known Kevlar, Nomex and Technora. Aramids are generally prepared by the reaction between an amine group and a carboxylic acid halide group (aramid) (Smallman & Bishop 1999), commonly this occurs when an aromatic polyamide is spun from a liquid concentration of sulfuric acid into a crystallized fibre (Erhard & Gunter2006). Fibres are then spun into larger threads in order to weave into large ropes or woven fabrics (Aramid). Aramid fibres are manufactured with varying grades to based on varying qualities for strength and rigidity, so that the material can be somewhat tailored to specific design needs concerns, such as cutting the tough material during manufacture.

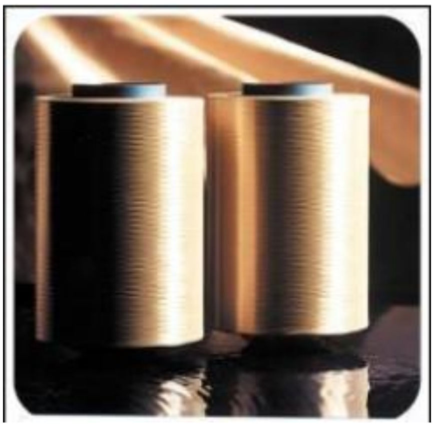


Fig 1.3: Aramid fibre material.

Table 1.6 shows the Materials ingredients of respective Reinforcing materials and the properties improved by them. All three types of FRP composites, namely Glass fiber (GFRP), carbon fiber (CFRP) and Aramid (AFRP) have been used for strengthening RC struvctures in both practical application and research. Table 1.7 illustrates the wide variety of strength and stiffness that FRC may possess (originally given by Head 1999).

Reinforcing Material	Most Common Matrix Materials	Properties Improved
Glass Fibres	UP, EP, PA, PC, POM, PP, PBT, VE	Strength, Elasticity, heat resistance
Wood Fibres	PE, PP, ABS, HDPE, PLA	Flexural strength, Tensile modulus, Tensile Strength
Carbon and Aramid Fibres	EP, UP, VE, PA	Elasticity, Tensile Strength, compression strength, electrical strength.
Inorganic Particulates	Semi-crystalline Thermoplastics, UP	Isotropic shrinkage, abrasion, compression strength

Table 1.6: Ingredients and properties improved by FRP materials.

The stress strain curves of CFRP and GFRP is shown in Fig 1.1. The summary of the material properties given by Kachlakev and McCurry 2000 is shown in Table 1.9.

Unidirectional advanced composite materials	Fibre content (% by weight)	Density (kg/m3)	Longitudinal tensile modulus	Tensile strength (MPa)
Glass Fibre / Polyester GFRP laminate	50-80	1600-2000	20-55	400-1800
Carbon Fibre / Epoxy CFRP laminate	65-75	1600-1900	120-250	1200-2250
Aramid Fibre / Epoxy AFRP laminate	60-80	1050-1250	40-125	1000-1800

Table 1.7: Properties of FRP materials given by Head 1999.

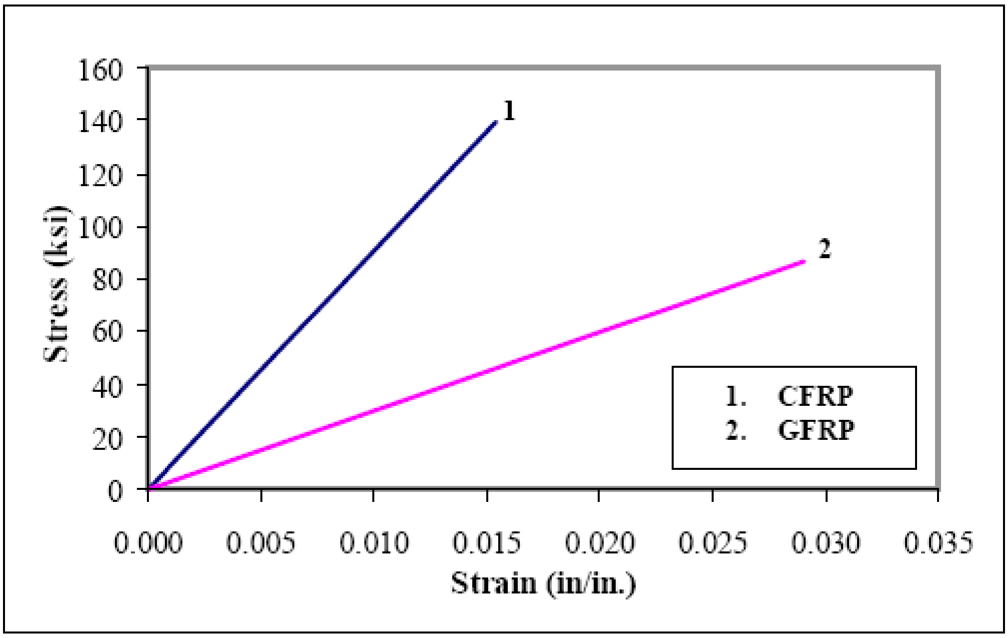


Fig 1.4: Stress strain curve of CFRP and GFRP.

FRP composite	Elastic modulus (MPa)	Major Poisson`s ratio	Tensile strength (MPa)	Shear modulus (MPa)	Thickness of laminate (mm)
CFRP	E_x=62000 E_y=4800 E_z=4800	ν_{xy}=0.22 ν_{xz}=0.22 ν_{yz}=0.30	958	G_{xy}=3270 G_{xz}=3270 G_{yz}=1860	0.3
GFRP	E_x=21000 E_y=7000 E_z=7000	ν_{xy}=0.26 ν_{xz}=0.26 ν_{yz}=0.30	600	G_{xy}=1520 G_{xz}=1520 G_{yz}=2650	1.0

Table 1.9: Mechanical properties of FRP materials given by Kachlakev & McCurry 2000.

1.4.5 Applications

Fibre-reinforced plastics are best suited for any design program that demands weight savings, precision engineering, finite tolerances, and the simplification of parts in both production and operation. A moulded polymer artefact is cheaper, faster, and easier to manufacture than cast aluminium or steel artefact, and maintains similar and sometimes better tolerances and material strengths. The Mitsubishi Lancer Evolution IV also used FRP for its spoiler material. Rudder of A310 Airbus also used Carbon Fiber composite plastic with its various advantages. The advantages over a traditional rudder made from sheet aluminium are:

- 25% reduction in weight.
- 95% reduction in components by combining parts and forms into simpler moulded parts.

- Overall reduction in production and operational costs, economy of parts results in lower production costs and the weight savings create fuel savings that lower the operational costs of flying the aeroplane.

FRP can be applied to strengthen the beams, columns and slabs in buildings. It is possible to increase strength of these structural members even after these have been severely damaged due to loading conditions.

Glass fibre reinforced polymers or concrete Engine intake manifolds are made from glass fibre reinforced PA 66. Advantages of this over aluminium manifolds are:

- Up to a 60% reduction in weight.
- Improved surface quality and aerodynamics.
- Reduction in components by combining parts and forms into simpler moulded shapes.

Automotive gas and clutch pedals made from glass fibre reinforced PA 66 (DWP 12-13) Advantages over stamped aluminium are:

- Pedals can be moulded as single units combining both pedals and mechanical linkages simplifying the production and operation of the design.
- Fibres can be oriented to reinforce against specific stresses, increasing the durability and safety.

For strengthening beams, two techniques are adopted. First one is to paste FRP plates to the bottom (generally the tension face) of a beam. This increases the strength of beam, deflection capacity of beam and stiffness (load required to make unit deflection). Alternatively, FRP strips can be pasted in 'U' shape around the sides and bottom of a beam, resulting in higher shear resistance. Columns in building can be wrapped with FRP for achieving higher strength. This is called wrapping of columns. The technique works by restraining the lateral expansion of the column. Slabs may be strengthened by pasting FRP strips at their bottom (tension face). This will result in better performance, since the tensile resistance of slabs is supplemented by the tensile strength of FRP. In the case of beams and slabs, the effectiveness of FRP strengthening depends on the performance of the resin chosen for bonding.

1.4.6 Advantages and Limitations

FRP allows the alignment of the glass fibres of thermoplastics to suit specific design programs. Specifying the orientation of reinforcing fibres can increase the strength and resistance to deformation of the polymer. Glass reinforced polymers are strongest and most resistive to deforming forces when the polymers fibres are parallel to the force being exerted, and are weakest when the fibres are perpendicular. Thus this ability is at once both an advantage and a limitation depending on the context of use. Weak spots of perpendicular fibres can be used for natural hinges and connections, but can also lead to material failure when production processes fail to properly orient the fibres parallel to expected forces. When forces are exerted perpendicular to the orientation of fibres the strength and elasticity of the polymer is less than the matrix alone. In cast resin components made of glass reinforced polymers such as UP and EP, the orientation of fibres can be oriented in two-dimensional and three-dimensional weaves. This means that when forces are possibly perpendicular to one orientation, they are parallel to another orientation; this eliminates the potential for weak spots in the polymer. Besides these, there are some possible structural failure conditions where emphasis can be laid upon:

1. Tensile forces stretch the matrix more than the fibres, causing the material to shear at the interface between matrix and fibres.
2. Tensile forces near the end of the fibres exceed the tolerances of the matrix, separating the fibres from the matrix.
3. Tensile forces can also exceed the tolerances of the fibres causing the fibres themselves to fracture leading to material failure.

1.4.7 Design considerations

FRP is used in designs that require a measure of strength or modulus of elasticity that non-reinforced plastic and other material choices are either ill suited for mechanically or economically. This means that the primary design consideration for using FRP is to ensure that the material is used economically and in a manner that takes advantage of its structural enhancements specifically. This is however not always the case, the orientation of fibres also creates a material weakness perpendicular to the fibres. Thus the use of fibre reinforcement and their orientation affects the strength, rigidity, and elasticity of a final form and hence the operation of the final product itself. Orienting the direction of fibres either, unidirectional, 2-dimensionally, or 3-dimensionally during production affects the degree of strength, flexibility, and elasticity of the final product. Fibres oriented in the direction of forces display greater resistance to distortion from these forces and vice versa, thus areas of a product that must withstand forces will be reinforced with fibres in the same direction, and areas that require flexibility, such as natural hinges, will use fibres in a perpendicular direction to forces. Using more dimensions avoids this either or scenario and creates objects that seek to avoid any specific weak points due to the unidirectional orientation of fibres. The properties of strength, flexibility and elasticity can also be magnified or diminished through the geometric shape and design of the final product. These include such design consideration such as ensuring proper wall thickness and creating multifunctional geometric shapes that can be moulding as single pieces, creating shapes that have more material and structural integrity by reducing joints, connections, and hardware (Erhard & Gunter 2006).

1.4.8 Disposal and recycling concerns

As a subset of plastic FRP plastics are liable to a number of the issues and concerns in plastic waste disposal and recycling. Plastics pose a particular challenge in recycling processes because they are derived from polymers and monomers that often cannot be separated and returned to their virgin states, for this reason not all plastics can be recycled for re-use, in fact some estimates claim only 20% to 30% of plastics can be material recycled at all. Fibre reinforced plastics and their matrices share these disposal and environmental concerns. In addition to these concerns, the fact that the fibres themselves are difficult to remove from the matrix and preserve for re-use means FRP amplify these challenges. FRP are inherently difficult to separate into base a material that is into fibre and matrix, and the matrix into separate usable plastic, polymers, and monomers. These are all concerns for environmentally informed design today, but plastics often offer savings in energy and economic savings in comparison to other materials, also with the advent of new more environmentally friendly matrices such as bio plastics and UV-degradable plastics, FRP will similarly gain environmental sensitivity (Smallman & Bishop 1999).

1.5 Objective of present work

After studying about the properties of Carbon Fibre Reinforced Plastic its scope in retrofitting purposes can be very well judged. Based on this fact the research is being carried out to use the Carbon Fibre wrap on retrofitting of the damaged beams. The main objective of this research is to study the effect of pre-stressed Carbon Fibre wrap on the different levels of damaged beams which have failed in flexure. The beams are damaged at different levels which are retrofitted using pre-stressed Carbon Fibre Reinforced Plastic. The retrofitted beams are tested again and their Load-deflection curves are plotted which are then compared with that of the control beam.

1.6 Organization of thesis

The thesis has been organized into the following five chapters:

Chapter 1: This chapter deals with general introduction of retrofitting and Carbon Fibre Reinforced Polymer.

Chapter 2: This chapter deals with literature review leading to the need of the present study.

Chapter 3: This deals with the experimental programme. In this the procedure of the work that has been done in this research is shown.

Chapter 4: This chapter deals with the results and discussion related to the work.

Chapter 5: This chapter presents the conclusions and also provides the scope of future work.

CHAPTER 2 REVIEW OF LITERATURE

2.1 Introduction

Reinforced concrete in one of the most abundantly used construction material not only in the developed world, but also in the remotest parts of the developing world. In the rural areas of the developing world, however, majority of the houses are constructed in traditional manner using indigenously developed techniques preferably following simpler and economical procedures. Unfortunately such non-engineered construction is mostly prevalent in earthquake prone areas of the developing world e.g. Turkey, Pakistan, India and Iran. Earthquake can cause even more destruction in such areas where the building structures are not structurally sound. Such are the cases where we use the word "Retrofitting". It is the modification of existing structures to make them more resistant to seismic activity, ground motion, or soil failure due to earthquakes. With better understanding of seismic demand on structures and with our recent experiences with large earthquakes near urban centres, the need of seismic retrofitting is well acknowledged.

2.2 Rise of Retrofitting in India

India also lies in high alert earthquake zone as recent past justifies this with various earthquakes causing lot of destruction to human life. Based on these past earthquakes especially after Bhuj earthquake in 2001, a lot of retrofitting measures are adopted. A report has been prepared by Rajendra Desai, Hon. Director National Centre for Peoples-Action in Disaster Preparedness (NCPDP) Ahmadabad, Gujarat, India in 2008. The report deals with four case studies where retrofitting measures are adopted first in India. This report can also be regarded as "The rise of retrofitting in India".

2.3 Case studies and lessons learnt

Seismic Retrofitting for reduction of vulnerability of a structure is a relatively new concept in India. It was only after Latur Earthquake of 1993 that retrofitting was taken up on a substantial scale as a part of the earthquake rehabilitation program under which many public buildings were retrofitted. The quantum got substantially increased in the aftermath of 2001 Kutch Earthquake. In spite of all that got done retrofitting as an option, a technique, a profession or a business is still in its infancy. This is manifested by five indicators –

(a) A delivery system for retrofitting does not exist.

(b) Official Schedule of Rates (SOR) of any government agency does not include seismic retrofitting.

(c) Contractors and skilled artisans knowledgeable in this are scarce.

(d) People at large have no knowledge of the option of retrofitting.

(e) Information on retrofitting is hard to find.

As a result the use of retrofitting as a tool for managing the earthquake risk is fraught with too many obstacles, putting it beyond the reach of an ordinary person. In India, it would not be an exaggeration to say that over 80% buildings that consist of non-engineered masonry are vulnerable against the hazard of future earthquake. These cover a broad range of buildings starting from small mud houses in remote villages all the way to the moderately large infrastructure buildings in cities. With the country witnessing a large number of deaths and incurring huge losses every year resulting from disasters it is important that the vulnerability of these non-engineered masonry structures is reduced through retrofitting. Fortunately, a substantial amount of pioneering work has been done in different parts of the country on seismic retrofitting of “non engineered masonry” buildings, although by a few individuals. This includes the:

(a) Development of regional technical guidelines in a number of regions,

(b) Making of public awareness materials in the regional languages,

(c) Most importantly, the actual execution of retrofitting of local variants of masonry structures coupled with some artesian training on retrofitting.

Since each region poses significantly different context, such an effort required fresh approach to evolve the solutions to tackle the problems on hand peculiar to the area. This involved different building technologies, different materials, difficulties of access, unreliability of electric power, unavailability of basic as well as special materials needed for retrofitting etc. The retrofitting work carried out in various regions, although on a small scale, offers a number of lessons that could be valuable for the further development of retrofitting as well as for its promotion as the most attractive option for reducing vulnerability.

2.3.1 House of Haribhau in Nagarsoga village Latur, Maharashtra, Year 1994.

Primary objective: To get first hand understanding of retrofitting: “Learning while doing”.

Building System: Heavy Mud roofing on self supported timber deck and Random Rubble Masonry walls in mud mortar.

Building Area: 2 large rooms – 40 sq.m.

Retrofitting Measures:

Restoration of damages followed by:

(a) Stitching of stone wythes with Cast in-situ RC Stitching Elements.

(b) Installation of roof level RC Band after removal of the upper part of the walls including the projection above the roof.

(c) Installation of Knee Braces at the junction of timber columns and beams.

Executing Agency: ASAG with the help of local masons and the house owner.

Special Features Used:

- Installation of Cast in-situ RC Stitching Elements involving making of dumbbell shaped holes through the stone wall.
- Installation of RC Band in the "Maalwad" style existing house involving the partial dismantling of masonry wall.
- Knee Braces for different configurations of timber columns and beams – fabrication by local metal work shops.

Problems Encountered:

- No awareness of retrofitting option among people. As a result not many people were interested in this option. Lack of confidence among engineers did not help this process.
- Complicated selection process of a house (simple with no major damage) to work upon because of lack of experience of restoration, retrofitting and random rubble masonry.

	
Retrofitting & Restoration completed	Knee braces at beam-column joint
	
Upper Wall removed for band installation & Roof deck opened for repairs	Cast in-situ RC Stitching Element being plaster

Fig 2.1: Repair work at House of Haribhau in Nagarsoga village Latur, Maharashtra, Year 1994

- The risk and the skills involved in the installation of Cast in-situ RC Stitching Elements.

- Thick stone walls precluded use of electric drill for making holes.
- Improvisation of tools for making holes in random rubble wall.
- Lack of necessary skill with masons required intensive hands on training of masons.

Two more houses were retrofitted subsequent to which a large area level retrofitting program was taken up. A complete delivery system was evolved for effective and efficient execution of the program. In all 900 masons were trained and 150 houses retrofitted.

2.3.2 R & B Office cum Storage at Patadi town in Gujrat, Year 2002.

Building System: AC Sheeting over wooden under structure supported on Random Rubble Masonry in cement mortar.

Building Area: 4 rooms and a passage, 80sq.m. covered area.

Retrofitting Measures:

- Stitching of stone wythes with Cast in situ RC Stitching Elements.
- WWM Seismic Belt at eave level.
- Vertical Reinforcing Bars in corners anchored to walls and encased in micro concrete.
- Encasing of wall openings with WWM Seismic Straps.
- Roof diaphragm improvement with the help of Diagonal Ties made of 13 gauge pre-tensioned multiple strand GI wires and timber struts.
- Strengthening of connections between roofing elements.
- Anchoring of elements of roof under structure to walls.
- Closing off of a window opening.
- Restoration of earthquake damage.

Executing Agency: NCPDP with the help of local masons.

Special Features Used:

- Additional Seismic Belt with WWM for extra high walls.
- Extensive use of Seismic Belt with WWM for encasement of openings.
- Anchoring of roofing elements to support walls.
- Blocking off of a window opening.

Problems Encountered:

• Absence of awareness about the significance of retrofitting in public as well as R & B engineers resulted in to little learning by them for future use.

• Absence of necessary skills with masons required their training and more supervision.

	
Gable & Lintle Seismic Belt, Stitching element, window blocking	Lintle & Ridge Seismic Belt, Roof Bracing, Stitching Elements

Fig 2.2: Repair work at R & B Office cum Storage at Patadi town in Gujrat, Year 2002.

With wide firsthand experience of retrofitting in Latur region from technical angle it wasn't difficult to take up retrofitting activities. A major program was taken up for the government for awareness building in 480 villages and training of 6000 masons.

2.3.3 Primary School in Sultan Daki village, Year 2006.

Building System: CGI sheeting over wooden under structure anchored to wooden attic floor deck which in turn supported on Random Rubble Masonry in cement mortar.

Building Area: 3 rooms and a verandah, 150sq.m.

Retrofitting Measures:

- Stitching of stone wythes with Cast in-situ RC Stitching Elements.
- WWM Seismic Belt at eave level.
- Vertical Reinforcing Bars in corners anchored to walls and encased in micro concrete.
- Encasing of wall openings with WWM Seismic Straps.
- Roof diaphragm improvement with the installation of timber bracings and struts.
- Strengthening of connections between roofing elements.
- Anchoring of attic floor to walls with the help of vertical rebars and special MS brackets.
- Diagonal timber bracings between timber columns in verandah.
- Restoration of earthquake damage.

Executing Agency: NCPDP with the help of local masons who were all getting trained while working on this building.

	
Retrofitting completed with bracings between columns	Roof to Wall Anchoring
	
Installation of WWM for Lintle level Seismic Belt	Seismic Belt & Opening Encasement

Fig 2.3: Repair work at Primary School in Sultan Daki village, Year 2006.

Problems Encountered:

- Absence of awareness about retrofitting in public as well as local masons resulted in to some scepticism about retrofitting the school in the beginning.
- Local village level politics and lack of awareness for retrofitting called for meetings and lobbying.
- Retrofitting was not a part of government program. This meant that no help would come from government engineers and in turn they did not learn anything from this.
- Lack of necessary skill with masons required intensive hands-on training of masons.
- Reaching the remote site with limited transportation and security checks made it more difficult.
- Procurement of galvanized WWM of the desired specifications demanded a lot of extra efforts and resulted in to a delay of one month.
- Thick stone walls and unreliable electric power supply precluded the use of electric drill.

With wide firsthand experience of retrofitting in Latur, mountainous Uttarakhand, and Gujarat from technical angle it wasn't difficult to take up retrofitting activities. Based on the experience of this school and Kupwada District Hospital a detailed manual on Retrofitting and Restoration was prepared for UNESCO.

Special Features Used:

- Anchoring of the triangular roof/attic box to stone walls using special brackets.
- Diagonal timber bracings between timber columns.

2.3.4 Ramnagar Primary School of Delhi Municipal Corporation, Delhi, Year 2007.

Building System: Three storey building with load bearing brick masonry walls supporting RC slab roof and intermediate floors.

Building Area:: G=2 storeys, 24 rooms and lobby, 1816 sq. mt.

Retrofitting Measures:

- WWM Seismic Belt at lintel and sill levels.
- Vertical Reinforcing Bars in corners anchored to walls and encased in micro concrete.
- Encasing of wall openings with WWM Seismic Straps.
- Anchoring of slabs to walls.
- Jacketing of masonry columns.

Executing Agency: NCPDP with the help of a team of experienced masons and labourers brought from Gujarat.

Problems Encountered, their impact & solution:

- Lack of awareness on the significance of retrofitting meant limited cooperation and support from all different quarters, making the execution task more difficult and more time consuming.
- Working in the functioning school building resulted in to unplanned delays and called for regular coordination with school authorities.
- No noisy activity such as plaster breaking or drilling work could be done during school hours.
- School children played with construction materials resulting in to waste and also disturbed the incomplete work.
- Extreme caution had to be exercised for the safety of children.
  Procurement of galvanized WWM of the desired specifications and the galvanized 6mm MS bars resulted in to frequent and long delays. Significant variations observed in the quality of galvanizing in the absence of standardization. 6mm galvanized MS bars were delivered in smaller lengths and in tangled conditions.
- Extensive removal of plaster for seismic belts and vertical reinforcement tackled with electric rotary grinder.

	
Jacketed Masonry Columns & Seismic Belts	Jacketing reinforcement
	
Lintle & Sill Belts & Window Encasement	Encasement of door way

Fig 2.4: Repair work at Ramnagar Primary School of Delhi Municipal Corporation, Delhi, Year 2007.

- Extensive drilling in to RC slabs for the vertical reinforcement and in masonry columns for installing shear connectors was tackled with good quality heavy duty electric drill.
- In the documentation of large buildings some details are always missed, many times due to closed access or obscurity due to plaster. At the time of execution of retrofitting this missing information results in to changes in work plan affecting the budget and time table.
- In large buildings extra items crop up easily. To make sure contractor gets paid for these the MOU would have to be suitably prepared.
- A large team of suitably skilled artisans and laborers had to be taken from Ahmadabad to Delhi since locally they were not available. This increased the labor cost.

- No technology transfer could take place. Thus the unavailability of skilled masons and labourers did not change.

Special Features Used:

- Three storey structure required scaffolding and safety equipment for the workers who were not used to such heights.

After studying these case studies where retrofitting measures were taken an initiative step, following conclusions can be laid:

- Awareness creation in the community at large about the possible dangers of a future disaster and the significance of retrofitting is a prerequisite for the promotion of retrofitting.
- Simple booklets and brochures on the subject must be made easily available in local language.
- Government engineers must receive rigorous onsite training in the retrofitting of non-engineered buildings to enable the respective agency to take up retrofitting projects.
- Retrofitting skills must become easily available in the market through the Hands-on training programs for masons.
- Public agencies owning buildings must be made aware of the need for retrofitting these buildings to help them assign right priority to retrofitting.
- Special materials required for retrofitting must be available easily.
- For the most common building systems the retrofitting items must be standardized with their specifications, and SOR must be developed and recognized.
- Documentation system also needs to be standardized so that all engineers use the same language of communication.
- Engineering community needs to become proactive in this direction just like in many of the Western countries.

2.4 Research Findings

Realising the importance of retrofitting in today's world, a lot of research work has been carried on all across the globe.

R G wight et al (2001) investigated the effect of Pre-stressed FRP sheets, applied to post strengthened RC beams for strengthening. In their study they investigated the behaviour of reinforced concrete beams strengthened with pre-stressed FRP sheets, and compared the effectiveness of FRP pre-stressed sheets with non pre-stressed sheets. In their study they casted four beams of size 5000×300×575 mm. One beam was made control beam, one beam was strengthened with CFRP sheet and two beams were strengthened with pre-stressed CFRP sheet. CFRP sheets were applied on the tension face of the beam. All beams were strengthened with five layers of CFRP sheets. Mechanical pre-stressing and anchorage system was used for pre-stressing the beam. The mechanical anchorage system consisted of steel roller anchors bonded to the sheets and steel anchor assemblies fixed to the beam. The roller anchors that gripped the sheet consisted of two stainless-steel rollers bonded to each end of the sheet. To pre-stress 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 movable (jacking end). During pre-stressing process CFRP sheets tended to separate itself from the beam due to imperfections in the tension face of the beam. To make this contact, force was applied perpendicularly to CFRP sheets. Epoxy was used for making bond between CFRP sheet and beam surface, after applying epoxy it was cured for two weeks before the beams were tested to failure. Load was applied to the beam using a 500 KN capacity actuator. After applying loads it was observed that stiffness of the beams strengthened with non pre-stressed and pre-stressed CFRP sheets were the same as the control beam. Strengthening the beam with CFRP sheet slightly increased the first crack load, but when the sheets were pre-stressed, the cracking load increased significantly. Cracking of pre-stressed beams was delayed to applied loads by 45 to 55 KN above the cracking load of the control beam which represented an increase in cracking load over the control beam by 150%. CFRP sheets contributed to the load-bearing capacity of the beams and the stresses carried by steel reinforcement were redistributed to the CFRP. This led to increased magnitudes of moment when the steel yielded. The beam strengthened with non pre-stressed sheets, had yielding of the steel occurred at a load 20% higher than that of the control beam. When pre-stressed CFRP sheets were used for beams, the internal steel reinforcement was relieved of tensile stresses and placed slightly into compression. Because of the pre-stress, a greater portion of the tensile stresses were transferred from the steel reinforcement to the CFRP. When pre-stressed sheets were bonded to the concrete beams, yielding occurred at loads that were 35 to 40% higher than that of the control beams. Increase of 35 % in ultimate load was observed in when beam strengthened with CFRP and 45% was observed when strengthened with pre-stressed sheets. Displacement and curvature at any load were significantly less with the addition of the non pre-stressed and pre-stressed CFRP sheets. Un-strengthened beams have wide cracks than strengthened beams. Displacements and curvatures were further reduced when the sheets were applied with a pre-stress because of the initial camber in the system and relatively larger tensile force was carried by the sheets which reduces crack widths. It was concluded from the study that Pre-stressed FRP sheets can significantly improve the serviceability of a reinforced concrete structure and reduces crack widths and delays the onset of cracking, can dramatically decrease the strains in the reinforcing steel and delay its yielding .Because if cracking is reduced, the beams with pre-stressed sheets have smaller deflections and curvatures at failure. Although FRP sheets can significantly increase the ultimate strength of concrete beams, pre-stressed sheets are slightly more effective at strengthening than unstressed sheets, because pre-stressing can prevent premature failures.

Czaderski & Motavalli (2005) took a 17 m long internally pre-stressed concrete girder from a bridge in southern Switzerland and strengthened it flexurally using pre-stressed CFRP plates and then tested it to failure. The load carrying behaviour of this test girder was compared to a reference girder and a girder strengthened using non-pre-stressed plates. The test girders were taken from one of the middle spans of a continuous four span member which were constructed using prefabricated pre-stressed concrete I-sections and near the support T-sections as shown in Fig 2.5.

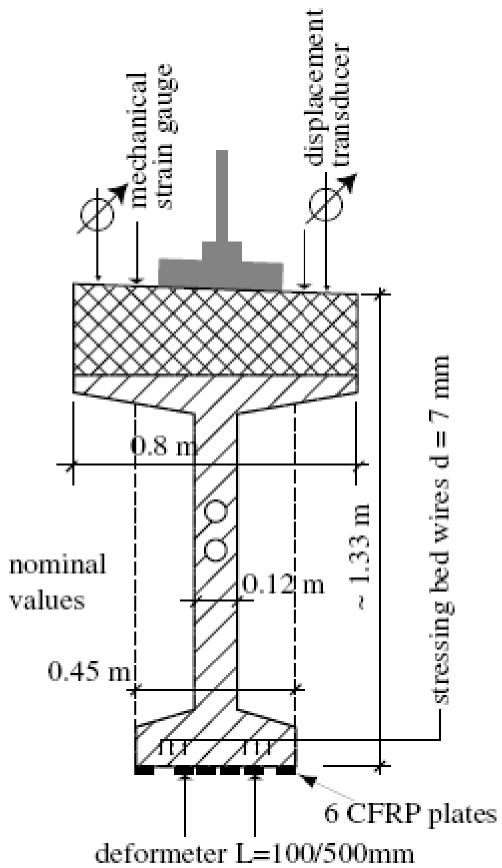


Fig 2.5: Section used for testing (Czaderski & Motavalli (2005))

The test program consisted of five similar test girders. An overview of the test program is shown in Table 2.1.

Test Girder	Investigation
No.1	Material, pre-stressing force.
No.2	Strengthening with pre-stressing CFRP plates.
No.3	Reference girder.
No.4	Strengthening with non-pre-stressed CFRP plates.
No.5	Strengthening of damaged girder with CFRP plates.

Table 2.1 Test setup programme (Czaderski & Motavalli (2005)).

The girders were tested according to the test program and retrofitted respectively and the material used was Sika CarboDur S512 CFRP plates with uni-directional carbon fibres. The CFRP plates were bonded to the concrete by using an epoxy based adhesive which cures faster at elevated temperatures as shown in Fig 2.6. A special computer controlled device fixed directly to the test girder was used for the pre-stressing and heating process.

Fig 2.6 Test setup by Czaderski & Motavalli (2005).

Load deflection curve were drawn and it was observed that smaller deflections at the same load were there for girder 2, strengthened with pre-stressed CFRP plates, than for the other two girders. At low load levels until approximately 250 kN, the curve displayed higher stiffness of girder 2 than girders 3 and 4 indicating fewer cracks and crack width. Furthermore, the maximum recorded load for girders 2 and 4 were 45% and 24% higher than the reference girder, respectively. In addition to measurement of the deflections using inductive displacement transducers and strains using electrical resistance strain gauges, extensive strain measurements using mechanical strain gauges were performed. The strains and deflections were taken twice and the final results reported were the mean values of measurements taken from both sides of the girder. Finally their research concluded that the use of pre-stressed CFRP plates for strengthening of the test girder resulted in:

(a) Lower deflections.

(b) Lower strains at top and bottom face of the girder due to loading.

(c) Fewer cracks and crack width.

(d) Less debonding length between the CFRP plates and concrete at mid-span, because of the lower strain peak in the CFRP plates.

(e) Higher maximum load.

Therefore, the pre-stressing of the CFRP plates improved the behaviour at serviceability (SLS) and ultimate limit state (ULS).

Amery & Mahaidi (2005) carried out the experiments for the retrofitting of beams through CFRP straps as well as by CFRP sheets and checked out the variation in their behaviour on the beams. Six RC beams with various CFRP retrofitting schemes were manufactured and tested. One concrete beam RR1 was kept without retrofitting and was considered as a control beam for comparison. Beams RR2 was retrofitted with CFRP Straps whereas RR3 was retrofitted with CFRP sheets. The rest 3 beams, RR4, RR5, RR6, were retrofitted with the combination of CFRP sheets and straps. Also, to increase the applied moments at the critical sections of the beams, beams RR1 and RR2 were loaded under four point loading whereas beams RR3 to RR6 were loaded with three point loading arrangements (shown in Fig. 2.7a and 2.6b).

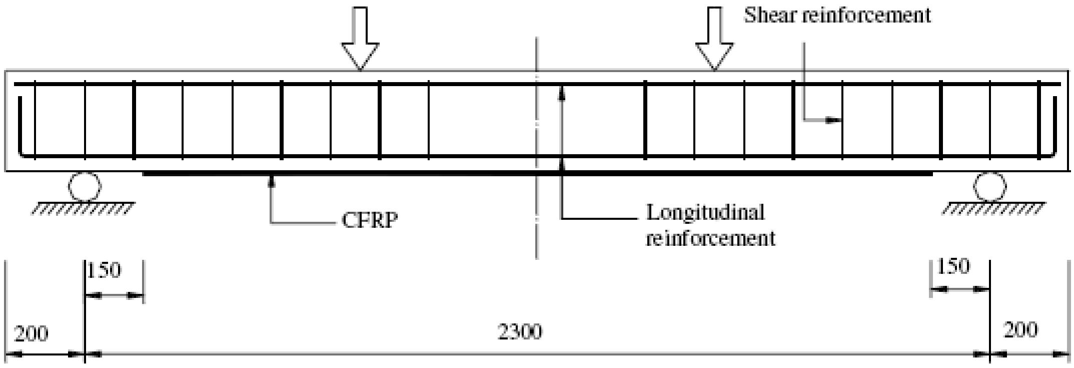


Fig 2.7a: Four point bending test setup (Amery & Mahaidi (2005))

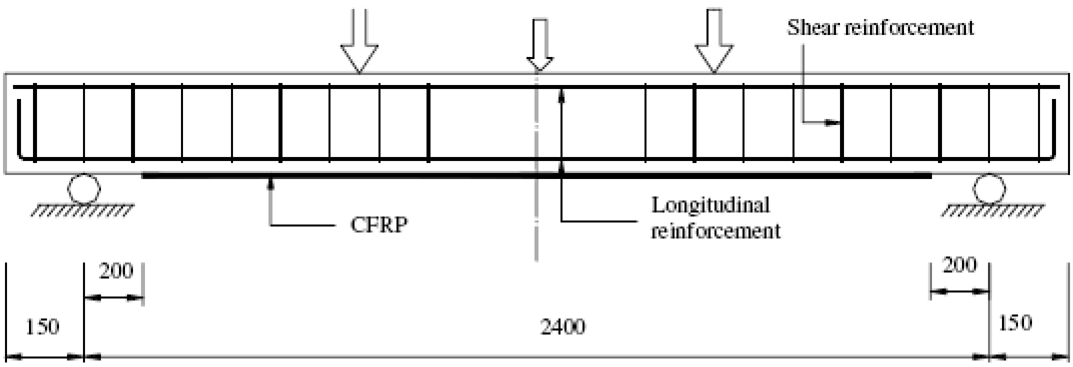


Fig 2.7b: Three point bending test setup (Amery & Mahaidi (2005)).

The CFRP straps are one layer 50 mm wide CFRP in a complete loop with 75 mm overlap and spacing of 200 mm along the beam span. The CFRP sheets are of three layers applied centrally in a wet lay-up process along the bottom surface of the beams having a width of 100 mm and a length of 2000 mm. The carbon fibre used was M Brace CF 130. Proper care was taken in the application of retrofitting materials on the beams as it is regarded as an important factor in enhancing the strength of beams. The bonding surface of the concrete beam were prepared using high pressure water jet to remove the exterior thin film of past and expose the coarse aggregate. This method proved to be efficient and led to relatively rough surfaces necessary for achieving strong bond. Wet lay-up process was used for CFRP application through which a resin undercoat layer was applied first, then first layer of carbon fibre sheet, resin over-coat, second layer of carbon fibre sheet and so on. The complete system was left to cure for one week at room temperature. Load cells, linear voltage displacement transducers (LVDT), electric resistance strain gauges and fibre optics strain sensors were used for instrumentations.

The results showed some really interesting facts shown in Table 2.2.

Beam	Failure load	% Increase in moment capacity	Failure mode
RR1	106.2	0	Shear
RR2	121.4	14.3	Flexure
RR3	100.3	61.8	Shear
RR4	112.1	80.9	Flexure (CFRP break)
RR5	126.3	103.8	Flexure (CFRP break)
RR6	123.2	98.7	Flexure (CFRP break)

Table 2.2: Results after retrofitting (Amery & Mahaidi (2005).

Also Figure 2.8 a,b,c shows the load deflection graph of all the beams. Results showed that CFRP sheets alone increases the flexural strength by 15% and when CFRP straps are used to anchor the CFRP sheets, increase of 95 % was obtained. They also concluded that the use of the CFRP straps prevents the occurrence of debonding failure by providing an extra anchorage mechanism for the CFRP sheets.

Bank & Arora (2006) tested the beams retrofitted with a unique combination of FRP strips with carbon and E-glass unidirectional fiberss and continuous strand mats that were fastened to the concrete beams with steel powder-actuated (PA) fasteners and expansion anchors (EA) and were tested to different failure mode. The beams were designed to fail in a ductile manner. The results concluded that beams retrofitted by this unique combination increase the yield moment and ultimate moment carrying capacities by upto 25% and 58 % respectively. The failure of the beam occurs due to concrete compression failure at large deflections and

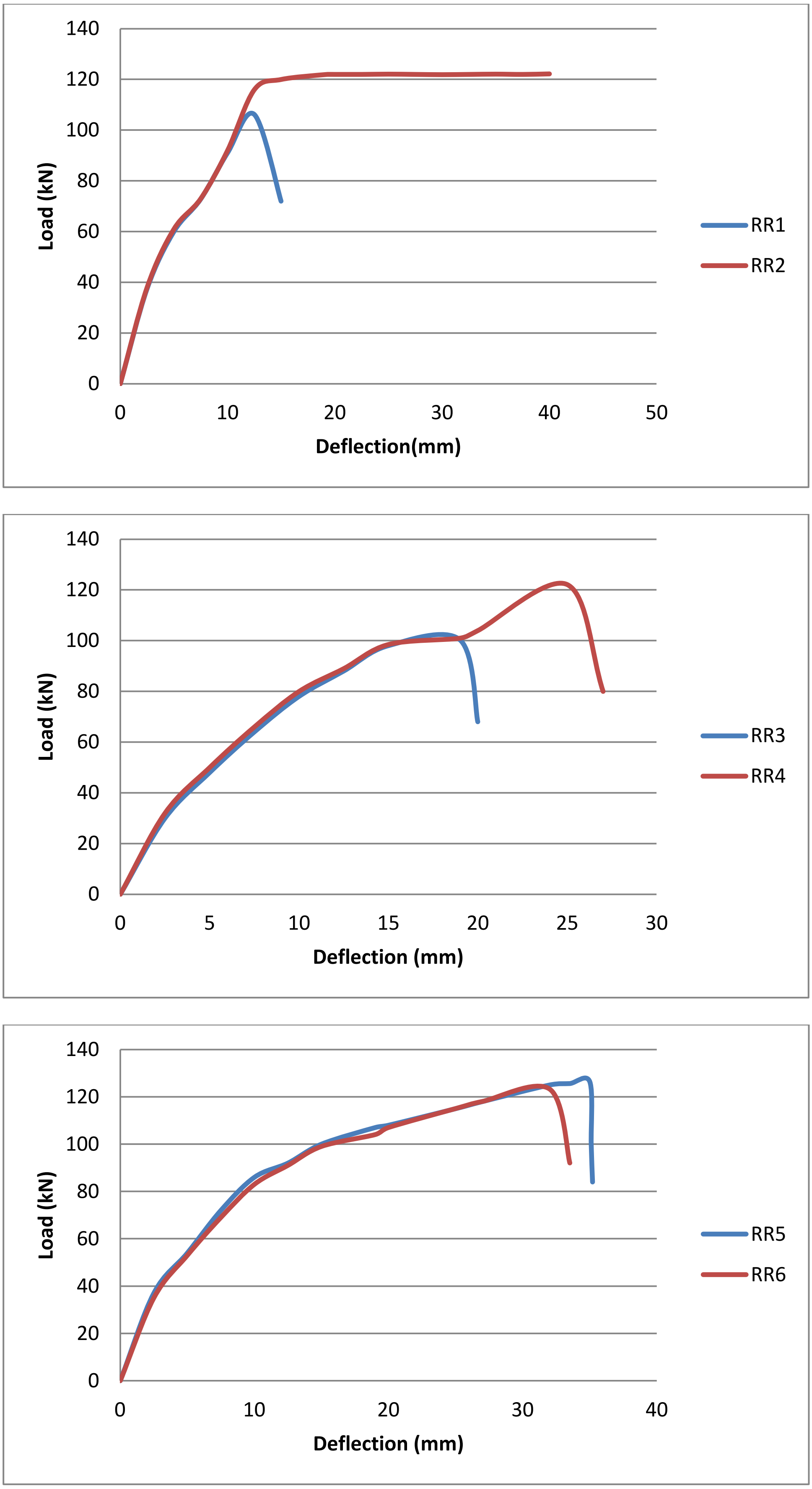


Fig 2.8 a,b,c: Comparison load deflection curves of beams (Amery & Mahaidi 2005)

not by tensile failure and also even after failure, the FRP was seen still attached to the beam and the failure was reported as ductile failure.

Yail J Kim et al (2008) founded the flexure behaviour of slabs with pre-stressed and non-pre-stressed CFRP sheets. In this research, flexural behaviour of the tested slabs, including the load-deflection response, strain distribution, crack propagation, and crack mouth opening displacement were found for four large-scale flat plate slabs 3,000 mm×3,000 mm ×90 mm having span length of 2700mm.Slabs were subjected to a monotonic patch load at centre of span with the help of plate. One slab was kept control slab and other three were strengthened with CFRP sheets out of which one was strengthened with non pre-stressed CFRP sheet and two were strengthened with pre-stressed CFRP sheets. After applying load an increase in the flexural load-carrying capacities of up to 25 and 72% was achieved for the slab strengthened with non-pre-stressed and pre-stressed CFRP sheets, as compared to the un-strengthened control slab. Failure pattern of the control slab was very ductile but for strengthened slabs it was step wise failure. High local strains were observed in the steel and the CFRP sheets under the loading area, accompanying large crack openings. The pre-stressed CFRP sheets provided a notable load sharing mechanism with the steel reinforcement that resulted in higher yield loads with respect to the un-strengthened slab. Crack mouth opening displacements at the location of the CFRP strengthened region were slightly lower than those at the centre span.

Mukherjee & Rai (2009) experimentally investigated the flexural behaviour of reinforced concrete (RCC) beams that have reached their ultimate bearing capacities and then retrofitted with externally pre-stressed carbon fibre reinforced composite (CFRC) laminates. The main objectives of the rehabilitation were:

- to increase capacity of the beams,
- avoid tensile and shear failure,
- Recover permanent deformations.

For testing, Beam of length 1.8 m and cross-section 90×180 mm was used. Concrete mix was prepared using Portland cement blended with fly ash. For longitudinal reinforcement of beams, high yield strength deformed steel bars and for shear links, mild steel were used. Detailed dimensions and reinforcements are shown in Fig.2.9. The beam sections that were made were under-reinforced. After testing the beam to their maximum load carrying capacity, they were retrofitted using two types of CFRC sheets and laminates. The laminates were pre-stressed and used in the longitudinal direction of the beams, and the sheets were used in the ends to protect the edges. For proper adhesion, a compatible epoxy system was used. It had two components, A – resin and B – hardener. Ratio of the components by weight is 100 parts of component A to 23 parts of component B. Mixing was done thoroughly for 5 min with low speed mixer at 400 rpm until components were thoroughly dispersed.

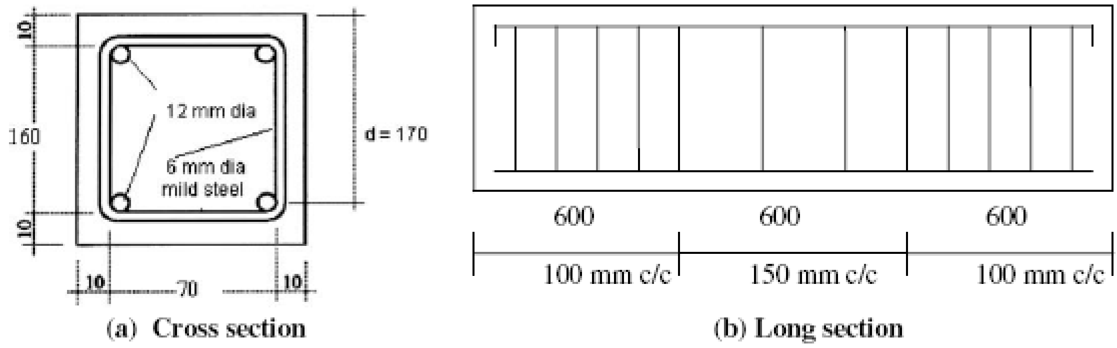


Fig. 2.9: Cross section of beam used for testing (Mukherjee & Rai 2009).

For testing, all the reinforced concrete (RC) beam specimens were loaded in a four point bend test setup as shown in Fig.2.9. The test was carried out on the RC beams prior to the application of any CFRC. The set up ensured pure bending in the central third portion of the beam. The beams were loaded with equal force on the two load points until the beams deformed did not take any further load. A deflection controlled experiment was carried out and the load rate was kept slow at 0.5 mm/s. The deflection of the beam was monitored with linear variable diode transducers (LVDT). As the specimens were under-reinforced, the steel had yielded in all the specimens. The damage in the beams started with bending cracks in the central region of the beam.

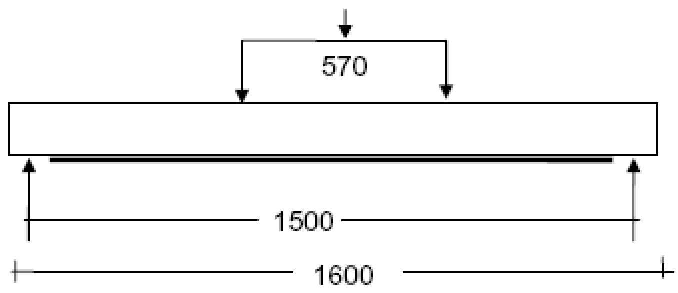


Fig. 2.10: Four point bending test setup (Mukherjee & Rai 2009).

At higher load levels the shear and shear-bending cracks at the end sections was initiated. At still higher levels of deformation, the cracks coalesced with a rapid loss of stiffness. The loading was discontinued when the load deflection curve was flat and no increase load was observed due to the increase in deflection. After unloading the permanent deformations in all the specimens have been recorded. Fig.2.10 shows the average load-displacement curve of the beams tested.

After the beams were damaged, they were repaired by using the pre-stressed FRC. As the fresh beams had failed due to the yielding of tension steel, the aim of the rehabilitation was to enhance the tension reinforcement to a level so that the tension failure of the reinforcement was avoided. 350 mm wide unidirectional CFRC wraps were used at both ends of the beams. It was applied on the tension face of the beam and as a result, the failure was shifted to the compression region. The resultant increase in the load capacity of the beam also enhanced the shear forces. The rehabilitation aims at augmenting the shear capacity of the beam by applying a CFRC wrap in the shear critical zone.

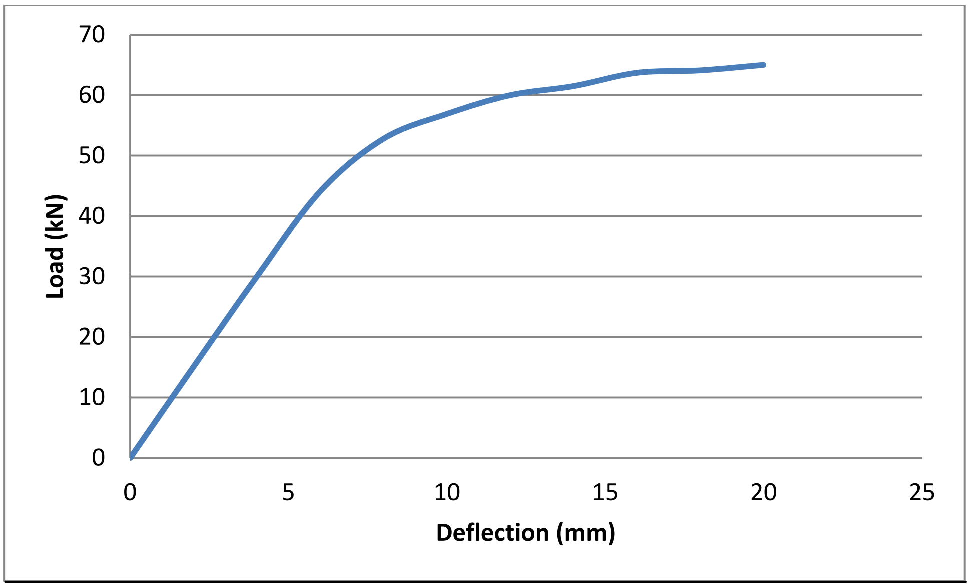


Fig 2.11: Load-deflection curve of beam specimen tested by Mukherjee & Rai (2009)

The wrap would also anchor the laminates to prevent their delimitation to augmentation of strength. In addition the permanent deformation in the beams after the first test was reverted by applying pre-stress on the laminates. The main parameter was the level of pre-stress that was decided based on the capacities of the pre-stressing facilities and the strength of the beams.

For pre-stressing of CFRC laminates, a specially designed machine was used as shown in Fig. 2.11. The laminate was mounted on the two drums of the pre-stressing machine and secured at the ends to prevent slippage of the laminate. The surface of the laminate was cleaned and the adhesive was uniformly spread on the top surface of the laminate and the bottom surface of the beam. The beam was then kept over the laminate. To avoid peeling off of CFRC laminates, the ends of the laminates were secured by means of a wrap of CFRC sheet. The set up was left for 6 days for curing of epoxy. On the sixth day, the laminate was slowly released by operating the screw jack to transfer the load in laminate to the RCC beam.

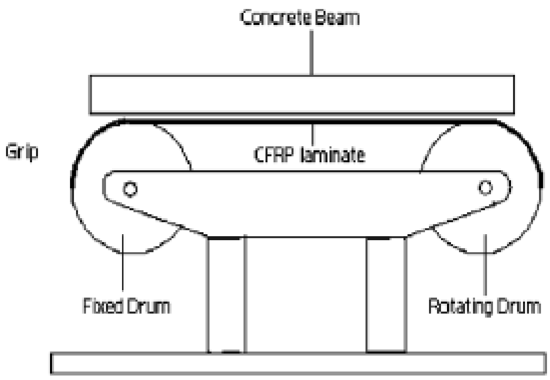


Fig 2.12: Special designed machine for pre-stressing of CFRP (Mukherjee & Rai 2009).

After the specimens were damaged and repaired by pre-stressing CFRC, the load-deflection curves were again plotted and their variation with the original specimen were compared as shown in Fig.2.12.

After testing they concluded that the fresh beam showed permanent deformation while retrofitting with pre-stressed fibres resulted in the recovery of deformation and the rehabilitated beam exceeded the load carrying capacity of fresh beam by a large margin.

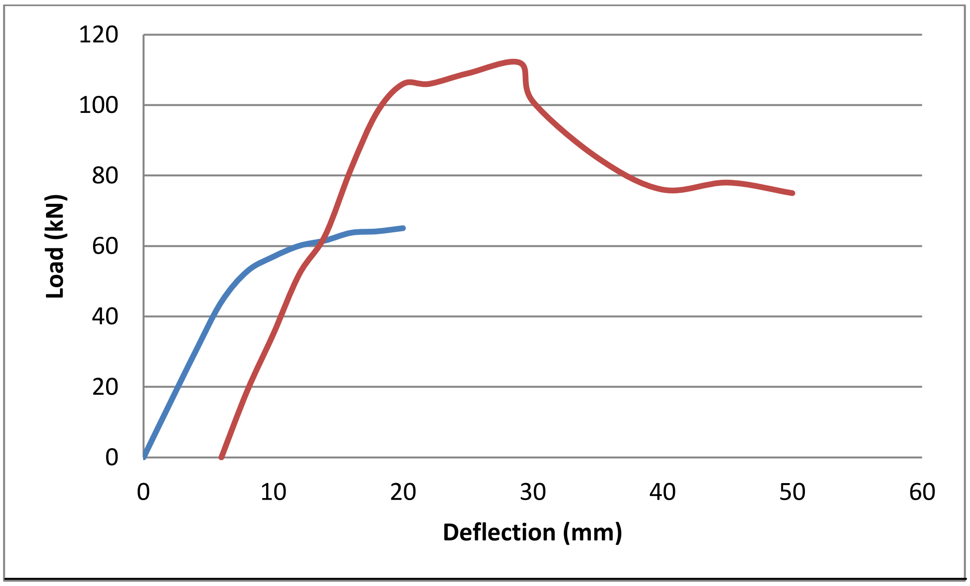


Fig 2.13: Comparison of Load-deflection graph by Mukherjee & Rai 2009.

Also the stiffness of rehabilitated beam was close to that of fresh beam and the maximum load was reached at a deflection beyond the maximum deflection of the fresh beam. The variation of pre-stress level of CFRP has also been done by them which has not been studied here.

Saidy et al (2010) experimentally studied the behaviour of retrofitted beams repaired with CFRP sheets, damaged at different levels of corrosion. The corrosion rate was varied with 5%, 10%,, and 15 %. For this 10 reinforced concrete rectangular beam specimens were tested. Six of the beam specimens were strengthened with CFRP sheets using three different strengthening schemes. The tensile reinforcement of six of the strengthened beams and three of the un-strengthened beams were corroded to various degrees by means of an impressed electrical current. To accelerate the corrosion process for the tension steel reinforcement, salt (NaCl) was added to the mixer before concrete casting so that 3% chloride ions, by weight of cement, were uniformly distributed along the bottom third of each specimen to be corroded. Following the corrosion phase, the beam specimens were tested to failure in flexure, in four-point bending. The reinforcement details of the specimens are shown in Fig. 2.13. Specimens C0, C5, C10 and C15 represent control specimens that were subjected to monotonic loading and not strengthened with CFRP sheets. The number describes the level of corrosion which ranges from 0% (no corrosion) to 15% Specimens M5S1 and M5S2 were subjected to 5%

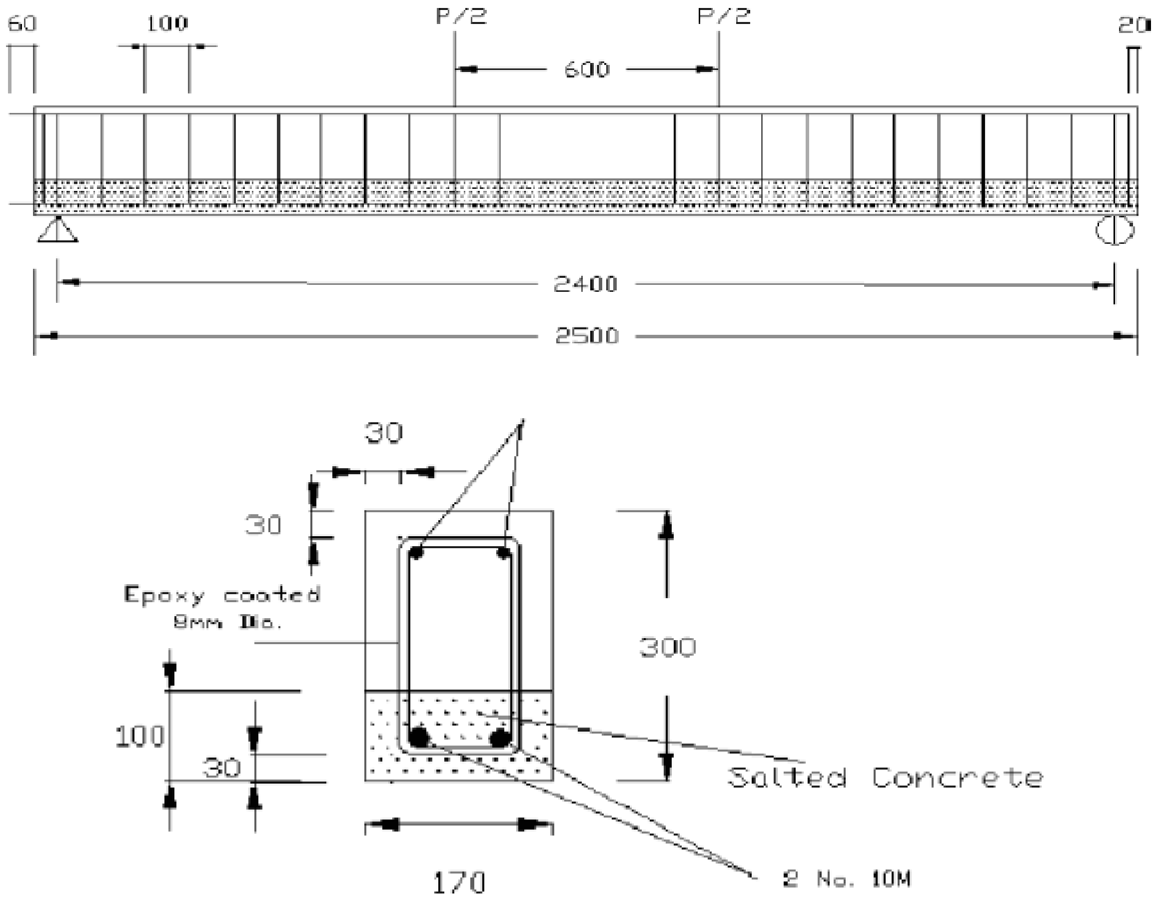


Fig 2.14: Reinforcement details of test specimen by Saidy et al (2010).

mass loss (corrosion) and strengthened with CFRP sheet using Scheme 1 (S1) and Scheme 2 (S2), respectively. All beams were strengthened with one CFRP layer except M15S2-2L was strengthened with two layers (2L) using Scheme 2 (S2). The definition of Schemes 1, 2 and 3 are described in Fig.2.11 a,b,c respectively.

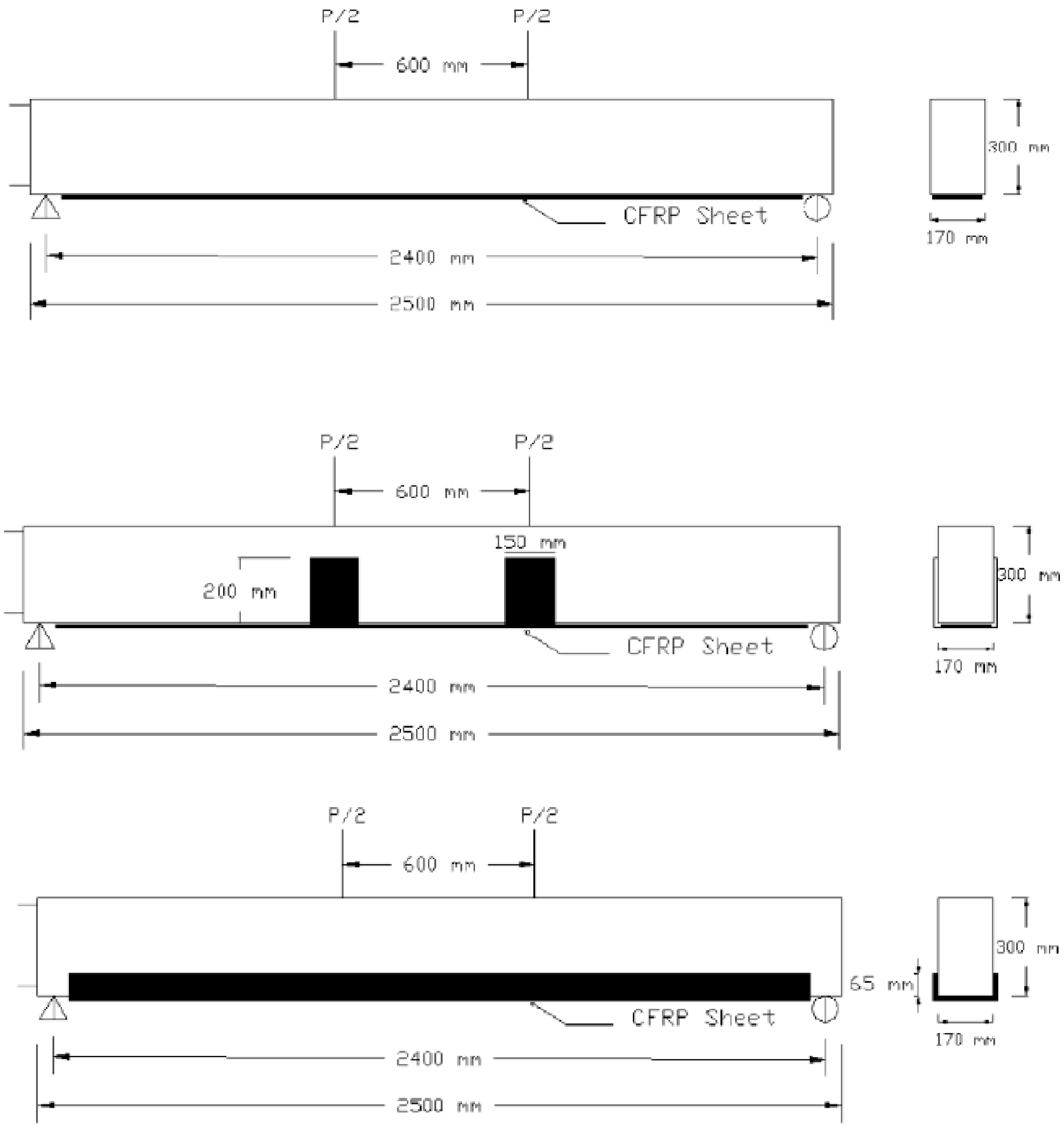


Fig. 2.15 a,b,c: Sceme1,2,3 respectively (Saidy et al (2010)).

Following the corrosion period, all specimens were loaded in four-point loading. The load was applied using a 250 kN hydraulic actuator through a spreader steel beam to the specimen. Each specimen spanned 2400 mm and was loaded symmetrically about its centre line at two points 600 mm apart. A linear variable displacement transducer (LVDT) with a range capacity of 100 mm was used to measure the span displacement of the test specimens. The load–mid-span deflection results for beam specimens are summarized in Table 2.3 in which P_c, P_y, and P_u are the cracking load, yielding load, and ultimate load, respectively; D_c, D_y, and D_u are cracking deflection, yielding deflection, and ultimate deflection, respectively.

Beam	**P_c**	**P_y**	**P_u**	**D_c**	**D_y**	**D_u**	**Failure mode**
C0	21	51	58	0.93	8.4	39.5	Flexural failure
C5	24	43.5	50	1.7	8	48.3	Flexural failure
C10	25	43.6	51.1	1.2	8.3	44	Steel rupture
C15	23.4	35.6	41.6	0.95	5.6	23.4	Steel rupture
M5SI	27.7	53.9	67	1	8.1	19.9	Debonding of CFRP
M5S2	26	54.1	71.8	1.5	9.2	24.5	CFRP rupture
M10S2	25	50.9	63.4	1.3	9.1	18.7	CFRP rupture
M15S2	28.1	43.8	56.9	1.5	7	17.2	Debonding of CFRP
M15S2-2L	25.9	52.1	68.5	1.3	7.8	15.6	Debonding of CFRP
M15S3	21.9	50.6	69.9	1.3	9.5	22	CFRP rupture

Table 2.3: Test results by Saidy et al (2010).

After analysing the results carefully, the author concluded that the use of CFRP sheets for strengthening corroded RC beams is capable of maintaining the structural integrity and increasing the ultimate strength of these beams to a level above the ultimate strength of the control beam. Also they concluded that the use of CFRP sheets for strengthening RC beams decreased the ultimate deflection of these beams to a level below the control beam as all strengthened specimens recovered the original strength except Specimen M15S2. Ultimate strength of Specimens M5S2 and M10S2 were 23.9% and 9.3%, respectively, greater than the ultimate strength of Specimen C0, while the ultimate strength of Specimen M15S2 was 2% less than the ultimate strength of Specimen C0 which was mainly due to premature debonding failure of Specimen M15S2. In addition, the ductility of all strengthened specimens was less than the control beam. Ultimate deflection of Specimens M5S2, M10S2 and M15S2 were, 38%, 52.8% and 56.6%, respectively, less than the ultimate deflection of Specimen C0.

CHAPTER 3 EXPERIMENTAL PROGRAMME

3.1 Introduction

A lot of research work has been carried on retrofitting of structural elements using Carbon Fibre Reinforced Plastics (CFRP) all across the globe with various successful results of using CFRP with single layered, double layered or with systems like Pre-stressing. The objective of this research is to study the effect of initial stress levels on beams retrofitted using pre-stressed carbon fibre sheets. The load-deflection curves of the retrofitted beams are then compared with that of the control beams.

3.2 Materials and Equipments

For the entire experimental programme the following materials and equipments are used.

3.2.1 Cement

Cement is a fine, grey powder. It is mixed with water and materials such as sand, gravel, and crushed stone to make concrete. The cement and water form a paste that binds the other materials together as the concrete hardens. Grade 53 Ultra Tech cement is used for casting beams. The cement was of uniform colour i.e. grey with a light greenish shade and was free from any hard lumps. The tests are conducted on the cement before its use and the results are shown in Table 3.1.

Sr. No.	Characteristics	Test Values	Value specified by IS :1489-1991 (Part 1)
1.	Standard Consistency (%)	32	-
2.	Fineness of cement as retained on 90 micron sieve (%)	0.7	10 (max)
3.	Setting time (minutes) 1.Initial 2.Final	 105 255	 30 (min) 600 (max)
4.	Specific gravity	3.10	-
5.	Compressive Strength (N/mm^2) 7 days 28 days	 27 39	 22 (min) 33 (max)

Table 3.1: Test results on Cement

3.2.2 *Aggregates*

Aggregates are the collection of items that are gathered together to form a total quantity. This includes sand, gravel, crushed stone, slag, or recycled crushed concrete. The aggregates generally used are of following two types:

3.2.2.1 Coarse Aggregates

The material which is retained on IS sieve no. 4.75 is termed as a coarse aggregate. The crushed stone is generally used as a coarse aggregate. The nature of work decides the maximum size of the coarse aggregate. The aggregates were washed to remove dust and dirt and were dried to surface dry condition. The aggregates were tested as per IS: 383-1970. The results of various tests conducted on coarse aggregate are given in Table 3.2 and Table 3.3 shows the sieve analysis results.

S.No.	**Characteristics**	**Value**	
		CA-I	CA-II
1.	Type	Crushed	Crushed
2.	Maximum Nominal Size (mm)	20	10
3.	Specific gravity	2.60	2.66
4.	Total water absorption (%)	1.90	1.81
5.	Fineness modulus	6.87	6.36

Table 3.2: Physical Properties of Coarse Aggregates

S.No.	Sieve No.	Mass retained (g)	Percentage retained (%)	Cumulative percentage retained,(C)	Percentage Passing, (100- C)
1	80 mm	0	0	0	100
2	40 mm	0	0	0	100
3	20 mm	0	0	0	100
4	10 mm	1225	40.83	40.83	59.17
5	4.75 mm	1624	54.13	94.96	5.04
6	Pan	151	5.04	Sum=135.79	-

Total weight taken = 3Kg

FM of 10 mm Coarse aggregate = (135.79+500)/100 = 6.36

Table 3.3: Sieve Analysis of Coarse Aggregates (10mm)

S.No.	Sieve No.	Mass retained (g)	Percentage mass retained	Cumulative percentage retained, C	Percentage Passing, (100 – C)
1	80 mm	0	0	0	100
2	40 mm	0	0	0	100
3	20 mm	0	0	0	100
4	10 mm	2648	88.26	88.26	11.74
5	4.75 mm	324	10.80	99.06	0.94
6	Pan	28	0.94	Sum=187.32	

Total weight taken = 3Kg

FM of 10 mm Coarse aggregate = (187.32+500)/100 = 6.87

Table 3.4: Sieve Analysis of Coarse Aggregates (20mm)

3.2.2.2 Fine Aggregates

The sand used for the experimental programme was locally procured and conformed to Indian Standard Specifications IS: 383-1970. The sand was first sieved through 4.75 mm sieve to remove any particles greater than 4.75 mm and then was washed to remove the dust. Properties of the fine aggregate used in the experimental work are tabulated in Table 3.5

Sr. No.	Characteristics	Value
1.	Specific gravity	2.46
2.	Bulk density	1.4
3.	Fineness modulus	2.56
4.	Water absorption	0.85
5.	Grading Zone (Based on percentage passing 600 µm sieve)	Zone III

Table 3.5: Physical Properties of fine aggregates

The aggregates were sieved through a set of sieves to obtain sieve analysis and the same is presented in Table 3.6. The fine aggregated belonged to grading zone III.

Sr. No.	Sieve Size	Mass retained	Percentage Retained	Cumulative Percentage Retained	Percent Passing
1	4.75mm	4.0	0.4	0.4	99.6
2	2.36 mm	75.0	7.50	7.90	92.1
3	1.18 mm	178.0	17.8	25.70	74.3
4	600µm	220.0	22.0	47.70	52.3
5	300µm	274.0	27.4	75.10	24.9
6	150µm	246.5	24.65	99.75	0.25
7	2.50	0.25	0.25	∑=256.55	

Table 3.6: Sieve analysis of fine aggregate

Total weight taken = 1000gm

Fineness Modulus of sand = 2.56

3.2.3 Water

Generally, water that is suitable for drinking is satisfactory for use in concrete. Water from lakes and streams that contain marine life also usually is suitable. When water is obtained from sources mentioned above, no sampling is necessary. When it is suspected that water may contain sewage, mine water, or wastes from industrial plants or canneries, it should not be used in concrete unless tests indicate that it is satisfactory. Water from such sources should be avoided since the quality of the water could change due to low water or by intermittent discharge of harmful wastes into the stream. In the present experimental programme, potable tap water is used for casting.

3.2.4 Reinforcing Steel

HYSD steel of grade Fe-500 Tata Tiscon steel of 12mm and 8mm diameters are used as longitudinal steel. 12mm dia bars are used as tension reinforcement and 8mm bars are used as compression steel and steel of 8mm diameter bars are used as shear stirrups.

3.2.5 Concrete Mix

Concrete mix is prepared using 53 grade Portland pozzolana cement, fine aggregate (medium-sized natural river sand) and crushed stone coarse aggregate with nominal size of 20 mm. The mix is taken as a nominal mix of M20 and as per Indian Standard Guidelines, the ratio of cement: sand: aggregate taken is 1:1.5:3. The water-cement ratio is 0.5 and compressive strength of concrete after 28 days is 29 MPa.

3.2.6 CFRP Material

Unidirectional CFRP sheets 300 mm wide and 0.117mm thickness are used for the retrofitting. The CFRP sheets are obtained from BASF construction chemicals and building systems (shown in Fig 3.3). Under stress, fibre utilizes the plastic flow of matrix to transfer the load to the fibre which results in high strength and high modulus composite. Properties of the fibre used are given in Table 3.7. Main function of matrix is to combine and to protect the fibre against external environment into which the composite will be placed.

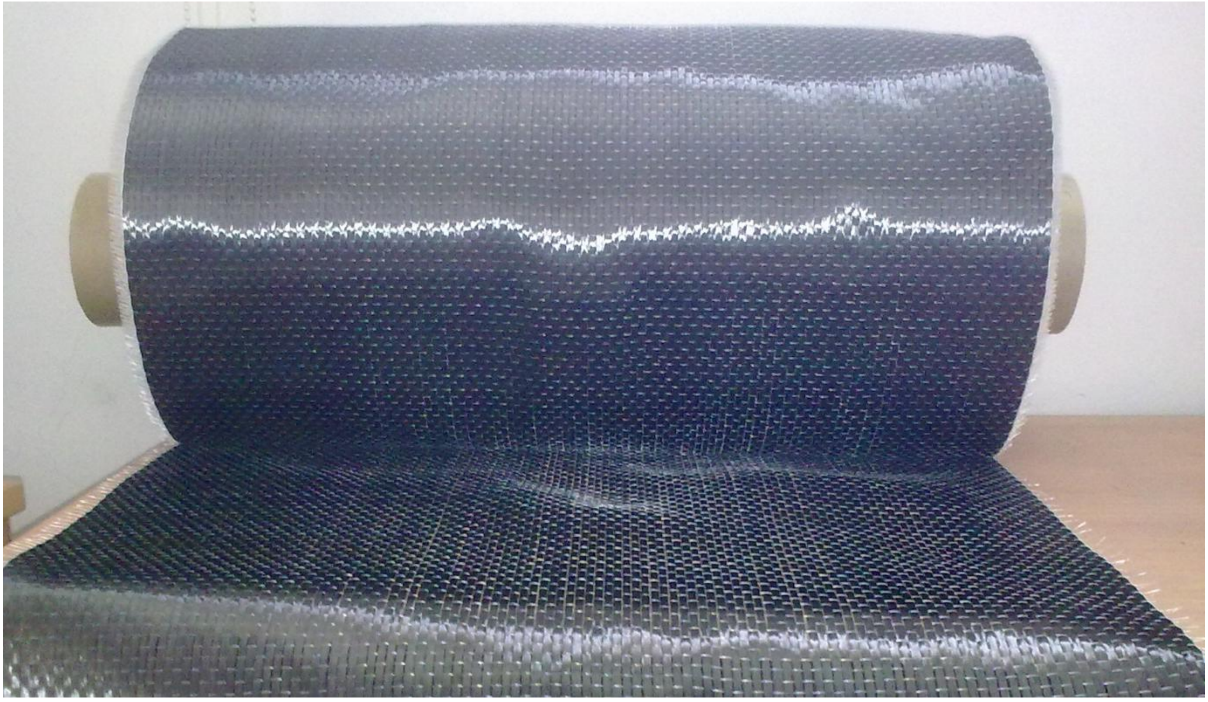


Fig 3.1: CFRP sheet used for retrofitting

Sr. No.	Physical properties	Value
1	Tensile Strength	3800 MPa
2	Modulus of elasticity	240 GPa
3	Density	1.7g/cm^3
4	Weight of Carbon fibre before stiching	200gsm
5	Weight of CFRP after stiching	230gsm
6	Thickness	0.117mm

Table 3.7: Properties of fibre used for retrofitting

3.2.7 Data Acquisition Servo Control System

The machine used for applying load to the beam is WAW-E series Electro-hydraulic servo universal Oil-source control system which is mainly used for servo control of single-cylinder testing machine. The system also equips with another set of oil outlet. It can be connected to two sets of hydraulic cylinders but at the same time it can control one cylinder. Measurement and control system is a universal one, so it can be used for control device for testing machines such as Test cylinder, Compression testing machine and Universal testing machine. The main technical specifications are under:

1. Oil source rated pressure: 26 MPa
2. Oil source rated flow: 4L/min
3. Total power: 3 kW
4. Oil tank capacity: 80 liters
5. Maximum displacement measurement: 1000mm
6. Maximum deformation measurement: 50mm

The structure and working principle is composed of following systems:

- *Oil source system*

It includes control cabinet box, control valves, oil source and power electricity boards as shown in Fig.1. Motor driven high pressure pump produces high pressure oil. Through the pupm outlet, it is divided into two circuits, manual control and servo control circuit. The circuit through which the work has carried on is the manual control circuit.

- *Measuring System*

It is for test data collection and output and data display. It further consists of load measuring system, deformation measuring system and displacement measuring system.

- *Computer System*

The system is computer-centered, synchronous acquisition of load, deformation and displacement measuring system`s output signal realised by the computer and also is stored for data processing, meanwhile testing data displayed by the computer.

3.2.8 LVDT

The linear variable differential transformer (LVDT) is a type of electrical transformer used for measuring linear displacement. The transformer has three solenoid coils placed end-to-end around a tube. The centre coil is the primary, and the two outer coils are the secondary. A cylindrical ferromagnetic core, attached to the object whose position is to be measured, slides along the axis of the tube. The transducer which is used to calculate the deflections is of the range of 50mm and has the least count of 0.01mm shown in Fig 3.2. It is connected to the adapter box through wire and touched to the positions where the deflection is to be found out.

Fig 3.1: Data Acquisition Servo Control System

Fig 3.2: LVDT used for measuring deflection

3.2.9 Adhesives

The adhesive used for bonding FRP sheets with concrete is a compatible epoxy system provided by the manufacturer. It is blue pigmented epoxy resin for saturation of MBrace fibre sheet to form in-situ FRP Composite. It is made by mixing base saturant and hardner in ratio 100:40. Mixing of saturant and hardner is done thoroughly for five minutes until components are thoroughly dispersed. The properties of the adhesive saturant are shown in Table 3.8.

3.2.10 Roller

A special type of roller for impregnating the adhesive substance to the fibre and to the beam is used (shown in Fig.3.4). The roller is provided by the manufacturer.

Aspect	Translucent blue liquid
Volume solids	100%
Mix density	1.13±0.03
Mixing ratio (by weight)	100 to 40
Mixed viscosity (cps at 25°C)	4000 ± 500
Pot life (in minutes)	25 min. at 25°C
Setting time	<3 hrs. at 25°C
Full cure	7 days at 18°C
Compressive strength	>40MPa at 1 day >60MPa at 7 days
Tensile strength	>17MPa
Flexure strength	>35MPa
Density	0.8 to 1.0 kg/m^2

Table3.8: Properties of Mbrace Saturant provided by manufacturer

Fig 3.4: Roller for proper adhesion of saturant to fibre

3.2.11 Pre-stressing Machine

It is used to pre-stress the fibre sheet. Hydraulic jack is fitted in the machine with which force is applied and pre-stressing of fibre is done. On both the sides of this machine dial gauges are fixed to measure the elongation in fibre after applying the force. Front part of the machine iss curved and moveable as shown in Fig 3.5.

Fig 3.5: Pre-stressing machine

3.2.12 Anchor Bolts

12 mm diameter Anchor bolts are used to fasten the plates of pre-stressing machines and plate. This bolt consists of two parts, bolt and rod.

3.2.13 Hydraulic Jack

Hydraulic jack was used to apply the load. Range of Hydraulic jack is 200 KN.

3.2.14 Dial Gauges

Dial gauges were used to measure the elongation in the fibre. These gauges are located on both sides of the pre-stressing machine. As the movable arm proceeds to elongate the fibre, it calculates the elongation shown in Fig 3.6. .Least count of dial gauge was .01 mm and range of 5.0 cm.

Fig3.6: Dial gauge used for measuring the elongation

3.4 Design of beam

The beam is designed to behave as under-reinforced section with M20 grade concrete and Fe500 steel of cross dimensions 13`6``×2`×1` (4100×600×300 mm). The reinforcement to be provided is 4 bars of 8 mm diameter in the compression zone and 4 bars of 12 mm diameter in the tension zone. Stirrups are also provided as 4-legged stirrups with 75mm spacing centre to centre. The detailed structural detailing of the beam is shown in Fig 3.7.

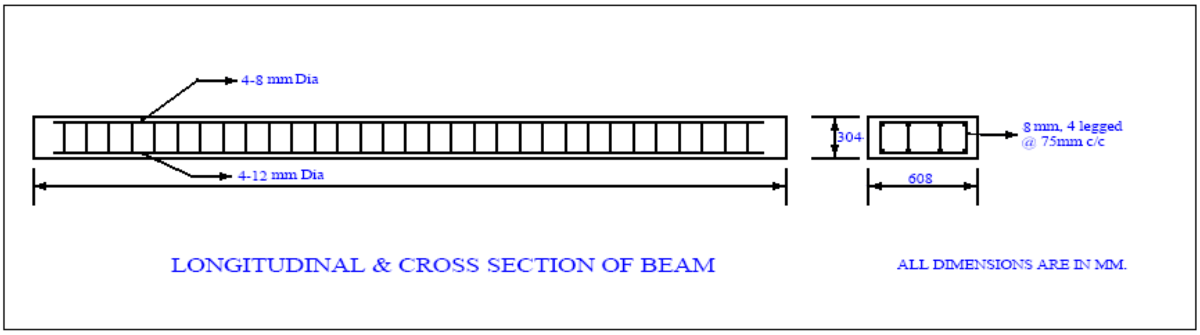


Fig 3.7: Structural detailing of the specimens

3.5 Casting of beams

As per the design, the beams are casted. The shuttering steel plates are joined together on the levelled ground through nut bolts to form the formwork to the required dimensions as shown in Fig 3.8. Proper oiling of the plates is done to ensure the proper debonding of the plates at their removal after curing.

Fig 3.8: Formation of formwork by joining shuttering plates

The steel mesh is formed by proper cutting and winding as shown in Fig 3.9.

Fig 3.9: Steel mesh formed by cutting and winding of steel

4 bars of 12mm diameter are kept at bottom and 4 bars of 8mm diameter are kept at top and proper stirrups are made 4-legged of 8mm diameter with spacing of 75mm c/c. The steel mesh so formed is kept inside the formwork with proper cover blocks of 25mm below the mesh. The concrete is made with proper design mix of 1:1.5:3. Vibrators are also used properly to ensure the proper compaction as shown in Fig 3.10.

Fig 3.10: Use of vibrators for proper compaction

After the beams have been moulded in the formwork they are left for 48 hours and the plates are removed. Proper curing is done with the jute bags for 28 days.

3.6 Testing Arrangement

To implement the desired objectives, eight beams are casted. Out of these eight beams, two beams are taken as Control Beams whose load-deflection curves are drawn and from average of these curves three corresponding stress levels are chosen such that first is from elastic zone, second is from elasto-plastic zone and third is from plastic zone. Two beams each from each stress level are retrofitted using special pre-stressed carbon fibre wrap which are pasted to the damaged beams using adhesives and their load-deflection graphs are then compared with that of the control beams. The test matrix of the experimental programme is shown in Table 3.9.

Variables	Beams	Number of beams
Control Beam 01	CB01	2
	CB02	
Initial Stress level 1	DB11	2
	DB12	
Initial Stress level 2	DB21	2
	DB22	
Initial Stress level 3	DB31	2
	DB32	
Total number of beams		8

Table 3.9: Test matrix of the experimental programme.

The beams after curing are tested on the two-point loading setup on the simply supported arrangement as shown in Fig 3.10. LVDT`s are placed beneath the beam at centre and at quarter of the span. The load is applied on the beam through hydraulic jack over it as shown in Fig 3.11.

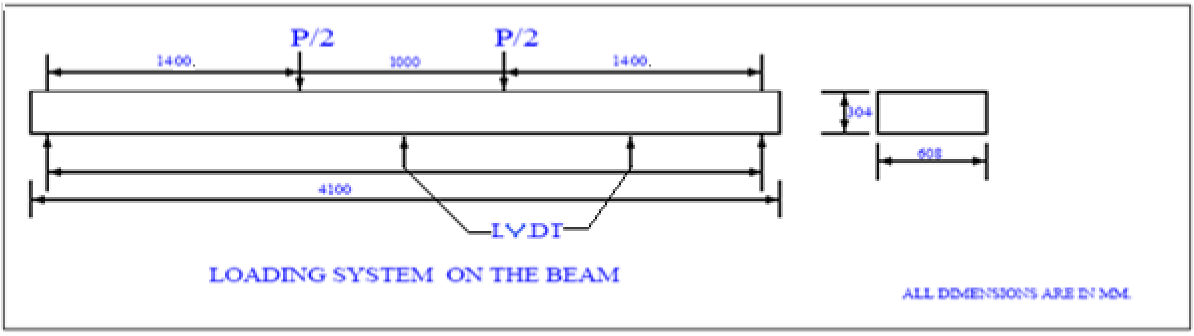


Fig 3.11: Two-point loading testing arrangement.

Fig 3.12: Two-point loading arrangement used for testing.

The LVDT`s are connected to the servo controller machine and the software directly gives the load-deflection curves of both the locations.

3.7 Retrofitting Using Pre-stressed CFRP

After the beams are stressed to the respective stress levels, they are put upside down for the retrofitting. This is because the fibre sheets have very high tensile strength and so they are incorporated to the bottom surface of the beam. Prior to that preparation of the surface is done. This is a two-step process which includes:

- *Grinding*

  Beam was grinded with the help of grinder having Diamond Plates. Purpose of grinding was to expose the aggregates so as to have better adhesion of the aggregates with the fiber.
- *Anchoring Bolts*

  The pre-stressing machine setup consists of an arrangement which is to be fixed properly to the soffit of the beam. It consists of three parts:
  1. Machine fixed to the beam.
  2. Movable arm connected to the machine.
  3. Plate which is to be anchored on the beam on the other side of the beam where machine is fixed.

For this, holes are drilled in the beam and fasteners of 12mm size are anchored inside by hammering its rod shown in Fig 3.13 on both the sides of the beam. The machine and the plate are anchored to it which is tightened through nuts.

Fig 3.13: Drilling of holes for fastners for anchoring the machine and plate on both sides of the beam.

Two layers of CFRP sheets are laid on the beam and the Pre-stressing machine is anchored on one side of the beam over the sheets. The sheets are taken to the front movable portion of the machine, pasted to it through epoxy made in the proportion 100 (Base): 40 (Hardener). The movable part of the machine is further fixed by another plate below the FRP sheets, which are fastened together with bolts to make proper grip of sheet so that uniform force should be applied shown in Fig 3.14.

Fig 3.14: Fibre fixed on the movable part of the machine by an extra plate attached to it.

On the other side of the beam, plate is fixed over the sheets pasted on the beam with adhesive material, anchored to the drilled fasteners and are tightened with the nut bolt system as shown in Fig 3.15.

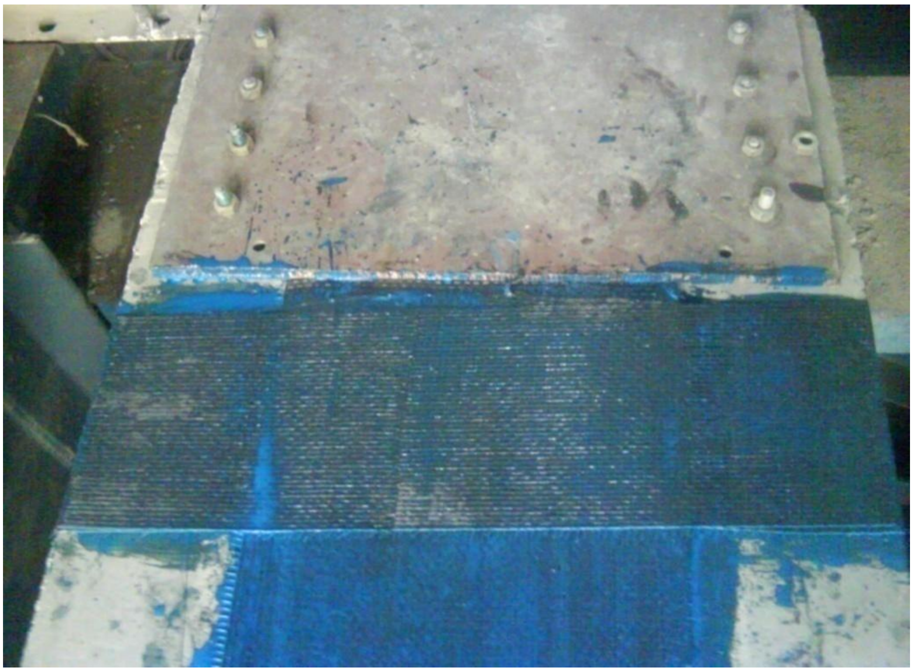


Fig 3.15: Plate anchored to one side of the beam over the CFRP sheets

After fixing the fiber it is left for curing for one day. Only fibers with double layers of length equal to plate are fixed to the beam with adhesive and rest of the fiber is not pasted. On the other end of the beam Hydraulic jack is fitted inside the fixed portion of the machine. Load is applied through the jack on the movable portion of the machine. The sheets fixed to this, starts elongating which is measured from the dial gauges attached to it on both sides (shown in Fig 3.16 and this is done till the elongation reached to 50mm.

.

Fig 3.16: Dial gauges attached to both sides of the machine for measuring the elongation in fibre.

Fiber sheets are elongated upto 50 mm and the pre-stressing load corresponding to this elongation can be calculated as follows:

Stress can be defined as:

$$Stress = \frac{P\ (load)}{\text{Area}}$$

Similarly, Strain can be found out by:

$$Strain = \frac{\Delta l}{l}$$

Therefore, the value of load, which has been applied for the elongation of 50mm, can be calculated as:

$$E = \frac{\text{P} \times \text{l}}{A \times \Delta}$$

Now,

E (Elastic modulus of CFRP) = 240×10^3 N/mm^2

A (Area of fibre under stress) = b×t = 300×(0.117+0.117) = $70.2mm^2$

Δ (elongation) = 50mm

L (length of fibre under pre-stressing) = 2700mm

Therefore,

$$P = 312kN$$

The sheets are then pasted to the remaining portion of the beam. Adhesive used is Mbrace Saturant which is mixed with the ratio of 100(Base):40(Hardener). After mixing base and hardener a translucent blue liquid has formed known as epoxy which is applied on the beam. The epoxy is applied on the sheets and is rolled properly with the roller till it starts coming out. The corners of the beams are wrapped with the fiber so that CFRP sheet should not stuck out. It is left for curing for 48 hours and after that the jack is released and plats are removed. The retrofitted beam is now ready for testing to check its strength. The beams are tested in the same way as the two control beams are tested and their load-deflection curves are plotted which are then compared with the control beams.

CHAPTER 4 RESULTS AND DISCUSSION

4.1 Introduction

In the present work the retrofitting of beams using pre-stressed Carbon Fibre Reinforced Plastics is done which are already stressed at different stress levels. As per the test matrix, the load-deflection graphs of the two control specimens are plotted and their average graph is adopted as reference labelled as RC. The stress level on which the other beams are stressed, are chosen from the load-deflection curves of the average of the two control beams (RC). The stressed beams are then retrofitted and their load-deflection curves are compared with that of the RC. Also, the effect of retrofitting is studied on the various initial stress levels.

4.2 Results of Control Beam

4.2.1 Introduction

The beams are tested on the testing arrangement as discussed in Section 3.6. Load is applied through the hydraulic jack and deflections in the beam are checked. With the increasing load and the deflections, load-deflection curves are plotted by the software. The beams are tested with the results shown under.

4.2.2 Control Beam 01 (CB01)

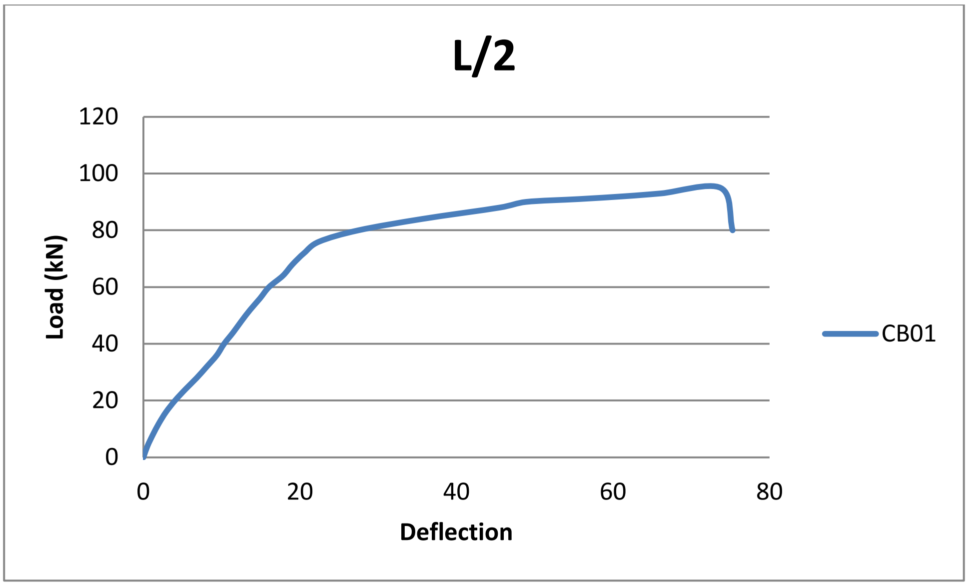
First control beam (CB01) is tested. As it is designed as an under-reinforced section, it is estimated that steel would get yield before the failure. The load-deflection data is given in Table 4.1 and corresponding to it the load-deflection curves are shown in Fig 4.1.

The graph shows that the beam deflects maximum at the centre. The maximum load carrying capacity of the beam is found to be 94.9kN and at this load the deflection at centre is found to be 73.7mm. Also, at this load, the deflection at quarter of the span (L/4) is found to be 31.3mm. As seen from the curve, after 78kN of load, yielding has started till the failure. This is because the beam is designed as under-reinforced section which accounts to the yielding zone.

The damage in the beam started with bending cracks in the central region of the beam. First crack is seen near to the left point load at 60kN. When the load reaches 70kN, Second crack is seen just inside the left point load and near to first crack. With the increasing load, third & fourth cracks are developed between centre point of beam and right point load. No shear cracks are observed. Major cracking has been observed at 88kN. Beam stopped taking load at 94.9kN. Almost all cracks are vertical near the bottom edge, sub cracks are developed connecting to main crack as shown in Fig.4.2. The cracking pattern shows that the failure in beam is pure flexural failure as all the cracks are seen near the bottom edge.

Load (kN)	Deflection at L/2 (mm)	Deflection at L/4 (mm)
0	0	0
4	0.52	0.1
8	1.21	0.51
12	1.98	0.84
16	2.89	1.33
20	4.03	2.01
24	5.37	2.87
28	6.81	4.09
32	8.11	5.23
36	9.38	6.12
40	10.31	6.93
44	11.46	8
48	12.52	9.18
52	13.64	9.94
56	14.89	10.59
60	16.05	11.61
64	17.82	12.6
68	19.05	13.63
72	20.54	14.7
76	22.45	15.37
80	27.47	16.84
84	35.62	18.81
88	45.51	21.41
90	48.89	22.89
91	55.63	24.27
92	61.53	26.12
93	66.16	27.98
94.9	73.77	31.29
80	75.27	33.15

Table 4.1: Load-deflection data for CB01



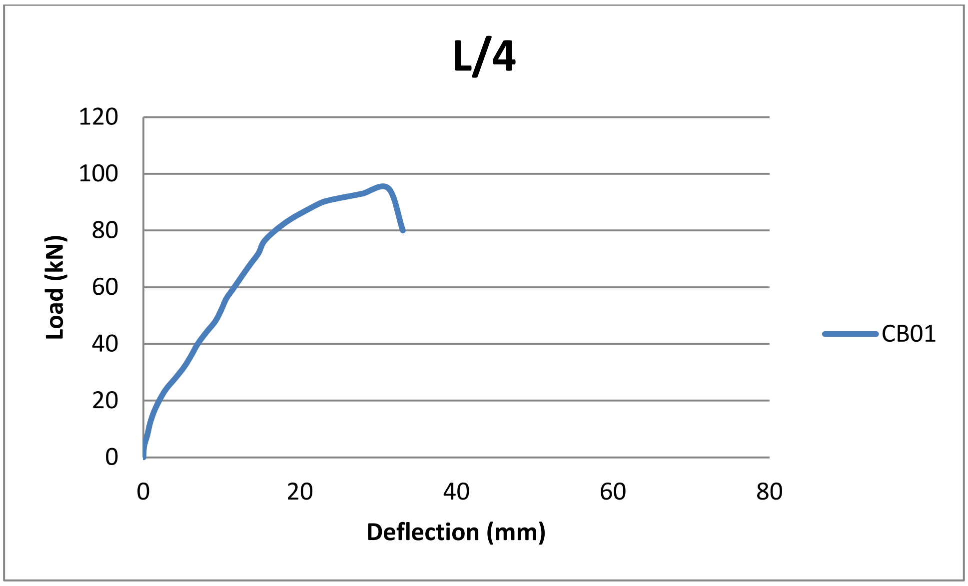


Fig 4.1: Load-deflection curves of CB01 a) at centre and b) at quarter of the span

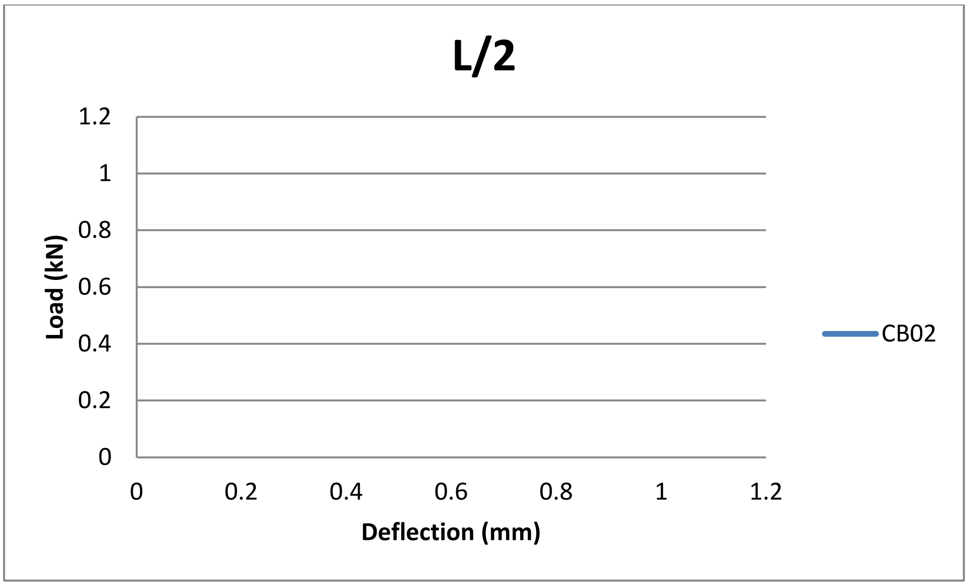
Fig 4.2: Crack pattern observed after CB01 is failed

4.2.3 Control Beam 02 (CB02)

Another control specimen (CB02) is tested on the same grounds as CB01 is tested. Similar procedure is followed. The load-deflection data is shown in Table 4.2 and the load-deflection curves so formed are shown in Fig.4.3.

Load (kN)	Deflection at L/2 (mm)	Deflection at L/4 (mm)
0	0	0
4	0.61	0.23
8	1.34	0.62
12	2.07	1.21
16	3.53	2.09
20	4.83	2.97
24	6.21	3.93
28	7.98	5.36
32	9.62	6.24
36	10.84	6.94
40	12.08	8.18
44	13.21	9.03
48	14.05	9.69
52	14.94	10.56
56	16.41	11.37
60	17.72	11.68
64	19.13	12.61
68	20.92	13.58
72	22.13	15.81
76	27.26	17.76
80	33.11	20.37
84	38.48	22.28
88	47.96	24.51
90	52.53	26.43
91	58.51	28.73
92	62.86	30.13
93.8	69.85	33.15
85	72.13	33.61

Table 4.2: Load-deflection data for CB02



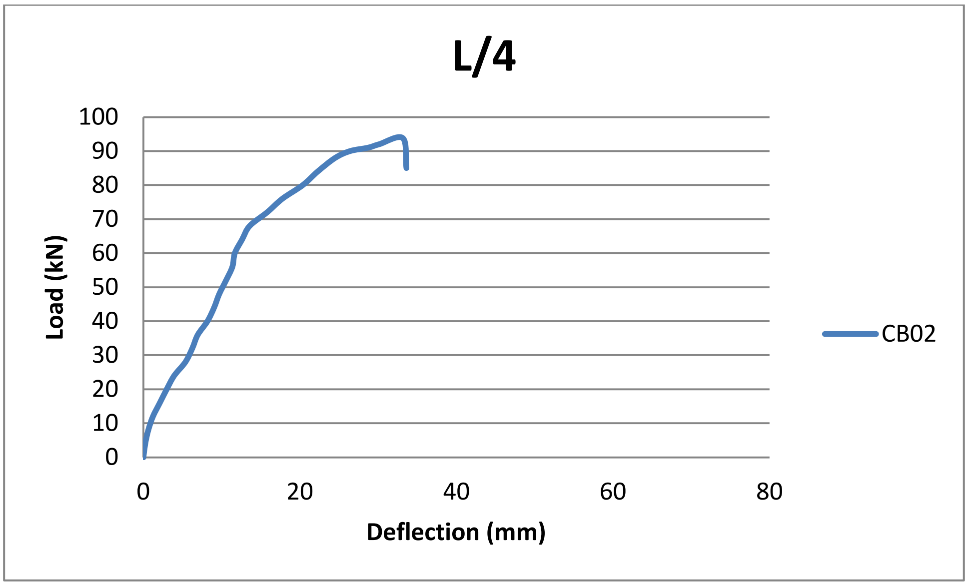


Fig 4.3: Load-deflection curves of CB02 a) at centre and b) at quarter of the span.

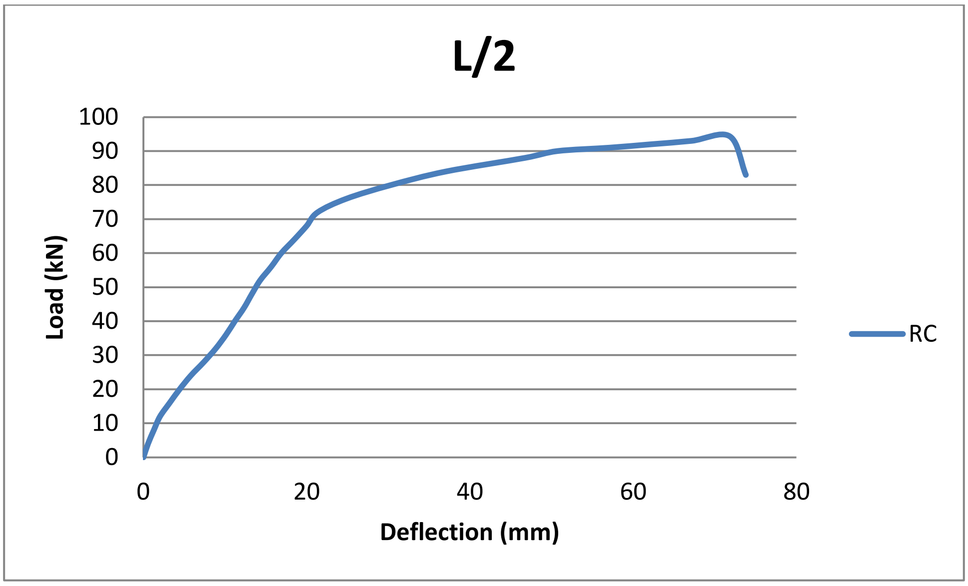
It can be seen that the beam deflects maximum at the centre. The maximum load carrying capacity of the beam is found to be 93.8kN and at this load the beam deflects maximum at centre with deflection of 69.85mm. Also, at this load, the deflection at quarter of the span (L/4) is found to be 33.15mm. After 74kN of load, the curve shows the yielding pattern as discussed in section 4.2.2. The damage in the beam started with bending cracks in the centre right region of the beam around 600mm from the centre shown in Fig.4.4. First crack is observed at 51kN. As load reaches 70kN more cracks are observed. Second crack is seen just inside the right point load and near to first crack. Third crack is seen towards the left point load. Forth is seen near the second crack. Fifth and sixth cracks are seen around the centre of

Fig 4.4: Crack pattern observed after CB02 is failed

the beam. No shear cracks are observed. Major cracking has been observed at 86kN. Beam stopped taking load at 93.8kN. Almost all cracks are vertical near top and bottom edge, sub cracks are developed connecting to main crack.

4.2.4 Reference Control Specimen (RC)

After the two control beams are tested to the failure and corresponding graphs are drawn, from the results of the deflection of both the beams, their average values are taken and graphs are plotted as load versus average deflection of both the beams of the same location that is at centre as well as at quarter of the span. This is done because this average graph would be chosen as reference as the control specimen (RC) and these results are then further compared to the retrofitted samples. Also, the damage levels at which other beams are to be damaged, are also chosen from this reference graph. The average Load-deflection curves at L/2 of the two control beams, taken as the control specimen for further reference is shown in Fig 4.5.



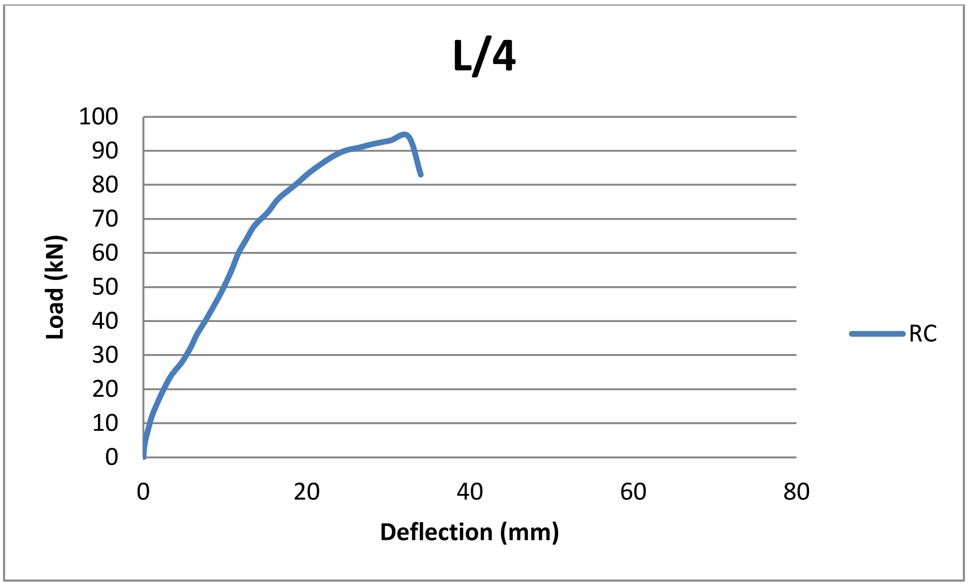


Fig 4.5: Load-deflection graph of RC: a) at centre and b) at L/4 of the span

The graph shows that up to 75kN, the beam is under elastic limit. After that the steel has started yielding which is seen through gradual increase of deflection which is due to its good ductility which comes under the elasto-plastic zone. The curve further shifts to the plastic zone at 91kN. After that the beam has stopped taking further load and it failed at the failure load of 94.35kN. The stiffness is also checked from the elastic limit of the graph and is calculated to be 3.91kN/mm. Also, from this reference graph, the initial stress levels are to be decided. As the maximum deflection is seen at L/2, so the stress levels are chosen from load-deflection curve at L/2 shown in Fig.4.5. The three stress levels so chosen are such that first is taken from elastic zone as 50kN, second from elasto-plastic zone as 84kN and the third is taken from plastic zone as 93kN. Their designations are shown in Table 4.3.

Variables	Damage level	Beams	Average
Control Beam 01	Fully damaged	CB01	RC
	Fully damaged	CB02	
Initial stress level 1	50kN (53%)	DB11	RDB1
	50kN (53%)	DB12	
Initial stress level 2	84kN (89%)	DB21	RDB2
	84kN (89%)	DB22	
Initial stress level 3	93kN (98%)	DB31	RDB3
	93kN (98%)	DB32	

Table 4.3: Designations of beams with their respective damage levels

As per the curve, the three loads are chosen as 50kN, 84kN and 93kN from the three respective zones. Of these three chosen loads, two beams each are damaged. The damage levels are calculated in terms of the percentage with respect to the failure load. The designations of beams with their respective damage levels are shown in Table 4.3. For accurate and reference, of each damage level, the average results of both the beams are calculated which are also labelled in the Table 4.3.

4.3 Results of Retrofitted Beams

4.3.1 Introduction

After deciding the initial stress levels, the beams are stressed to the decided levels and are then retrofitted using the Pre-stressed double layered Carbon Fibre wrap as discussed in section 3.7. The retrofitted beams, after curing for 72 hours as specified in the experimental programme, are tested again and damaged to their failures with the results discussed below.

4.3.2 Initial Stress level 1

With reference to the Table 4.1 initial stress level 1 corresponds to the retrofitting of beams which are already stressed at 53% of failure load. This level has two beams. Both the beams are tested under the same testing arrangement on which control specimens are tested.

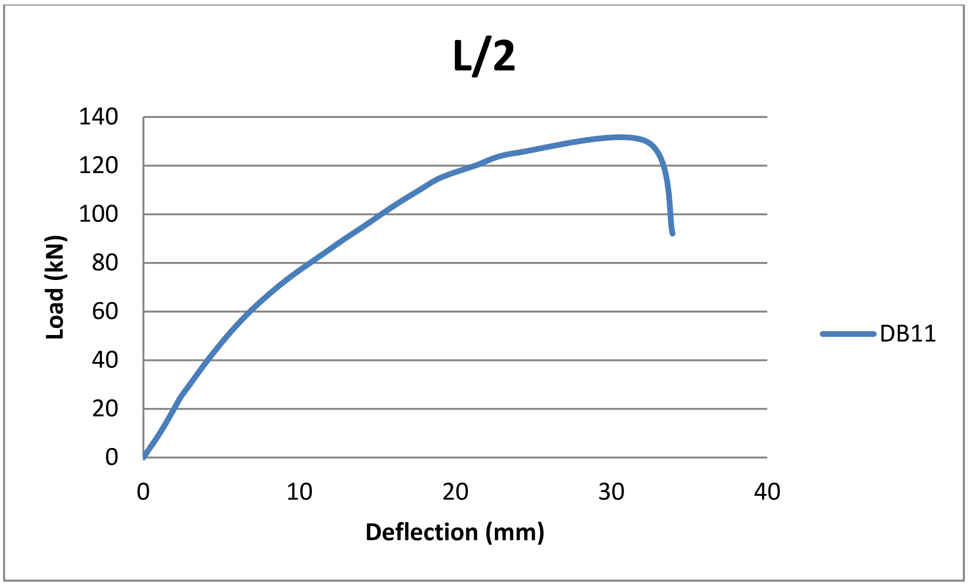
4.3.2.1 DB11

The beams after retrofitting are tested and the load-deflection data at L/2 and at L/4 are shown in Table 4.4 and corresponding to it load-deflection graphs are shown in Fig 4.6.

Load (kN)	Deflection at L/2 (mm)	Deflection at L/4 (mm)
0	0	0
5	0.53	0.36
10	1.052	0.71
15	1.522	1.01
20	1.962	1.3
25	2.408	1.64
30	2.98	1.99
35	3.538	2.4
40	4.118	2.81
45	4.74	3.24
50	5.384	3.68
55	6.078	4.15
60	6.836	4.69
65	7.678	5.31
70	8.59	5.96
75	9.588	6.66
80	10.696	7.4
85	11.824	8.13
90	12.938	8.91
95	14.13	9.73
100	15.264	10.5
105	16.446	11.29
110	17.724	12.09
115	19.066	13.02

120	21.306	14.43
125	23.74	16.69
129.82	32.272	19.89
95	33.93	21.53

Table 4.4: Load-deflection data for DB11



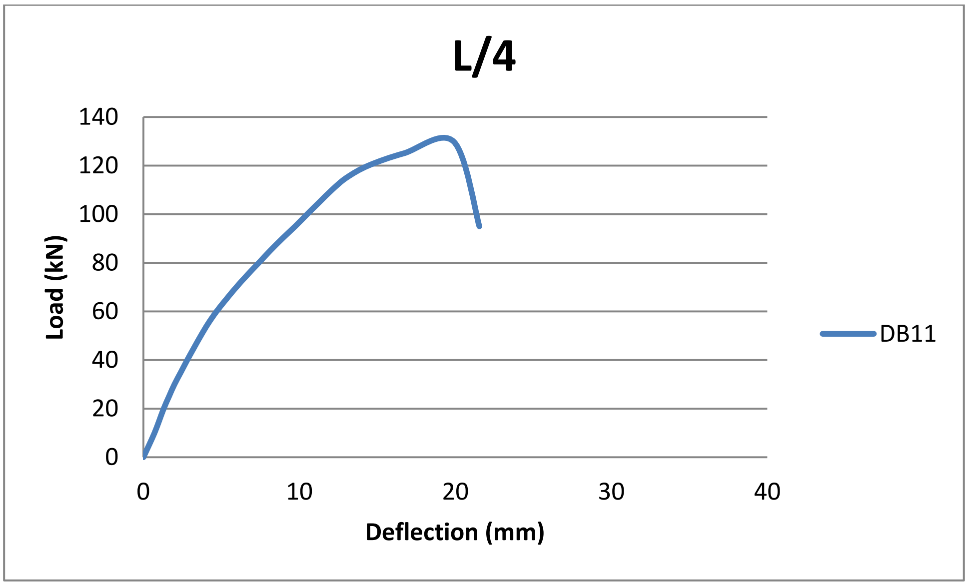


Fig 4.6: Load-deflection curves of DB11 a) at centre and b) at quarter of the span

During testing it is observed that during initial stages not much of the cracks are observed. Initial cracks are observed at 100kN of load towards the right half of the beam. After that more intense cracks are observed with the increasing load. Cracking sound is heard at 110 KN and at this point concrete start spalling out. The cracking pattern is shown in Fig 4.7. Sheet started debonding at 120kN. Loud cracking sound is heard at 129.82kN and the beam is failed due to debonding of the sheets as shown in Fig 4.8.

The maximum deflection of the beam is seen to be 32.37mm which is very less than that of the control beam. The retrofitted beam showed significant increase in load carrying capacity. Also the crack pattern of the beam is quite different to those of control specimens. They are seen to be more shifted towards the compression face of the beam. That is why the cracks are extended towards the upper edge of the beam.

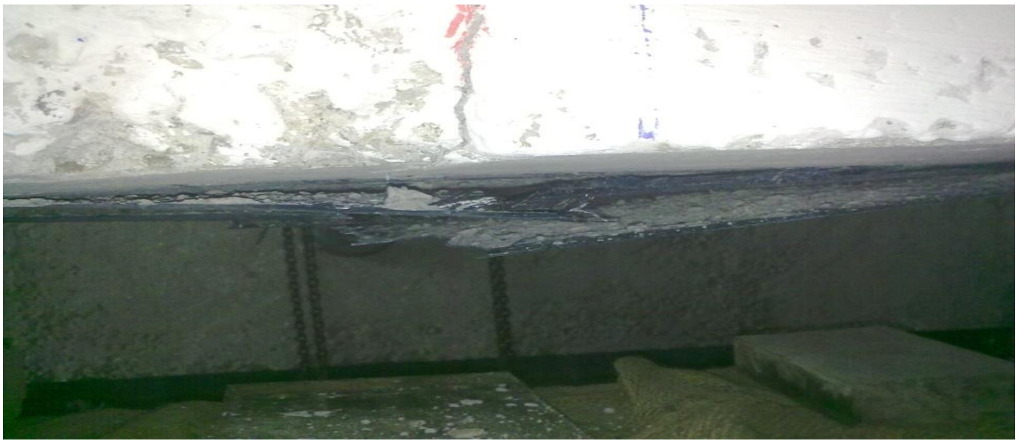


Fig 4.7: Failure of DB11 by debonding of sheets

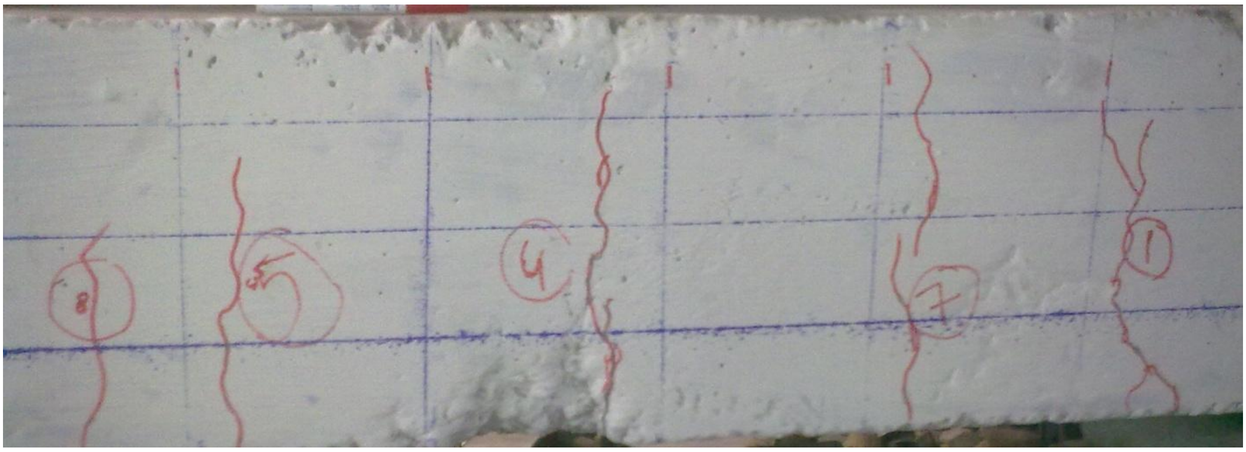


Fig 4.8: Cracking pattern of DB11

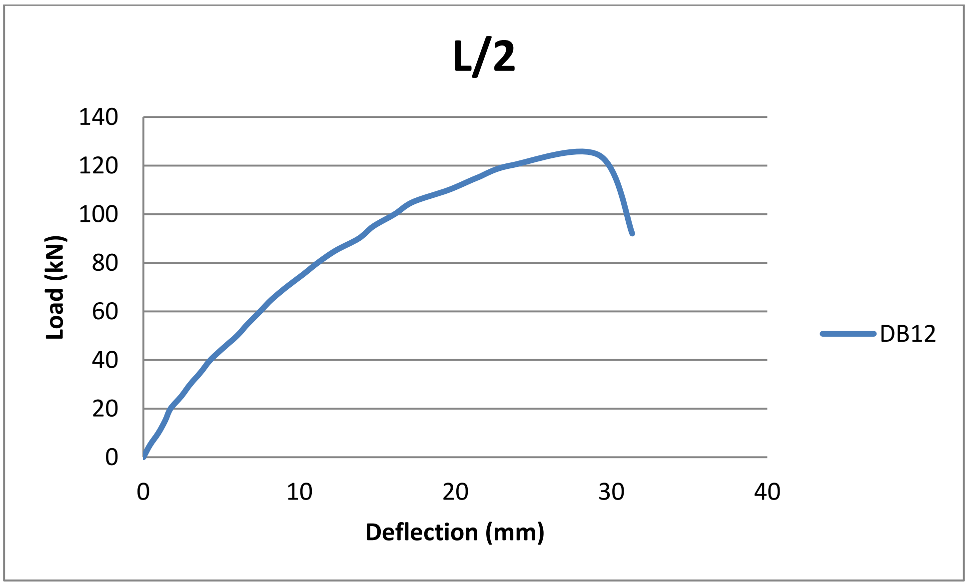
4.3.2.2 DB12

Another stressed beam at the same level is retrofitted and tested under arrangement discussed in section 3.6. The load-deflection data is shown in Table 4.5 and coresponding to it the load-deflection curves are shown in Fig 4.9.

Quite similar results are observed. Initial cracks are observed at 95kN of load towards the right half of the beam. More cracks are observed after the first crack with the increasing load. Second and third cracks are seen at 105kN and 108kN. Cracking sound is heard at 110kN and at this point concrete start spalling out. The cracking pattern is shown in Fig 4.11. Sheet started debonding at 118kN. Loud cracking sound is heard at 124.23kN and the beam is failed due to debonding of the sheets as shown in Fig 4.10.

Load (kN)	Deflection at L/2 (mm)	Deflection at L/4 (mm)
0	0	0
5	0.42	0.25
10	0.96	0.618
15	1.39	0.938
20	1.74	1.2
25	2.42	1.652
30	3	2.01
35	3.68	3.68
40	4.29	4.29
45	5.13	5.13
50	6.01	6.01
55	6.71	6.71
60	7.48	7.48
65	8.24	8.24
70	9.17	9.17
75	10.2	10.2
80	11.18	11.18
85	12.32	12.32
90	13.81	9.364
95	14.74	14.74
100	16.11	16.11
105	17.28	17.28
110	19.59	19.59
115	21.41	14.98
120	23.53	23.53
124.23	29.21	29.21
90	31.34	21.26

Table 4.5: Load-deflection data for DB12



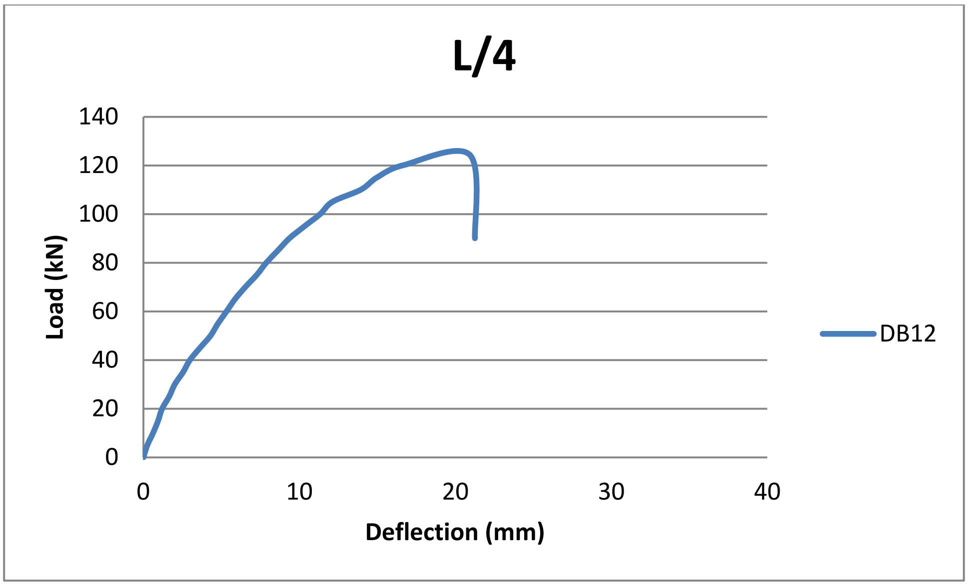


Fig 4.9: Load-deflection curves of DB12 a) at centre and b) at quarter of the span

At the failure load, the maximum deflection of the beam is seen to be 29.21mm which is very less than that of the control beam. The retrofitted beam showed significant increase in load carrying capacity.

Fig 4.10: Failure of DB12 by debonding of sheets

4.3.2.3 Average stress level 1(RDB 1)

The two beams, which are already damaged at 53% of the failure load, are retrofitted and tested to the failure. For the reference and most accurate results, the load-deflection curves of the average of both the beams are plotted as shown in Fig 4.11. These average curves with the deflection at centre of the span, is then compared to that of the control specimen RC.

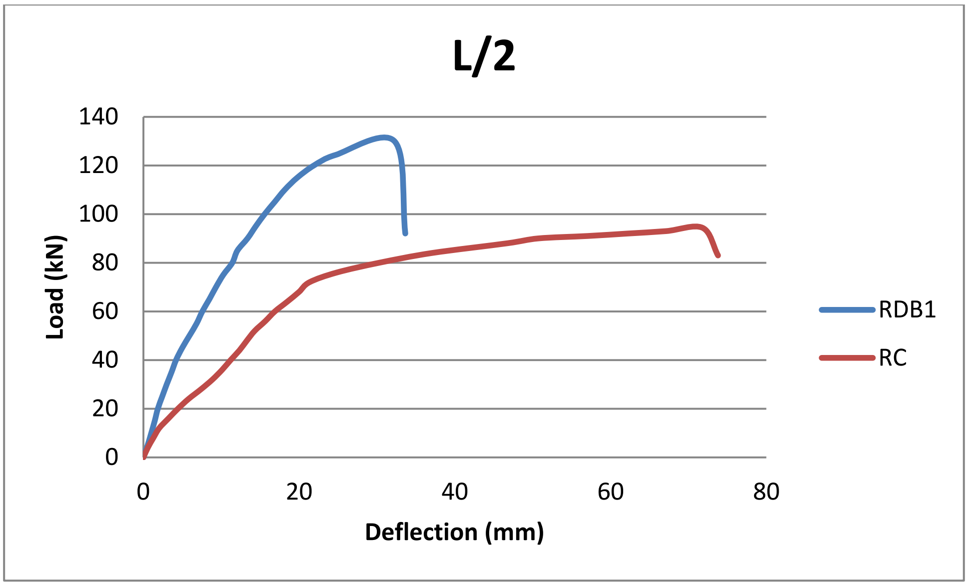


Fig 4.11: Load-deflection curves of RDB1 and its variation with RC at centre of span

The curve showed the wide variation of the retrofitted beam as compared to that of the control specimen. The failure load of the average of the two specimens is calculated to be 127kN and corresponding to this failure load, the maximum deflection is calculated to be 30.74mm. The stiffness from the load-deflection graph is calculated to be 5.42kN/mm. A very interesting thing is noted that beam before getting to the failure load of 127.11kN, yielded pattern is observed after 120kN as seen from the graph. This accounts for the ductility of the beam still present in the beam before retrofitting as it is stressed only to 53% of its failure load.

4.3.3 Initial Stress level 2

Damaged beam level 2 corresponds to the retrofitting of beams which are already damaged at 88% of the failure load. This level also has two beams. Both the beams are tested under the same testing arrangement on which control specimens are tested.

4.3.3.1 DB21

After retrofitting, the beams are tested under arrangement discussed in section 3.7. Table 4.6 shows the load-deflection data and Fig 4.12 shows the load-deflection curve so formed.

During testing it is observed that at initial stages not much of deflection is observed. Initial cracks are observed at 100kN of load. With the increasing load more intense cracks are observed. Cracking sound is heard at 120kN and at this point concrete start spalling out. The cracking pattern is shown in Fig 4.13. Sheet started debonding at 124kN. Loud cracking sound is heard at 133kN and the beam is failed due to debonding of the sheets as shown in Fig 4.14.

Load (kN)	Deflection at L/2 (mm)	Deflection at L/4 (mm)
0	0	0
5	0.562	0.61
10	1.07	1.3
15	1.97	1.93
20	2.79	2.56
25	3.89	3.18
30	4.72	3.71
35	5.88	4.17
40	6.69	4.6
45	7.97	5.07
50	9.12	5.49
55	10.25	5.97
60	11.38	6.37
65	12.45	6.8
70	13.23	7.2
75	14.16	7.62
80	14.89	8.01
85	15.71	8.44
90	16.45	8.88
95	17.17	9.27
100	17.98	9.93
105	18.64	10.45
110	19.25	10.98
115	20.11	11.79
120	21.34	12.64
125	22.98	13.69
130	24.57	14.57
131	27.51	15.45
90	28.42	16.38

Table 4.6: Load-deflection data of DB21

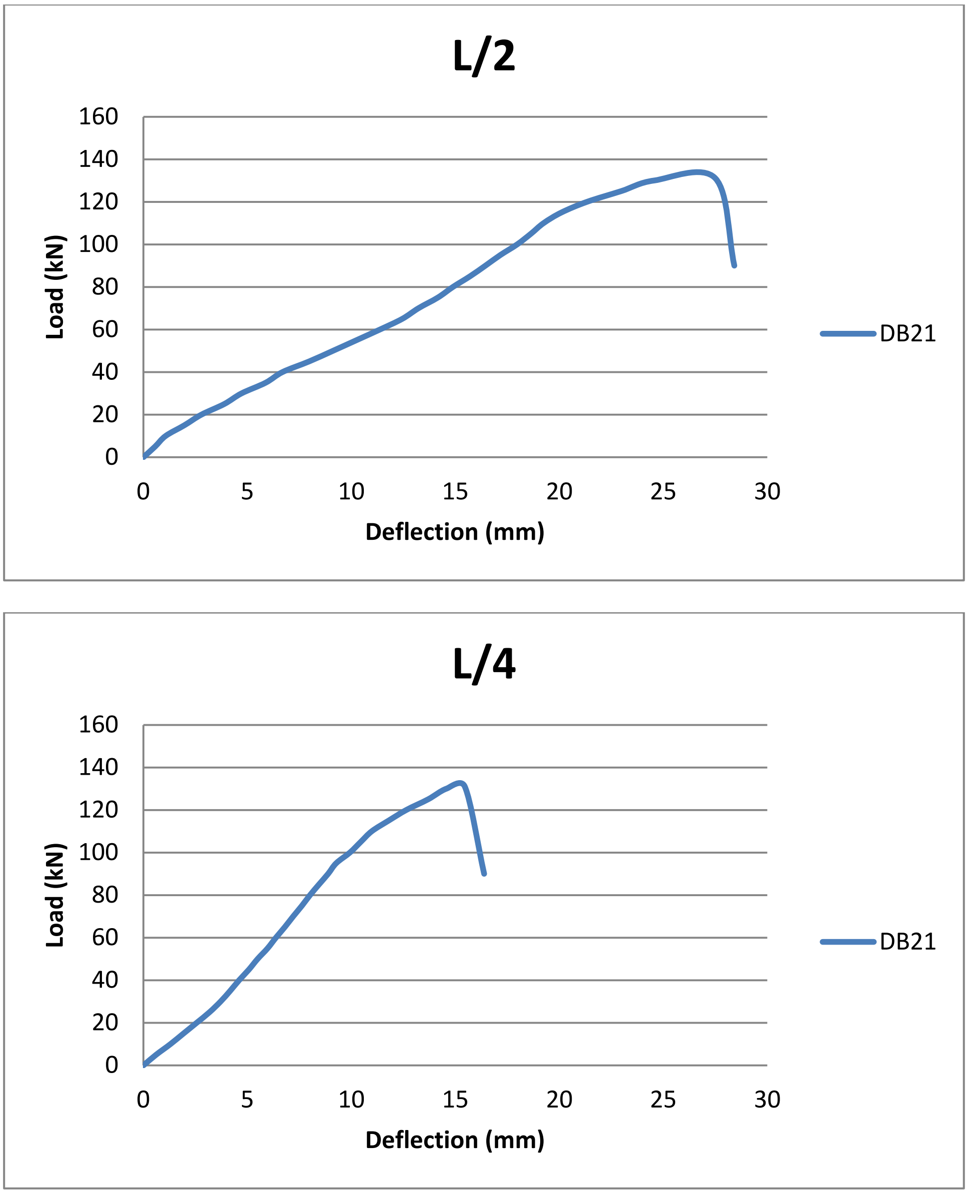


Fig 4.12: Load-deflection curves of DB21 a) at centre and b) at quarter of the span

Fig 4.13: Failure of DB12 by debonding of sheets

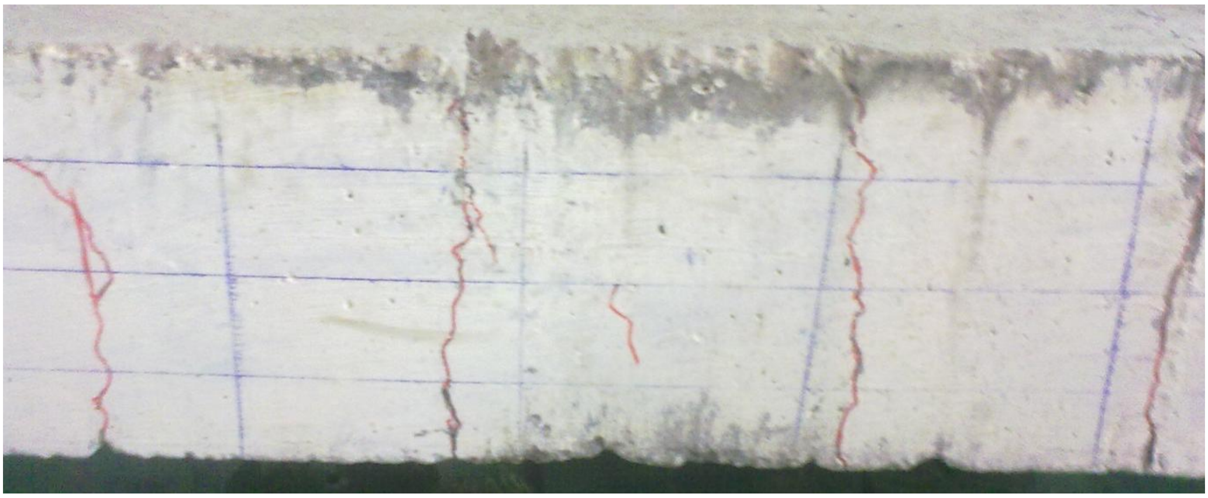


Fig 4.14 Cracking pattern of DB12

The maximum deflection of the beam is seen to be 27.51mm which is very less than that of the control beam. Significance increase in the load carrying capacity is observed. Cracks are seen to be more shifted towards the compression face of the beam thats they are extended towards the upper edge of the beam.

4.3.3.2 DB22

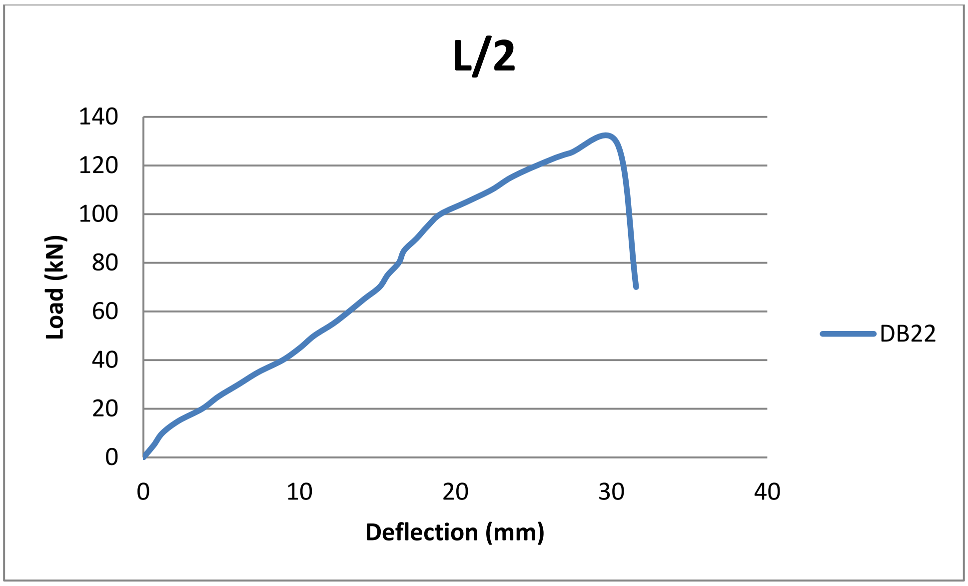
Another beam of same stressed level is retrofitted and tested. The load-deflection data is given in Table 4.7 and the graph so formed is shown in Fig 4.15.

Quite similar results are observed. Initial cracks are seen at 84kN of load towards the right half of the beam. More cracks are observed after the first crack with the increasing load. Second and third cracks are seen at 100kN towards the centre of the beam. Cracking sound is heard at 108kN and at this point concrete start spalling out. The cracking pattern is shown in Fig 4.17. Sheet started debonding at 118kN. Loud cracking sound is heard at 129kN and the beam is failed due to debonding of the sheets as shown in Fig 4.16.

Load (kN)	Deflection at L/2 (mm)	Deflection at L/4 (mm)
0	0	0
5	0.662	0.48
10	1.216	1.15
15	2.25	1.67
20	3.77	2.13
25	4.81	2.85
30	6.1	3.36
35	7.36	3.98
40	8.93	4.42

45	10.03	5.32
50	10.94	5.89
55	12.17	6.31
60	13.18	6.97
65	14.13	7.42
70	15.134	7.98
75	15.66	8.42
80	16.37	8.91
85	16.71	9.44
90	17.51	9.98
95	18.21	10.35
100	19.04	10.93
105	20.72	11.45
110	22.33	11.98
115	23.55	12.79
120	25.23	13.84
125	27.3	14.98
129	30.37	16.43
70	31.58	17.53

Table 4.7: Load-deflection data for DB22



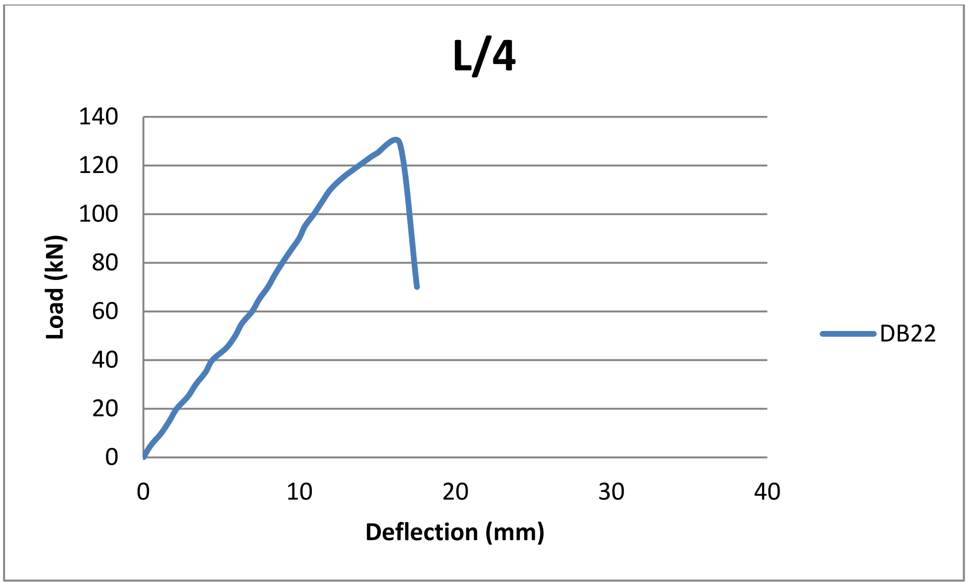


Fig 4.15: Load-deflection curves of DB22 a) at centre and b) at quarter of the span

At the failure load, the maximum deflection of the beam is seen to be 30.37mm which is very less than that of the control beam.

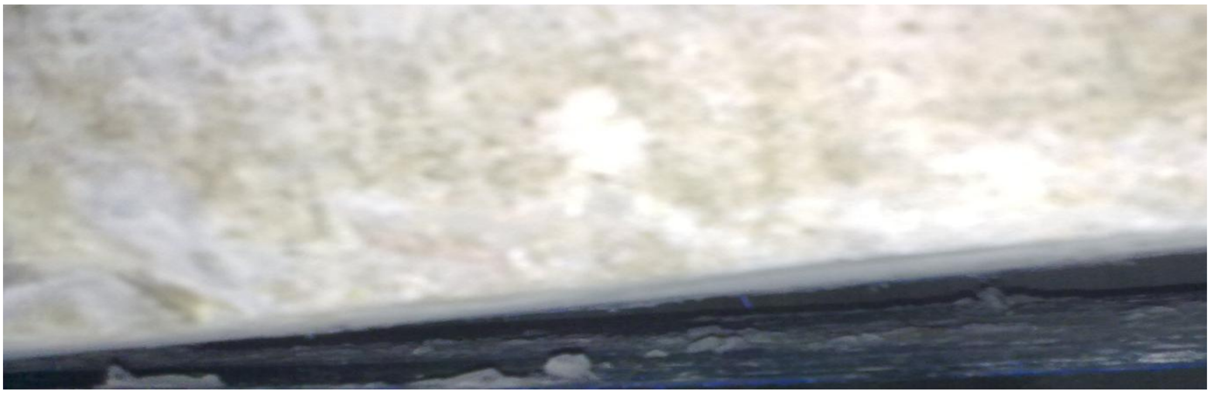


Fig 4.16: Failure of DB12 by debonding of sheets

Fig 4.17: Cracking pattern of DB12

4.3.3.3 Average stress level 2 (RDB2)

The two beams, which are already damaged at 89% of the failure load, are retrofitted and tested to the failure. For the reference and most accurate results, the load-deflection curves of the average of both the beams are plotted as shown in Fig 4.18. This average curve with the deflection at centre of the span is then compared to that of the control specimen RC.

The curve showed the wide variation of the retrofitted beam as compared to that of the control specimen. The failure load of the average of the two specimens is calculated to be 131kN and corresponding to this failure load, the maximum deflection is calculated to be 29.83mm. The stiffness from the load-deflection graph is calculated to be 4.47kN/mm.

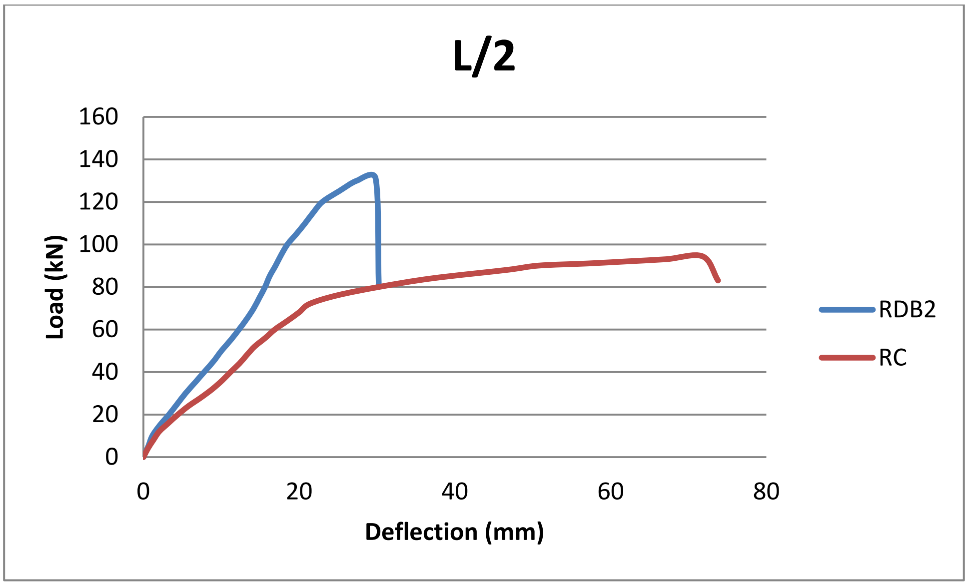


Fig 4.18: Load-deflection curves of RDB at centre of the span

It can be easily seen from the graph that before reaching at the failure load, it yielded a little bit which is slowly referring that the beam is now about to fail. This is simply due to fact that the beam is already damaged at 89% and not fully damaged before retrofitting. So this accounts that the beam still have some ductility which gets add up with that of the fibre.

4.3.4 Initial Stress level 3

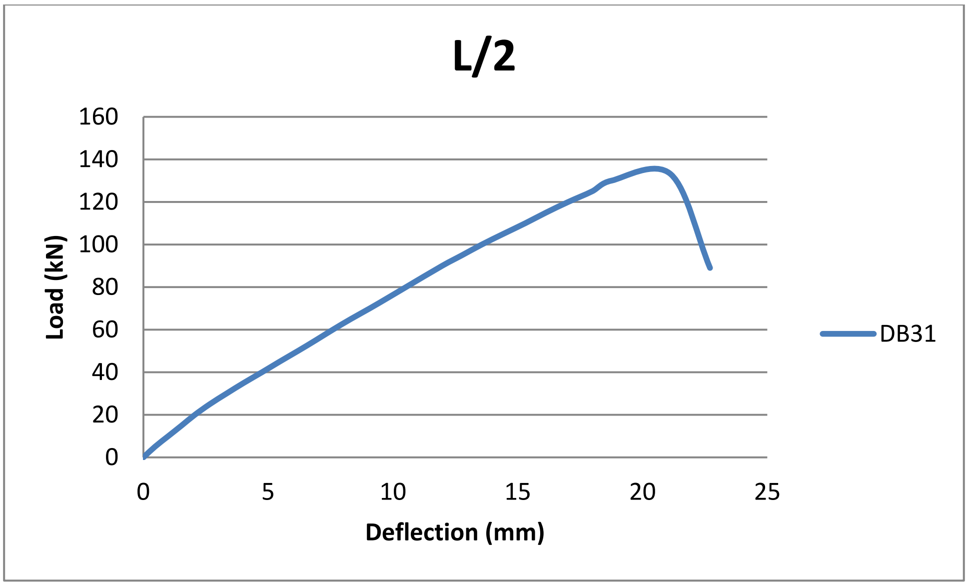
Damaged beam level 3 corresponds to the retrofitting of beams which are already stressed at 98% of the failure load. This level also has two beams. Both the beams are tested under the same testing arrangement on which control specimens are tested.

4.3.4.1 DB31

The load-deflection data is given in Table 4.8 and with this data the curves are drawn at centre and at quarter of the span are shown in Fig 4.19.

Load (kN)	Deflection at L/2 (mm)	Deflection at L/4 (mm)
0	0	0
5	0.462	0.05
10	0.998	0.36
15	1.534	0.67
20	2.06	0.96
25	2.66	1.38
30	3.33	1.83
35	4.02	2.28
40	4.75	2.75
45	5.46	3.24
50	6.19	3.69
55	6.9	4.14
60	7.59	4.59
65	8.31	5.05
70	9.07	5.49
75	9.8	5.94
80	10.52	6.44
90	11.97	7.39
95	12.77	7.95
100	13.56	8.66
105	14.42	9.2
110	15.3	9.74
115	16.13	10.39
120	17.02	11
125	17.98	11.96
130	18.75	12.89
133	21.13	13.53
89	22.7	14.76

Table 4.8: Load-deflection data for DB31



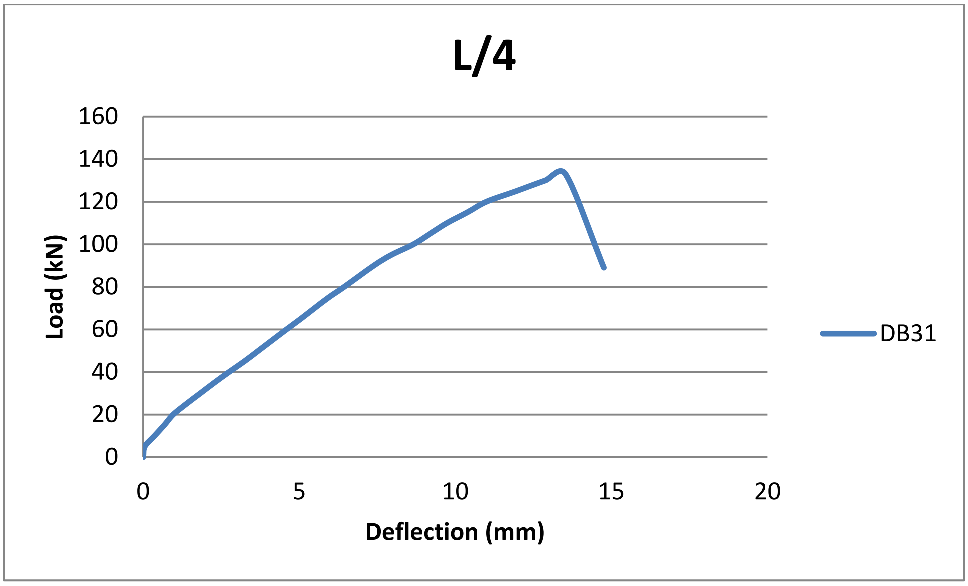


Fig 4.19: Load-deflection curves of DB31 a) at centre and b) at quarter of the span

The retrofitted beams after testing showed some really good results. It is observed that during initial stages not much of cracks are observed. They are initiated at 112kN and many cracks are developed in very short span of load. Cracking sound is heard at 121 KN and at this point concrete start spalling out. Loud cracking sound is heard at 133kN and the beam is failed due to debonding of the sheets as shown in Fig 4.20. The failure is very sudden and no yielding pattern is observed before failure. This shows the brittleness of the concrete due to fibre as beam is almost fully stressed before and it has already lost its ductility before retrofitting. The maximum deflection of the beam is seen to be 21.13mm which is very less than that of the control beam.

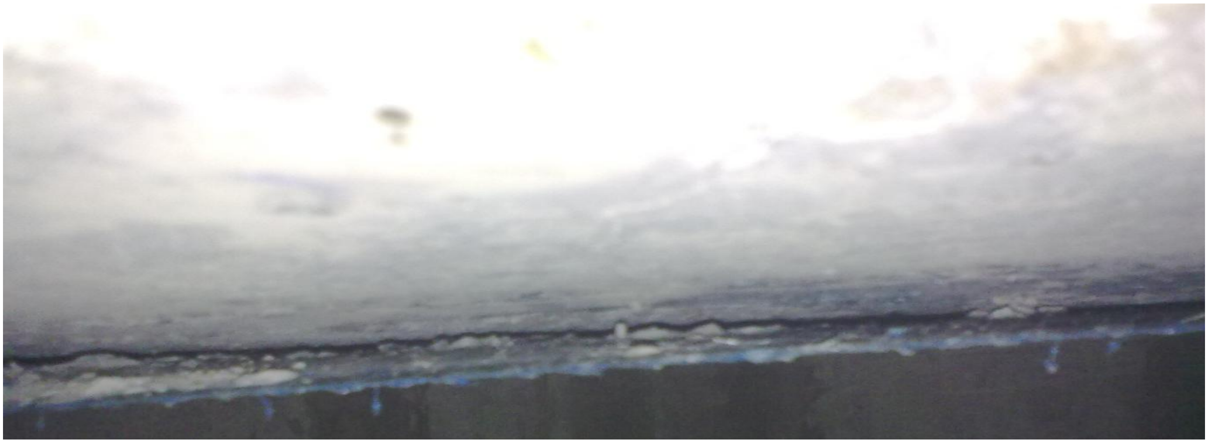


Fig 4.20: Failure of DB31 by debonding of sheets

4.3.4.2 DB32

Another beam stressed at the same level is retrofitted and tested. The load-deflection data is given in Table 4.9 and the load-deflection curve so obtained is shown in Fig 4.21.

Load (kN)	Deflection at L/2 (mm)	Deflection at L/4 (mm)
0	0	0
5	0.444	0.27
10	0.978	0.54
15	1.432	0.87
20	1.964	1.22
25	2.514	1.65
30	3.068	1.96
35	3.622	2.25
40	4.138	2.56
45	4.75	2.87
50	5.436	3.2
55	6.042	3.65
60	6.676	4.05
65	7.3	4.48
70	7.95	4.9
75	8.624	5.32
80	9.304	5.79
90	10.714	6.63
95	11.43	7.05
100	12.142	7.52
105	12.89	7.96
110	13.638	8.45
115	14.418	8.96
120	15.29	9.5
125	16.318	10.15
130	17.06	11.02
133	17.91	11.75
135	18.41	12.1
137.32	19.23	12.5
85	21.46	13.26

Table 4.9: Load-deflection data for DB32

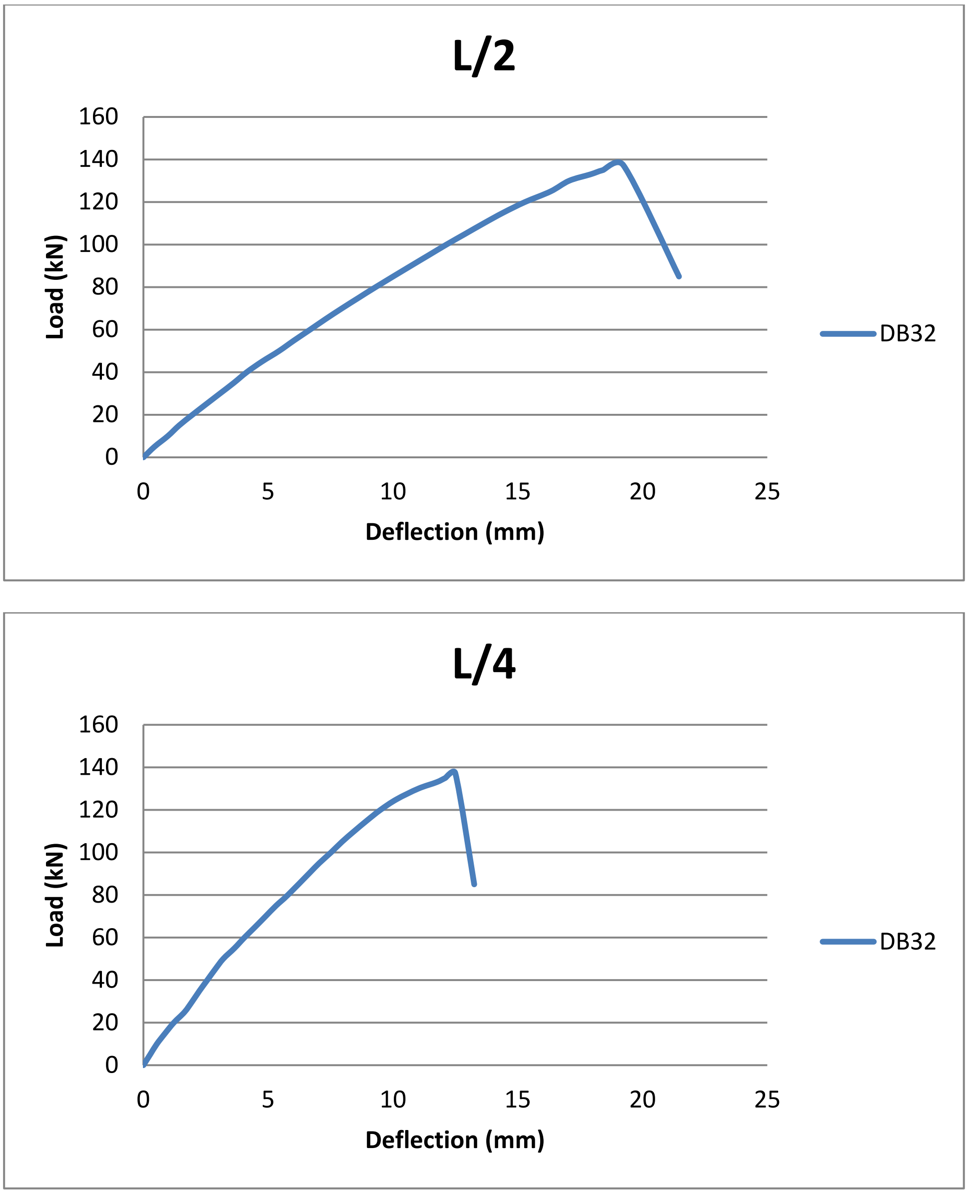


Fig 4.21: Load-deflection curves of DB32 a) at centre and b) at quarter of the span

Quite similar results are observed when another beam damaged at same stress level is tested under the same setup. Initial cracks are observed at 98kN of load towards the right half of the beam. More cracks are observed after the first crack with the increasing load. Second and third cracks are shifted towards the centre. Cracking sound is heard at 120kN and at this point concrete start spalling out. The cracking pattern is shown in Fig 4.22. Sheet. Loud cracking sound is heard at 137.32kN and the beam is failed due to debonding of the sheets as shown in Fig 4.23.

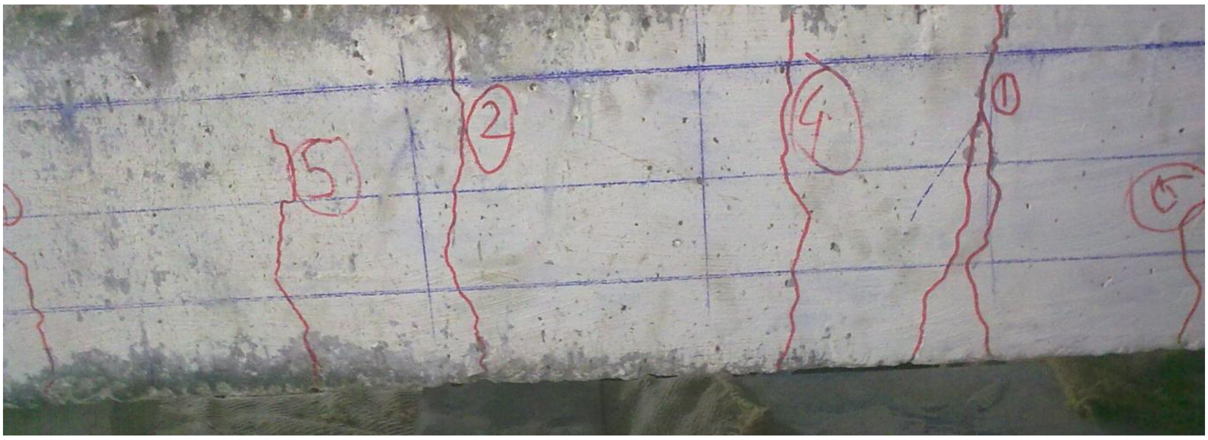


Fig 4.22: Cracking pattern of DB32

At the failure load, the maximum deflection of the beam is seen to be 19.23mm which is very less than that of the control beam. The retrofitted beam showed significant increase in load carrying capacity.

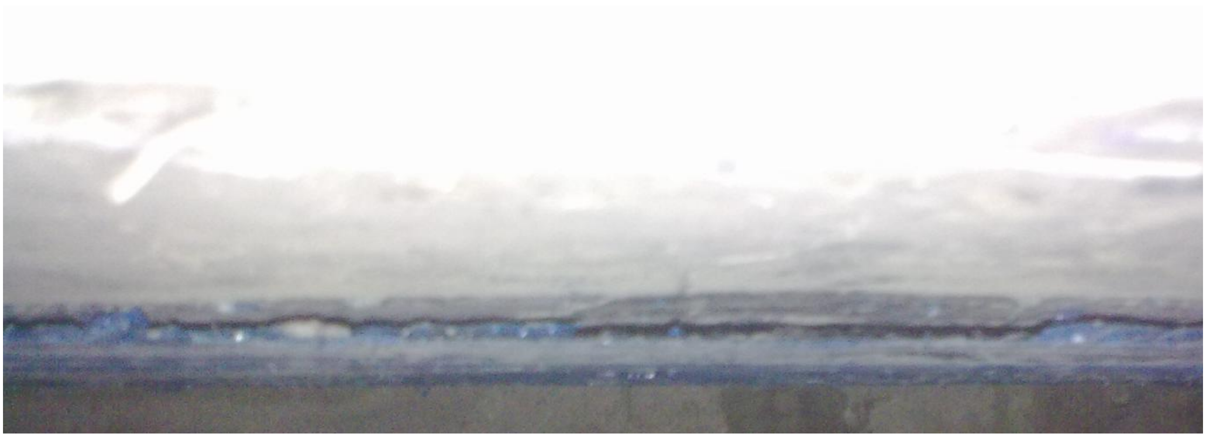


Fig 4.23: Failure of DB32 by debonding of sheets

4.3.4.3 Average stress level 3

The two beams, which are already 98% stressed, are retrofitted and tested to the failure again. For the reference and most accurate results, the load-deflection curves of the average of both the beams are plotted as shown in Fig 4.24. These average curves with the deflection at centre of the span, is then compared to that of the control specimen RC. The curve showed the wide variation of the retrofitted beam as compared to that of the control specimen. The failure load of the average of the two specimens is calculated to be 135.15kN and corresponding to this failure load, the maximum deflection is calculated to be 21.36mm. The stiffness from the load-deflection graph is calculated to be 5.83kN/mm. A very interesting thing is noted here from the graph that there is no yielding at all before failure load. The deflections are getting increased with the increasing load but there is no alarm prior to the failure. It just simply failed at the failure load.

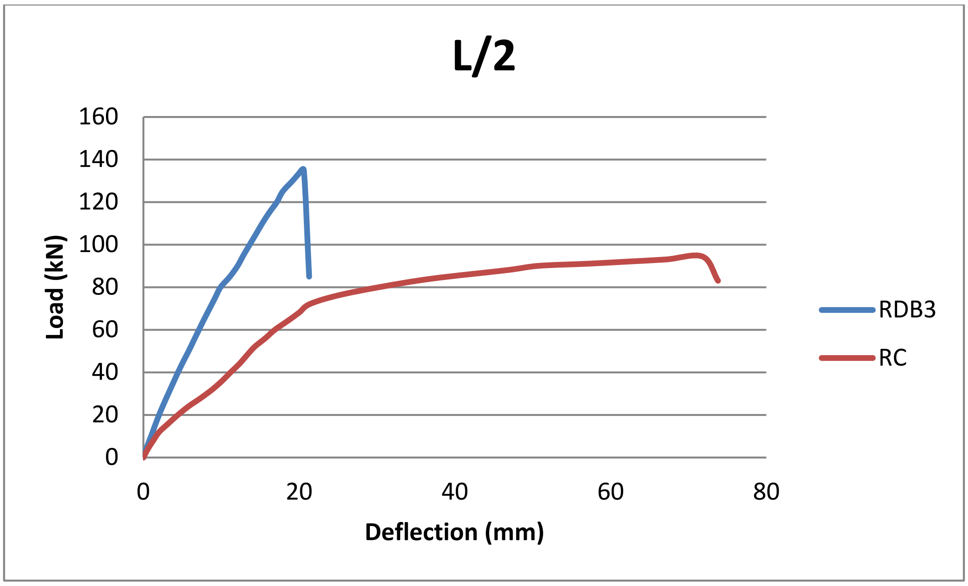


Fig 4.24: Load-deflection curves of RDB3 and its variation with RC at centre of span

This is due to the fact that these beams are 98% stressed before the retrofitting is done. This accounts to almost full loss of its ductility and stiffness. So the retrofitted beam has only the ductility of the sheets which is very less. This is the reason why it showed the brittleness characteristics.

4.4 Effect of initial stress levels on retrofitting

As per the test matrix the three levels are chosen from the load-deflection curve of the control specimen. The damaged beams are retrofitted and results are compared with that of the control specimen as shown in Fig 4.25.

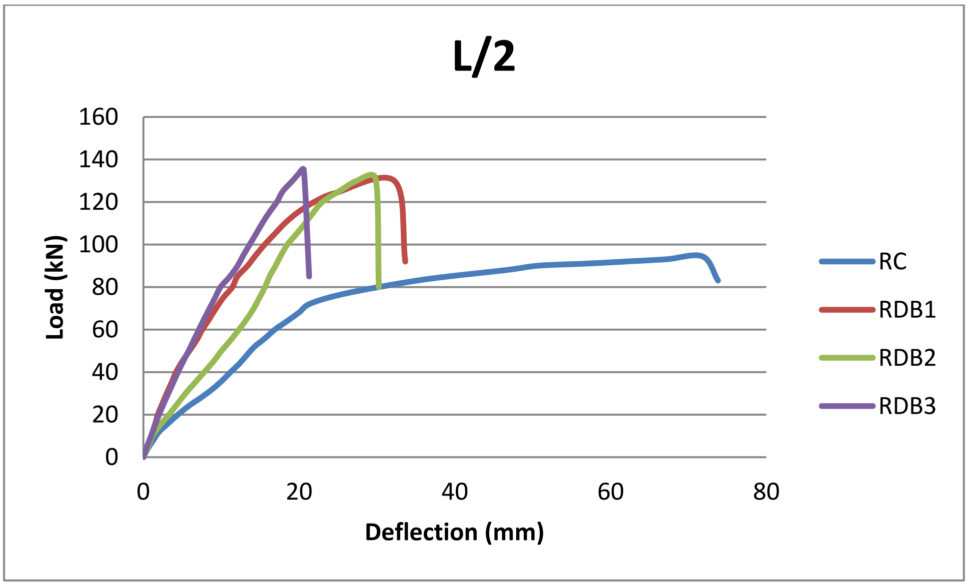


Fig 4.25: Variation of initial stress levels

The results of retrofitted beams at the three initial stress levels are also compared with themselves to check the variation of retrofitting on various initial stress levels. The variation of RC, RDB1, RDB2 and RDB3 over several parameters is shown in Table 4.10 and Fig 4.26. The load-deflection curves are not recorded upto the failure of each test since the LVDT transducer has to be removed when a significant vertical displacement is seen in case of control specimen. For the retrofitted beams, as the debonding failure of sheets are expected so LVDT`s are removed to prevent any damage to it otherwise the steel at the soffit of the beam is expected to get yielded more which might get increase the ductility of the specimens.

Beam	Damage level	Load (kN)	Deflection (mm)	Stiffness (kN/mm)
RC		94.35	71.77	3.91
RDB1	50kN (53%)	127	30.74	5.42
RDB2	84kN (89%)	131	29.83	4.47
RDB3	93kN (98%)	135.15	21.36	5.83

Table 4.10: Variation of RC, RDB1, RDB2 and RDB3

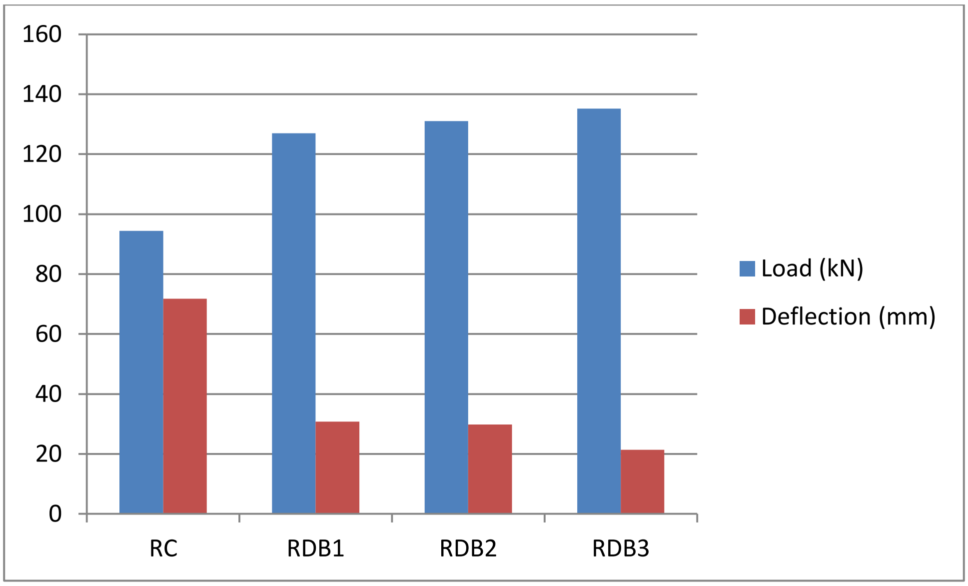


Fig 4.26: Variation of RC, RDB1, RDB2 and RDB3

The variation in load carrying capacity and maximum deflection can be seen in Fig 4.26. It can be easily seen that RDB3, which refers to 98% damage level, shows the minimum deflection as compared to the other stress levels. It also has the maximum load bearing capacity and also has the maximum stiffness calculated from the curve as shown in Fig 4.27, calculated in elastic limit, as compared with other stress levels. From Fig 4.25 it can be easily seen that there is sudden failure in the beam which is resulted due to the increase in brittleness. This is due to the fact that 98% damage means the steel has almost completely yielded and the specimen is no more ductile. As a result the beam has completely lost its own ductility and the retrofitted beam has ductility that of the CFRP sheets only which is quite less. On the other hand RDB1, which refers to 53% damage level, has the maximum yielding zone with minimum brittleness as compared with other stress levels. This is because the beams are damaged at 53% damage which is in Elastic zone only which means it already has enough ductility in it. That is the reason why the failure in this case is not sudden. It yielded for some time and then it failed to the failure load.

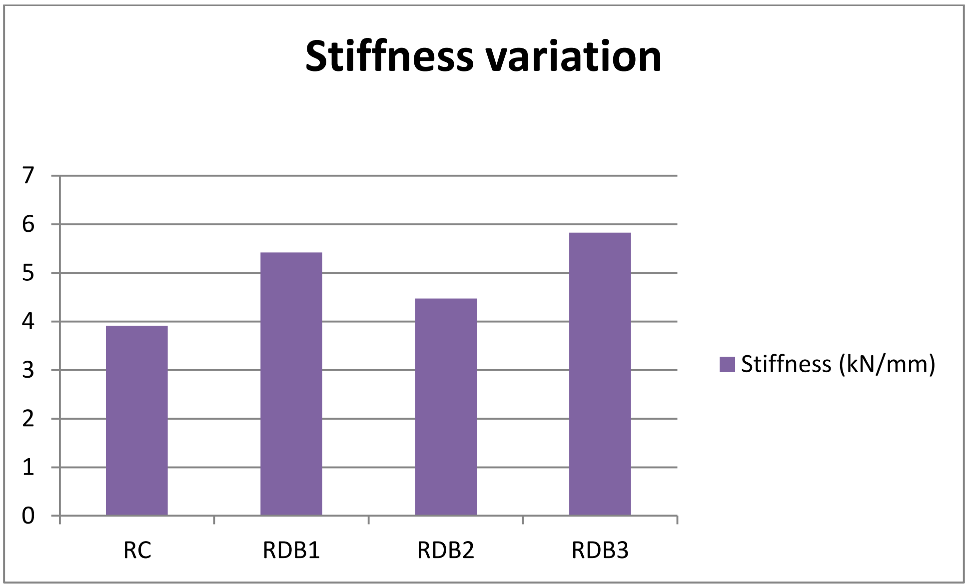


Fig 4.27: Variation of Stiffness at each stress level

Talking about RDB2, as it was 89% damaged before retrofitting, it also has some stiffness left to it which results in the yielding before reaching the failure load. But the yielding in this case is lesser than that of the RDB1.

CONCLUSIONS

The results revealed by the retrofitting of beams using pre-stressed carbon fibre reinforced sheets are phenomenal. Keeping these results in mind, this method of retrofitting can be used with far more benefits over the conventional retrofitting measures. With more researches are been carried all across the globe on pre-stressed CFRP, the scope of this system can be seen very high. On its basis the following conclusions can be drawn:

1. Load carrying capacity of the retrofitted beams is remarkably improved compared to that of fresh control beam.
2. The retrofitted beams initially stressed at various levels do not show the big variation in terms of the load carrying capacity. The control specimen is failed at 94.35kN and the retrofitted beams are seen to be failed in the range of 125kN to 136kN.
3. There is subsequent decreases in the deflection also as compared to that of the fresh control beam at all the stresssed levels. The 53% stressed beam showed the maximum deflection whereas the 100% stressed beam showed the minimum deflection.
4. Stiffness has also increased after retrofitting at all stressed levels as compared to the control beam. The 53% and 89% stressed beams showed almost same stiffness but the 98% stressed beam showed the maximum stiffness.
5. With the increase in damage level, the ductility of the retrofitted beams is found to be decreased as compared to the fresh beam. The 53% stressed beam showed more ductility as compared to the 89% stressed beam. Yielding is observed in these beams before they failed at the ultimate load whereas beams with 98% stressed, sudden failure is observed which accounts for the maximum brittleness in the beam.
6. The failure in the retrofitted beam is due to the debonding of the sheet from the beam. This meaNs even if three layers of sheets are used, the results would still not vary much as the failure occurs due to debonding of sheets and not by failure of sheets.
7. Shifting of cracks is seen towards the compression zone as compared to the pure bending failure of the fresh control beam which cracked on the tension side only.

REFERENCES

A.Benachour, S. Benyoucef, A. Tounsi, E.A. Adda bedia, "Interfacial stress analysis of steel beams reinforced with bonded prestressed FRP plate", Engineering Structures Vol 30, page 3305-3315, 2008.

A.H. Al-Saidy, A.S. Al-Harthy, K.S. Al-Jabri, M. Abdul-Halim, N.M. Al-Shidi, "Structural performance of corroded RC beams repaired with CFRP sheets", Composite Structures Vol 92, page 1931–1938, 2010.

A. Pan and J. Moehle, "Displacement Ductility of Flat Plates," ACI Structural Journal, 1989

Abhijit Mukherjee and Gopal L. Rai, "Performance of reinforced concrete beams externally prestressed with fiber composites", Construction and Building Materials Vol 23, page 822–828, 2009.

Bank L. C and Arora D, "Analysis of RC beams strengthened with mechanically fastened FRP (MF-FRP) strips." Journal Of Composites Structures, page 180-191, 2006.

Cem Yalcin, Osman Kaya, Mustafa Sinangil, "Seismic retrofitting of R/C columns having plain rebars using CFRP sheets for improved strength and ductility", Construction and Building Materials Vol 22, page 295–307, 2008.

Christoph Czaderski and Masoud Motavalli, "40-Year-old full-scale concrete bridge girder strengthened with prestressed CFRP plates anchored using gradient method", Composites Part B Vol 38, page 878–886, 2007.

Constantin E. Chalioris, "Analytical model for the torsional behaviour of reinforced concrete beams retrofitted with FRP materials", Engineering Structures Vol 29, page 3263–3276, 2007.

Constantin E. Chalioris, "Torsional strengthening of rectangular and flanged beams using carbon fibre-reinforced-polymers – Experimental study", Construction and Building Materials Vol 22, page 21–29, 2008.

Erhard and Gunter, "Designing with Plastics",Hanser Publisher, Munich, 2006.

G. Krause and J. Wight, "Strengthening of Reinforced Concrete Frame Structures," 4USNCEE, May 1990.

Head P. R, “Advanced composites in Civil engineering- a critical review”, Advanced composite materials in bridges and structures, Proceedings of second international conference, Quebec , Canada, page 3-15, 1999.

Jun Deng, Marcus M.K. Lee, Shiqing Li, “Flexural strength of steel–concrete composite beams reinforced with a prestressed CFRP plate”, Construction and Building Materials Vol 25, page 379–384, 2011.

Kachlakev D and McCurry DD. Behaviour of full-scale reinforced concrete beams retrofitted for shear and flexural with FRP laminates. Composites: Part B, Vol 31, page 445–52, 2000.

Kim J yail, Longworth Jesse M, Wight R Gordon, Green Mark F, “Flexure of slabs strengthened with prestressed and Non prestressed CFRP sheets.” Journal of composites for construction ASCE Vol 12, page 366-374, 2008.

Kim J Yail, Wight R.Gordon, Green Mark F, “Flexure strengthening of RC beams with prestressed CFRP sheets: Using Non Metallic Anchor Systems.”, Journal of composites for construction ASCE Vol 12, page 44-52, 2008.

Pierluigi Colombi and Carlo Poggi, “An experimental, analytical and numerical study of the static behaviour of steel beams reinforced by pultruded CFRP strips”, Composites: Part B Vol 37, page 64–73, 2006.

Rajendra Desai, “Managing Earthquake Risk”, Action in Disaster Preparedness (NCPDP) Ahmedabad, Gujarat, Government of India, 2008.

Riyadh Al-Amery and Riadh Al-Mahaidi, “Coupled flexural–shear retrofitting of RC beams using CFRP straps”, Composite Structures Vol 75, page 457–464, 2006.

Sang-Kyun Woo, Jin-Won Nam, Jang-Ho Jay Kim, Sang-Hoon Hanc, Keun Joo Byun, “Suggestion of flexural capacity evaluation and prediction of prestressed CFRP strengthened design”, Engineering Structures Vol 30, page 3751-3763, 2008.

Sen R, Liby L, Mullins G, “ Strengthening steel bridge sections using CFRP laminates”, Composites: Part B, Vol 32, page 309–22, 2001.

Smallman R.E and R.J Bishop, “Modern Physical Metallurgy and Materials Engineering", 6th ed. Oxford: Butterworth-Heinemann, 1999.

Wight R G, Green M F and Erki M, “ Prestressed FRP sheets for Post strengthening Reinforced Concrete Beams ”, Journal of Composite for construction Vol 5, page 214-220, 2001.