EXPERIMENTAL INVESTIGATION AND BEHAVIOUR OF REINFORCED CONCRETE BEAMS STRENGTHENED WITH CFRP

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ABSTRACT

Reinforced Concrete (RC) is the most common and most popular building material in the world. Most structures such as buildings, bridges etc. uses reinforced concrete as their main construction material. Some of these structures or parts of it are not fulfilling their structural functions due to some reasons. Replacing these deficient structures requires huge investments and is not an enticing option, hence strengthening has become the suitable way for improving the load carrying capacity and prolonging their service life. Even though the effectiveness of other techniques are widely accepted, a new effective and promising technique of strengthening civil engineering structures externally is gaining popularity, where fibre-reinforced polymer (FRP) is used.

The aim of this thesis is to investigate and improve the understanding and the properties of externally RC beams strengthened with Carbon Fibre Reinforced Polymer (CFRP) sheets. To achieve the aim, two groups of beams were designed according to ACI318-11 and investigated experimentally. The first group consisting of four beams strong in flexure but weak in shear and the second group beams consist of three beams strong in shear but weak in flexure. The beams were strengthened using CFRP externally. The behaviours of the beam specimens under the effect of loads were investigated according to various application of CFRP orientation. According to results of experiments, the effectiveness of the rehabilitation method both on the load carrying capacity and energy absorption capacity were evaluated.

Key words: *FRP, CFRP, Beams, Strengthening, ACI318-11.*

ÖZET

Betonarme, dünyanın en yaygın ve en popüler yapı malzemesidir. Binalar, köprüler vs. gibi birçok yapının ana malzemesinde betonarme kullanılmaktadır. Bu yapıların veya yapı elemanlarının bazı nedenlerden dolayı kendi yapısal işlevlerini yerine getirememektedir. Bu eksikliği gidermek için yapıları değiştirmek uygun seçenek olmamakla beraber, yüksek maliyetlidir. Dolayısıyla, güçlendirme yöntemi, yapının yük taşıma kapaistesini artırmak ve ömrünü uzatmak için uygun bir yol haline gelmiştir. Diğer kullanılan tekniklerin etkinliği kabul edilir olsa dahi, fiber takviyeli polimerlerle (FRP) dıştan güçlendirilen mühendislik yapılarının etkili ve gelecek vaat eden bir yöntem olduğu bilinmektedir.

Bu tezin amacı, karbon fiber takviyeli polimer (CFRP) ile güçlendirilmiş kirişlerin davranışını araştırmak ve geliştirmektir. Amaca ulaşmak için, iki grup kiriş, ACI318-11'e göre tasarlanmış ve deneysel olarak incelenmiştir. Birinci gruptaki dört kiriş numunesi eğilmeye karşı güçlü, kesmeye karşı zayıftır. İkinci gruptaki üç kiriş numunesi ise kesmeye karşı güçlü ve eğilmeye karşı zayıftır. Kirişler dıştan CFRP kullanılarak güçlendirilmiştir.

Kiriş numunelerin yük altında eğilme ve kesmeye karşı davranışı, çeşitli CFRP uygulamaları ile yönlendirilmiş ve incelenmiştir. Yapılan deneysel araştırmalara göre, kirişlerin yük taşıma kapasiteleri, sargılı numunelerin eksenel yük altında enerji yutma kapasiteleri ve rehabilite yönteminin etkinliği değerlendirilmiştir.

Anahtar Kelimeler: *FRP, CFRP, Kirişler, Güçlendirme, ACI318-11.*

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LIST OF ABBREVIATIONS

FRP	Fibre Reinforced Polymer
TEC2007	Turkish Earthquake Code 2007
TRNC	Turkish Republic of Northern Cyprus
ACI	American Institute of Concrete
CFRP	Carbon Fibre Reinforced Polymer
AFRP	Aramid Fibre Reinforced Polymer
GFRP	Glass Fibre Reinforced Polymer
RC	Reinforced Concrete
ISIS	Intelligent Sensing for Innovative Structures
EMPA	Swiss Federal Laboratories for Material Testing and Research
FIB	Fédération Internationale du Béton
TR 55	Technical Report
DPÖ	State Planning Organization
CBM1-G1	Control Beam Group 1
SBM1-G1	Strengthened Beam1 Group 1
SBM2-G1	Strengthened Beam2 Group 1
SBM3-G1	Strengthened Beam3 Group 1
CBM1-G2	Control Beam Group 2
SBM1-G2	Strengthened Beam1 Group 2

SBM2-G2 Strengthened Beam2 Group 2

LIST OF SYMBOLS

f_y	Grade of steel
A_s	Area of tensile reinforcement
d	Effective depth of beam
S_f	Spacing of FRP strips
t_f	Thickness of CFRP layers
w_f	Width of CPRP strip
b_w	Web width of the member
f'_c	Characteristic strength for concrete
Ψ	Additional reduction factor from FRP contribution
E_m	Elastic modulus of matrix
E_f	Elastic modulus of fibres
V_{mi}	Volume fraction of matrix
V_{fi}	Volume fraction of fibre
ρ_{eff}	Effective steel reinforcement ratio

Chapter 1

INTRODUCTION

1.1 Background

Reinforced Concrete (RC) is one of the common and most popular building material in the world. Most structures such as buildings, bridges etc. uses reinforced concrete as their main construction material. Some of these structures or parts of it are not fulfilling their structural functions due to defects on the concrete caused by poor construction practices, corrosion damage as shown in Figure 1.1, fire damage, accidental damage as shown in Figure 1.2 or deterioration caused by environmental action as shown in Figure 1.3. While some reinforced concrete structures need to be upgraded due to design and construction faults and in cases of load increment or damage induced to the structural members by a seismic or other action. In addition, increase in volume of traffic may result to bridge upgrade. Strengthening the structural members in a bridge, gained popularity. Replacing these deficient structures requires huge investments and is not an enticing option, hence strengthening has become the suitable way for improving the load carrying capacity and prolonging their service life.

Even though the effectiveness of other techniques are widely accepted, engineers develop a new, better and most promising technique using advanced material in strengthening civil engineering structures with external bonding advanced fibre-reinforced polymer (FRP) which is more advantageous and gained popularity worldwide. In recent years, civil engineers and researchers are showing great interest in using these new and advanced methods of repairing, retrofitting, and strengthening of RC structures. Thus, theoretical and experimental researches have been conducted extensively on the behaviour of FRP strengthened reinforced concrete structural elements, including beams, columns, and slabs.

To make the use of FRP easier, some manufacturing techniques have been introduced and the property characteristics of FRP composites have shown that they have many advantages over the more conventional civil engineering material.

Figure 1.1: Typical Corrosion Damage Caused by Chloride-Contamination Leaks in Bridge Deck Joints [http://www.structuremag.org/article.aspx?articleID=509]

Figure 1.2: Accidental Damage on a Bridge (Shahawy, 2010)

Figure 1.3: Concrete Deck Deterioration Caused by Environmental Factors (Shahawy, 2010)

The most common type of structural member is beam and has the function of withstanding loads. However, in some cases due to the reasons explained before i.e poor construction, poor design etc., members in structure of a buildings or bridges are not adequate of resisting the applied loads. Hence using an effective method of strengthening beams is significant in terms of maintaining the safety of the structures. With respect to these, in the recent years the emerging technology of using FRP for strengthening simply supported RC beams is fascinating much attention. FRP is listed as one of the successful technique which is currently interesting to the structural engineers as a modern and promising material in the construction industry (Jumaat et al., 2010). Due to the increase use of FRP as an externally bonded material with concrete, design codes were developed by some professional organizations such as American Concrete Institute Committee Report ACI 440.2R-2008, Fédération Internationale du Béton FIB 2003, Technical Report TR 55 2004, and Intelligent Sensing for Innovative Structures ISIS Canada (Andrei, 2011).

The strengthening of RC members using FRP is influenced by the type of fiber, fiber direction, fiber distribution, and bond scheme. A large number of research showed that the success of FRP is maximized by bonding FRP on the surface and parallel to the direction of the principal tensile stress.

There are three most commonly used FRP composites used for rehabilitation of structural members in civil engineering works. But the most commonly used one is classified as carbon fibre reinforced polymer (CFRP) which is preferred due to its greater advantages to be used as reinforcing bars, and externally bonded reinforcement for strengthening, retrofitting and repairing of deficient ageing bridges and buildings as shown in Figure 1.4 and Figure 1.5 (Jumaat et al., 2010).

Figure 1.4: Shear Strengthening Inclined Orientation: Efficient Use of CFRP (Shahawy, 2010)

(a) Pre-repair (before repair) (b) After Repair (1996) (c) Seven Years after Repair (2003)

Figure 1.5: Example of a Column Wrapped with CFRP (Shahawy, 2010)

1.2 Statement of Problems of Reinforced Concrete Structures in TRNC

Concrete been the prevalent building material in Turkish Republic of Northern Cyprus (TRNC), was used for structural buildings that were designed and constructed in the 70s and 80s when there was intense reconstruction after the war. The concrete were far below the standards according to the current design codes. As a result, designs might be deficient in strength according to the current codes.

After conducting a study in 2009, the TRNC "State Planning Organization" (DPÖ) published a report on "Urban and Rural Building Construction Statistics" assessing the development of the building sector over a period of 15 years as shown in Figure 1.6. According to the report, the number of new constructions after 2004 has exceeded 2000 per year showing that the housing industry peaked, after the Annan plan. These buildings have deteriorated badly during their lifetime because of the environmental action, due to poor quality control during the construction or due to the poor structural detailing and lack of planning. Hence, many of the structures cannot be sold or used appropriately. In addition, increase in population in the country is affecting the rate of land prices making it unaffordable for the masses.

Generally, most reinforced concrete structures life span is shorter than the expected design life span. These brings about a way of finding and using alternative solutions to these problems, hence the need for the aging structures to be strengthen, repair, or retrofit action to meet the new usage. Although the issue of strengthening or retrofitting of buildings is quite complex, the method of applying CFRP externally for strengthening existing concrete structures is one of the easiest solution in the field of civil engineering.

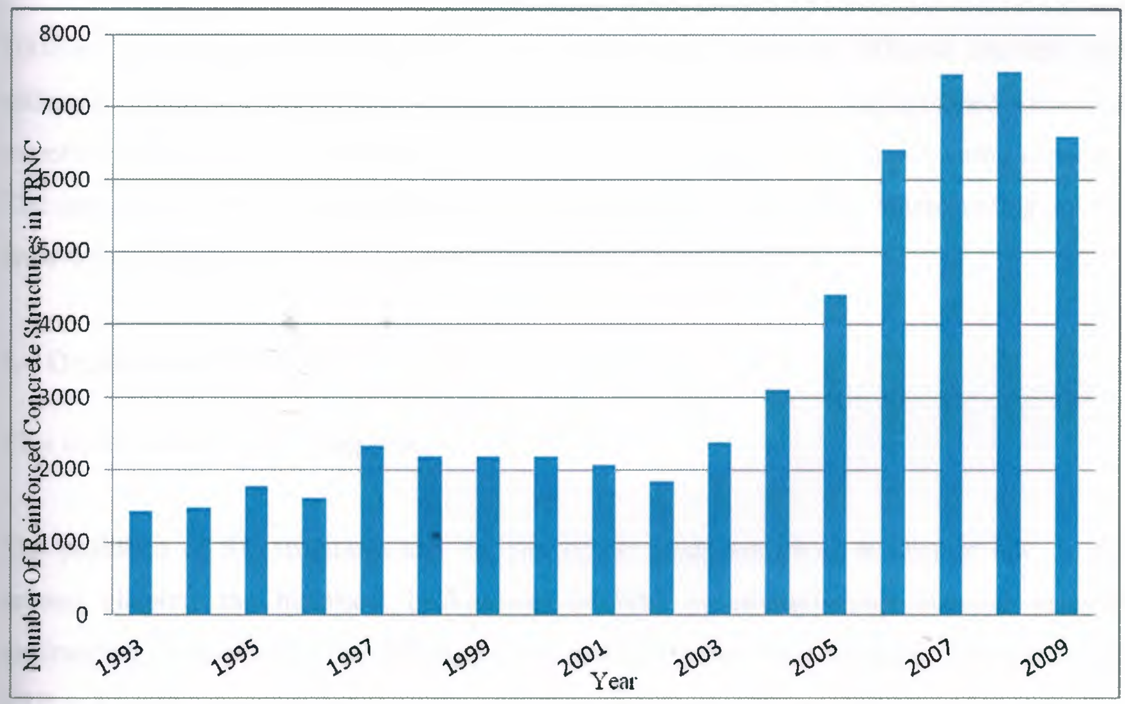

Figure 1.6: Number of RC Structures in TRNC with Year (DPÖ, 2009)

1.3 Aim of the Research

There is need for increase ultimate capacity of reinforced concrete (RC) structures without increasing their self-weight in North Cyprus. Past research concluded that applying CFRP to the tension face (bottom side) of a RC beam increases the rigidity, load bearing capacity and decreases the deflection. In this research experiment, two groups of beams will be investigated. The first group consisting of four beams strong in flexure but weak in shear and the second group beams consist of three beams strong in shear but weak in flexure. Reinforced concrete strengthened and unstrenghtened beams will be tested for flexure and shear under four point loading. To check for the energy absorption capacity, a concrete cylindrical specimen wrapped with CFRP will be compared to the unwrapped under compressive load.

Ultimate strength of RC beams will be examined with applying different amounts and different orientation of CFRP, to understand the behaviour of a strengthened structural member well and realize what parameters affect the failure mode and load-bearing capacity. The aim of this thesis is therefore to investigate and improve the understanding of the behaviour of reinforced concrete beams strengthened with CFRP.

1.4 Organisation of Thesis

This study consists of six chapters:

The problems of RC structures and the aim of the study are given in chapter one. In the second chapter, the historical background of FRP as strengthening material in civil engineering is examined. The different types and some important mechanical properties of FRP and finally a concise literature review of experimental work on CFRP is presented.

In the third chapter, the general mode of failures in simple beam are mentioned. Also the wrapping schemes according to ACI 440.2R-08 and TEC2007 are presented.

Chapter four reports the CFRP schemes for strengthening RC beams for flexure and shear strengthening through an experiment conducted. The general description of all the experimental specimens, geometry of the specimens, reinforcement details are given. Material properties of steel and epoxy resin are also presented. The detail of the bonding schemes adopted in the research and the test procedures are outlined.

In the fifth chapter, experimental observations are discussed to give insight to the overall behaviour of the specimens. Observation noted during the experiment such as cracking or crushing of the concrete or debonding of the CFRP strips/sheets are presented.

In the sixth and last chapter, results and suggestions are presented.

CHAPTER 2

FRP AS STRENGTHENING MATERIAL IN CIVIL ENGINEERING

2.1 General

This chapter presents the historical background of FRP as strengthening material in civil engineering. It also presents the different types and some important mechanical properties of FRP. In specific, CFRP is discussed in more detail due to the interest of this research and finally a concise literature review of experimental work on CRFP was conducted.

2.2 FRP as Strengthening Material

2.2.1 Background

The technique of using FRP for strengthening concrete structure started at almost the same time in the late 1980s, both in Europe and Japan. The concept of using FRP as an external bonding material was borrowed from the experience of strengthening structures using steel plates and steel jackets which was a well known successful technology in the 1980s especially for bridge retrofitting and rehabilitation. Research on bonding CFRP on the tension face (bottom side) for flexural strengthening of reinforced structure was first studied in Switzerland in 1987 by Urs Meier and his team members at the Swiss Federal Laboratories for material testing and research (EMPA). Since 1982, carbon fibre reinforced polymer composites have been successfully used at EMPA for post-strengthening of reinforced concrete beams, with tests performed on more than 70 flexural beams having spans of between 2 and 7 m (Meier, 1995). Since after, research on FRP as an external bonding material for reinforced concrete in strengthening different structural members were conducted in the most important research laboratories all over the world, with very interesting experimental results being reported (Andrei, 2011). Some of these important researchers include Meier 1987, Mufti et al. 1989, Khalifa and Nanni 2002, Rizkalla 2003, Anania et al. 2005. By 1997, more than 1,500 concrete structures worldwide had been strengthened with externally bonded FRP materials (Mirmiran et al., 2004).

2.2.2 Definition of FRP

FRPs are special materials classified under composite materials consisting of fibers with high strength enclosed in a polymer matrix shown in Figure 2.1. The polymer matrix has an important function of transferring stress between the fibres and also serves as a barrier against the effects of the environment. Combination of different matrix type and fibre type creates FRP with different properties which, is an advantage and disadvantage as an engineering material hence make its study more complicated. FRP materials can be used in most application thus leading to a greater number of properties and difficult in many cases to conclude FRP behaviours generally. Due to their remarkable formability, FRP systems gives flexibility to be applied as strengthening method on any flat, curved or geometrically irregular surfaces (Andrei, 2011).

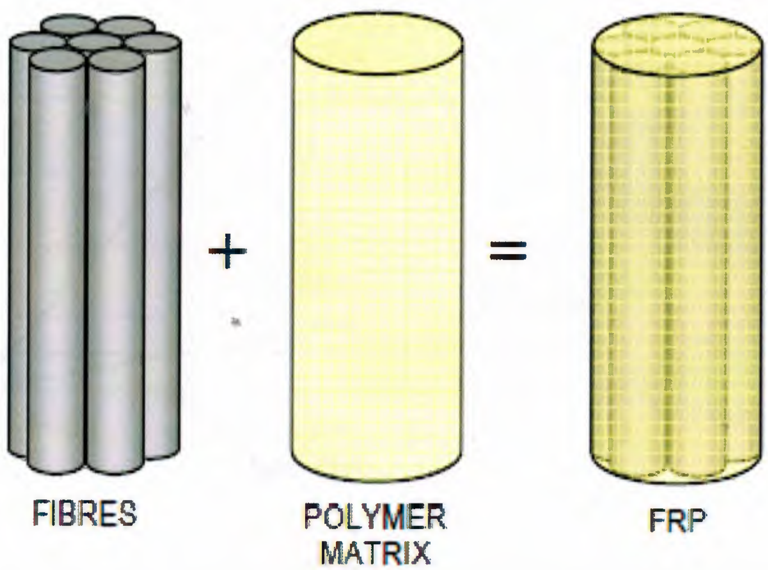

Figure 2.1: Components of FRP Material
(Bisby and Fitzwilliam, 2006)

2.2.3 Type of Fiber and their Properties

There are different types of FRP composites, but the most commonly used ones are classified into three types: Aramid fibre reinforced polymer also known as Kevlar 49 (AFRP), Carbon fibre reinforced polymer (CFRP), and Glass FRP (GFRP), depending on fibres type. Some mechanical properties of the FRPs are shown in Table 2.1. Even though FRP materials are more expensive than steel, they have become an attractive substitute for steel in strengthening systems for concrete structures due to their great number of advantages shown in Figure 2.2 and Table 2.1 which includes: high modulus of elasticity, high tensile strength, high strength to weight ratio, easy and reliable surface preparation, high fatigue resistance, corrosion resistance, easy and reliable surface application, durability of strengthening system, reduced mechanical fixing, and reduced construction period (Andrei, 2011), (Al-Salloum and Almusallam, 2002).

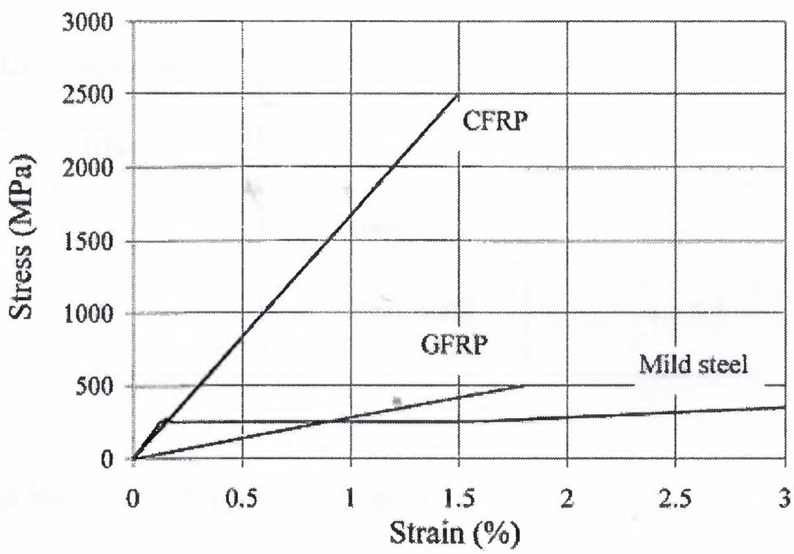

Figure 2.2: Typical Stress-Strain Relation for FRP Materials and Mild Steel (Al-Salloum and Almusallam, 2002)

Table 2.1: Properties of Fiber Reinforced Polymers (Andrei, 2011)

Material	Tensile Modulus (GPa)	Ultimate Tensile Strain (%)	Elastic Modulus (GPa)
Aramid			
High Modulus	3500-4000	2.5-3.5	115-130
Low Modulus	3500-4100	4.3-5.0	70-80
Carbon			
High strength	3500-4800	1.4-2.0	215-235
Ultra high strength	3500-6000	1.5-2.3	215-235
High modulus	2100-2400	0.5-0.9	350-700
Ultra high modulus	2100-2400	0.2-0.4	500-700
Glass			
E	1900-3000	3.0-4.5	70
S	3500-4800	4.5 5.5	85-90

2.2.4 Mechanical Properties of FRPs

The combined effect of the fibre and the matrix give the unique physical and mechanical properties of FRP. Although the strength and stiffness of FRP composites are greatly influenced by the fibres, the overall material properties depend also on the mechanical properties of the matrix, fibre volume fraction, cross sectional area, orientation of the fibre within the matrix and the method of manufacturing. Some important mechanical properties are discussed below.

2.2.4.1 Strength

Most FRPs are applied as tensile reinforcement in concrete structures due to the effectiveness in tension and impotent in compression. The two component material of the FRP and their failure strain are two factors that determine how effective an FRP material is in tension (Bisby and Fitzwilliam, 2006).

Ultimate compressive strength on the other hand is attained due to micro buckling of the fibre, shear failure and lastly the matrix transverse failure, but in few cases like in the pultruded FRP in bending, the ultimate compressive strength is determine by the total load. Figure 2.3 represents a stress-strain curve for different unidirectional (fibre in one direction) FRP materials. It can be concluded from the graph shown that both GFRP and AFRP have moduli that are substantially less than steel in the zone just before yielding, but CFRP have moduli that are almost same to or even higher than steel in some cases. The graph concluded that ultimate strength of FRP is higher than steel (Bisby and Fitzwilliam, 2006).

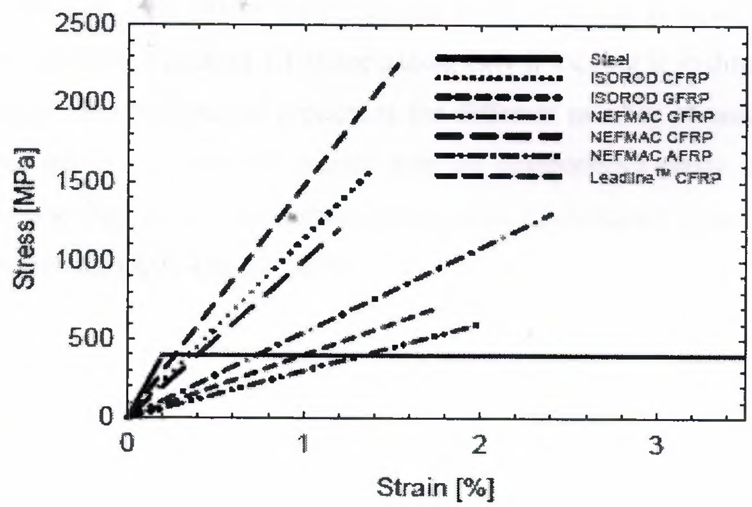

Figure 2.3: Tensile Stress-Strain Curves for Different FRP Materials (Bisby and Fitzwilliam, 2006)

2.2.4.2 Modulus of Elasticity

Modulus of elasticity is the slope of stress vs. strain curve of the material. The modulus of elasticity of unidirectional (fibres in one direction) FRP composite material is obtained from the ultimate strength and stiffness when the composite is loaded in tension parallel to fibre direction (Bisby and Fitzwilliam, 2006). The elastic modulus of the FRP, E_{frp}, is expressed in Equation 2.1.

$$E_{frp} = E_m V_{mi} + E_f V_{fi} = (E_f - E_m) V_{fi} + E_m \qquad 2.1$$

Where,

E_m Elastic modulus of matrix

E_f Elastic modulus of fibres

V_{mi} Volume fraction of matrix

V_{fi} Volume fraction of fibre

Because the modulus of elasticity perpendicular to the fibres is generally very much lower for unidirectional fibre, Equation 3.1 is applicable only if loading is in direction of the fibres. Tables 2.2 shows the mechanical properties for different number of unidirectional FRP in tension, hence tension is generally greater than the compressive elastic modulus achieved. Depending on the fibre in use, the compressive elastic modulus are always about 50-80% of the tensile test results (ACI 440.2R, 2008).

Table 2.2: Comparison between the Three Major Types of FRP Material
(Bisby and Fitzwilliam, 2006)

Factor	Type of Fibre		
	Aramid	Glass	Carbon
Tensile Strength	Very Good	Very Good	Very Good
Bulk Density	Excellent	Adequate	Good
Modulus of Elasticity	Good	Adequate	Very Good
Long term behaviour	Very Good	Adequate	Good
Alkaline Resistance	Very Good	Adequate	Good
Fatigue Behaviour	Good	Adequate	Excellent
Price	Adequate	Very Good	Adequate

2.2.4.3 Fatigue

Fatigue is a process of progressive, permanent, and internal structural change in a material subjected to repetitive stresses. As an excellent engineering material, FRP display good fatigue behaviour when compared with steel used in most civil engineering strengthening application with CFRP displaying superior fatigue characteristics (Bisby and Fitzwilliam, 2006). Despite the great number of publications on the basic mechanical properties of carbon fiber such as flexural behavior, creep, compressive strength, and tensile strength, there are insufficient to predict the useful life remaining for a strengthened member under fatigue loading (Al-Rousan and Issa, 2011).

The fatigue behaviour of FRP is gaining more consideration in the recent years. Al-Rousan and Issa (2011) reported the fatigue performance of reinforced concrete beams strengthened with CFRP sheets conducted both experimentally and analytically. The results indicate that increase in number of CFRP layers and CFRP contact area with concrete have a considerable decrease on mid- span permanent deflection, and an increase in stiffness and ultimate load. Tensile fatigue tests conducted on unidirectional CFRP materials have indicated that it can sustain much greater stresses than steel. GFRP are considered to be less stiff, and therefore GFRP matrices experience larger strains during load cycling which lead to more matrix cracking and can eventually lead to failure, while AFRP display intermediate fatigue behavior between GFRP and CFRP. Through de-fibrillation, AFRP will eventually fail in fatigue because AFRP is innately sensitive (Bisby and Fitzwilliam, 2006).

2.3 CFRP and its Application Field

Carbon fibre reinforced polymers also known as graphite fibres, that are thin fibres with diameter about 0.005-0.010 mm carbon atoms. The carbon atoms are bonded together in the microscopic crystals and exactly aligned parallel to the long axis of the fibre as shown in Figure 2.4 and Figure 2.5. Alignment of the crystals makes the fibre incredibly strong considering its size. The density of carbon fibre reinforced polymer is noticeably lower than steel, which is an excellent property for low weight.

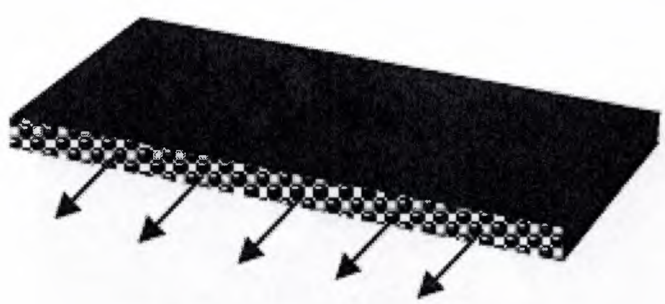

Figure 2.4: Direction of Stresses Carried by the Fibres (Triantafillou, 2003)

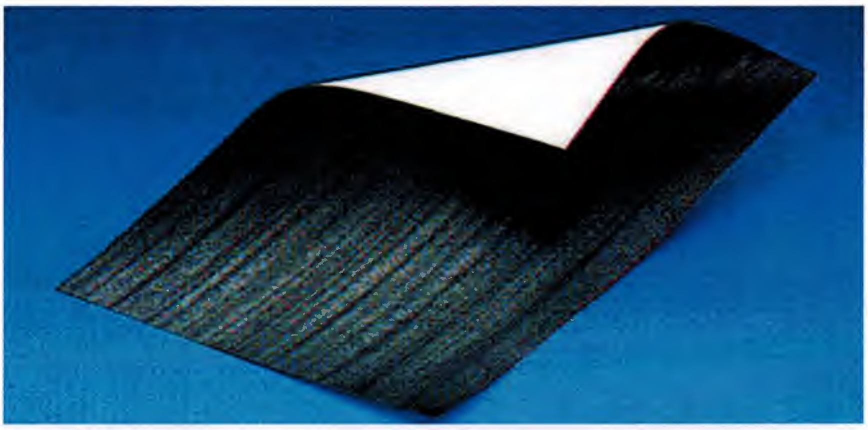

Figure 2.5: Unidirectional Carbon Fibre Reinforced Polymer Sheet (Bisby and Fitzwilliam, 2006)

Mechanical properties of carbon fiber such as high tensile strength, low weight, etc., make it application well accepted in other field which includes military, motor sports as shown in Figure 2.6, aerospace as shown in Figure 2.7, and other competition sports. It is also used in compressed gas tanks, racing vehicles with vehicle shell commonly composed of the material often in combination with AFRP and GFRP. Carbon fibre is extensively used in the bicycle industry, especially for high-performance racing bikes (Teng, 2001). Carbon fibre reinforced polymer CFRP are considerably more expensive than AFRP and GFRP, but are more widely used in structural engineering applications, due to their superior properties.

Figure 2.6: CFRP Thermoset Car Body Structure (Shahawy, 2010)

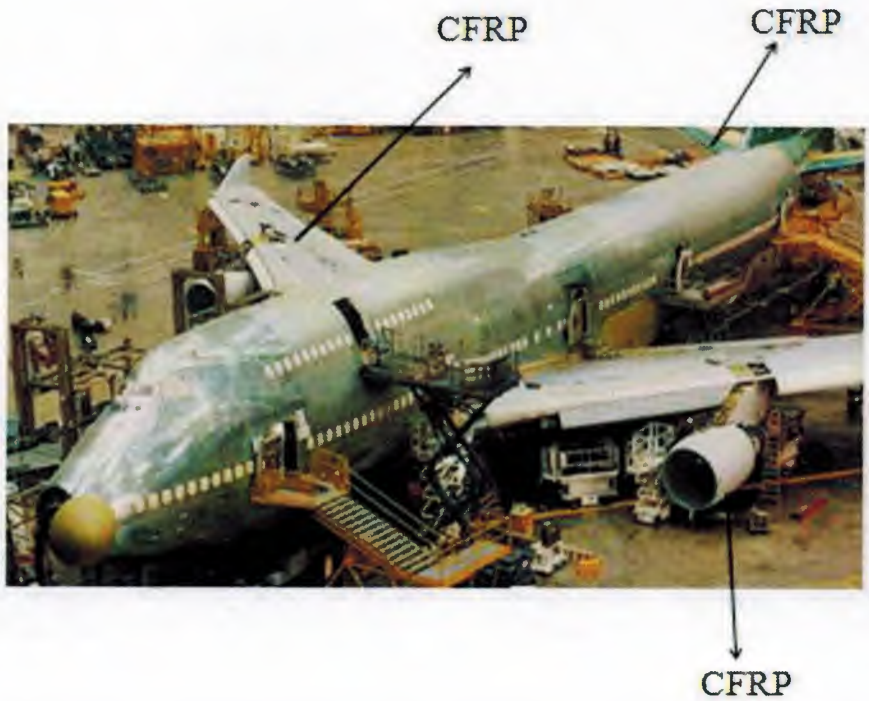

Figure 2.7: Airbus 380 Wings Constructed with CFRP (Shahawy, 2010)

2.3.1 CFRP Application in Civil Engineering

The used of CFRP is an accepted solution for repairing or strengthening action in the field of civil engineering around the world. It gained popularity in recent years.

One of the world largest CFRP application for strengthen is the "West Gate Bridge" in Melbourne, Australia. The bridge was designed in 1960 and the construction was completed in 1978. The idea to strengthen the structure without any additional construction work on the superstructure was due to the heavily increased load in traffic. This was achieved by using external post-tensioning within the box cells together with externally bonding CFRP sheets (Hollaway, 2004).

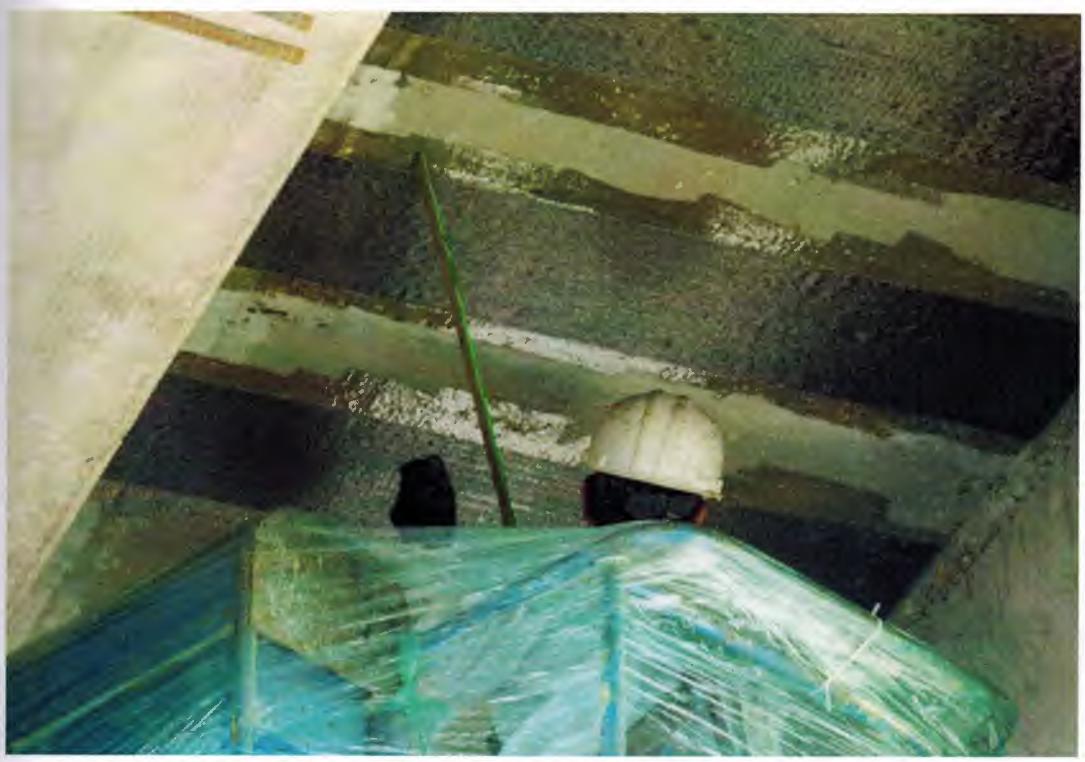

Figure 2.8: Deck Strengthening of Bridge Using CFRP (Shahawy, 2010)

Another CFRP application technique was in Quebec, Canada. After badly deteriorated due to corrosion of steel reinforcement, the 50 years old deck slab of the famous Joffre Bridge was reconstructed using CFRP. This was the first Canadian Bridge deck slab completely reinforced with FRP. Other numerous projects involving CFRP includes, rehabilitation of Webster Parkade Sherbrooke, Canada, Portage Creek Bridge Victoria, British Columbia, Ibach Bridge, Lucerne, Bible Christian Bridge Cornwall, UK, Hythe Bridge Oxford, UK, Silver Springs Equestrian Bridge Maryland, USA, Clear Creek Bridge Kentucky, USA, among others (Hollaway, 2004). The technique of using FRP reinforcement instead of bonded steel plates was for reasons of substantial economy. Additional material costs of CFRP over steel were neglected by practical aspects, as no heavy lifting, cutting or welding equipment would have been required as is the case with steel, and labour hours would have been significantly less (Irwin and Rahman 2002).

Figure 2.9: CFRP Laminates Strengthening of a Box Girder (West Gate Bridge) (Irwin and Rahman 2002)

2.4 Previous Experimental Studies

In the course of this investigation, a review of the broader literature was undertaken. In the interest of space, only a brief summary is given below.

Shahawy (1995), conducted a research which involved an experimental investigation of the flexural behavior of reinforced concrete beams bonded with carbon fiber reinforced polymer CFRP laminates. Six numbers of reinforced rectangular beams were investigated. Four were casted with minimum steel reinforcement according to ACI 318-89 with 203×305×2744mm. One was left as a control beam. The beams were cured for 4 weeks before the application of the CFRP strengthening. Ultimate strength and the moment-deflection behavior of the beams were obtained theoretically; the results were compared with the experiment results. The effect of CFRP laminates, cracking behavior, deflections, serviceability loads, ultimate strength and failure modes were also examined. Significant reduction in deflection with increasing number of CFRP laminates was also concluded.

Swamy and Mukhopadhyaya (2000), research study was to check the plate debonding occurance when CFRP laminates are used as externally reinforcement to strengthen reinforced concrete beam structural elements. It also investigated the amount of tension reinforcement, the concrete strength, the amount of shear reinforcement and the location and arrangement of the externally bonded anchorages. Seven beams were tested, one was the control (unplated) while the other six beams were strengthened with one 1.5mm thick CFRP bonded on the tension face. Both beams failed in flexure and the maximum load at failure was recorded as 199.9KN. Anchoring of the CFRP plate against vertical and horizontal movement is very important to maintain the effectiveness and integrity of the bonded plate and consequently ensure a ductile failure. The results also confirmed that premature plate debonding must be avoided in order to achieve the optimum ductility of CFRP plate-bonded beams.

Balendran et. al. (2001), investigated the flexural behavior of plain and composite beams and compared the results. The reinforced concrete beams were externally bonded with steel plates or CFRP on the tension face. A three point bending test was carried out on each of the three sets of beams classified as small size beams, medium size beams, and large beams with each type of strengthening. The influence of the plate thickness, beam size, concrete strength and adhesive type, on the moment capacity, deflection, and failure mode of the strengthen beams were reported. It was reported that there was 120% increase in the moment capacity and a 40% increase in the stiffness of the plain beam bonded with plates as thin as 1mm. With the increase in plate thickness, moment capacity of bonded beams increase almost proportionally and the failure mode varied gradually from flexure to shear. It was reported that the failure mode changes from shear in small beams to flexure/shear in medium beams and inclined to more flexure in large beam. There was no change in the failure mode of plain and CFRP bonded beams by increasing the grade of the concrete.

Khalifa and Nanni (2002), concluded that the shear strength of beams is increased by CFRP composite, from the experimental investigations conducted on twelve full-scale reinforced concrete simply supported beams which were designed to fail in shear. The parameters investigated are steel stirrups, shear span- depth ratio, finally amount of, and the distribution of CFRP. Also the results were used to validate a shear design approach.

Using thinner laminates allows full utilization of CFRP fabric and the laminates should be placed perpendicular to shear crack if possible, this was reported in a paper title "*Strengthening Concrete beams For Shear with CFRP Sheets*" by Täljsten (2003), on an experimental investigation conducted on seven beams under four point loading. One of the beams was left as a reference beam (without CFRP bonding) while the other six were strengthened with varying angles and thickness (weight) of the fibre. Also no steel stirrups was used in the shear region, this to differentiate the shear contribution from the concrete and CFRP sheets.

Buyukozturk, et. al. (2003) stated that, the failure pattern which demands attention and raises concerns is the sudden brittle manner in which the CFRP plate debonded prior to ultimate failure. Hence, this particular failure pattern deserves further close and critical examination. Almost all the failure that occurred on the tested beams indicates that the FRP was not fully utilized and failure type was changed from ductile to brittle. Such failures, unless adequately considered in the design process, may significantly decrease the effectiveness of the strengthening or repair application.

A research paper authored by Pham and Al-Mahaidi (2004) report that retrofitting RC beams with thicker CFRP does not always lead to higher capacity. The average ultimate load for the beams retrofitted with 2, 6 and 9 layers of CFRP are 148.3, 145.3 and 126.4 kN, respectively. They also reported that concrete cover and stirrup spacing have insignificant effect on the beam ultimate load capacity, but the stirrup spacing changes the failure mode. The research was conducted experimentally on a total of eighteen (18) RC beams. Two were control beams and sixteen were retrofitted with CFR fabrics. All of the beams were tested in four-point bending with the span of 2300 mm and the shear span of 700 mm. The beam width was 140 mm and the CFRP fabric width was 100 mm. Parameters that were checked in the test includes the CFRP bond length, the area of tension reinforcement, the concrete cover, the number of plies and the amount of shear reinforcement.

Fereig et al. (2005), conducted a research on the repair and rehabilitation of reinforced concrete beams using CFRP and GFRP fabrics. A total number of five beams were designed to fail in shear and were loaded to failure. Results showed that both GFRP and CFRP regained an improved the ultimate carrying capacity and ductility of the investigated beams, with GFRP showing excellent result compared to CFRP.

Ma'en et al. (2007), conducted an investigation on the shear behavior of reinforced concrete beams strengthened by different orientation type and varying number of CFRP bonded layers using epoxy-adhesives. Thirty eight beams were tested experimentally with vertically, horizontally and 45^0 orientation of CFRP laminates. It was recorded that an increase in ultimate strength occurred as the CFRP layers in the beam increases. They also concluded the beam bonded over the entire depth and shear span showed the greatest increase in shear strength.

Han (2008), conducted an experiment on T beams with the intention of increasing deformability in FRP strengthened beams. One was left as unstrengthened control beam, and the other 16 beams were classified based on the strengthened methods as non prestressed EB, non-prestressed NSM, 40% prestressed NSM, and 60% prestressed NSM. To allow investigation of the effect of partially unbonding, each group has different unbonded lengths and includes a fully bonded beam as the variables to compare. It was reported that the prestressed NSM strengthening system is much more effective to improve the ultimate load carrying capacity and the serviceability in comparison to the non-prestressed NSM beams.

Siddiqui (2009), published a paper on the study of reinforced beams strengthened with FRP composite materials using an effective and efficient practical strengthening schemes. Two groups of beams were designed according to ACI 318-02 and investigated experimentally. First group beams were designed to be strong in shear and weak in flexure, and the second group beams were designed to be strong in flexure and weak in shear. All beams were tested under same loading conditions, and was concluded that the U shaped end anchorage beam has the higher flexural strengthening and inclined strips are effective in improving shear capacity of the beam.

Bukhaari et al (2010), studied the shear strengthening of reinforced concrete beams with Carbon Fiber Reinforced Polymer (CFRP) sheet. Seven, two span continuous reinforced concrete (RC) rectangular beams of cross section 152mm×305mm and beam length 3400mm were tested. One beam was left as control beam (un-strengthened) and the remaining six were strengthened with different arrangements of CFRP sheet. They studied orientation of fiber vertical and some oriented at 45^0 as main variables. The tests showed that it is beneficial to orientate the fibres in the CFRP sheet at 45^0 so that they are approximately perpendicular to the shear cracks. Also the paper reviewed existing design guidelines for strengthening beam in shear with CFRP sheets and proposed a modification to the concrete society technical report TR55.

Andrei (2011), studied and analyzed the response behavior of retrofitted reinforcement concrete beam with CFRP laminates. The load-displacement analysis, deformation behavior, displacement profile, crack pattern and type of failure modes were investigated. Five reinforced concrete beams 200×400×4000mm were tested, one as control, two strengthened with single layer of CFRP laminate and the last two were strengthened with two layers of CFRP laminates on the tension side of the beams. All the beams were bonded externally with CFRP beam performed better than the control beam both in strength and stiffness. It was also reported that the strength of the strengthen beams are factor of the original stiffness of the beams, type and amount of the CFRP layers. The four beams failed in FRP debonding while the control failed due to steel yielding. The properties of the adhesive are probably important in relation to the debonding failure. The strain recorded in the CFRP laminates indicated that the capacity of the composite system was not fully utilized (40%).

Obaidat et al. (2011), conducted an experiment study and presented the results. The investigation was to ascertain the behaviour of structurally damaged full-scale reinforced concrete beams retrofitted with CFRP laminates in shear/flexure. Load was applied on the RC beams until crack was noted, then retrofitted with CFRP. The parameters considered were the internal reinforcement ratio, position of retrofitting and the length of CFRP. The results showed that the stiffness of the retrofitted beams in shear and flexure using CFRP laminates are restored and the strength nearly equal to or greater than those of the control beams.

To investigate the effect of CFRP laminates on the flexural behavior of reinforced concrete beams, Guibing et al. (2012) compared the side-bonded beams with the soffit-bonded beams.

6 side-bonded CFRP sheets RC beams, 2 soffit-bonded beams, and 1 control beam were tested using 4 point load. The side-bonded beams were anchored using two different schemes. It was reported that the flexural behavior of the beams with side-bonding CFRP laminates is worse than that of the beams by soffit bonding, and the flexural behavior of the strengthened beams with rigid anchor of CFRP laminates is better than that of the beams with flexible anchor. To improve the ultimate load capacity of beams, the side-bonded arrangement is not needed.

Venkatesha et al, (2012) investigated the efficiency of Carbon Fibre Reinforced Polymer (CFRP) strips in enhancing shear capacity and/or changing the mode of failure brittle shear failure to ductile flexural failure. Reinforced concrete beams simple supported rectangular section 100×200×1500mm, 3 wrapped and 3 controlled (unwrapped), cured for 28days were tested experimentally. The type of wrapping (CFRP) was one layer discrete u-type CFRP strip. The variables in the research were the shear span-depth ratio and the spacing between the steel stirrups. Results showed a change of failure mode from shear to almost flexural mode and the ultimate strength of wrapped beams increased.

Singh (2013) concluded that there exists a critical value of shear force in the control beam, up to which shear strain in the beam is not significant. However, the strength is far beyond this critical value for the beams strengthened externally with CFRP sheets. The conclusion was based on a research experiment on four precast RC beams with inadequate amount of steel stirrups (design for shear failure) with the aim to determine the optimum lay-ups most efficient combination of fabric lay-up for sheets used to strengthen RC beams in shear and to determine the most efficient combination of the lay-ups. Three of the beams were strengthened on the vertical side with different orientation lay-ups strengthening fabric, 45°, 0°/90°, and 0°/90°/45°, while the last one was left as a control beam (unstrenghtened). The results shows that Beam-45°, Beam-0°/90°, and Beam-0°/90°/45° have 25%, 19%, and 40% increases in shear-load carrying capacity in comparison to the control beam, respectively.

CHAPTER 3

FAILURE IN SIMPLE BEAM AND CFRP WRAPPING SCHEMES GUIDELINES.

3.1 General

The main aim of this research is to investigate shear and flexure behaviour of reinforced concrete beams which are externally wrapped by CFRP. The failures in simple reinforced concrete beam and its mechanism, is important to provide solutions. This chapter covers the general mode of failure in simple beam. The flexural failure is more preferable and more clear than the shear failure, because shear failure occurs without warning and should be avoided as much as possible. Also the wrapping schemes according to ACI 440.2R-08 and TEC2007 were presented.

3.2 Failure Types in Simple Beam

3.2.1 Flexural Failure

Tension cracks are the element of flexural failure and occur when the principal tensile stress in beam almost reaches the tensile strength of the concrete. When the beam is reinforced with an adequate amount of reinforcements and loaded beyond its ultimate capacity, the longitudinal reinforcement yields excessively resulting in failure in the concrete known as flexural failure. Yielding of reinforcement is due to large amount of stresses above its yield point, making the tension cracks in the concrete widen visibly and propagates upward with significant deflection in the beam. The crack is almost vertical and causes failure of the beams as shown in Figure 3.1. Flexural failure in reinforced concrete beams occurs in the region of maximum moment and when the ultimate bending capacity is exceeded (bottom mid span of the beam). Flexural failure is more preferably to other types of failure because it is gradual and precedes by visible signs of distress and show increase in deflection. It is seen from Figure 3.1 that the vertical flexural cracks occur at the mid-span, resulting in a redistribution of stress (Winter and Nilson 1972).

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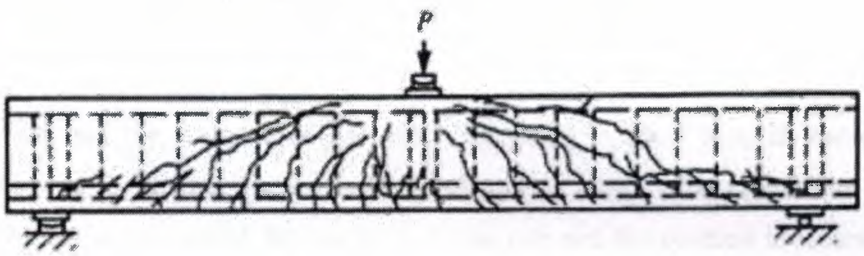

Figure 3.1: Flexural Failure (Chen, 2007)

3.2.2 Diagonal Tension Failure

Shear failure also known as (diagonal tension failure) in reinforced concrete beams is the most undesirable mode of failure due to its rapid progression and have high levels of unprediction. It happens so sudden and is hazardous. Diagonal tension failure was the major problem in reinforced concrete beam since the initial usage of concrete. Experimental test were conducted for decades to have the knowledge of the phenomena of how it occurs and its causes. Diagonal tension failure mechanism varies depending upon the cross-sectional dimension, geometry, type of loading and properties of member. Diagonal cracks are main mode of diagonal tension failure in reinforced concrete beams, located near the supports and caused by excess applied shear forces. The diagonal crack starts after the last flexural crack at mid span, where it follows direction of the reinforcing steel and the concrete at support as shown in Figure 3.2. Beams fail immediately upon formation of critical cracks in the high shear region near the beam supports (Winter and Nilson 1972).

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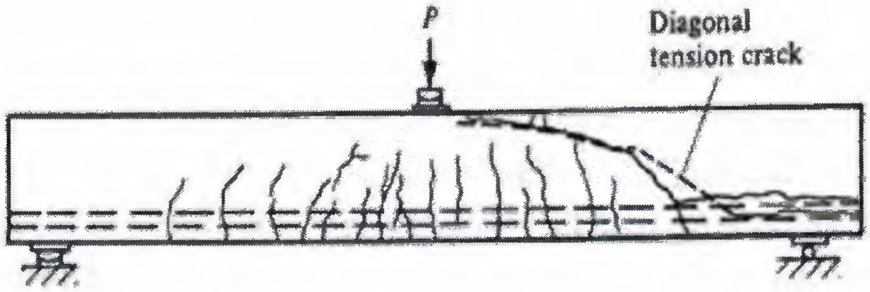

Figure 3.2 Diagonal Tension Failure (Chen, 2007)

The behaviour and the analysis of RC beam members in shear is quite more complex compared to that in flexure. Hence, it is mandatory to study what causes the shear and principal stresses to understand the cracking of concrete and the position it occurs along the span of the concrete. Principal tensile stress acts on the tension face at an approximately 45° plane with respect to the axis of the beam at section close to the support. By investigating other stresses at different cross sections of the beam, it was observed that the principal stresses vary throughout the beam. Two systems of orthogonal curves, called stress trajectories, that give the directions of the principal stresses can be constructed (Gere and Timoshenko, 2001). The crack pattern can be predicted from these trajectories as shown in Figure 3.3.

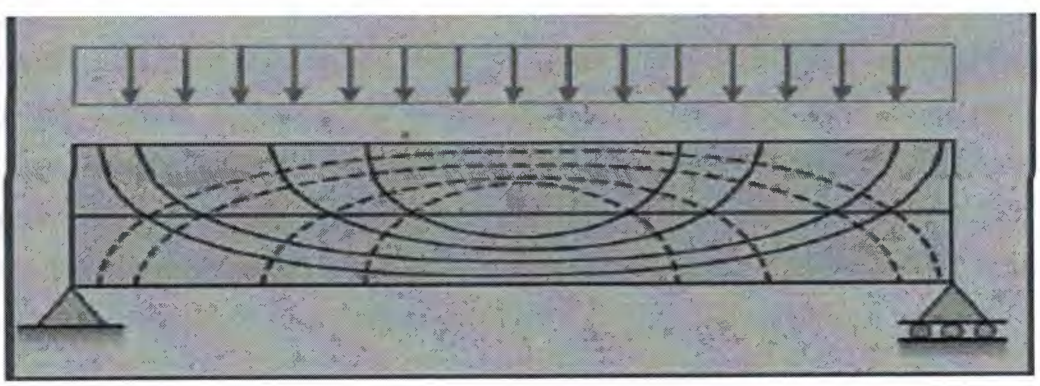

Figure 3.3 Principal Stress Trajectories (Gere and Timoshenko, 2001)

3.2.3 Shear Compression Failure

Shear compression failure occurs when the diagonal shear crack propagates through a beam and reaches the compression zone without any sign of secondary cracks as in the case of shear tension failure and with crushing of concrete near the compression flange above the tip of the inclined crack as shown in Figure 3.4. It mostly occurs in short beams. The ultimate load at failure is considerably more than at diagonal cracking as a result of arch action, and load is transferred to the supports in direct compression in a truss-like action (Winter and Nilson 1972).

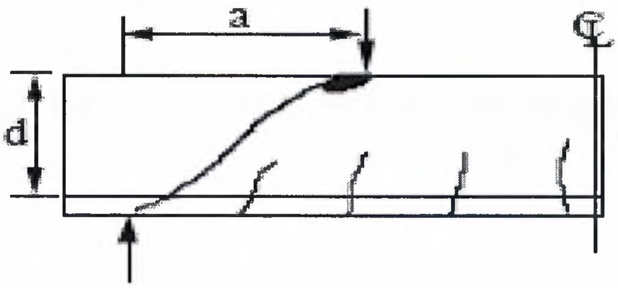

Figure 3.4: Shear Compression Failure

3.3 Wrapping Schemes According to ACI 440.2R-08 and TEC2007

To increase the shear capacity of a beam, ACI 440.2R-08 and TEC2007 recommends three wrapping schemes as; completely wrapping scheme, U-wrapping scheme, and two side wrapping scheme, as shown in Figure 3.5. For all wrapping schemes, the CFRP can be applied either continuously along the longitudinal axis of the beam or discrete strips. The continuous wrapping has disadvantage of preventing migration of moisture. All wrapping schemes can be applied with different orientation to achieve the best possible results.

Complete wrapping is the most effective and efficient scheme. However, this is almost not possible in most structures due to the presence of slab. Therefore U-wrapping scheme and two side wrapping scheme are more applicable (ACI 440.2R-08).

Figure 3.5: Typical Wrapping Schemes for Shear Strengthening using FRP Laminates (ACI 440.2R-08)

3.4 Design Philosophy of ACI 440.2R-08

The design philosophy of RC externally strengthened beams with CFRP in ACI 440.2R-08 is based on the traditional reinforced concrete design principles stated in ACI 318-05 and knowledge of the mechanical behaviour of CFRP materials. FRP materials have high tensile strength, so they are designed to resist tensile forces while maintaining strain compatibility between CFRP and concrete surface and neglecting compressive forces. Therefore the overall nominal strength of a member is concluded based on the possible failure modes and the outcome from the strain and stress of all the materials. ACI 440.2R-08 also recommend for an additional reduction factor to be applied as a contribution of CFRP reinforcement to reflect uncertainties attributed in CFRP systems. These reduction factors were determined based on the statistical evolutions of differences in mechanical properties, field application and predicted versus full-scale test results.

The nominal shear strength of a beam member externally strengthened with FRP material can be determined by adding the shear contribution of, the concrete, the reinforcing bars, and the CFRP material as derived by Khalifa et.al (1998) and by Traintafillou and Antonopoulous (2000). Figure 3.6 shows the parameters used in shear strength calculation.

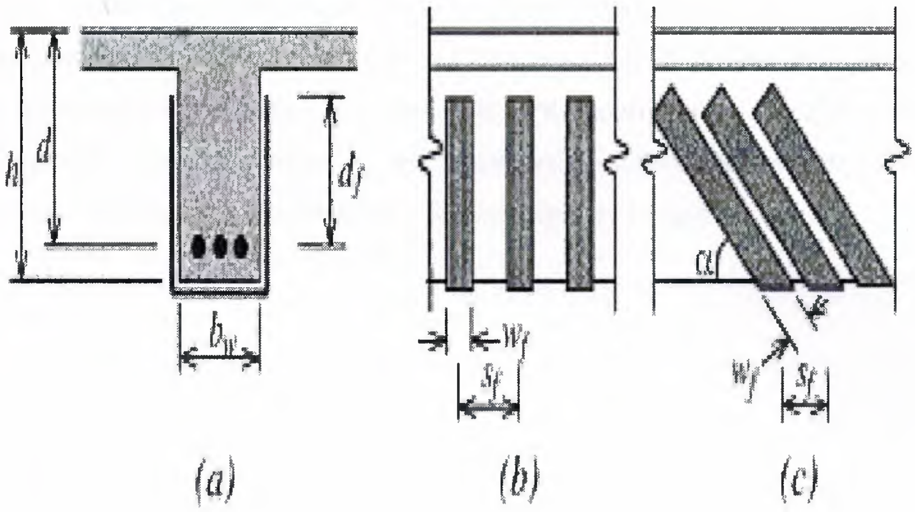

Figure 3.6: Dimensional Parameters Used in Shear Strengthening Calculations with CFRP Laminates (ACI 440.2R-08)

According to ACI 440.2R-08 and TEC2007, the spacing "s" in discrete strip bonding should not be greater than one fourth of the depth plus width of the strip, as shown in Equation 3.1 or according to ACI 318-05 provision for shear spacing in ACI 440 2R-08.

$$s_f \leq w_f + \frac{d}{4} \qquad 3.1$$

3.5 FRP Shear Strength Contribution

Due to the complexity nature of shear design even in simple reinforced concrete beams, determining the precise contribution from FRP in shear is still under investigations with the results not conforming to the generalized prediction models. Research conducted by Traintafillou and Antonopoulous, Sas et al., Bousselham and chaallal etc. held out great data information, with the investigated parameters are shear span-depth ratio, compressive strength of concrete, internal transverse reinforcement bar, longitudinal reinforcement bar and the relationship between failure modes with CFRP configuration. Overall contribution of FRP to the shear strength is based on orientation and assumed crack pattern and considered the bond mechanism that exists between FRP and concrete surface.

CHAPTER 4

EXPERIMENTAL STUDY

4.1 General

This chapter reports the CFRP schemes for strengthening RC beams for flexure and shear strengthening through an experiment conducted at Near East University, Department of Civil Engineering, Nicosia, TRNC. The general description of all the experimental specimens, geometry of the specimens, reinforcement details are given. Material properties of steel and epoxy resin are also presented. The detail of the bonding schemes adopted in the research and the test procedures are outlined.

4.2 Description of the Specimens

The beam with section dimension of 300x300mm was initially considered. Using a scale factor of 1:2, (which is a ratio comparing the scaled measurement to the actual measurement) the beams were casted with sectional dimension of 150x150mm. The beams were casted in steel and wooden moulds as shown in Figure 4.1-2 and Figure 4.4. The reinforced concrete beams were categorized in two groups. The first group consist of four specimens which were designed to be strong in flexure and weak in shear (shear failure) and the second group consist of three specimens designed to be strong in shear and weak in flexure (flexural failure). All beams were strengthened with CFRP strips/sheets externally. In each group, one beam was taken as a control beam.

4.3 Geometry of the Specimens

The beams were designed in accordance with American Concrete Institute ACI 318-11. The dimension of the reinforced concrete beams was 150×150×750 mm. The beams were designed with a concrete cover of 30 mm. Group 1 beams: Strong in flexure and weak in shear were designed with 0.0129 steel ratio and the group 2 beam: Strong in shear and weak in flexure have a steel ratio of 0.0057. Table 4.1 shows the parameters used in design calculations. Detail design calculations are shown in Appendix 1.

Table 4.1: Summary of Group 1 and Group 2 Beam Design

Beam	b (mm)	h (mm)	d (mm)	f_c(MPa)	f_y(MPa)	A_s (mm^2)	ρ	ρ_{eff}
Group 1	150	150	116	30	420	226.2	0.0129	0.0102
Group 2	150	150	117	30	420	157	0.0089	0.0062

Figure 4.1: View of Group 2 Beam Reinforcement

Figure 4.2: Moulds with Reinforcement before Casting

4.4 Reinforcement Details

A total of seven reinforced concrete beams were casted in two groups. Group 1 containing four beams and group 2 containing 3 beams.

Group 1 beams designed to be strong in flexure and weak in shear. Group 1 beams had a cross-section of 150×150mm and a span of 750mm. As the beam of group 1 were made strong in flexure, reinforced with 2Ø12mm steel bars at bottom part of the beam, with Ø8mm stirrups at 200mm spacing. A Ø8mm steel bar was used at the upper part of the beam to tie up the stirrups as shown in Figure 4.3 and Figure 4.4.

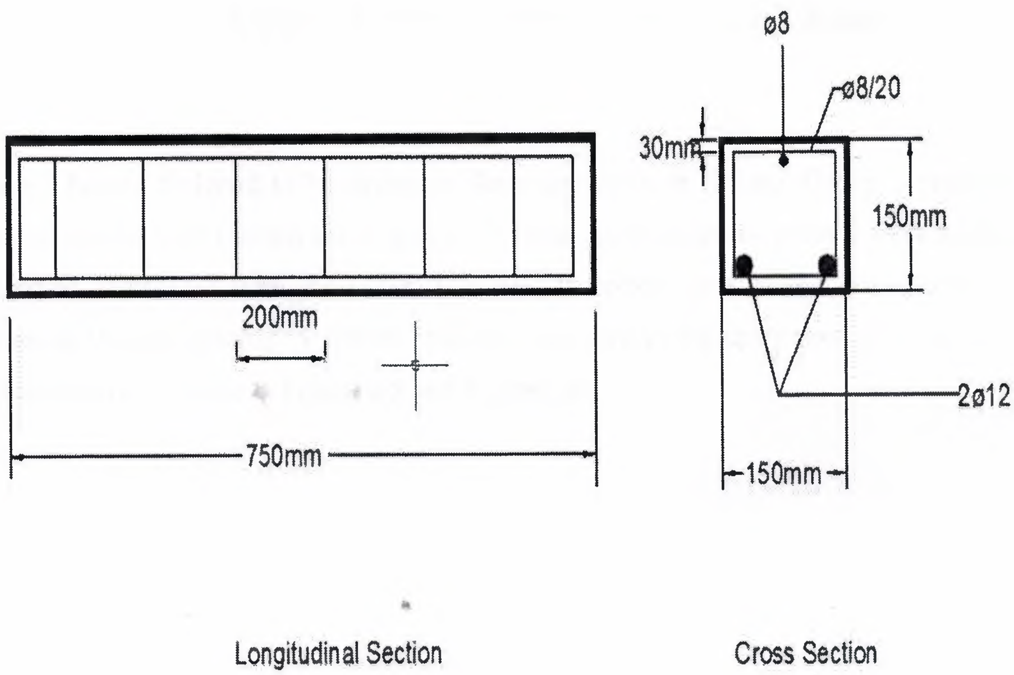

Figure 4.3: Detail of Group 1 Beams

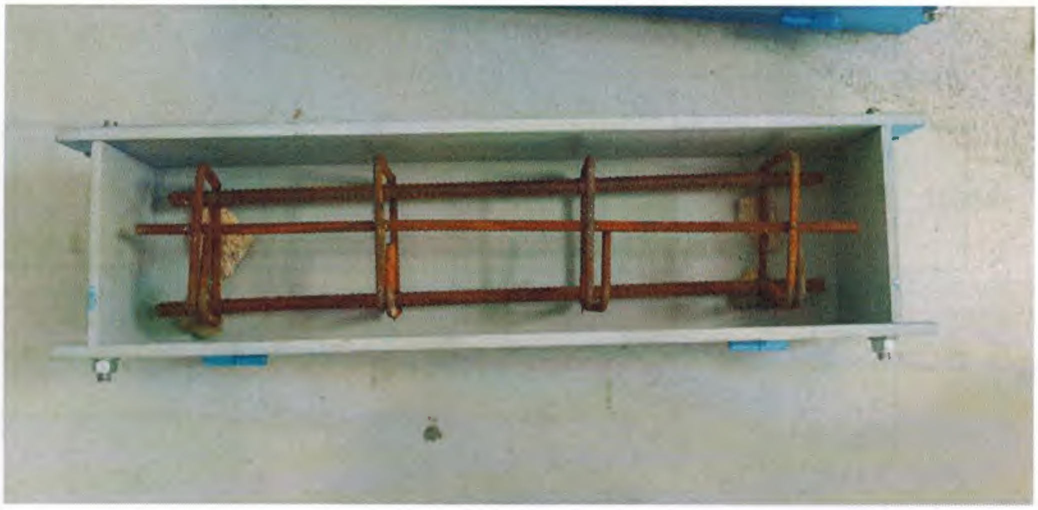

Figure 4.4: Internal Reinforcement of Group 1 Beams

Group 2 beams designed to be strong in shear and weak in flexure. Group 2 beams had a cross-section of 150×150mm and a span of 750mm. As the beam of group 2 were made weak in flexure, reinforced with 2Ø10mm steel bars at bottom part of the beam, with Ø8mm stirrups at 100mm spacing. A Ø8mm steel bar was used at the upper part of the beam to tie up the stirrups as shown in Figure 4.5 and Figure 4.6.

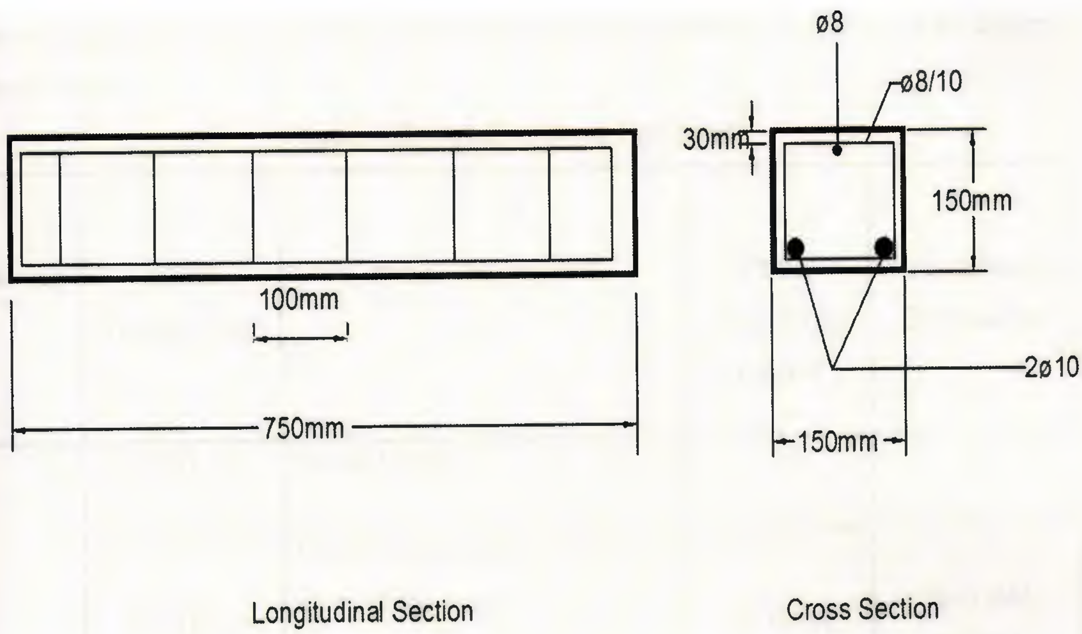

Figure 4.5: Detail of Group 2 Beams

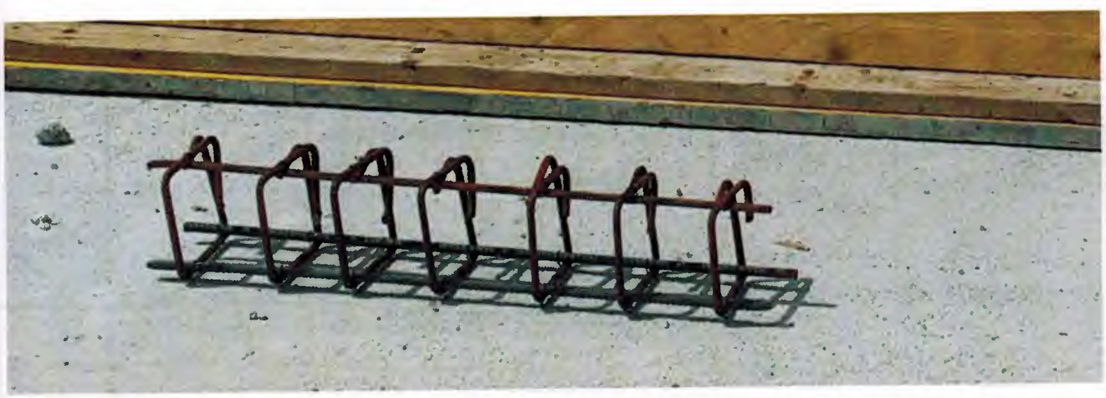

Figure 4.6: Internal Reinforcement of Group 2 Beams

The nomenclature used for different beam configurations are shown in Table 4.2 for easier identification.

Table 4.2: Detail of Beam Specimens

Group	Beam Designation	Beam Description	CFRP Sheet/Strip Layers	Strengthening Orientation
1	CBM1-G1	Control beam	-	-
	SBM1-G1	CFRP was bonded to both sides of the beam perpendicular to longitudinal axis	Strip (1)	Two side bonded
	SBM2-G1	CFRP U-strips were bonded to the beam	Strip (1)	U-bonded
	SBM3-G1	Inclined CFRP strips (45^0) were bonded to both side of the beam, parallel to the shear crack	Strip (1)	45^0 side bonded
2	CBM1-G2	Control beam	-	-
	SBM1-G2	CFRP sheet was bonded to the beam on the tension side	Sheet (1)	Tension face
	SBM2-G2	CFRP sheet was bonded to the beam on the tension side with U-strip end anchorage	Sheet (1)	Tension face

4.5 Material Properties

4.5.1 Concrete

Ready mixed concrete of grade C30 was used for conducting the research, which was obtained from Near East University Mosque construction site, and was ordered from a local supplier, Tüfekçi Ltd. Seven rectangular reinforced concrete beams, four cylindrical specimens and three cube specimens were casted as shown in Figure 4.7 and Figure 4.8 which were later tested experimentally. The concrete was poured in the upright position of the formwork and was carefully vibrated in order to provide proper consolidation. The curing was made by water spraying for three days, continuously. After three days, formworks were removed and specimens were lifted and placed in curing tank. The specimens were later cured in water for 25 day with 21 ± 2^0 C temperature and 100% relative humidity. The three cubes and four cylindrical specimens were tested in the 28^{th} day. The 28 days old specimens tested under uniaxial compression, using fully Automatic Compression Test Machine (UTC-4320) 2000 kN load capacity, and BC 100 control unit, as shown in Figure 4.9. This machine has been designed for reliable and consistent testing of concrete samples. After weighing the cured specimens, load was applied at a constant pace rate of 0.6MPa/s on the cube and cylindrical specimens until failure occurred on the specimens. Table 4.3 and Table 4.4 shows the results obtained from the compressive strength test, in which the readings were taken carefully.

Figure 4.7: Beams Casting

Figure 4.8: View of Casted Specimens

Figure 4.9: Automatic Compression Test Machine (UTC-4320)

Three cube and cylindrical specimens were cast, according to procedure EN12390-3 to determine the average values of the concrete compressive strength. The beam specimens, the cylindrical and the cube specimens were all kept in the same place in the laboratory under the same condition. They were moisture cured for the first three days and then lifted and placed in curing tank for 28 days. The beams, cylinders and cubes were not tested the same day, but in the same week. Appendix 2 shows the graphical result obtained from the test.

Table 4.3: Compressive Strength of Concrete Cylindrical Specimens

Specimen	**Mass (kg)**	**Compressive Strength (MPa)**
1	7999.5	42.06
2	8043.5	40.71
3	7991.0	39.48
Average	**8011.3**	**40.75**

Table 4.4: Compressive Strength of Concrete Cube Specimens

Specimen	Mass (kg)	Compressive Strength (MPa)
1	12730	32.6
2	12795	35.1
3	12810	37.2
Average	**12778.3**	**35.0**

4.5.2 Reinforcing Steel Bars

S420 deformed bars were used for both longitudinal and transverse reinforcements of all members. Ø10mm and Ø12mm bars were used for longitudinal reinforcements and Ø8mm bars were used for as transverse ones. To evaluate the mechanical properties of steel bars, tensile tests were conducted on samples of each diameter. Tensile test was carried out at Near East University, Civil engineering laboratory, using Universal Testing Machine (UTM-4000) 600kN load capacity, with BC 100 control unit to obtain the yield strength, ultimate strength, modulus of elasticity and percentage elongation values of the steel reinforcing bars as shown in Figure 4.10. The load was applied at a constant pace rate of 0.6MPa/s until failure occurred on the specimens. Table 4.5 shows the valuable information about the mechanical properties of steel reinforcement. Appendix 2 shows the graphical result obtained from the test.

Figure 4.10: Universal Testing Machine (UTM-4000)

Table 4.5: Mechanical Properties of Steel Reinforcement

Steel Bars diameter (mm)	Ultimate Stress (MPa)	Yield Stress (MPa)	% Elongation	Modulus of Elasticity (GPa)
Ø8	567	446	17.4	210
Ø10	588	456	21.3	208
Ø12	705	551	14.5	210

4.5.3 CFRP Material

FRP is a new class of composite material manufactured from fibres and resins which has proven to be efficient and economical for the development and repair of new and deteriorating structures in civil engineering. The CFRP material used is unidirectional sheet with fibres oriented in the one direction only (along the longitudinal axes). The strength of fibre material in longitudinal direction is far greater than that in latitudinal and diagonal directions. There are three main types of CFRP according to elastic moduli known as high strength, high modulus, and ultra-high modulus. For this research, SikaWrap-300C CFRP sheet of 0.17mm thickness was obtained from a local supplier in Nicosia, TRNC. The CFRP sheet and mechanical properties of the sheet are shown in Figure 4.11 and Table 4.6 as obtained from the supplier respectively. The data sheets are shown in and Appendix 3.

CFRP application can be summarized briefly as mounting (bonding) of CFRP strips/sheets to concrete surfaces by aid of special resins and requires special care. Generally, environmental conditions of CFRP application process must be examined carefully during the winter (lower than 5^0C) and summer (in excess of 20^0C) season.

Figure 4.11: CFRP Sheet Used for Flexural Strengthening

Table 4.6: Properties of CFRP Obtained from supplier

Density of Fibre	$1.8 g/cm^3$
Thickness	0.17mm
Tensile Strength	$3900 N/mm^2$
Tensile Elastic Modulus of Fibre	$230000 N/mm^2$

4.5.4 Epoxy Resins

Epoxy resins are generally used to bond the CFRP on the concrete surface, which can be applied in both shear strengthening and flexural strengthening of beams. Success of the strengthening technique primarily depends on the performance of the epoxy resins used for the bonding. Varieties of epoxy resins are commercially available for usage with wide range of physical and mechanical properties. The epoxy resins are generally available in two parts, a resin and a hardener shown in Figure 4.12.

Sikadur-330 epoxy resin (A and B) were used in this research study. Sikadur-330 epoxy resin (A) is the epoxy (white colour) and Sikadur-330 epoxy resin (B) the hardener (grey colour) which were mixed in the ratio 4:1. They were mixed thoroughly with a mixing tool for ten 10 minutes until the color was a grey and applied on the concrete surface using trowel. The mechanical properties of above mentioned materials is given in Table 4.7 and Appendix 3. These values were taken from the manufacturer product data sheet.

Figure 4.12: Sikadur-330 Epoxy Resin (A and B)

Table 4.7: Properties of Sikadur-330 Obtained from Supplier

Density of Mixed Resin	1.31kg/lt
Tensile Strength	$30N/mm^2$
Flexural Elastic Modulus	$3800N/mm^2$
Tensile Elastic Modulus	$4500N/mm^2$
Elongation at Break	0.9%

4.6 Wrapping Orientation

In the first group, first scheme, SBM1-G1 of group 1 beam that were designed to be strong in flexure, was externally bonded with CFRP strips both sides of the beam, perpendicular to the longitudinal axis. SikaWrap-300C unidirectional CFRP strips were used to strengthen the RC beam that is weak in shear. The strips are bonded perpendicular at an angle of 90^0 to the longitudinal axis of the beam with 80mm center to center spacing as shown in Figure 4.13

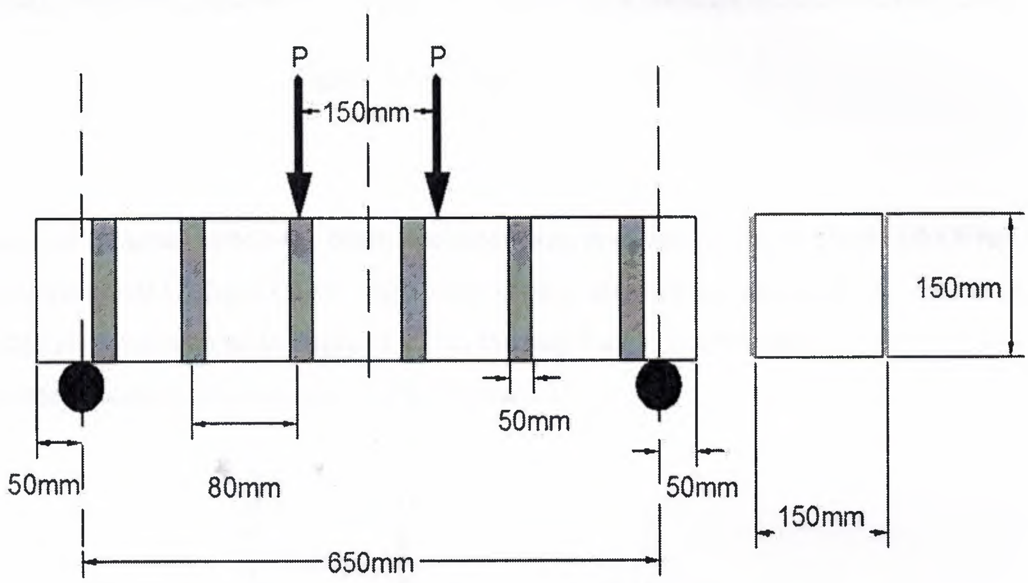

Figure 4.13: Schematic Representation of SBM1-G1 Beam

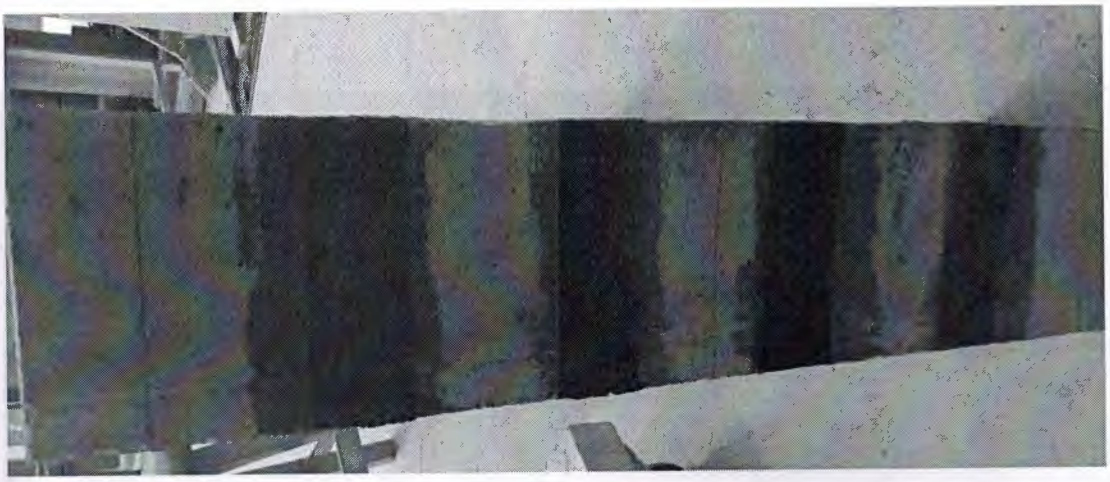

Figure 4.14: SBM1-G1 Beam

In the second scheme, SBM2-G1 beam specimen was designed to fail in shear. SikaWrap - 300C unidirectional U shape CFRP strips were used to strengthen beam in shear. The strips are bonded perpendicular at an angle of 90^0 to the major axis (longitudinal) of the beam with 80mm center to center spacing as shown in Figure 4.15

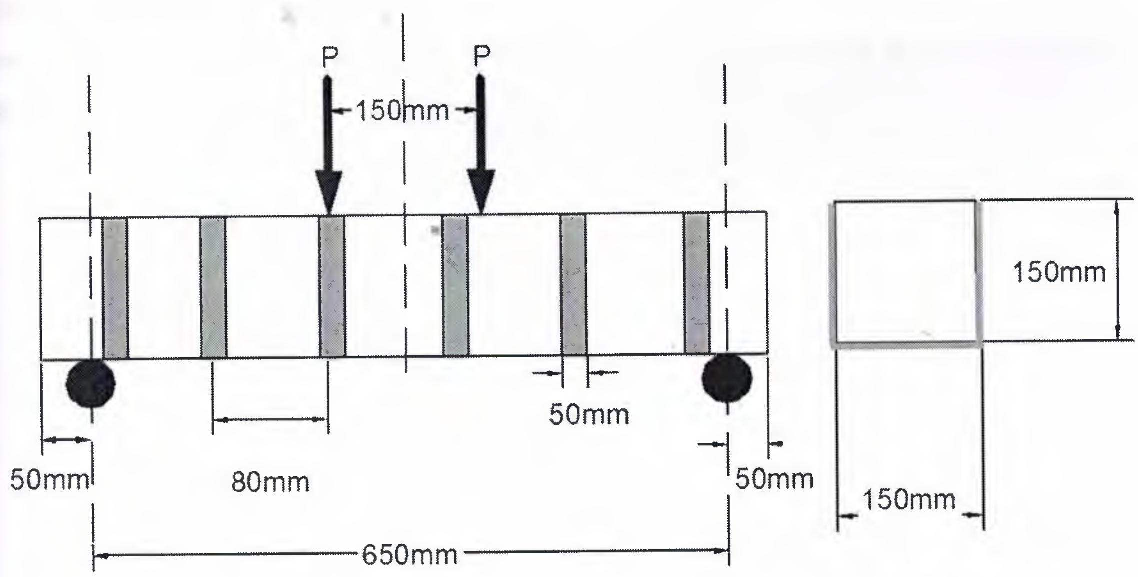

Figure 4.15: Schematic Representation of SBM2-G1 Beam

Figure 4.16: SBM2-G1 Beam

In the third scheme, SBM3-G1 beam specimen was designed to fail in shear. The beam was strengthened with SikaWrap-300C unidirectional CFRP strips bonded on both two sides. The strips are bonded at an angle of 45^0 to the major axis (longitudinal) of the beam and parallel to the expected diagonal shear failure at 80mm center to center spacing as shown in Figure 4.17

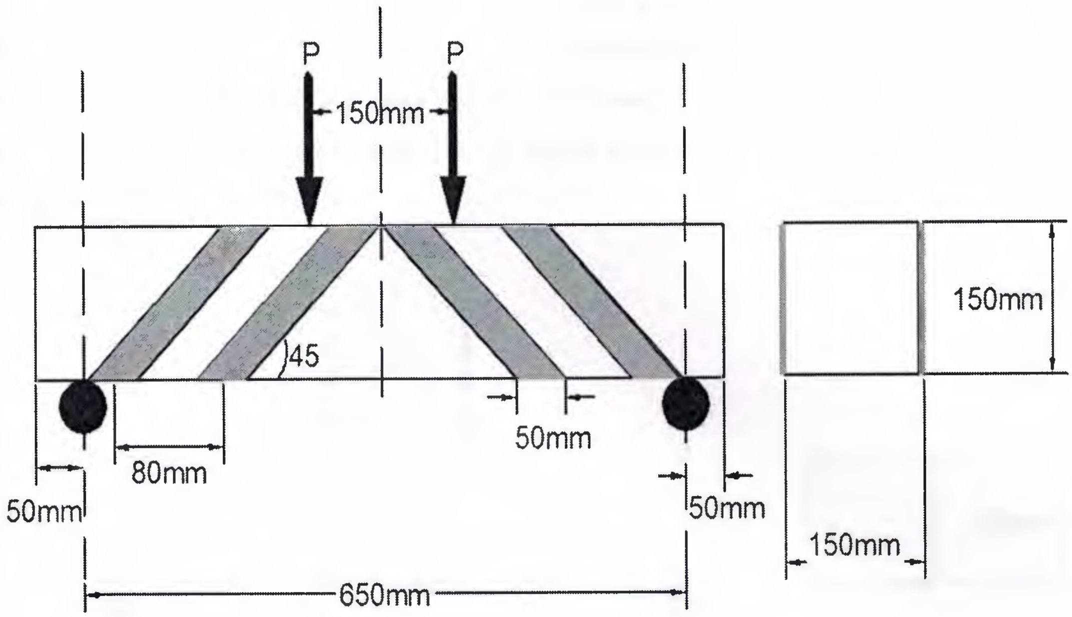

Figure 4.17: Schematic Representation of SBM3-G1 Beam

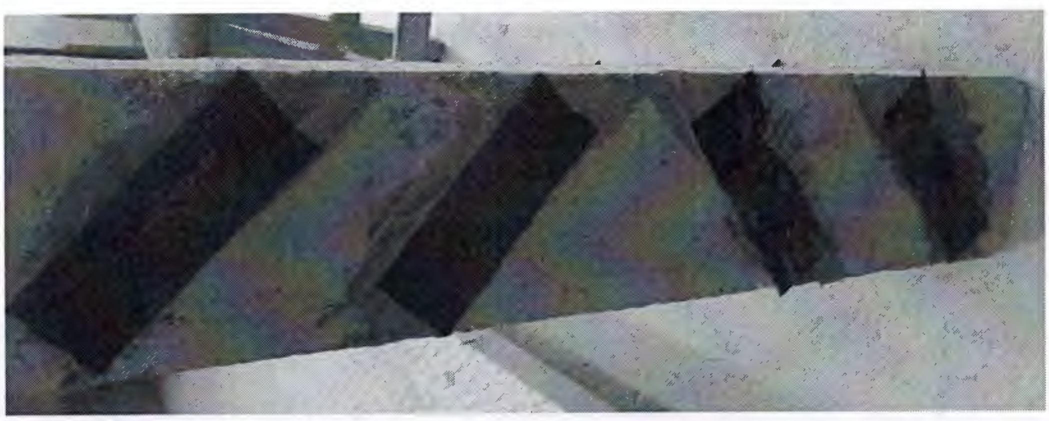

Figure 4.18: SBM3-G1 Beam

In the second group, first scheme, SBM1-G2 of group 2 beams that were designed to be strong in shear and weak in flexure, was externally bonded with SikaWrap-300C unidirectional CFRP sheet at bottom part of the beam in order to strengthen the beam in flexure. The sheet is orientated parallel to the major axis (longitudinal) of the beam as shown in Figure 4.19.

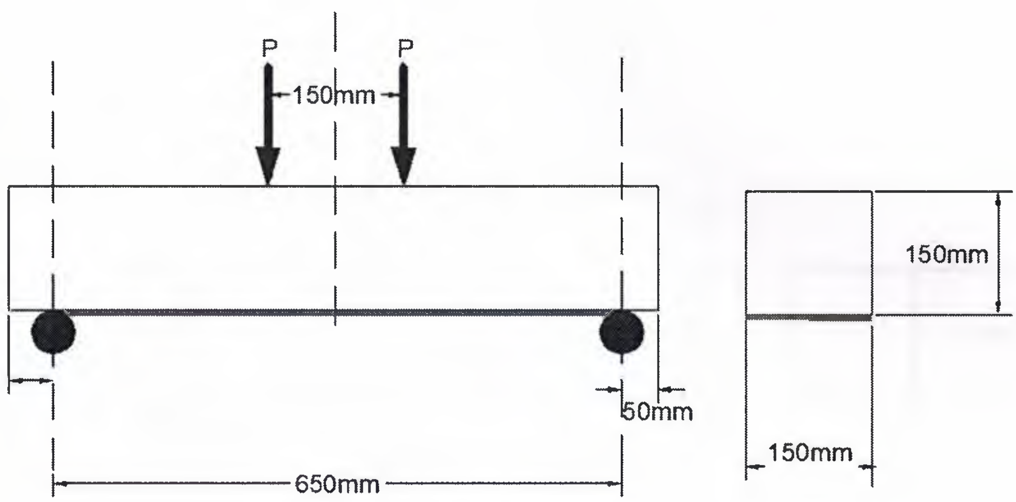

Figure 4.19: Schematic Representation of SBM1-G2 Beam

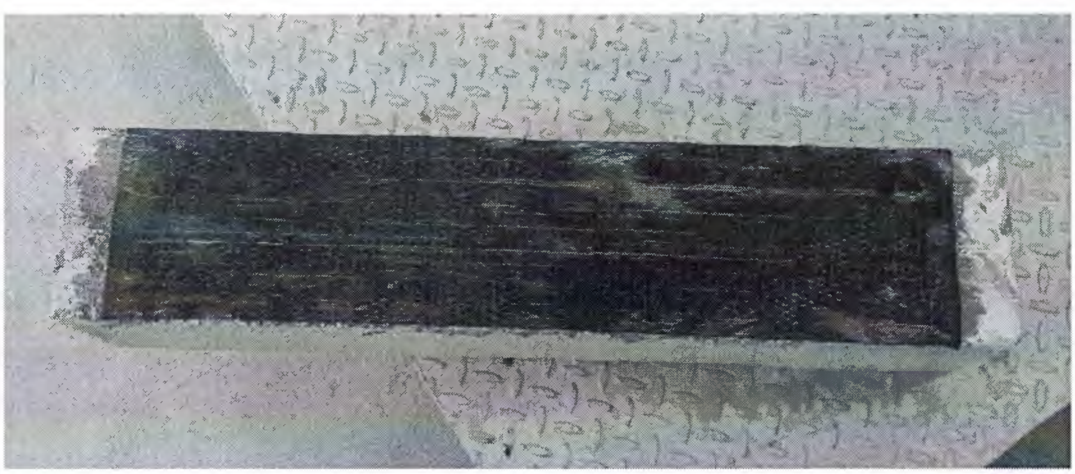

Figure 4.20: SBM1-G2 Beam

In the second scheme, SBM2-G2 beam specimen was designed to fail in flexure. SikaWrap - 300C unidirectional CFRP sheet bonded on the tension side (bottom of the beam) in single layer and CFRP U-strips were bonded at the ends of the beam to prevent any possible debonding of sheet. The sheet at the tension side is bonded parallel to the major axis (longitudinal) of the beam as shown in Figure 4.21

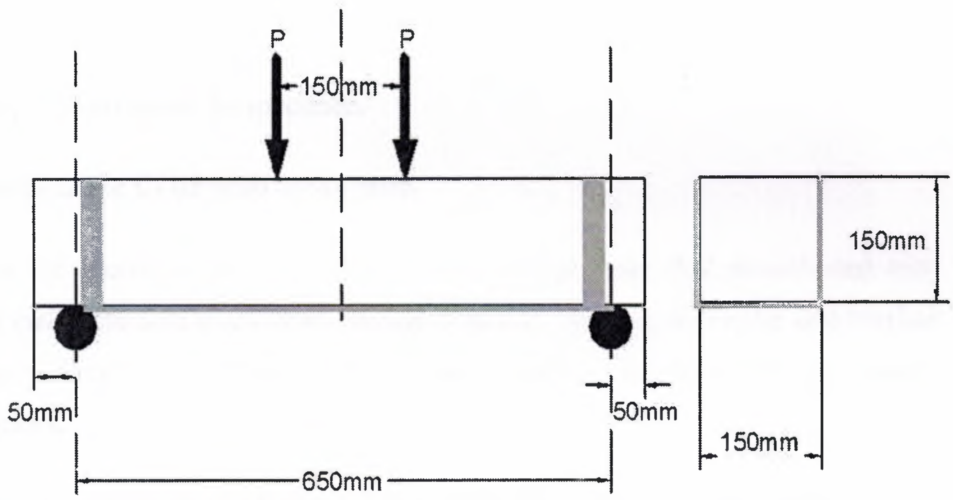

Figure 4.21: Schematic Representation of SBM2-G2 Beam

4.7 Experimental Procedure

The experimental research was carried out at Near East University, Civil engineering laboratory, Nicosia, TRNC. A total of seven reinforced concrete beams were tested with similar loading.

In group 1, three beams were strengthened externally and in group 2, two beams were strengthened. In order to investigate the behaviour of as-built structural members, 2 specimens, which were called as control specimens (CBM1-G1 and CBM1-G2), were tested first. The others were strengthened through a CFRP application to overcome the deficiencies observed in the tests of the control beams. All these specimens are labeled and explained in accordance as shown in Table 4.2.

The procedure followed for installing the CFRP strips/sheets to the subassemblies was as following:

1. Preparing the surface of the specimen.
2. Application of the CFRP with epoxy resin.

All the faces of the specimen, where CFRP is to be applied, were first smoothened with scratch paper. Finally, the specimens were cleaned from dust by using air blower and brushes to obtain a clean surface. CFRP strips/sheets were cut beforehand into prescribed sizes using appropriate scissors.

Sikadur-330 epoxy resin mixture of about 2 mm thick was applied on the surface of the concrete beams where the CFRP strips/sheets were positioned. Using a roller, the strips/sheets were squeezed against the surface to assure that there was no void between the strip/sheet and the concrete surface. All CFRP strips/sheets used in strengthening were of unidirectional SikaWrap-300C. After strengthening, the specimens were left undisturbed in the laboratory for 3 day before testing to make sure that the epoxy had enough time to cure.

All the beams were tested using Automatic Flexural Testing Machine (UTC-4620) with 200kN load capacity as shown in Figure 4.22. The beams were tested as simply supported with two points loading placed at equal distance from the supports. The loads was applied at a constant pace rate of 0.2MPa/s until failure occurred on the specimens.

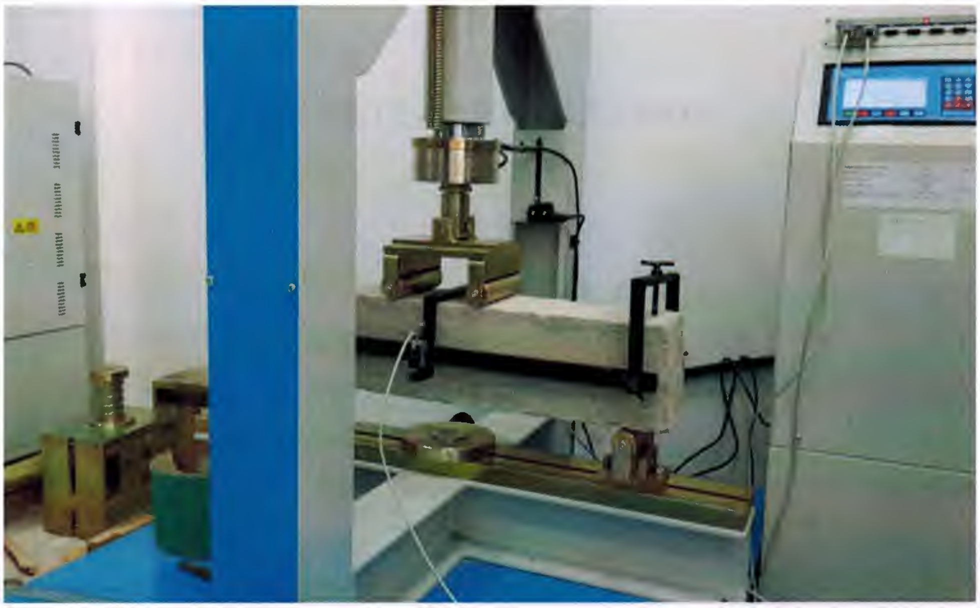

Figure 4.22: Four Point Loading on Beam Specimen Using Automatic Flexural Testing Machine (UTC-4620) 200kN

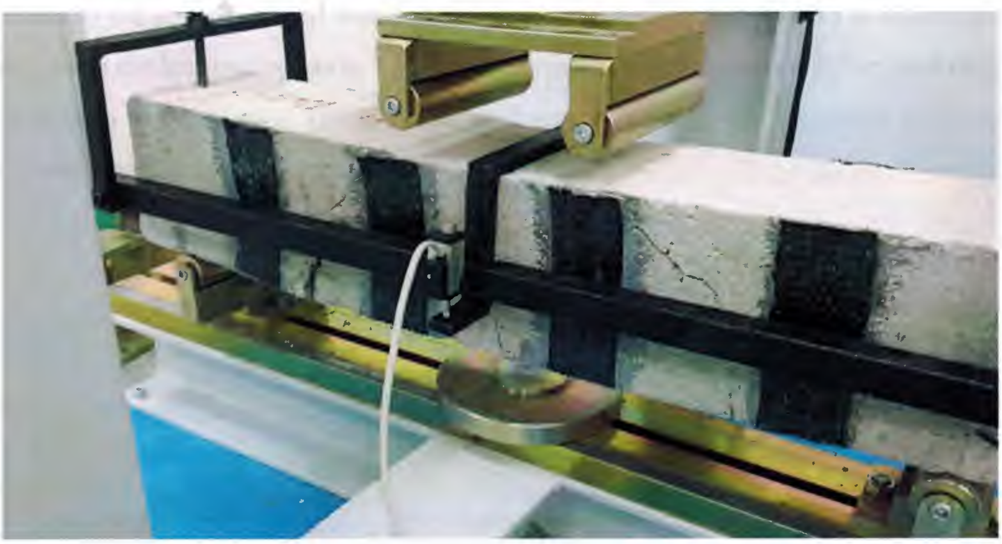

Figure 4.23: View of SBM3-G1 on Loading Frame

CHAPTER 5

RESULTS AND DISCUSSION

5.1 General

In this chapter, experimental observations are discussed to give insight to the overall behaviour of the specimens. Observation noted during the experiment such as cracking or crushing of the concrete or debonding of the CFRP strips/sheets.

5.2 Experimental Observations of Group 1 Beams

CBM1-G1

The control beam was designed as strong in flexure and weak in shear. It was loaded until failure. The ultimate capacity of CBM1-G1 was 74.2kN and failed due to diagonal tension failure as expected. The failure was observed from one support and started as a thin hair-like crack close to the support which widened gradually and propagated upward at approximately 45^{o} inclined to the longitudinal axis. This failure is pure diagonal tension failure and occurred sudden as seen in Figure 5.1. The ultimate load was reached in 409.2 seconds as shown on the load-time graph in Figure 5.2.

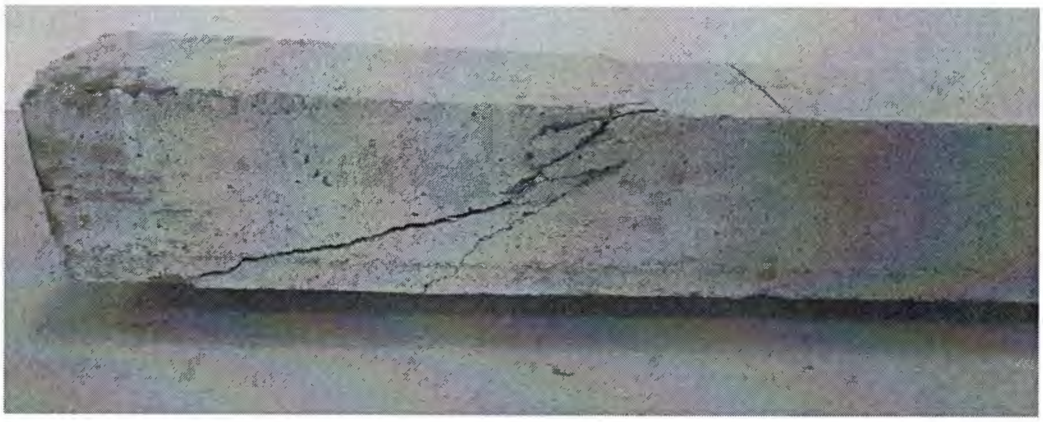

Figure 5.1: Failure in CBM1-G1

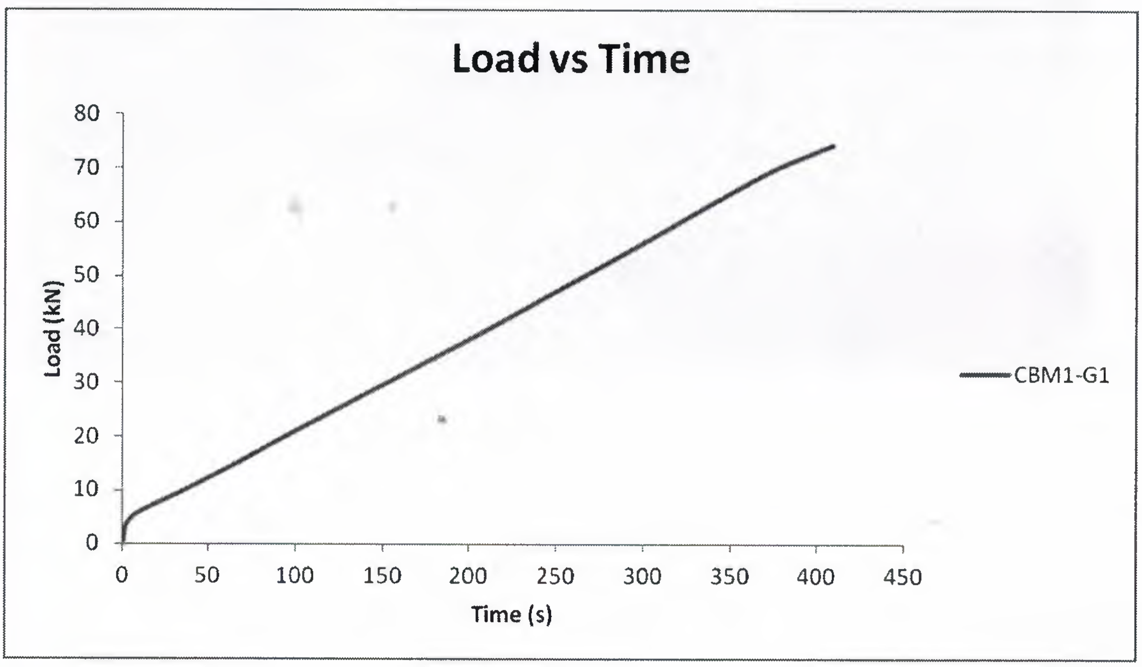

Figure 5.2: Load-Time Curve for CBM1-G1

SBM1-G1

The strengthened beam was loaded until failure. The ultimate load capacity of SBM1-G1 was 84.10kN which showed an increase in ultimate capacity compared to the control beam. The failure observed in both sides of the beam was due to concrete crushing close to the CFRP strips and some flexural crack at the mid-span of the beam as shown in Figure 5.3. The CFRP strips were not detached from the concrete. The ultimate load was reached in 517.83 seconds as shown on the load-time graph in Figure 5.4.

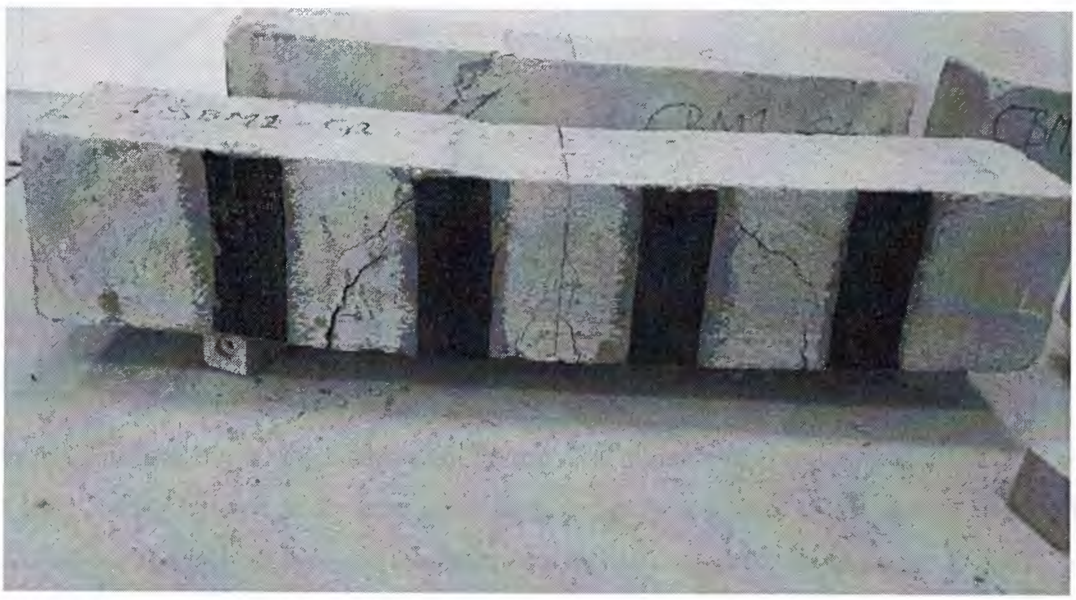

Figure 5.3: Failure in SBM1-G1

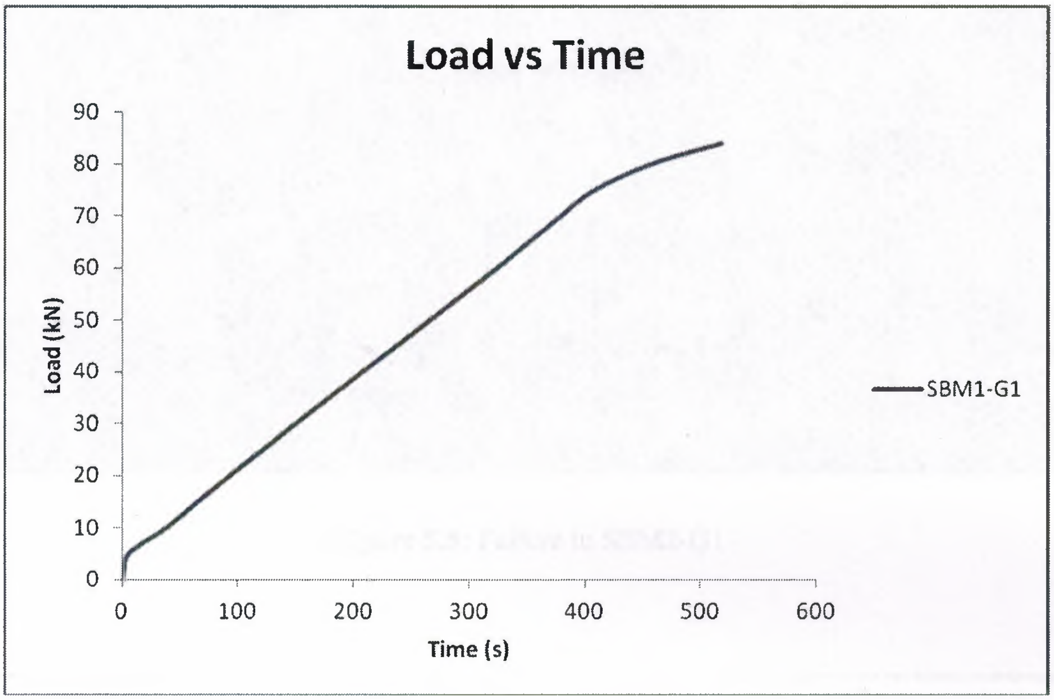

Figure 5.4: Load-Time Curve for SBM1-G1

SBM2-G1

The strengthened beam was loaded until failure. The ultimate load capacity of SBM2-G1 was 94.69kN which showed an increase in ultimate capacity compared to the control beam. The failure observed in both sides of the beam was due to concrete crushing close to the CFRP strips and some flexural crack as shown in Figure 5.5. The failure was almost sudden. The ultimate load was reached in 549.17 seconds as shown on the load-time graph in Figure 5.6.

Figure 5.5: Failure in SBM2-G1

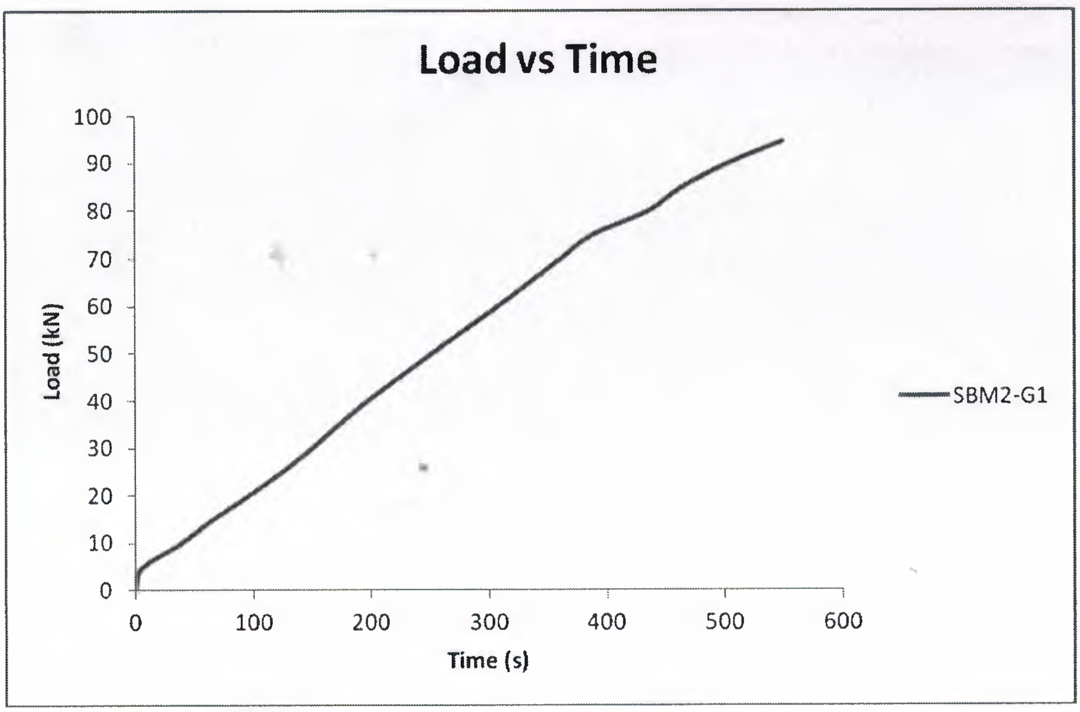

Figure 5.6: Load-Time Curve for SBM2-G1

SBM3-G1

The strengthened beam was loaded until failure. The ultimate load capacity of SBM3-G1 was 81.85kN which showed an increase in ultimate capacity compared to the control beam. The failure observed in both sides of the beam was due to concrete bursting close to the CFRP inclined strips and some flexural crack that started from the center and bottom of the beam to almost the center of the beam as shown in Figure 5.7. The CFRP strips were not detached from the concrete. The ultimate load was reached in 516.38 seconds as shown on the load-time graph in Figure 5.8.

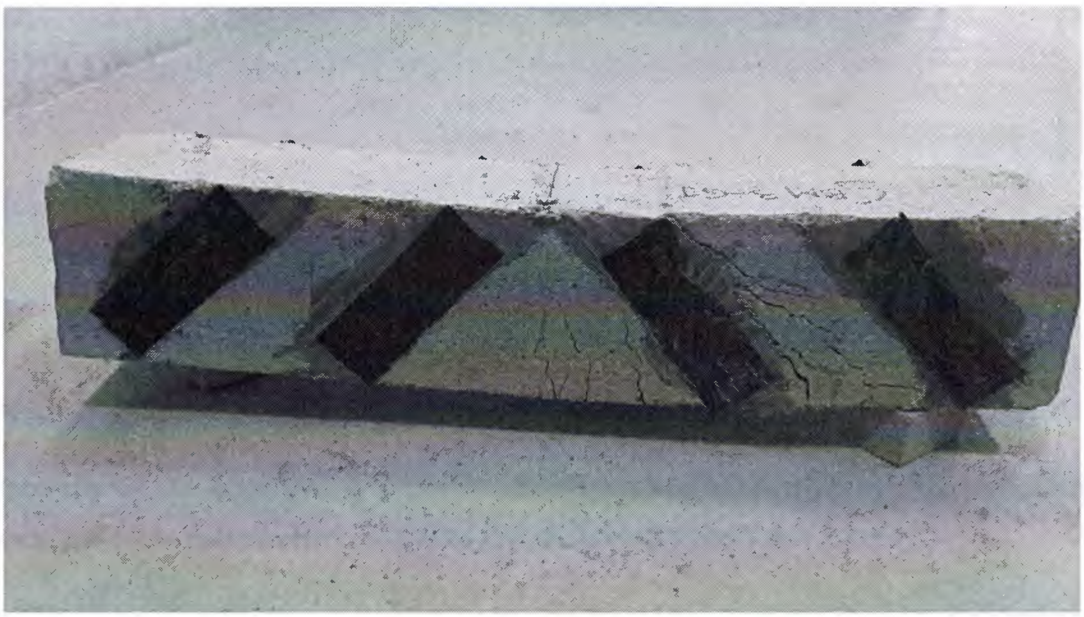

Figure 5.7: Failure in SBM3-G1

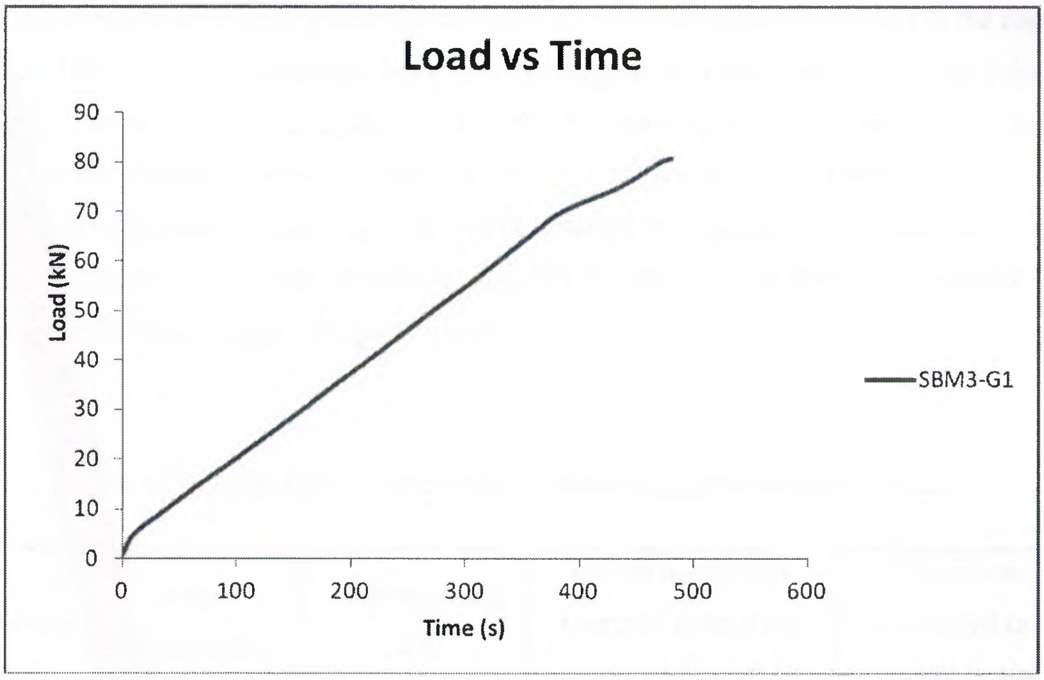

Figure 5.8: Load-Time Curve for SBM3-G1

5.3 Summary of the Experimental Observations of Group 1 Beams

Generally all the strengthened beams in group 1 showed an increase in ultimate capacity compared to the control beam as showed in both Table 5.1 and Table 5.2.

Table 5.1: Results of Group 1 Beams

Group	Beam Designation	Ultimate load (kN)
1	CBM1-G1	74.20
	SBM1-G1	84.03
	SBM2-G1	94.69
	SBM3-G1	81.85

All the strengthened beams showed an increase in ultimate capacity compared to the control beam. The U-strip strengthened beam has the highest increase and 45^0 wrapped beams lowest. It shows that wrapping beams with 45^0 strips parallel to the diagonal tension failure increase the ultimate strength less that the 90^0 this is because the CFRP did not arrest the propagated diagonal tension crack. The result obtained by Siddiqui (2009), showed that the ultimate capacity of the beam strengthen with CFRP strips inclined at 90^0 to the longitudinal axis as 17.06% compared to the control beam.

Table 5.2: Comparison of Control beam with Strengthened Beams (Group 1)

Group	Beam Designation	Ultimate load (kN)	Ultimate capacity Increase compared control beam	% increase compared to control beam
1	CBM1-G1	74.20	-	-
	SBM1-G1	84.03	9.83	13.25
	SBM2-G1	94.69	20.49	27.61
	SBM3-G1	81.85	7.65	10.31

5.4 Experimental Observations of Group 2 Beams

CBM1-G2

The control beam was designed as strong in shear and weak in flexure. It was loaded. The ultimate capacity of CBM1-G2 was 85.31kN and failed due to flexural failure as expected and crushing of concrete at the position of applied load. The flexural failure was observed at the mid-span of the beam and started as thin hair-like flexural cracks, which widened as it propagated to the top of the beam as shown in Figure 5.9. The change in the position of the neutral axis was easily and clearly observed at the mid-span due to maximum deflection that occurred at the position. The ultimate load was reached in 516.17 seconds as shown on the load-time graph in Figure 5.10

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Figure 5.9: Failure in CBM1-G2

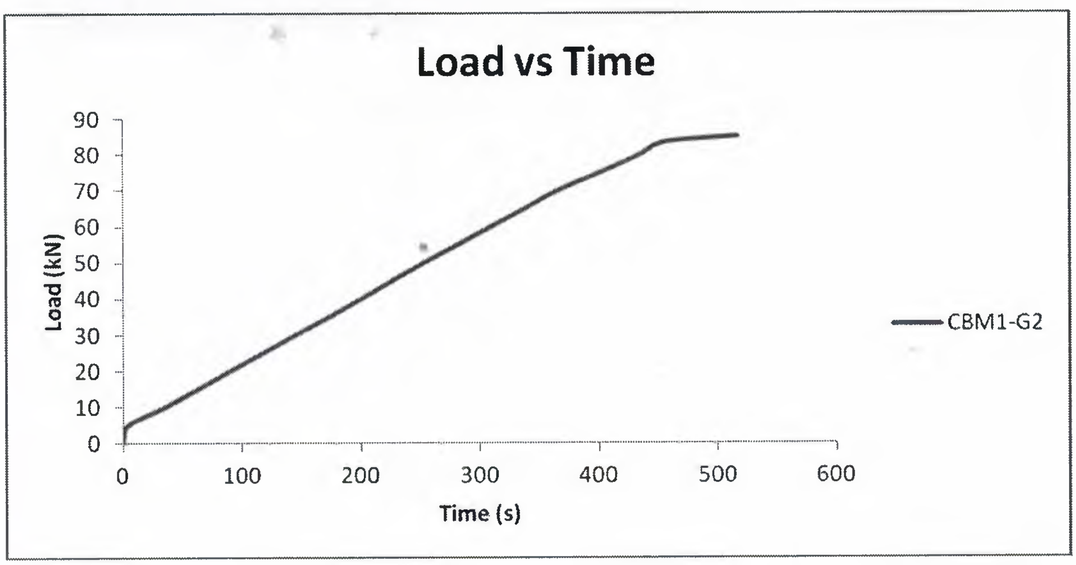

Figure 5.10: Load-Time Curve for CBM1-G2

SBM1-G2

The strengthened beam was loaded until failure. The ultimate load capacity of SBM1-G2 was 94.74kN which showed an increase in ultimate capacity compared to the control beam. The failure observed due to debonding of CFRP from the end followed by shear compression failure as shown in Figure 5.11. The ultimate load was reached in 517.83 seconds as shown on the load-time graph in Figure 5.12.

Figure 5.11: Failure in SBM1-G2

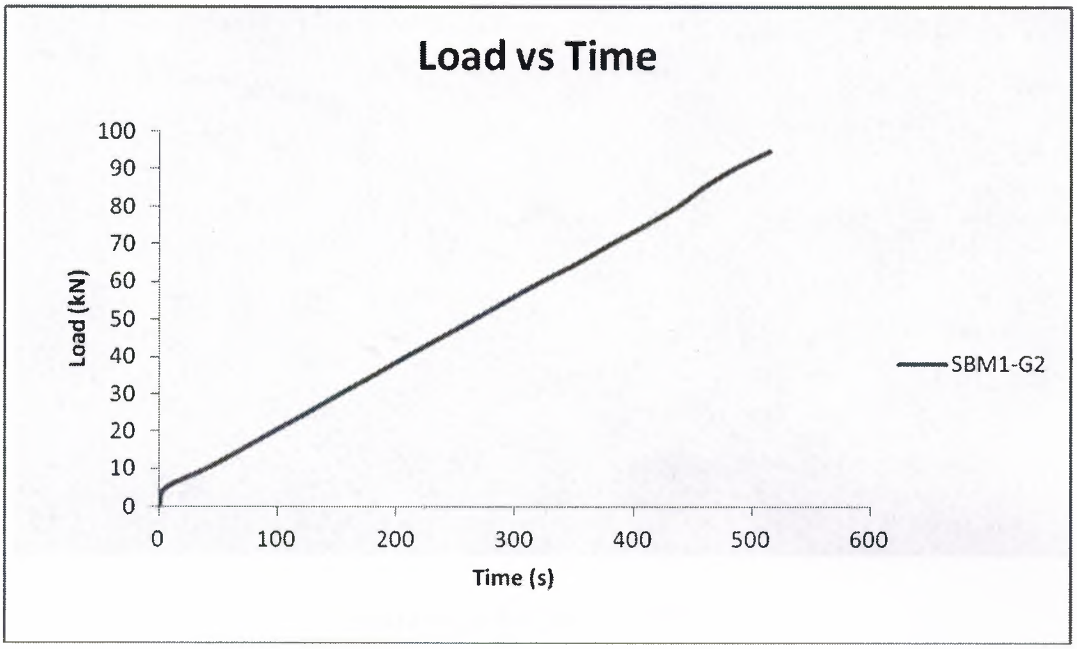

Figure 5.12: Load-Time Curve for SBM1-G2

SBM2-G2

The strengthened beam was loaded until failure. The ultimate load capacity of SBM1-G2 was 100.6kN which showed a significant increase in ultimate capacity compared to the control beam. Bursting of concrete was observed together with shear compression failure as shown in Figure 5.13. No debonding occurred, this improvement was due to U-strips attached to the end of the CFRP. The ultimate load was reached in 556.33 seconds as shown on the load-time graph in Figure 5.14.

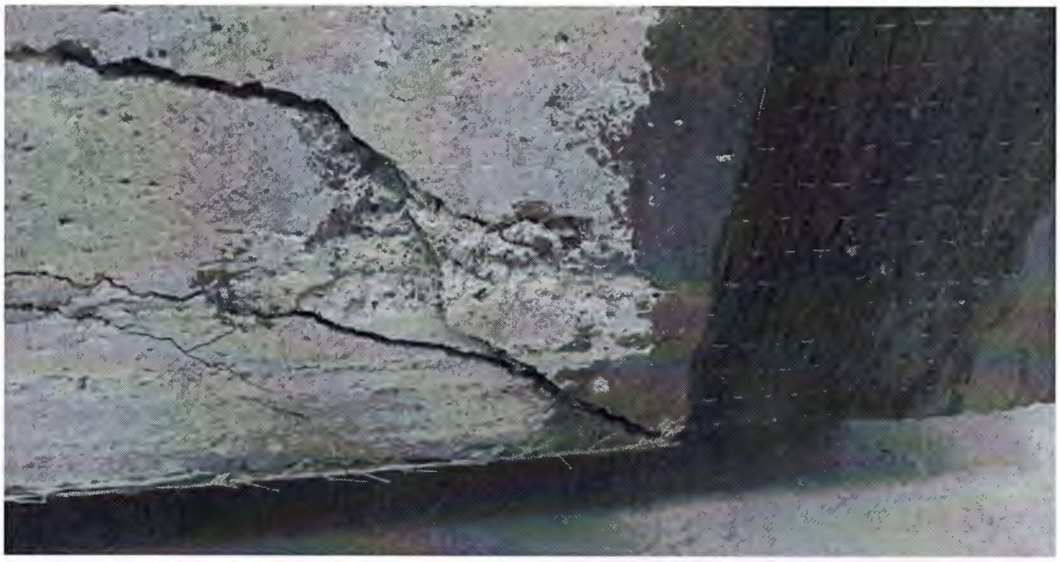

Figure 5.13: Failure in SBM2-G2

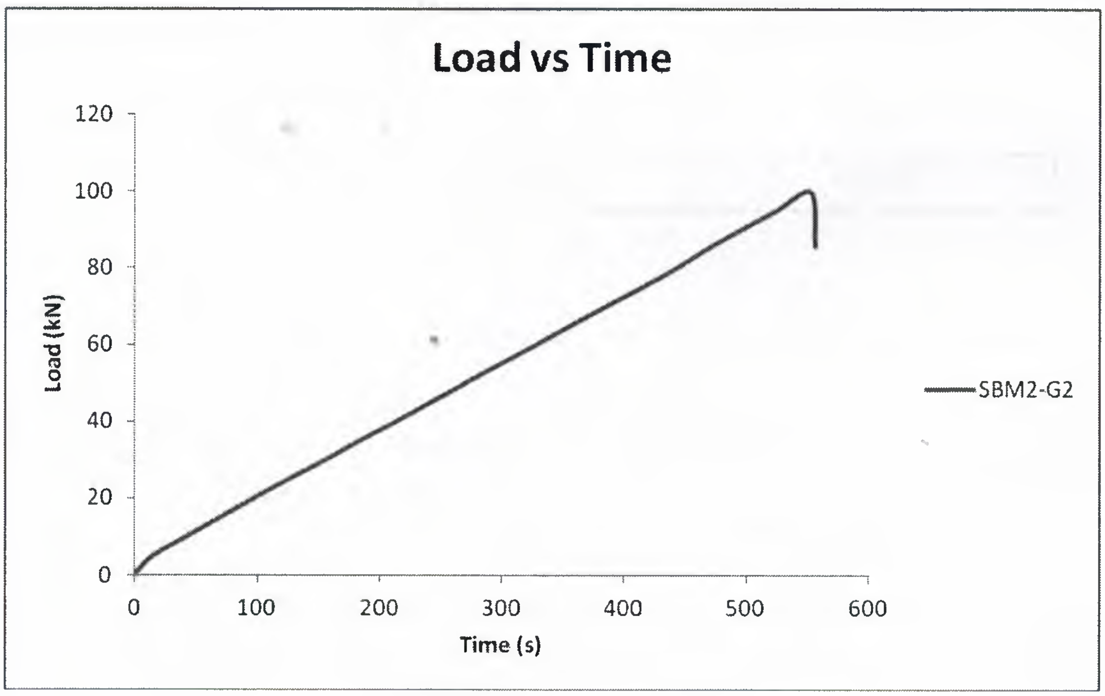

Figure 5.14: Load-Time Curve for SBM1-G2

5.5 Summary of the Experimental Observations of Group 2 Beams

Generally all the strengthened beams in group 2 showed an increase in ultimate capacity compared to the control beam as showed in both Table 5.3 and Table 5.4. The strengthened beams show an increase in ultimate strength compared to the control beam. With tension side plus U-strip end anchorage wrapping showing the highest increase. The addition of end U-strip also eliminates the effect of debonding as in the SBM1-G2.

Experiment conducted by Siddiqui (2009), showed that end anchorages increase the load carrying capacity. It also was stated that tension side bonding of CFRP sheets with U-shaped end anchorages is very effective in flexural strengthening of RC beams as this scheme not only increases the flexural capacity substantially, but also maintains sufficient deformation capacity.

Table 5.3: Results of Group 2 Beams

Group	Beam Designation	Ultimate load (kN)
2	CBM1-G2	85.31
	SBM1-G2	94.78
	SBM2-G2	100.60

Table 5.4: Comparison of Control beam with Strengthened Beams (Group 2)

Group	Beam Designation	Ultimate load (KN)	Ultimate capacity Increase compared control beam	% increase compared to control beam
2	CBM1-G2	85.31	-	
	SBM1-G2	94.78	9.47	11.11
	SBM2-G2	100.60	15.29	17.92

5.6 Energy Absorption Capacity

To ascertain the damage caused by loading a structure, engineers use energy absorption as an important parameter. Energy absorption is obtained to evaluate the strength of a structure or structural member after impact of loading.

In this research, one cylindrical specimen was wrapped using a single layer of CFRP sheet as shown in Figure 5.15 and was compared with the average compressive strength of cylindrical specimens that were not wrapped (the specimens used to obtain compressive strength). The compressive strength of the wrapped specimen was 45.9MPa. This shows an increase of 12.64% compared to the average of unwrapped specimens where the compressive strength was 40.75MPa. The failure of the wrapped specimen was observed to be sudden after the ultimate load, which was due to the rupture of CFRP from the concrete cylinder.

Increase in compressive strength of the wrapped specimen, indicates that CFRP can be used to reduce the damage of compression members.

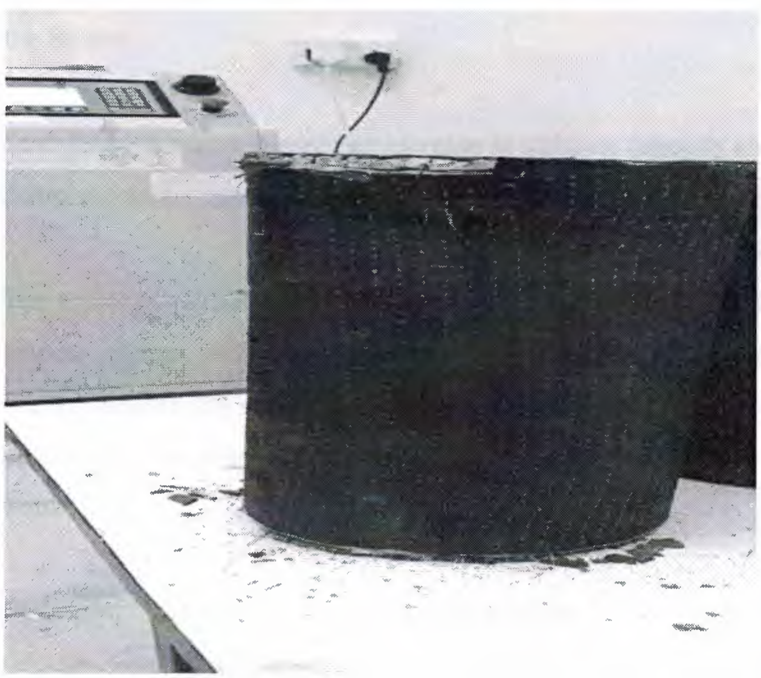

Figure 5.15: Failure of Wrapped Cylindrical Specimen

CHAPTER 6

CONCLUSIONS AND RECOMMENDATIONS

According to the research experiment conducted, the following conclusions and key findings can be drawn on the basis of the results and is presented in this chapter.

- The failure mode in group 1 control beam was due to flexural failure, while group 2 control beam failed due to diagonal tension (shear failure) as expected. Change in failure mode was observed in all the strengthened beams compared to the control beam for both groups and are more brittle.

- It was observed in all strengthened beams that the ultimate capacity of CFRP was not reached. All the failures in the strengthened beams are either due to debonding of CFRP from the surface, rupture or crushing of the concrete around the CFRP. Concentration of stress near the CFRP end was the cause of the failure.

- Strengthened beams showed an increase in ultimate capacity and stiffness compared to the control beams in each group.

- The most undesirable mode of failure is due to diagonal tension failure which caused failure without reaching the yield strength capacity of the reinforcements. By applying the CFRP U-strip scheme, the effectiveness and efficiency was observed with 27.61% increase with respect to the control beam.

- In group 1, SBM1-G1 showed a greater ultimate capacity compared to SBM3-G1, so bonding CFRP strips at 45^0 inclined to the longitudinal axis and parallel to the shear failure (diagonal tension failure) is not as effective as bonding at 90^0 to the longitudinal axis.

- In seismic regions, CFRP can be applied on compression members to increase their absorption capacity. In this experiment, the tested cylinder wrapped with 1 layer of CFRP showed an increase in compressive strength of 12.64% with respect to the control.

Generally, the recent and new effective technique of CFRP strengthening can be used in reinforced structures including bridges, high rised buildings, etc. to increase the ultimate carrying capacity and stiffness without increase in the overall weight of the system. The most effective and efficient orientation is applying CFRP as U-strips with 27.61% increase with respect to the control beam in shear strengthening. For flexural strengthening, applying the sheet in the tension face with U-strip end anchorage was the most effective.

From the research conducted, it is obvious that there is need for further investigation into the behaviour of CFRP externally bonded beams. Other parameters such as, orientation of CFRP, spacing between the CFRP strips, beam design, fully wrapped beams, shear depth ratio are to be investigated. The design guidelines provided by ACI 440.2R-08 should be used for numerical calculations and compared with the experimental results as well.

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APPENDIX I

Beam Design According to ACI 318-11

Group 1 beams

Tension reinforcement = 2Ø12mm

Compression reinforcement = 1Ø8mm   Concrete cover = 20mm

d = h- cover- Ø/2 –$Ø_{stirrup}$

d = 150- 20-6-8

d= 116mm

$$A = \frac{\pi d^2}{4} = 226.2mm^2$$

$$A_s' = \frac{\pi d^2}{4} = 50.27mm^2$$

$$\rho = \frac{As}{bd} = \frac{226.2}{150 \times 116} = 0.0129$$

$$\rho' = \frac{As'}{bd} = \frac{50.27}{150 \times 117} = 0.0027$$

$$\rho_{eff} = \rho - \rho' = 0.0129 - 0.0027 = 0.0102$$

$$\rho_{min} = \frac{1.4}{f_y} = 0.0033$$

$$\rho_{bal} = 0.85\beta_l \frac{f_c'}{f_y}\left(\frac{600}{600 + f_y}\right)$$

$$= 0.85 \times 0.85 \times \frac{30}{420}\left(\frac{600}{600 + f_y}\right)$$

$$\rho_{bal} = 0.030$$

$$\rho_{max} = 0.75\rho_b + \rho'$$

$$= 0.75(0.03) + 0.0027$$

$= 0.0252$

$$\rho_{min} < \rho_{eff} < \rho_{max}$$ Beam O.K

Group 2 beams

Tension reinforcement = 2Ø10mm

Compression reinforcement = 1Ø8mm Concrete cover = 20mm

d = h- cover- Ø/2 –$Ø_{stirrup}$

d = 150- 20-5-8

d= 117mm

$$A = \frac{\pi dia^2}{4} = 157mm^2$$

$$A_s' = \frac{\pi dia^2}{4} = 50.27mm^2$$

$$\rho = \frac{As}{bd} = \frac{157}{150 \times 117} = 0.0089$$

$$\rho' = \frac{As'}{bd} = \frac{50.27}{150 \times 117} = 0.0027$$

$$\rho_{eff} = \rho - \rho' = 0.0089 - 0.0027 = 0.0062$$

$$\rho_{min} = \frac{1.4}{f_y} = 0.0033$$

$$\rho_{bal} = 0.85\beta_{,}\frac{f_c'}{f_y}\left(\frac{600}{600 + f_y}\right)$$

$$= 0.85 \times 0.85 \times \frac{30}{420}\left(\frac{600}{600 + f_y}\right)$$

$$\rho_{bal} = 0.030$$

$$\rho_{max} = 0.75\rho_b + \rho'$$

$= 0.75(0.03) + 0.0027$

$= 0.0252$

0.0062< 0.0033 But very weak in flexure

Shear Design for Group1 and Group 2 Beams

$$V_c = \phi V_c + \phi V_s$$

$$V_u \leq \phi V_c + \frac{\phi A_v f_y d}{s}$$

$$V_c = 0.17\sqrt{f_c'} b_w d$$ ACI 318-05

$$V_c = 0.17\sqrt{30} \times 150 \times 117 = 17.65kN$$

$$\phi V_c = 0.85 \times 17.5 = 15kN$$

$$V_u = 60k\text{N}$$

$V_u > \phi V_c$ Shear reinforcement is required

$$\frac{A_v}{s} = \frac{V_u - \phi V_c}{f_y d \phi}$$

$$= \frac{(60 - 15) \times 10^3}{420 \times 117 \times 0.85}$$

$$\frac{A_v}{s} = 1.07$$

Using Ø8mm

$$A_v = 2\frac{\pi d^2}{4} = 100.53mm^2$$

$$s = \frac{100.53}{1.07}$$

$$\text{s} = 93.31mm$$

Provide spacing s = 100mm for Group 2 beams (Strong in Shear)

Provide spacing s = 100mm for Group 1 beams (Weak in Shear)

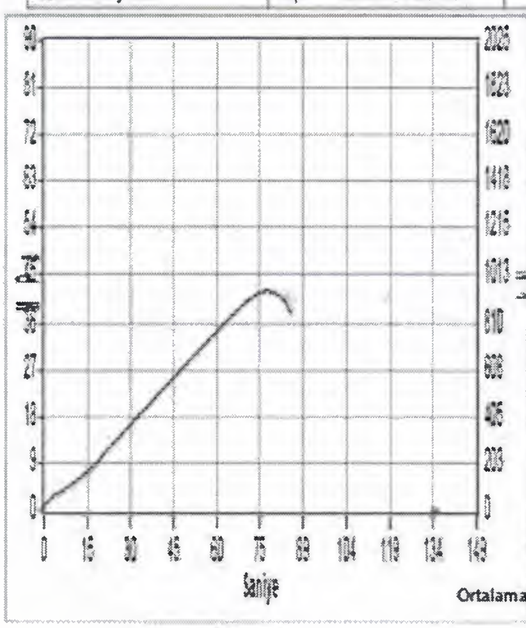
APPENDIX II

Graphical Results Obtained From Compression Test and Tensile test

YAKIN DOĞU ÜNİVERSİTESİ LABORATUVARI BETON BASINÇ DAYANIM DENEY RAPORU

Rapor Tarihi	30/05/2014			Referans No	MSC Thesis

FİRMA VE ŞANTİYE BİLGİLERİ		DENEY SONUÇLARI			
İnşaat Sahibi	Mustapha Nasır Said	Gerilme (Mpa)	Kırım Kuvveti (KN)	Ağırlık (gr)	Birim Ağırlık(Kg/m3)
Deneyi Yaptıran	Mustafa Türk	42.1	946.3	8000.0	2.370
İnşaat Adresi	NEU Mosque Construction Site	0.0	0.0	0.0	0.000
Standart	TS EN 206-1 / TS EN12390-3	0.0	0.0	0.0	0.000
		0.0	0.0	0.0	0.000
NUMUNE BİLGİLERİ		0.0	0.0	0.0	0.000
Numuneyi Alan	Mustafa Turk	0.0	0.0	0.0	0.000
Üretici Firma	Tufekci				
Döküm Tarihi	2/05/2014				
Kırım Tarihi	30/05/2014				
Alındığı Yer (Kiriş,Kolon vs)	Colomn	Numune Adedi	3		
Beton Sınıfı	C30	Yaş 1	28 days		
Numune Boyutları	Küp 150.0x150.0x150.0	Yaş 2			



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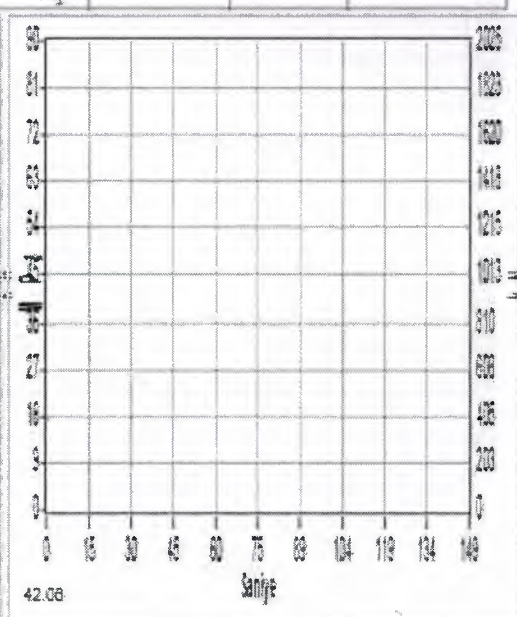


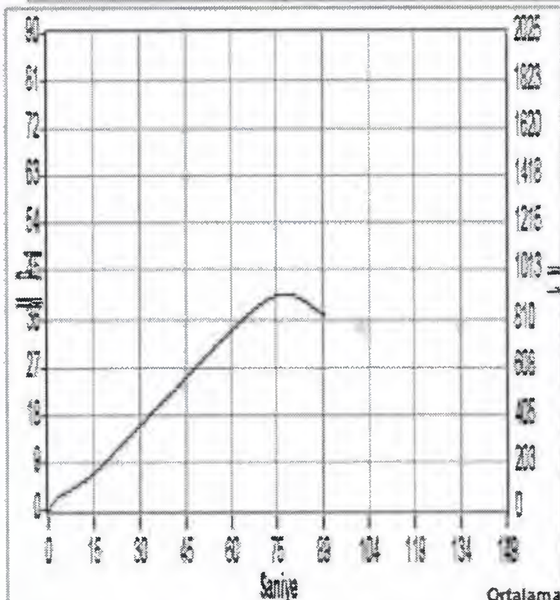
DENEYİ YAPAN

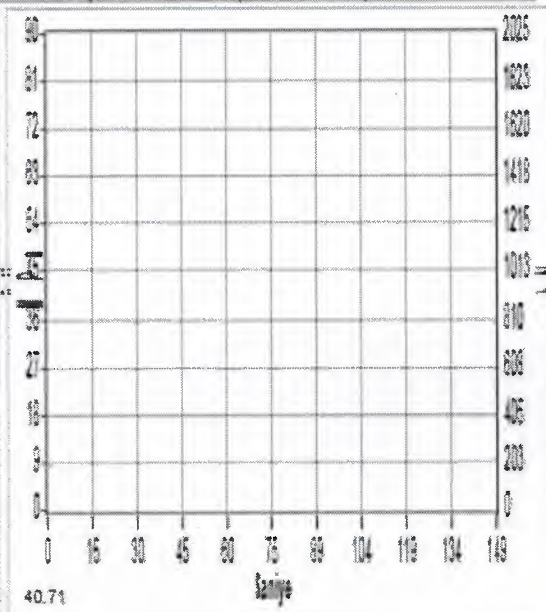
ONAYLAYAN

YAKIN DOĞU ÜNİVERSİTESİ LABORATUVARI BETON BASINÇ DAYANIM DENEY RAPORU

Rapor Tarihi	30/05/2014			Referans No	Msc Thesis

FİRMA VE ŞANTİYE BİLGİLERİ		**DENEY SONUÇLARI**			
İnşaat Sahibi	Mustapha Nasır Said	Gerilme (Mpa)	Kırım Kuvveti (KN)	Ağırlık (gr)	Birim Ağırlık(Kg/m3)
Deneyi Yaptıran	Mustafa Turk	40.7	915.9	8044.0	2.383
İnşaat Adresi	NEU Mosque Construction Site	0.0	0.0	0.0	0.000
Standart	**TS EN 206-1 / TS EN12390-3**	0.0	0.0	0.0	0.000
		0.0	0.0	0.0	0.000
NUMUNE BİLGİLERİ		0.0	0.0	0.0	0.000
Numuneyi Alan	Mustafa Turk	0.0	0.0	0.0	0.000
Üretici Firma	Tufekcı				
Döküm Tarihi	02/05/2014				
Kırım Tarihi	30/05/2014				
Alındığı Yer (Kiriş,Kolon vs)	COLOMN	Numune Adedi	3		
Beton Sınıfı	C30	Yaş 1	28days		
Numune Boyutları	Küp 150.0x150.0x150.0	Yaş 2			





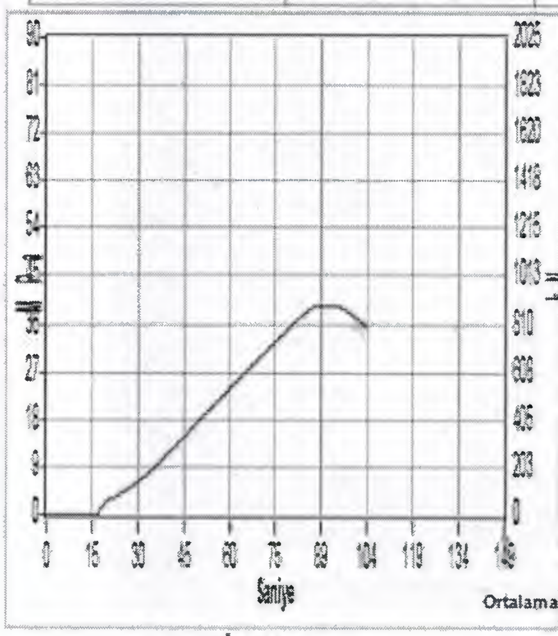
Ortalama 40.71

DENEYİ YAPAN **ONAYLAYAN**

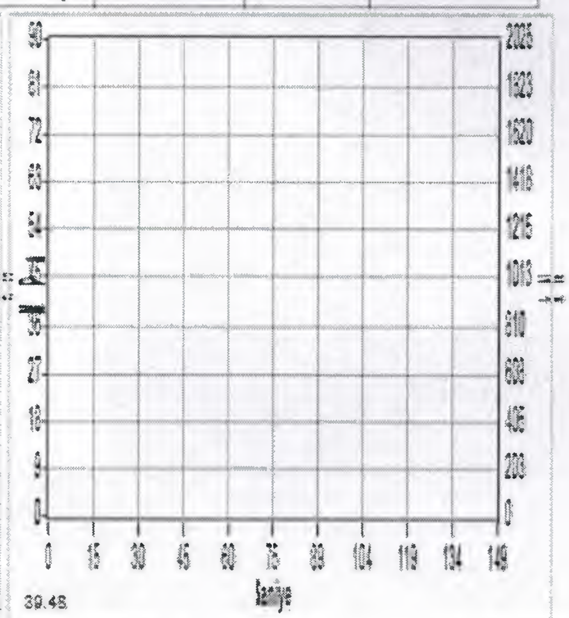
YAKIN DOĞU ÜNİVERSİTESİ LABORATUVARI BETON BASINÇ DAYANIM DENEY RAPORU

Rapor Tarihi	30/05/2014			Referans No	Msc Thesis

FİRMA VE ŞANTİYE BİLGİLERİ		**DENEY SONUÇLARI**			
İnşaat Sahibi	Mustapha Nasır Said	Gerilme (Mpa)	Kırım Kuvveti (KN)	Ağırlık (gr)	Birim Ağırlık(Kg/m3)
Deneyi Yaptıran	Mustafa Turk	39.5	888.4	7991.0	2.368
İnşaat Adresi	NEU Mosque Construction Site	0.0	0.0	0.0	0.000
Standart	**TS EN 206-1 / TS EN12390-3**	0.0	0.0	0.0	0.000
		0.0	0.0	0.0	0.000
NUMUNE BİLGİLERİ		0.0	0.0	0.0	0.000
Numuneyi Alan	Mustafa Turk	0.0	0.0	0.0	0.000
Üretici Firma	Tufekci				
Döküm Tarihi	02/05/2014				
Kırım Tarihi	30/05/2014				
Alındığı Yer (Kiriş,Kolon vs)	COLOMN	Numune Adedi	3		
Beton Sınıfı	C30	Yaş 1	28 days		
Numune Boyutları	Küp 150.0x150.0x150.0	Yaş 2			



Ortalama 39.48



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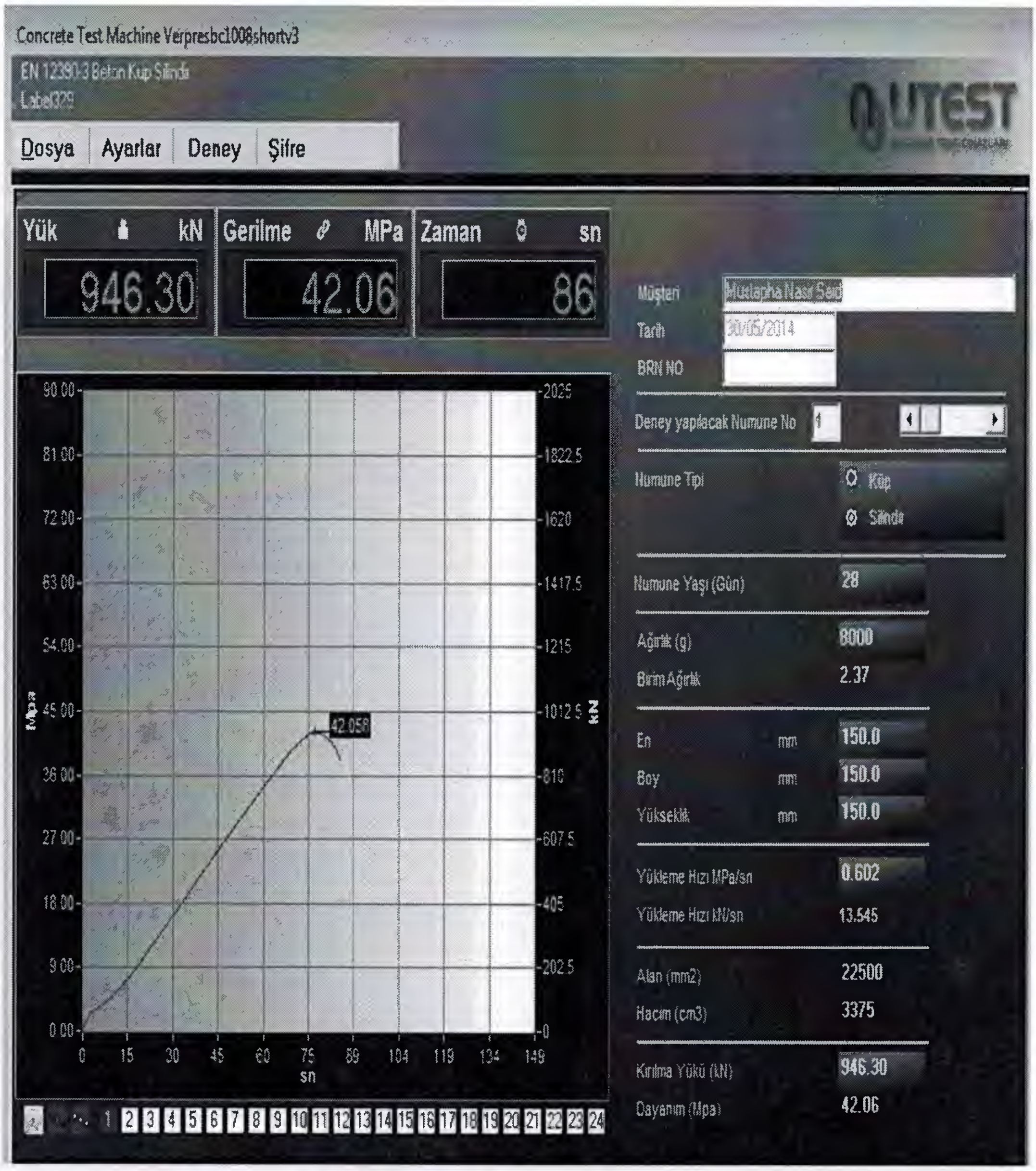
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Ayarlar
Deney
Şifre
Yük kN
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Gerilme MPa
42.06
Zaman sn
86
Müşteri
Mustapha Nasir Said
Tarih
30/05/2014
BRN NO
Deney yapılacak Numune No
1
Numune Tipi
Küp
Silindir
Numune Yaşı (Gün)
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Ağırlık (g)
8000
Birim Ağırlık
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En mm
150.0
Boy mm
150.0
Yükseklik mm
150.0
Yükleme Hızı MPa/sn
0.602
Yükleme Hızı kN/sn
13.545
Alan (mm2)
22500
Hacım (cm3)
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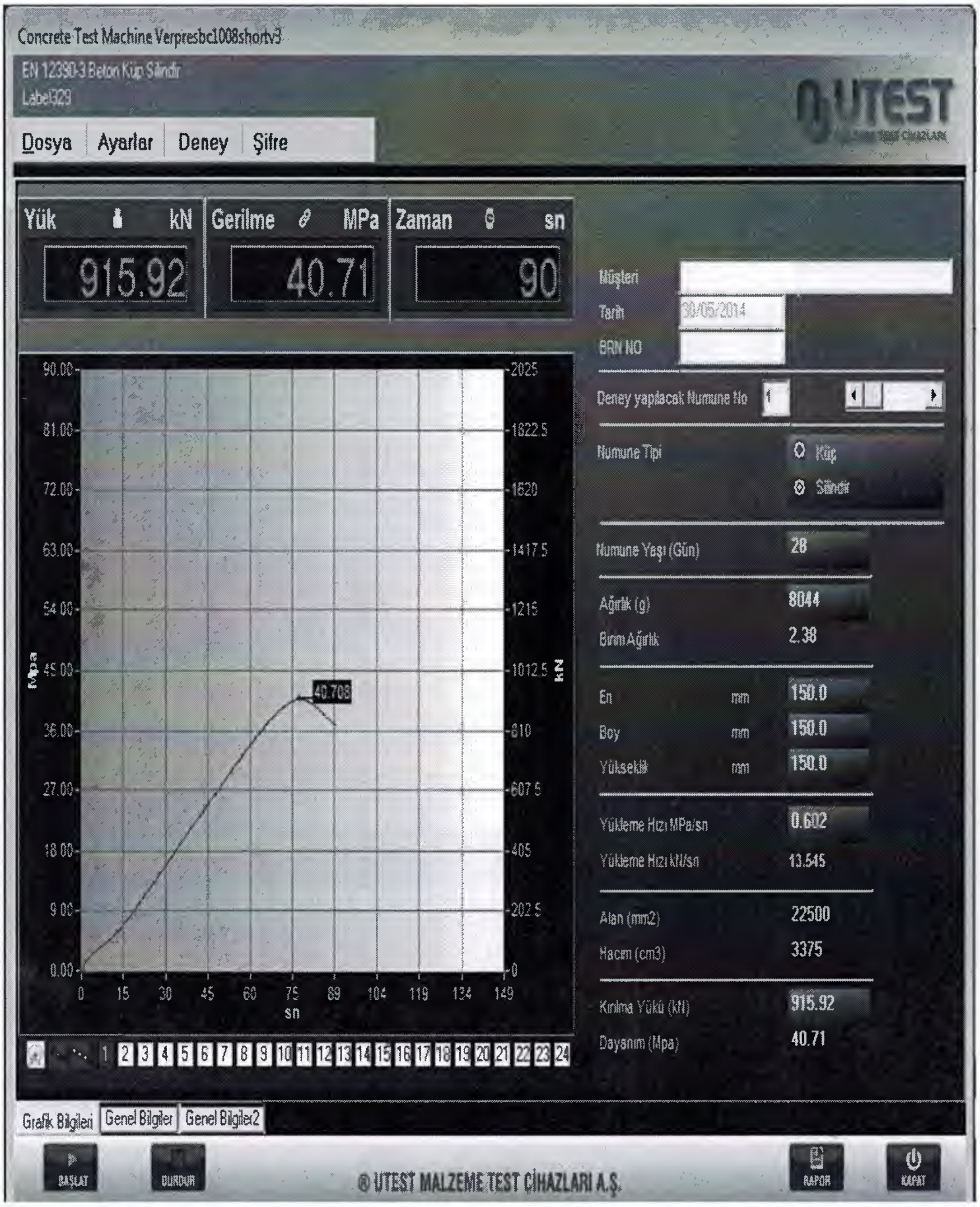
Kırılma Yükü (kN)
946.30
Dayanım (Mpa)
42.06
42.058
sn
kN
Mpa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Dosya
Ayarlar
Deney
Şifre
Yük kN
915.92
Gerilme MPa
40.71
Zaman sn
90
Müşteri
Tarih
30/05/2014
BRN NO
Deney yapılacak Numune No
Numune Tipi
Küp
Silindir
Numune Yaşı (Gün)
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Ağırlık (g)
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Birim Ağırlık
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Boy mm
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Yükseklik mm
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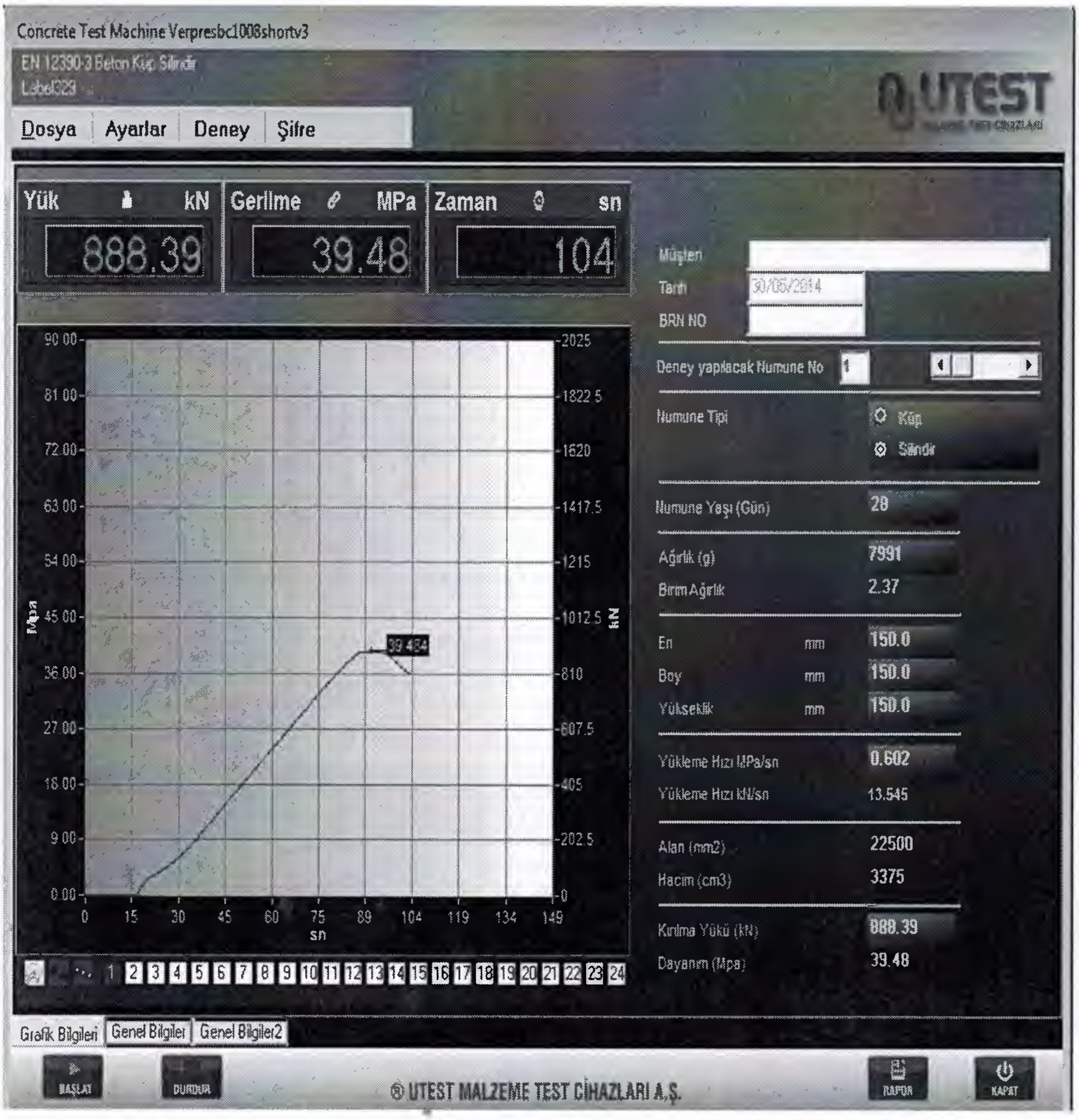
Yükleme Hızı MPa/sn
0.602
Yükleme Hızı kN/sn
13.545
Alan (mm2)
22500
Hacim (cm3)
3375
Kırılma Yükü (kN)
915.92
Dayanım (Mpa)
40.71
40.708
Mpa
kN
sn
Grafik Bilgileri
Genel Bilgiler
Genel Bilgiler2
BAŞLAT
DURDUR
® UTEST MALZEME TEST CİHAZLARI A.Ş.
RAPOR
KAPAT



Concrete Test Machine Verpresbc1008shortv3
EN 12390-3 Beton Küp Silindir
Dosya
Ayarlar
Deney
Şifre
UTEST
Yük kN
888.39
Gerilme MPa
39.48
Zaman sn
104
Müşteri
Tarih
BRN NO
Deney yapılacak Numune No
1
Numune Tipi
Küp
Silindir
Numune Yaşı (Gün)
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Ağırlık (g)
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Birim Ağırlık
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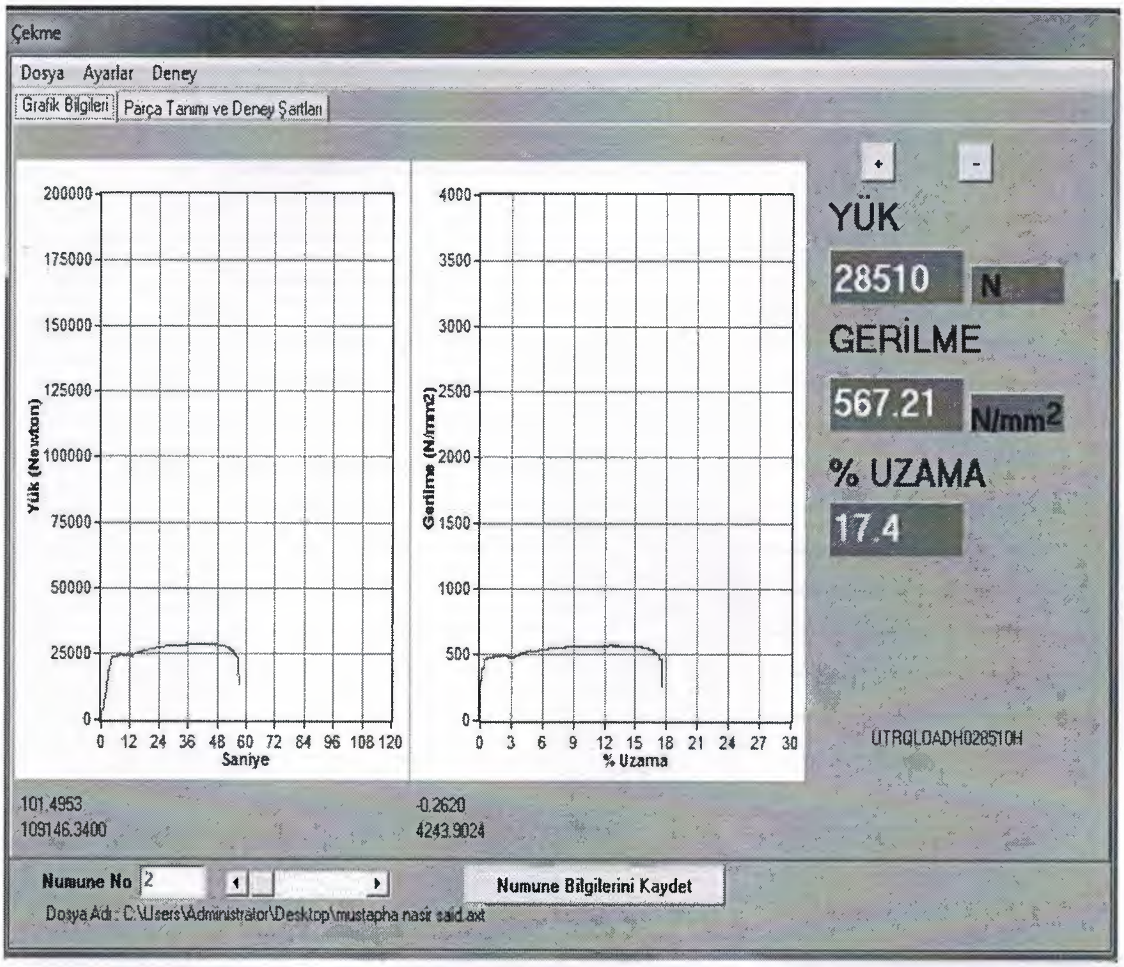
En mm
150.0
Boy mm
150.0
Yükseklik mm
150.0
Yükleme Hızı MPa/sn
0.602
Yükleme Hızı kN/sn
13.545
Alan (mm2)
22500
Hacim (cm3)
3375
Kırılma Yükü (kN)
888.39
Dayanım (Mpa)
39.48
Mpa
kN
sn
39.484
Grafik Bilgileri
Genel Bilgiler
Genel Bilgiler2
BAŞLAT
DURDUR
® UTEST MALZEME TEST CİHAZLARI A.Ş.
RAPOR
KAPAT



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Grafik Bilgileri
Parça Tanımı ve Deney Şartları
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N
GERİLME
567.21
N/mm2
% UZAMA
17.4
Yük (Newton)
Saniye
Gerilme (N/mm2)
% Uzama
UTRGLOADH028510H
101.4953
109146.3400
-0.2620
4243.9024
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Numune Bilgilerini Kaydet
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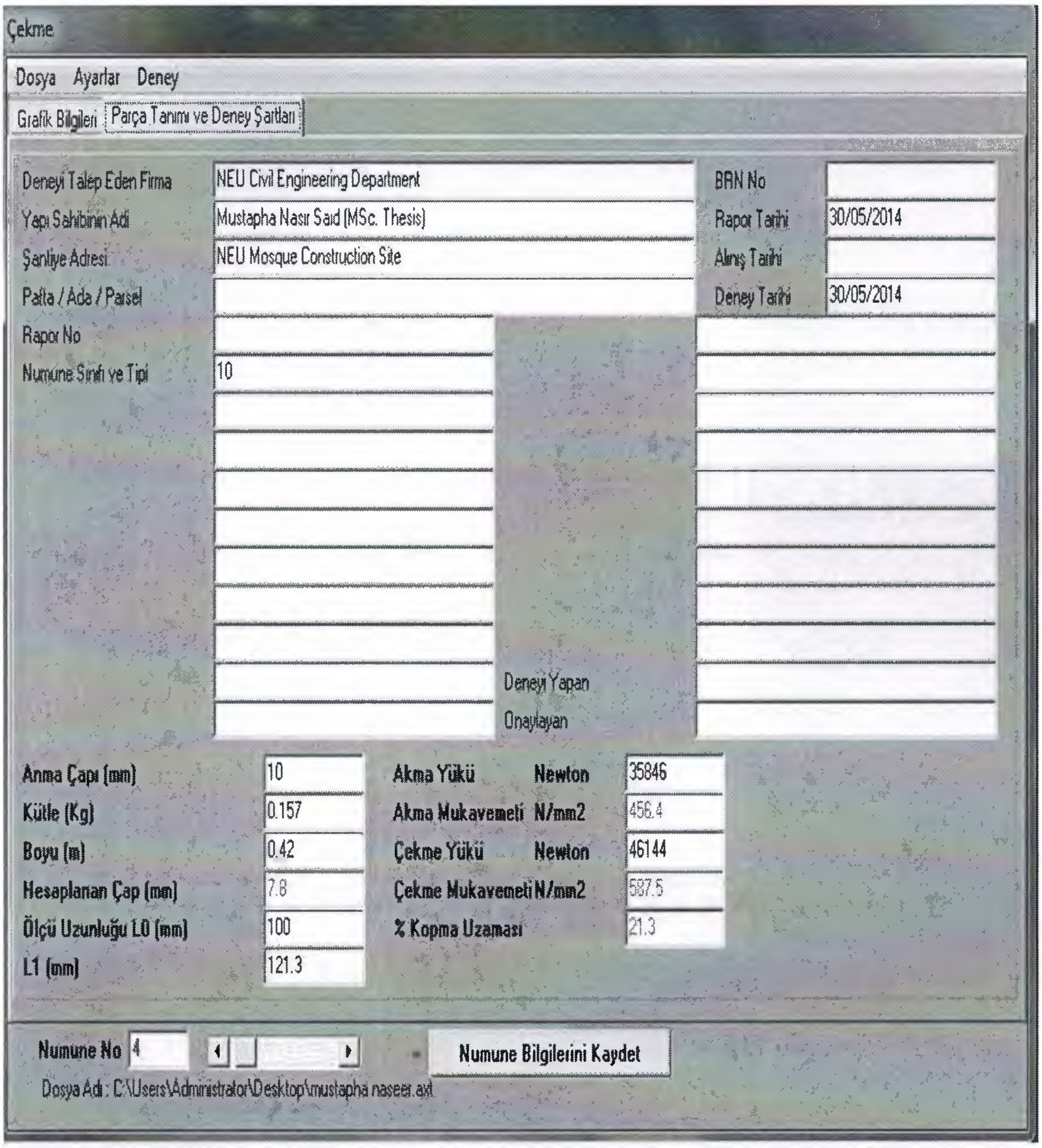
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Dosya Ayarlar Deney
Grafik Bilgileri
Parça Tanımı ve Deney Şartları
Deneyi Talep Eden Firma
NEU Civil Engineering Department
BRN No
Yapı Sahibinin Adı
Mustapha Nasır Saıd (MSc. Thesis)
Rapor Tarihi
30/05/2014
Şantiye Adresi
NEU Mosque Construction Site
Alınış Tarihi
Pafta / Ada / Parsel
Deney Tarihi
30/05/2014
Rapor No
Numune Sınıfı ve Tipi
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Deneyi Yapan
Onaylayan
Anma Çapı (mm)
10
Kütle (Kg)
0.157
Boyu (m)
0.42
Hesaplanan Çap (mm)
7.8
Ölçü Uzunluğu L0 (mm)
100
L1 (mm)
121.3
Akma Yükü Newton
35846
Akma Mukavemeti N/mm2
456.4
Çekme Yükü Newton
46144
Çekme Mukavemeti N/mm2
587.5
% Kopma Uzaması
21.3
Numune No
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Numune Bilgilerini Kaydet
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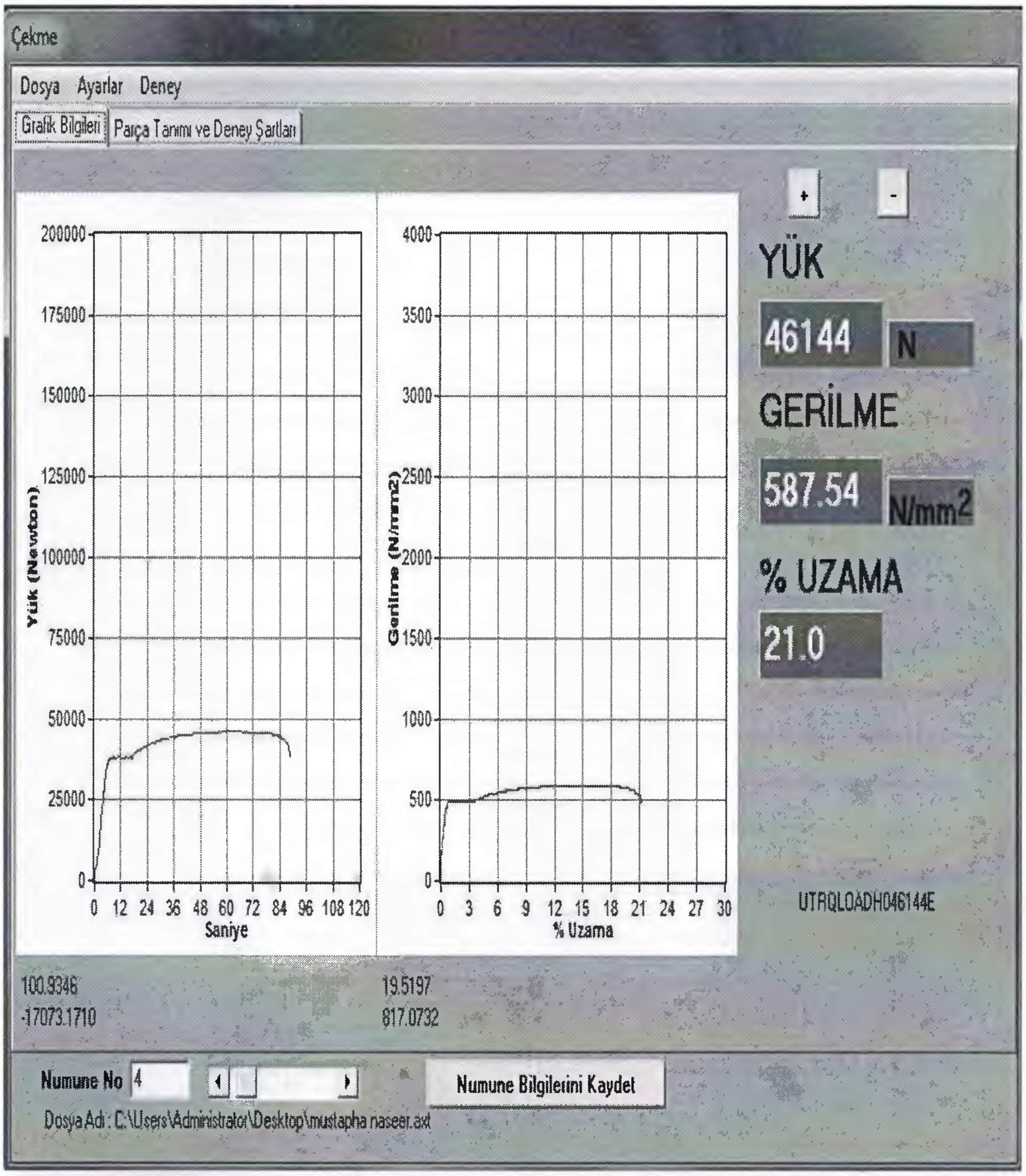
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Dosya Ayarlar Deney
Grafik Bilgileri
Parça Tanımı ve Deney Şartları
Yük (Newton)
Saniye
Gerilme (N/mm2)
% Uzama
YÜK
46144
N
GERİLME
587.54
N/mm2
% UZAMA
21.0
UTRQLOADH046144E
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-17073.1710
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4
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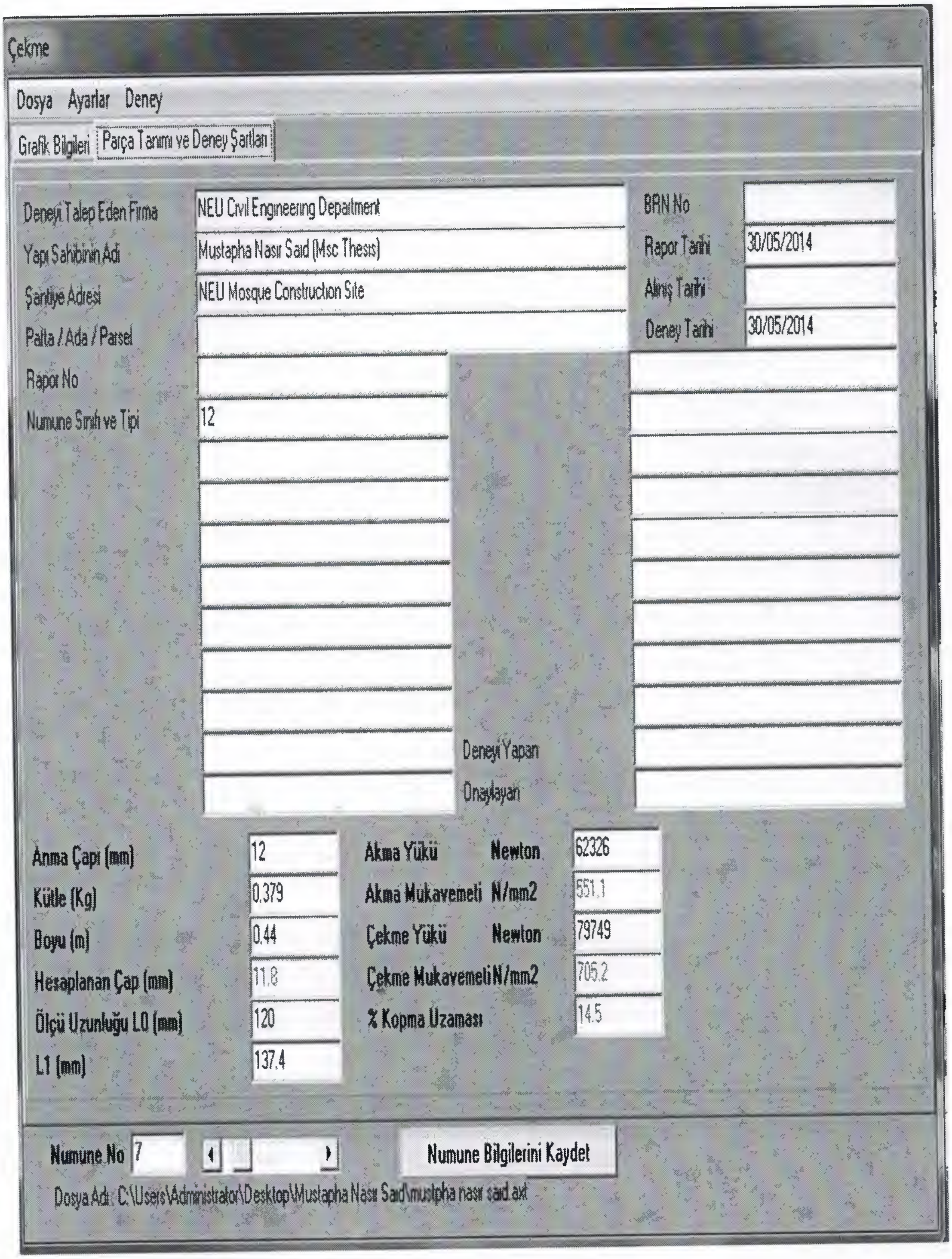
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Grafik Bilgileri
Parça Tanımı ve Deney Şartları
Deneyi Talep Eden Firma
NEU Civil Engineering Department
Yapı Sahibinin Adı
Mustapha Nasır Said (Msc Thesis)
Şantiye Adresi
NEU Mosque Construction Site
Palta / Ada / Parsel
Rapor No
Numune Sınıf ve Tipi
12
BRN No
Rapor Tarihi
30/05/2014
Alınış Tarihi
Deney Tarihi
30/05/2014
Deneyi Yapan
Onaylayan
Anma Çapı (mm)
12
Kütle (Kg)
0.379
Boyu (m)
0.44
Hesaplanan Çap (mm)
11.8
Ölçü Uzunluğu L0 (mm)
120
L1 (mm)
137.4
Akma Yükü Newton
62326
Akma Mukavemeti N/mm2
551.1
Çekme Yükü Newton
79749
Çekme Mukavemeti N/mm2
705.2
% Kopma Uzaması
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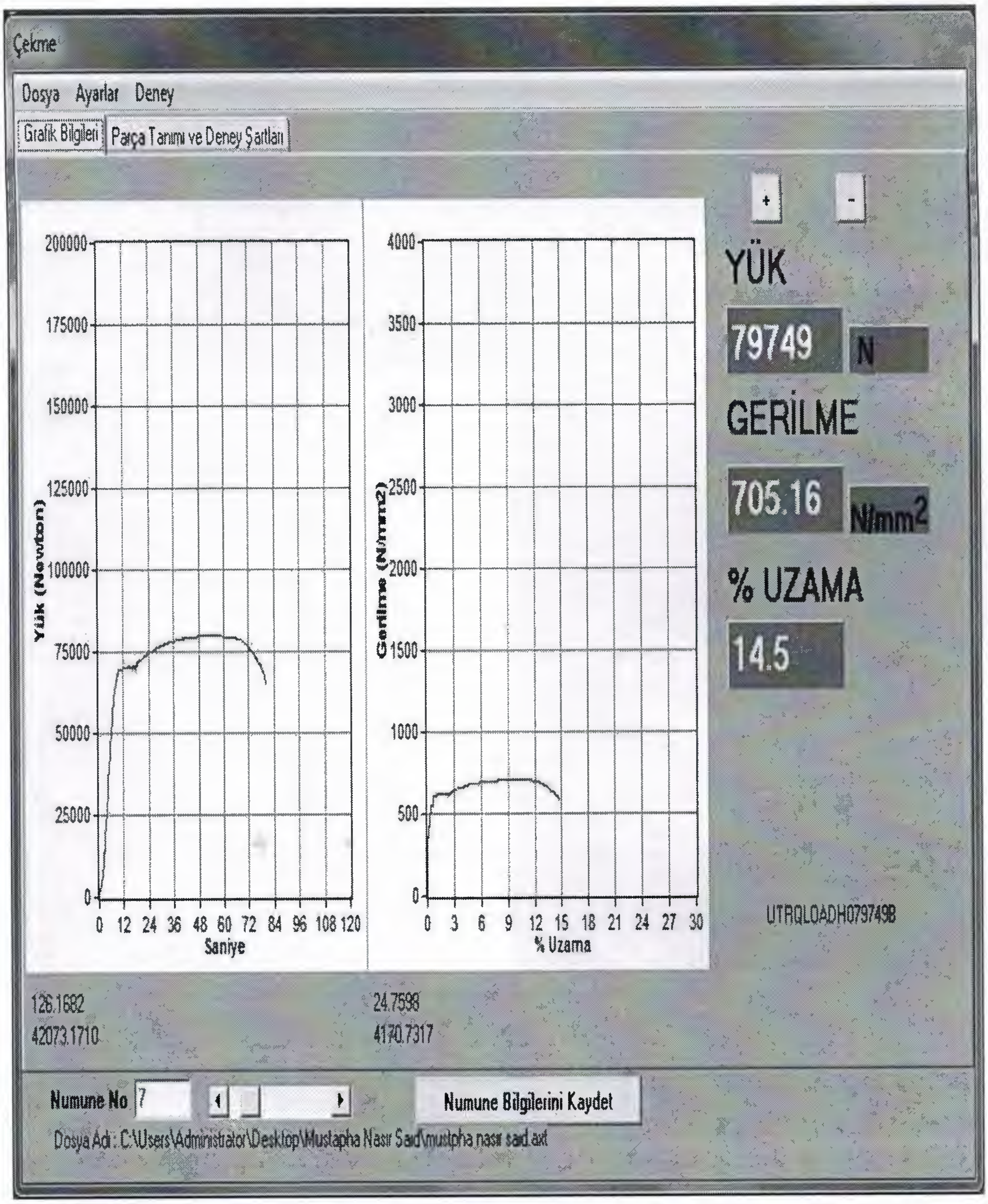
Numune No
7
Numune Bilgilerini Kaydet
Dosya Adı: C:\Users\Administrator\Desktop\Mustapha Nasır Said\mustpha nasır said.axt


Çekme
Dosya Ayarlar Deney
Grafik Bilgileri
Parça Tanımı ve Deney Şartları
Yük (Newton)
Saniye
Gerilme (N/mm2)
% Uzama
YÜK
79749
N
GERİLME
705.16
N/mm2
% UZAMA
14.5
UTRQLOADH079749B
126.1682
42073.1710
24.7598
4170.7317
Numune No
7
Numune Bilgilerini Kaydet
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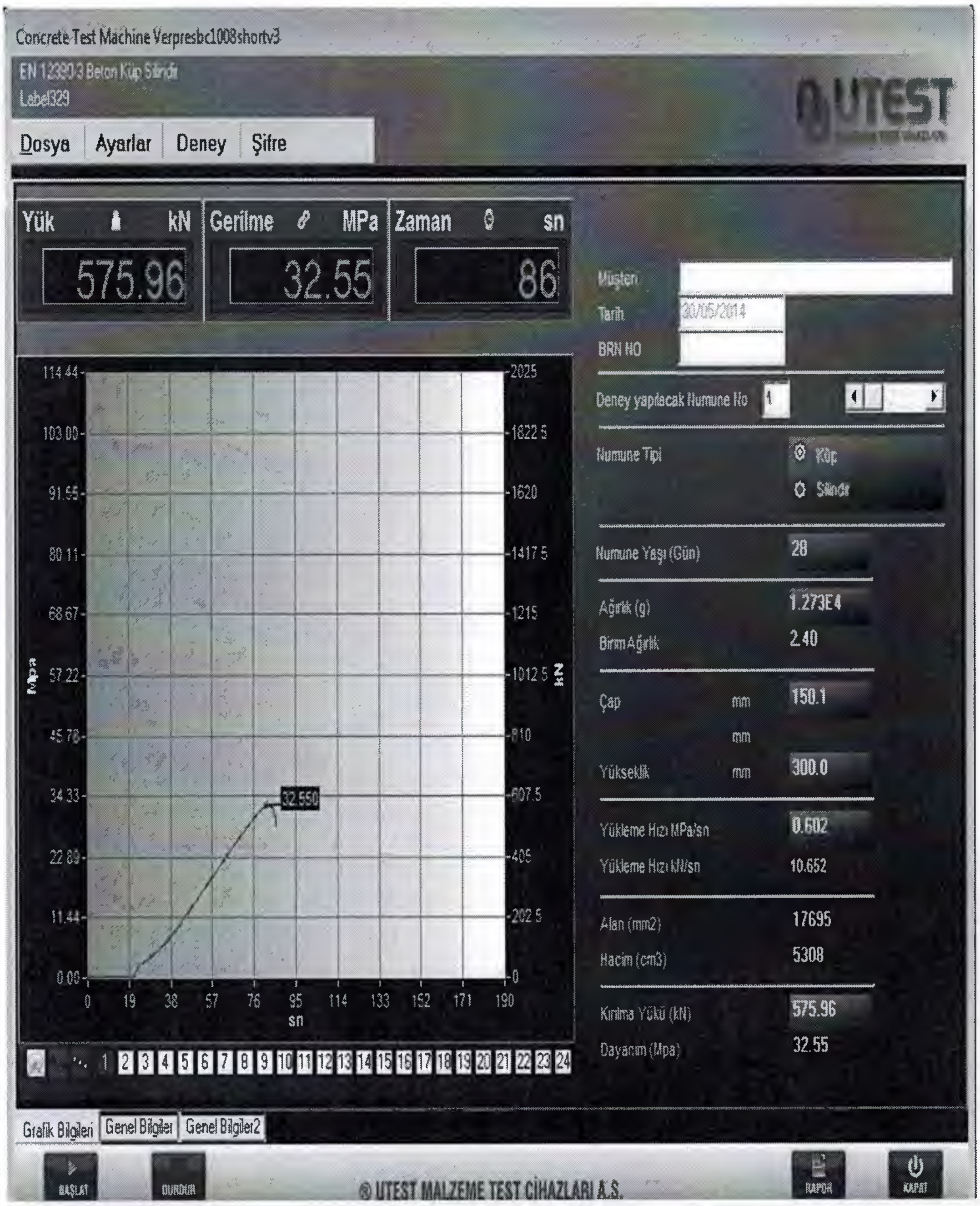
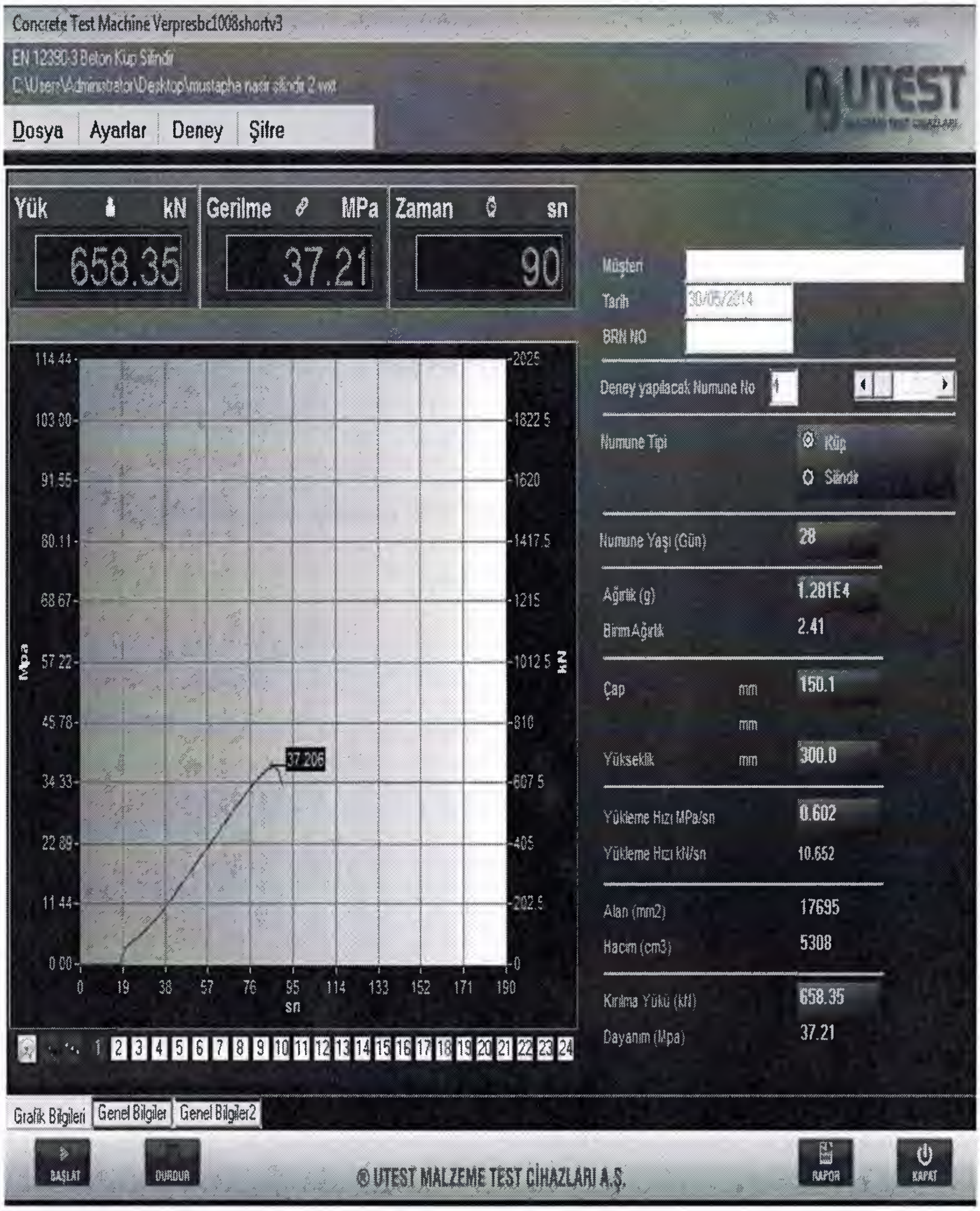

Concrete Test Machine Verpresbc1008shortv3
EN 12390-3 Beton Küp Silindir
Label329
Dosya
Ayarlar
Deney
Şifre
Yük kN
575.96
Gerilme MPa
32.55
Zaman sn
86
Müşteri
Tarih 30/05/2014
BRN NO
Deney yapılacak Numune No 1
Numune Tipi
Küp
Silindir
Numune Yaşı (Gün) 28
Ağırlık (g) 1.273E4
Birim Ağırlık 2.40
Çap mm 150.1
Yükseklik mm 300.0
Yükleme Hızı MPa/sn 0.602
Yükleme Hızı kN/sn 10.652
Alan (mm2) 17695
Hacim (cm3) 5308
Kırılma Yükü (kN) 575.96
Dayanım (Mpa) 32.55
32.550
sn
Grafik Bilgileri
Genel Bilgiler
Genel Bilgiler2
BAŞLAT
DURDUR
® UTEST MALZEME TEST CİHAZLARI A.Ş.
RAPOR
KAPAT


Concrete Test Machine Verpresbc1008shortv3
EN 12390-3 Beton Küp Silindir
UTEST
Dosya
Ayarlar
Deney
Şifre
Yük kN
658.35
Gerilme MPa
37.21
Zaman sn
90
Müşteri
Tarih
30/05/2014
BRN NO
Deney yapılacak Numune No
Numune Tipi
Küp
Silindir
Numune Yaşı (Gün)
28
Ağırlık (g)
1.281E4
Birim Ağırlık
2.41
Çap
mm
150.1
mm
Yükseklik
mm
300.0
Yükleme Hızı MPa/sn
0.602
Yükleme Hızı kN/sn
10.652
Alan (mm2)
17695
Hacim (cm3)
5308
Kırılma Yükü (kN)
658.35
Dayanım (Mpa)
37.21
37.206
Mpa
kN
sn
Grafik Bilgileri
Genel Bilgiler
Genel Bilgiler2
BAŞLAT
DURDUR
® UTEST MALZEME TEST CİHAZLARI A.Ş.
RAPOR
KAPAT

APPENDIX III

Data Sheet for Sikadur -330 Epoxy Resin and SikaWrap-300c

Product Data Sheet
Edition 13/6/2006
Identification no:
02 04 01 04 001 0 000004
Sikadur® -330

Sikadur®-330

2-Part Epoxy Impregnation Resin

Product Description	**Sikadur®-330** is a two part, solvent free, thixotropic epoxy based impregnating resin / adhesive.
Uses	■ Impregnation resin for SikaWrap® fabric reinforcement for the dry application method ■ Primer resin for the wet application system ■ Structural adhesive for bonding Sika® CarboDur® plates to even surfaces
Characteristics / Advantages	■ Easy mix and application by trowel and impregnation roller ■ Manufactured for manual saturation methods ■ Excellent application behaviour to vertical and overhead surfaces ■ Good adhesion to many substrates ■ High mechanical properties ■ No separate primer required ■ Solvent free
Tests	
Approval / Standards	Conforms to the requirements of: ■ SOCOTEC (France): Cahier des charges Sika® CarboDur, SikaWrap®. ■ Road and Bridges Research Institute (Poland): IBDiM No AT/2003-04-336.

Product Data

Form	Very soft brushable paste
Appearance / Colours	Resin part. A: paste Hardener part B: paste Colour: Part A: white Part B: grey Part A+B mixed: light grey
Packaging	Standard: 5 kg (A+B) pre-dosed units Industrial: Part A: 24 kg pails Part B: 6 kg pails

Storage

Storage Conditions / Shelf life	24 months from date of production if stored properly in original unopened, sealed and undamaged packaging in dry conditions at temperatures between +5°C and +25°C. Protect from direct sunlight.

Technical Data

Chemical Base
Epoxy resin.

Density
Mixed Resin: 1.31 kg/lt (at +23°C)

Viscosity
Shear rate: 50 /s

Temperature	Viscosity
+10°C	~ 10'000 mPas
+23°C	~ 6'000 mPas
+35°C	~ 5'000 mPas

Thermal Expansion Coefficient
45 x 10^{-6} per °C (-10°C to +40°C)

Thermal Stability
Heat Distortion Temperature (HDT) (ASTM D648)

Curing	Temperature	HDT
7 days	+10°C	+36°C
7 days	+23°C	+47°C
7 days	+35°C	+53°C
7 days, +10°C plus 7 days, +23°C	-	+43°C

Service Temperature
-40°C to +50°C

Mechanical / Physical Properties

Tensile Strength	30 N/mm² (7 days at +23°C)	(DIN 53455)
Bond Strength	Concrete fracture on sandblasted substrate: > 1 day	(EN 24624)
E-Modulus	Flexural: 3800 N/mm² (7 days at +23°C) Tensile: 4500 N/mm² (7 days at +23°C)	 (DIN 53452) (DIN 53455)
Elongation at Break	0.9% (7 days at +23°C)	(DIN 53455)

Resistance

Chemical Resistance
The product is not suitable for chemical exposure.

Thermal Resistance
Continuous exposure +50°C.

System Information

System Structure
Substrate primer - **Sikadur®-330**.
Impregnating / laminating resin - **Sikadur®-330**.
Structural strengthening fabric - SikaWrap® type to suit requirements.

Application Details

Consumption	This will be dependant on the roughness of the substrate and the type of SikaWrap® fabric to be impregnated. See respective SikaWrap® fabric Product Data Sheet. Guide: 0.7 - 1.5 kg/m²
Substrate Quality	The substrate must be sound and of sufficient tensile strength to provide a minimum pull off strength of 1.0 N/mm² or as per the requirements of the design specification. The surface must be dry and free of all contaminants such as oil, grease, coatings and surface treatments etc. The surface to be bonded must be level (max. deviation 2 mm per 0.3 m length), with steps and formwork marks not greater than 0.5 mm. High spots can be removed by abrasive blasting or grinding. Wrapped corners must be rounded to a minimum radius of 20 mm (depending on the SikaWrap® fabric type) or as per the design specification. This can be achieved by grinding edges or by building up with Sikadur® mortars.
Substrate Preparation	Concrete and masonry substrates must be prepared mechanically using abrasive blast cleaning or grinding equipment, to remove cement laitance, loose and friable material to achieve a profiled open textured surface. Timber substrates must be planed or sanded. All dust, loose and friable material must be completely removed from all surfaces before application of the **Sikadur®-330** preferably by brush and industrial vacuum cleaner. Weak concrete/masonry must be removed and surface defects such as honeycombed areas, blowholes and voids must be fully exposed. Repairs to substrate, filling of blowholes/voids and surface levelling must be carried out using Sikadur®-41 LP or a mixture of Sikadur®-30 LP and Sikadur®-501 quartz sand (mix ratio 1 : 1 max parts by weight). Bond tests must be carried out to ensure substrate preparation is adequate. Inject cracks wider than 0.25 mm with Sikadur®-52 LP or other suitable Sikadur® injection resin.

Application Conditions / Limitations

Substrate Temperature	+10°C min. / +35°C max.
Ambient Temperature	+10°C min. / +35°C max.
Substrate Humidity	≤ 4% moisture content. Test method: Sika-Tramex meter.
Dew Point	Beware of condensation! Ambient temperature during application must be at least 3°C above dew point.

Application Instructions

Mixing	Part A : part B = 4 : 1 by weight When using bulk material the exact mixing ratio must be safeguarded by accurately weighing and dosing each component.

Mixing Time

Pre-batched units: Mix parts A+B together for at least 3 minutes with a mixing spindle attached to a slow speed electric drill (max. 600 rpm) until the material becomes smooth in consistency and a uniform grey colour. Avoid aeration while mixing. Then, pour the whole mix into a clean container and stir again for approx. 1 more minute at low speed to keep air entrapment at a minimum. Mix only that quantity which can be used within its potlife.
Bulk packing, not pre-batched:First, stir each part thoroughly. Addthepartsinthecorrectproportionsintoasuitablemixingpail and stir correctly using an electric low speed mixer as above for pre-batched units.

Application Method / Tools

Preparation:
Prior to application confirm substrate moisture content, relative humidity and dew point.
Cut the specified SikaWrap® fabric to the desired dimensions.

Resin Application:
Apply the **Sikadur®-330** to the prepared substrate using a trowel, roller or brush.
Fabric Placement and Laminating:
Place the SikaWrap® fabric in the required direction onto the **Sikadur®-330**. Carefully work the fabric into the resin with the Sika plastic impregnation roller parallel to the fiber direction until the resin is squeezed out between and through the fiber strands and distributed evenly over the whole fabric surface. Avoid excessive force when laminating to prevent folding or creasing of the SikaWrap® fabric.
Additional Fabric Layers:
For additional layers of SikaWrap® fabric, apply **Sikadur®-330** to previous applied layer wet on wet within 60 minutes (at +23°C) after application of the previous layer and repeat laminating procedure.
If it is not possible to apply within 60 minutes, a waiting time of at least 12 hours must be observed before application of next layer.
Overlays:
If a cementitious overlay is to be applied over SikaWrap® fabric an additional **Sikadur-330** resin layer must be applied over final layer at a max. 0.5 kg/m². Broadcast with quartz sand while wet which will serve as a key for the overlay.
If a coloured coating is to be applied the wet **Sikadur®-330** surface can be smoothed with a brush.

Overlaps
Fiber Direction:

- Overlapping of the SikaWrap® fabric must be at least 100 mm (depending on the SikaWrap® fabric type) or as specified in the strengthening design.

Side by Side:

- Unidirectional fabrics: when placing several unidirectional SikaWrap® fabrics side by side no overlapping is required unless specified in the strengthening design.
- Multi-directional fabrics: overlapping in the weft direction must be at least 100 mm (depending on the SikaWrap fabric type) or as specified in the strengthening design.

Cleaning of Tools

Clean all equipment immediately with Sika® Colma Cleaner. Cured material can only be mechanically removed.

Potlife

Potlife:

Temperature	Time
+10°C	90 minutes (5 kg)
+35°C	30 minutes (5 kg)

Potlife starts with the mixing of both parts (resin and hardener). At low ambient temperature pot life will be extended, at elevated temperatures this will be reduced. The higher the quantity of material mixed, the shorter the potlife. To achieve a longer potlife at high temperatures the mixed material may be divided into smaller units or both parts may be cooled before mixing.
Open time:

Temperature	Time
+10°C	60 minutes
+35°C	30 minutes

Waiting Time / Overcoatability

To (pre-) cured resin:

Products	Substrate Temperature	Minimum	Maximum
Sikadur®-330 onto **Sikadur®-330**	+10°C	24 hours	Cured resin older than 7 days has to be degreased with Sika® Colma Cleaner and gently grinded with sandpaper before coating.
	+23°C	12 hours	
	+35°C	6 hours	

Products	Substrate Temperature	Minimum	Maximum
Sikadur®-330 overcoated with Sikagard®-coloured coatings	+10°C	5 days	Cured resin older than 7 days has to be degreased with Sika® Colma Cleaner and gently grinded with sandpaper before coating.
	+23°C	3 days	
	+35°C	1 day	

Times are approximate and will be affected by changing ambient conditions.

Notes on Application / Limitations

This product may only be used by experienced professionals.
The **Sikadur®-330** must be protected from rain for at least 24 hours after application.
Ensure placement of fabric and laminating with roller takes place within open time.
The SikaWrap® fabric must be coated with a cementitious overlay or coating for aesthetic and/or protective purposes. Selection will be dependent on exposure requirements. For basic UV protection use Sikagard®-550W Elastic, Sikagard® ElastoColor-675W or Sikagard®-680S.
At low temperatures and / or high relative humidity, a tacky residue (blush) may form on the surface of the cured **Sikadur-330** epoxy. If an additional layer of fabric, or a coating is to be applied onto the cured epoxy, this residue must first be removed to ensure adequate bond. The residue can be removed with water. In both cases, the surface must be wiped dry prior to application of the next layer or coating.
For application in cold or hot conditions, pre-condition material for 24 hours in temperature controlled storage facilities to improve mixing, application and pot life limits.
The number of additional fabric layers applied wet on wet must be closely controlled to avoid creeping, creasing or slippage of the fabric during curing of the **Sikadur®-330**. The number of layers will be dependent on the type of SikaWrap® fabric used and the ambient climate conditions.

SikaWrap®-300C
Carbon Fiber Fabric for Structural Strengthening

Description
- Unidirectional, woven, carbon fiber fabric for the dry and wet application process
- Equipped with weft fibers that keep the fabric stable (heat-set process)

Uses
Strengthening of reinforced concrete structures, brickwork and timber in case of flexural and shear load due to:
- Increase of loading capacity
- Changes of building utilisation
- Repair of defects
- Prevention of defects caused by earthquakes
- Meeting of changed standards or specifications

Advantages
- Multifunctional use as bending or shear reinforcement
- Flexibility of surface geometry (beams, columns, chimneys, piles, walls)
- Resistance against chemicals and environmental conditions (no corrosion)
- Fabric available in 2 widths for optimum utilisation
- Fiber orientation tailor-made

Technical Data

Fiber type	High strength carbon fibers
Fiber orientation	0° (unidirectional)
Construction	Warp: Carbon fibers (99 % of total areal weight) Weft: Thermoplastic heat-set fibers (1 % of total areal weight)
Areal weight	300 g/m² ± 5 %
Fiber Density	1.80 g/cm³
Fabric design thickness	0.17 mm (based on total carbon content)
Tensile strength of fibers	3'900 N/mm² (nominal)
Tensile E-modulus of fibers	230'000 N/mm² (nominal)

Strain at break of fibers	1.5 % (nominal)
Fabric length/ roll	≥ 50 m
Fabric width	300 / 600 mm
Shelf life	2 years from date of production
Package	1 roll in card board box
	The achievable laminate properties in the tensile test are depending on impregnating resin used and the type of laminate testing method. A loss of up to 30 % for the measured actual strength compared to the theoretical values is possible. Apply material reduction factors according to the standard used for design.

Application

SikaWrap-300C can be impregnated either with the dry or the wet application method.

Dry Application:

Sikadur-330 is a high quality mid-viscosity resin that requires no prior use of a primer to the prepared substrate.

For detailed resin properties and application details see Product Data Sheet Sikadur-330.

Wet Application

As a primer/sealer on smooth surface Sikadur-300 epoxy is used. On rough surfaces either Sikadur-330 or Sikadur-300 mixed with max. 5 % thixotropic agent Sikadur-513 is preferred.

Distribute 2/3 of the expected impregnating resin amount on a clean PE sheet and impregnate then the fabric with the resin: always roll at the fiber direction. Distribute the remaining 1/3 of the resin on the fabric and roll it in.

Apply the wetted fabric within the open time of the primer/sealer.

For detailed resin properties and application details see also Product Data Sheet Sikadur-300.

Resin Consumption

For both application methods the resin consumption is depending on the roughness of the substrate.

Dry Application:

- First layer: approx. 1.0...1.5 kg/m² (Sikadur-330)
- Following layers: approx. 0.5 kg/m² (Sikadur-330)

Wet Application Primer/Sealer Consumption:

- Smooth surface: approx. 0.5 kg/m² (Sikadur-300)
- Rough surface: approx. 0.5...1.0 kg/m² (Sikadur-330 or Sikadur-300 mixed with max. 5 % thixotropic agent Sikadur-513)

Wet Application Resin Consumption:

- approx. 0.7 kg/m² (Sikadur-300)

Important information

- The fabric can be cut with special scissors or a very sharp knife, but may never be folded.
- In fiber direction, overlapping of the fabric must be at least 100 mm or as per the project specifications. Overlapping sections of additional layers should be distributed over the column circumference.