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GFRP external strengthening of structural concrete

Tarek Mohamed Nabil Fathy Rizk
The American University in Cairo AUC

GFRP EXTERNAL
STRENGTHENING OF
STRUCTURAL CONCRETE

TAREK MOHAMED NABIL
FATHY RIZK

1997

THE AMERICAN UNIVERSITY IN CAIRO
SCHOOL OF SCIENCES AND ENGINEERING

GFRP EXTERNAL STRENGTHENING OF STRUCTURAL CONCRETE

A Thesis Submitted to

THE ENGINEERING DEPARTMENT

In partial fulfillment of the requirements for the degree of

Master of Science in Engineering
with specialization in

Construction Engineering

BY

Tarek Mohamed Nabil Fathy Rizk
B.Sc. (1994) in Construction Engineering, American University in Cairo
Graduate Diploma in Engineering (1995), American University in Cairo

under the supervision of:
Dr. Safwan Khedr, Associate Prof. and Head of Construction Engineering Unit, American University in Cairo
Dr. Ibrahim Mahfouz, Prof. and Vice Dean of Graduate Studies, Faculty of Engineering, Zagazig University

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has been approved by

Dr. Safwan Khedr, (Advisor)
Head, Construction Engineering Unit,
Engineering Department, AUC

Dr. Ibrahim Mahfouz,
Professor, Faculty of Engineering,
Zagazig University

Dr. Omar Aly El Nawawy,
Professor, Faculty of Engineering,
Ain Shams University

Dr. Medhat Haroun,
Professor, Engineering Department, AUC

Dr. Maher Y.A. Younan,
Head, Engineering Unit,
Engineering Department, AUC

[redacted]

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DEDICATION

To my mother and the memory of my father

Acknowledgment

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ABSTRACT

The backbone of any country's economy consists of constructed facilities that include public buildings, airports, highways, etc. Many of the constructed structures are suffering from continuous deterioration. A reliable system that would maintain the structural integrity, and extend life of constructed facilities would save countries an enormous amount of money. The emergence of high strength resins made possible the strengthening and repair of concrete members using externally bonded fiber reinforced plastic sheets. The aim of this work is to investigate external strengthening and repair of concrete members using glass fiber reinforced plastic (GFRP) sheets which have superior properties including high strength to weight ratio, high resistance to chemical attacks and non corroding.

Experimental and theoretical studies were carried out to investigate the behavior of concrete members when strengthened or repaired using externally bonded advanced composite material. Experimental work included investigating tensile properties of glass GFRP sheets, concrete cylinders strengthened using GFRP sheets, and beams repaired using GFRP sheets. Finite element analysis using ABAQUS was conducted in order to compare experimental and theoretical results. Preliminary design formulae were developed to anticipate the loading capacity of repaired beams.

Both the experimental and the theoretical results indicate that GFRP externally bonded to concrete members improve both their strength limit state in the form of increased flexural and shear load capacities and their serviceability limit state in the form of reduced cracks.

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LIST OF SYMBOLS

GFRP	Glass fiber reinforced plastics
FRP	Fiber reinforced plastics
PVA	Polyvinyl alcohol
UP	Unsaturated polyester
DGEBA	Diglycidyl ether of bisphenol
DETA	Diethylenetriamine
M_u	Beam's ultimate moment capacity
A_s	Cross-sectional area of tensile steel
F_y	Steel yield strength
A_f	Cross-sectional area of GFRP longitudinal reinforcement
F_f	GFRP ultimate strength
Q_u	Total shear capacity of beam
Q_c	Contribution of concrete to shear capacity of beam
Q_s	Contribution of steel to shear capacity of beam
Q_f	Contribution of FRP to shear capacity of beam

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CHAPTER I

INTRODUCTION AND LITERATURE REVIEW

1.1- Introduction

The backbone of any country's economy consists of assets of constructed facilities that include public buildings, airports, highways, bridges, historical structure etc. Many of the constructed structures are suffering from continuous deterioration at a rate faster than our ability to renovate them. In the USA, for example, studies (McConnell, 1993) estimate that a total of over 3 trillion dollars will be required during the next 15 years to bring these facilities to efficient operating levels. Studies, (America's Highway, 1984) also, estimate that the cost of replacing a disintegrated structure is as twice as the original cost of construction. The problem of deteriorating structures is even more serious in developed countries including Egypt because of their economic problems which may lead to neglecting maintenance of the buildings which represent tremendous national investment.

A reliable structural strengthening system that would maintain the structural integrity , and extend the life of constructed facilities would save countries an enormous amount of money. Strengthening of concrete structural members is a must because of the problems which occur in concrete as it is cast or afterwards due to inadequate design, poor construction, improper selection of material, normal deterioration with aging, events like fire, earthquakes, severe weather conditions, settlement or overloading. Once this has happened, there are two main alternative actions to the problem; strengthen or replace the structure. Because of the large number of deficient structures, and of the very high replacement cost, structural strengthening is often the viable solution.

The emergence of high strength resins may provide a reliable and economical method of strengthening concrete members especially in the long term. By using the very high adhesive strength resins, additional material can be bonded to concrete members in order to improve its structural capacity. Traditionally, steel plates were bonded to critical areas of concrete members increasing their strength and stiffness. Lately, research efforts have been devoted to explore the use of advanced composite materials (used by aerospace industry for more than 50 years) with superior mechanical and physical properties such as high strength to weight ratio, high energy absorption, good chemical resistance, non corrosive, and high fatigue resistance properties. A member of such class of materials is fiber reinforced plastics (FRP) and many indications show that FRP will revolutionize reinforced concrete strengthening techniques because of its excellent properties that would extensively prolong the life of existing facilities by decreasing their rate of deterioration as will be shown in the literature review later.

FRP are mainly composed of high performance fibers, the most common of which are glass, carbon, and aramid embedded in a polymer of either thermosetting (e.g. ,polyester, vinyl ester, epoxy) or thermoplastic resins (e.g., nylon, PET). An explanation of the constituents and properties of FRP products is provided in the next chapter.

1.2- Historical background

The use of composites started in the USA after world war II. The main applications at that time were in the aerospace industry because of the electromagnetic transparency and high strength to weight capabilities, advanced composites were used by the aerospace industry 50 years ago. Research efforts which explore the use of resins in concrete started in the late 60's with a program at the Bureau of Reclamation on polymer impregnated concrete. Marshall-Vega Corporation was the first company to manufacture a glass FRP reinforcing bar in North America.

In spite of the fact that those research programs started in the late 60's, commercial application of FRP products was not recognized until late 70's. It was only during early 1980's that another FRP company was established in North America (International Grating, Inc.).

In 1986, the world's first highway bridge using composite reinforcement was built in Germany. Since then, several bridges have been constructed throughout Europe, the USA, and Japan. By the end of 1993, there were several companies marketing FRP products for civil engineering purposes in North America, Europe, and Japan.

1.3- Literature survey

The development of epoxy resins in the 1960's made possible the strengthening of existing structures by bonding of external steel plates. Although this method has proven to be effective in repairing and upgrading concrete structures, it had several drawbacks. The most important of which are the corrosion problems at the steel to adhesive interface, also, steel plates are heavy thus they add to the dead load of the member and are difficult to handle in site (usually installation requires heavy equipment and skilled labor).

Recently, research has been carried out to investigate using FRP sheets (instead of steel plates) to externally reinforce concrete members because of their superior properties:

1- Light weight which makes FRP sheets easy to handle in site. Also repair can be done in a short period of time because there is no need for heavy equipment. This is extremely useful in many real life situations such as, highway bridges where you don't want to disturb the traffic for long periods , military emergencies, etc.

2- Non corroding material where there is no corrosion problem like as the case of steel plates.

3- High strength to weight ratio; accordingly, a very thin sheet of FRP can efficiently replace a thick steel plate.

4- High resistance to chemical attacks from acids, alkalis and salts; consequently, FRP is the material to use for repairing and upgrading all structures subjected to chemical attacks for example water desalination plants, waste water treatment stations, and offshore petroleum platforms.

5- The coefficient of thermal expansion of FRP is close to that of concrete; thus the two materials are compatible and there are no problems of expansion or shrinkage.

The premium properties listed above attracted a number of civil engineering researchers to study the external reinforcement of concrete members using FRP. Saadatmanesh et al, 1990 and Ritchie et al, 1991 studied the external strengthening of beams using fiber composite plates. FRP plates of glass, aramid and carbon were adhered to the tension side of the tested beams. Ritchie et al, found out that an increase in stiffness ranging between 17 to 99 percent and an increase in the ultimate strength ranging between 40 to 97 percent was achieved in the plated beams. Therefore, they concluded that bonded plates of fiber reinforced plastics is a feasible method of improving stiffness and strength of reinforced concrete beams. Also, they developed an iterative analytical method to predict the stiffness and maximum strength in bending of the plated beam. Meier et al, 1992 and Deblois et al, 1992 used non corroding, light weight, high strength fiber sheets instead of steel plates to externally reinforce beams. Faza et al, investigated the ability of rehabilitating deteriorated concrete structural members using steel plates or carbon fiber wraps. They reached several conclusions including that external carbon wrapping is a convenient system that is resistant to corrosion and that beams previously loaded and later strengthened with carbon wrapping exhibited similar performance without any change in strength and deflection when compared to the control beams. However, they pointed out that the performance of carbon wrapping under adverse environmental conditions, long term effects on adhesives and fibers, ductility, and creep need evaluation. Sharif et al, 1994 worked on the repair of initially loaded reinforced concrete beams with epoxy bonded fiber glass reinforced plastic plates. They used different repair and anchoring schemes. The results indicated that generally the flexural strength of the repaired beams is increased but the ductile behavior of the repaired beams is inversely proportional to the plate thickness. . Chajes et al, 1995 studied the ability of externally bonded composite reinforcement to improve the beams' flexural and shear capacity. The composite used was a unidirectional carbon-fiber-reinforced tow-sheet. The tow-sheet has a paper backing and a glass-fiber scrim which serves to keep the dry fibers in place. The results indicated an increase in flexural stiffness ranging between 103 to 178 percent and an increase in the ultimate beam capacity ranging between 158 to 292 percent over that of the control beams having no external reinforcement. In addition, simple formulae were produced to predict the ultimate capacity of the externally reinforced beams. James et al (Royal Military College of Canada) investigated the flexural behavior of plain concrete beams, with and without external strengthening using carbon fiber and glass fiber sheets bonded to their tension face. They came up with several conclusions, the most important of which is that the load versus displacement behavior of the carbon fiber upgraded concrete beams was more repeatable compared to that of glass fiber. They, also, concluded that two failure mechanisms are responsible for failure in the glass and carbon sheets: 1- the rupture of the sheet in the constant moment region 2- sheet delamination as a result of relative displacement at a flexural shear crack and the shearing of the concrete at the epoxy resin interface. Erki et al, 1995 investigated the external reinforcement of slabs using GFRP. The slabs were subjected to transverse patch loads. The additional reinforcement by FRP increased the flexural strength of the slabs, delaying flexural cracking to higher loads, therefore increasing it's resistance to punching shear. James et al, also concluded that the modulus of elasticity of the FRP used depends on the orientation of the fibers and the components of the FRP used.

1.4-Research objective and scope

The main objective of this research is to study the behavior of concrete members when externally strengthened (upgraded or repaired) using advanced composite material (FRP sheets). To accomplish our purpose, the following was studied :

1- The tensile properties of glass fiber reinforced plastic (GFRP) samples.

2- The magnitude of variation in strength provided by wrapping concrete cylinders with GFRP sheets. The effect of changing the number of wrapping layers on strength was studied.

3- The magnitude of variation in both the ultimate flexural and shear capacities provided by GFRP sheets to pre-loaded and cracked beams (85% of the ultimate loaded).

CHAPTER II

FIBER REINFORCED PLASTIC MATERIALS

2.1- Introduction

A composite material is formed when two or more materials are combined so that the properties of the composite are different from, and usually better than those of the individual constituents. Today modern advanced composites refer primarily to fiber reinforced plastics (FRP). Fiber reinforced plastics are composed of continuous fibers made of, carbon, glass, aramid, or polyvinyl alcohol (PVA), embedded in resins, alone or joined with other traditional materials like concrete and steel to form hybrids or super-composites. The fibers can also be discontinuous but in this case the fiber volume fraction should be 10 percent or more in order to provide a significant reinforcement function. Resins can be either thermosetting or thermoplastic. Thermosetting resins harden upon application of heat and can not be reliqufied, such as polyesters and epoxies. Thermoplastic resins solidify by cooling and are liquefied by heating, such as PVC, polyethylene, and polypropylene. FRP products for civil engineering applications depend on thermosetting resins.

The performance of any composite depends on the reinforcing fibers which are the load bearing constituent of any structural composite, and the interaction between the fibers and the polymer matrix.

2.2- Importance of the polymer matrix

A polymeric material such as plastic, consists of a collection of a large number of polymer molecules of similar chemical structure. Reinforcing fibers are impregnated with polymers to form fiber reinforced plastics. Although the reinforcing fibers are the load bearing element of any structural composite, it is important to understand the role played by the matrix polymer which may be summarized in the following points:

1- Transfer applied load and stresses to fiber.

2- Protect the surface from damage during handling.

3- Protect the fibers from mechanical and environmental damage.

4- Resin provides lateral support against fiber buckling under compression loading.

5- The polymer matrix properties influence interlaminar shear.

6- The polymer matrix properties influence in-plane shear.

7- The polymer matrix has to be chemically and thermally compatible with the fibers.

8- The polymer matrix stops to some extent a crack from propagating straight through a mass of fibers.

2.3- Types and properties of the most commonly used resins

The most common commercial thermosetting matrix polymers used in the production of composites that are used to reinforce concrete structures are polyester resins, epoxied resins, and vinyl esters.

2.3.1- Polyester resins

Polyester resins can be divided into two classes, (i) the so-called saturated polyester resins such as polyethylene terephthalate (ii) the unsaturated polyester resins which can be cross-linked, by the addition of a catalyst, into thermoset polymers. Unsaturated polyester (UP) is the polymer resin most commonly used to produce large composite structural parts. The composite institute (ACI Committee 440, 1996) estimates that 85 percent of U.S. composite production is based on unsaturated polyester resins.

Unsaturated polyester resins are prepared by the polycondensation of dihydroxyl derivatives and diabasic organic acids or anhydrides. In order to convert this material into a usable resin, a low viscosity unsaturated monomer such as styrene is added. This monomer acts as a solvent for the polyester resin to produce a liquid with a suitable handling viscosity as a cross-link of the polyester chain to give a fully cross-linked thermoset structure. Once the polyester has been dissolved in a monomer, the system may gel at room temperature in the absence of a catalyst. To prevent this and to obtain a storage life for the resin, an inhibitor is added after the polycondensation reaction is completed and before the resin is dissolved in the monomer. Inhibitors are reducing agents that absorb free radicals preventing them from polymerizing the resin. Therefore, once an inhibitor is exhausted, any free radicals will cause gelatin of the resin. In summary, four components are needed to produce a usable resin 1- a glycol, 2- a diabasic organic acid (a proportion of that acid must contain an unsaturated group or a double bond), 3- a reactive monomer, 4- in the case of storing, an inhibitor.

Later, addition of heat and/or a free-radical initiator such as organic peroxide, causes a chemical reaction that results in non reversible cross-linking between the unsaturated polyester polymer and the monomer. This can be done at room temperature by using peroxides and the appropriate additives. Several types of commercial polyesters are available, the most common of which are:

1-Orthophthalic polyesters

They include phthalic anhydride and maleic anhydride, or fumaric acid. Orthophthalic polyesters are seldom used in structural applications because they do not have the strength, thermal stability, moisture resistance, or chemical resistance that other polyesters resins.

2-Isophthalic polyester

They include isophthalic acid and maleic anhydride or fumaric acid. Iso polyesters have better mechanical properties, greater moisture resistance, improved chemical resistance, and superior thermal resistance.

2.3.2- Vinyl ester resins

Vinyl ester resins were developed especially for corrosive environment applications. They are produced by reacting a monofunctional unsaturated acid with a bisphenol di-epoxide. They contain styrene as monomer and can be crosslinked by peroxide catalysts. They are mainly used wherever high chemical resistance is required for example pipes, and storage tanks.

2.3.3- Epoxy resins

Epoxy resins contain one or more epoxide groups or more generally glycidyl groups per molecule. The most common material used to produce epoxy is diglycidyl ether of bisphenol-A (DGEBA). The uncured resins vary from flowing liquids to high melting solids. A curing agent or a hardener is used to cross-link the resin into hard infusible materials with excellent electrical, chemical, and mechanical properties. The curing reaction is achieved by the addition of a suitable di- or poly-functional curing agent to the resin. One popular curing agent is diethylenetriamine (DETA). Hydrogen atoms in the amine group of the DETA react with the epoxide groups of DGEBA molecules causing the DGEBA molecules to cross-link producing the solid cured epoxy. Table 2.1 shows a comparison of properties between reinforced epoxy and selected metals.

Table 2.1 Comparison of properties between reinforced epoxy and selected metals [ACI Committee 440 (1996)]

Material	Density	Unidirectional tensile strength	
	gr./cm^3	Gpa	(10 ksi)
Carbon AS-4	1.55	145	(21.0)
Carbon HMS	1.63	207	(21.0)
S-Glass TM	1.99	59	(8.6)
E-Glass	1.99	52	(7.6)
Aramid	1.38	83	(12.0)
Aluminum (7075-T6)	2.76	69	(10.0)
Titaniun(6A1-4v)	4.42	114	(16.5)
Steel (4130)	8.0	207	(30.0)

Epoxy polymer matrix resins are much more expensive than polyester and vinyl ester resins, however, they provide the following characteristics:

1- No emission of volatile monomers during curing,

2- Excellent chemical properties,

3- Good adhesion properties,

4- A wide range of mechanical and physical properties can be obtained,

5- Low shrinkage during the curing process.

On the other hand, epoxy resins have several disadvantages, the most serious of which is that they are expensive, and must be carefully processed to maintain moisture resistance. They also require long curing time and special care during handling in order not to cause skin irritation.

Although most of the FRP used for reinforcing concrete was based on epoxy resins, it is most likely that new generations of FRP will heavily depend on polyesters because they are cheaper and have improved alkaline resistance and better retention of tensile elongation.

2.4-Types and properties of fibers

Reinforcing fibers are the principal load bearing constituent of any structural composite element. Typical types of structural fibers include aramid, carbon, glass, polyvinyl alcohol (PVA). Fibers are generally made from materials with the following properties:

1- high modules of elasticity.

2- high ultimate strength.

3- low variation of mechanical properties between individual fibers.

4-stability and retention of mechanical properties.

5- uniform fiber cross-section.

The most commonly fibers used in engineering applications are discussed below.

2.4.1-Glass Fibers

Glass has been the predominant fiber for many civil engineering applications because of its economical cost and specific strength properties. Glass fibers are available in: E-Glass, and S-Glass (high silica), and C-Glass (chemical grade).

Table 2.2 shows compositional ranges for commercial glass fibers.

E-glass: Is a general purpose type that offers good heat resistance and electrical properties. E-glass comprises approximately 80 to 90 percent of the glass fiber commercial production.

Table 2.2 Compositional ranges for commercial glass fibers (units = percent by weight) [From ACI Committee 440 (1996)]

	E-Glass range	S-Glass range	C-Glass range
Silicon dioxide	52-56	65	64-68
Aluminum oxide	12-16	25	3-5
Boric Oxide	5-10	–	4-6
Sodium oxide and potassium oxide	0-2	–	7-10
Magnesium oxide	0-5	10	11-25
Calcium oxide	16-25	–	0-1
Barium oxide	–	–	–
Zinc oxide	–	–	–
Titanium oxide	0-1.5	–	–
Zirconium oxide	–	–	–
Iron oxide	0-0.8	–	0-0.8
Iron	0-1	–	–

S-Glass: Is a proprietary magnesium alumino-silicate which is used wherever resistance to high heat is required. S-glass and S-2 glass have the same composition but different surface treatment. This type of fiber is the most expensive among glass fibers and is produced under specific quality control to meet military requirements.

C-Glass: A soda- lime -borosilicate composition which is used wherever resistance to chemical corrosion is required. It is usually used in the chemical processing industry.

Typical properties of glass fibers are summarized below followed by properties of commercially available glass fibers in table 2.3.

1- Glass fibers are elastic until failure

2- Low cost with respect to other fibers

3- Low tensile modulus

4- High tensile strength

5- Low resistance to cyclic loads

6- Softening point of about 800-1000 C

7- Sensitivity to alkaline environment

8- High insulating property

9- Surface active and hydrophilic

When glass fibers are under a constant load at stresses below the instantaneous strength, they usually fail at some point as long as stress is maintained above a minimum value. This is called creep rupture.

Table 2.3 Typical properties of commercial glass fibers [ACI Committee 440 (1996)]

Fiber	Typical diameter (microns)	Specific gravity	Tensile modulus GPa	Tensile strength GPa	Strain to failure (%)	Coefficient of thermal expansion	Poisson's ratio
E-Glass	10	2.54	72.4	3.45	4.8	5	0.2
S-Glass	10	2.49	86.9	4.3	5	2.9	0.22

2.4.2-Carbon Fibers

There are three sources for commercial carbon fibers: pitch, a by-product of petroleum distillation; PAN (polyacrylonitrile), and rayon. The properties of carbon fibers are controlled by molecular structure and the degree of freedom from defects. The formation of carbon fibers requires processing temperatures above 1000C.

Properties of carbon fibers are listed below:

1-High strength and stiffness to weight ratio.

2- Low ultimate strain

3- Low impact resistance

4- Low sensitivity to fatigue loads

5- Excellent chemical and moisture resistance

6- Sensitivity to galvanic corrosion

7- Thermal stability up to 2000C

8- Highly conductive to heat and electricity

2.4.3- Aramid fibers

Several organic fibers can be used in structural applications, the most common of which is aramid. It is poly-para-phenyleneterephthalamide, known as PPDT. Properties of aramid fibers are listed below:

1- Tensile properties decrease by 5% when moisture exists.

2- Good resistance to chemical attack.

3- Low electrical conductivity.

4- Tensile strength ranges between 2414-3150 MPa.

5- Elastic modulus ranges between 62-142 GPa.

2.4.4- Polyvinyl alcohol

PVA is obtained from saponification of polyvinyl acetate. Properties of PVA are summarized below:

1- They are not expensive.

2- Good resistance to salt, alkalis, and acids.

3- Low stiffness.

4- Elastic modulus ranges between 8 - 28 GPa.

5- Tensile strength is in the range of 870 to 1350 MPa.

2.4.5- Hybrid reinforcement

If different types of fibers used together, we have hybrid reinforcement. One of the most common cases where we have carbon and glass fibers impregnated by polyester resin. The higher the ratio of carbon to glass, the higher the cost but the better the properties which include tensile strength, modulus of elasticity, flexural strength, flexural modulus, interlaminar shear.

2.5- Types and properties of FRP products

2.5.1- 1-D FRP reinforcing

Under 1-D reinforcement, there are several products including, reinforcing bars, gripping devices for concrete formworks, and prestressing tendons.

2.5.1.1- Reinforcing bars

Several companies in North America are marketing FRP reinforcing bars, intended for non prestressed reinforcement. Most of the available products contain E-glass fibers with choice of thermoset resin.

2.5.1.2- Prestressing tendons

Several types of tendons are commercially marketed worldwide.

2.5.2- 2-D FRP reinforcement

FRP gratings and grid-type are used in the North America and Japan for two dimensional reinforcement. NEFMAC is a 2-D grid-type FRP reinforcement consisting of glass and carbon fibers impregnated with resin. In Canada, research was done by Rahman et al (ACI Committee, 1996), to investigate its suitability for reinforcing barrier walls and bridge decks in the Canadian climate.

2.5.3- 3-D FRP reinforcement

3-D systems are now successfully produced in Japan (ACI Committee 440, 1996). Products include 3-D cage where 2-D trusses are manufactured and then combined into 3-D cages using a combination of Pultrusion and filament winding. Another product is 3-D fabric that is used as a reinforcement for louvers and curtain walls.

2.5.4- FRP products for external reinforcement

Several companies in North America and Japan are producing FRP fibers and plates that are used for the external reinforcement of concrete members. The most commonly used composites in repairing and upgrading concrete members are carbon or glass fiber sheets or plates used with epoxy or polyester resins.

CHAPTER III

EXPERIMENTAL PROGRAM

3.1- Introduction

An experimental program was undertaken to study the behavior of concrete elements strengthened or upgraded using GFRP sheets. The first step of the experimental work was to perform tension tests on samples of the GFRP sheets to determine it's experimental strength and Young's modulus. The obtained values were used in the finite element analysis of the beams and in the analytical calculations as will be shown in the chapter IV. The second step studying concrete cylinders wrapped with GFRP sheets. This was a step to check the potential of the strengthening system before testing reinforced concrete members. The results obtained from testing the cylinders were promising which encouraged moving to the third step that is studying the effect of GFRP sheets on the shear and flexural loading capacities of damaged reinforced concrete beams.

3.2- GFRP composite used in the research

The material used in strengthening concrete members in this research is Tyfo S Fibrwrap manufactured by Hexcel Fyfe Corporation. The main constituents of that system are Tyfo S epoxy and woven fiberglass sheets commercially produced under the name SEH 51.

The fiberglass sheet is made of continuos filaments made of E-glass that are drawn or pulled together in a batch of molten glass through precise multi-hole brushing and combined into strands. The strands are combined to form yarns see fig(3.1). The yarns are, then, interlocked over and under each other in an alternating fashion, this is called plain weave see fig (3.2). The GFRP sheet is cut to any required size using a scissors see fig(3.3). The material used in this research is produced by Hexcel Fyfe. Throughout this chapter, epoxy refers to Tyfo S epoxy and GFRP sheets refers to SEH 51.

Tyfo S epoxy consists of two parts namely hardener and resin. The supplier's instructions were to mix the two components with the ratio of 100 : 42 by weight of the resin and hardener respectively. This should be done using an electric mixer for at least 5 minutes. The motor speed must not be less than 600 rpm. There was no such mixer available, therefore, the epoxy components were manually mixed for all purposes of this research.

Fig(3.1) Yarn construction

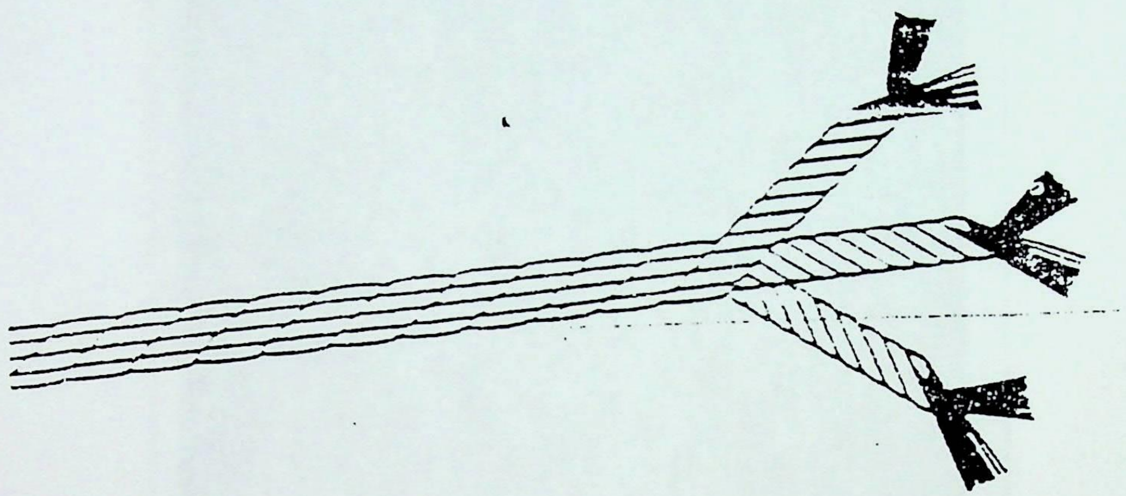


Fig (3.2) SEH51 plain weave sheets

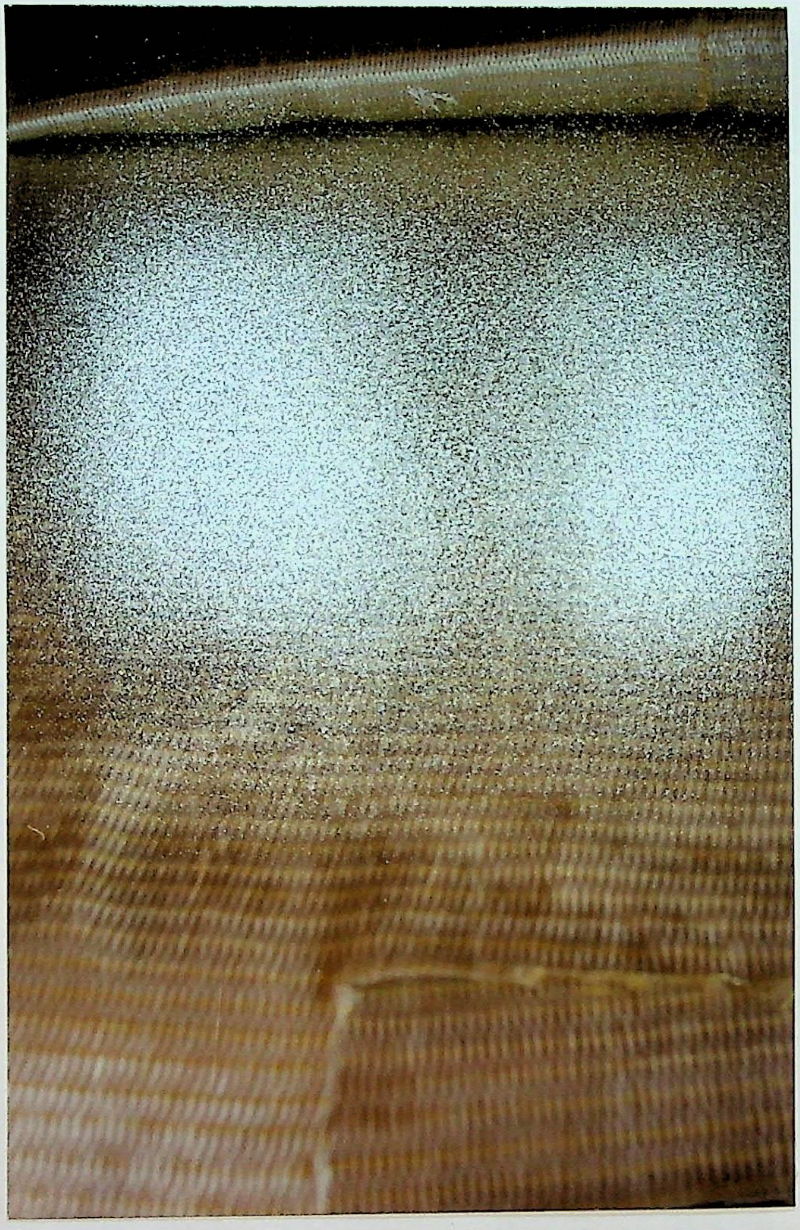


Fig (3.3) Scissors used to cut GFRP to required size

3.3- Tensile properties of fiber resin composite

3.3.1- Specimen preparation

The GFRP sheet was cut to the required size (using normal scissors). The two components of the epoxy were mixed and used to impregnate the GFRP sheet. A roller was manually applied over the soaked composite to get rid of excess epoxy and to ensure impregnation. Samples were prepared using 2 layers of GFRP sheets. Extra layers were used in the gripping area as shown in fig (3.4) in order to ensure failure within the gage length. The specimens were left to cure on a polyethylene sheet for 7 days.

3.3.2- Instrumentation and test procedure

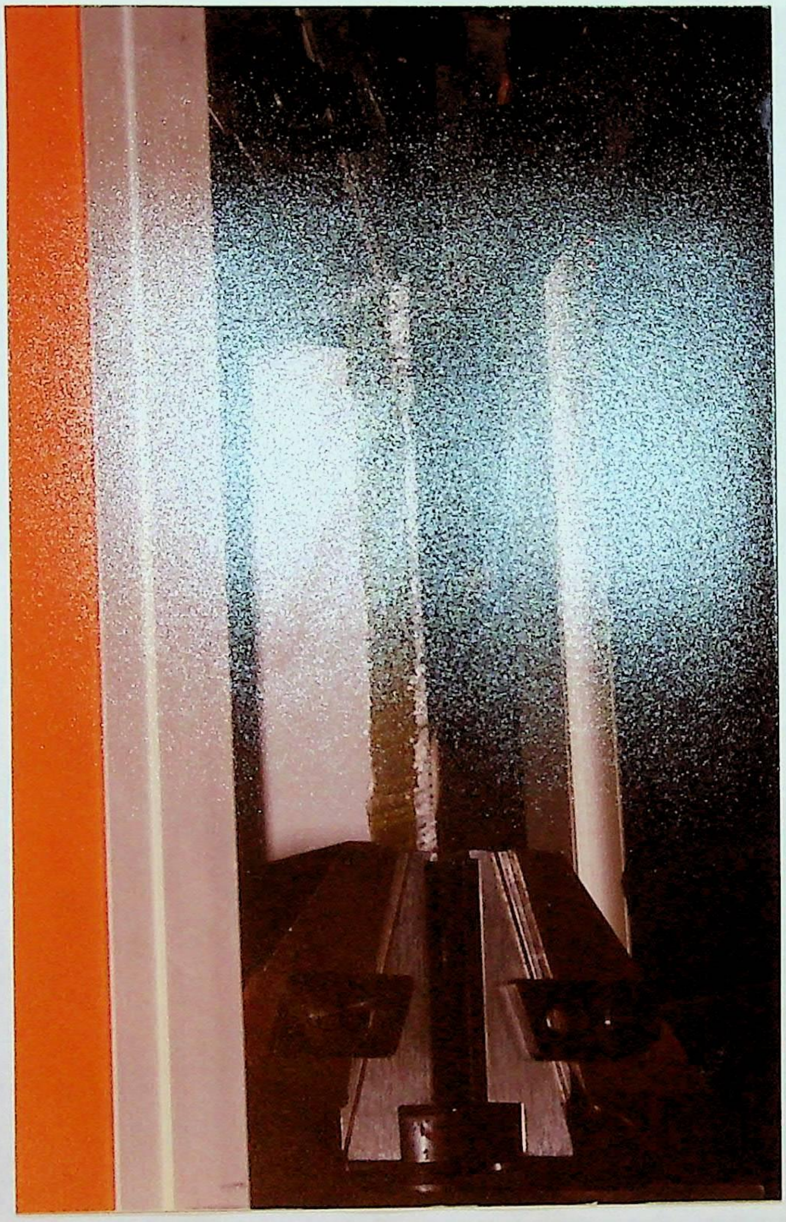
The test was performed according to ASTM D3039, "Standard Test Method for Tensile Properties of Fiber-Resin Composites". The specimens were zero degree oriented and tested parallel to the major glass direction of the SEH 51 glass using a universal testing machine as shown in fig(3.5). Stress versus strain curves for all the specimens were obtained.

3.4- Compression test on cylinders

3.4.1- Casting of Cylinders

The concrete used for casting the cylinders was designed to have an ultimate compressive strength of 300 Kg/cm^2 at 28 days. The mix proportions by weight were 0.5:1:0.3 for fine aggregate : coarse aggregate : cement, and water per cement ratio of

Fig(3.4) Extra layers in gripping area to ensure failure within gage length



Fig(3.5) Univesal testing machine used to perform tension tests on GFRP

0.5. Because of a defect in the cement used, the maximum compressive strength that the control cylinders reached was 160 Kg/cm^2. Since the same bag of cement was used for pouring all the cylinders, comparative results between compressive strength of the control cylinders to those of the cylinders wrapped by GFRP sheets are still valid. Cylinders were left to cure for 28 days.

3.4.2- Wrapping of cylinders

The cylinders were divided into 4 groups. The first group consisted of cylinders C1, C2, C3, the second group of 3 cylinders C4, C5, C6, the third group of cylinders C7, C8, finally the fourth group of cylinder C9 . The wrapping process was done manually. The GFRP sheet was cut to the required size of the cylinder using a scissors. The cut sheets were soaked in the epoxy and then squeezed very well to get rid of excess epoxy. Finally the sheets were manually wrapped around the cylinders. Hand pressure was applied to get rid of air bubbles and excess epoxy as much as possible. The first group of cylinders was used for control. The second group was strengthened using 1 layer of GFRP sheet wrapped around the cylinder with the main direction of the fiber jacketing the cylinder, similarly, the third group strengthened using 2 jacketing layers and the fourth strengthened using 3 jacketing layers. The wrapped cylinders were left to cure (epoxy curing) for 7 days on polyethylene sheets as shown in fig (3.6).

3.4.3- Instrumentation and test procedure

The compressive strength of the concrete cylinders was determined according to ASTM C39.

Fig(3.6) Wrapped cylinders left for epoxy curing on polyethelene sheets for 7 days

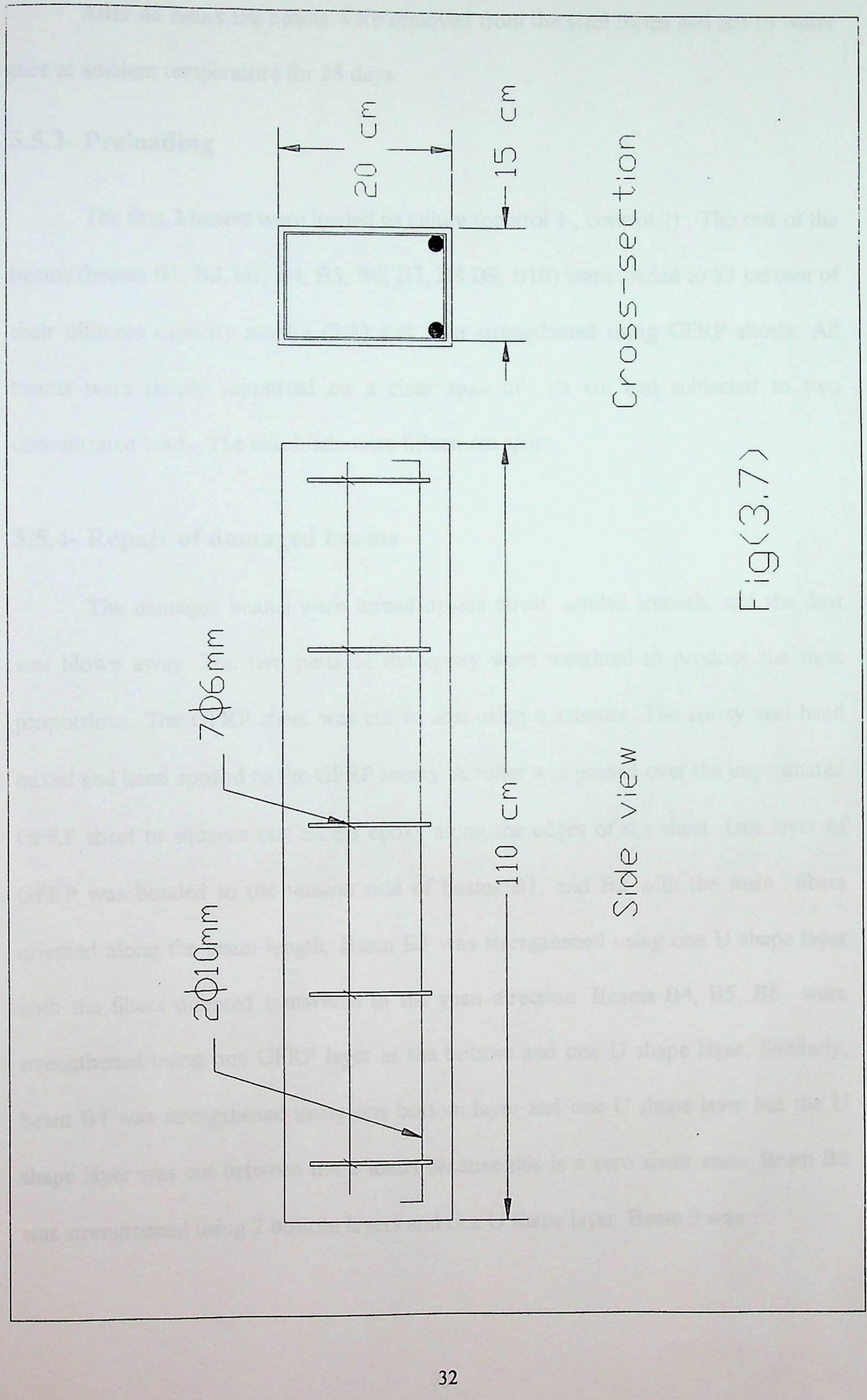
3.5- Tests on damaged beams

3.5.1- Design of concrete beams

The design of the concrete beams was carried out according to the 1995 Egyptian Code of Practice (using the ultimate design method. Steel reinforcement was chosen far below the maximum limit (As/bd=0.006) to allow for external reinforcement using FRP without having a brittle compression failure. The dimensions of the beam are 15 cm wide by 20 cm deep by 110 cm long. The tested span was 98 cm. The internal reinforcement consisted of 2 φ 10 mm steel bars (10 mm diameter, and 4200 Kg/cm^2 yield stress). Shear reinforcement consisted of 7 φ 6 mm stirrups along the whole length of the beam see fig(3.7).

3.5.2- Casting of beams

12 reinforced concrete beams (15 x 20 x110) were cast in steel forms for the research program. After assembling the steel forms, the surfaces were oiled for easy removal of the concrete beam. Then, the steel reinforcing cage was assembled and placed in the form. Twelve batches of concrete (one for each beam) were needed to fabricate the beams. The mix proportions by weight were 0.5 : 1 : 0.3 for fine aggregate : coarse aggregate : cement respectively with water per cement ratio of 0.4. Before placing the concrete in the form, the steel cage was checked for accuracy in position and dimension. An electrical vibrator was used to vibrate the beams. During final stages smooth surface finishing was done at the exposed surface of the beam. Concrete test cylinders were cast with every beam and the average compressive strength of the cylinders was 300 Kg/cm^2.



Fig(3.7)

After 48 hours the beams were removed from the steel forms and left to water cure at ambient temperature for 28 days.

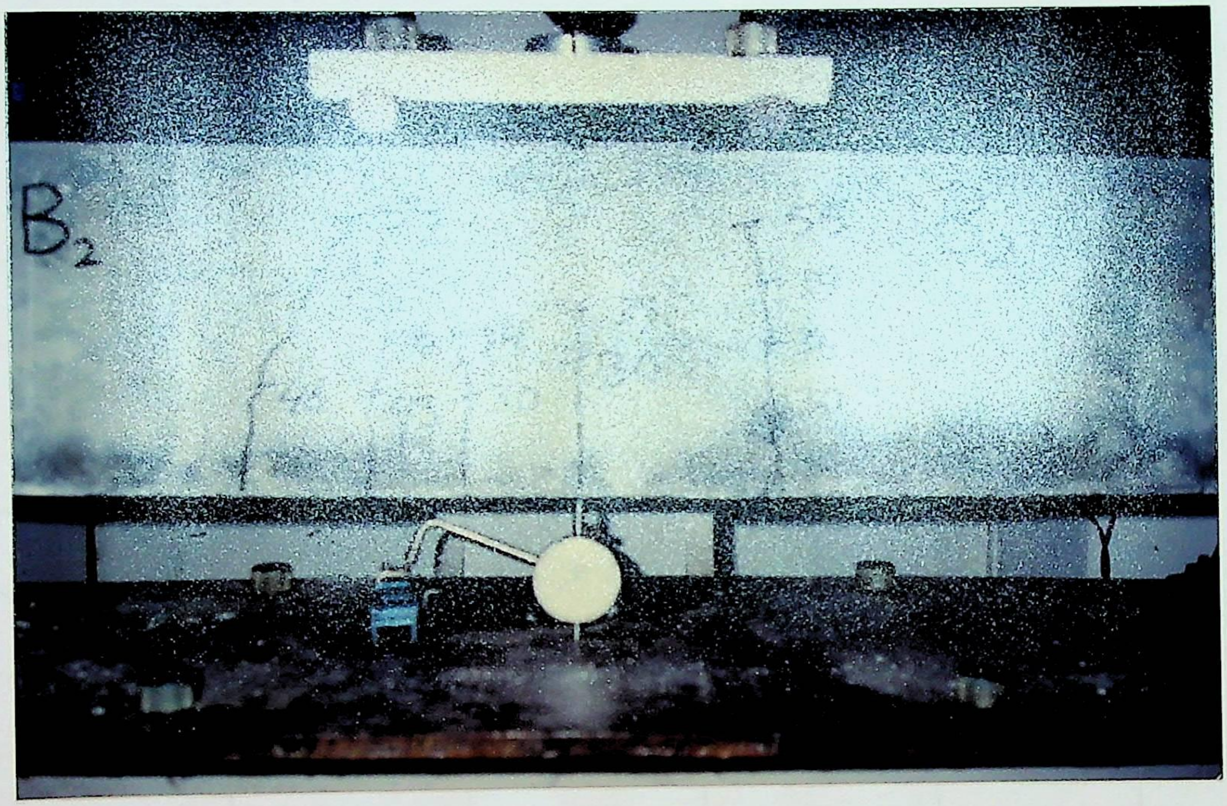
3.5.3- Preloading

The first 2 beams were loaded to failure (control 1 , control 2) . The rest of the beams (beams B1, B2, B3, B4, B5, B6, B7, B8 B9, B10) were loaded to 85 percent of their ultimate capacity see fig (3.8) and then strengthened using GFRP sheets. All beams were simply supported on a clear span of 98 cm and subjected to two concentrated loads. The two loads were fifteen cm apart.

3.5.4- Repair of damaged beams

The damaged beams were turned upside down, sanded smooth, and the dust was blown away. The two parts of the epoxy were weighted to produce the right proportions. The GFRP sheet was cut to size using a scissors. The epoxy was hand mixed and hand applied to the GFRP sheets. A roller was passed over the impregnated GFRP sheet to squeeze out excess epoxy along the edges of the sheet. One layer of GFRP was bonded to the tension side of beams B1, and B2 with the main fibers oriented along the beam length. Beam B3 was strengthened using one U shape layer with the fibers oriented transverse to the span direction. Beams B4, B5, B6 were strengthened using one GFRP layer at the bottom and one U shape layer. Similarly, beam B7 was strengthened using one bottom layer and one U shape layer but the U shape layer was cut between the 2 loads because this is a zero shear zone. Beam B8 was strengthened using 2 bottom layers and one U shape layer. Beam 9 was

Fig(3.8) Preloading of beams to 85% of ultimate load

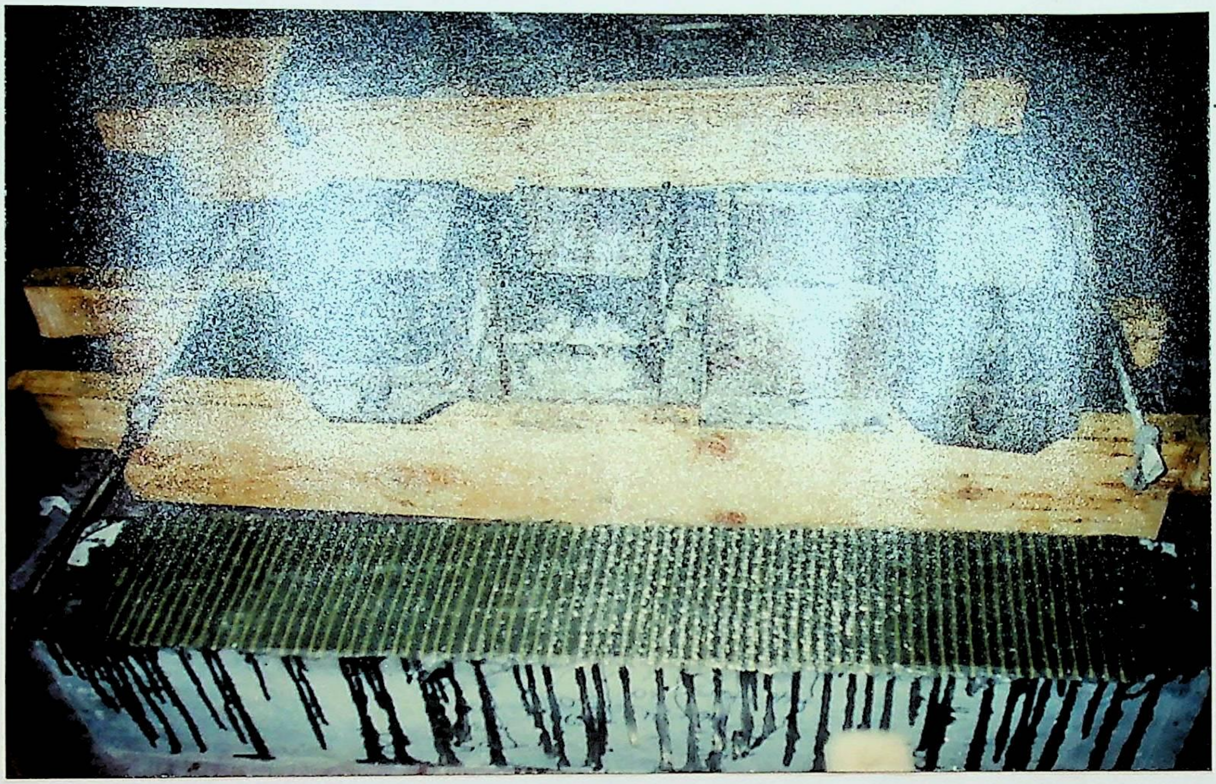


strengthened using 3 separate layers, one layer of GFRP was bonded to the tension side and 2 separate layers bonded to the 2 sides of the beam with the main direction of the fiber along the length of the beam. Finally, Beam B10 was strengthened using 2 bottom layers and 2 U shape layers. The beams were left to cure (epoxy curing) for 7 days. 2 wooden sheets were tied using steel rods to the sides of the beams to get rid of air bubbles and excess epoxy see fig(3.9). Table 3.1 summarizes the reinforcement of the beams.

Table 3.1- Reinforcement of beams using external GFRP sheets

Beam	Pre-loading Level	Bottom Layers	U-Shape Layer	Side Layers
B1	85%	1	0	0
B2	85%	1	0	0
B3	85%	0	1	0
B4	85%	1	1	0
B5	85%	1	1	0
B6	85%	1	1	0
B7	85%	1	1	0
B8	85%	2	1	0
B9	85%	1	0	2
B10	85%	2	2	0
Control 1	100%	0	0	0
Control 2	100%	0	0	0

Fig(3.9) wooden sheets to get rid of excess epoxy and air bubbles



3.5.5- Instrumentation and test procedure

The test procedure consisted of loading all beams to failure in 2 point bending using universal testing machines. Vertical deflections were measured at the mid-span. The load versus deflection curves were obtained.

Chapter IV

FINITE ELEMENT ANALYSIS

4.1-Introduction

The finite element method, a numerical procedure for solving differential equations, is one of the most effective and flexible techniques for simulating reinforced concrete members. This method provides a description of the structural response within both the elastic and post elastic loading conditions.

The finite element method was used in analyzing some of the beams that were strengthened using GFRP sheets. A description of the finite element technique will be presented in this chapter. Also, the procedure for modeling the various components of the GFRP strengthened beams will be provided in later sections of this chapter.

The package ABAQUS was used in analyzing the performance of some of the beams upgraded using GFRP in order to compare theoretical and experimental results. ABAQUS is a multipurpose finite element program produced and distributed by the Electric Power Research Institute (EPRI) under the title ABAQUS-EPGEN. The program is commonly used in solving structural problems.

4.2- The ABAQUS computer program input file

The ABAQUS computer program is a multipurpose finite element program. A data deck for ABAQUS consists of model and history data. History data provides the sequence of events for which the model is analyzed. Model data provides the finite element model. The model includes nodes, elements and their properties, constraints, and boundary conditions.

4.3- Finite element modeling of beams using ABAQUS

A three dimensional finite element model was used to represent the tested beams. beams were divided into 3 main components; the concrete, reinforcing steel, and the GFRP sheets. The program has many built in elements within its library. The element used in modeling each of the 3 components is explained in detail below.

4.3.1- Modeling of concrete

The ABAQUS has a number of three-dimension built in solid elements in its library. C3D8, an eight node element, was used to model the concrete beams in this research. C3D8 has 3 displacement and 3 rotation degrees of freedom at each node. In other words, six degrees of freedom at each node. The element took the shape of a rectangular prism.

4.3.2- Modeling of GFRP sheets

GFRP sheets were modeled using the built in S4R5 which is a four-nodes shell element with six degrees of freedom. S4R5 has 3 displacement and 3 rotation degrees of freedom at each of it's 4 nodes.

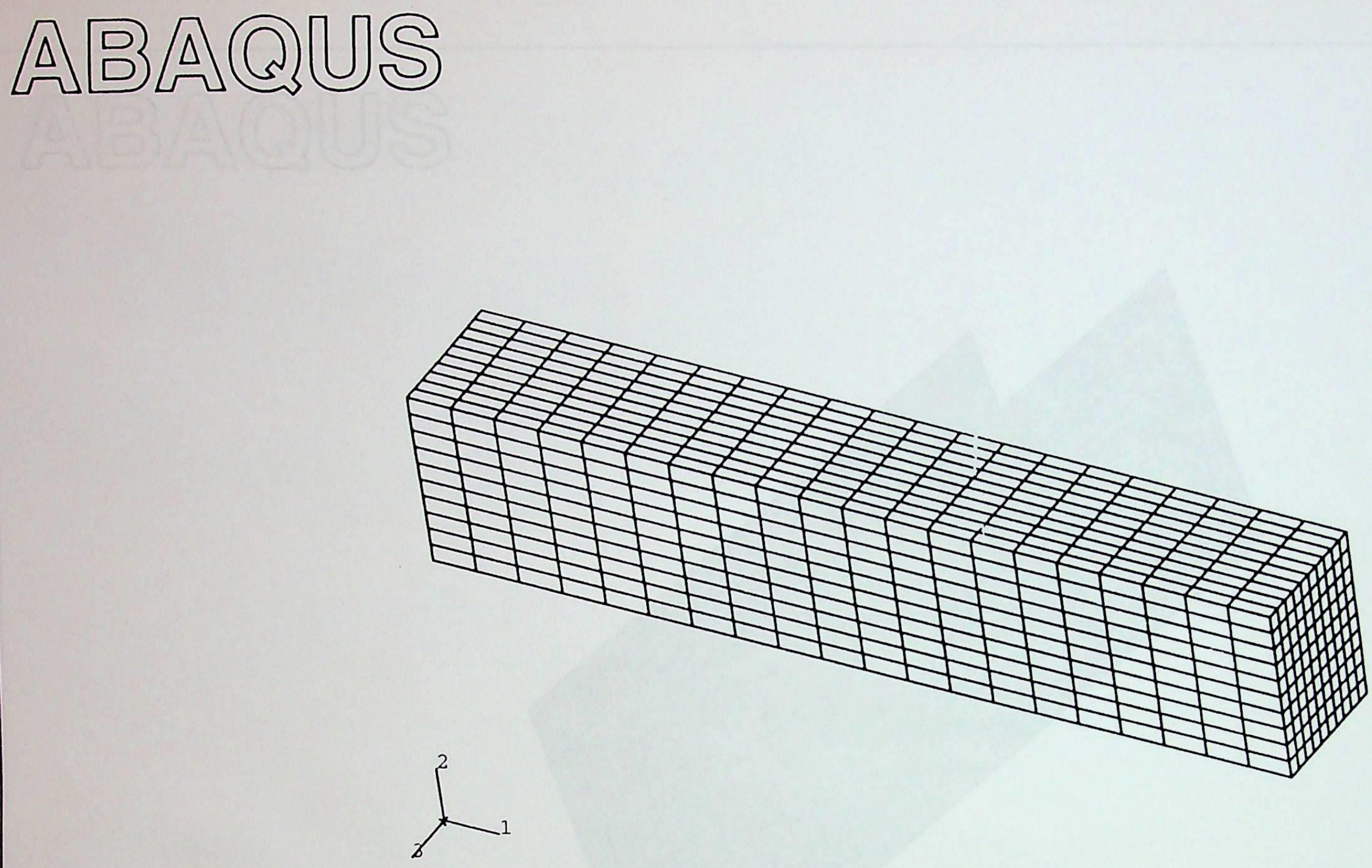
The uniaxial GFRP sheets used have 2 different properties in 2 perpendicular directions. The properties of the GFRP material were modeled using orthtropic material named LAMINA in ABAQUS. The failure of the GFRP sheets takes place suddenly because the stress-strain curve is a straight line and there is no yield strength as will be shown in chapter 5.

4.3.3- Modeling of reinforcing steel bars

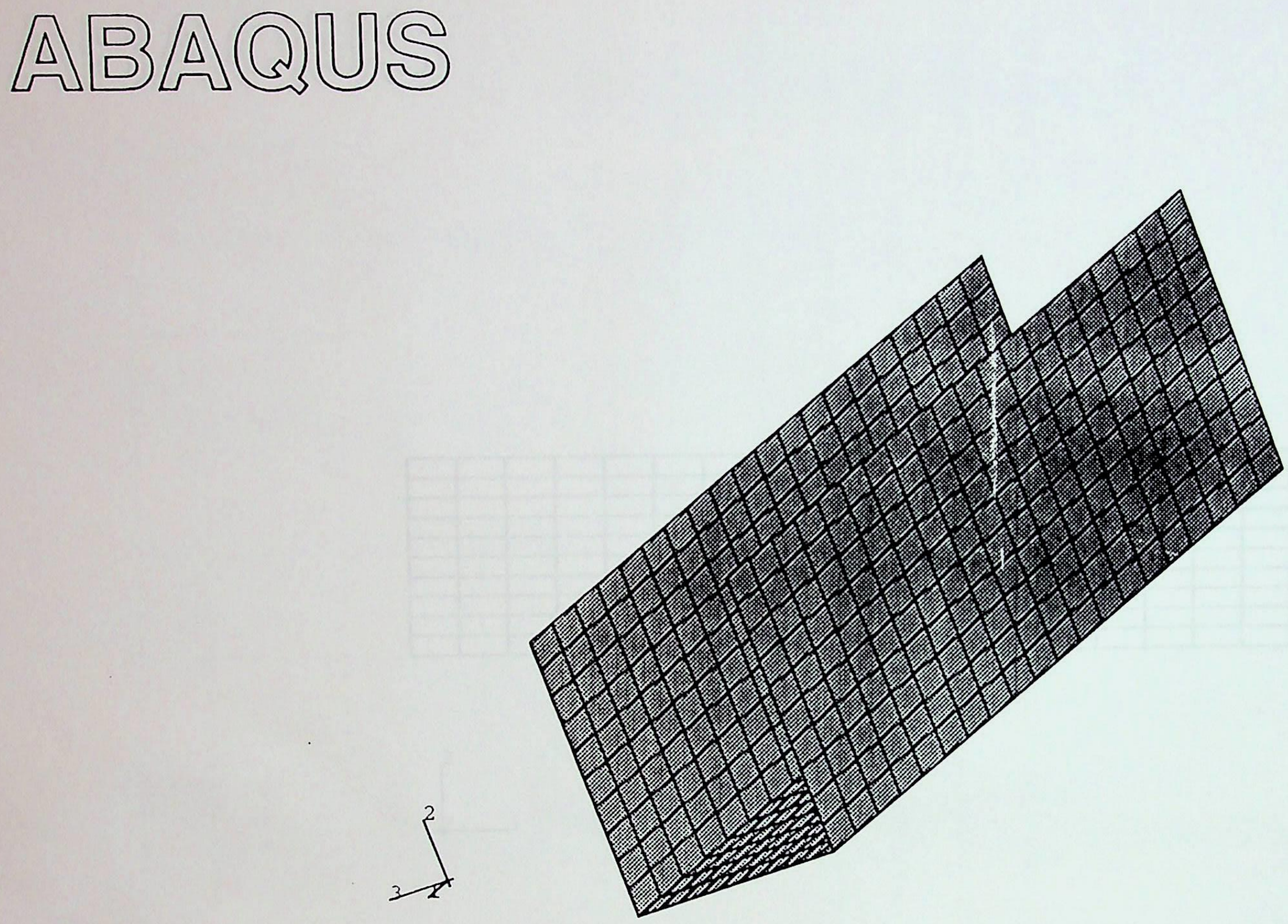
The ABAQUS built in REBAR option was used in modeling the reinforcing steel bars. Reinforcing bars were superimposed on the concrete mesh in the required position. The TENSION STIFFING built in option was used to simulate bond slip, and energy release with the propagation of cracks.

4.4- Verification of experimental results

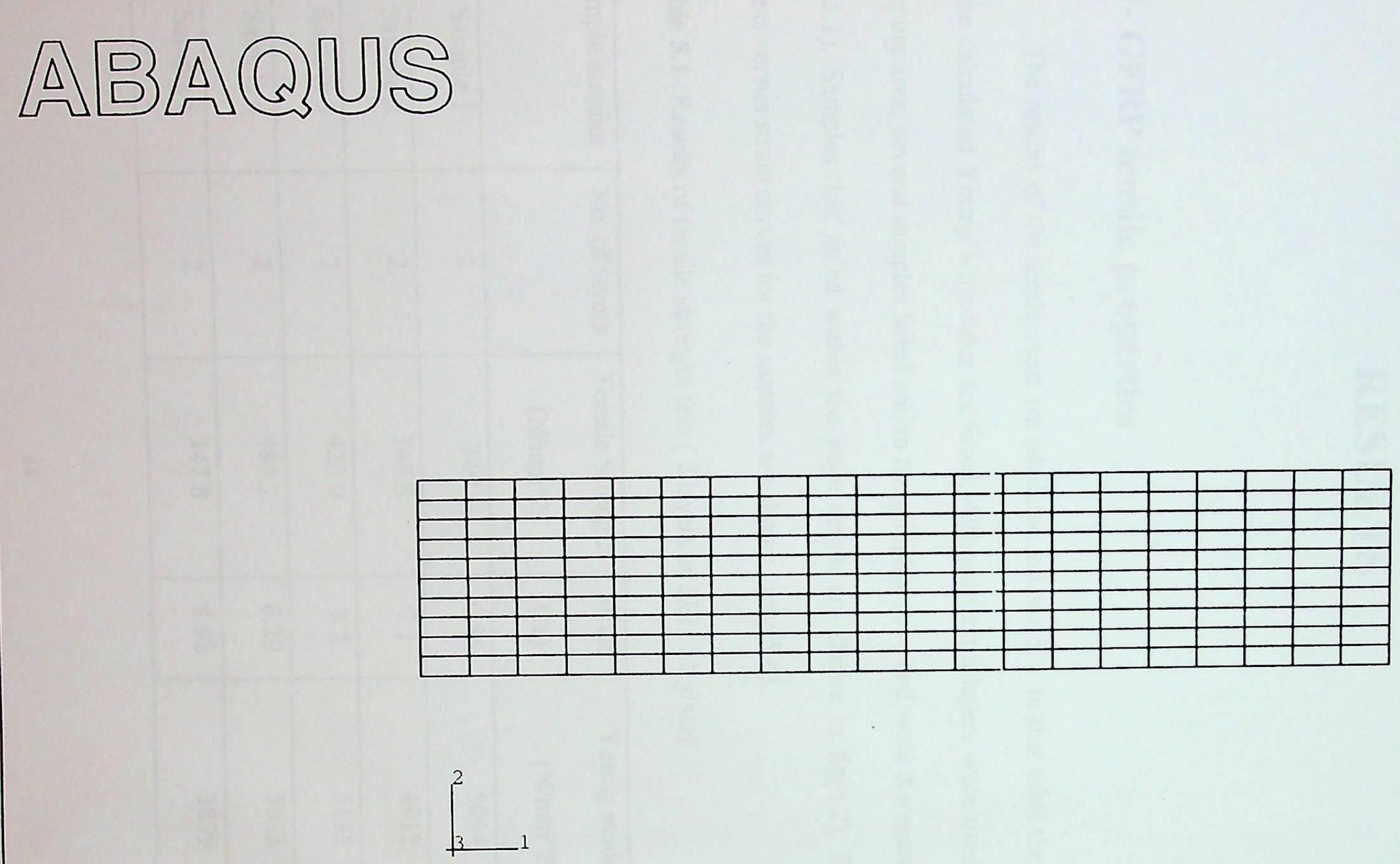
Finite element analysis using ABAQUS was conducted in order to compare experimental and theoretical results. The theoretical study included beams reinforced using one U shape layer and one bottom layer, which was the case of beams B4, B5, B6. and B7 respectively. Typical finite element meshes for the concrete, U shape GFRP, and bottom layer GFRP are shown in figures 4.1, 4.2, and 4.3 respectively.



Fig(4.1) Finite element mesh for concrete using ABAQUS



Fig(4.2) Finite element mesh for U shape GFRP sheet using ABAQUS



Fig(4.3) Finite element mesh for bottom layer GFRP sheet using ABAQUS

CHAPTER V

EXPERIMENTAL AND THEORETICAL RESULTS

5.1- GFRP tensile properties

The results of the tension test are tabulated in table 5.1. In that table the values of the calculated Young's modulus are listed. Although extra layers were used in the gripping area, several samples failed within the gripping zone and were disregarded see fig(5.1). Samples that failed within the gage length are shown in fig(5.2), fig(5.3). Stress versus strain curves for the samples are shown in fig(5.4).

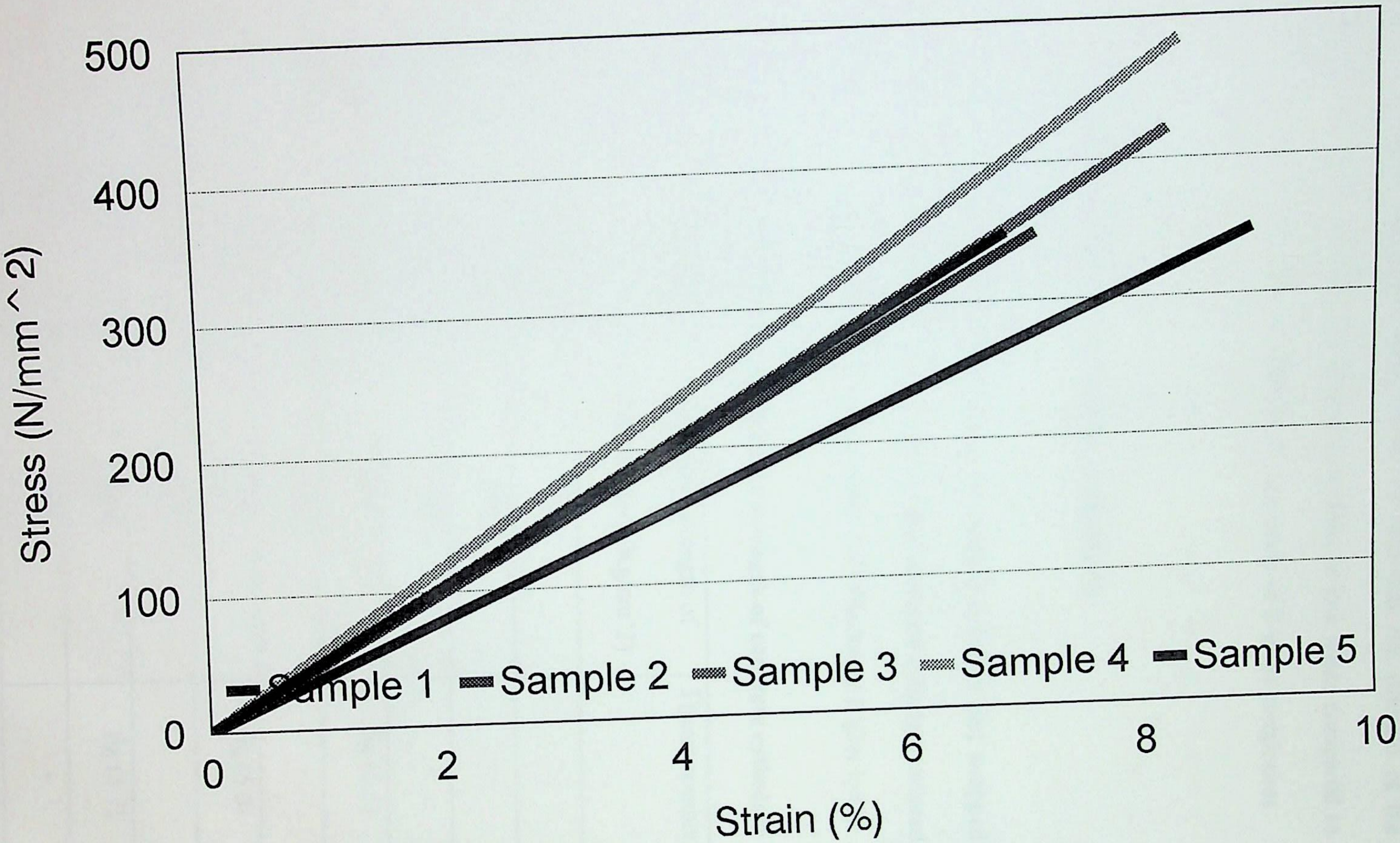
Table 5.1- Results of tensile strength test (2 layers of SEH 51 glass)

Sample number	No. of layers	Tensile Strength (N/mm^2)	Strain (%)	Young modulus (N/mm^2)
Sample 1	2	349	6.85	5094
Sample 2	2	348.8	7.1	4912
Sample 3	2	420.9	8.2	5132
Sample 4	2	484.2	8.29	5838
Sample 5	2	347.8	8.92	3899

Fig (5.1) Tensile failure inside the grip

Fig(5.2) Tensile failure of composite within gage length

Fig(5.3) Tensile failure of composite within gage length



Fig(5.4) Stress strain curves of GFRP samples shown in table 5.1

From the experimental test results, it can be seen that the strength of the composite is close to that of steel, but Young's modulus is low when compared to that of steel. Relatively low modulus of elasticity is a property of E-glass composites.

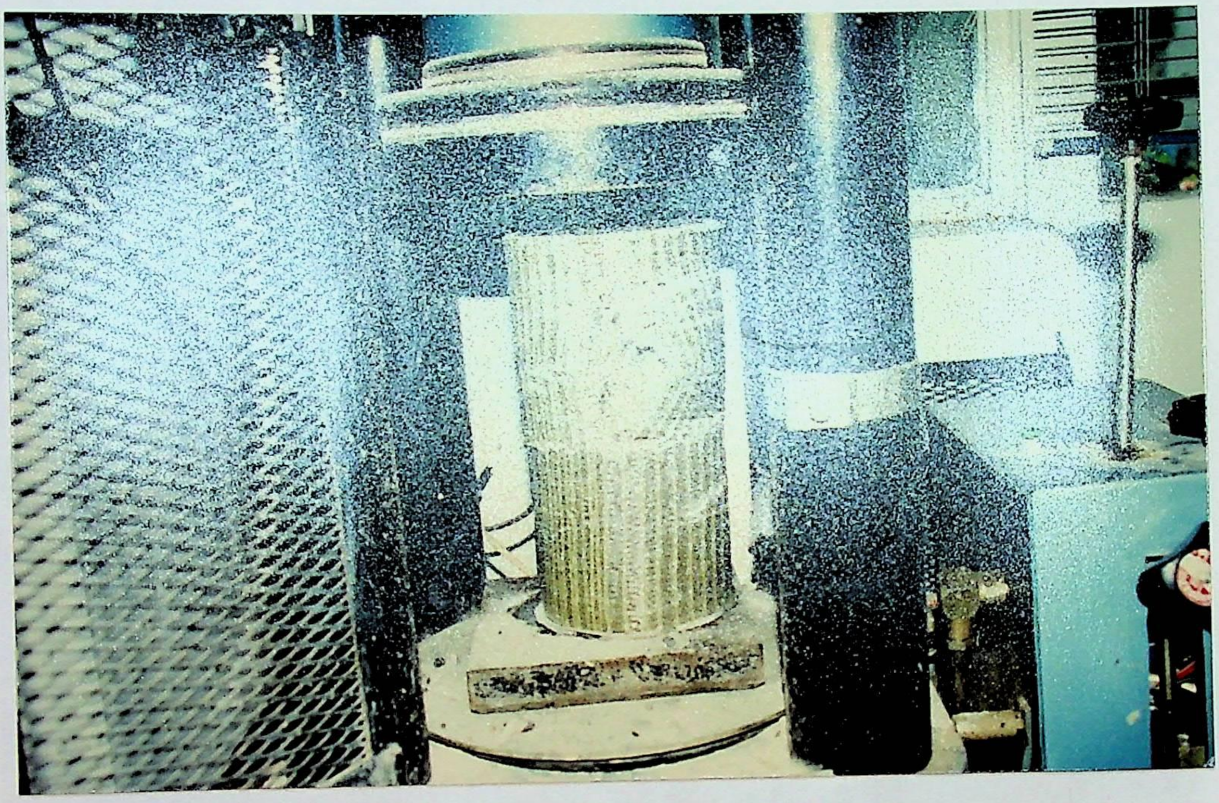
5.2- Compression tests on concrete cylinders

The results of the compressive strength of the control cylinders and wrapped cylinders is listed in table 5.2. The failure of all the wrapped concrete cylinders initiated by the crushing of the concrete followed by failure of the GFRP jacket at higher loads.

Table 5.2- Experimental results of the compressive strength of concrete cylinders

Cylinder No.	No. of GFRP layers	Compressive strength of concrete cylinders (Kg/cm^2)	Figure number
C1	0	172	-
C2	0	168	-
C3	0	169	-
C4	1	441	fig (5.5)
C5	1	452	-
C6	1	475	fig (5.6)
C7	2	633	-
C8	2	622	fig (5.7)
C9	3	973	-

Fig(5.5) Cylinder C4 at failure during compressive strength test



Fig(5.6) Cylinder C6 at failure during compressive strength test

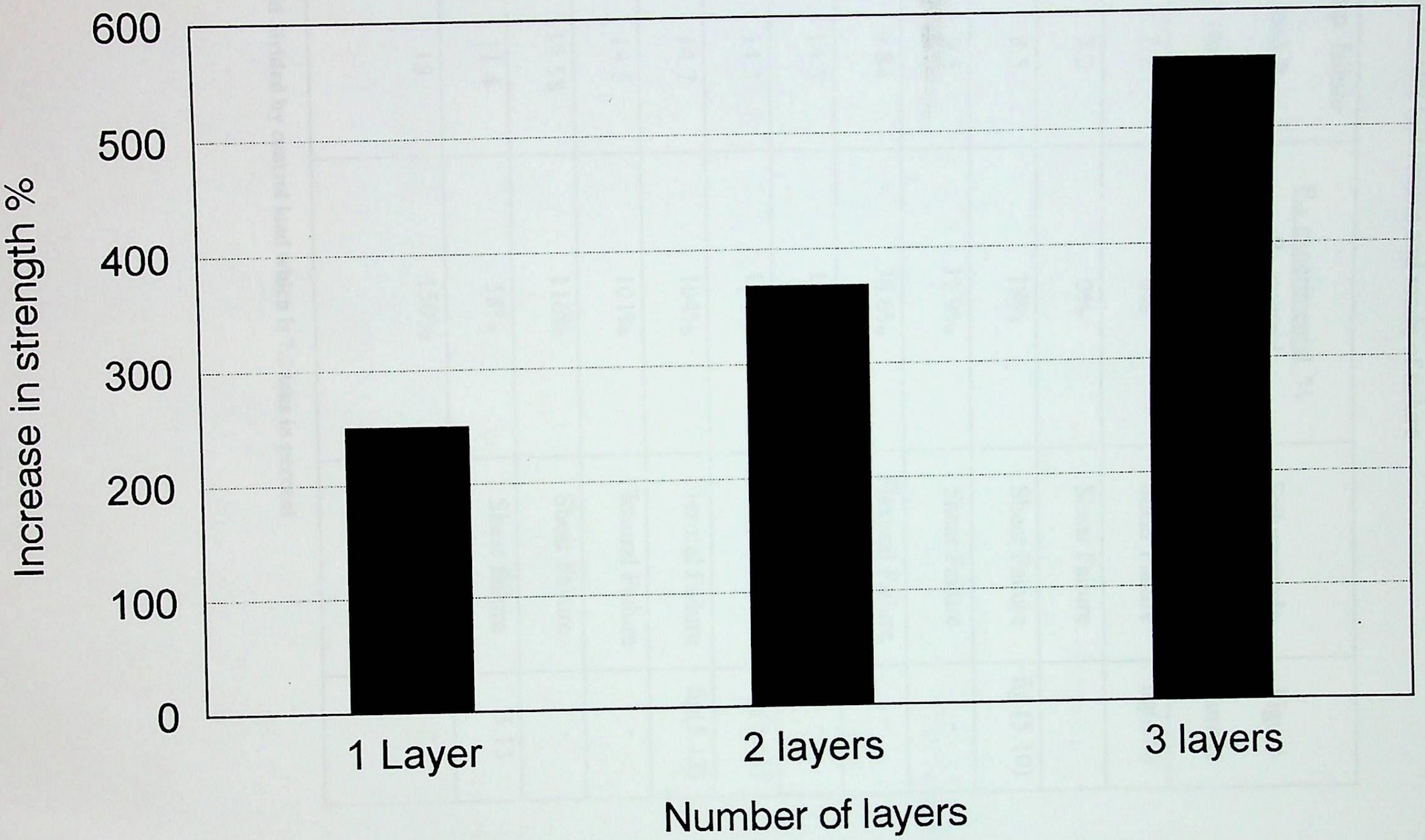
Fig (5.7) Cylinder C8 at failure during compressive strength test

The obtained results illustrate the ability of external GFRP jackets to greatly improve the compressive strength of concrete cylinders. One GFRP layer increased the compressive strength by around 250%, 2 layers by 368%, and 3 layers by 560% see fig(5.8).

The cylinders strengthened using GFRP had more than 1 plane of failure, see cup and cone failure of cylinder C8. Several planes of failure are a result of the confining effect of GFRP which holds cracks and prevent them from causing failure until the composites reaches it's ultimate strength.

5.3- GFRP repair of damaged beams

As previously mentioned, the first 2 beams were loaded to failure for evaluating cracking, yielding and ultimate loading. The control beams were found to develop flexural tensile cracks in the constant moment region at loads starting from 3 tons. At around 5 tons, a flexural tensile crack starts outside the constant moment region. Finally a shear crack initiates and flexural shear cracks cause failure of the control beam. Table 5.3 summarizes the results of the 2 point bending experiments on the control and the repaired beams (loaded to 85 % and repaired using GFRP as explained in previous chapter).



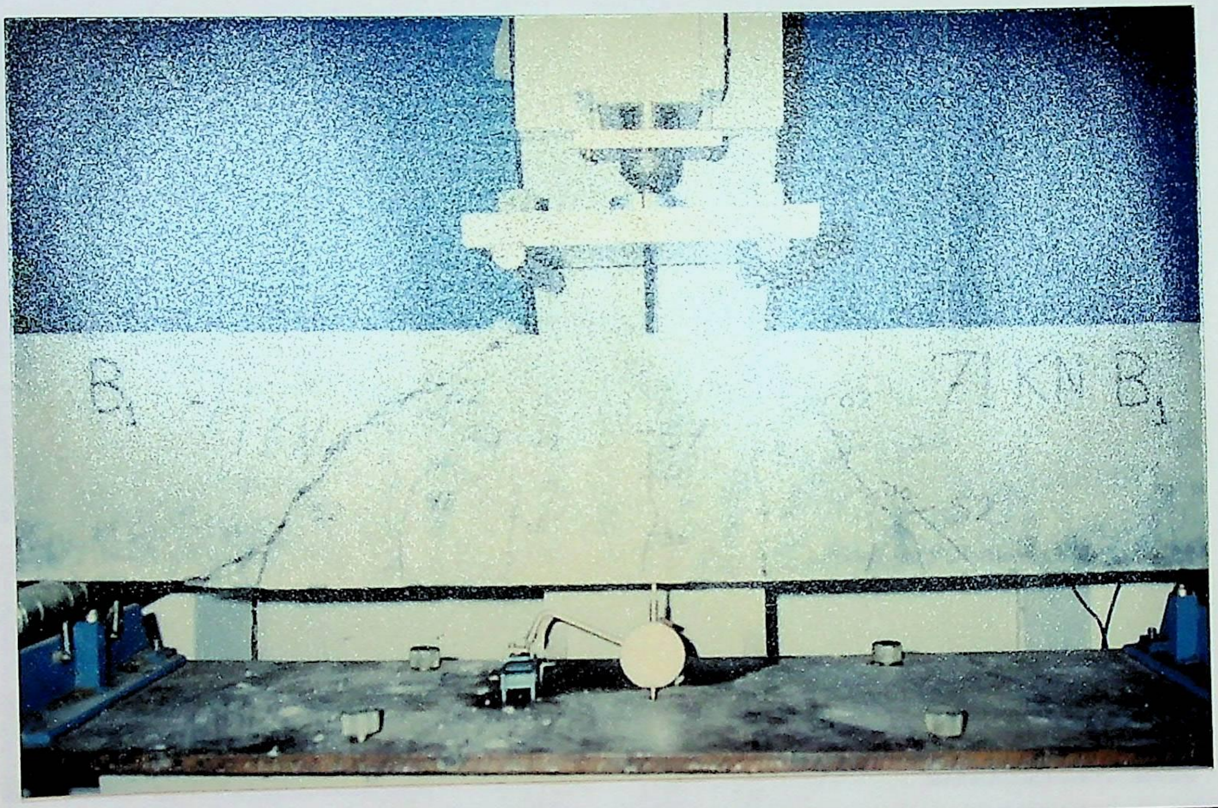
Fig(5.8) Increase in compressive strength of wrapped cylinders over control

Table 5.3- Experimental test results and mode of failure

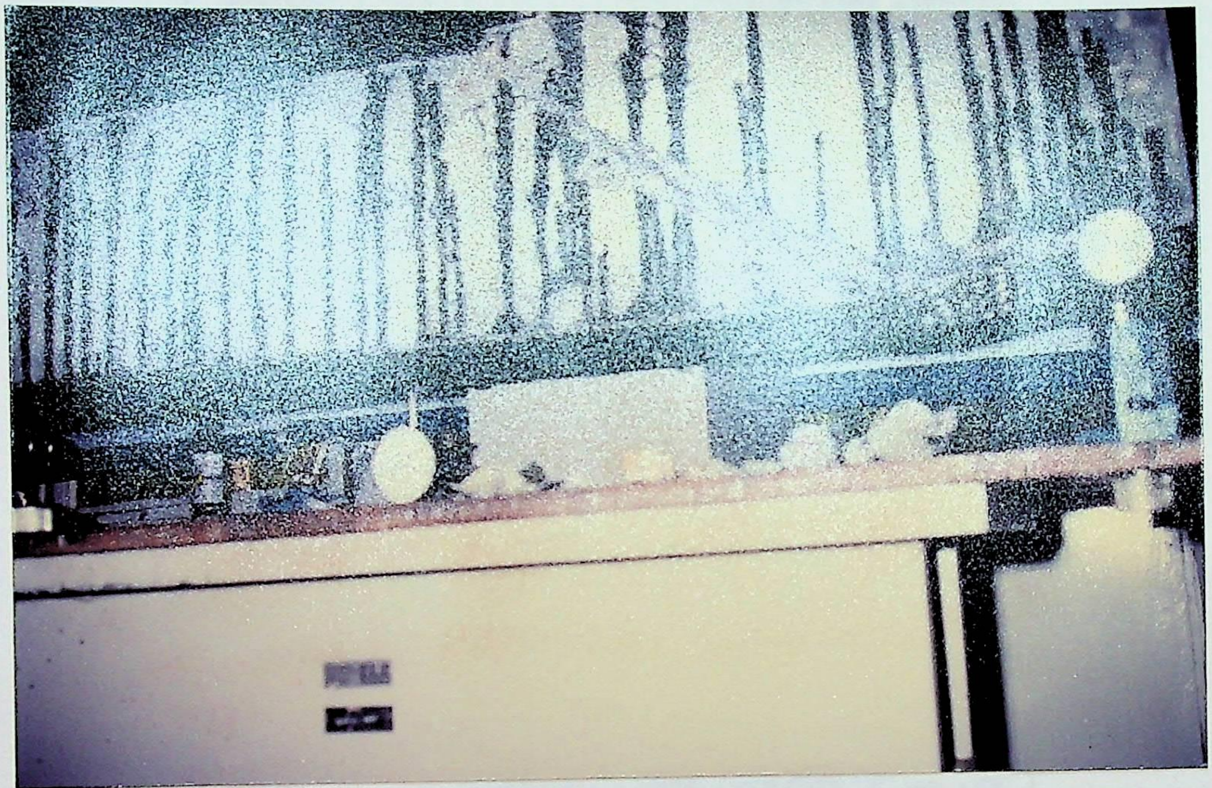
Beam	Exp. failure load P_{ult} (ton)	$\frac{P_{ult}\ \text{experimental}}{P_{ult}\ \text{control}}$ %	Failure mode	Figure number
Control 1	7.1	0%	Shear Failure	fig(5.9)
Control 2	7.2	0%	Shear Failure	
B1	8.5	18%	Shear Failure	fig (5.10)
B2	9.5	31.9%	Shear Failure	-
B3	9.84	36.6%	Flexural Failure	-
B4	14.5	104%	Flexural Failure	-
B5	14.3	104%	Flexural Failure	fig(5.11)
B6	14.7	104%	Flexural Failure	fig(5.12)
B7	14.5	101%	Flexural Failure	-
B8	15.58	116%	Shear Failure	-
B9	11.4	58%	Shear failure	5.13
B10	18	150%	Compression shear Failure	-

*** Maximum load divided by control load which is 7.2 tons in percent**

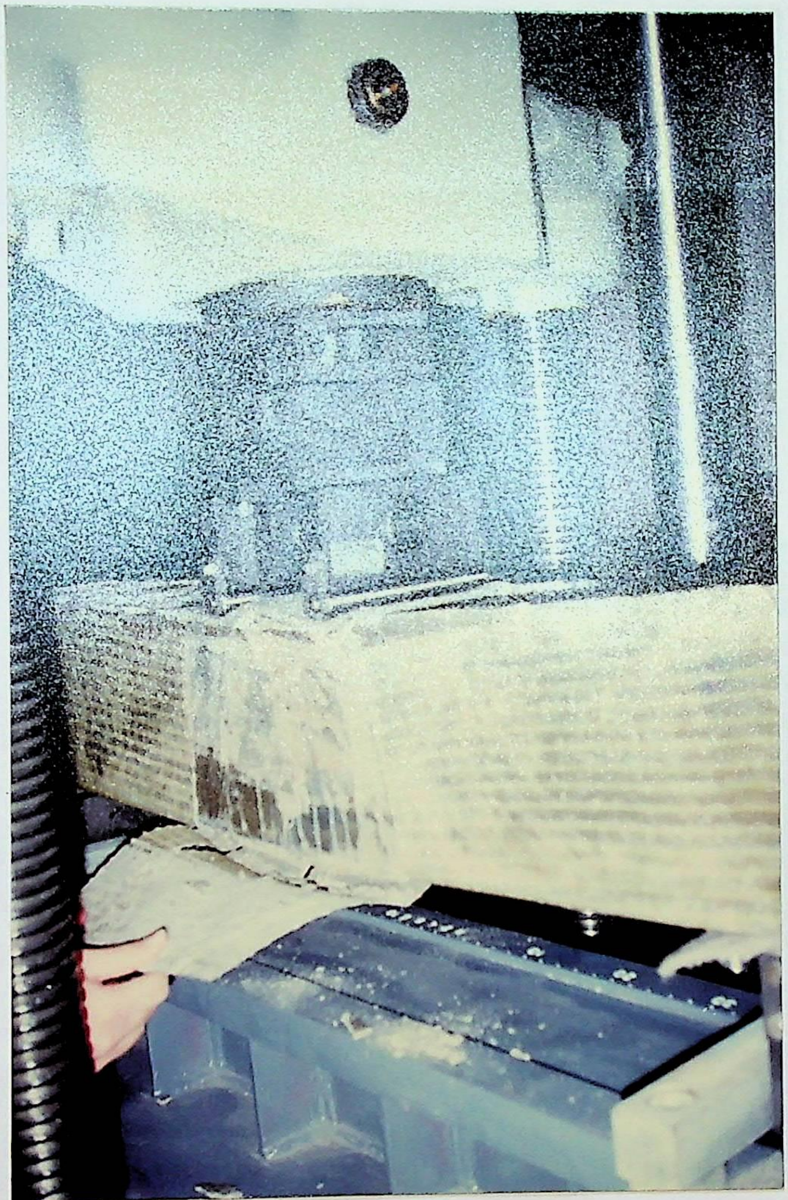
Fig(5.9) Control 1 beam at failure during 2 point flexural test



Fig(5.10) Beam B1 at failure during 2 point flexural test

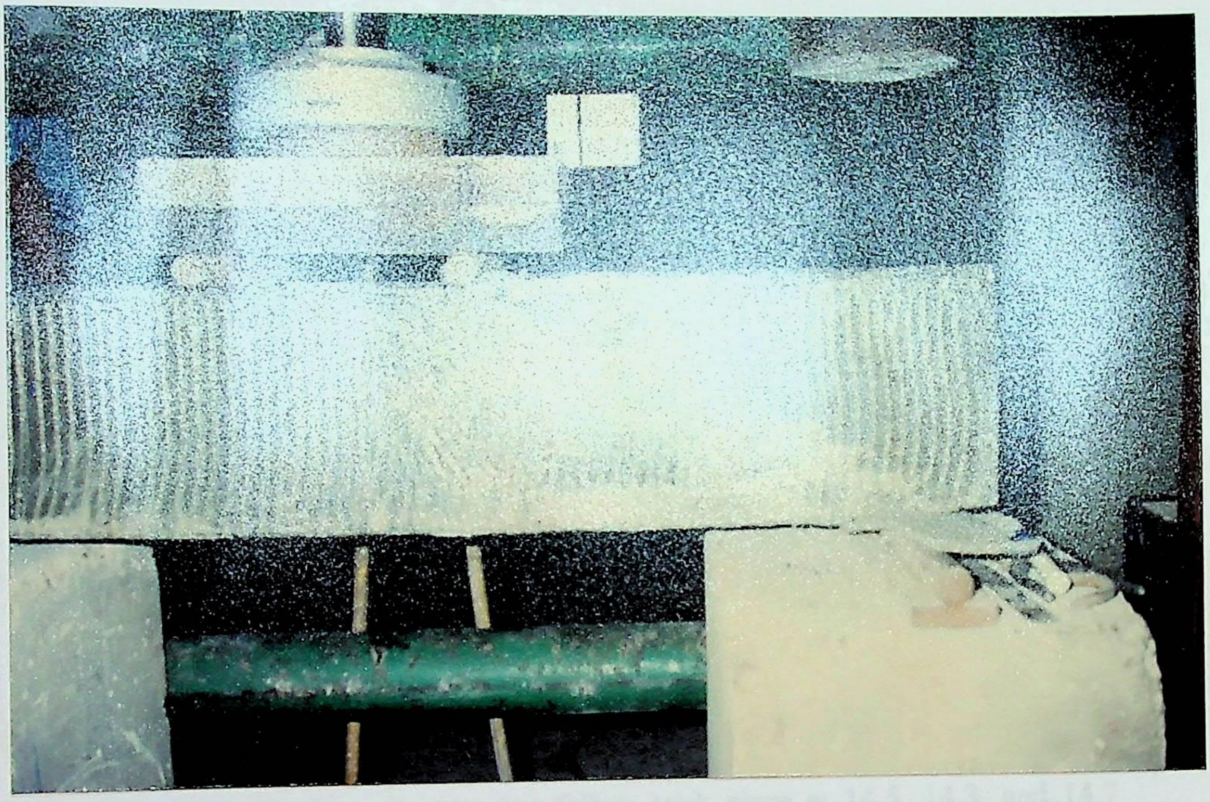


Fig(5.11) Beam B5 at failure during 2 point flexural test



Fig(5.12) Beam B6 at failure during 2 point flexural test

Fig(5.13) Beam B9 at failure during 2 point flexural test

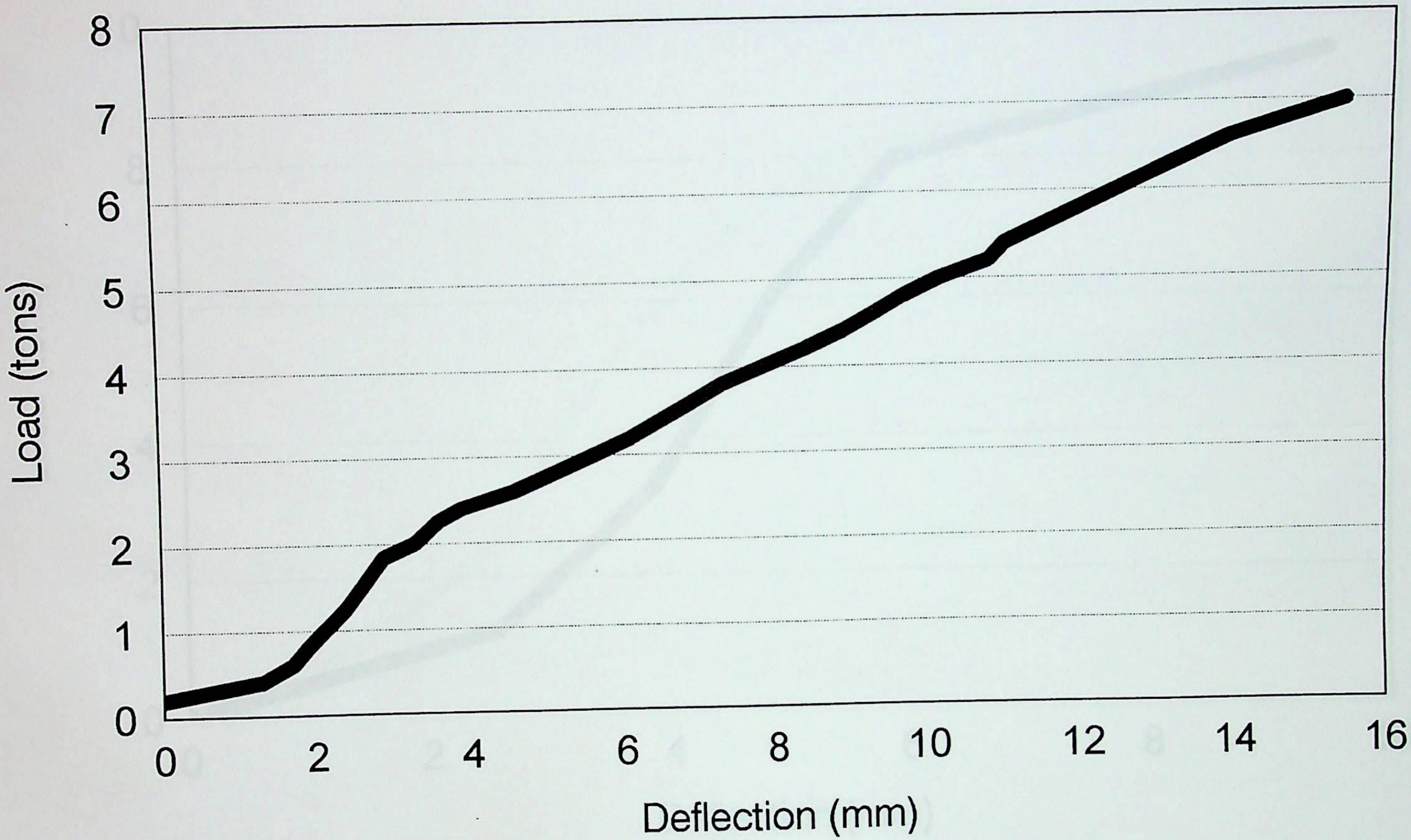


The load versus deflection curves are shown for some beams in fig(5.14) for control 1 beam, fig(5.15) for beams B2, fig(5.16) for beam B7, fig(5.17) for beam B8, fig(5.18) for beam B10.

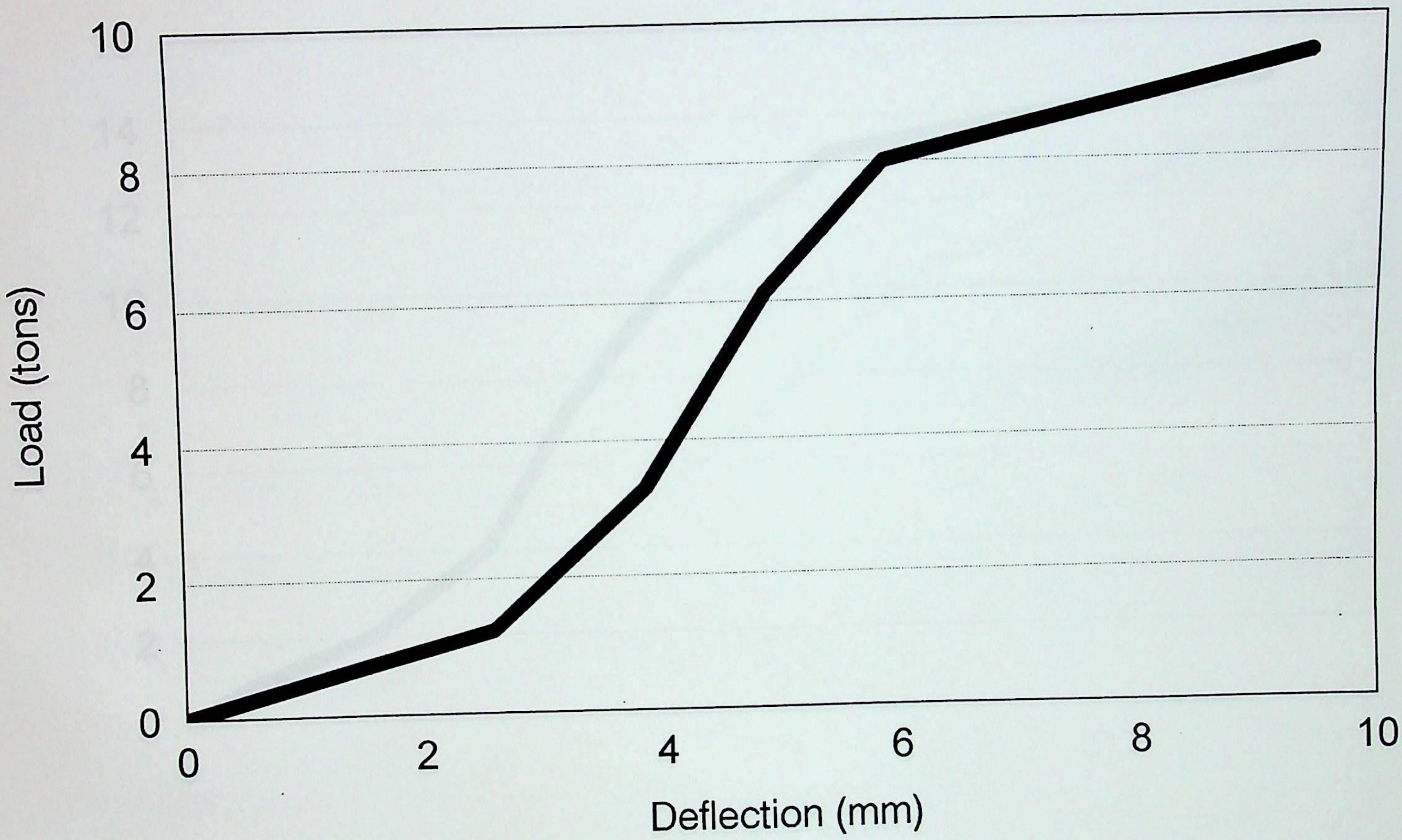
The repaired beams show a logical progression of failure modes. Adding one layer of GFRP to the tension face of the beam increased both the flexural and the shear capacity of the beam. Beam B1 failed at 8.5 tons in shear, beam B2 also failed in shear but at 9.5 tons. The increase in the flexural capacity is due to the fact that the bottom GFRP layer acts as additional reinforcement. The increase in the shear capacity of the beam when adding 1 layer of GFRP to the tension side of the beam is due to the fact that the bottom GFRP layer increased the resistance of concrete to shear by increasing the dual action and the shear interlocking. Adding a U shape layer with the main fibers orientated transverse to the span direction of the beam prevented the shear failure, however, a flexural failure of beam B3 took place at 9.5 tons because the steel reached its ultimate tensile capacity. The U shape layer increased the shear loading capacity by providing shear resistance similar to that provided by stirrups and by increasing the shear interlocking. Wrapping the beams with a U shape layer and a longitudinal layer prevented the shear failure but a flexural failure due to the failure of both the steel and the composite followed by a compression failure took place at 14.5, 14.3, and 14.7 tons for beams B4, B5, B6 respectively. In order to save material, a longitudinal layer was used with a U shape layer that was cut between the 2 loads (beam B7) because this is a zero shear zone and therefore there is no need for shear reinforcement. The results obtained indicate that the saving in the material had no effect on the shear capacity of the beam. This makes it important to perform some analysis before repairing in order to provide efficient and economic design by saving material.

Repairing the beam using a single U shape layer and 2 longitudinal layers, prevented the flexural failure because the longitudinal reinforcement was increased to 2 layers and a shear failure of beam B8 took place at 15.58 tons. Repairing the beam using a longitudinal layer and 2 separate layers bonded to both sides of the beam with the main direction of the fiber oriented along the beam length, prevented the flexural failure and beam B9 failed in shear at 11.4 tons. The 2 separate side layers improved the shear loading capacity of the beam to 11.4 tons while a single U shape layer increased the shear capacity to around 14.5 tons. Again, this emphasizes the importance of analysis before repair because if there is no need for a very high shear loading capacity, 2 separate layers can be used in order to save effort and cost. Finally, repairing the beam using 2 longitudinal layers and 2 U shape layers prevented, as expected, both the shear and flexural failures and beam B10 failed in compression at 18 tons.

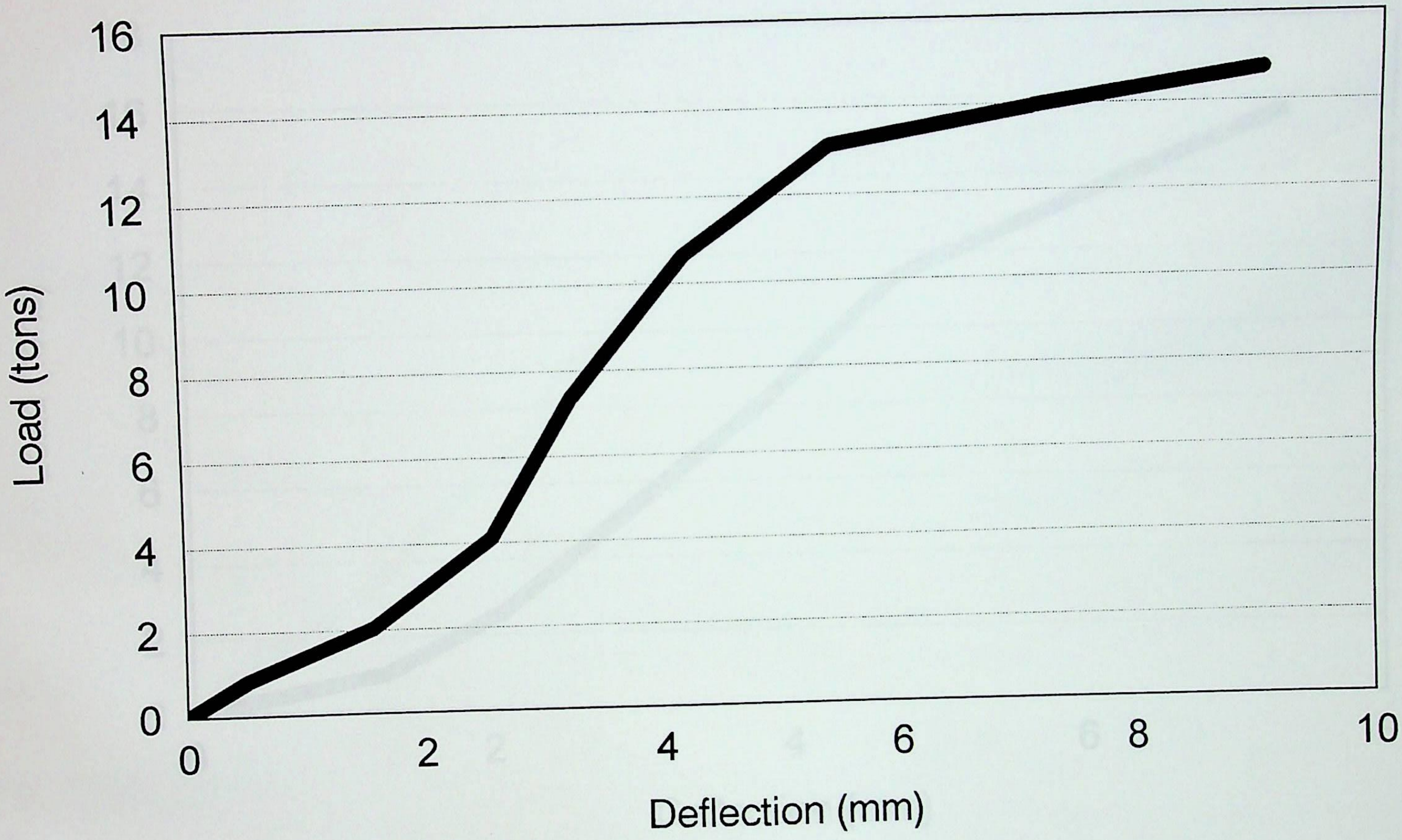
It can be concluded that external reinforcing of beams using GFRP sheets improved both the strength limit state performance in the form of increased ultimate flexural and shear loading capacities and the serviceability limit state in the form of reduced crack width.



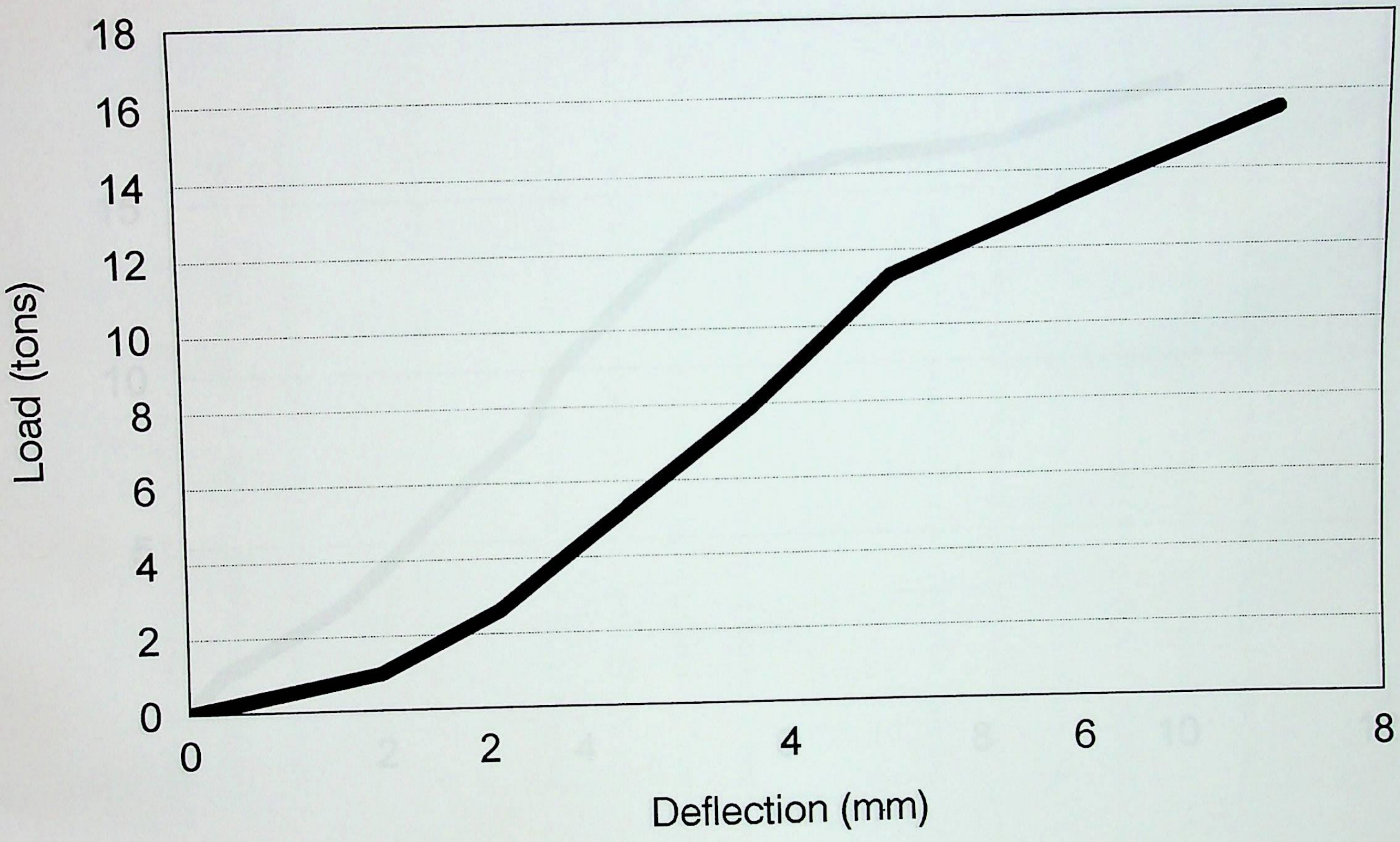
Fig(5.14) Mid-span load-deflection curve of control 1beam



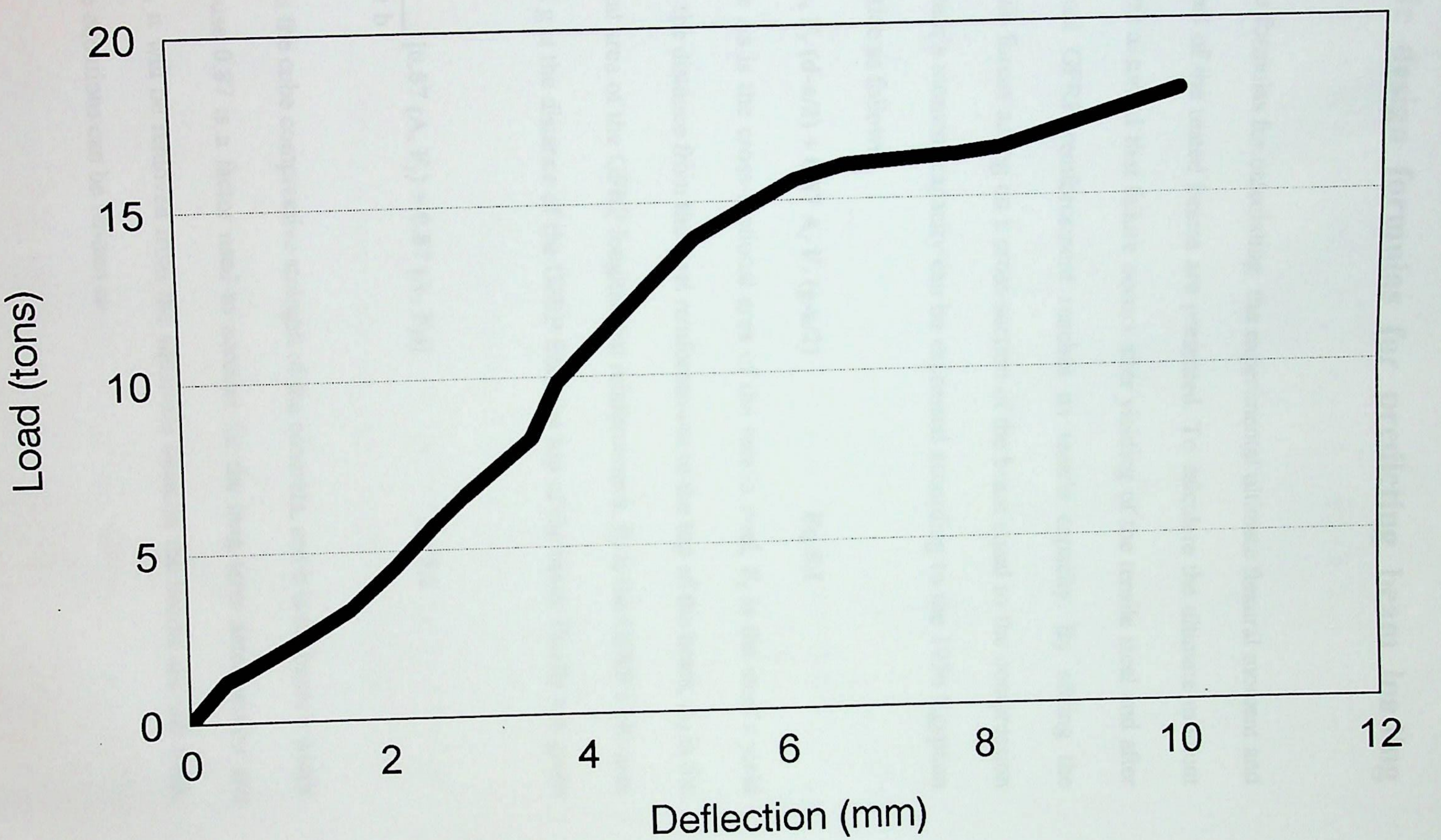
Fig(5.15) Mid-span load-deflection curve of B2 beam



Fig(5.16) Mid-span load-deflection curve of B7 beam



Fig(5.17) Mid-span load-deflection curve of B8 beam



Fig(5.18) Mid-span load deflection curve of B10 beam

5.4 Simple design formulas for predicting beam loading capacity

Simple formulas for calculating the experimental ultimate flexural moment and shear capacities of the tested beams are presented. To calculate the ultimate moment capacity, it was assumed that failure occurs after yielding of the tensile steel and after the longitudinal GFRP reinforcement reaches its tensile capacity. By setting the resultant tensile forces acting on a cross-section of the beam equal to the compression forces, the beam's moment capacity can be expressed according to the 1996 Egyptian Code of Practice as follows

$$\mathbf{Mu = 0.87\ A_s\ F_y\ (d\text{-}a/2) + 0.87\ A_f\ F_f\ (g\text{-}a/2)} \qquad \textbf{Eq 5.1}$$

where As is the cross-sectional area of the tensile steel, F_y is the steel's yield strength, d is the distance from the steel reinforcement to the top of the beam, A_f is the cross-sectional area of the GFRP longitudinal reinforcement, F_f is the GFRP's ultimate strength, and g is the distance of the GFRP from the top of the beam. Finally a is given by

$$\mathbf{a = \frac{1.5}{0.67\ Fcu\ b}\ [0.87\ (A_s\ F_y) + 0.87\ (A_f\ F_f)]} \qquad \textbf{Eq 5.2}$$

where Fcu is the cube compressive strength of the concrete, and b is the beam's width.

Because 0.87 is a factor used to account for the long term deterioration and other errors, it will be removed from the equations because the beams are still fresh, therefore the equations can be written as

$$\mathbf{a = \frac{1.5}{0.67\ Fcu\ b}\ [\ (A_s\ F_y) + (A_f\ F_f)]} \qquad \textbf{Eq 5.3}$$

$M_u = A_s\, Fy\, (d\text{-}a/2) + A_f\, F_f\, (g\text{-}a/2)$ **Eq 5.4**

The shear capacity Qu of the beam can be expressed as

$Q_u = Q_c + Q_s + Q_f$ **Eq 5.5**

where Qc, Qs, and Qf are the contributions of the concrete, steel, and FRP respectively.

According to The Egyptian 1996 Code of Practice

$Q_c = 0.75/2(\sqrt{Fcu/\delta_c}\,) \times bd$ **Eq 5.6**

where Fcu is the compressive strength of concrete

δc is 1.5

$Q_s = (A_s/S) \times (F_y/\delta_s/b) \times bd$ **Eq 5.7**

where A_s is the cross-sectional area of steel stirrups, S= spacing between stirrups, F_y is the steel yield strength, δ_s=1.15, b is the beam width.

In work by Al-Sulaimani et al (Chajes et al, 1195b) contribution to the total shear capacity from FRP plates bonded to a beans web was approximated by

$Q_f = (2\, \tau\, d\, g\, /2)$ **Eq 5.8**

where τ is the interface shear strength and is taken 28 Kg/cm2.

Equation 5.4 was used to predict ultimate experimental moments. The theoretical predictions were found conservative by around 16% see table 5.4.

Table 5.4- Theoretical versus experimental flexural beam capacity

Beam	M_{ult} experimental Kg.cm	M_{ult} theoretical Kg.cm	$\frac{M_{ult}\text{ experimental}}{M_{ult}\text{ theoretical}}$ %
B4	295437.5	247234.6	16.3%
B5	291362.5	247234.6	15.1%
B6	299512.5	247234.6	17.4%
B7	295437.5	247234.6	16.3%

The difference between the experimental and calculated ultimate moments may be attributed to uncertainty in behavior of concrete or due to the experimental error in determining F_f, the estimated value of lamination strength or code factors.

Equation 5.8 was used to determine the theoretical shear capacity of GFRP. $Q_f = 2*28*18*20/2 = 10.08$ tons. From the experimental work, the shear capacity of both the concrete and steel was 7.1 tons (control 1). Thus, experimentally the load capacity of GFRP is 8.84 tons. This shows that the equation is conservative by about 15%.

5.5- Finite element analysis of beams using ABAQUS

As explained in chapter IV of this work, a model was constructed using the finite element program ABAQUS to simulate the case of the control beam, and that of beams strengthened using 1 bottom layer and 1 U shape layer which is the case of beams B4, B5, B6, and B7 respectively. Table 5.5 shows the finite element analysis results.

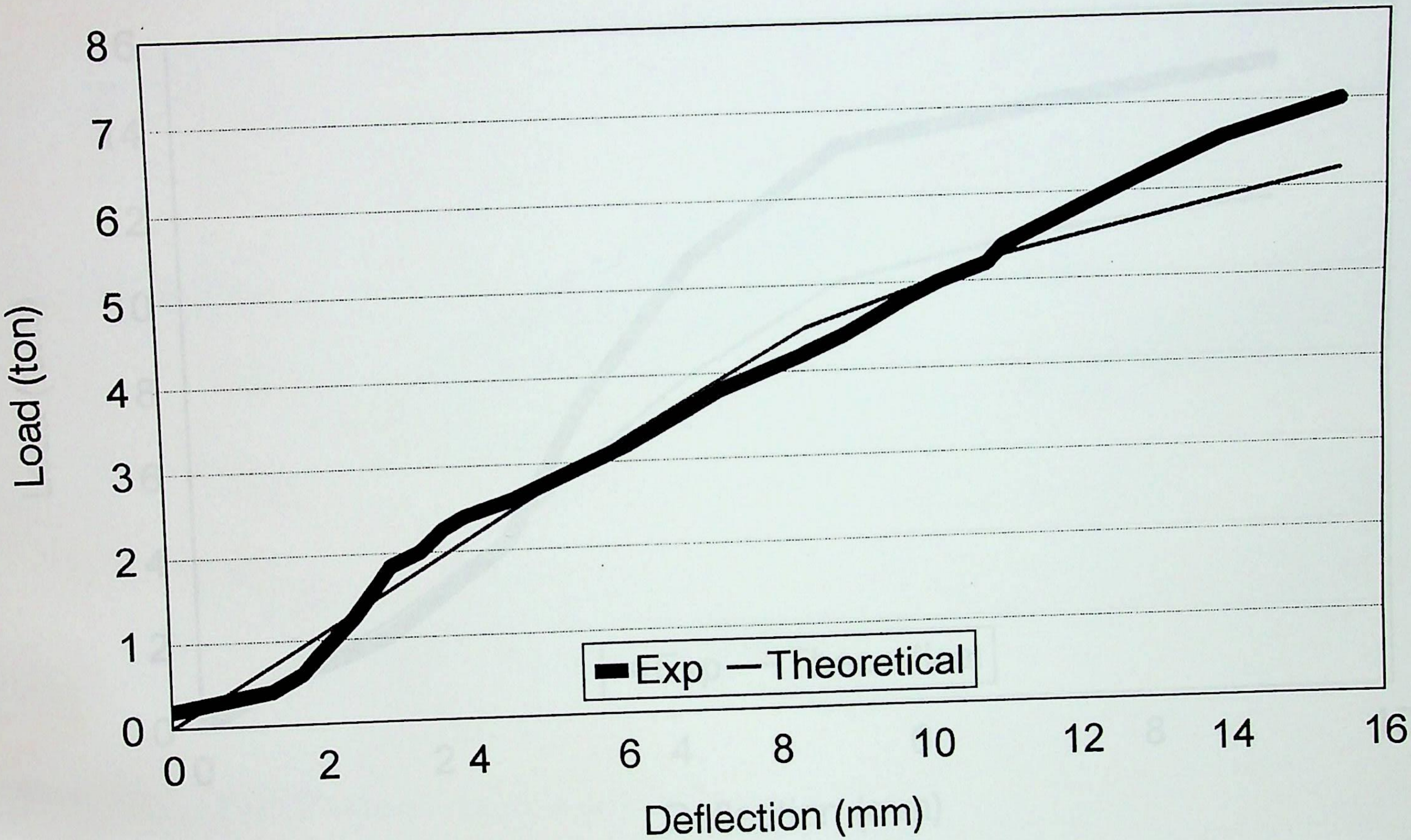
Fig(5.19), and fig(5.20) show the experimental vs. theoretical load-deflection curves for the control beam and strengthened beams respectively.

Table 5.5- Finite element analysis results

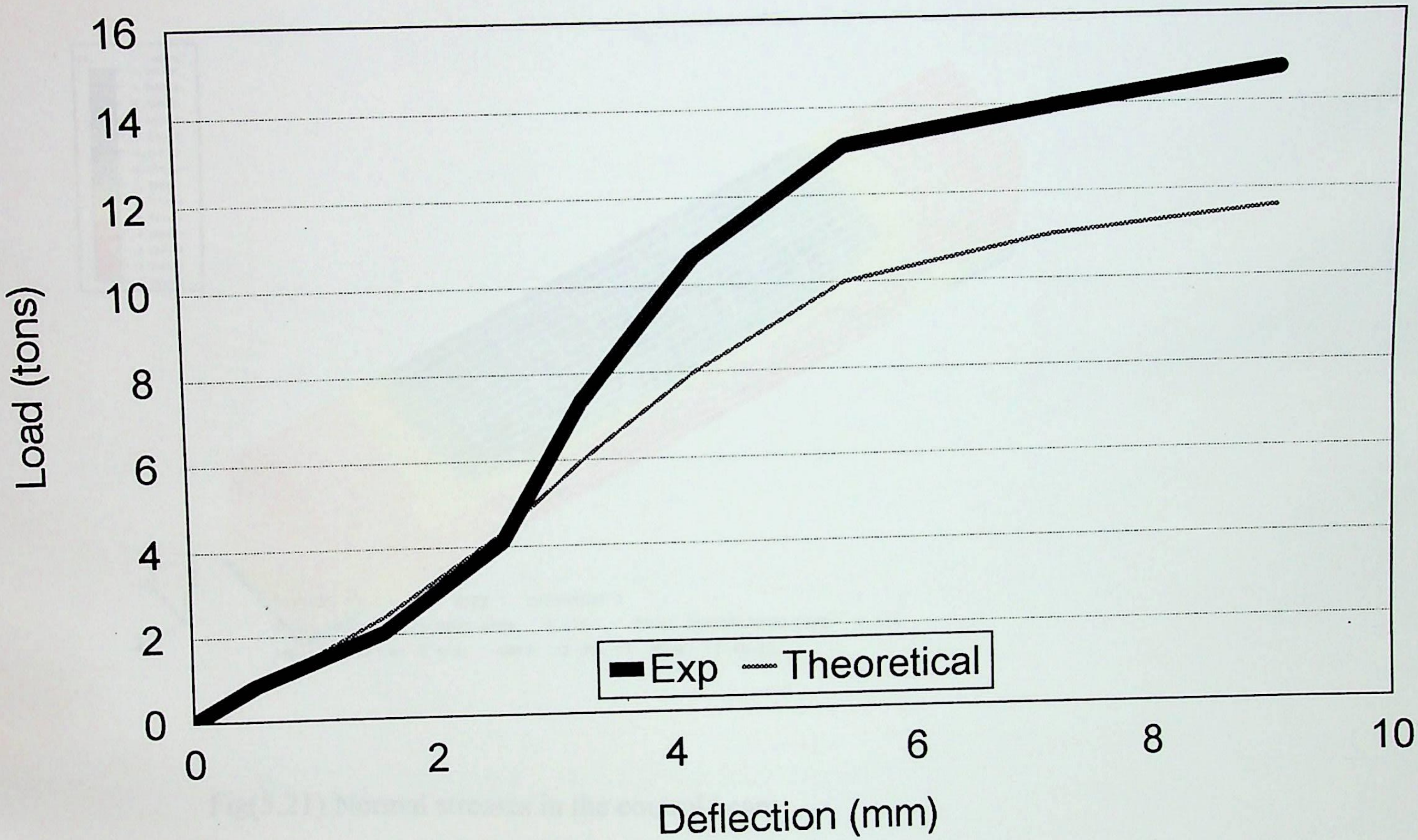
Beam	Experimental failure (ton)	Theoretical failure (ton)	Percent difference
Control 1	7.1	6.2	13%
B4,B5,B6,B7	14.5	11.6	20%

In both the experimental and the theoretical investigations, the control beam failed in shear while the strengthened beam failed in flexure. The results obtained from the finite element analysis performed using the ABAQUS computer package were close to those obtained experimentally. ABAQUS results were conservative by 20% in the case of beams strengthened using 1 bottom layer with the main direction of fibers oriented along the span of the beam and 1 U shape layer with the main direction of the fibers transverse to the span of the beam. The ABAQUS results were conservative by 13% in the case of the control beams. In both cases, cutting the GFRP sheet in the zero shear zone was found to have no influence on the load carrying capacity of the beam. The ABAQUS package made it possible to visualize the behavior of the experimentally tested beams. Fig(5.21) represent normal stresses in the control beam, and fig(5.22) the maximum load and deflection. As for the strengthened beams, fig(5.23) to fig(5.26) show the stresses in the concrete, U shape GFRP sheet, bottom GFRP layer, and the maximum load and deflection respectively. By comparing the stresses obtained from ABAQUS in the concrete, U shape GFRP sheet, and the bottom GFRP layer, it is clear that the highest stresses at failure were exerted in the bottom GFRP layer shown in fig(5.25) and this explains the experimental flexural failure of the strengthened beams.

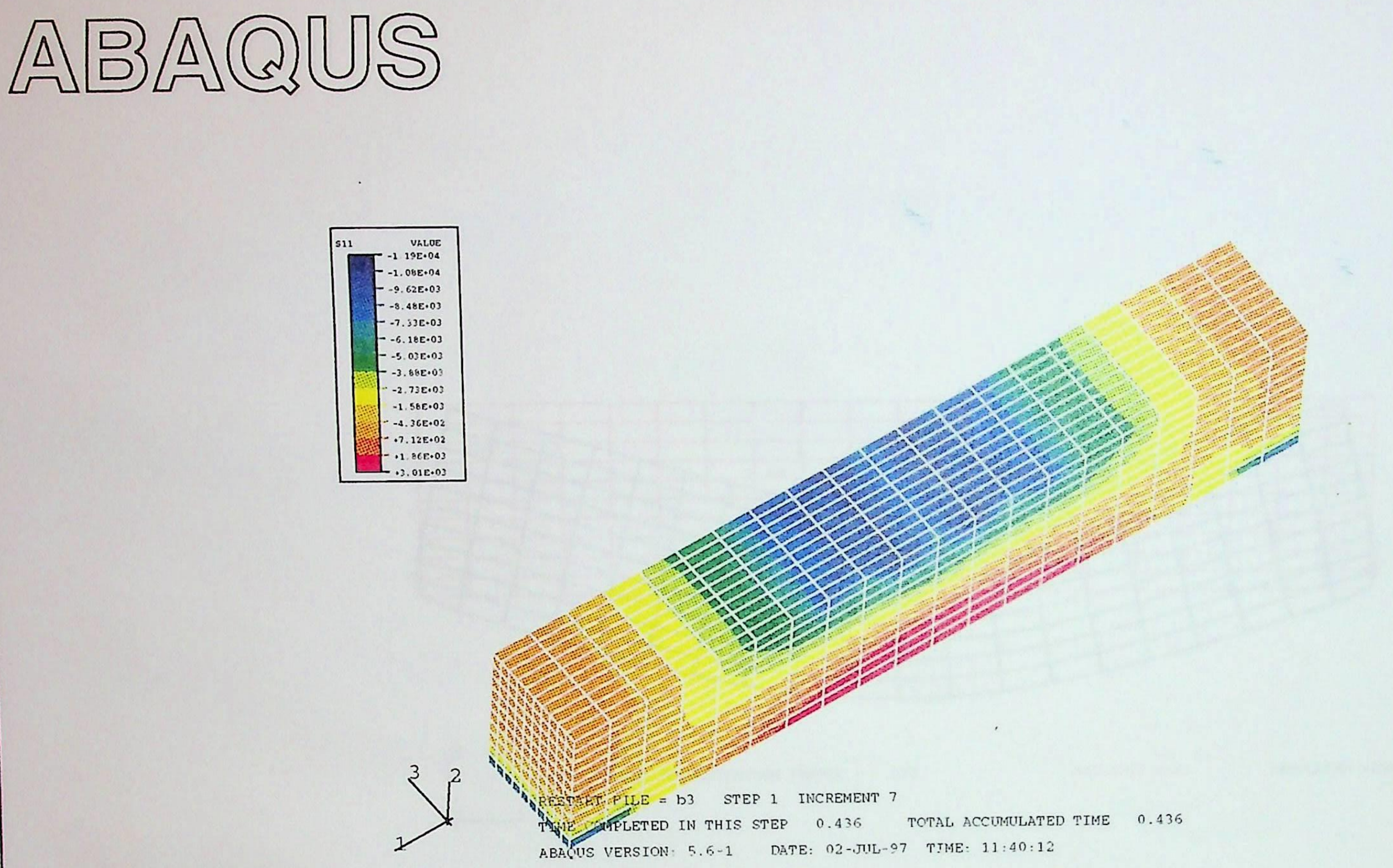
Finite element programs can be used in designing future experimental research, and in predicting the behavior of externally reinforced concrete members using GFRP sheets. However, the user must be careful while using the built in curves and be well acquainted with the method of solution used by the program before inputting data because the process of modeling is quite complicated.



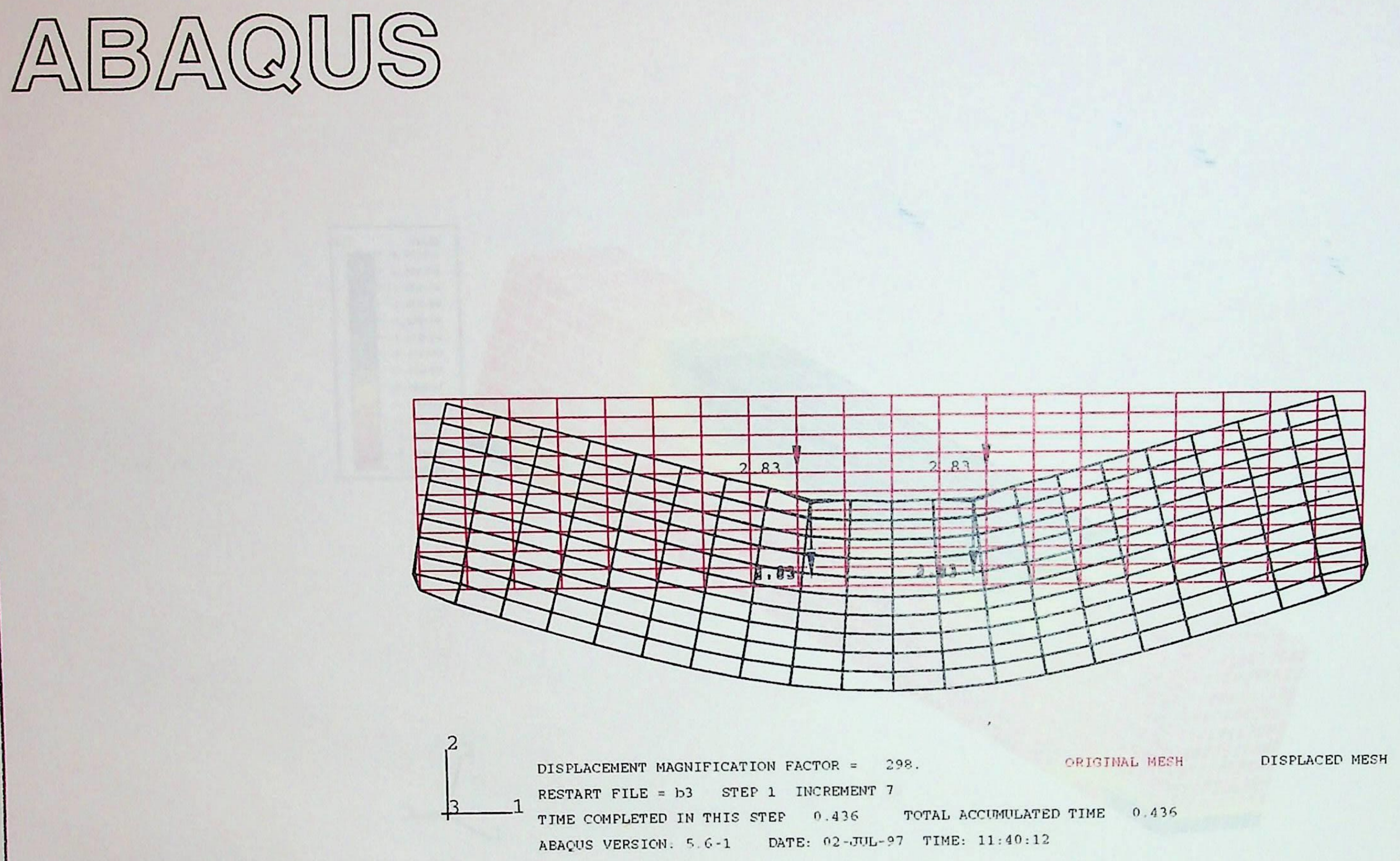
Fig(5.19) Load-deflection curve for beam control 1 experimentally and using ABAQUS



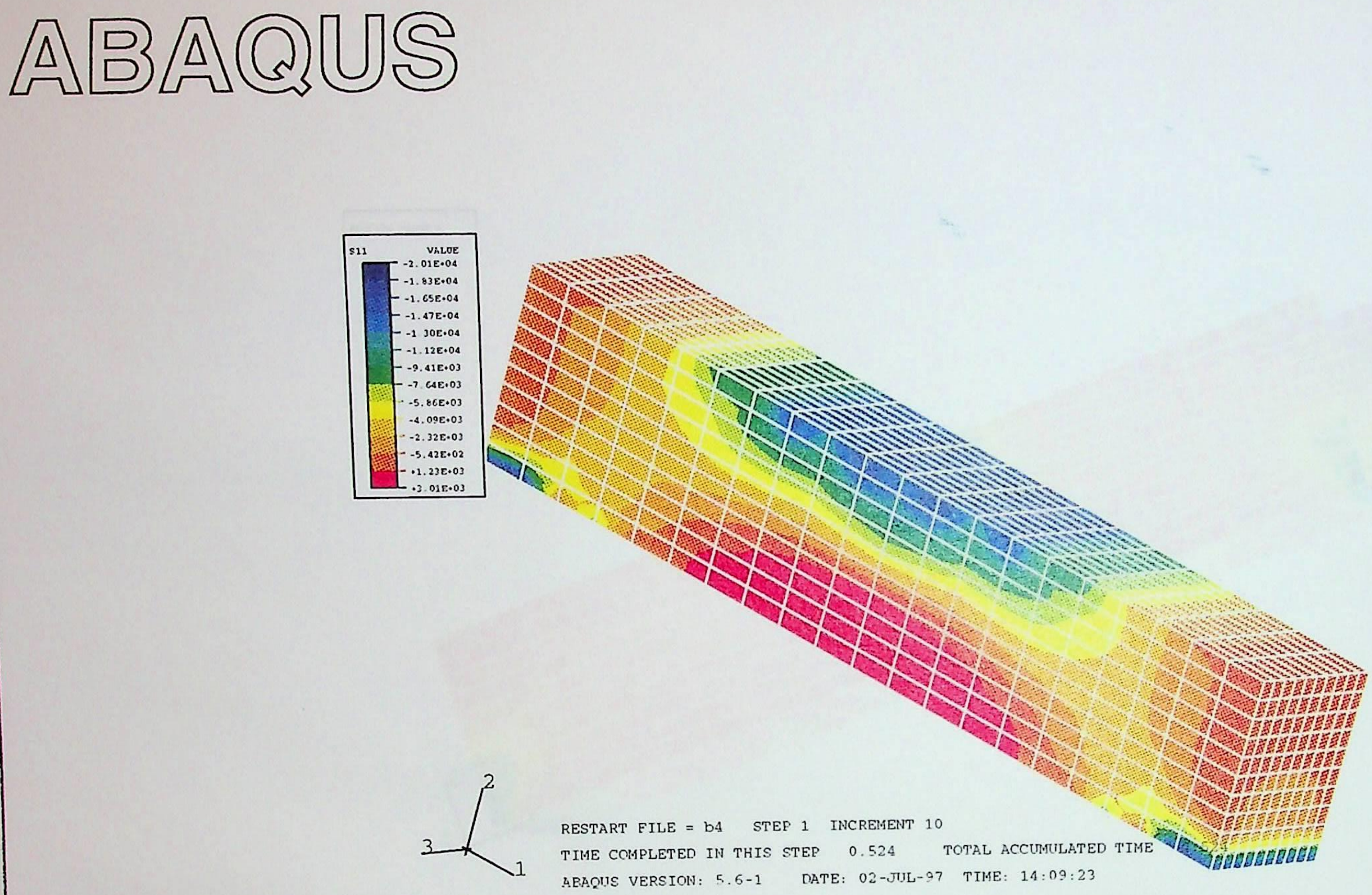
Fig(5.20) Load-deflection curve for beam B7 experimentally and usin ABAQUS



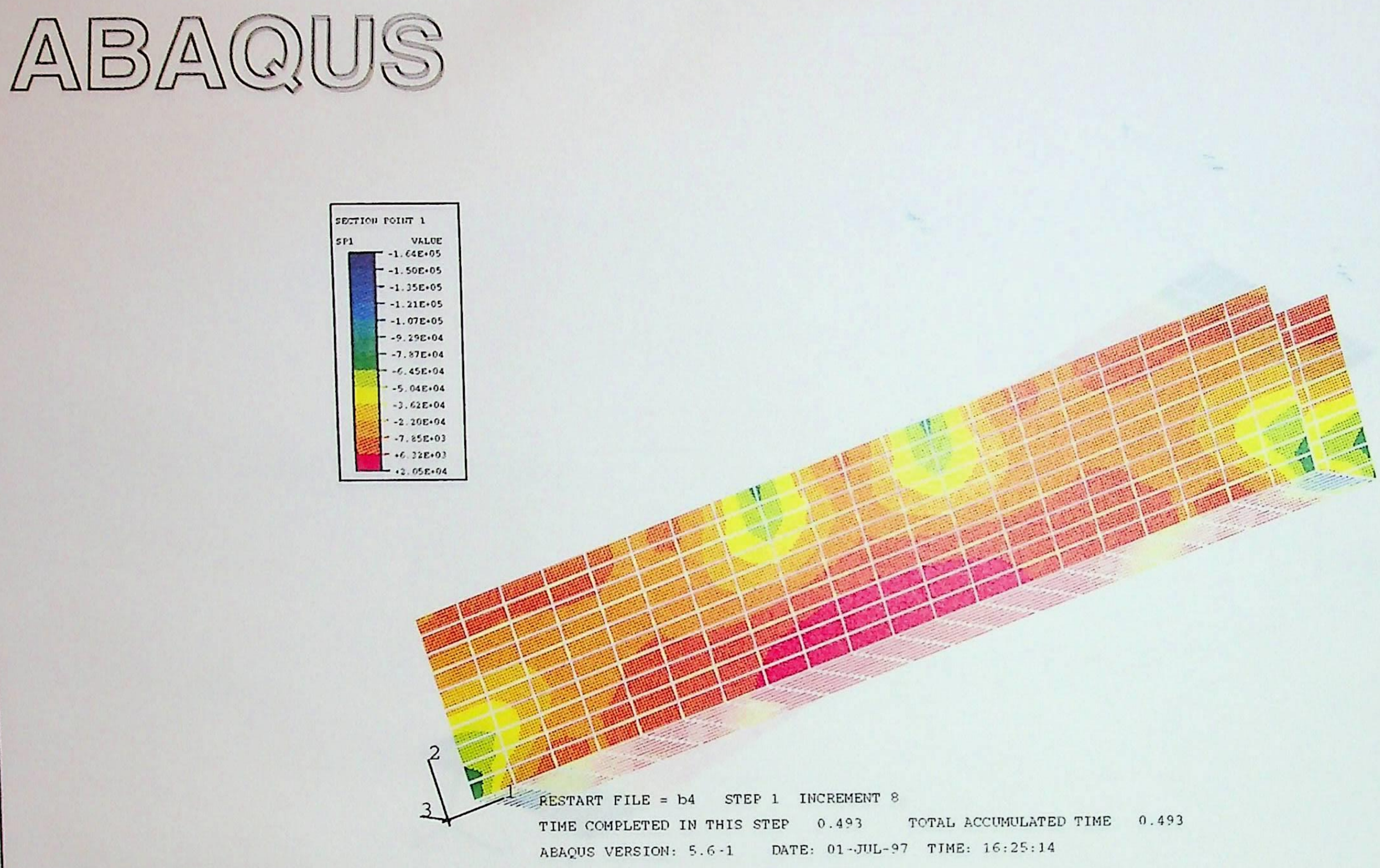
Fig(5.21) Normal stresses in the control beam



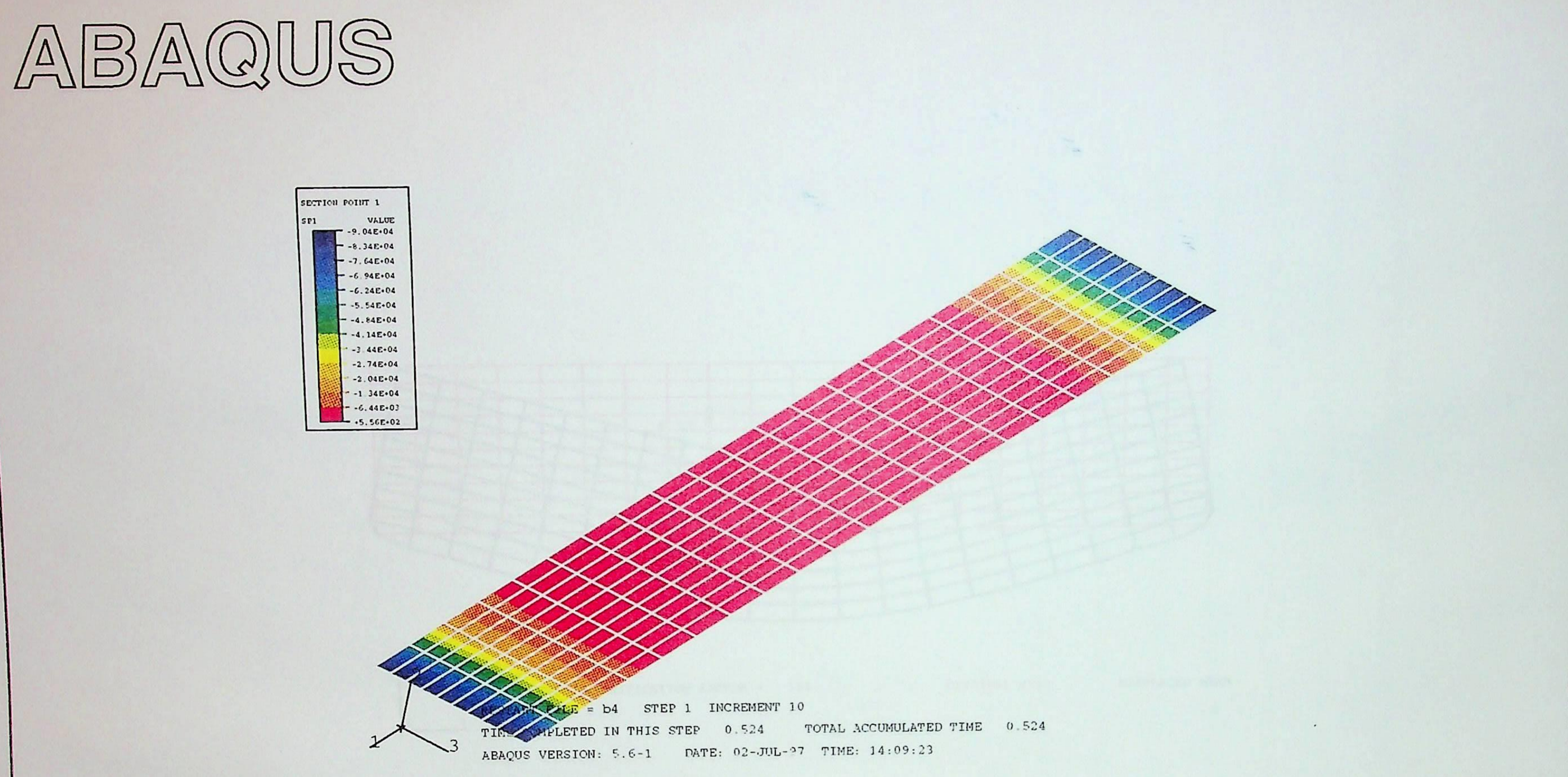
Fig(5.22) The deflected shape of the control beam



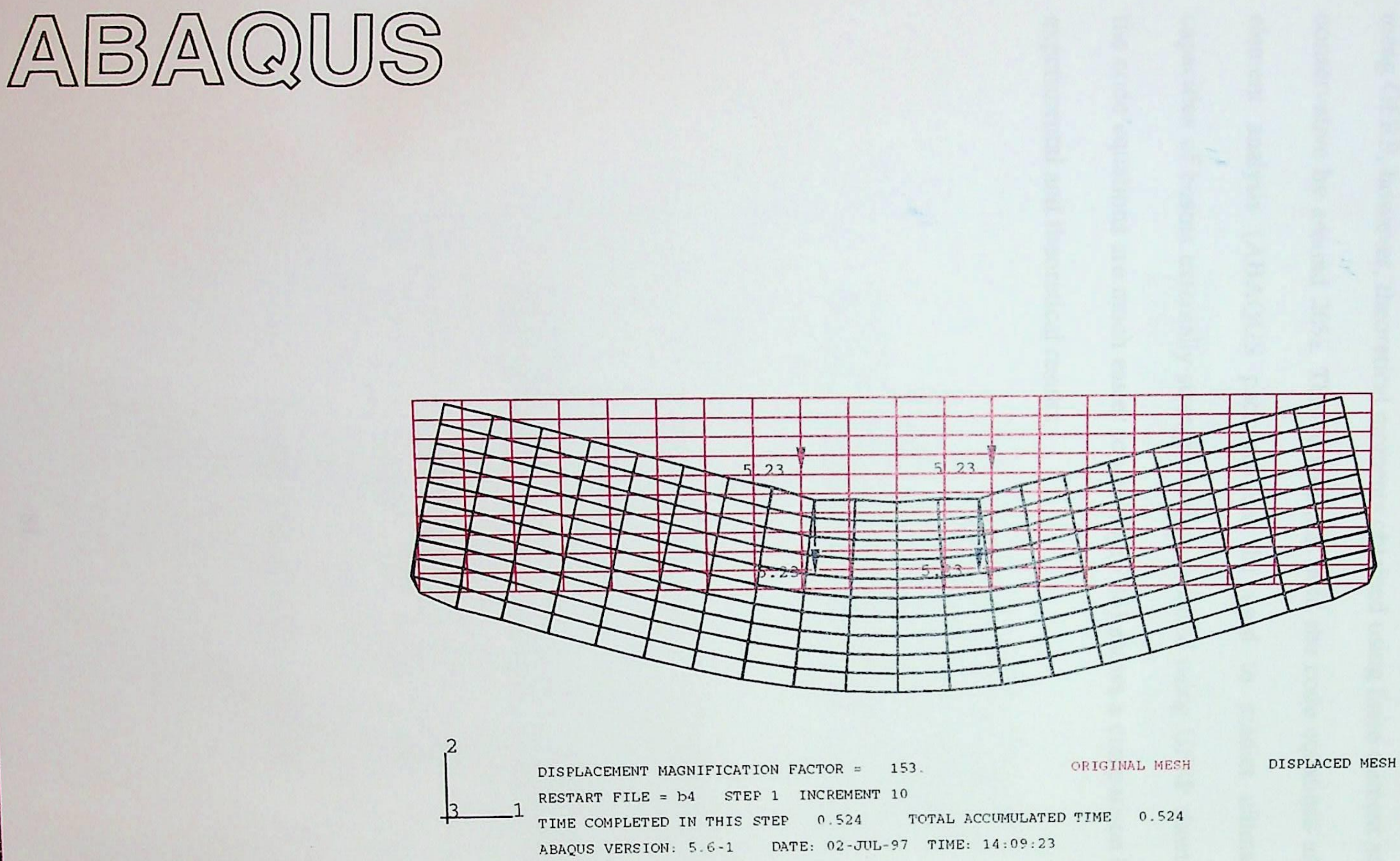
Fig(5.23) The maximum stresses in the concrete of strengthened beam using 1 bottom GFRP layer and 1 U shape GFRP layer



Fig(5.24) The maximum stresses in the U shape GFRP sheet



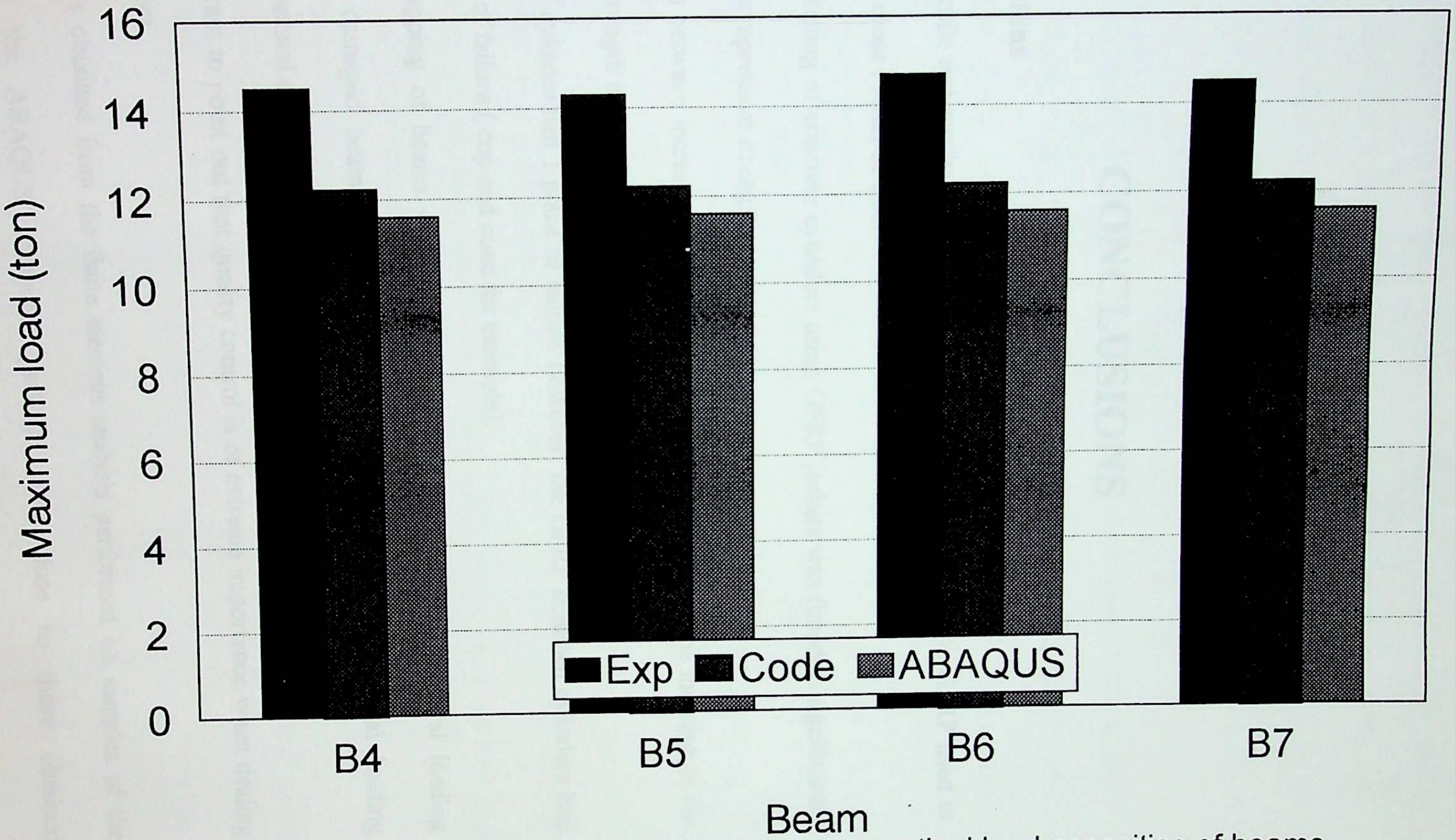
Fig(5.25) The maximum stresses in the bottom GFRP sheet



Fig(5.26) The deflected shape of concrete beam strengthened using 1 bottom GFRP layer and 1 U shape GFRP layer.

5.6- Comparison between experimental and theoretical results

The theoretical ultimate moments obtained using the code equations were conservative than the experimental moments by around 16% for beams strengthened using GFRP, however, theoretical moments obtained using finite element analysis were conservative by around 20%. This proves that both the code equations and the finite element analysis (ABAQUS package) can be used to predict ultimate moment capacities of beams externally strengthened or repaired using GFRP sheets, however, the code equations are much easier to use. Fig(5.26) shows a comparison between the experimental and theoretical results.



Fig(5.27) Comparison between experimental and theoretical load capacities of beams

CHAPTER VI

CONCLUSIONS

6.1- Conclusions

1- From the tensile test results, it is clear that the tensile properties of the GFRP used in this research is close to that of steel.

2- The strengthening of concrete cylinders using GFRP jackets was found to significantly increase their compressive strength.

3- The relation between increasing the number of confining layers and the increase in the compressive strength is not directly proportional.

4- The control cylinder had 1 plane of failure, however, the GFRP confined cylinders had several planes of failure (cup and cone for example).

5- GFRP wrapping of beams was found to improve the shear and flexural loading capacities of damaged beams. The repaired beams exceeded their original loading capacities in flexural and shear by values up to 150%.

6- It is important to point out that quality control is of extreme importance when dealing with composites.

7- The results obtained from the finite element analysis performed on samples of the beams using the ABAQUS computer package were close to those obtained experimentally. ABAQUS results were conservative by 20% in the case of beams strengthened using 1 bottom layer with the main direction of fibers oriented along the span of the beam and 1 U shape layer with the main direction of the fibers transverse to the span of the beam. The ABAQUS results were conservative by 13% in the case of the control beams.

8- Finite element programs can be used in designing future experimental research, and in predicting the behavior of externally reinforced concrete members using GFRP sheets. However, the user must be careful while using the built in curves and be well acquainted with the method of solution used by the program before inputting data because the process of modeling is quite complicated.

9- The results obtained using equations derived from the 1995 Egyptian code and those obtained using ABAQUS were close which means that code the derived equations can be used in designing GFRP externally reinforced beams.

10-Based on the experimental and the theoretical work, it can be concluded that fiber reinforced plastics have great potentials that can be used to revolutionize reinforced concrete repairing and upgrading techniques because GFRP sheets improved both the strength limit state performance in the form of increased ultimate flexural and shear loading capacities and the serviceability limit state in the form of reduced crack width, however, research efforts are still needed to come out with reliable design and quality control procedures.

6.2 Recommendations for future research

Most of the evidence indicate that FRP composites can be used in upgrading or strengthening the mechanical properties of reinforced concrete members. It is recommended that future research should focus on:

1- The impact of environmental conditions on concrete members upgraded or repaired using FRP composite material. Environmental conditions include but are not limited to effect of temperature, salt and alkaline attacks.

2- The behavior of FRP upgraded or repaired concrete members when subjected to seismic loads.

3- The effect of creep on reinforced concrete members upgraded and repaired using FRP.

4- Long term effects on resins and fibers.

5- The use of finite element packages and techniques to model externally reinforced concrete members using advanced composite materials.

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