JANUARY 2017 M.Sc. IN – Civil Engineering IMAD ABDULLAH MANSOORI

REPUBLIC OF TURKEY
UNIVERSITY OF GAZIANTEP
GRADUATE SCHOOL OF
NATURAL & APPLIED SCIENCES

REHABILITATION OF REINFORCED CONCRETE HAUNCHED BEAM WITH CARBON FIBER REINFORCED POLYMER (CFRP) STRIPS

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Imad Abdullah MANSOORI

ABSTRACT

REHABILITATION OF REINFORCED CONCRETE HAUNCHED BEAM WITH CARBON FIBER REINFORCED POLYMER (CFRP) STRIPS

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This thesis presents the shear behavior of damaged reinforced concrete haunched beams after repaired by injection epoxy and strengthened with unidirectional carbon fiber reinforced polymer strips (CFRP) at shear zone. The beams were designed without steel stirrups in order to fail in shear manner and after then rehabilitated. The experimental investigation was accomplished on twelve RC haunched beams of three different groups. The first group included two prismatic beams were repaired by epoxy injection and strengthened with vertical CFRP strips that have different width, the second group included eight haunched beams with different angles of inclination (10°and 15°). This group was repaired by epoxy injection and strengthened with CFRP strips by two methods, vertical (90°) and inclined (45°) according to the x-axis of the beam. The third group included two beams with an inclination of 5° which is repaired with epoxy injection.

Keywords: Rehabilitation, Haunched Beams, Strengthening, Carbon Fiber Reinforced Polymer, Debonding, Shear, Injection.

ÖZET

DEĞİŞKEN KESİTLİ BETONARME KİRİŞLERİN KESMEYE KARŞI KARBON LİFLİ POLİMER KUMAŞ ŞERİTLERLE İYİLEŞTİRİLMESİ

Imad Abdullah MANSOORI
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Ocak 2017
61 sayfa

Bu tez, hasar görmüş betonarme değişken kesitli kirişlerin çatlak tamir harcı ve tek yönlü karbon lifli polimer kumaş şeritlerle iyileştirilerek kesme bölgesinde kesme kuvvetine karşı davranışını inceleyen bir çalışmayı sunmaktadır. Çalışmada kullanılan kirişler daha önceden kesme modunda göçmeleri için etriyesiz üretilmişlerdir ve göçmeden sonra tamir edilmişlerdir. Tez çalışmaları kapsamında 3 farklı gruptan oluşan 12 değişken kesitli kiriş üzerinde deneysel çalışmalar yapılmıştır. Birinci grup çatlak tamir harcı ve değişik genişlikteki dikey pozisyonda karbon lifli polimer kumaş şeritlerle iyileştirilmiş 2 adet prizmatik kirişi içermektedir. İkinci grup farklı eğim açılarına sahip (10°and 15°) 8 adet değişken kesitli kirişten oluşmaktadır. Bu gruptaki kirişler çatlak tamir harcı ve kiriş eksenine 45° ve 90° olacak şekilde yerleştirilen karbon lifli polimer kumaş şeritlerle iyileştirilmiştir. Üçüncü grup ise, eğimi 5° olan ve sadece çatlak tamir harcıyla iyileştirilmiş 2 adet değişken kesitli kirişi içermektedir.

Anahtar Kelimeler: İyileştirme, Değişken Kesitli Kiriş, Güçlendirme, Karbon Lifli Polimer Kumaş, Ayrılma, Kesme, Enjeksiyon.

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TABLE OF CONTENTS

Page

LIST OF FIGURES

Page

LIST OF TABLES

Page

CHAPTER 1

INTRODUCTION

1.1 General

Haunched beams are widely used in structures like bridges and mid-rise buildings as seen in Fig 1.1. Despite the fact that it is widely used in current structures, the amount of experimental data available to learn its behavior in shear after rehabilitation and strengthening by carbon fiber reinforced polymer (CFRP) are very few. Like all structural members haunched beams can be exposed to design errors, leading to increase in the live loads and damage caused by earthquakes. The German Code of 2001 showed that there is a weakness in the factor of safety for several bridges because they were based on old codes (Hassan et el, 2015). Therefore, these beams need to be urgently strengthened or rehabilitated. One of the most widely used methods to strengthen concrete members is using the carbon fiber reinforced polymer (CFRP) in particular to strengthen shear zones.

Figure 1.1 Applications of the haunched beams.

1.2 Aims of research

The aims of this thesis are as follows:

1-To check the feasibility of using CFRP strips to strengthen the reinforced concrete haunched beam to resist shear forces.

2- To find the location of the critical section of the shear zone, where the critical section represents the location of the first debonding which happens to CFRP strips

3- To make comparison between vertical and inclined strengthening to find the best way of shear strengthening for the haunched beam.

4- To determine the effect of change in the angle inclination of haunched beam on the critical section location.

5- To investigate the effectiveness of epoxy injection to restore the capacity of haunched beam.

6- To provide a number of indicative data that could be used for rehabilitation of such kind of beams during maintenance of projects.

1.3 Thesis Organization

Chapter one: This chapter contains a brief introduction about haunched beams, purpose of research and the significance of research as well as a brief introduction about fiber reinforced polymers (FRP).

Chapter two: In this chapter was discussion the previous researches related to the external strengthening by CFRP at the shear zone for prismatic and haunched beams. Also discuss the types of failure and strengthening configuration.

Chapter three: This chapter contains all details related to the experimental program like reinforcement details, design of specimens, characteristics of the materials that were used in this research, setup of the test and the procedures of rehabilitation for all beams.

Chapters four: In this chapter have been presented and discussed the results of the test.

Chapter five: This chapter presents the summary, conclusions and recommendations for future Research.

CHAPTER 2

LITERATURE REVIEW

2.1 Overview

Nowadays reinforced concrete haunched beams (RCHB) are widely used in bridges and other structures. Especially in old bridges the need for repair, rehabilitation of RCHB in increasing day by day. An effective way for rehabilitation is to add an external transverse steel stirrup to remove the outer surface of the element that requires strengthening which increases the possibility of corrosion of reinforcing steel. However, recently, the use of CFRP has gained importance. This chapter provides a summary of the previous researches related to the use of the CFRP in the shear strengthening of reinforced concrete beams.

2.2 Strengthening by using CFRP system

The strengthening of structures by using externally bonded CFRP provides extra strength to the structures for restoring or increasing the ability of the structure to resist the external loads. The CFRP installation on the surfaces of structures is accomplished by using an appropriate adhesive (epoxy) (Matthys 2000). Strengthening approaches can be divided into two parts. The first appraoch is called **(Bond Critical Applications)** which depends on ensuring full coherence between the surface of concrete and materials. It is used to upgrade members to resist flexural moment or tensile and shear resistance.

The second approach of strengthening is called **(Contact Critical Applications)** which includes a full contact between the surface of the concrete and CFRP. An example may be the strengthening columns by enclosing the columns from all sides by using CFRP for upgrading the column to resist the axial pressure and increase ductility

2.3 Research significance

Much researches has been conducted in the past decade for evaluating the efficiency of using CFRP strips as external bonding to strengthen RC beam in shear (Adhilkary, B.B 2004; Khalifa et al. 1998; pellegrino, C 2006; Triantafillon 1998; zhang et al 2004). But, there is lack of studies of the shear behavior after repairing and strengthening of haunched beam by CFRP. This study focuses on finding the location of critical section for this type of beams where, the number of available studies on the strengthening and rehabilitation of this type of beams is very rare. Therefore this type of beams needs to be studied in advance as they are widely used in many buildings and particularly in bridges.

2.4 Raw materials of the fiber reinforced polymers (FRP)

The production of composite fiber reinforced polymer (FRP) needs two raw materials, the first one is fiber reinforced and the second is the polymer resin (matrix). The raw materials are produced at high temperatures in industrial processes that require very specialized equipment.

2.4.1 Fibers

The fibers are made from materials ranging in diameters 5-20μm consisting of long filaments placed parallel one-way or two-way cross (Zoghi 2010). The FRP composite material contains thousands of individual micrometer-diameter filament.

The vast majority of fiber used in FRP products consists of long indefinitely fiber called continuous fibers. The volumetric content of continuous fibers used to reinforce polymer resin are a relatively high (20 to 60 %) (Wiley 2006). The Fibers have important properties such as high elastic modulus, high ultimate strength, low variation of strength between individual fibers, uniformity of fiber diameter, surface, Availability in suitable forms and acceptable prices, high toughness and durability.

The mechanical characteristics of fibers are usually more in value than mechanical characteristics of resins. The most widely fibers using in the manufacture of FRP are glass, carbon, and aramid. These types of fibers show a linear elastic behavior under tensile loading up to failure without exhibiting any yielding as shown in Fig. 1.2.

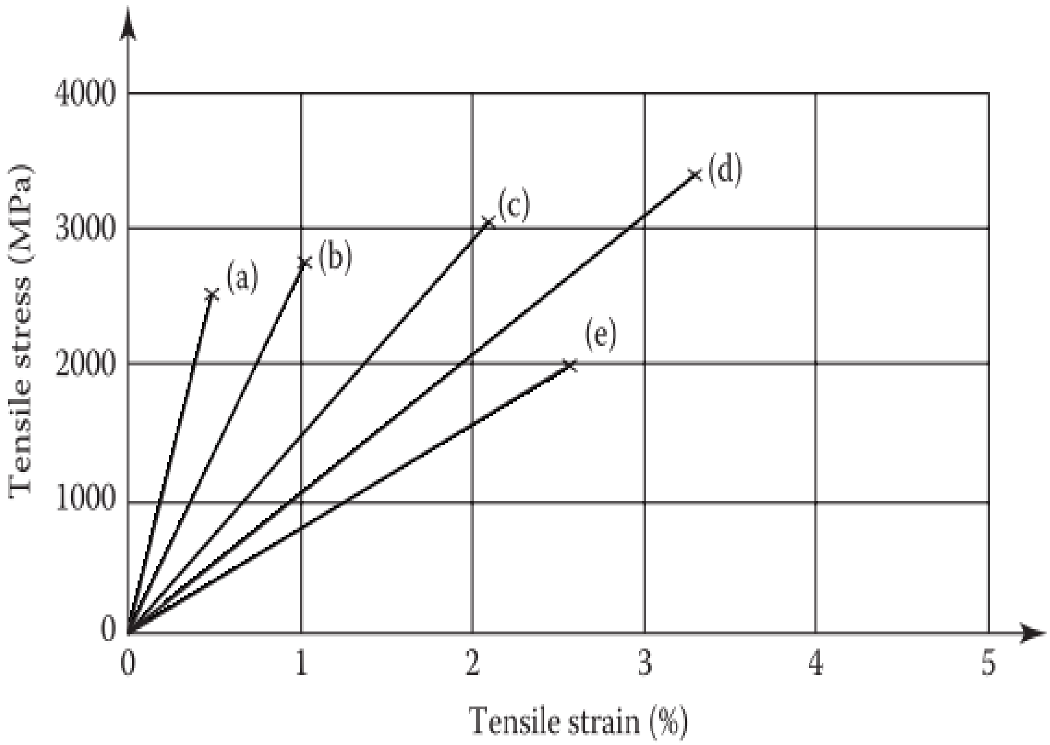


Figure 1.2 Stress–strain curves of typical reinforcing fibers (a) carbon (high modulus); (b) carbon (high strength); (c) aramid; (d) S-glass; and (e) E-glass(Nawaf Khaled 2014)

Carbon fiber: Carbon fibers are solid, semi crystalline organic materials containing atomic level for a planar two-dimensional array of carbon atom. Carbon fiber is produced at high temperatures [1200 to 2400℃ (~2200 to 4300℉)] from three possible precursor materials: a natural cellulosic rayon textile fiber, polyacrylonitrile (PAN), or pitch (coal tar). Pitch-based fibers are produced as a by-product of petroleum processing; it has generally lower cost than rayon-based fibers. Carbon fibers have high resistance to tensile strength and high modulus of elasticity compared to other fiber types and low density. It has a good resistance to wear, rust and corrosion. But the main disadvantage is that it is brittle. (Zoghi 2010).

There are several types of carbon fibers. The most important types are:

1- Carbon fibers general purpose (standard).

2- Carbon fibers, high strength.

3- Carbon fibers ultra- high strength.

4- Carbon fibers, high modulus.

5- Carbon fiber ultra-high modulus.

Characteristics of common grades of carbon fibers are listed in Table1.1.

Table 1.1 Approximate characteristics of common grades of carbon fibers

Grad of Carbon Fiber	Density (g/cm^3)	Tensile Modulus (GPa)	Tensile Strength (MPa)	Max Elongation (%)
Standard	1.7	250	3700	1.2
High strength	1.8	250	4800	1.4
High modulus	1.9	500	3000	0.5
Ultra high modulus	2.1	800	2400	0.2

2.4.2 Polymer Resins (matrix)

Polymer resins are materials with a big molecular weight up to hundreds of thousands of repetitive units called monomers. Polymers can be formed from the hydrogen and carbon atoms are called organic polymers or can be made up of atoms, such as Silicon (si), phosphorus (p), or sulfur(s) and called inorganic polymers. Polymers work as a mediator to assemble the fiber inside the polymer structure in multiple directions according to the required specifications as shown in the figure 1.2. Polymers can be divided into two categories according to their behavior under the influence of heat:

1-Thermoplast: This type is relent when subject to heat and solidifies when cooling and could be re-formed several times by heating and cooling.

2-Thermoset: This type does not relent by heating, but solidifies definitively by cooling. This type is characterized by possession of good electric properties. The polymers which do not relent by the heat and reinforce.

There are three main types of resins used in the manufacture of FRP like polyester, Vinyl ester and epoxy resins. The resins of epoxy are characterized by the following general performance characteristics (Leeming 1999):

1- Have wide range of mechanical properties due to the diversity of additives.

2- Does not emit volatile materials during the curing process.

3- Decreases the rate of shrinkage during curing.

4- Excellent resistance to chemicals and solvents.

5- Good adhesion to surfaces, fillers and fibers.

6- Superior strength retention under sustained load and low creep.

7- Good wetting properties for a variety of substrates and high surface activity.

2.5 Adhesives

The adhesive is known as a material used for the purpose of the glue FRP sheet onto the concrete surface to provide a path for load between the concrete surface and the FRP sheet(Fib 2006). The adhesive material must provide excellent performance over the life of the structure, even when exposed to environmental conditions or temperature change. There are several types of polymers that can be used, like polyesters, cross-linkable acrylics, polyurethanes and epoxies.(Matthys 2000), the most commonly used structural adhesive is epoxiy. In short, the adhesive requirements are as follows (Leeming 1999):

1- It must demonstrate sufficient adhesion in the case of bonding between the concrete and FRP.

2- The temperature of glass transition of the adhesive, (Tg) must be at least 40°C.

3- The tensile strength and bulk shear at 20°C should be equal or more than 12MPa.

4- It should not be affected by the alkaline nature of concrete and its possible effect on the durability of the joints.

2.6 Most important advantages and disadvantages of FRP using for external strengthening

The choice of appropriate strengthening or repairing technique is closely linked to the condition of the structure to be strengthened and environmental conditions. There are many alternatives, but the final assessment is mainly related to the cost effectiveness. The choice of the appropriate alternative is not related to reduce the initial costs, but also to minimize the maintenance cost. Due to the similarity between The strengthening by using CFRP and strengthening using steel plate, we will review of the most important advantages and disadvantages of using CFRP compared with steel plate.

- The most important advantages of CFRP are given below:

1- CFRP possesses excellent resistance to corrosion.

2- CFRP has a low weight and can be available in continuous lengths. This feature makes it easy to install and more flexible (e.g., in closed places), particularly in the case of wet lay-up systems.

3- High stiffness to weight ratio.

4- Lightweight, workability and also can be used for several layers up to seven layers and you can add it quickly.

5- High fatigue and impact strength.

6- Low heat conductivity.

- On the other hand, the most important disadvantage of CFRP is given as follows:

1- Its failure is brittle.

2- Weak when subjected to high temperatures or fire.

3- Not resistant to alkali.

2.7 Using of CFRP to strengthen the deficit at shear zones

The rehabilitation by using CFRP could be in the form of strengthening or repairing of damaged structural members or strengthening of structures to resist earthquakes. The reasons for the use of CFRP are that it possesses a good mechanical specification mentioned earlier.

2.8 Wrapping schemes of CFRP

The best ways to strengthen the shear zone is full wrapping of the structural member, but this method is impractical for some structural members such as beams that carry the roof. Full wrapping of the element in shear zone is preventing any possibility of a delaminating or debonding failure. This system is usually used in columns, in this type of strengthening, the CFRP wrap must be continuous around the element. Direct Bond between the CFRP and the concrete substrate is not critical because the continuous CFRP wrap adheres directly to itself as shown in the Fig. 2.1 (A). In the case of inability to complete wrapping of the element, there are alternative configurations of CFRP materials which are less efficient than full wrapping called as **U- wrapping.** This method includes the wrapping around three sides of the section beams to improve shear strength. This type of strengthening allows the design engineer to specify the angle of installation with respect to the longitudinal axis of the beam to get the best results as shown in the Fig. 2.1 (b).

In the case of inability to using U- wrapping because of the existence of section enlargement such as the bulb tee or in the case of a roof over the beam and the wall under beam as shown in the Fig. 2.1 (c, d), **Side bonding** technique is the least efficient of the three schemes (Hollaway & Teng 2008; Kaiser 1989; Kim et al. 2012; Tumialan 2007; Wiley, 2006).

2.9 Factors affecting on CFRP contribution to increase shear strength

There are several factors can influence on the efficiency of CFRP materials. Some of these factors are not linked to the material properties alone, but rather with the location and manner of application**.** These factors include, but are not limited to:

1- Different CFRP layouts and configurations.

2- Strain distribution across the critical crack.

3- Number of layers of CFRP material.

4- Shear span-to-depth ratio. The shear span is known as the distance between the closest face of a support and the location of a point load enforced to the beam. The latest ACI design guideline for FRP (ACI 440.2R-08) does not mention the effects of the shear span to depth ratio, but many researchers have pointed out that the shear span-to-depth ratio has an importance in design(Bousselham, Abdelhak 2006; Challal, o., Shahawry, M, Hassan 2002). When the shear span-to-depth ratio gets smaller, the concrete beam will try to show a different mode of shear failure than the conventional sectional shear mode.

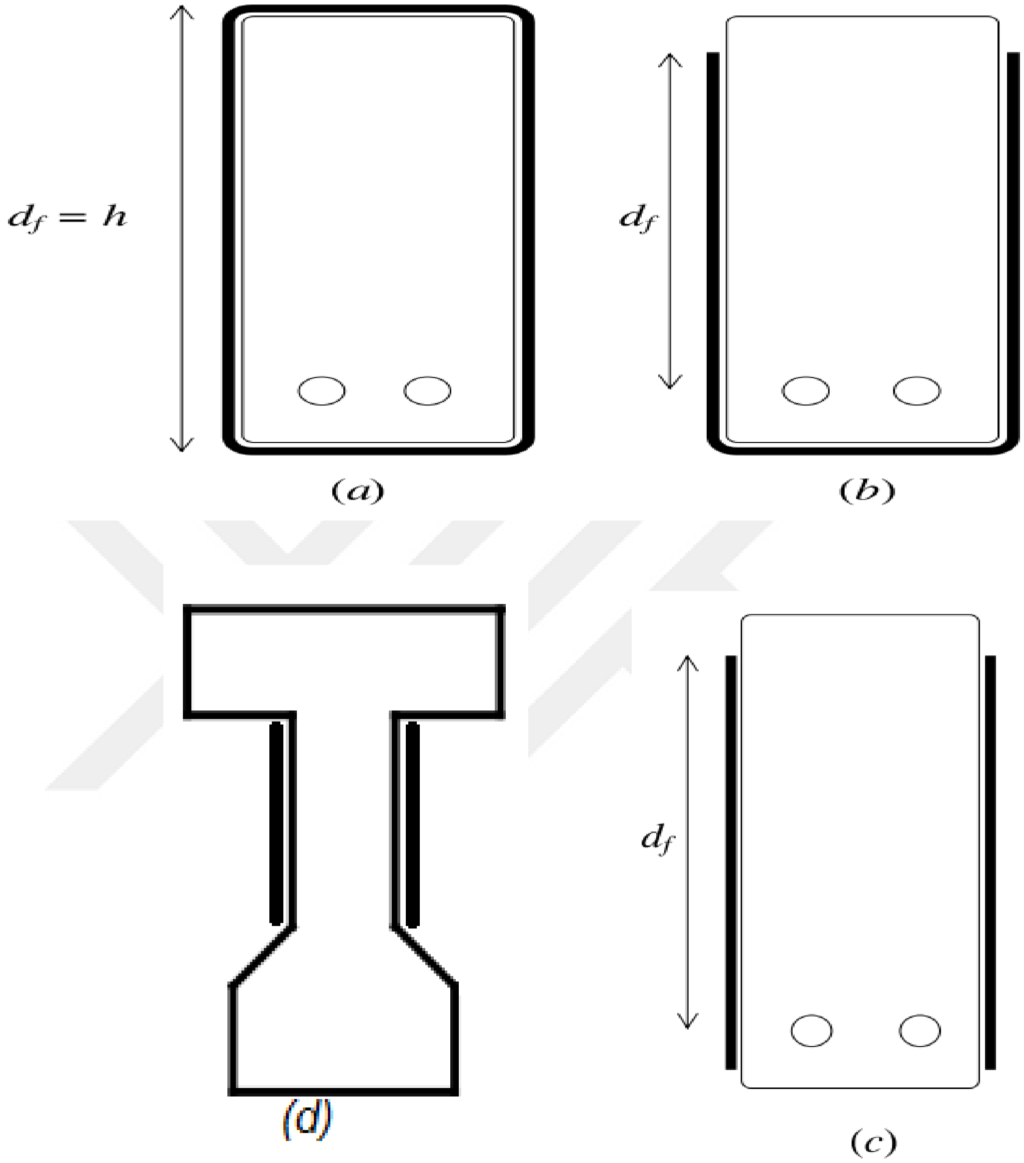


Figure 2.1 CFRP shear strengthening schemes and effective depths: (a) four-sided; (b) three-sided; (c, d) two-sided.

2.10 Repair of cracks using injection method

Cracks in reinforced concrete structures cannot be avoided because of the low tensile strength of concrete. The large cracks lead to expose the reinforced steel to weather and corrosion damages which cause delimitation or rapture to FRP layers and reduce the strength, stiffness, in addition to influence of the aesthetic of concrete structure(Ahmad et al. 2013; Ekenel & Myers 2007). Therefore, we must repair these cracks to avoid any damage the famous techniques for repairing cracks is injected epoxy resin.

The epoxy resins have good specifications which make it suitable for bonding cracks. These specifications are as follows:

1- Solvent-free.

2- Can be used in cases of repairs, dry and wet cracks.

3- No problems when used in low temperature.

4- High mechanical and adhesive strengths.

5- Possesses low viscosity which assisted it in the access of narrow cracks

6- Possible injection by simple equipment.

The epoxy resins have a susceptibility to seal cracks approaching 0.002 in (0.05 mm)

2.11 Effect of injection of cracks on the performance of reinforced concrete beams

The cracks in the reinforced structures cannot be avoided because of the low tensile resistance of concrete. Wider cracks do not affect the aesthetic of concrete only, but the steel may be exposed to the weather, which could lead to corrosion. Furthermore, the ACI 440.2R-08 recommends that the cracks bigger than 0.3 mm should be repaired because its effect on the performance of the externally bonded FRP which may lead to premature debonding or rupture of the FRP.

2.12 Shear strengthening of reinforced concrete haunched beams

Despite the widespread use of haunched beam in bridges and the buildings to offer advantages such as reducing the weight and to achieve some requirements architectural and aesthetic.

A very small amount of research has been conducted on using of CFRP materials to shear strengthening of reinforced concrete haunched beam(hasssan 2015; DIN 2001). Abed Elrahman, 2015 investigated the behavior of Thirty-six different (compressive strength and mode) haunched beams strengthened by using steel side plates and epoxy as adhesive. Details and geometry are explained in Fig. 2.2. The beams are divided into two groups N and P. The first group of N is divided into five series.

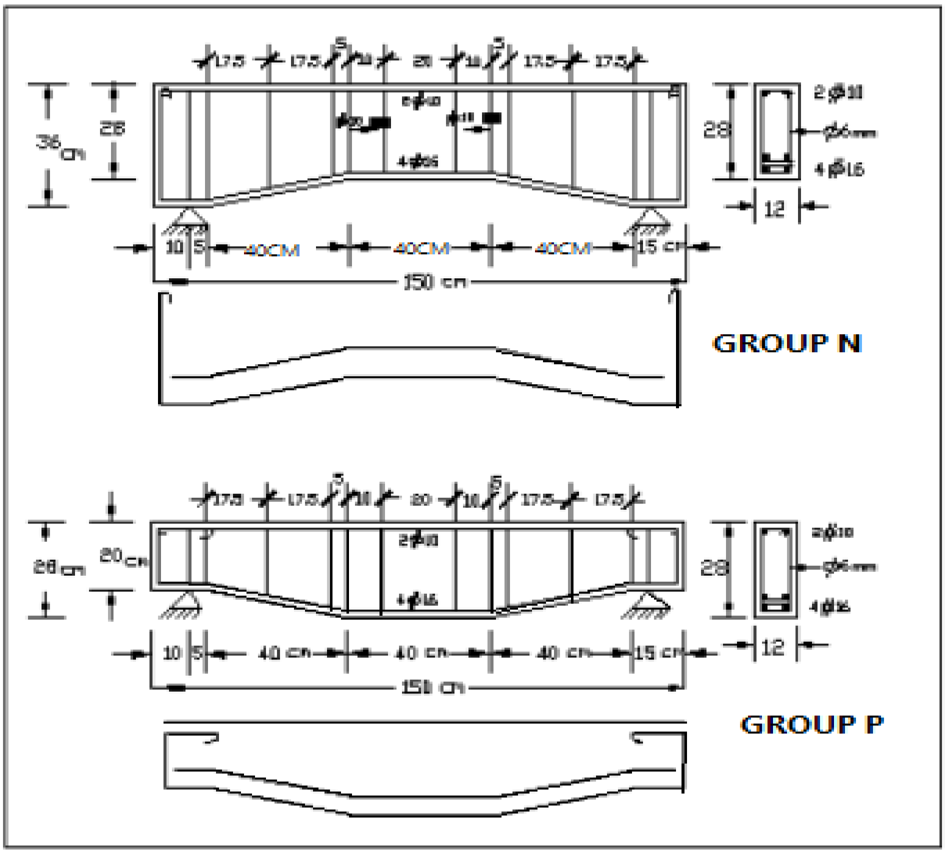
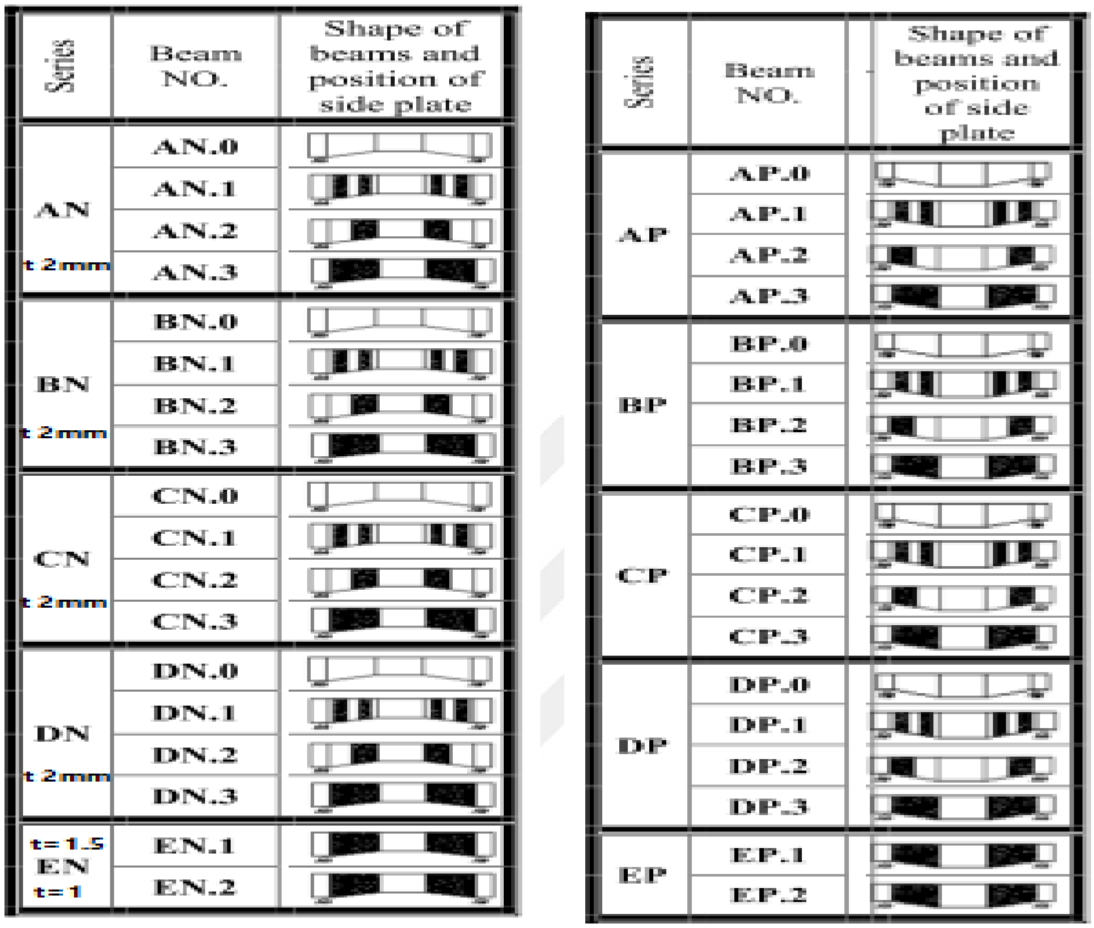


Figure 2.2 Details of reinforcement and geometry(Rashwan 2010).where is this reference in text.

The rest of details can be seen in Fig. 2.3. Through the experiment results, the researcher concluded that, the failure for all un-strengthened beams was shear failure, But after strengthening the type of failure changed from shear failure to shear, compression and flexural with crashing failure at the top of mid span. The stiffness and ultimate load for group N was observed to be greater than group P, as shown in the Fig. 2.4 (Rashwan 2010).



Series	Beam NO.	Shape of beams and position of side plate
AN t 2mm	AN.0	
	AN.1	
	AN.2	
	AN.3	
BN t 2mm	BN.0	
	BN.1	
	BN.2	
	BN.3	
CN t 2mm	CN.0	
	CN.1	
	CN.2	
	CN.3	
DN t 2mm	DN.0	
	DN.1	
	DN.2	
	DN.3	
EN t=1.5	EN.1	
t=1	EN.2	

Series	Beam NO.	Shape of beams and position of side plate
AP	AP.0	
	AP.1	
	AP.2	
	AP.3	
BP	BP.0	
	BP.1	
	BP.2	
	BP.3	
CP	CP.0	
	CP.1	
	CP.2	
	CP.3	
DP	DP.0	
	DP.1	
	DP.2	
	DP.3	
EP	EP.1	
	EP.2	

Figure 2.3 Details and data of testing beams (Rashwan 2010) where reference in text

Dawood and Nabbat,2015 examined shear and flexural behavior of high strength twelve reinforced concrete haunched beams with and without opening, strengthened with carbon fiber reinforced polymer (CFRP) and carbon rebar to strengthen shear zones. Six beams designed to fail in shear. And shear zones were strengthened by using near-surface-mounted (NSM) see Fig. 2.5. All results in the ultimate load and change in the type of failure from shear to flexural(Dawood & Nabbat 2015).

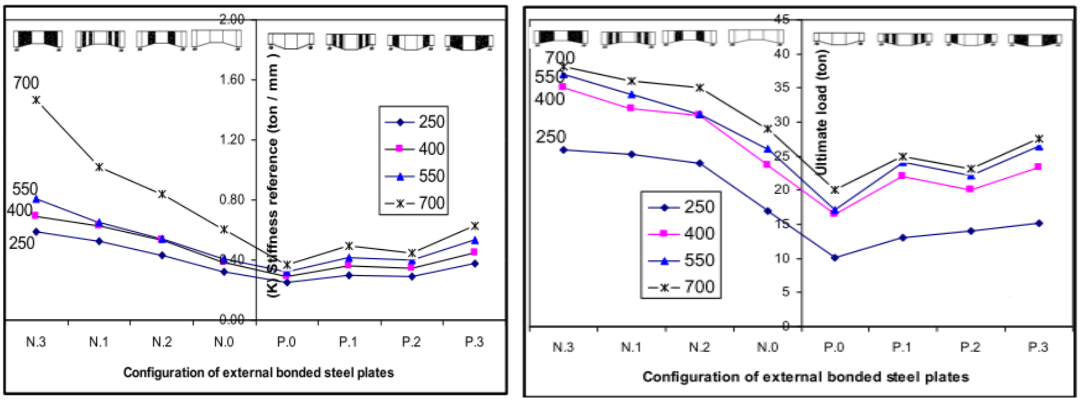


Figure 2.4 Relation between ultimate and stiffness (k) with the mode of beams for all series(Rashwan 2010). Where is this reference in text.

Figure 2.5 Types of failure after strengthening(Dawood & Nabbat 2015). where is this reference in text.

2.13 Failure types of externally applied CFRP

On the basis of previous research found that there are two basic types in the failure of the CFRP. The first failure kind is CFRP debonding; this type of failure occurs when the level of strain is much lower than the ultimate strain and includes separation of the CFRP from the concrete substrate(Chen & Teng 2003; Carloni & Nobile 2007; Khalifa et al. 1998; Sas et al. 2008). The second type of failure is rupture.

CHAPTER 3

EXPERIMENTAL PROGRAM

3.1 Overview

The experimental program in this study consists of twelve RC beams belonging to three different groups. The first group includes two prismatic beams which were repaired by epoxy injection and strengthened with vertical CFRP strips that have different width. The second group includes eight haunched beams that have different angles of inclination (10°and 15°) which were repaired by epoxy injection and strengthened with CFRP strips by two methods, vertical angle (90°) and the inclined angle (45°) according to the x-axis of the beam. The third group includes two beams with an angle (5°) of inclination. This group was just repaired with epoxy injection. The importance of studying this type of beams was to find the location of critical section that represents the location of the first debonding of CFRP strip. All of the beams were strengthened by U-wrapping manner.

3.2 Geometry and reinforcement details of tested beams

The RCHBs in this study were divided into three general modes. The first mode is prismatic, the second mode is A-mode haunched beam and the third mode is B-mode haunched beams which are shown in Fig. 3.1 in details All beams have 1700mm length, 150mm width and the depth is variable along shear span. Depth at mid of beams is constant which are 25 cm for B mode and 30 cm for a mode. The geometry and steel reinforcement details of beams are shown in the Table 3.1. The shear span of all beams is equal to 0.65 m and all beams are designed without steel stirrups except at points of load and supports as shown in the fig. 3.1.

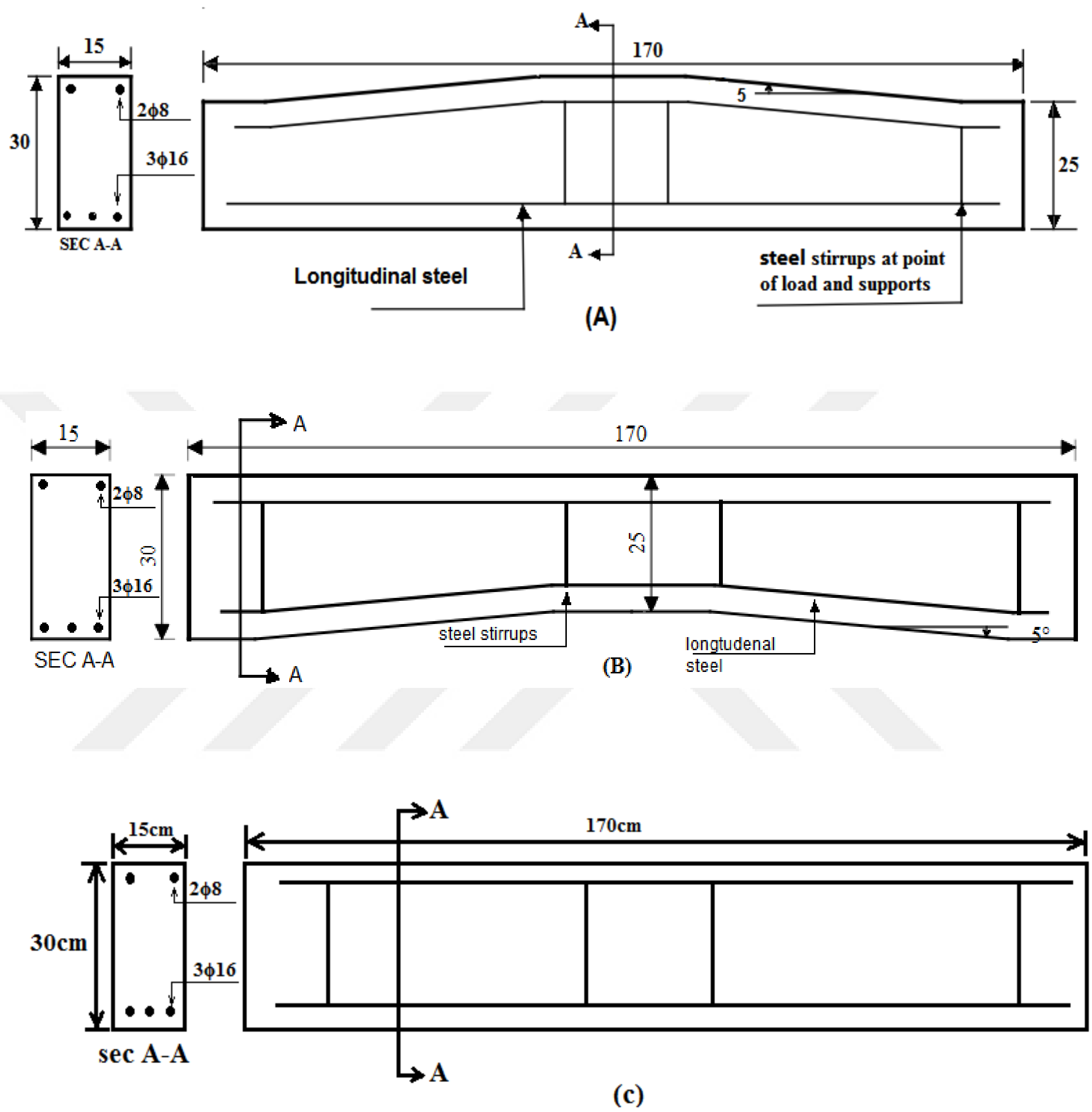


Figure 3.1 Geometry and steel detail for mode A, mode B and mode C haunched beams.

Table 3.1 Geometries of haunched beams.

Beam code	Mode	$F_{c'}$ (MPa)	E_C (MPa)	α^o	h_f^* (mm)	h_S^* (mm)	Reinforced details			
							A_s (mm^2)	N0. Steel	$A_{s'}$ (mm^2)	N0. Steel
Rf1*U90	A	44.5	34.2	0	300	300	603	3ϕ16	100	2ϕ8
A1-5INJ*	A	60	37	5	300	250	603	3ϕ16	100	2ϕ8
A1-10U90	A	49	35.26	10	300	200	603	3ϕ16	100	2ϕ8
A2-10U45	A	51.5	33.2	10	300	200	603	3ϕ16	308	2ϕ10
A1-15U90	A	42.5	34	15	300	150	603	3ϕ16	100	2ϕ8
A2-15U45	A	60	37.2	15	300	150	603	3ϕ16	308	2ϕ10
Rf2U90	-	60.7	34.2	0	250	250	603	3ϕ16	100	2ϕ8
B1-5INJ	-	58.5	37.26	-5	250	300	603	3ϕ16	100	2ϕ8
B1-10U90	B	44	33.1	-10	250	350	603	3ϕ16	100	2ϕ8
B2-10U45	B	61	37	-10	250	350	40	2ϕ16	100	2ϕ8
B1-15U90	B	62	37.4	-15	250	400	603	3ϕ16	100	2ϕ8
B2-15U45	B	50.1	32.74	-15	250	400	402	2ϕ16	100	2ϕ8

INJ: repaired with epoxy injection **RF**: reference beam (prismatic) **A**: Beam mode **15**: angle inclination of haunched **U**: type of wrapping **90**: angle inclination of CFRP strips according to x-axis **$h_{m:}$** depth at middle **$h_{s:}$** depth at the supports

3.3 Materials

3.3.1 Concrete

A number of experimental mixtures were prepared to obtain self-compacting concrete (SCC). To achieve the properties of SCC crushed stone and sand was used from the same source. The volumetric gradient for crushed stone and sand are shown in Fig. 3.2.

Moreover, the fine materials are used in this mixture like silica fume, fly ash (type F) and Portland cement (type 32.5). The resulting mixture has achieved all the fresh requirements set by the self-compacting concrete guide (EFNARC). The compressive strength was between 44.2 and 62 MPa. The proportions of components of the concrete mixture are listed in the Table 2.

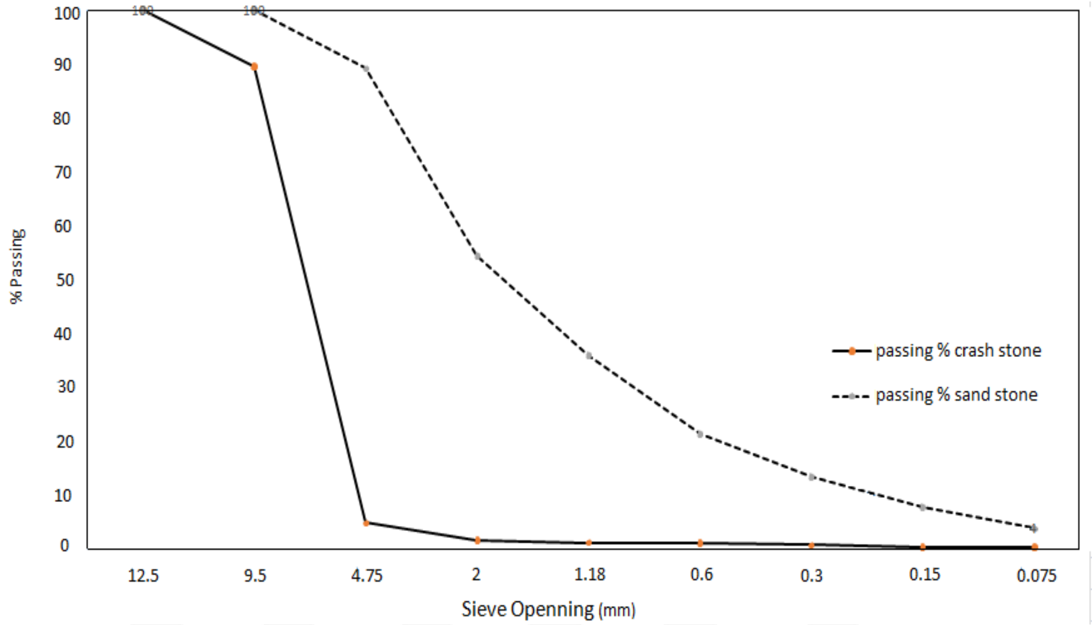


Figure 3.2 Volumetric gradients for gravel and sand

Table 3.2 Weight of concrete mix components of one cubic meter

Materiales	Agreggate (Kg)	Sand (Kg)	Cement (Kg)	Silica fum (Kg)	Fly ash (Kg)	Water (L)	Super plasticizer (L)
Weight/m^3	680	1100	250	35	215	150	5

3.3.2 Steel reinforcement

Three sizes of steel reinforcement were used. The diameters of steel reinforcement were (8, 10 and 16) mm. The yield strength of the bars was 550, 485 and 456 MPa, respectively; the ultimate strength was 640, 595 and 560 MPa respectively.

3.3.3 Epoxy injection

In this study, Sikadur 52 epoxy was used which consists of two-components A and B shown in Fig. 3.3. These two components were mixed in a ratio of 2:1 by electric mixer (drill) with slow rotation (maximum 250 RPM) for 3 minutes to get a homogeneous mixture according to the manufacturer's recommendations. Table 3.3 shows the properties of epoxy resin (Sikadur 52).

Figure3.3 Sikadur-52 two components A+B

Table 3.3 Properties of Sikadur 52

Type	Flexural Strength (N/mm^2)	Tensile Strength (N/mm^2)	Compressive Strength (N/mm^2)	Bond Strength (N/mm^2)	Pot life at 10C°Minute (s)	Mixing ratio	Thermal Expansion Coefficient for every C°
Sikadur 52	1800	37	52	4	80	2:1	$89*10^{-6}$

3.3.4 Adhesive and CFRP sheet materials

The epoxy adhesive used in this study was (Teknobond 300 Tix). It consists of two-component A and B shown Fig 3.4, mixed with a ratio of 1:4 by electric mixer (drill) with a speed of 400 RPM for 2-3 minutes to get a homogeneous mixture according to manufacturer's recommendation. Table 3.4 shows the properties of (Teknobond 300 Tix). The CFRP sheet that used in this study was unidirectional (SPN U 300). The Table 3.5 shows the general properties for the CFRP.

Figure 3.4 TEKNOBOND 300 TIX A+B

Table 3.4 General properties of TEKNOBOND 300 TIX

TYPE	Pot life at 20C° (Minutes)	Flexural Strength 7 days (N/mm2)	Compressive Strength7 days (N/mm^2)	Bond Strength 7 days (N/mm^2)	Mixing ratio	Modulus of Elasticity 7 days (Gpa)	Color (A + B)	Fluid density (kg./lt.)
TEKNOBOND 300 TIX	30	≥ 25	≥62.5	≥3	4:1	>20	Off-White	1,02 ± 0,02

Table 3.5 Properties of carbon fiber reinforced polymer fabric

Type	Pattren	Tensile Strength (Mpa)	Tensile Modulus (Gpa)	Strain (%)	Densityg/m^2	Thickness (mm)	Yarn	Weft
SPN U 300	Unidirectional	4900	240	2	300±%3	0.3±%10	12K DOWAKSA CARBON (800 TEX)	1/cm Glass hot melt

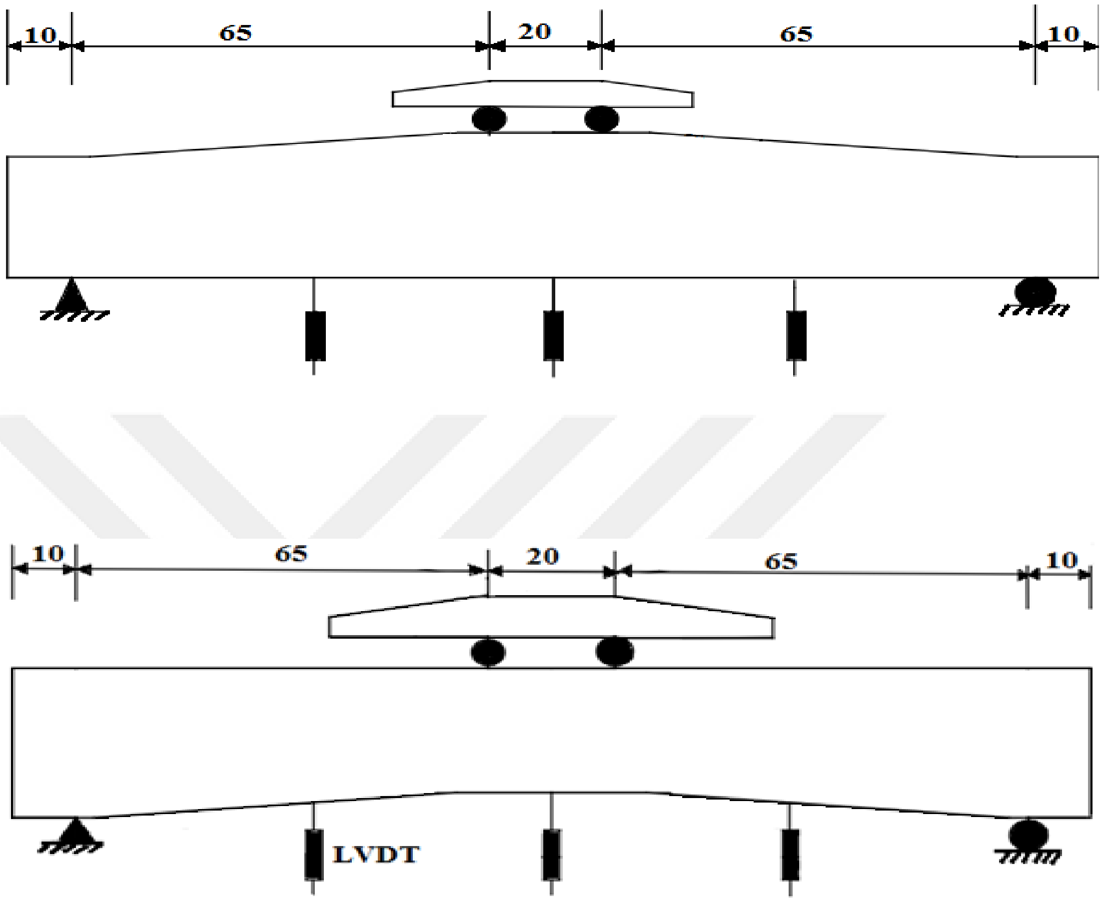


Figure 3.5 Load configuration of A and B modes.

3.4 Test procedures and Instrumentation

The clear span was 150 cm for all tested beams. 4-point bending test was applied .For the loading BESMAK hydraulic jack with 600 KN capacity was used. All beams were tested under displacement control at a rate (0.2 mm) up to the point of failure. The displacement was recorded automatically by three LVDTs placed at the center of the concrete beam in addition to the right and left sides shown in Fig. 3.5.

3.5 Repair and rehabilitation of beams

After testing all the beams which reached to ultimate failure (where the failure of all beams was shear failure), rehabilitation processes were started which include the following steps:

1- Preparation of beams to inject epoxy.

2- Preparation of concrete surface was started immediately after curing period of epoxy injection.

3- Installation of CFRP strips by using the wet lay-up method.

3.5.1 Repairing cracks by epoxy injection

The cracks were cleaned by using compressed air to remove dust and loose materials, which effect on the efficiency of bonding shown in Fig 3.6.

Figure 3.6 Cleaning of cracks by using compressed air.

The cracks were injected according to ACI 440.2R-08, where it’s recommended that cracks bigger than (0.3 mm) must be repaired may cause deboning or rupture of CFRP premature.

The injection process includes creating an entry at a distance of (15cm) along cracks. The cracks were sealed on three faces of beam by using a high viscosity epoxy (Teknobond 200) to keep up the low viscosity epoxy (Sikadur 52) from leaking out as shown in Fig. 3.7.

The two components of high viscosity epoxy A+B were mixed with a proper amount with a ratio of 2:1 by using an electric mixer (drill) with a speed of 250 RPM for three

Figure 3.7 Sealing cracks of three faces of the beam

Minutes. When the mixture became ready, it was used to seal cracks. After this procedure, we wait and finally start the injection process. All cracks were injected by low viscosity epoxy (Sikadur 52). The injection was done by gravity as shown in the Fig. 3.8 (A, B).

3.5.2 Surface preparation

Previous research has shown the importance of bond strength between concrete surface and CFRP strips (De lorenzis et el 2001; Nakaba et el 2001; Orton,et el 2009). Concrete grinder with rotating disk was used to prepare the concrete surface to remove a laitance layer and roughen the surface of concrete.

Fig. 3.19 shows the use of concrete grinder to clean and roughen the surface of beams. The bottom edges of beams were rotated with a radius about (0.5 in) according to ACI 440.2R-08 to avoid the concentration of stresses which lead to premature rupture of the CFRP strips as shown in Fig. 3.10.

Figure 3.8 (A) Mixing epoxy A+B by using a suitable mixer (b) injection epoxy by gravity.

Figure 3.9 Grinding concrete surface before applying CFRP strips.

Figure 3.10 Rounded bottom edge in order to reduce the stress concentrations on the CFRP strip

3.5.3 Installation of CFRP strips

Before proceeding with the installation of CFRP strips, the concrete surface was cleaned again by using compressed air to remove any remaining dust that results from surface grinding. Lines are then drawn on the surface of concrete to specify the locations as shown in Fig 3.11

Figure 3.11 Drawing lines for the application of CFRP strip on its specific location.

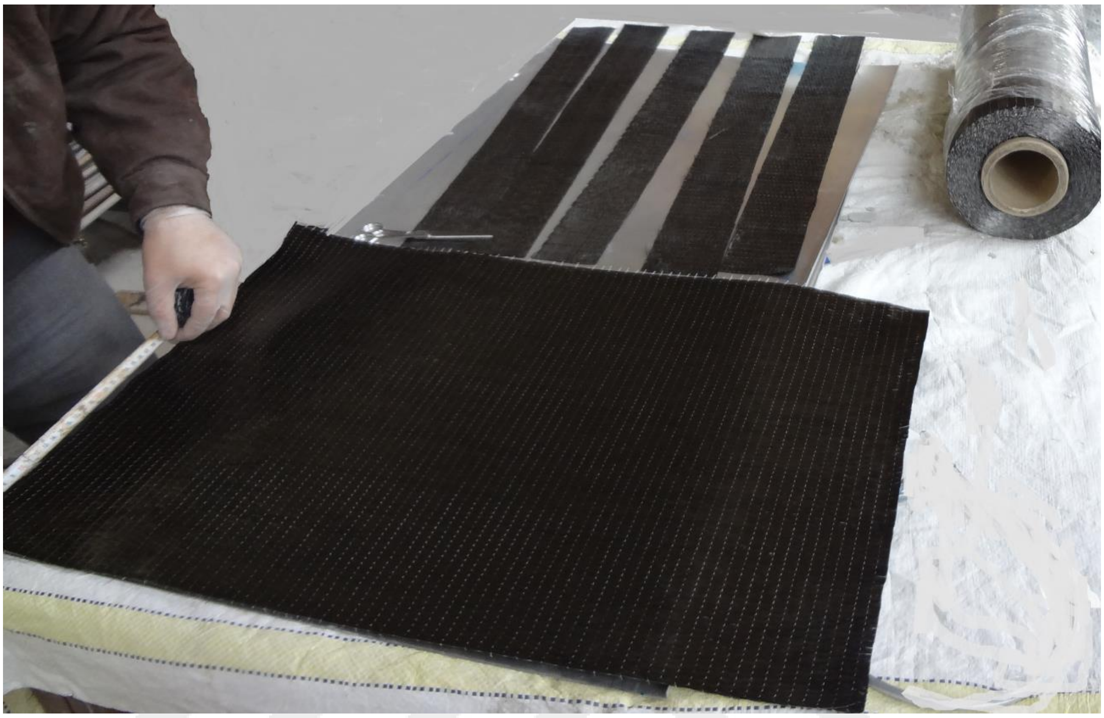


Figure 3.12 Cutting of CFRP strips by using a sharp scissors according to required lengths

All CFRP strips were cut into proper lengths to avoid wasting in quantity of the carbon sheet as shown in Fig. 3.12. After cutting the CFRP sheet, we started the processes of mixing a suitable amount of two components of epoxy that were used to bond CFRP sheet with concrete surface. The processes of preparing bonding epoxy included two components (A+B) with a ratio 4:1. The epoxy was mixed by using the electric mixer (drill) with a speed of 250 RPM for three minutes. The installation of CFRP sheet was by Wet Lay-up method, which requires to start with impregnation of CFRP sheet with epoxy for two faces to make sure that the process of fiber saturation with epoxy is sufficient as shown in Fig 3.13. The sheets become ready to be installed onto the surface of the beam. The CFRP strips were compressed carefully to expel air bubbles as shown in Fig 3.14 and Fig 3.15.

Figure 3.13 Process of impregnation of CFRP sheet with epoxy before installation.

It should be noted here that the configuration of strengthening was U-wrapping where the end of the CFRP strip reached to the upper edge of the beam and the distance between CFRP strips was 200mm from center to center. The width of strips was 50mm except one of the reference prismatic beam where the distance between strips was 180mm and the width of strips was 80mm from center to center.

This reference beam has been strengthened for the purpose of testing the ability of the contribution of the CFRP strip to increase the resistance of shear force to avoid excessive strengthening.

In this study, there are two types of strengthening schemes were applied: The first one is vertical strips the second is inclined strips with an angle of 45° shown in Fig. 3.14.

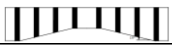
The configuration of strengthening for all beams is shown in Table 3.6. After the completion of the installation of all CFRP strips on the concrete surface, all beams are stored for seven days in the laboratory where the temperature was 21° to complete the treatment stage as recommended by the manufacturer of the epoxy resin as shown in Fig. 3.16.

Figure 3.14 Using of knife and plastic roller to press the CFRP

Figure 3.15 Method of compressing the CFRP using a knife.

Figure 3.16 Curing of all beams after strengthening.

Tables 3.6 Configuration of CFRP strips

Beam code	Strengthening scheme	W_f (mm)	S_f (mm)	t_f (mm)
Rf1U90		80	180	0.3
A1-5INJ		-	-	-
A1-10U90		50	200	0.3
A2-10U45		50	200	0.3
A1-15U90		50	200	0.3
A2-15U45		50	200	0.3
Rf2U90		50	200	0.3
B1-5INJ		-	-	-
B1-10U90		50	200	0.3
B2-10U45		50	200	0.3
B1-15U90		50	200	0.3
B2-15U45		50	200	0.3

$W_{f:}$ width of CFRP

S_f: spacing between CFRP strips.

$t_{f:}$ Thicknes of CFRP strips.

CHAPTER 4

EXPERIMENTAL RESULTS AND DISCUSSION

4.1 Overview

All beams were repaired by epoxy injection and strengthening by CFRP strips except two haunched beams with the angle of inclination equal to 5° which were just repaired by epoxy injection to check the efficiency of epoxy injection to restore the capacity of the haunched beam.

The effect of three parameters on the shear behavior of reinforced concrete haunched beams after rehabilitation was studied which are:

1) The efficiency of epoxy injection to restore the load capacity of the beam.

2) The effect of change in the configuration of CFRP strips on the location of the critical section.

3) The effect of change in the angle of inclination at shear zone.

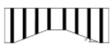
This chapter will review the discussion of the test results before and after rehabilitation and discuss the nature of the failure and load-deflection curves before and after rehabilitation.

The discussion of the test results will contain some of the following information:

1- Photos of the beam after failure.
2- Load –displacement curves.
3- Shear crack before and after rehabilitation.
4- The stages of the debonding failure of CFRP strips.

The test results for all beams are listed in the Table 4.1. The results in this table include the peak load and type of failure before and after rehabilitation.

Table 4.1 Results of experimental tests.

Beam code	Strengthening scheme	$F_{c'}$ MPa	Experimental			Mode of failure	
			p_u (KN)	$^*P_U$ (KN)	$\%^*P_U/p_u$	Before repairing	After repairing
Rf1U90		44.5	107	227	2.12	Shear	Crash
A1-5INJ		60	113.5	118.2	1.04	Shear	Shear
A1-10U90		49	114.5	170.5	1.48	Shear	Shear
A2-10U45		51.5	115.3	160	1.39	Shear	Shear&Splitin g
A1-15U90		42.5	121	158.5	1.3	Shear	Shear
A2-15U45		60	113.2	164	1.49	Shear	Shear&Splitin g
Rf2U90		60.7	94.4	134	1.42	Shear	Shear
B1-5INJ		58.5	108	142	1.31	Shear	Shear
B1-10U90		44	101.5	130.8	1.28	Shear	Shear
B1-15U90		62	104	136	1.31	Shear	Shear
B2-10U45		61	91.7	110	1.19	Shear	Flexural
B2-15U45		50.1	95.7	110	1.15	Shear	Shear& Flexural

$P_{U=}$ a peak load capacity before rehabilitation (kN).

$^*P_{u.\,=}$ a peak load capacity after rehabilitation (kN).

4.2 Failure modes

This section will review the stages of failure and loads associated with the first debonding of CFRP strips and the loads associated with the appearance of first crack. It is noteworthy that the first appearance of the cracks is not visible and it is difficult to find. The location of the new diagonal shear crack after rehabilitation was not in the same location of the diagonal shear crack before rehabilitation and was shown in red color. The experimental outcomes of the test results are summarized for each beam given below.

BEAM RF1U90

Reference and prismatic beam, the load of failure before rehabilitation was 83.7 kN with shear failure and after rehabilitation the load of failure increased to 227 kN with crash failure at the top of the mid beam. Formation of the shear cracks began in 155.5 kN and the new shear crack did not occur in the same location of the previous shear crack. Debonding occurred at the end of strip on right side near the point of load as shown in Fig. 4.1 (A). In addition to this, in the middle of the second strip debonding was observed, which is not along the strip as shown in Fig. 4.1(B, C).

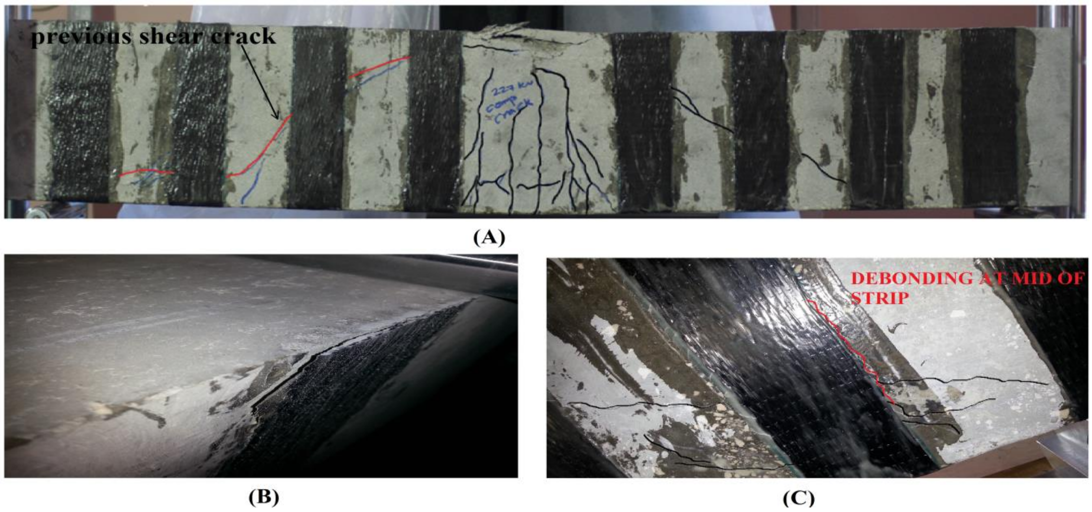


Figure 4.1 Failure mode of RF1U90.

BEAMS A1-5INJ and B1-5INJ

The angle inclination of these haunched beams is 5°. These beams were repaired only by epoxy injection, in order to investigate the possibility of restoring the capacity of this type of haunched beams. For both **A1-5INJ** and **B1-5INJ** there was an increase in load capacity after rehabilitation (%4 and 31% respectively). The new shear crack occurred near to the former shear crack with the same behavior. The wavy red lines are showing the lactation of previous cracks while the black lines are showing the current cracks as shown in Fig. 4.2 and Fig. 4.3 respectively.

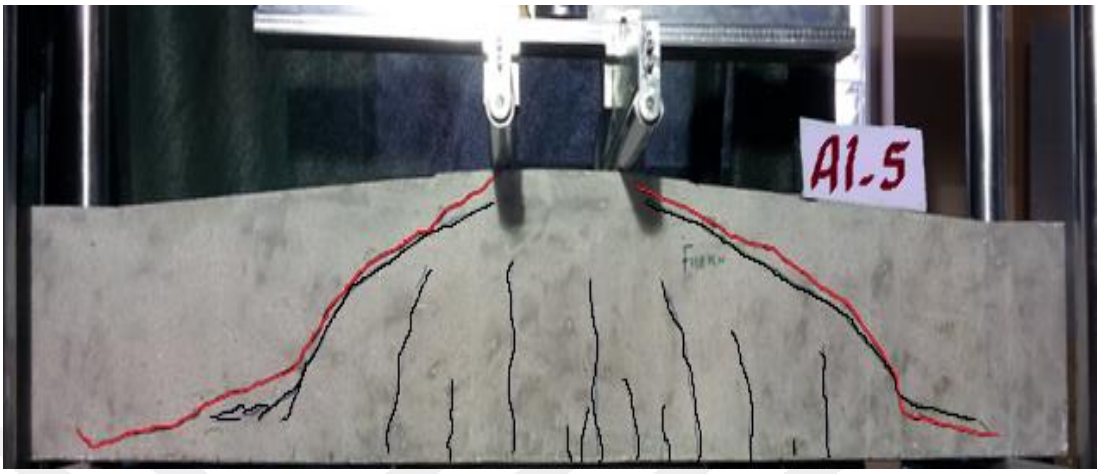


Figure 4.2 Failure mode of A1-5INJ beam.

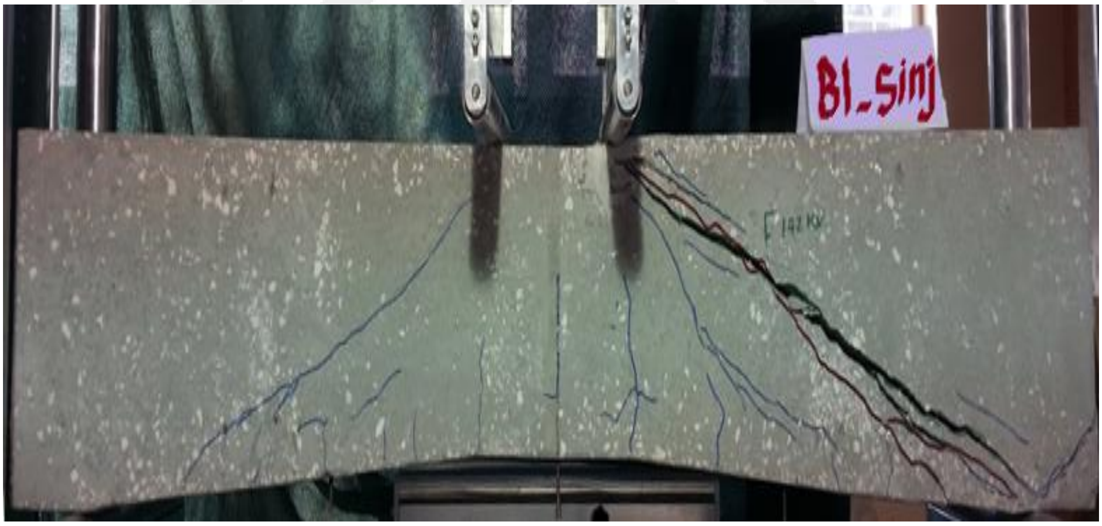


Figure 4.3 Failure mode of B1-5INJ beam

BEAM A1-10U90.

The failure load before rehabilitation was 114.5 kN with shear crack and after rehabilitation the failure load increased to 170.5 kN with shear crack as shown in Fig. 4.4. The first appearance of a shear crack was at 65.3 kN. The shear cracks continued to expand with increasing load when the load reached up to 169 kN debonding was observed on the third strip on the right side of the beam as shown in Fig. 4.4 (A, B, C).

It was noted that the first debonding occurred in the third strip number where the length of the debonding was (5) cm. There was no debonding at the beginning of loading on the second strip because the bonding length of CFRP strip was enough (13cm) to withstand the stresses caused by the expansion of a shear crack.

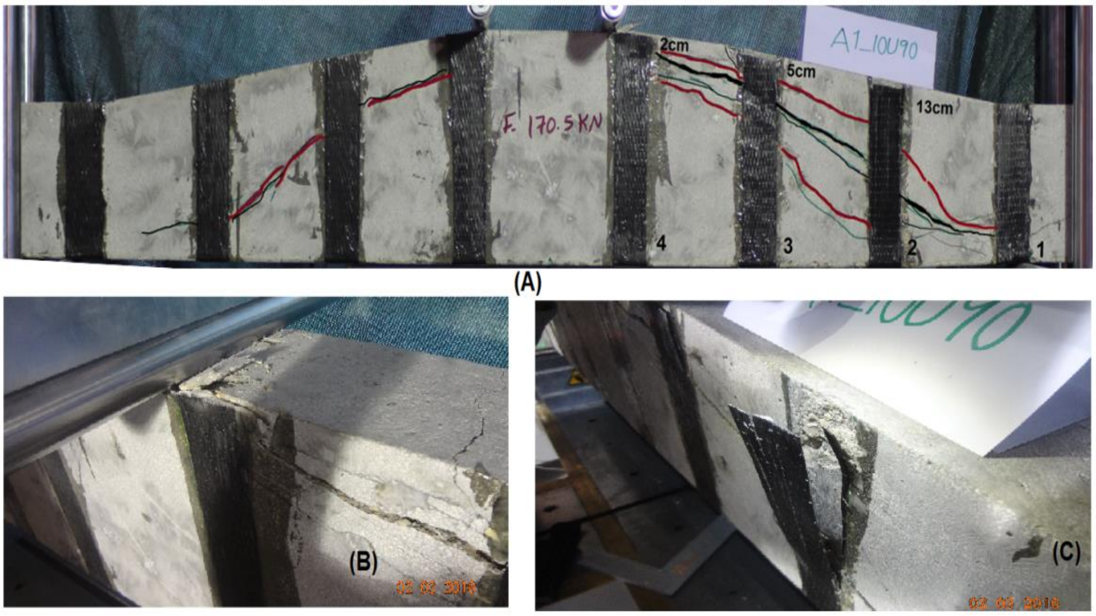


Figure 4.4 Failure mode of A1-10U90 beam

BEAM A1-15U90

The failure load before rehabilitation was 116 kN with shear failure AND after rehabilitation the failure load increased to 158.5 N with shear splitting crack Fig. 4.5 (A, C). The first appearance of a shear crack was at 69.6 kN where this shear crack continued to expand with increase in load. When the load reached up to 146 kN debonding was observed in the second strip number, on the back of the right side as shown in the Fig. 4.5 (B).

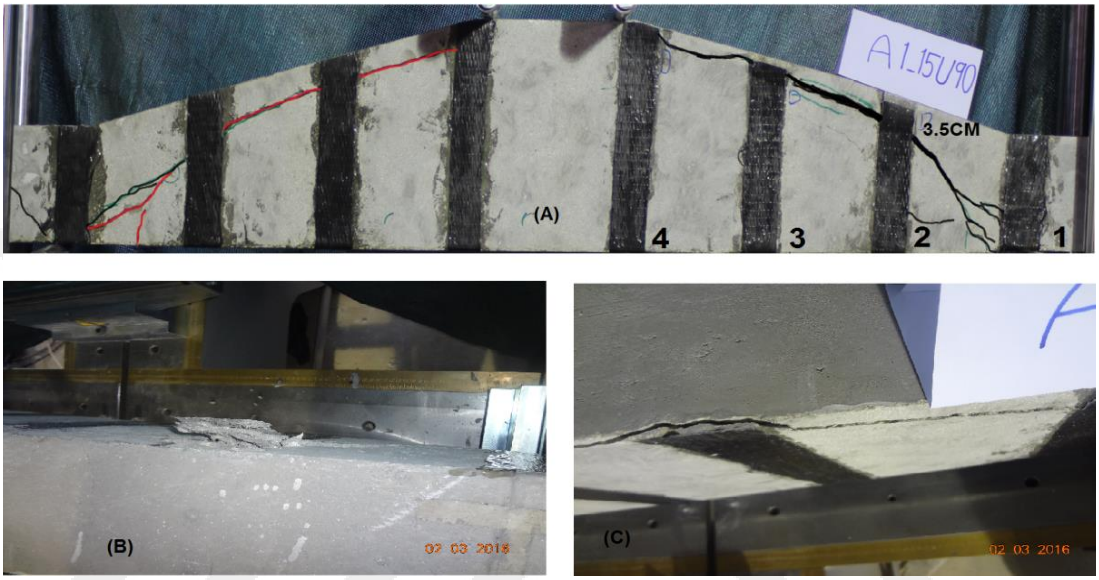


Figure 4.5 failure mode of A1-15U90 beam.

BEAM A2-10U45

The failure load before rehabilitation was 115.3 kN with shear failure and after rehabilitation increased to 160 kN with shear failure sees (fig. 4.6A). The first appearance of a shear crack was at 69.6 kN and the first debonding was noted in the first strip number at load about 160 kN on the front face Fig 4.6 (B, C).

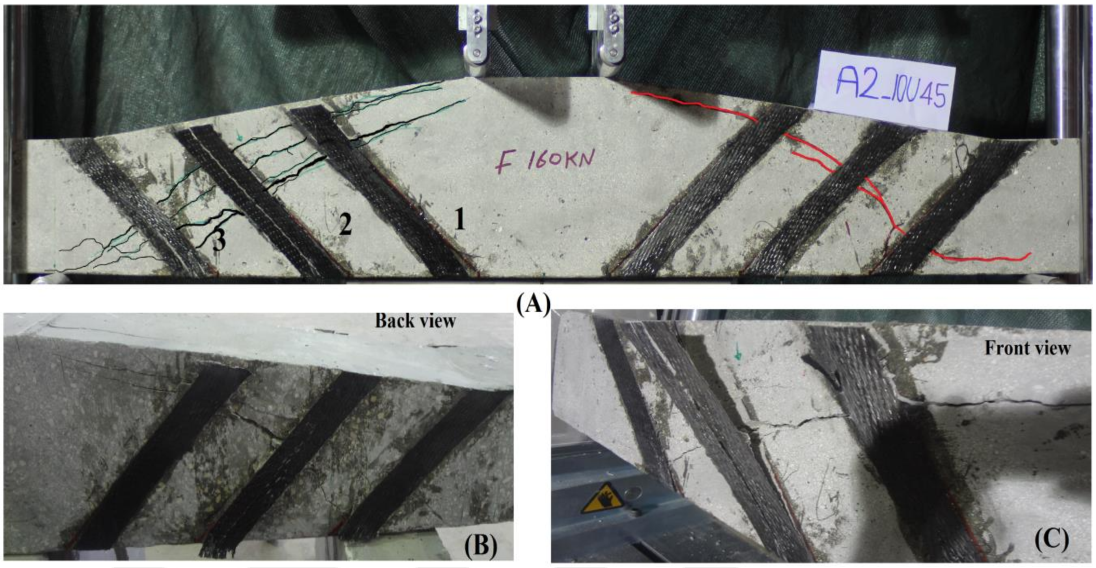


Figure 4.6 Failure mode of A2-10U45 beam.

Beam A2-15U45

The failure load before rehabilitation was 113.2 kN with shear failure and after rehabilitation the failure load increased to 164 kN with shear failure. The first debonding was noted in the third strip at a load about 149 kN as shown in fig. 4.7A, After increasing the load, the shear crack began to expand and transform to split crack at the second strip. When the load reached to 164 kN debonding was observed into the third strip at front face accompanied by a splitting crack at the second strip as shown in Fig. 4.7(B, C).

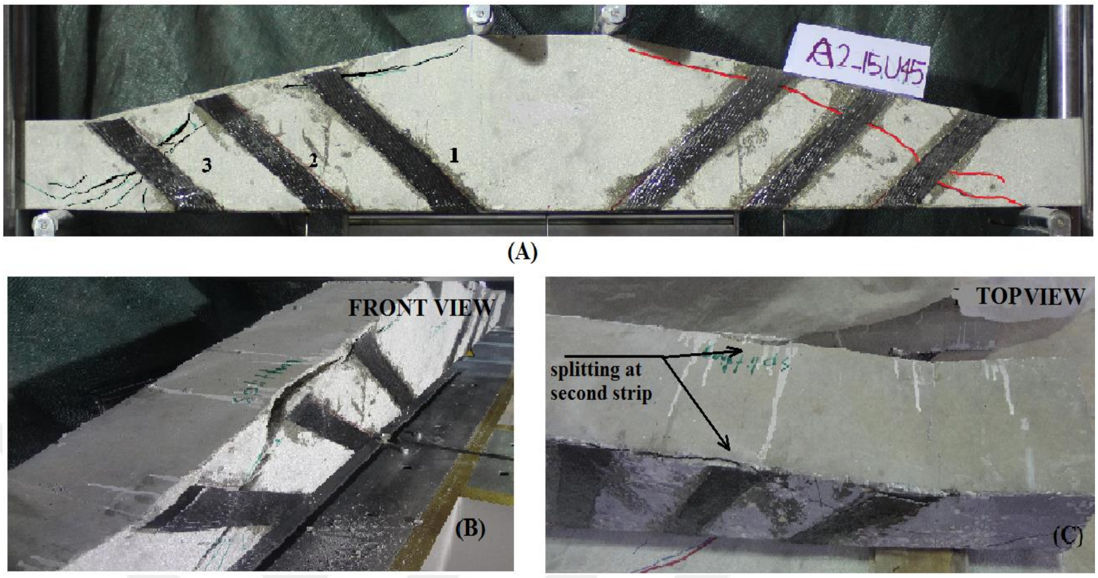


Figure 4.7 Failure mode of A2-15U45 beam.

Beam Rf2U90

This is the reference (prismatic) beam. The type of rehabilitation for this beam was U-wrapping where the width of CFRP strips was 50mm, with 200mm spacing between the strips from Centre- to-center as shown in the Fig. 4.8 (A). The first debonding was noted in the strip number (3) at load about 119KN on the right side of the back with a shear crack shown Fig. 4.8 (B).

After increasing the load until it reached to 134KN, debonding occurred on the strip number (2) and the final failure of the beam was shear shown Fig. 4.8 (C). The failure load before rehabilitation was 94.4 kN, and after rehabilitation increased to 134 kN.

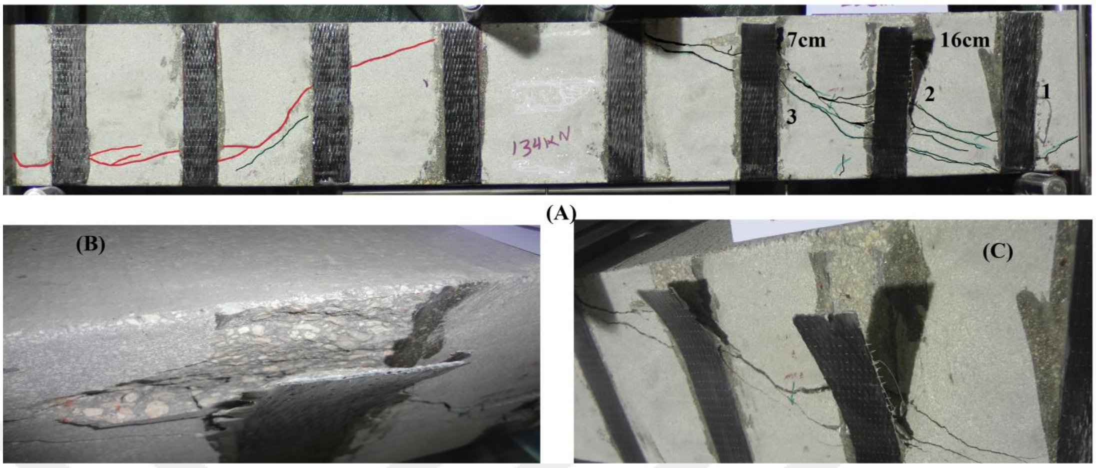


Figure 4.8 Failure mode of Rf2U90 beam

Beam B1-10U90

The first debonding was noted in the strip number (2) at a load about 115 kN on the right side of the back face with shear crack as shown in Fig. 4.9 (B). When the load increased to 130.8 KN, the shear crack began to expand and debonding occurred at the strip number (2) on the right side of the front face which led to reduce the load on the beam. The load was increased again to 127.7 kN, which led to debonding at the strip number (1, 3) as shown in Fig. 4.9 (A, C). The critical section was located at the strip number (2). The failure load before rehabilitation was 101.5 kN with shear failure, after rehabilitation increased to 130.8 kN with shear failure as well.

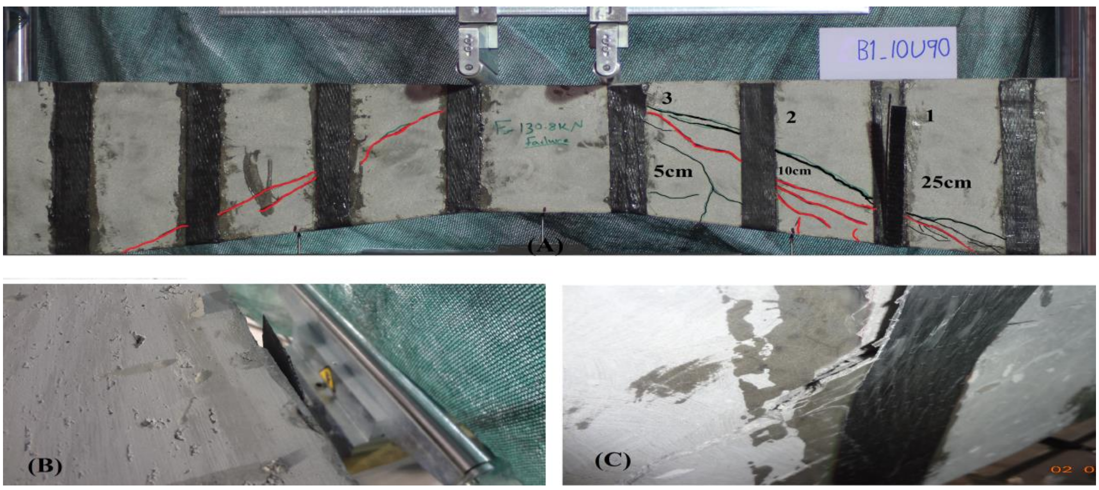


Figure 4.9 Failure mode of B1-10U90 beam

Beam B1-15U90

The first debonding was noted in the strip number (1) at a load about 104.7kN on the right side of the back face with shear crack as shown in the Fig. 4.10. After increasing the load to 104.2 kN sudden debonding occurred at the strip number (2) on the right side of the front face, followed by debonding at the strip number (1, 3) with shear failure. The failure load before rehabilitation was 104KN with shear failure, after rehabilitation increased to 136 kN with shear failure as well.

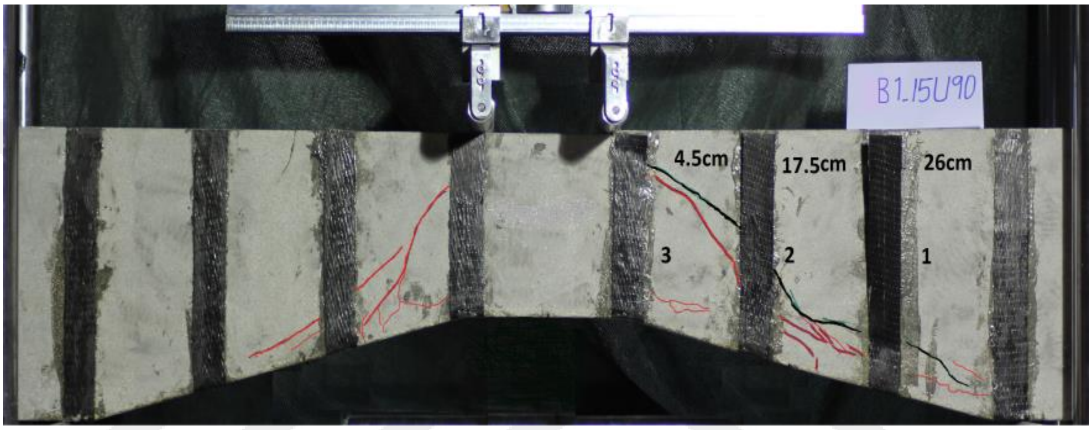


Figure 4.10 failure mode of B1-15U9

Beam B2-10U45

This type of haunched beam failed with flexural manner after rehabilitation, and didn't no debonding was observed as shown in Fig 4.11. The failure load before rehabilitation was 91.7 kN with shear failure and after rehabilitation the failure load increased to 110 kN with flexural failure as well.

The reason why no debonding was observed in CFRP strips may be due to the high strength of concrete 62 MPa, which improves bonding ability between the CFRP strips.

Figure 4.11 Failure mode of B2-10U45 beam

Beam B2-15U45

This type of haunched beams was failed by shear and flexural cracks after rehabilitation as shown in the Fig. 4.12 (A, B). The previous failure mode before rehabilitation was shear failure.

The first debonding occurred in strip number (2) as shown in Fig. 14 (B) when the load reached to 110 kN. The failure load before rehabilitation was 95.7 kN and after rehabilitation increased to 110 kN.

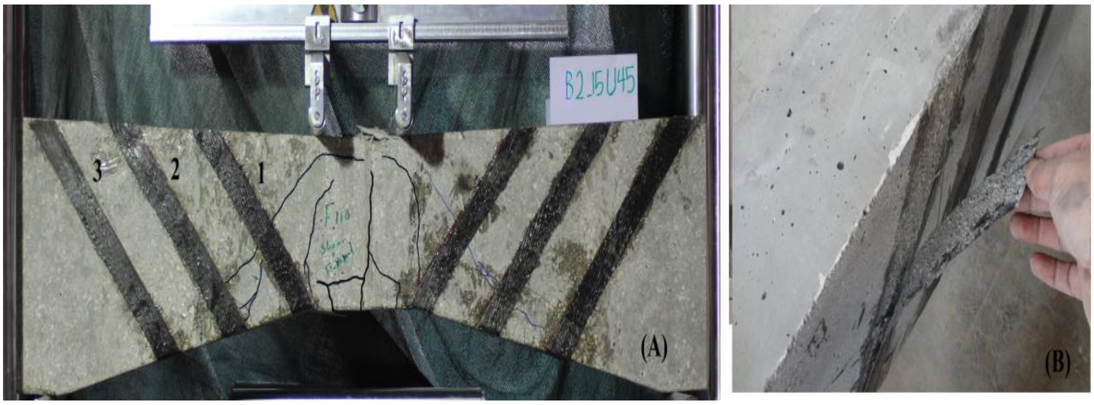


Figure 4.12 Failure mode of B2-15U45 beam.

4.3 Load–displacement curves before and after rehabilitation.

In this section we will discuss the load-displacement curves, the rate of increase in the peak load and stiffness of the beams before and after rehabilitation for each beam.

4.3.1 Beam RF1U90

As seen from Fig. 4.13, Stiffness decreased after rehabilitation. On the other hand, the ultimate load and ductility increased 112%. The ultimate load before and after rehabilitation was 107 kN and 227 kN, respectively.

The maximum displacement before and after rehabilitation were 3.9 mm and 5.6 mm, respectively. The first shear crack in concrete before and after rehabilitation was appeared at 94.4 kN and119 kN respectively.

4.3.2 Beam RF2U90

The stiffness after rehabilitation decreased as shown in the Fig 4.14. On the other hand, the ultimate load and ductility increased to 42%. The ultimate load before and after rehabilitation was 94.4 kN and 134 kN respectively.

The maximum displacement before and after rehabilitation were 3.9 mm and 5.6 mm, respectively. The first shear crack in concrete before and after rehabilitation was appeared at 94.4 kN and 119 kN respectively.

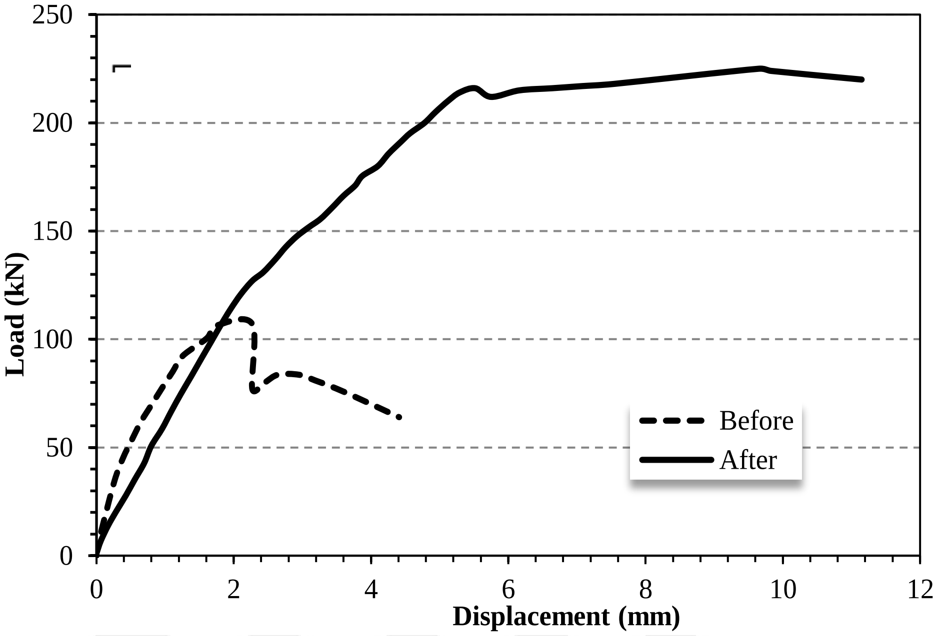


Figure 4.13 Load-displacement curve for RF1-U90 beam.

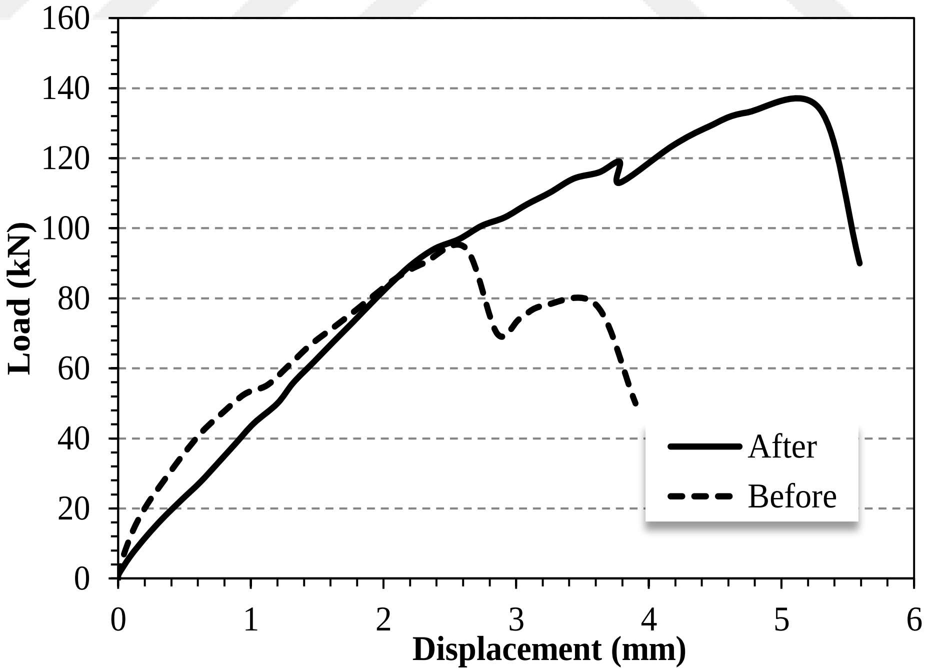


Figure 4.14 Load-displacement curve for RF2-U90 beam

4.3.3 Beam A1-5INJ

As seen from Fig. 4.15, Stiffness decreased after repairing cracks by epoxy injection. On the other hand, the ultimate load and ductility increased 4.7%. The ultimate load before and after repairing was 113 kN and 118.2 kN, respectively. The maximum displacement before and after rehabilitation were 2.37 mm and 3.4 mm, respectively. The first shear crack in concrete before and after repairing was appeared at 113.5 kN and111 kN respectively.

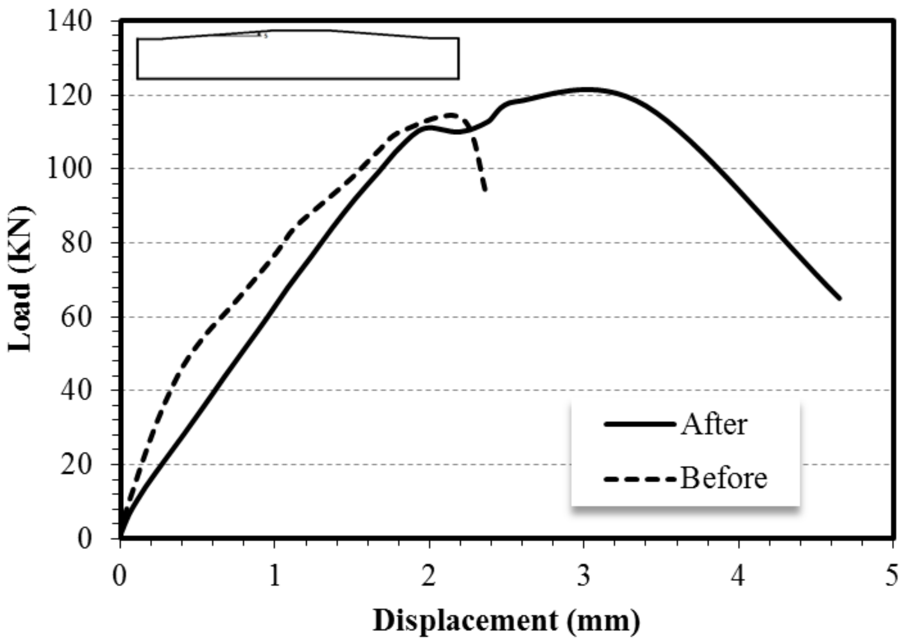


Figure 4.15 Load- displacement curve of the A1-5INJ beam.

4.3.4 Beam B1-5INJ

As seen from Fig. 4.16, Stiffness decreased after repairing cracks by epoxy injection. On the other hand, the ultimate load and ductility increased 31%. The ultimate load before and after repairing was 108 kN and 142 kN, respectively. The maximum displacement before and after rehabilitation were 3 mm and 5.1 mm, respectively. The first shear crack in concrete before and after repairing was appeared at 80.2 kN and 137 kN respectively.

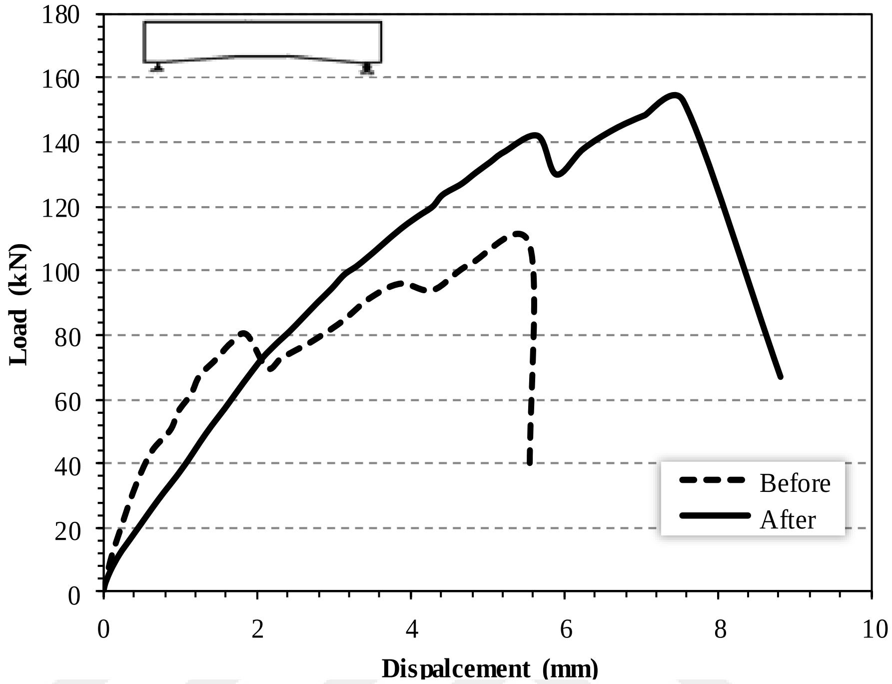


Figure4.16 Load- displacement curve of the B1-5INJ beam.

4.3.5 Beam A1-10U90

The stiffness after rehabilitation decreased as shown in the Fig. 4.17. On the other hand, the ultimate load and ductility increased to 48%. The ultimate load before and after rehabilitation was 114.5 kN and 170.5 kN respectively.

The maximum displacement before and after rehabilitation were 3 mm and 5.1 mm, respectively. The first shear crack in concrete before and after rehabilitation was appeared at 114.5 kN and 170.5 kN respectively.

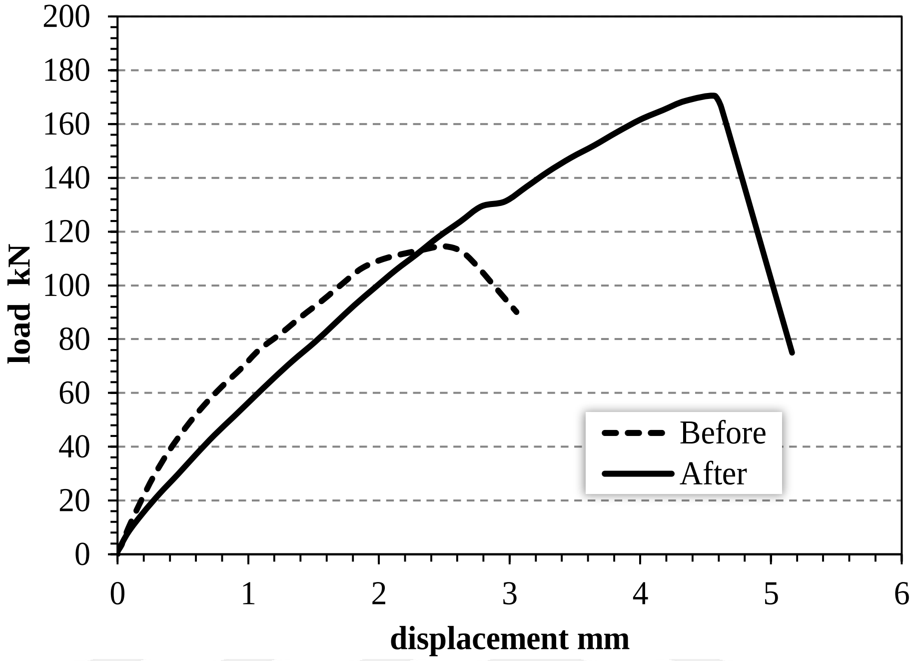


Figure 4.17 Load- displacement curve of the A1-10U90 beam

4.3.6 Beam A1-15U90

The stiffness after rehabilitation decreased as shown in the Fig. 4.18. On the other hand, the ultimate load and ductility increased to 30%. The ultimate load before and after rehabilitation was 121 kN and 158.5 kN respectively.

The maximum displacement before and after rehabilitation were 4.32 mm and 5.94 mm, respectively. The first shear crack in concrete before and after rehabilitation was appeared at 121 kN and 143 kN respectively. This beam was showed high displacement when comparison with A1-10U90, this is due to the decrease in the effective depth.

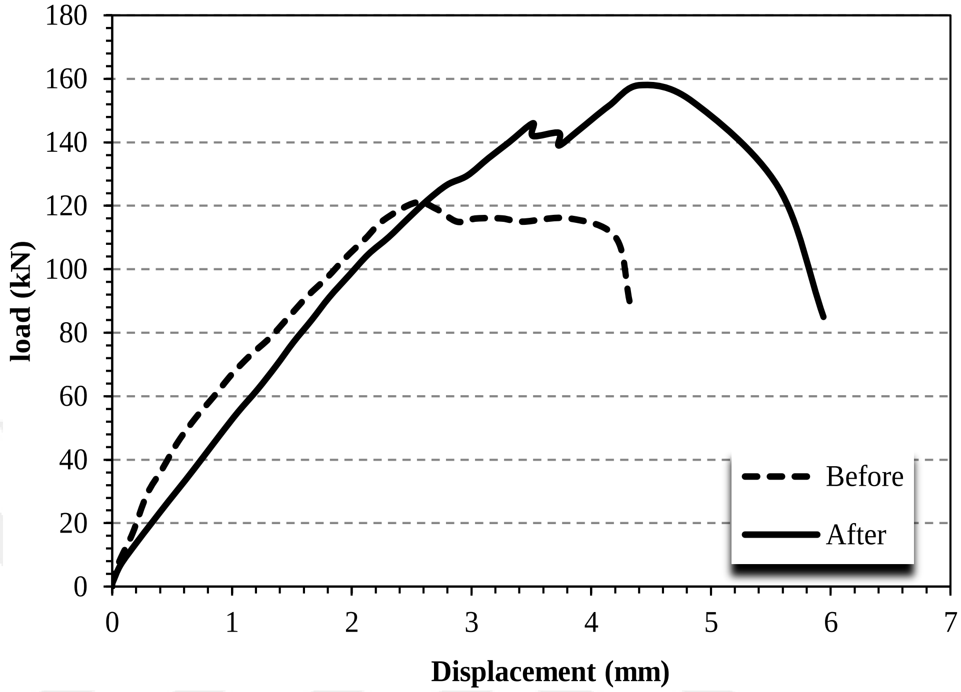


Figure 4.18 Load- displacement curve of the A1-15U90 beam

4.3.7 Beam A2-10U45

As seen from Fig. 4.19 the stiffness after rehabilitation decreased in the. On the other hand, the ultimate load and ductility increased to 39%. The ultimate load before and after rehabilitation was 115.3 kN and 160 kN respectively.

The maximum displacement before and after rehabilitation were 3.08 mm and 4.9 mm, respectively. The first shear crack in concrete before and after rehabilitation was appeared at 115.53 kN and 160 kN respectively.

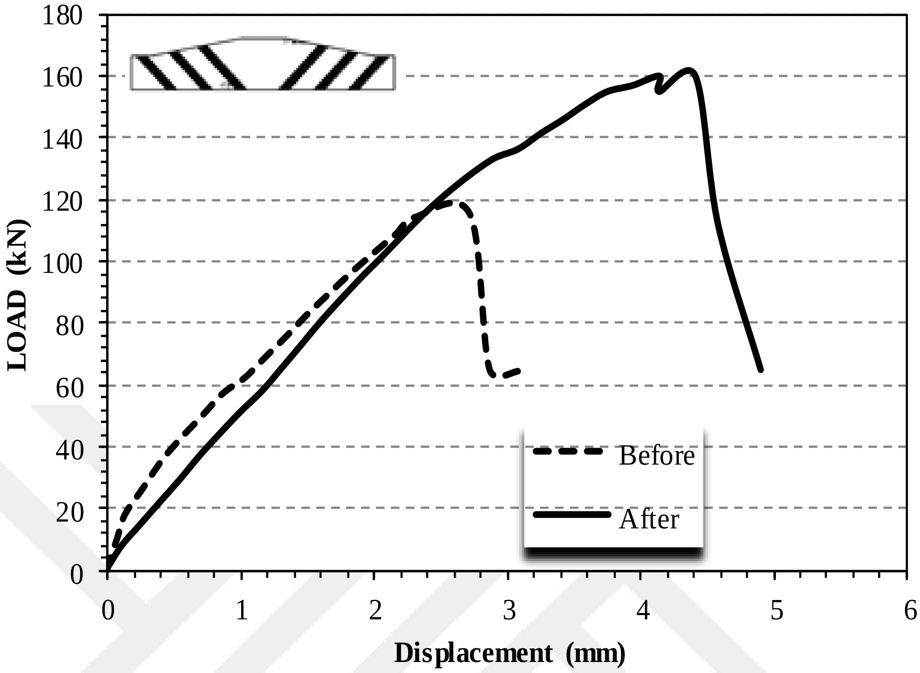


Figure 4.19 Load- displacement curve of the A2-10U45 beam

4.3.8 Beam A2-15U45

As seen from Fig. 4.20 the stiffness after rehabilitation decreased in the. On the other hand, the ultimate load and ductility increased to 49%. The ultimate load before and after rehabilitation was 113.2 kN and 164 kN respectively.

The maximum displacement before and after rehabilitation were 3.5 mm and 4.94 mm, respectively. The first shear crack in concrete before and after rehabilitation was appeared at 112 kN and 149 kN respectively. The percentage increase in the ultimate load for this haunched beam is more than other beams this is due to the high compressive strength where it was 60 MPa.

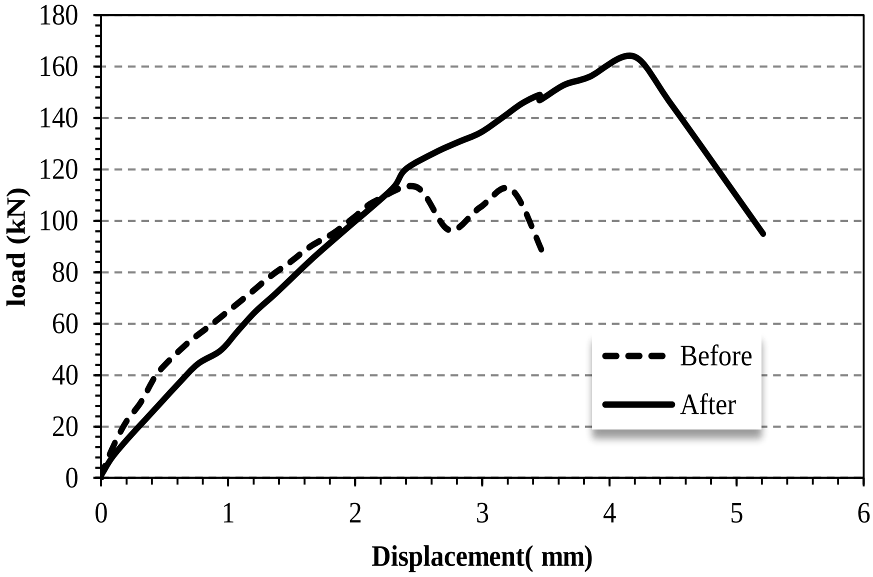


Figure 4.20 Load- displacement curve of the A2-15U45beam

4.3.9 Beam B1-10U90

The stiffness after rehabilitation decreased as shown in the Fig. 4.21. On the other hand, the ultimate load and ductility increased to 28%. The ultimate load before and after rehabilitation was 101.5 kN and 130.8 kN respectively.

The maximum displacement before and after rehabilitation were 6.2 mm and 6.357 mm, respectively. The first shear crack in concrete before and after rehabilitation was appeared at 64 kN and 115 kN respectively.

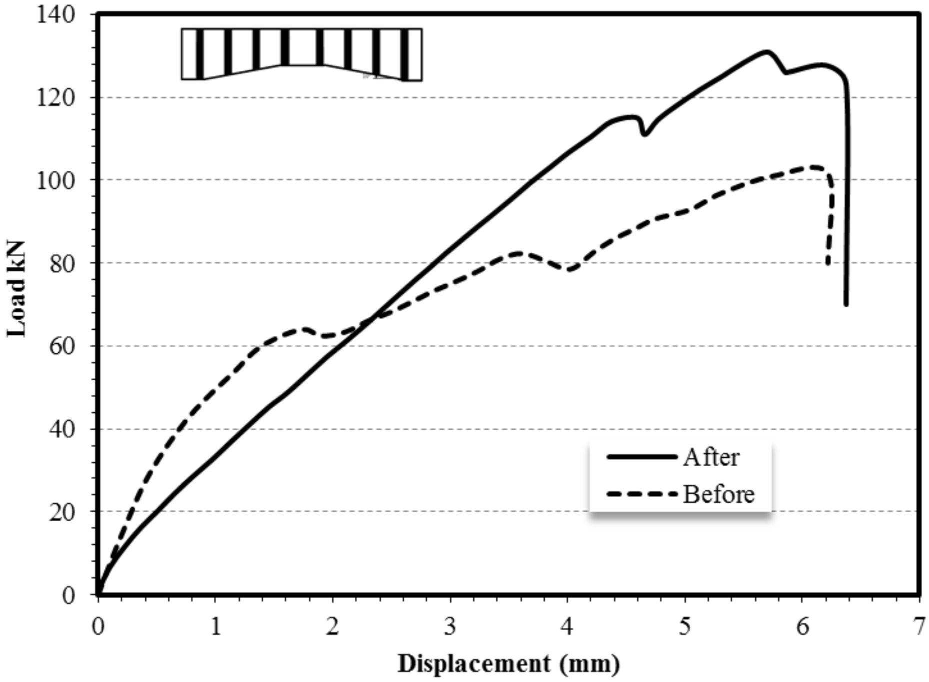


Figure 4.21 load- displacement curve of the B1-10U90 beam

4.3.10 Beam B1-15U90

As seen from Fig. 4.22 the stiffness after rehabilitation decreased. On the other hand, the ultimate load and ductility increased to 31%. The ultimate load before and after rehabilitation was 104 kN and 136 kN respectively.

The maximum displacement before and after rehabilitation were 5.32 mm and 5.38 mm, respectively. The first shear crack in concrete before and after rehabilitation was appeared at 74.2 kN and 104.7 kN respectively.

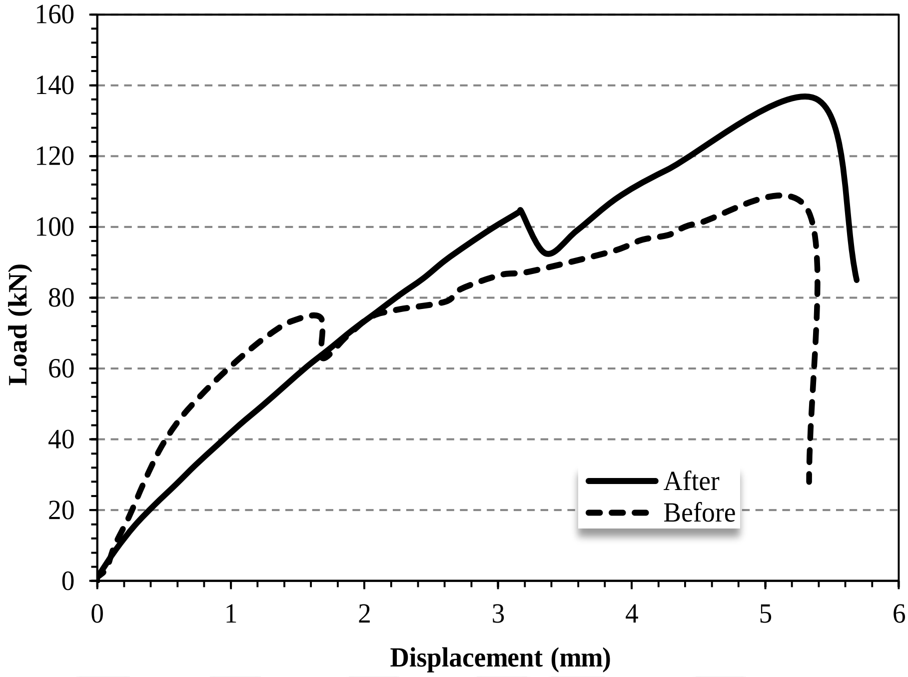


Figure 4.22 Load- displacement curve of the B1-15U90 beam

4.3.11 Beam B2-10U45

The stiffness after rehabilitation decreased as shown in the Fig. 4.23. On the other hand, the ultimate load and ductility increased to 15%. The ultimate load before and after rehabilitation was 95.7 kN and 110 kN respectively.

The maximum displacement before and after rehabilitation were 7.25 mm and 7.71 mm, respectively. The first shear crack in concrete before and after rehabilitation was appeared at 91.7 kN and 110 kN respectively.

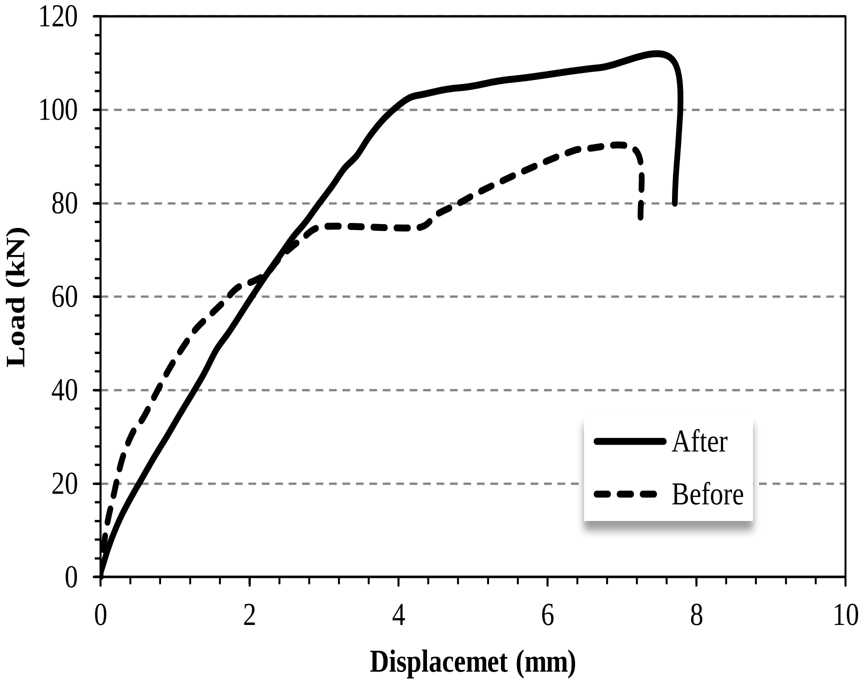


Figure 4.23 Load- displacement curve of the B2-10U45beam

4.3.12 Beam B2-15U45

As seen from Fig. 4.24 the stiffness after rehabilitation decreased. On the other hand, the ultimate load and ductility increased to 19%. The ultimate load before and after rehabilitation was 104 kN and 136 kN respectively.

The maximum displacement before and after rehabilitation were 8.36 mm and 10.64 mm, respectively. The first shear crack in concrete before and after rehabilitation was appeared at 78 kN and 105 kN respectively

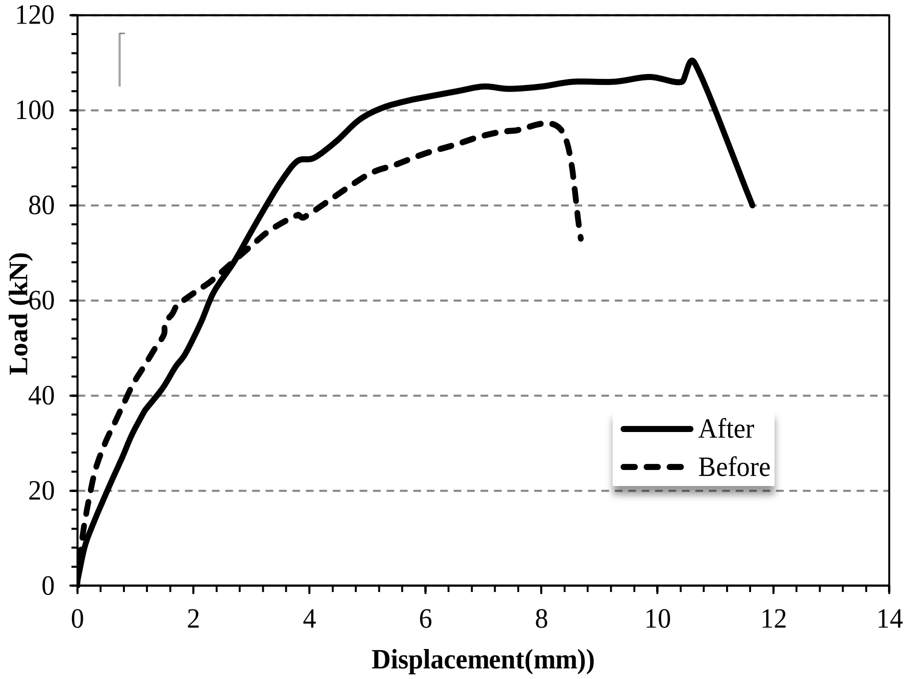
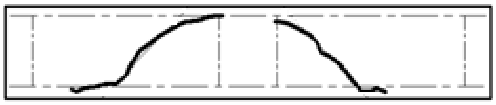


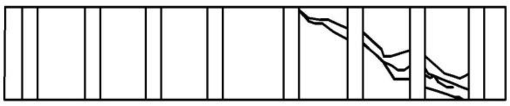
Figure 4.24 Load- displacement curve of the B2-15U45 beam

4.4 Shear crack behavior

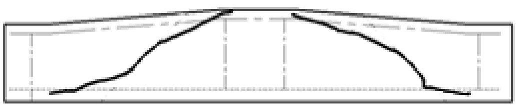
The control of cracks in concrete members is of significant importance, especially for shear cracks because this type of crack is sudden. From Fig. 4.37 it was noted that there is no definite explanation on the behavior of cracks after strengthening by CFRP strip.



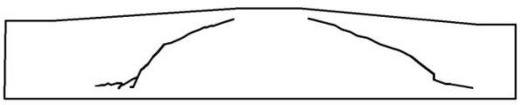
Rf1U90 Before rehabilitation



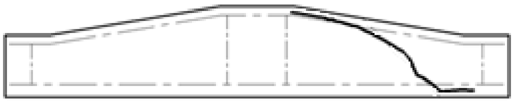
Rf1U90 After rehabilitation



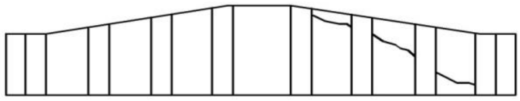
A1-5INJ Before injection



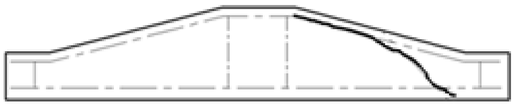
A1-5INJ After injection



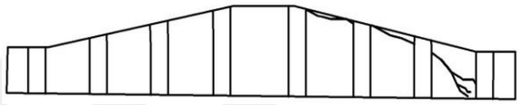
A1-10U90 Before rehabilitation



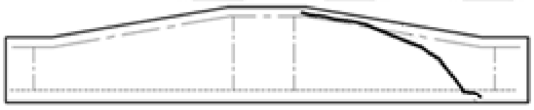
A1-10U90 After rehabilitation



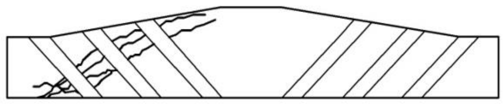
A1-15U90 Before rehabilitation



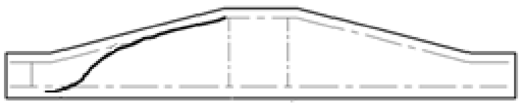
A1-15U90 After rehabilitation



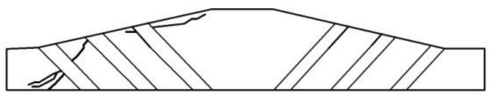
A2-10U45 Before rehabilitation



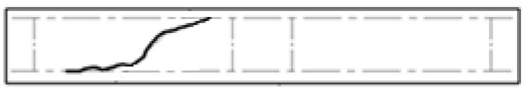
A2-10U45 After rehabilitation



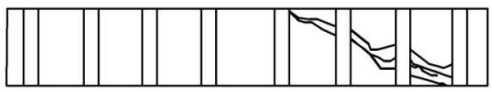
A2-15U45 Before rehabilitation



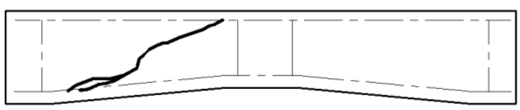
A2-15U45 After rehabilitation



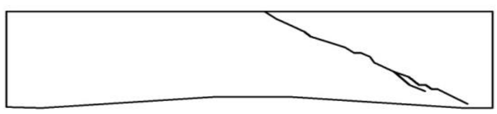
Rf2U90 Before rehabilitation



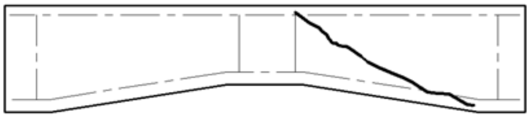
Rf2U90 After rehabilitation



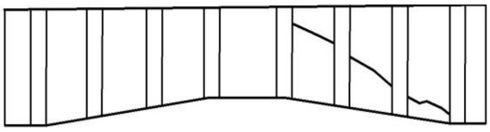
B1-5INJ Before injection



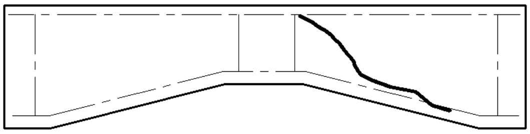
B1-5INJ After injection



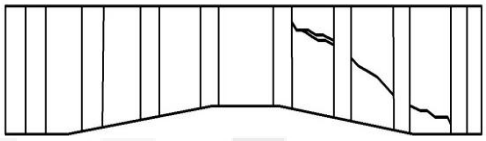
B1-10U90 Before rehabilitation



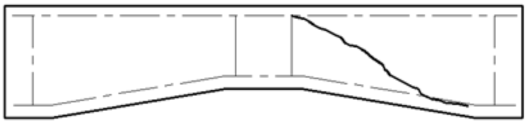
B1-10U90 After rehabilitation



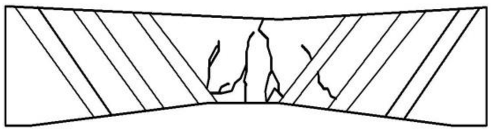
B1-15U90 Before rehabilitation



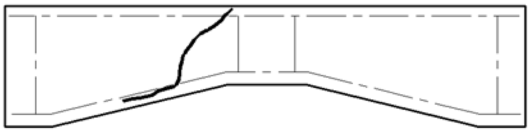
B1-15U90 After rehabilitation



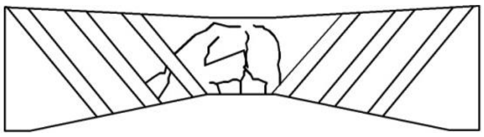
B2-10U45 Before rehabilitation



B2-10U45 After rehabilitation



B2-15U45 Before rehabilitation



B2-15U45 After rehabilitation

Figure. 4.25 Failure of diagonal shear cracks

CHAPTER 5

CONCLUSIONS

5. Overview

This thesis presents the shear behavior of damaged reinforced concrete haunched beams after repaired by injection epoxy and strengthened with unidirectional carbon fiber reinforced polymer strips (CFRP) at shear zone. The beams were designed without steel stirrups in order to fail in shear manner and after then rehabilitated. The experimental investigation was accomplished on twelve RC haunched beams of three different groups.

The main parameters examined in this study were:

- The effect of configuration of CFRP strips on the efficiency of strengthening,
- The effect of changing in the degree of slope at the shear zone of haunched beams on the location of critical section.
- The effect of injection epoxy on the upgrade of haunched beams after rehabilitation.

The results showed that the location of critical section depends on the inclination of haunched beams, where the location of critical section changes with increased inclination. On the other hand, the vertical CFRP strips showed to be more effective than the inclined strips to increase the capacity of the beam after rehabilitation.

5.1 Regarding Haunched Beams Belonging to Mode A The Following Conclusions Can be Withdrawn:

-The critical section location for all haunched beams are constant regardless the type of strengthening and compressive strength of concrete. It can be concluded that as inclination of haunhed beam increases, the location of critical section tend to become nearer to the point of support.

- The vertical strengthening is a more effective than inclined strengthening for this mode of haunched beams.

5.2 Regarding Haunched Beams Belonging to Mode B the Following Conclusions Can Be Withdrawn:

- The location of the critical section for this mode of haunched beam is the same as the location of the haunched beam mode (A).

- There is no noticeable difference in increase of capacity of beams at peak load for both types of vertical and inclined strengthening.

- The mode (A) of haunched beams showed a better performance regarding it increase of ultimate load after rehabilitation as compared to mode B haunched beams.

5.3 Recommendations for future research

Recent decades have seen the development and use of new technologies in the field of rehabilitation and strengthening of structural members, so it is necessary to conduct further studies and research on these technologies, including the use of polymer fibers.

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