EXPERIMENTAL INVESTIGATION ON FLEXURAL SHEAR CAPACITY OF BEAMS CAST IN TWO PHASES

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Bangladesh University of Engineering and Technology

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This thesis titled "**Experimental Investigation On Flexural Shear Capacity Of Beams Cast In Two Phases.**", submitted by Shamim Ara Bobby, Roll No. 1009042319F, Session: October 2009, has been accepted as satisfactory in partial fulfillment of the requirement for the degree of **Master of Science in Civil Engineering (Structural)** on 9th September, 2015.

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The thesis or any part of the thesis has not been submitted to any other university or educational institute for the award for any degree, except for publication.

(Shamim Ara Bobby)

DEDICATION

This Thesis is dedicated to my parents and teachers.

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At first, the author acknowledges the blessings of almighty, kindness, merciful, the universal king, Allah for enabling me to complete this thesis successfully.

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ABSTRACT

The aim of this research is to investigate on flexural shear capacity of RC Beams which are cast in two phases. Suitability of using epoxy as bonding agent under one point loading was investigated and compared with two phases of concrete joined without any bonding agent. To achieve the objectives, total thirty half scale specimens were constructed and tested. Ten of them, Group A, were constructed to obtain concrete strength 3ksi with and without using bonding agent. Similarly for Group B and C target strength were respectively 4ksi and 5ksi.

All the specimens were constructed in two phases except the control specimen of each group. At first phase, for three groups total three beams were fully cast and twenty seven beams were half cast. At second phase, after one day nine half cast beams (from each group three beams) were cast fully with and without using bonding agent. Same procedures were followed after three days and seven days. So, total twenty seven half cast beams were fully cast at phase two.

Specimens were tested under incremental one point loading. The load was applied by Universal Testing Machine at midspan of the beam. A constant strain-controlled loading was applied with the movement rate of the platform being 5mm/min. Time vs load was continuously monitored and data was saved automatically. Deflections of the beams were also monitored with a video extensometer and time versus deflection data was stored continuously. These data are then combined to prepare load versus deflection curves which are the basis of analysis for beam strength.

Load-deflection curves of beams were plotted to study their behavior under one point loading. Behavior of the beams such as ultimate load, deflection and failure pattern are compared with that of fully cast beams and beams cast in two phases by using bonding agents. By means of constructing beams in two phases with using bonding agent 50MPa Epoxy and 75MPa Epoxy, the flexural shear capacity of the beams can be increased. The beams cast in two phases with using bonding agent 75MPa Epoxy have higher values of ultimate load than using 50MPa Epoxy. So, 75MPa Epoxy is more effective as a bonding agent. The effect of age of concrete after which the second phase of construction is started on ultimate load with using bonding agent 50MPa Epoxy and 75MPa Epoxy is very less. So, the effect of applying epoxy as a bonding agent with the variation of days is insignificant once the concrete is set.

For concrete strength of 3ksi, the increase in ultimate load for beams cast in two phases after one day respectively by using 75MPa Epoxy and using 50MPa Epoxy are almost 18.4% and 3.9% than compared to that of fully cast beam. For concrete strength of 4ksi, it was 11.72% for beams cast in two phases after one day by using 75MPa Epoxy than compared to that of fully cast beam but for the beam cast in two phases after one day by using 50MPa Epoxy, ultimate load did not increase than the fully cast beam. Again for concrete strength of 5ksi, the ultimate load increases almost 7.82% and 5.97% respectively for beams cast in two phases after one day by using 75MPa Epoxy and using 50MPa Epoxy than compared to that of fully cast beam. Similarly for concrete strength of 5ksi, ultimate load increases almost 8.2% and 4.2% respectively for beams cast in two phases after three days by using 75MPa Epoxy and using 50MPa Epoxy than compared to that of fully cast beam. Again for concrete strength of 5ksi, increase in ultimate load for beams cast in two phases after seven days respectively by using 75MPa Epoxy and using 50MPa Epoxy are almost 6.5% and 1.2% than compared to that of fully cast beam.

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CHAPTER 1
INTRODUCTION

1.1 GENERAL

This thesis is focusing on how the flexure strength is affected when the beam is cast in two phases. The first chapter of this thesis presents the study background and problem statement, defines the objectives of this study, overviews of the scope to satisfy all the objectives, and outlines the thesis structure.

1.2 BACKGROUND AND PROBLEM STATEMENT

Beams are the horizontal membranes of a structure that are responsible for carrying all vertical loads and transmitting the pressure towards the columns. It has similarity in form with the column except that it is designed to be horizontal in arrangement. The horizontal arrangement of beams predicts its job to hold and protect the entire structure from vertical collapse. Without them, all other members will fall to the ground.

Generally reinforced concrete (RC) beams fail in two ways: flexural failure and diagonal tension (shear) failure. In nature the shear failure is more sudden and brittle [1] .It gives no warning prior to failure except for large cracks and it is more dangerous than flexural failure.

It is important that the structural engineer be able to predict with satisfactory accuracy the ultimate strength of a structural member. By making this strength larger by an appropriate amount than the largest loads which can be expected during the lifetime of the structure, an adequate margin of safety is assured .Actual inspection of many concrete stress-strain curves which have been published, show that the geometrical shape of the stress distribution is quite varied and depends on a number of factors such as cylinder strength, the rate, and duration of loading.

In the design of reinforced concrete flexural members, to apply the higher resistance factor φ of 0.9, a member should exhibit desirable behavior. At service load, small deflections and minimal cracking are desired. At higher loads, however, the member should exhibit large deflections and/or excessive cracking to provide warning before reaching nominal strength. Both deflection and cracking are primarily a function of steel strain near the tension face of the member.

Shear in concrete has always been a challenge for researchers to understand. The shear behavior and shear strength of reinforced concrete elements has been studied for more than 100 years. The problem is that it is difficult to predict accurately the shear failure e.g. in simple concrete beams, more properly called “diagonal tension failure”. This is a problem that has generated a lot of debates for researchers and engineers for many years. The past years, models and methods have been suggested to describe and to predict shear failure and shear strength. [2, 3]

Epoxies are frequently used as repair materials because they bond well to almost all materials, cure rapidly, attain high strengths and exhibit good chemical resistance. Applications include use in bonding concrete (hardened to hardened, and hardened to fresh), and in patches, overlays and protective coatings. The addition of resin was found to improve the strength, ductility, durability, and chemical resistance of concrete. In a recent experimental program,[4] it was found that replacing 20% of the cement with epoxy increased the shear strength of longitudinally reinforced beams by up to 11%.

In spite of using epoxy as replacing cement, it can be used as bonding agent. In practical situation there are many cases for buildings and other structures where beams cannot be cast at a time. For example if there is a case where two slabs at different levels are joined with a beam, then this beam cannot be cast at a time. It is cast in two phases. Half of the beam is cast with lower slab then another half is cast with upper slab. To better understand the bonding mechanism at the interface between two parts of beams, it is essential to measure bond strength at the interface**.** The development of concrete-to-concrete bond is important for performance of reinforced concrete beam strength.

The main purpose of this experimental work is to investigate and understand the performance of concrete beam cast in two phases at different times and with different strengths by using concrete and epoxy bonding agent.

1.3 RESEARCH OBJECTIVES AND AIM OF THE STUDY

The overall aim of this research is to investigate and gather knowledge on RC Beams flexural shear capacity which were cast in two phases and to investigate the suitability of using epoxy as bonding agent and without using any bonding agent in strengthening of RC beams under one point loading .

The main objectives of this research are:

- To study the structural behavior of RC beams cast in two phase
- To evaluate and analyze the flexural shear capacity of RC beams cast in two phases at various age of concrete (one day, three days and seven days).
- To investigate the suitability of using epoxy as a bonding agent and without using any bonding agent in the casting procedure.
- To investigate the effect of shear capacity of RC beams prepared using different concrete strength (3ksi, 4ksi & 5ksi).
- To research the performance of concrete beams cast in two phases with concrete and epoxy bonding agent under one point loading and compare the performances of all the beams.

1.4 METHODOLOGY

To investigate the behavior of reinforced concrete beams cast in two phases with concrete and epoxy, one point loading will be applied to test the beams. Half scale RC model beams will be prepared with 5ft span. The cross section of beams will be 6 inch x 8 inch for all specimens prepared. Then the following parameters will be considered for the study:

1. Different concrete strength (3ksi, 4ksi & 5ksi).
2. Various age of concrete (one day, three days and seven days)
3. Two different types of epoxy grade.

A total of 30 (thirty) concrete beams were constructed for the above study:

1. Ten beams for concrete strength 3ksi were constructed, one of which was reference specimens and fully cast at the first day and the another three were cast in two phases after one day, three days and seven days and the rest three were also cast in two phases after applying two different grades of epoxy on the half cast beams top surface after one day, three days and seven days.

2. Ten beams for concrete strength 4ksi were constructed, one of which was reference specimens and fully cast at the first day and the another three were cast in two phases after one day, three days and seven days and the rest three were also cast in two phases after applying two different grades of epoxy on the half cast beam top surface after one day, three days and seven days.

3. Ten beams for concrete strength 5ksi were constructed, one of which was reference specimens and fully cast at the first day and the another three were cast in two phases after one day, three days and seven days and the rest three were also cast in two phases after applying two different grades of epoxy on the half cast beam top surface after one day, three days and seven days.

Finally, the load deflection curves of beams cast in two phases with concrete and epoxy are compared.

1.5 SCOPE OF THE WORK

This study considered only the half scale RC beams. All the samples had similar dimensions and they were constructed using stone chips. The outcome of this study may be helpful in deciding appropriate alternative methods to improve the flexural shear capacity of beams when the beams cannot be cast at a time.

1.6 THESIS ORGANIZATION

To achieve the goals that are mentioned here above an organized step by step procedure has to be used the thesis comprises of five chapters. The content of the chapters summarizes below:

Chapter 1: The introductory chapter describes the background and objective of this study.

Chapter 2: This chapter implies the beam behavior under flexural strength, different types of failure modes, and design criteria for flexural and shear strength of beams. Different factors that affect the bond strength when beams cast in two phases are also discussed briefly in this chapter.

Chapter 3: Various types of materials such as cement, coarse aggregate, fine aggregate, epoxy bonding agent etc. were used for the preparation of test specimen. Properties and relevant information about the material are discussed in chapter 3. Also the selection and preparations of the models are discussed deliberately in Chapter three. This study needed deliberate experimental preparation and set up which are also discussed in this chapter.

Chapter 4: The chapter 4 covers the most significant part of the thesis – results of the experiments and their analysis. This chapter represents the findings of the experimental results and provides explanation for the behavior deserved.

Chapter 5: This chapter summarizes the overall research work and presents the conclusion of the study and presents the final conclusions based on the outcome of the experimental study and tracks out some recommendations for future studies.

CHAPTER 2

LITERATURE REVIEW

2.1 INTRODUCTION

This chapter provides an introduction to the structural member beam, the beam behavior under flexural strength and failure modes.

2.2 BEAM

Beams are usually long, straight prismatic members, as shown in the photo on the previous page. Steel and aluminum beams play an important part in both structural and mechanical engineering. Timber beams are widely used in home construction (Fig. 2.1a). In most cases, the loads are perpendicular to the axis of the beam. Such a transverse loading causes only bending and shear in the beam. When the loads are not at a right angle to the beam, they also produce axial forces in the beam.

The transverse loading of a beam may consist of concentrated loads P1, P2, . . . , expressed in newtons, pounds, or their multiples, kilonewton and kips (Fig. 2.1a), of a distributed load w, expressed in N/m, kN/m, lb/ft, or kips/ft (Fig. 2.1b), or of a combination of both. When the load w per unit length has a constant value over part of the beam (as between A and B in Fig. 2.1b), the load is said to be uniformly distributed over that part of the beam.

Figure 2.1: Beam

2.3 DESIGN CRITERIA

One of the main objectives of the design of reinforced concrete beams is safety. Sudden failure due to shear low strength is not desirable mode of failure. The reinforced concrete beams are designed primarily for flexural strength and shear strength. Beams are structural members used to carry loads primarily by internal moments and shears. In the design of a reinforced concrete member, flexure is usually considered first, leading to the size of the section and the arrangement of reinforcement to provide the necessary resistance for moments. For safety reasons, limits are placed on the amounts of flexural reinforcement to ensure ductile type of failure. Beams are then designed for shear. Since shear failure is frequently sudden with little or no advanced warning, the design for shear must ensure that the shear strength for every member in the structure exceeds the flexural strength. The shear failure mechanism varies depending upon the cross-sectional dimensions, the geometry, the types of loading, and the properties of the member.

2.3.1 Basis of Current Design Practice

Based on current design practice, the design of a flexural member should take into account the overall behavior of the member throughout the service range and up to the nominal capacity of the member. Beginning with the ACI 318-63[5] Code, flexural members were required to have reinforcement ratios not greater than 75% of the balanced reinforcement ratio ρb. By 1993, this criterion had been in use for 30 years, and the behavior of flexural members was judged to be satisfactory. The current criterion based on tensile strain in the reinforcement was selected to provide similar behavior to that experienced under the $0.75\rho b$ criterion.

2.3.2. Past and Present ACI 318 Code Requirements

Ultimate strength design (now called strength design) was introduced in the ACI 318 Code in 1963[5] for flexural members, the maximum reinforcement ratio ρ was limited to 0.75 of the balanced reinforcement ratio ρb. The purpose of this requirement was to ensure that the member would be under-reinforced such that the reinforcement would yield before the concrete reached its limiting strain of 0.003 prior to failure. This requirement remained essentially in effect from 1963 to 1999.

In the 1995 ACI 318 Code [6] an alternative requirement was introduced. Rather than limiting the maximum reinforcement ratio ρ, a minimum strain level in the reinforcement at nominal strength was required for the use of the high φ factor of 0.9 for flexure. A tension-controlled section was defined as a cross section in which the net tensile strain in the extreme tension steel at nominal strength was greater than or equal to 0.005. For tension-controlled sections, a φ factor of 0.9 was used. If the steel strain at nominal strength was less than 0.005, a reduced φ factor was used to account for the less desirable behavior of these sections. Compression-controlled sections are defined as having steel strains at nominal strength at or below the yield strain of the reinforcement. For compression-controlled sections, the φ factor for compression was used. For sections with steel strains between the aforementioned two limits, the φ factor was determined by linear interpolation between the φ factor for compression and the φ factor for tension. In the 2002 ACI 318 Code [7] these requirements were moved to the main body of the code, replacing the former limit on ρ for flexural members.

The background information on the development of these limits has been provided by Mast[8].Whereas the tension controlled strain limit of 0.005 was developed from studies of the behavior of members reinforced with Grade 60 (400 MPa) steel, it was also permitted to be used with Grade 75 (520 MPa) steel reinforcement

2.4 FLEXURE STRENGTH

It is of interest in structural practice to calculate stresses and deformations which occur in a structure in service under design load. For reinforced concrete beams this can be done by the methods just presented, which assume elastic behavior of both materials. It is equally, if not more, important that the structural engineer be able to predict with satisfactory accuracy the ultimate strength of a structural member. By making this strength larger by an appropriate amount than the largest loads which can be expected during the lifetime of the structure, an adequate margin of safety is assured. Until recent times, methods based on elastic analysis like those just presented have been used for this purpose. It is clear, however, that at or near the ultimate load, stresses are no longer proportional to strains.

Actual inspection of many concrete stress-strain curves which have been published, show that the geometrical shape of the stress distribution is quite varied and depend on a number of factors such as cylinder strength, the rate, and duration of loading. Below is a typical stress distribution at the ultimate load.

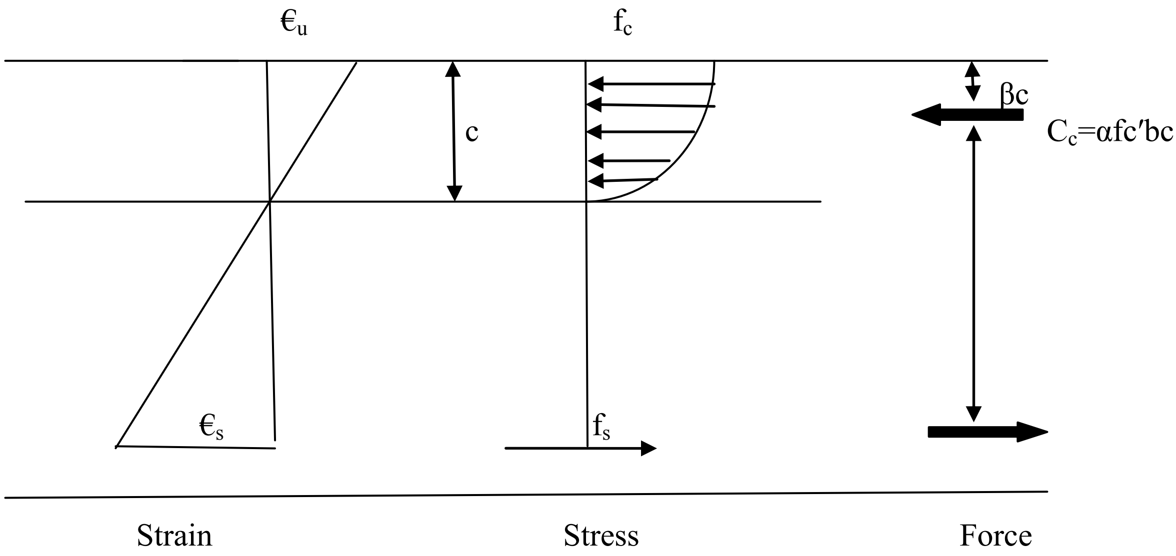

Figure: 2.2 Strain, Stress, and Force Diagrams

2.4.1 Two Different Types of Failure

There are two possible ways that a reinforced beam can fail:

- Beam will fail by tension of steel

Moderate amount of reinforcement is used. Steel yields suddenly and stretches a large amount, tension cracks become visible and widen and propagate upward (Ductile Failure)

- Compression failure of concrete

Large amount of reinforcement is used. Concrete fails by crushing when strains become so large (0.003 to 0.004). Failure is sudden, an almost explosive nature and occur with no warning (Brittle Failure).

In a rectangular beam the area that is in compression is *bc*, and the total compression force on this area can be expressed as $C = f_{av}bc$, where f_{av} is the average compression stress on the area *bc*. Evidently, the average compression stress that can be developed before failure occurs becomes larger the higher the cylinder strength f_c' of the particular concrete. Let

$$\alpha = \frac{f_{av}}{f_{c'}} \qquad (2.1)$$

Then

$Cc = f_{av}bc = \alpha f_c'bc$ --- (2.2)

Compression force is applied at βc distance from top fiber, and *c* is the distance of the N.A. from top fiber.

Based on research we have:

$\alpha = 0.72 - \frac{f_{c'} - 4{,}000}{1000} \times 0.04 \; and \; 0.56 < \alpha < 0.72$ -- (2.3)

$\beta = 0.425 - \frac{f_{c'} - 4{,}000}{1000} \times 0.025 \; and \; 0.324 < \beta < 0.425$ ------------------------------------ (2.4)

Forces

From equilibrium we have $C_c = T$ or

$\alpha f_c'bc = A_sf$ --- (2.5)

$M = TZ = A_s f_s (d - \beta c)$ -- (2.6)

or

$M = C_cZ = \alpha f_c'bc\,(d - \beta c)$ -- (2.7)

Tension Failure

$f_s = f_y$ *steel yielding*

from eq. (2.5) we have

$c = \frac{A_s f_y}{\alpha b f_{c'}} \times \frac{d}{d} = \frac{A_s f_y d}{bd\,\alpha f_{c'}} = \rho \frac{f_y}{f_{c'}} \frac{d}{\alpha}$ --- (2.8)

Substitute *c* from Eq. (2.8) in Eq. (2.6)

$M_n = A_s f_y (d - \rho \frac{\beta}{\alpha} \frac{f_y}{f_{c'}} d)$ -- (2.9)

With the specific, experimentally obtained values for α and β we always have

$\frac{\beta}{\alpha} = 0.59$ for $f_c' = 4{,}000$ *psi or any other strength*

Therefore, Eq. (2.9) simplifies as

$M_n = A_s f_y (d - 0.59\rho \frac{f_y}{f_{c'}} d)$ --- (2.10)

Or

$M_n = \rho b d^2 f_y (1 - 0.59\rho \frac{f_y}{f_{c'}})$ --- (2.11)

Where M_n = nominal moment capacity.

Compression Failure

In this case, the criterion is that the compression strain in the concrete becomes $\varepsilon_u = 0.003$, as previously discussed. The steel stress f_s, not having reached the yield point, is proportional to the steel strain, ε_s; i.e. according to Hooke's law:

$\varepsilon_u = 0.003$ (ACI 10.2.3) and $f_s < f_y$ --- (2.12)

$f_s = E_s\ \epsilon_s$ *Hooks law, since* $f_s < f_y$ -- (2.13)

From similar triangles we have

$$\frac{\epsilon_u}{c} = \frac{\epsilon_s}{d-c} \rightarrow \epsilon_s = \frac{d-c}{c} \quad (2.14)$$

Substitute Eq. (14) in Eq. (12)

$$f_s = E_s\,\epsilon_s = E_s\,\epsilon_s\frac{d-c}{c} < f_y \quad (2.15)$$

From Eq. (5) we have

$$\alpha f_s' bc = A_s\, f_s = A_s\, E_s \epsilon_u \frac{d-c}{c} \quad (2.16)$$

Using Eq. (2.16) solve for c, and then find M_n, the nominal moment capacity.

Balance Steel Ratio

We like to have tension failure, because it gives us warning, versus compression failure which is sudden. Therefore, we want to keep the amount of steel reinforcement in such manner that the failure will be of tension type.

Balanced steel ratio, ρ_b represents the amount of reinforcement necessary to make a beam fail by crushing of concrete at the same load that causes the steel to yield. This means that neutral axis must be located at the load which the steel starts yielding and concrete starts reaching its compressive strain of $\varepsilon_u = 0.003$.

$$c^b = \frac{\epsilon_u}{\epsilon_y + \epsilon_u} d$$

T = C $\rightarrow A_s^b f_y = \alpha f_c' bc^b$

$$A_s^b f_y = \rho_b bdf_y = \alpha f_c' b\frac{\epsilon_u}{\epsilon_u + \epsilon_y} d$$

$$\overline{\rho_b} = \alpha\frac{f_c'}{f_y} = \frac{\epsilon_u}{\epsilon_u + \epsilon_y}$$

2.4.2 Load Deflection Behavior of Reinforced Concrete Beams

Fig. 2.3 shows the relation between load and concrete strain for the moderately reinforced concrete beam.

The first part of the curve is a straight line where the deflection is directly proportional to the load and where, once the load is removed, the beam will return to its original state; i.e. it retains its elasticity. With increasing load, a limit point of proportionality is reached after which the increase in amount of deflection is greater than (i.e. no longer proportional with) the increase in load; but elasticity is still retained until an elastic limit is reached. If stress is further increased, the material loses elasticity and becomes plastic (i.e when the load is removed the deformation caused by deflection will be more or less permanent). At the point of maximum load, ultimate load or ultimate strength, the material begins to yield and will fracture unless load is substantially reduced.

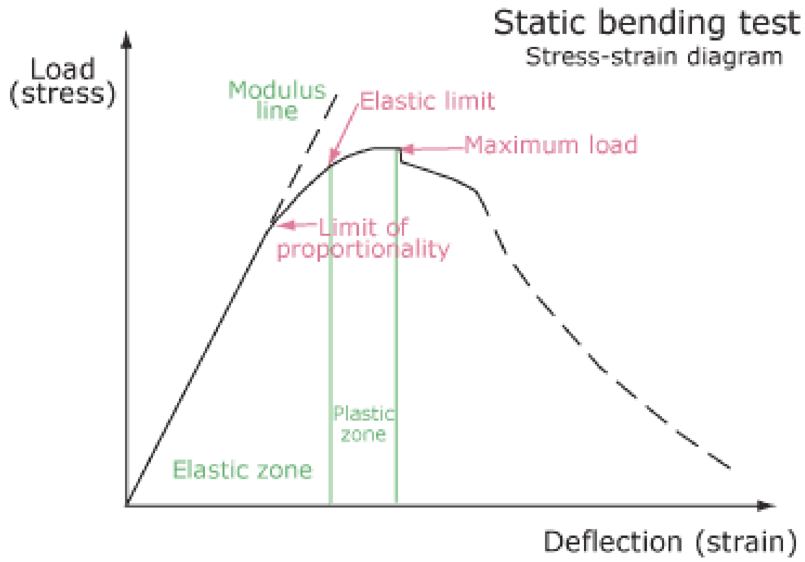

Figure: 2.3 Typical Stress-Strain Diagram

2.5 SHEAR STRENGTH

Reinforced concrete beams must have an adequate safety margin against bending and shear forces, so that it will perform effectively during its service life. At the ultimate limit state, the combined effects of bending and shear may exceed the resistance capacity of the beam causing tensile cracks. The shear failure is difficult to predict accurately despite extensive experimental research. Retrofitting of reinforced concrete beams with multiple shear cracks is not considered an option [9].

Diagonal cracks are the main mode of shear failure in reinforced concrete beams located near the supports and caused by excess applied shear forces. Beams fail immediately upon formation of critical cracks in the high-shear region near the beam supports. Whenever the value of actual shear stress exceeds the permissible shear stress of the concrete used, the shear reinforcement must be provided. The purpose of shear reinforcement is to prevent failure in shear, and to increase beam ductility and subsequently the likelihood of sudden failure will be reduced.

Normally, the inclined shear cracks start at the middle height of the beam near support at approximately 450 and extend toward the compression zone. Any form of effectively anchored reinforcement that intersects these diagonal cracks will be able to resist the shear forces to a certain extent. In practice, shear reinforcement is provided in three forms; stirrups, inclined bent-up bars and combination system of stirrups and bent-up bars. In reinforced concrete building construction, stirrups are most commonly used as shear reinforcement, for their simplicity in fabrication and installation. Stirrups are spaced closely at the high shear region. Congestion near the support of the reinforced concrete beams due to the presence of the closely spaced stirrups increase the cost and time required for installation. Bent up bars are also used along with stirrups in the past to carry some of the applied shear forces. In case where all the tensile reinforcement is not needed to resist bending moment, some of the tensile bars where bent-up in the region of high shear to form the inclined legs of shear reinforcement. The use of bent-up bars is not preferred nowadays. Due to difficulties in construction, bent-up bars are rarely used. In beams with small number of bars provided, the bent-up bar system is not suitable due to insufficient amount of straight bars left to be extended to the support as required by the code of practice.

Shear analysis procedure

The shear analysis procedure involves the following:

- Cracking the shear strength in an existing member
- Verifying that the various ACI code requirements have been satisfied and met.

2.5.1 Shear Reinforcement Design Requirement

ACI Code Provisions for Shear Design:

– According to the ACI Code 318 , the design of beams for shear is based on the following relation:

$$\varphi V_n \geq V_u \quad (2.17)$$

Where

φ = strength reduction factor (= 0.85 for shear)

$V_n = V_c + V_s$

V_s = nominal shear strength provided by reinforcement

$$= \frac{A_v f_y d}{s} \text{ for inclined stirrups} \quad (2.18)$$

– Symbols

A_v = total cross-sectional area of web reinforcement within a distance s, for single loop stirrups, $A_v = 2A_s$

A_s = cross-sectional area of the stirrup bar (in^2) b_w = web width = b for rectangular section (in.)

s = center-to-center spacing of shear reinforcement in a direction parallel to the longitudinal reinforcement (in.)

f_y = yield strength of web reinforcement steel (psi)

– For inclined stirrups, the expression for nominal shear strength provided by reinforcement is

$$V_s = \frac{A_v f_y d\,(\sin\alpha + \cos\alpha)}{s} \quad (2.19)$$

– For $\alpha = 45^0$, the expression takes the form

$$V_s = \frac{1.414\,A_v f_y d}{s} \quad (2.20)$$

– The design for stirrup spacing can be determined from

$$\text{Required } S = \frac{A_v f_y d}{V_s} \quad (2.21)$$

$$\text{And required } S = \frac{1.414\,A_v f_y d}{V_s} \quad (2.22)$$

$$V_s = \frac{V_u - \emptyset V_c}{\emptyset} \quad (2.23)$$

– According to the ACI Code, the maximum spacing of stirrups is the smallest value of

$$S_{max} = \frac{A_v f_y}{50 b_w}$$
$$S_{max} = \frac{d}{2}$$
$s_{max} = 24$ in.

If V_s exceeds $4\sqrt{f_c{}'}b_w d$, the maximum spacing must not exceed $d/4$ or 12 in.
– It is not usually good practice to space vertical stirrups closer than 4 in.

– It is generally economical and practical to compute spacing required at several sections and to place stirrups accordingly in groups of varying spacing. Spacing values should be made to not less than 1-in. increments.
– Critical Section
The maximum shear usually occurs in this section near the support.
For stirrup design, the section located a distance *d* from the face of the support is called the "Critical section"
Sections located less than a distance *d* from the face of the support may be designed for the same V_u as that of the critical section.
The stirrup spacing should be constant from the critical section back to the face the support based on the spacing requirements at the critical section. The first stirrup should be placed at a maximum distance of $s/2$ from the face of the support, where s equals the immediately adjacent required spacing (a distance of 2 in.) is commonly used.

2.5.2 Stirrup Design Procedure

The design of stirrups for shear reinforcement involves the determination of stirrup size and spacing pattern.

A general procedure is as follows:

1. Determine the shear values based on clear span and draw a shear diagram for V_u.
2. Determine if stirrups are required.
3. Determine the length of span over which stirrups are required.
4. On the V_u diagram, determine the area representing "required φ V_s." This will display the required strength of the stirrups to be provided.
5. Select the size of the stirrups. Find the spacing required at the critical section (a distance *d* from the face of the support.
6. Establish the ACI Code maximum spacing requirements.
7. Determine the spacing requirements based on shear strength to be furnished by web reinforcing.
8. Establish the spacing pattern and show detailed sketches.

2.5.3 Beam Shear Failure Modes

Three distinct modes of shear failure are observed, which describe the manner in which concrete fails:
- Diagonal tension failure
- Shear compression failure
- Shear tension failure

Diagonal Tension Failure

This type of failure is usually a flexure-shear crack. The diagonal crack starts from the last flexural crack at mid span, where it follows direction of the bond reinforcing steel and the concrete at the support. After that, few more diagonal cracks develop with further load, the tension crack will extend gradually until it reaches its critical point where it will fail without warning. This type of shear failure is always in the shear-span when the a/d ratio is in the range of 2.5 to 6. Such beams fail either in shear or in flexure [10, 11, and 12].

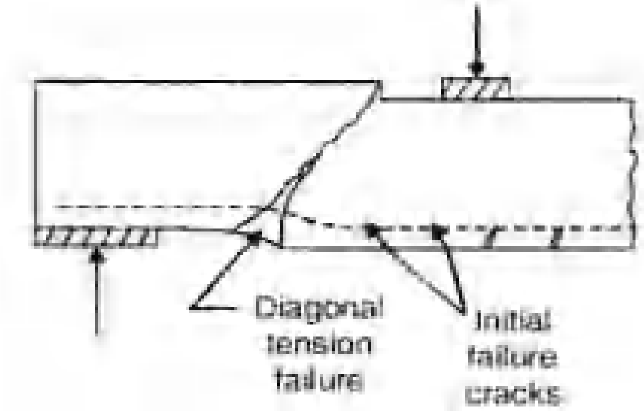

Figure: 2.4 Diagonal Tension Failure

Shear Compression Failure

This type of failure is common in short beams with a/d ratio between 1 and 2.5. It's called a web shear crack, it's crushing the concrete in the compression zone due to vertical compressive stresses developed in the vicinity of the load [10, 12].

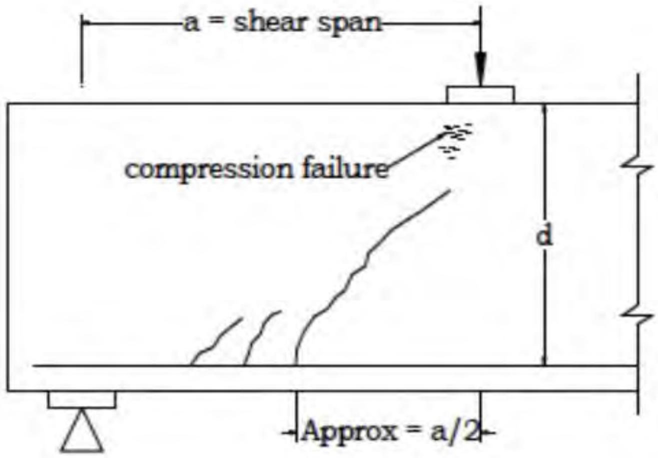

Figure: 2.5. Shear Compression Failure.

Shear Tension Failure

This type of failure is also common in short beams and it is similar to diagonal tension failure. First we can see a shear crack that is similar to the diagonal crack that goes through the beam; the crack extends toward the longitudinal reinforcement and then propagates along the reinforcement those results in the failure of the beam [13].

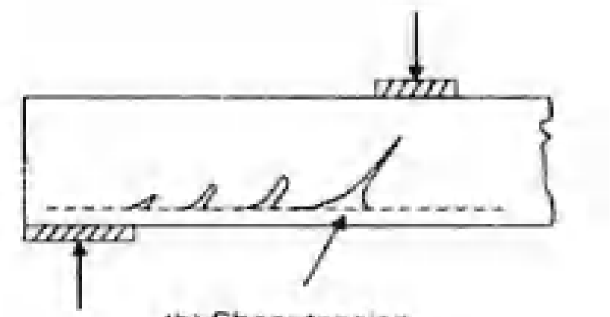

Figure: 2.6 Shear Tension Failure

2.6. BOND STRENGTH BETWEEN OLD CONCRETE AND NEW CONCRETE

Knutson [14] found that the bond strength between the old and new concrete is affected by the surface texture and moisture conditions of existing concrete and bonding agents. The surface of the old concrete should be cleared of other factors. Milling and shot blasting are usually used to clean and roughen the surface of the substrate. Several controversies still exist on the effects of bonding agents and moisture condition of substrate. Using the results of seven site survey projects, Gillette [15] indicated that a better bond may be obtained by applying the grout and brushing it into the base pavement surface with brooms rather than applying it pneumatically.

The surveys showed that relatively thin or thick concrete overlays will perform adequately if proper surface preparation and construction procedures are followed. It means that bonding agents increase bond strength of both layers. By covering laboratory bond tests, experimental field projects, and survey of projects in use, Felt [16] found that the two main factors governing bond were: (1) the strength and integrity of the old base concrete, and (2) the cleanness of old surface, a good bond may be achieved without using a grout layer. However, he pointed out that the chance of increasing bond strength can be improved by using grout and best bond could be obtained with dry and grouted base concrete.

By determined specimen size, maximum aggregate size of repair materials, types of repair materials, interface roughness and age at loading as variables, Momayez [17] pointed out that the bond strength was improved with surface roughness, silica fume content, and age at testing. Larger specimens resulted in lower bond strength. Bi-surface test was found to be a reliable test for determining bond between existing concrete and repair materials.

2.6.1 Factors Affecting Interfacial Shear Bond Strength

This section provides a discussion on various factors affecting interfacial shear bond strength between old and new concrete.

Moisture Condition

It is not clearly known how the moisture condition affects the bond strength. Gillette [18] founded that thin watery grout or free water left standing on the surface of the base pavement tends to weaken the bond strength. However, Pigeon and Saucier [19] concluded from their tests that moisture condition does not affect bond strength. According to Austin [20], SSD condition is most favorable for higher bond strength. Chorinsky [21] reported that, if an unmodified cement mortar is applied to a dry concrete surface, part of the mixing water will be sucked into the concrete before any soluble and reactive components in the cement paste are formed. The reaction of calcium hydroxide and soluble silicates is restricted to the outer areas of aggregates. On the other hand, the surface of substrate concrete is saturated with water before the mortar is applied. The capillary pores are closed for penetration of hydration products out of the cement paste. The excess water from the capillaries will raise the w/c ratio in the boundary of the fresh mortar and lower its mechanical properties. The bond strength is weakened in the surface. Austin [20] concluded that, the effect of moisture condition on crystalline bonding of the cement is similar to that of unmodified cementitious mortar. Austin's early work [22] with patch test also suggested that an SSD condition gave higher bond strength than saturated surface wet condition. The moisture condition of the interior and surface of the substrate will influence the development of bond strength. When conducted study on bond between concrete and steel, Fu [23] found that water treatment increased bond strength. However, the longer the water immersion time, the weaker the bond. These diverse results might be caused by different environmental condition, material of substrate and overlay.

Water to Cement (w/c) Ratio

The concept of water to cement ratio (w/c) was developed by Duff A. Abrams [24] and was first published in 1918. The w/c ratio is the ratio of the weight of water to the weight of cement used in a concrete mix. The compressive strength of new concrete with lower w/c ratio (up to 0.42) results in higher compressive strength [25]. Of all the diverse factors influencing the strength of concrete, the principle one is the amount of mixing water used with respect to the weight of cement. On the basis of experimental data, Belyaev [26] proposed an equation of concrete strength

R30=Rc/ (3(w/c) 1.5)
Where R30: is the concrete strength on the 30th day;
Rc: activity of the cement;
w/c: water to cement ratio.

From this equation, greater the w/c ratio, the weaker the cement, and lower the concrete strength. The compressive strength of concrete is known to decrease with an increase of w/c ratio due to an increase in porosity, but the bond strength increases with an increase of w/c ratio by having decreased void content at the interface [26].

Bonding Agent

Bonding agents have been used to enhance bond with the existing concrete. Bonding agents are natural, compounded or synthetic materials used to enhance the joining of individual members of a structure without employing mechanical fasteners. These products are often used in repair applications such as the bonding of fresh concrete, sprayed concrete or sand/cement repair mortar to hardened concrete.
Two of the critical factors affecting the bonding between new and old concrete, provided sound concrete practices are followed, are (i) the strength and integrity of the old surface and (ii) the cleanliness of the old surface.

When a weak layer of concrete (laitance) exists on the old surface or when the old surface is dirty, a poor bond is obtained. The surface condition thus plays a critical role in bond development, although the strength of the bond depends on other factors such as proper compaction of the new concrete and proper surface preparation that takes into account the density of the base concrete. For a sound base concrete, for example, acid etching will suffice, while mechanical cleaning will be essential if the old concrete contains a weak or deteriorated surface.

Bonding agents play a significant role where it is critical to ensure bond at the interface. For example, a weak and pliable substrate may need strengthening to match the modulus of the repair material. A bonding agent may be required because of the prevailing poor ambient conditions. Notwithstanding the advantages provided by bonding agents, they should not be used to compensate for poor workmanship.

2.7 IMPORTANCE OF USING BONDING AGENTS DURING REPAIR

Properly selecting and applying a bonding agent between repair materials and existing concrete has been shown to improve bond strength between repair materials and new concrete [27, 28, 29] Selected bonding agents depend on the required performance of the repair. When the repair concrete is portland cement-based grouts, epoxy-based bonding agents and latex bonding agents can be used. Rapid setting repair materials such as magnesium phosphate don't require bonding agents, and if bonding agents are used the bond strength is typically lowered.

2.7.1 Portland Cement Grouts

Portland cement grouts use cement and water to produce bonding agents that can be used between existing concrete and repair concrete. Grout with a 0.3 w/c has demonstrated to increase bond strength [27]. A field investigation was completed on existing concrete pavement where a dry substrate, 0.3 w/c ratio grout, wet substrate, and water/silica fume slurry were used. After the repair material was placed pull off tensile test were performed after 7 days and 10 months of ageing and weather exposure. The pull off tensile strength were 200 psi for the portland cement grout, 145 psi for the water/ silica fume slurry, and 130 psi for the wet and dry surface conditions [27]

2.7.2 Epoxy Bonding Agent

Epoxy bonding agents must be high modulus, moisture tolerant, and compliant with ASTM C881 [30] requirements. Structural epoxies are typically made up of a two-part system of chemicals that are mixed immediately before application. The hardener and the modifier must be thoroughly mixed before the bonding agent is applied between the repair material and the existing concrete. Epoxies must have a minimum gel time of 30 minutes [30]. Like many chemical reactions, the epoxy hardening process is a temperature-dependent process. Hot weather conditions decrease epoxy gel time and cold weather increases gel time and must be accounted for in the field [31].

In a laboratory study where epoxy bonding agents were used on multiple substrates surface preparations the samples that used epoxy bonding agents had higher bond strengths [29] then with samples that did not. The surface examined were left as cast, wire brushed, and shot blasted [29]. Both dry and saturated surface conditions were examined. The samples were examined using a direct shear test, and the samples made with epoxy agents after shot blasting the substrate had the highest bond strength of 700 psi. The same sample with no agent had bond strength of 530 psi. Even the samples with left as cast substrate surfaces which had bond strength of 200 psi with no bonding agent had strength of 420 psi when using epoxy bonding agents.

2.7.3 Application

Bonding agents are applied to the existing concrete with a brush in a thin continuous layer before the repair material is placed. The entire repair section surface must be covered by the bonding agents [31]. When using epoxy, the repair concrete should be applied before the working time is exceeded. Exceeding the gel time will inhibit bond strength development [30].

CHAPTER 3

EXPERIMENTAL WORK

3.1 INTRODUCTION

This chapter reports the experimental program of the research that was conducted. to achieve the research objects The experimental work of this research consisting of sample preparation of half scale models of concrete beams . Details of the specimens, test set-up and the properties of the materials used are also presented in this chapter. The test setups and procedure are discussed below.

3.2 BEAM TEST SPECIMENS

3.2.1 Description of Beam Specimens

The test specimens were comprised of 30 half scale RC beams of rectangular cross-section. The designation of the specimens reflects its major properties .For example in FB3: FB stands for fully cast at a time and 3 is the concrete strength. Similarly in HB31C: HB stands for fully cast after completing 1^{st} half portion then 2^{nd} half portion, 3 stands for concrete compressive strength , 1 is for 2 nd half portion casting done after one days and C stands for casting is done on the concrete surface without applying any epoxy resin.
The beams were cast in three groups, see Table 3.1. These groups were:

Group A. Comprised of ten beams for concrete strength 3ksi, one of which was reference specimens and fully cast at the first day and the another three were cast in two phases after one day, three days and seven days and the rest six were also cast in two phases after applying two different grades of epoxy on the half cast beams top surface after one day, three days and seven days.

Group B. Comprised of ten beams for concrete strength 4ksi, one of which was reference specimens and fully cast at the first day and the another three were cast in two phases after one day, three days and seven days and the rest six were also cast in two phases after applying two different grades of epoxy on the half cast beam top surface after one day, three days and seven days.

Group C. Comprised of ten beams for concrete strength 5ksi, one of which was reference specimens and fully cast at the first day and the another three were cast in two phases after one day, three days and seven days and the rest six were also cast in two phases after applying two different grades of epoxy on the half cast beam top surface after one day, three days and seven days.

The overall dimensions of the all test beams were remained same, i.e. beam width b= 150mm(6 inch), beam height,h=200mm (8 inch), effective depth d=175 mm(7 inch) and a beam length of 1500mm (5ft).The span length for the test beams was 1350mm(54inch).
No variation was made as far as longitudinal main reinforcement and shear reinforcement for all the beams. The beams were doubly reinforced with longitudinal main reinforcement at top and bottom to avoid premature shear failure of the beams during loadings and 8mm two-leg stirrups were provided .Shear reinforcements were provided 37.5mm(1.5 inch) from both

supports and then @350mm(14inch) in the middle part .Details of beam's design are illustrated in Figure 3.1

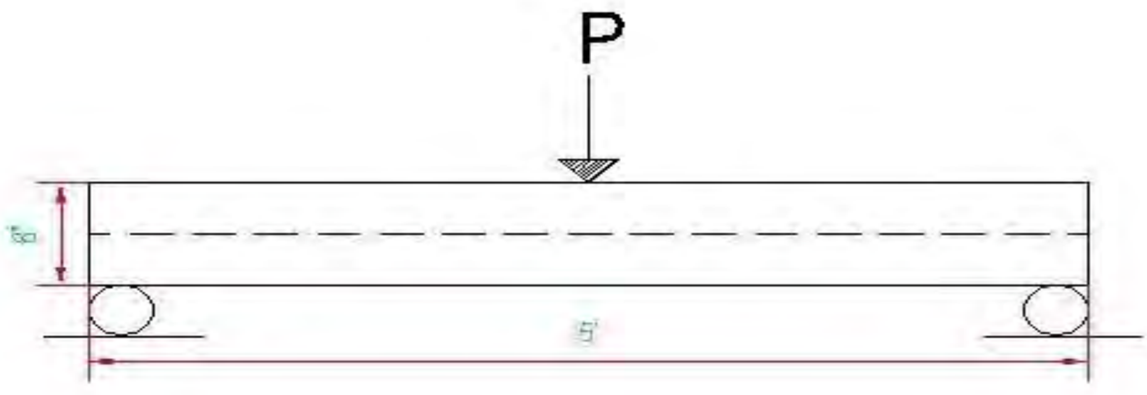

Figure: 3.1 Figure Of Beam

Table 3.1 Description Of Test Specimens

Group	Specimen Designation	Concrete Compressive Strength (ksi)	Cast at a time	Cast in two phase	Use of Bonding Agent
Group A	Specimen-FB3 Control specimen	3 ksi	beam was fully cast at a time		No needed
	Specimen-HB31C	3 ksi		beam was cast in two phase after one day	without using any bonding agent
	Specimen-HB31EP	3 ksi		beam was cast in two phase after one day	with using bonding agent 50MPa Epoxy
	Specimen-HB31PC	3 ksi		beam was cast in two phase after one day	with bonding agent 75MPa Epoxy
	Specimen-HB33C	3 ksi		beam was cast in two phase after three days	without using any bonding agent
	Specimen-HB33EP	3 ksi		beam was cast in two phase after three days	with using bonding agent 50MPa Epoxy
	Specimen-HB33PC	3 ksi		beam was cast in two phase after three days	with using bonding agent 75MPa Epoxy

	Specimen-HB37C	3 ksi		beam was cast in two phase after seven days	without using any bonding agent
	Specimen-HB37EP	3 ksi		beam was cast in two phase after seven days	with using bonding agent 50MPa Epoxy
	Specimen-HB37PC	3 ksi and		beam was cast in two phase after seven days	with using bonding agent 75MPa Epoxy
Group B	Specimen-FB4 Control specimen	4 ksi	beam was fully cast at a time		No needed
	Specimen-HB41C	4 ksi		beam was cast in two phase after one day	without using any bonding agent
	Specimen-HB41EP	4 ksi		beam was cast in two phase after one day	with using bonding agent 50MPa Epoxy
	Specimen-HB41PC	4 ksi		beam was cast in two phase after one day	with bonding agent 75MPa Epoxy
	Specimen-HB43C	4 ksi		beam was cast in two phase after three days	without using any bonding agent
	Specimen-HB43EP	4 ksi		beam was cast in two phase after three days	with using bonding agent 50MPa Epoxy
	Specimen-HB43PC	4 ksi		beam was cast in two phase after three days	with using bonding agent 75MPa Epoxy
	Specimen-HB47C	4 ksi		beam was cast in two phase after seven days	without using any bonding agent
	Specimen-HB47EP	4 ksi		beam was cast in two phase after seven days	with using bonding agent 50MPa Epoxy
	Specimen-HB47PC	4 ksi and		beam was cast in two phase after	with using bonding agent 75MPa Epoxy

				seven days	
Group C	Specimen-FB5 Control specimen	5 ksi	beam was fully cast at a time		No needed
	Specimen-HB51C	5 ksi		beam was cast in two phase after one day	without using any bonding agent
	Specimen-HB51EP	5 ksi		beam was cast in two phase after one day	with using bonding agent 50MPa Epoxy
	Specimen-HB51PC	5 ksi		beam was cast in two phase after one day	with bonding agent 75MPa Epoxy
	Specimen-HB53C	5 ksi		beam was cast in two phase after three days	without using any bonding agent
	Specimen-HB53EP	5 ksi		beam was cast in two phase after three days	with using bonding agent 50MPa Epoxy
	Specimen-HB53PC	5 ksi		beam was cast in two phase after three days	with using bonding agent 75MPa Epoxy
	Specimen-HB57C	5 ksi		beam was cast in two phase after seven days	without using any bonding agent
	Specimen-HB57EP	5 ksi		beam was cast in two phase after seven days	with using bonding agent 50MPa Epoxy
	Specimen-HB57PC	5 ksi		beam was cast in two phase after seven days	with using bonding agent 75MPa Epoxy

3.2.2 Material properties

Reinforcement

BSRM XTREME 500W steel bars were used as internal reinforcement .For all groups A, B and C the longitudinal steel reinforcement was 2Ø10 and 1Ø8 at bottom and 2Ø8 at top and shear reinforcement was Ø8. Samples were tested for yield and ultimate capacity. The summary and details of the test result are given in Table 3.2 and Table A.7 respectively.

Table 3.2 Strength of Reinforcing Bars

Diameter (mm)	Elongation (%)	Cross Section of Bar (mm^2)	Yield Strength (MPa)	Ultimate Strength (MPa)
10	14	76.7	587.69	672.75
8	13.67	49.1	559.12	650.77

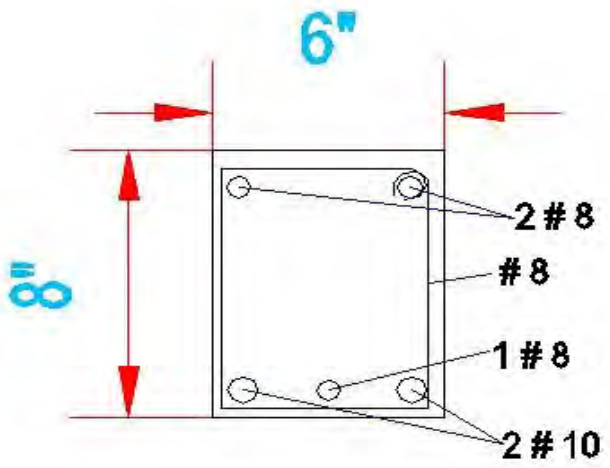

FIGURE: 3.2 Cross-Section of Beam

Cement

Cement is a binder, a substance that sets and hardens and can bind other materials together. The most important uses of cement are as a component in the production of mortar in masonry, and of concrete, a combination of cement and an aggregate to form a strong building material.

The research was conducted using Bashundhara Portland Composite Cement (CEM II). The most important properties of the cement are hydration, setting, fineness and strength. The physical properties of the cement are summarized in the following Table 3.2

.

Table 3.3: Physical Properties of Cement

Physical properties	Test Value	Specification
Initial Setting Time (minutes)	168	>45
Final Setting Time (minutes)	290	<420
Compressive Strength (MPa)		
3 Days	13	> 16
7 Days	20	>22
28 Days	25	> 31

Fine Aggregate

In general, aggregate comprising of particles finer than 5mm (0.2 in.) e.g. sand, crushed stone or crushed slag screenings ete. can be classified as fine aggregate. Fine aggregate is an essential constituent of aggregate since its primary function is fill up the large voids between the particles of coarse aggregate and prevent honeycomb in the concrete matrix. For adequate consolidation of concrete, the desirable amount of air, water, cement and fine aggregate (that is, the mortar function) should be about 50% to 65% by absolute volume (45% to 60% by mass) [32].

Shylet Sand

Shylet sand extracted from Surma river bed is utilized as fine aggregate in the concrete prepared for the study. Figure 1 shows the gradation curve. Table 3.2 presents property test results.

Table 3.4: Fine Aggregate (Sylhet Sand) Properties According To ASTM C29/C29M-97 And ASTM C128-88 And ASTM C136-01

Basic Property	**Condition**	**Value**
Specific Gravity	Oven Dry Bulk Sp. Gr.	2.47
	SSD Bulk Sp.Gr	2.55
	Apparent Sp.Gr	2.70
Absorption Capacity		3.52%
Density	Oven Dry Rodded Unit Weight	1561kg/m^3
	SSD Rodded Unit Weight	1616 kg/m^3
Voids	Compacted by rodding	37%
Fineness Modulus		2.77

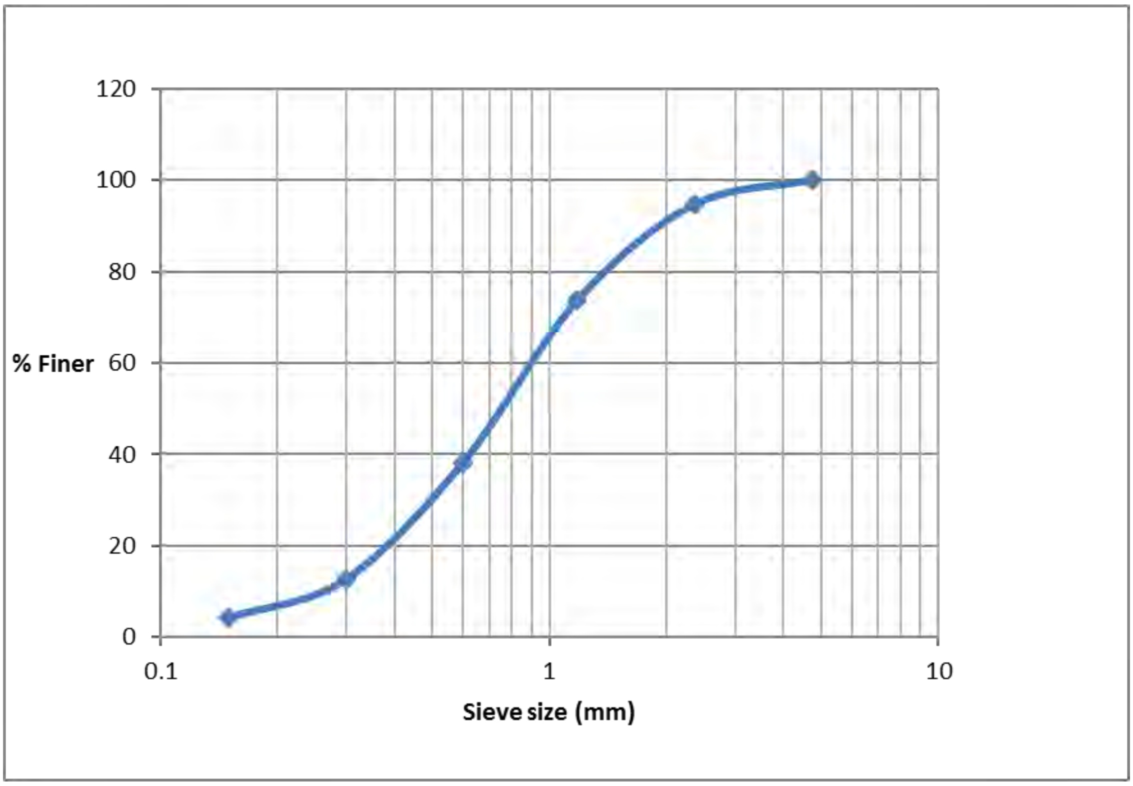

Figure 3.3 Grain Size Distribution Curve of Fine Aggregates

Coarse Aggregate

Coarse aggregate shall consist of naturally occurring materials such as gravel, or resulting from the crushing of parent rock, to include natural rock, slags, expanded clays and shales (lightweight aggregates) and other approved inert materials with similar characteristics, having hard, strong, durable particles, conforming to the specific requirements of this Section. Materials substantially retained on the No. 4 sieve, shall be classified as coarse aggregate.

Coarse aggregate consists of particles that are more than 9.5 mm (3/8 in) and generally less than 37.5mm (11/2 in)in size. Coarse aggregate can be extracted from various sources. The usual sources of coarse aggregate are quarry rock, boulders, cobbles, gravels etc. For the present research crushed stone (stone chips) are used.

Crushed Stone

Crushed stone is a form of construction aggregate, typically produced by mining a suitable rock deposit and breaking the removed rock down to the desired size using crushers. 12.5 mm downgrade stone chips are used as coarse aggregates. Major properties of the aggregate were tested in the laboratory and the properties are presented in table 1

Table 3.5: Coarse Aggregate (Stone Chips) Properties According To ASTM C29/C29M-97and ASTM C127-88

Basic Property	**Condition**	**Value**
Specific Gravity	Oven Dry Bulk Sp. Gr.	2.42
	SSD Bulk Sp.Gr	2.45
	Apparent Sp.Gr	2.48
Absorption Capacity		0.98%
Density	Oven Dry Rodded Unit Weight	1438kg/m^3
	SSD Rodded Unit Weight	1452 kg/m^3
Voids	Compacted by rodding	41%

Concrete

To prepare concrete, Portland Composite Cement was used along with shylet sand as fine aggregate and 12.5 mm downgrade stone chips as coarse aggregates.

The concrete was mixed in a mixer machine which was used for casting all the beams and the casting took place at the concrete lab in BUET. Before using concrete slump test was carried out to keep the slump value in between 1 to 2 inch.

Determination of Target Strength

Based on calculation several mixture proportions of concrete are selected and their 7 days and 28 days strength are determined to obtain the target strengths which are shown in Table 3.6 and Table 3.7.

Table 3.6: 7 days Cylinder Strength

Mix Ratio	Cylinder No.	Observed Load (KN)	Actual Load (KN)	Area,m2	Strength (KN/m^2)	Strength (ksi)	w/c Ratio
1:2.0:1.75	S1	230	214.14	0.008107	26414.20994	3.830	
	S2	230	214.14	0.008107	26414.20994	3.830	0.45
	S3	228	212.104	0.008107	26163.06895	3.793	
1:2.25:2.0	S7	210	193.78	0.008107	23902.80005	3.465	
	S8	204	187.672	0.008107	23149.37708	3.356	0.48
	S9	208	191.744	0.008107	23651.65906	3.429	
1:2.50:2.15	S13	194	177.492	0.008107	21893.67214	3.174	
	S14	186	169.348	0.008107	20889.10818	3.028	0.54
	S15	198	181.564	0.008107	22395.95411	3.247	

1:2.75:2.25	S19	217	105.513	0.008107	13015.04872	1.887	
	S20	235	123.315	0.008107	15210.92883	2.205	0.57
	S21	221	109.469	0.008107	13503.02208	1.957	
1:3.00:2.50	S25	220	108.48	0.008107	13381.02874	1.940	
	S26	219	107.491	0.008107	13259.0354	1.922	0.62
	S27	221	108.48	0.008107	13381.02874	1.940	
1:3.50:2.75	S31	187	75.843	0.008107	9355.248551	1.356	
	S32	185	73.865	0.008107	9111.261872	1.321	0.68
	S33	178	66.942	0.008107	8257.308499	1.197	
1:3.75:3.00	S37	175	63.975	0.008107	7891.328482	1.144	
	S38	178	66.942	0.008107	8257.308499	1.197	0.71
	S39	176	64.964	0.008107	8013.321821	1.161	

For 1:2:1.75, 1:2.25:2.0, and 1:2.50:2.15
Calibration equation is $y = 1.018x - 20$

For 1:2.75:2.25, 1:3.00:2.50, 1:3.50:2.75 and 1:3.75:3.00
Calibration equation is $y = 0.989x - 109.1$

Table 3.7:28 days Cylinder Strength

Mix Ratio	Cylinder No.	Observed Load (KN)	Actual Load (KN)	Area,m2	Strength (KN/m^2)	Strength (ksi)	w/c Ratio
1:2.0:1.75	S4	330	323.396	0.008107	39890.96	5.784	0.45
	S5	320	313.366	0.008107	38653.76	5.604	
	S6	330	323.396	0.008107	39890.96	5.784	
1:2.25:2.0	S10	284	277.258	0.008107	34199.83	4.958	0.48
	S11	268	261.21	0.008107	32220.3	4.671	
	S12	262	255.192	0.008107	31477.98	4.564	
1:2.50:2.15	S16	352	249.072	0.008107	30723.08	4.454	0.54
	S17	351	248.061	0.008107	30598.37	4.436	
	S18	348	245.028	0.008107	30224.25	4.382	
1:2.75:2.25	S22	230	223.096	0.008107	27518.93	3.990	0.57
	S23	240	233.126	0.008107	28756.14	4.169	
	S24	234	227.108	0.008107	28013.82	4.062	
	S28	291	187.401	0.008107	23115.95	3.351	

1:3.00:2.50	S29	301	197.511	0.008107	24363.02	3.532	0.62
	S30	301	197.511	0.008107	24363.02	3.532	
1:3.50:2.75	S34	242	137.862	0.008107	17005.3	2.465	0.68
	S35	230	125.73	0.008107	15508.82	2.248	
	S36	236	131.796	0.008107	16257.06	2.357	
1:3.75:3.00	S40	223	118.653	0.008107	14635.87	2.122	0.71
	S41	237	132.807	0.008107	16381.77	2.375	
	S42	239	134.829	0.008107	16631.18	2.411	

For 1:2:1.75, 1:2.25:2.0 and 1:2.75:2.25
Calibration equation is $y = 1.003x - 7.594$

For 1:2.50:2.15, 1:3.00:2.50, 1:3.50:2.75 and 1:3.75:3.00
Calibration Equation is $y = 1.011x - 106.8$

Depending on the above two table 3.6 and 3.7, three mixture proportions of concrete are selected to obtain target concrete strength 3 ksi, 4ksi and 5ksi.

The mixture proportions of the constituents of the target concrete strength are shown in Table 3.8.

Table 3.8: Concrete Mixture Proportions (Volumetric Ratio)

Concrete strength, f′c ksi	Mix ratio (cement: sand: stone chips)	Water /cement ratio	Attained Strength, ksi
3	**1:3:2.50**	**0.62**	**3.8**
4	**1:2.50:2.15**	**0.54**	**4.4**
5	**1:2:1.75**	**0.45**	**5.3**

The compressive strength for each concrete casting was determined on 4x8 inch standard concrete cylinders. The samples were in the moulds for 24 hours; thereafter they were taken from their moulds and stored at 100% relative humidity until testing. The compressive strength was tested after 28 days for the entire specimen for group A, B and C.

Workability measurement was carried out on the fresh concrete as slump value, was approximately 2 in, as shown in figure 3.5.

Figure 3.4: Concrete Mixing **Figure 3.5: Slump Test**

Figure 3.6: Concrete Cylinders are Stored in Water. **Figure3.7. Compressive Strength Testing of Cylinders**

Bonding Agent

For the present research purpose 50MPa Epoxy resin concrete bonding agent and 75MPa epoxy adhesive are used. Their specifications are shown in Appendix B.

50MPa Epoxy

The two materials (base and hardener) are thoroughly mixed until a uniform color is obtained. The measurement is as follows:

Hardener= base/2

75MPa Epoxy

The base and hardener are poured into a suitable container and mixed with a paddle to obtain a uniform grey color. The measurement is as follows:

Hardener= base/2

3.3 CONSTRUCTION OF SPECIMENS

The beams were made using wooden formwork and it was constructed at concrete lab at BUET. The concrete was placed into the wooden beam forms and then compacted with a mechanical vibrator.

3.3.1 Specimen Preparation for Group A, B and C

The total procedure was completed in four steps:

Step one- the longitudinal reinforcements along with shear reinforcements were kept in the formwork. A number of small mortar blocks were used on the inner base and on two sides of the formwork to maintain the clear cover and vibrator was used for proper compaction. Ten beams were made. Among them one was fully cast and others were cast in half depth of beam which was marked previously. So for three groups total three beams were fully cast and twenty seven beams were half cast.

Figure: 3.8 Formwork Ready for Casting. Reinforcements Were Kept In Formwork

Figure: 3.9 Concrete Was Poured In The Formwork

Figure: 3.10 Half Cast Beams

Step two-after one day, three half cast beams were completed on the remaining portion. Among them one was finished without using any resins on the half cast beam's top surface. The other two were cast after applying 50MPa Epoxy and 75MPa Epoxy on the exposed surface of half cast beams. As 75MPa Epoxy sets earlier (pot life is around 30 mins), so the beam was cast quickly after application of this bonding agent. But in case of 50MPa Epoxy we can delay as pot life is 4 to 6 hr. So for three groups total nine beams were cast.

Figure: 3.11 Half Made Beam Was Fully Cast Without Using Any Bonding Agent After One Day

Figure: 3.12 Preparation of 50MPa Epoxy

Figure: 3.13 Application of 50MPa Epoxy

Figure: 3.14 After 50MPa Epoxy Application Beam Casting

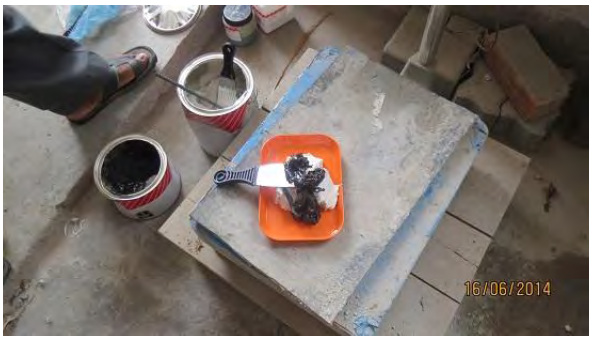

Figure: 3.15 75MPa Epoxy Preparation

Figure: 3.16 Application of 75MPa Epoxy

Figure: 3.17 After Application Of 75MPa Epoxy Half Prepared Beams Were Vibrated and Cast

Step three-Same procedure like step two was followed to complete another three beams but the only difference was it was done after three days. For three groups nine half prepared beams were cast.

Figure: 3.18 Application of 75MPa Epoxy And 50MPa Epoxy After Three Days

Figure: 3.19 After Application of Bonding Agent Beam Casting

Figure: 3.20 Curing of Beams

Step four-again after seven days we cast the remaining last three half cast beams following the similar procedure as like step two. For three groups nine half prepared beams were cast.

Figure .3.21 Casting of Last Nine Beam Specimens

Figure: 3.22 Curing of All Thirty Beams

The beams were stored covered in plastic sheet until before testing. The specimens were cured for more than 28 days. Curing stopped 2 to 4 days before testing to allow for painting and placement of the beam on the loading frame. The compressive strength for each concrete casting was determined by standard 100 mm x 200 mm concrete cylinders. The detailed cylinder test results are given in Table A.6 of Appendix A

Before testing all the specimens were all through white washed to find out the crack and their absolute location. Precautions were taken to avoid any potential damages during lifting and transporting of the specimens.

3.4 LOAD SELECTION

The Specimens were tested under incremental one point loading .As the perpous of the test was to investigate the flexural shear capacity, not completely flexure nor shear but both of them, so one point loading pattern was selected.

3.5 EXPERIMENTAL SET UP AND TESTING PROCEDURE

Figure 3.23 also shows the loading and support arrangement of the beams. The beams were mounted on a platform and two concrete blocks with semi-circular upper end were placed at the bottom at the points of support so that beam can deflect as a simply supported beam. The load was applied by Tinius Olsen Universal Testing Machine at midspan of the beam. A constant strain-controlled loading was applied with the movement rate of the platform being 5mm/min. Time vs load was continuously monitored and data was saved automatically .Deflection of the beams were also monitored with a video extensometer and time versus deflection data was stored continuously .These data are then combined to prepare load versus deflection curves which are the basis of analysis for flexural strength .The failure loads for specimens in all groups and the deflections for the specimens that were recorded will be presented in Chapter 4.

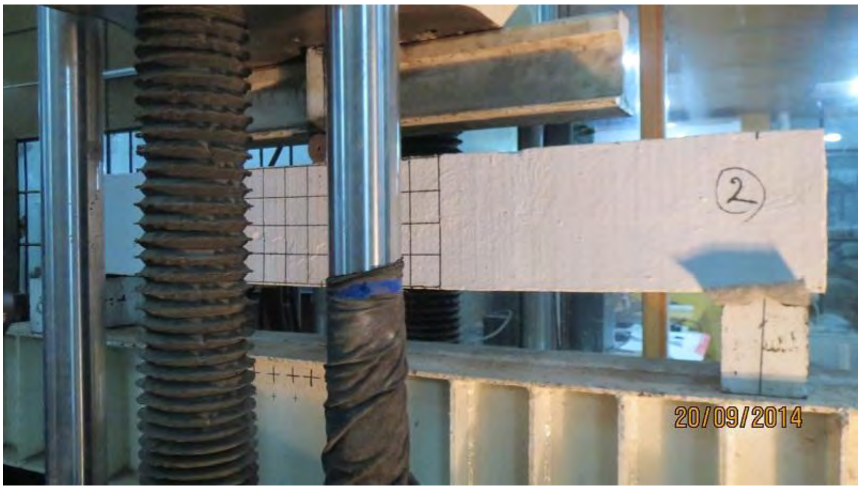

Figure: 3.23Experimental Set Up in Laboratory

CHAPTER FOUR

ANALYSIS AND INTERPRETATIOM OF RESULTS

4.1 GENERAL

This Chapter summarizes the qualitative and quantitative experimental results from test specimens Sample-1 to Sample-30. The qualitative results include photographs of each specimen through the course of testing and displaying the crack patterns. Load corresponding to displacements and different crack history were recorded for producing the quantitative results

4.2 FAILURE MODES OF SPECIMENS

Most of the beams failed in shear by means of diagonal tension. This type of shear failure is characterized by large diagonal shear crack where the crack develops with further load. It extends gradually until it reaches its critical point where it finally fails without a warning.

For the beams in three groups, flexural cracks were observed in the fully cast beams near the mid-span at the bottom of the beam, at a load level of about 15kN(except in case of fully cast beam FB4). The shear cracks began to appear at a load of approximately 20- 40 kN in the other beams. As the load increased, the shear crack developed further up to the critical failure of the beam. As explained in chapter 2, shear failure modes are extremely hard to predict. The failure progress of the reference beams observes that beams in three groups have not the same values of ultimate shear load. As displayed in Figure 4-1 to Figure 4-30 that the crack pattern differs between beams.

4.2.1 Crack Patterns of Group A

Specimen FB3

The cracks were marked by permanent black pen and the corresponding loadings were stored automatically.First flexural crack in the control specimen was generated at the mid-span at the bottom of the beam, at a load level of about 12kN .After that several flexure and diagonal tension cracks were generated as Figure 4.1. As the load increased, the flexural crack developed further up to the final failure.

Figure 4.1 Failure Pattern of Specimen-FB3

Specimen HB31C

At 20kN first flexural crack was initiated at the mid-span at the bottom of the beam. As the load increased, the flexural crack developed further up to the final failure. No diagonal crack and no bond crack were developed.

Figure 4.2 Failure Pattern of Specimen-HB31C

Specimen HB31EP

The cracks were marked by permanent black pen and the corresponding loadings were stored automatically.First flexural crack in the control specimen was generated at the mid-span at the bottom of the beam, at a load level of about 24 kN. After that several flexure and diagonal tension cracks were generated. Also bond crack was visible. As the load increased, the diagonal tension crack followed by very small bond crack developed further up to the final failure

Figure 4.3 Failure Pattern of Specimen-HB31EP

Specimen HB31PC

The cracks were marked by permanent black pen and the corresponding loadings were stored automatically.First flexural crack in the control specimen was generated at the mid-span at the bottom of the beam, at a load level of about 19 kN. After that several flexure and diagonal tension cracks were generated. Also bond crack was visible. As the load increased, the diagonal tension crack developed further up to the final failure.

Figure 4.4 Failure Pattern of Specimen-HB31PC

Specimen HB33C

First flexural crack in the specimen was generated near the mid-span at the bottom of the beam, at a load level of about 9 kN. After that several flexure and diagonal tension cracks were generated as Figure 5.5. As the load increased, the diagonal tension crack developed further up to the final failure. No bond crack was developed

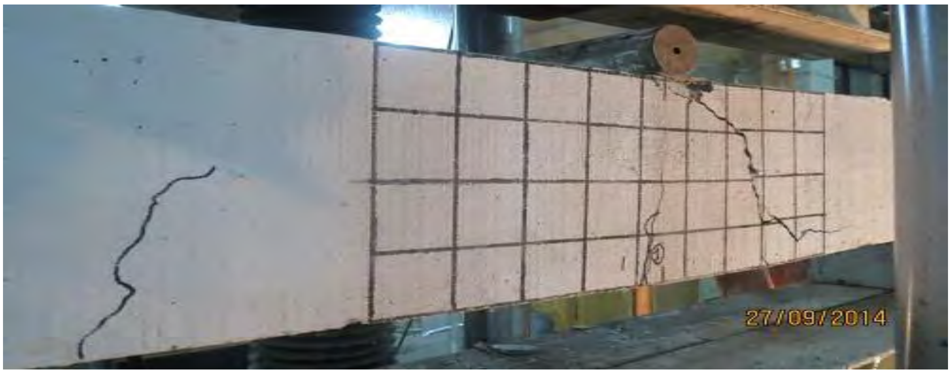

Figure 4.5: Failure Pattern of Specimen-HB33C

Specimen HB33EP

The cracks were marked by permanent black pen and the corresponding loadings were stored automatically.First flexural crack in the control specimen was generated at the mid-span at the bottom of the beam, at a load level of about 20 kN. After that several flexure and diagonal tension cracks were generated. Also bond crack was visible. As the load increased, the diagonal tension crack developed further up to the final failure.

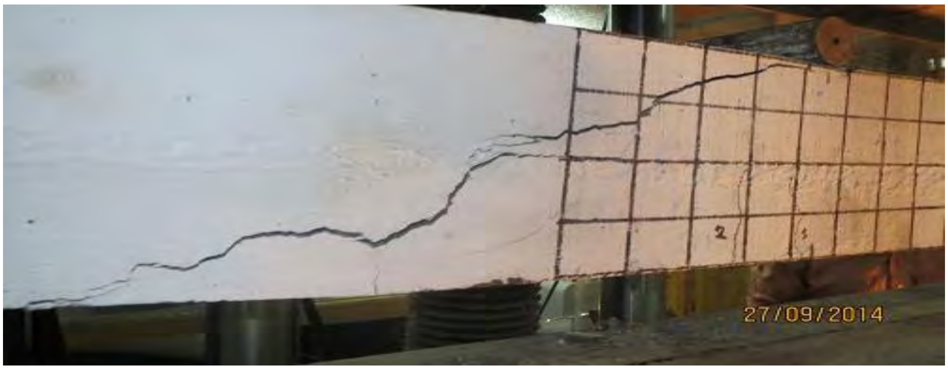

Figure 4.6 Failure Pattern of Specimen-HB33EP

Specimen HB33PC

The cracks were marked by permanent black pen and the corresponding loadings were stored automatically.First flexural crack in the control specimen was generated at the mid-span at the bottom of the beam, at a load level of about 15 kN. After that several flexure and diagonal tension cracks were generated. Also bond crack was visible. As the load increased, the diagonal tension crack followed by bond crack developed further up to the final failure.

Figure 4.7 Failure Pattern of Specimen-HB33PC

Specimen HB37C

The cracks were marked by permanent black pen and the corresponding loadings were stored automatically.First flexural crack in the control specimen was generated at the mid-span at the bottom of the beam, at a load level of about 19kN. After that several flexure and diagonal tension cracks were generated. Also bond crack was visible. As the load increased, the diagonal tension crack followed by bond crack developed further up to the final failure.

Figure 4.8 Failure Pattern of Specimen-HB37C

Specimen HB37EP

The cracks were marked by permanent black pen and the corresponding loadings were stored automatically.First flexural crack in the control specimen was generated at the mid-span at the bottom of the beam, at a load level of about 28kN. After that several flexure and diagonal tension cracks were generated. Also bond crack was visible. As the load increased, the diagonal tension crack developed further up to the final failure.

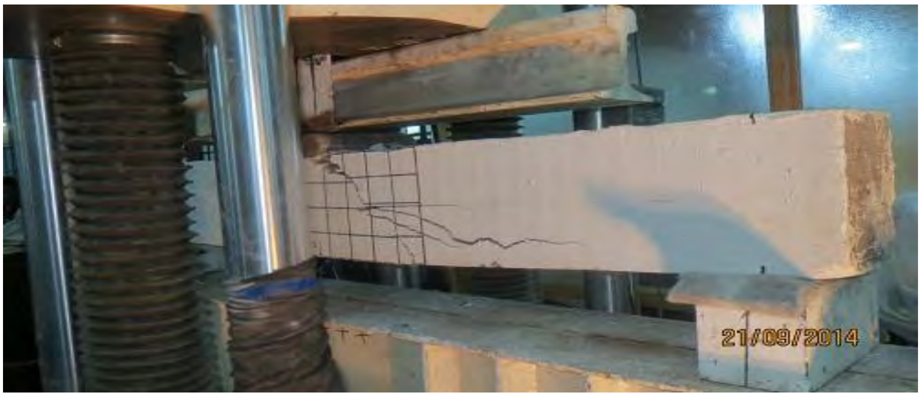

Figure 4.9 Failure Pattern of Specimen-HB37EP

Specimen HB37PC

The cracks were marked by permanent black pen and the corresponding loadings were stored automatically.First flexural crack in the control specimen was generated at the mid-span at the bottom of the beam, at a load level of about 22kN. After that several flexure and diagonal tension cracks were generated. Also bond crack was visible. As the load increased, the bond crack developed further up to the final failure.

Figure 4.10 Failure Pattern of Specimen-HB37PC

4.2.2 Crack Patterns of Group B

Specimen FB4

The cracks were marked by permanent black pen and the corresponding loadings were stored automatically.First flexural crack in the control specimen was generated near the mid-span at the bottom of the beam, at a load level of about 33kN .After that several flexure cracks and diagonal tension cracks were generated as Figure 5.11. At the loading point crushing of concrete was also occurred. As the load increased, the flexural crack developed further up to the final failure.

Figure 4.11 Failure Pattern of Specimen-FB4

Specimen HB41C

The cracks were marked by permanent black pen and the corresponding loadings were stored automatically.First flexural crack in the control specimen was generated at the mid-span at the bottom of the beam, at a load level of about 14kN. After that several flexure cracks were generated. As the load increased, the flexure cracks developed further up to the final failure. No diagonal tension crack and bond crack was developed.

Figure 4.12 Failure Pattern of Specimen-HB41C

Specimen HB41EP

The cracks were marked by permanent black pen and the corresponding loadings were stored automatically.First flexural crack in the control specimen was generated at the mid-span at the bottom of the beam, at a load level of about 16kN. After that several flexure and diagonal tension cracks were generated. Also bond crack was visible. As the load increased, the diagonal tension crack followed by bond crack developed further up to the final failure.

Figure 4.13 Failure Pattern of Specimen-HB41EP

Specimen HB41PC

The cracks were marked by permanent black pen and the corresponding loadings were stored automatically.First flexural crack in the control specimen was generated at the mid-span at the bottom of the beam, at a load level of about 15kN. After that several flexure and diagonal tension cracks were generated. As the load increased, the diagonal tension crack developed further up to the final failure. No bond crack was developed.

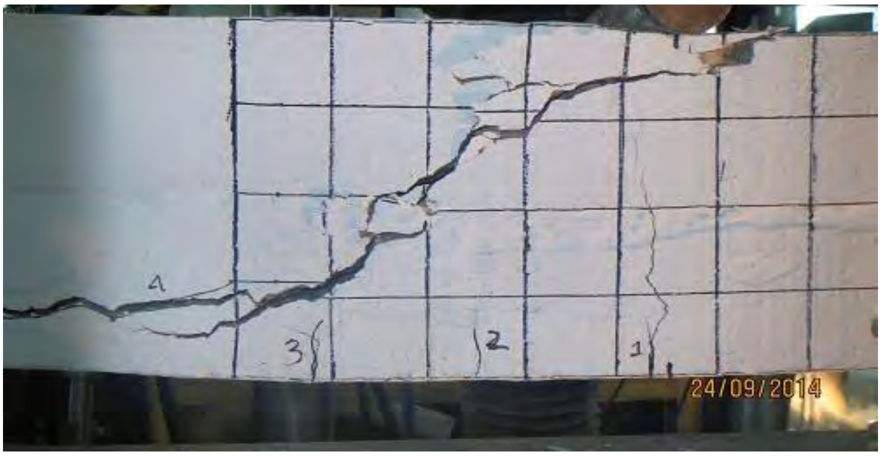

Figure 4.14 Failure Pattern of Specimen-HB41PC

Specimen HB43C

The cracks were marked by permanent black pen and the corresponding loadings were stored automatically.First flexural crack in the control specimen was generated at the mid-span at the bottom of the beam, at a load level of about 31kN. After that several flexure and diagonal tension cracks were generated. Also bond crack was visible. As the load increased, the diagonal tension crack followed by bond crack developed further up to the final failure.

Figure 4.15 Failure Pattern of Specimen-HB43C

Specimen HB43EP

The cracks were marked by permanent black pen and the corresponding loadings were stored automatically.First flexural crack in the control specimen was generated near the mid-span at the bottom of the beam, at a load level of about 21kN. After that several flexure and diagonal tension cracks were generated. Also bond crack was visible. As the load increased, the diagonal tension crack followed by bond crack developed further up to the final failure.

Figure 4.16 Failure Pattern of Specimen-HB43EP

Specimen HB43PC

The cracks were marked by permanent black pen and the corresponding loadings were stored automatically.First flexural crack in the control specimen was generated at the mid-span at the bottom of the beam, at a load level of about 24kN. After that several flexure and diagonal tension cracks were generated. No bond crack was visible. As the load increased, the diagonal tension crack developed further up to the final failure.

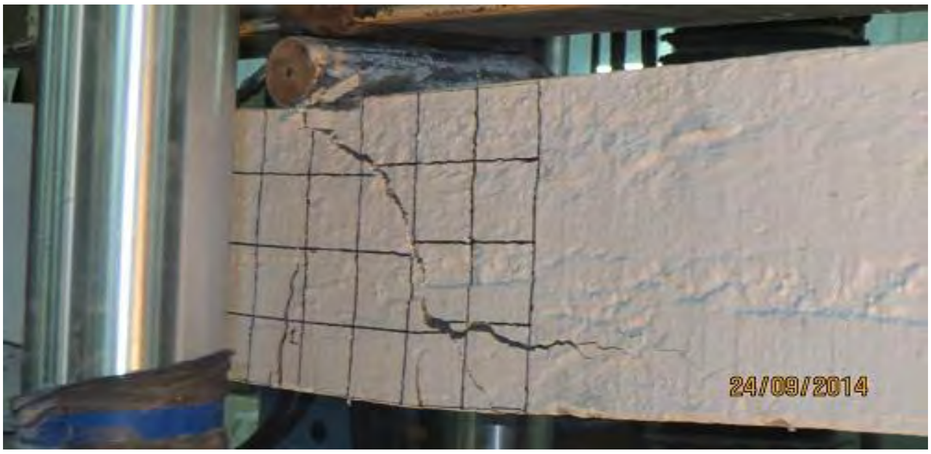

Figure 4.17 Failure Pattern of Specimen-HB43PC

Specimen HB47C

The cracks were marked by permanent black pen and the corresponding loadings were stored automatically.First flexural crack in the control specimen was generated at the mid-span at the bottom of the beam, at a load level of about 16kN. After that several flexure and diagonal tension cracks were generated. No bond crack was visible. As the load increased, the diagonal tension crack developed further up to the final failure.

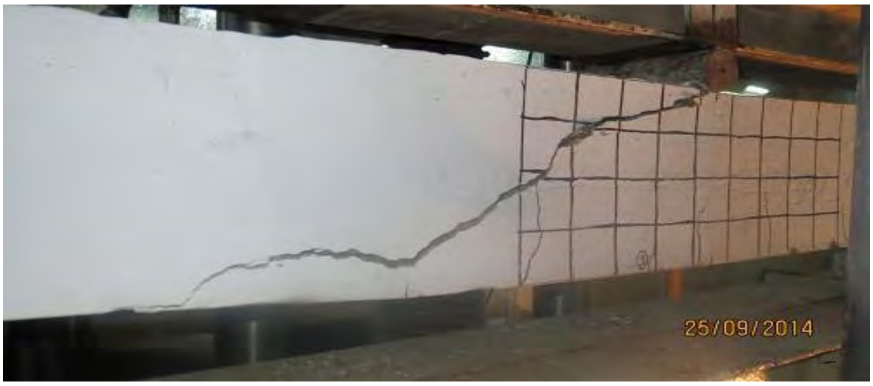

Figure 4.18 Failure Pattern of Specimen-HB47C

Specimen HB47EP

The cracks were marked by permanent black pen and the corresponding loadings were stored automatically.First flexural crack in the control specimen was generated at the mid-span at the bottom of the beam, at a load level of about 16kN. After that several flexure and diagonal tension cracks were generated. Also bond crack was visible. As the load increased, the diagonal tension crack developed further up to the final failure.

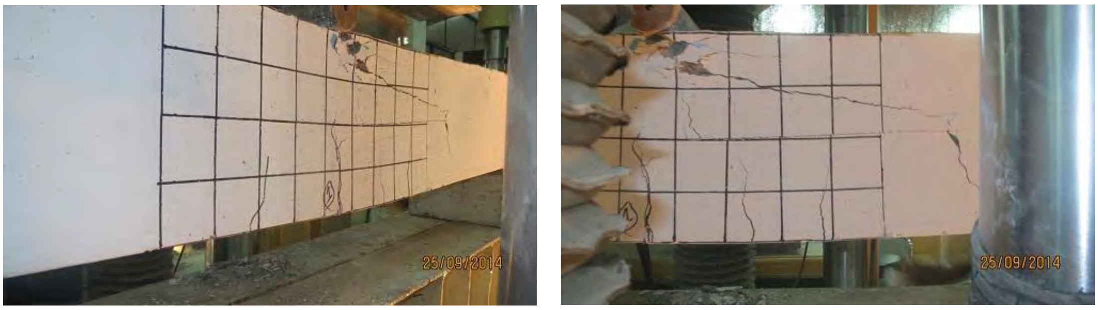

Figure 4.19 Failure Pattern of Specimen-HB47EP

Specimen HB47PC

The cracks were marked by permanent black pen and the corresponding loadings were stored automatically.First flexural crack in the control specimen was generated at the mid-span at the bottom of the beam, at a load level of about 26kN. After that several flexure and diagonal tension cracks were generated. Also bond crack was visible. As the load increased, the diagonal tension crack followed by bond crack developed further up to the final failure.

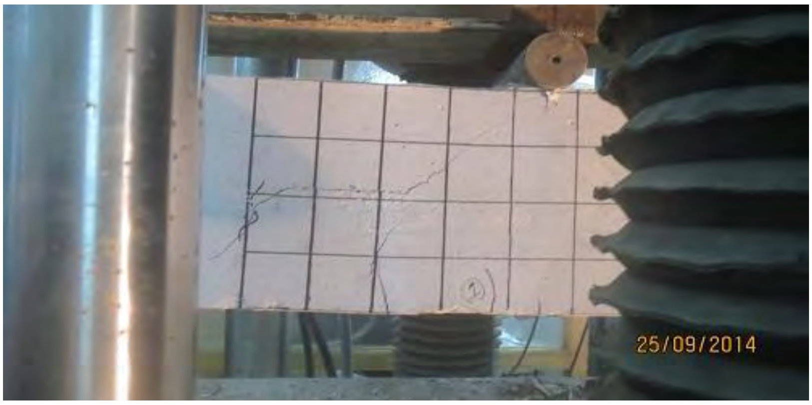

Figure 4.20 Failure Pattern of Specimen-HB47PC

4.2.3 Crack Patterns of Group C

Specimen FB5

The cracks were marked by permanent black pen and the corresponding loadings were stored automatically.First flexural crack in the control specimen was generated near the mid-span at the bottom of the beam, at a load level of about 18kN. As the load increased several flexure cracks and few diagonal tension cracks were generated as Figure 4.21.The flexural crack developed further up to the final failure.

Figure 4.21 Failure Pattern of Specimen-FB5

Specimen HB51C

The cracks were marked by permanent black pen and the corresponding loadings were stored automatically.First flexural crack in the control specimen was generated at the mid-span at the bottom of the beam, at a load level of about 15 kN. After that several flexure and diagonal tension cracks were generated. Also bond crack was visible. As the load increased, the diagonal tension crack followed by bond crack developed further up to the final failure.

Figure 4.22 Failure Pattern of Specimen-HB51C

Specimen HB51EP

At 31kN first flexural crack was initiated near the mid-span at the bottom of the beam. As the load increased, several flexure and few diagonal tension cracks were generated as Figure 5.23. The diagonal tension crack developed further up to the final failure. No bond crack was developed.

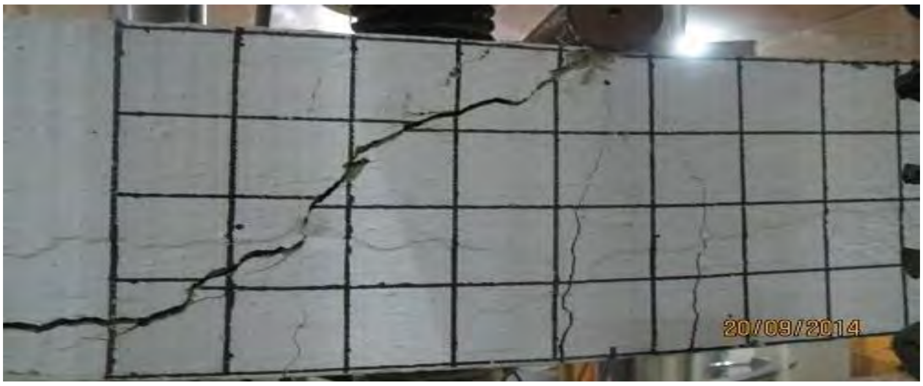

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Figure 4.23 Failure Pattern of Specimen-HB51EP

Specimen HB51PC

The cracks were marked by permanent black pen and the corresponding loadings were stored automatically.First flexural crack in the control specimen was generated at the mid-span at the bottom of the beam, at a load level of about 31kN. After that several flexure and diagonal tension cracks were generated. No bond crack was visible. As the load increased, the diagonal tension crack developed further up to the final failure.

Figure 4.24 Failure Pattern of Specimen-HB51PC

Specimen HB53C

The cracks were marked by permanent black pen and the corresponding loadings were stored automatically.First flexural crack in the control specimen was generated at the mid-span at the bottom of the beam, at a load level of about 23kN. After that several flexure and diagonal tension cracks were generated. Also bond crack was visible. As the load increased, the diagonal tension crack followed by large bond crack developed further up to the final failure.

Figure 4.25 Failure Pattern of Specimen-HB53C

Specimen HB53EP

The cracks were marked by permanent black pen and the corresponding loadings were stored automatically.First flexural crack in the control specimen was generated at the mid-span at the bottom of the beam, at a load level of about 30kN. After that several flexure and diagonal tension cracks were generated. Also bond crack was visible. As the load increased, the diagonal tension crack followed by bond crack developed further up to the final failure.

Figure 4.26 Failure Pattern of Specimen-HB53EP

Specimen HB53PC

The cracks were marked by permanent black pen and the corresponding loadings were stored automatically.First flexural crack in the control specimen was generated at the mid-span at the bottom of the beam, at a load level of about 28kN. After that several flexure cracks were generated. Also bond crack was visible. As the load increased, the flexure cracks developed further up to the final failure.

Figure 4.27 Failure Pattern of Specimen-HB53PC

Specimen HB57C

The cracks were marked by permanent black pen and the corresponding loadings were stored automatically.First flexural crack in the control specimen was generated at the mid-span at the bottom of the beam, at a load level of about 15 kN. After that several flexure and diagonal tension cracks were generated. Also bond crack was visible. As the load increased, the diagonal tension crack followed by bond crack developed further up to the final failure.

Figure 4.28 Failure Pattern of Specimen-HB57C

Specimen HB57EP

The cracks were marked by permanent black pen and the corresponding loadings were stored automatically.First flexural crack in the control specimen was generated at the mid-span at the bottom of the beam, at a load level of about 22 kN. After that several flexure and diagonal tension cracks were generated. Also bond crack was visible. As the load increased, the diagonal tension crack followed by bond crack developed further up to the final failure.

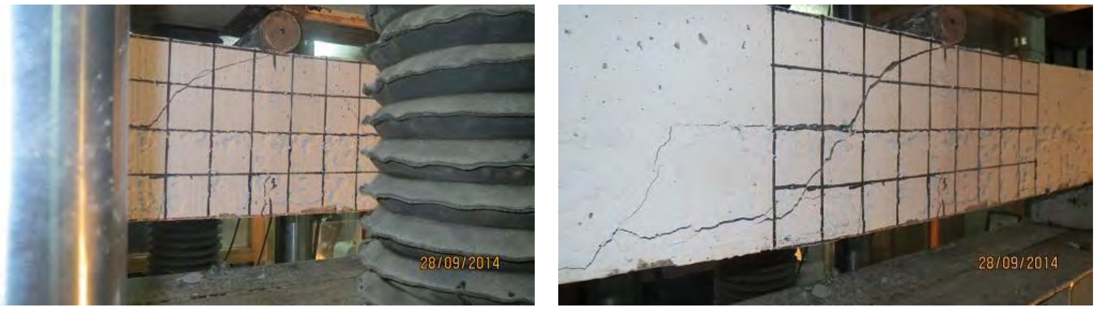

Figure 4.29 Failure Pattern of Specimen-HB57EP

Specimen HB57PC

The cracks were marked by permanent black pen and the corresponding loadings were stored automatically.First flexural crack in the control specimen was generated at the mid-span at the bottom of the beam, at a load level of about 24 kN. After that several flexure and diagonal tension cracks were generated. Also bond crack was visible in a small portion. As the load increased, the diagonal tension crack developed further up to the final failure.

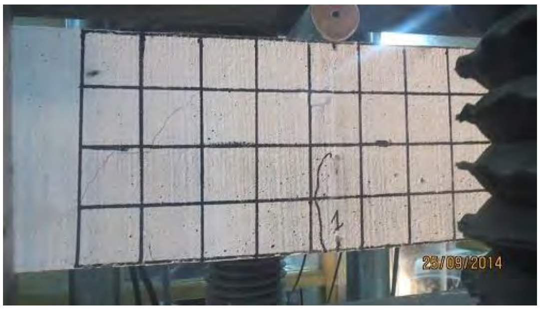

Figure 4.30 Failure Pattern of Specimen-HB57PC

4.3 LOAD DEFLECTION RESPONSE OF SPECIMENS

4.3.1. Load Deflection Behavior of Beam

Time vs load was continuously monitored and data was saved automatically .Deflection of the beams were also monitored with a video extensometer and time versus deflection data was stored continuously .These data are then combined to prepare load versus deflection curves which are the basis of analysis. The test continued till the specimen reached to the final failure.

Load vs. Deflection curves have been drawn for all the specimens to investigate their structural behaviour and to compare the specimens cast in two phases with the control specimen as shown in Figure 4.31 to Figure 4.48. For better justification at first group A (concrete strength 3ksi) has been compared with the control specimen as shown in Figure 4.31 to Figure 4.36. Then group B (concrete strength 4ksi) has been compared with the control specimen as shown in Figure 4.37 to Figure 4.42 .And finally group C (concrete strength 5ksi) has been compared with the control specimen as shown in Figure 4.43 to Figure 4.48.The load deflection curve for variation of strength among the three groups is also plotted in Figure 4.49 .When testing the beam for concrete strength 3ksi without using any bonding agent cast after one day (HB31C), the machine was malfunctioned. So, the testing result for this beam is not sufficient.

For Group A

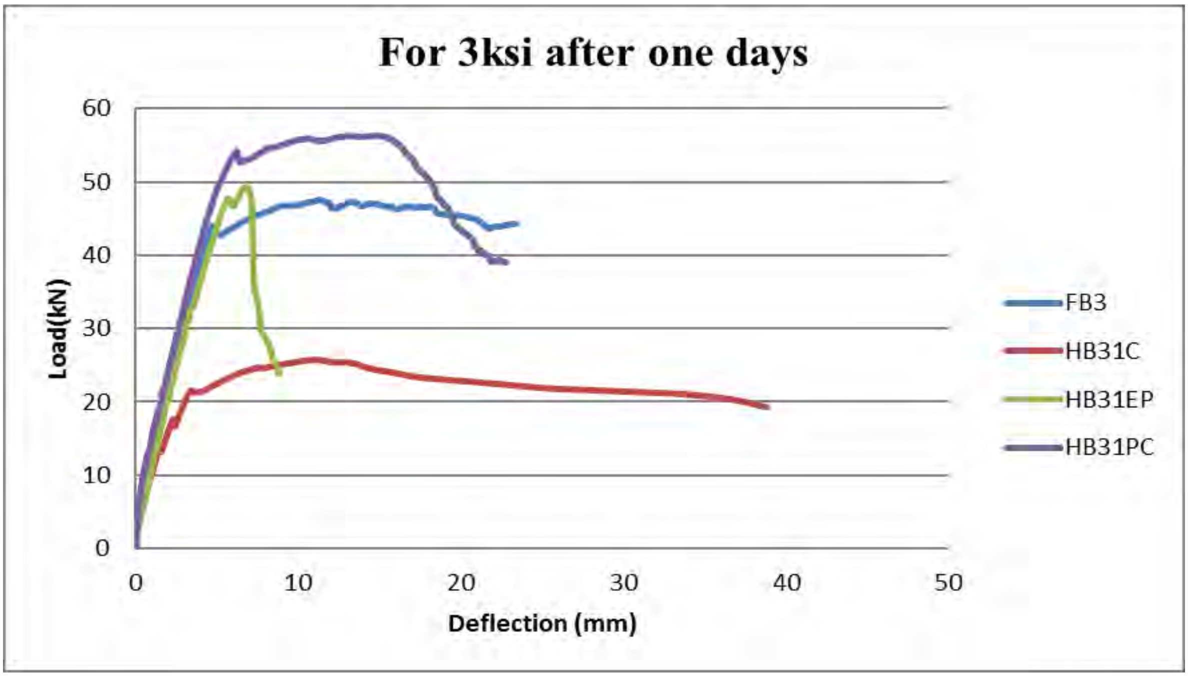

Figure 4.31 Comparison between Fully Cast Beam and Beams Cast in Two Phases after One Day

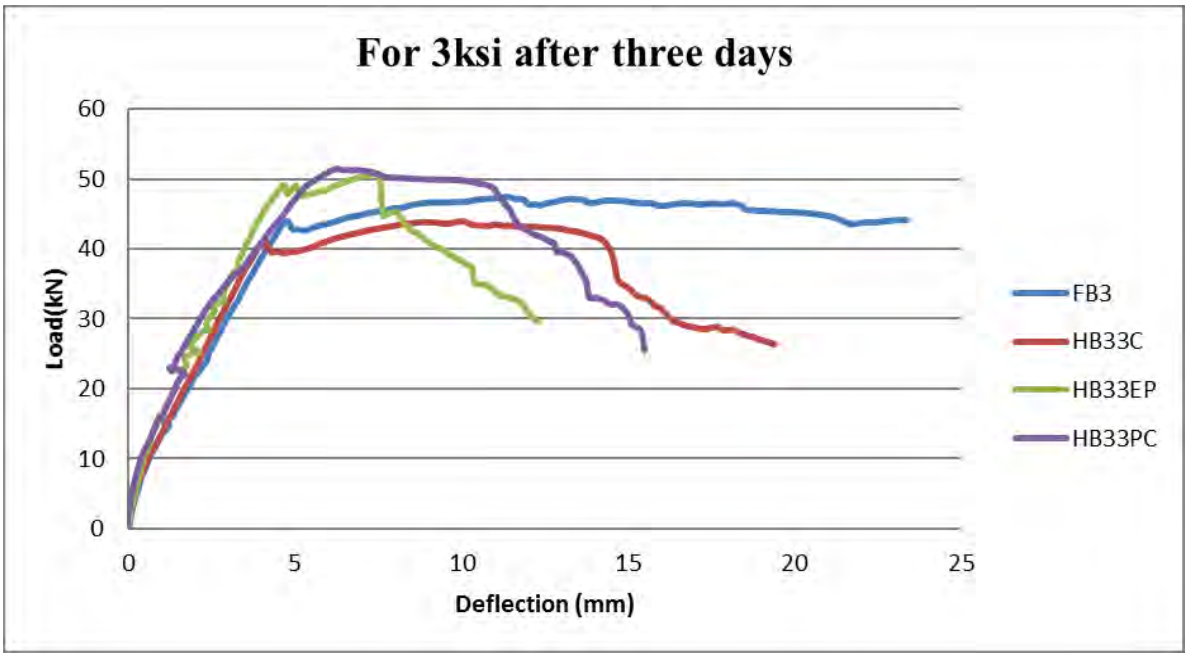

Figure 4.32 Comparison between Fully Cast Beam and Beams Cast in Two Phases after Three Days

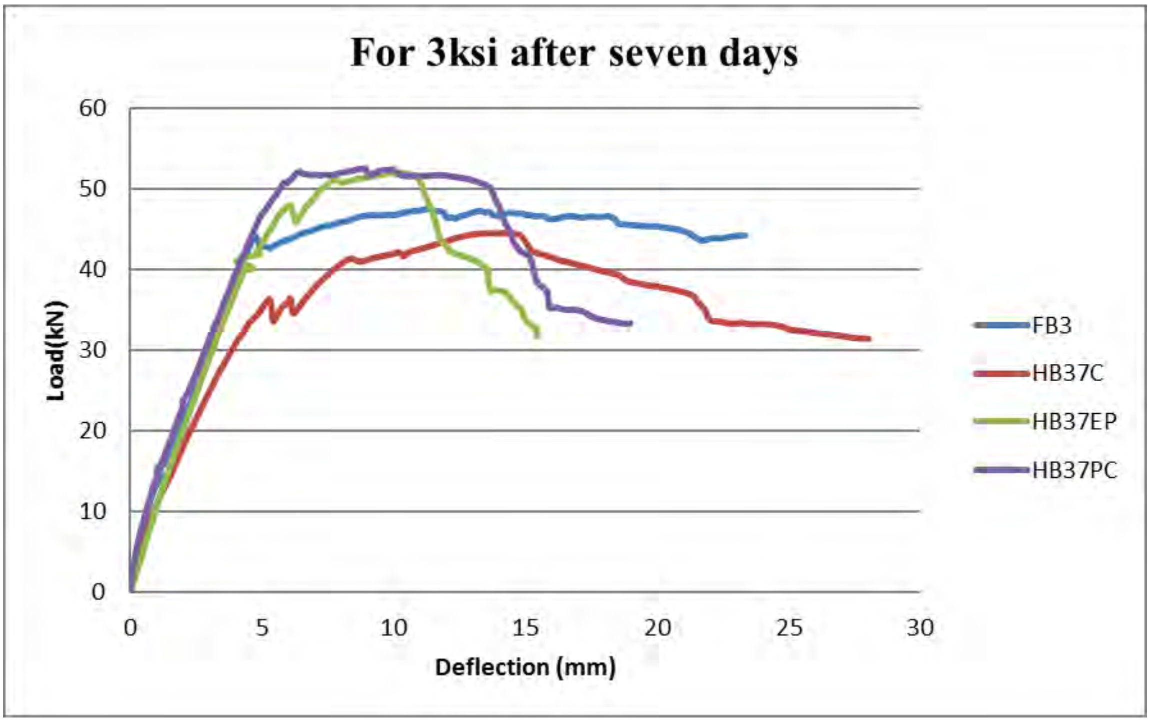

Figure 4.33 Comparison between Fully Cast Beam and Beams Cast In Two Phases after Seven Days

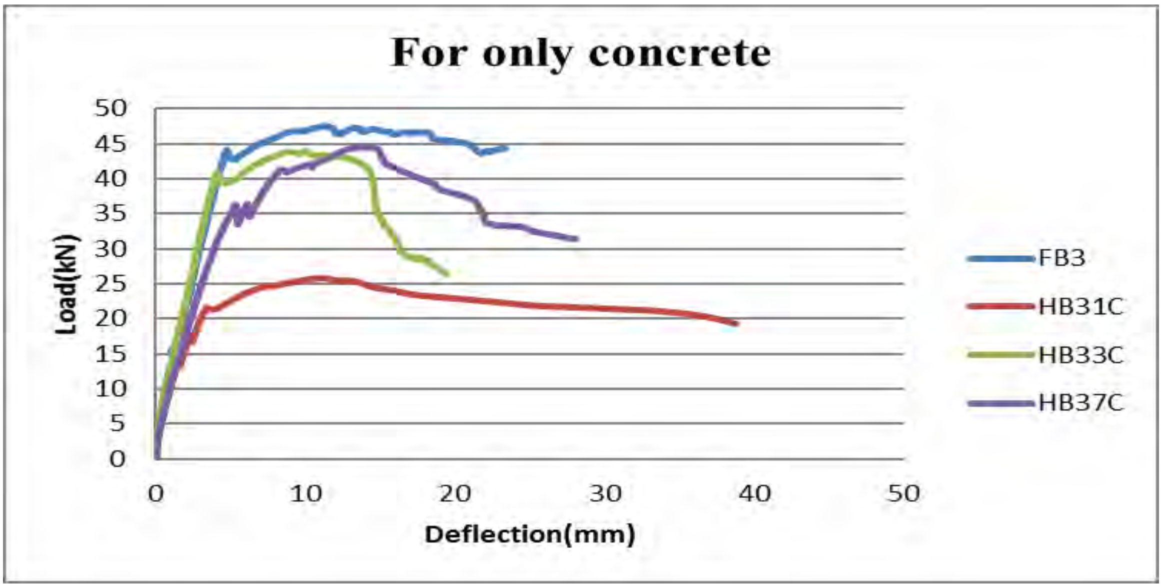

Figure 4.34 Comparison between Fully Cast Beam And Beams Cast In Two Phases After One Day, Three Days And Seven Days Without Using Bonding Agent

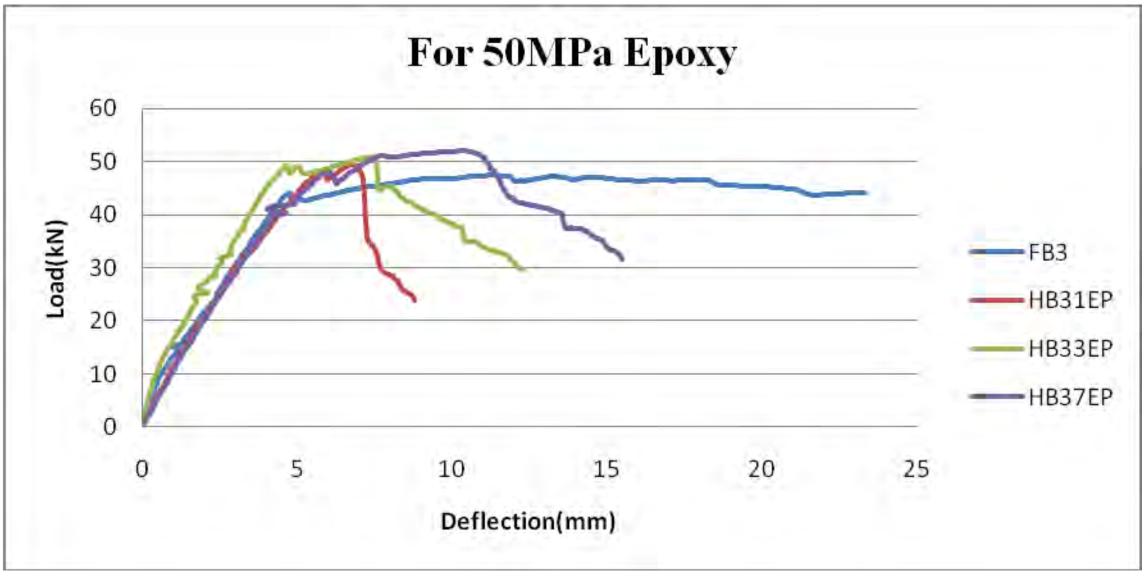

Figure 4.35 Comparison between Fully cast Beam and Beams Cast in Two Phases after One Day, Three Days and Seven Days with Using Bonding Agent 50MPa Epoxy

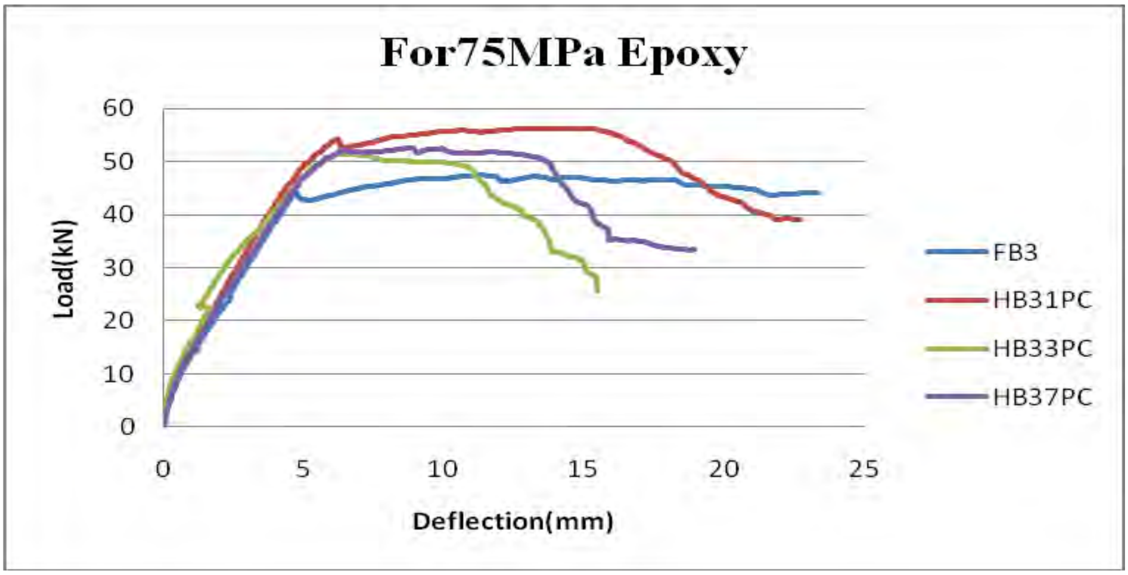

Figure 4.36 Comparison between Fully Cast Beam And Beams Cast In Two Phases After One Day, Three Days And Seven Days With Using Bonding Agent 75MPa Epoxy

By analyzing the above figures it is found that load deflection behavior of beams for all the specimens is very much similar to the control specimen within the elastics range and exhibits a linear relationship.

When load is further increased, the specimen loses elasticity and becomes plastic. At the point of maximum load, ultimate load or ultimate strength, specimens began to yield and fractured unless load was substantially reduced.

The load defection curve for beams cast in two phases after one day, three days and seven days by using bonding agent 50MPa Epoxy and 75MPa Epoxy are always higher than the control specimen(fully cast at a time, FB3) as shown in figures 4.31 to figure 4.33.And the curves for these beams when cast after one day, three days and seven days are almost similar as shown in figures 4.35 and figure 4.36.So, effect of applying epoxy with the variation of days are insignificant once the concrete is set. On the other hand, the load defection curve for beams cast in two phases without using any bonding agent are lower than the control specimen and other two beams cast by using bonding agent as shown in figures 4.31 to figure 4.33. The load deflection curves for these beams (cast in two phases without any bonding agent) are moving higher when they cast after one day, three days and seven days as shown in figure 4.34. By analyzing the load-deflection curve it is also found that 75MPa Epoxy is better as a bonding agent than 50MPa Epoxy.

For Group B

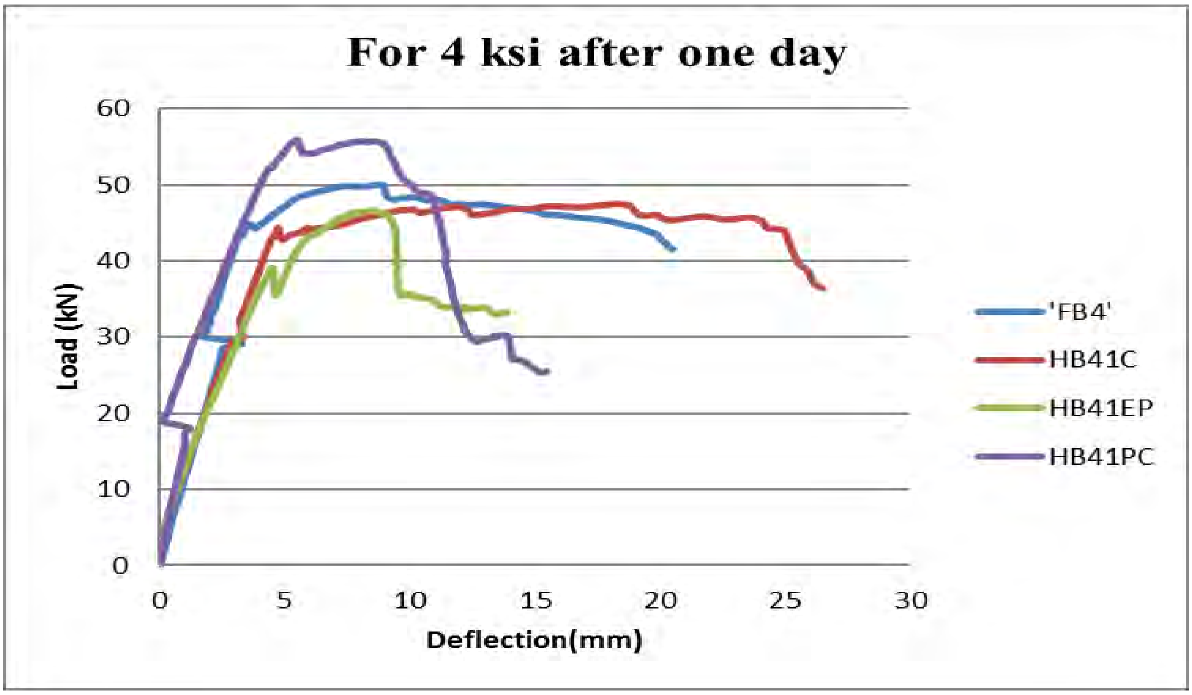

Figure 4.37 Comparison between Fully Cast Beam and Beams Cast In Two Phases after One Day

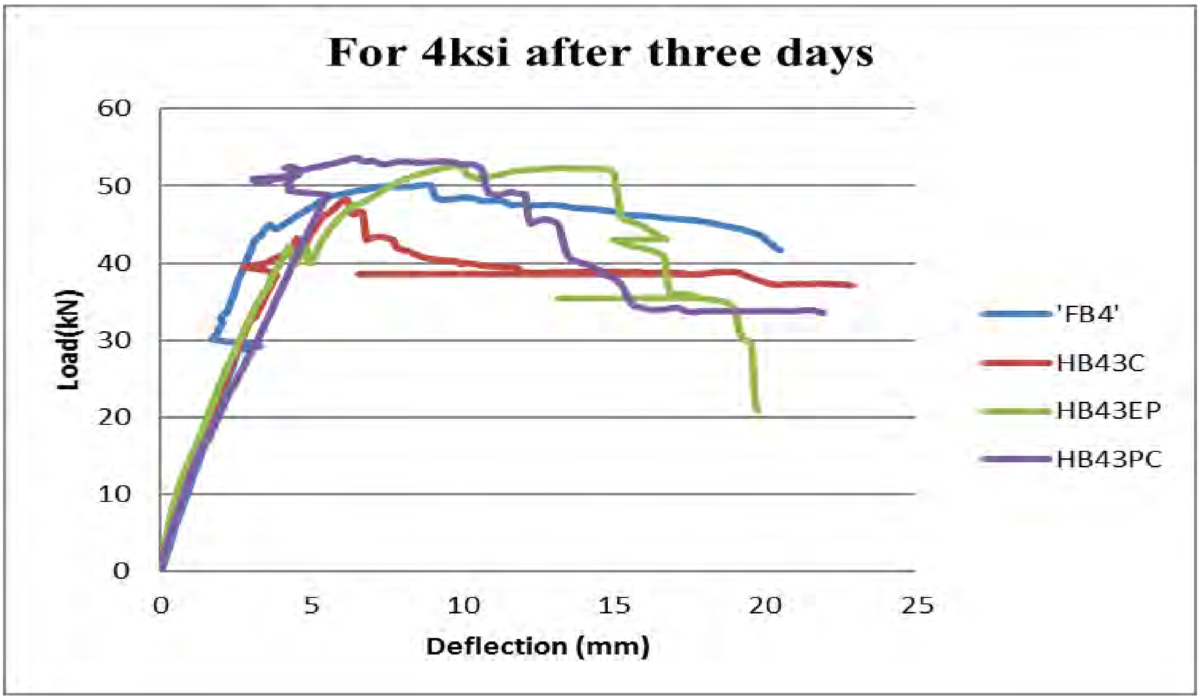

Figure 4.38 Comparison between Fully Cast Beam and Beams Cast In Two Phases after Three Days

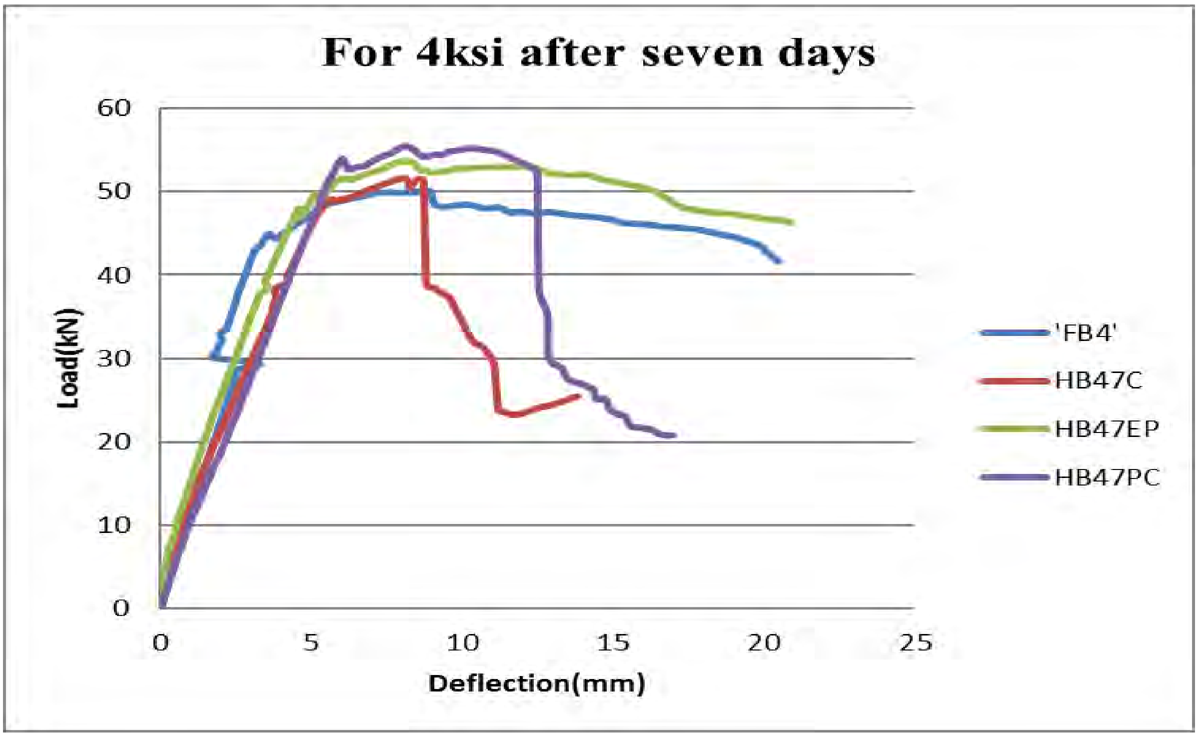

Figure 4.39 Comparison between Fully Cast Beam and Beams Cast In Two Phases after Seven Days

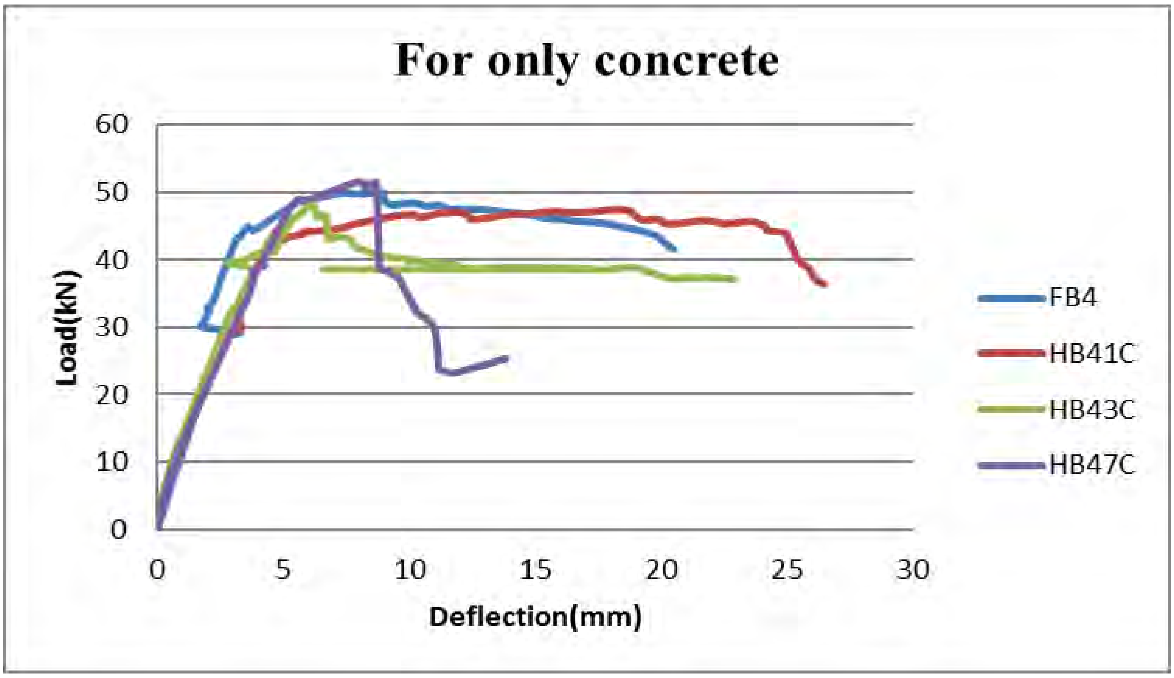

Figure 4.40 Comparison between Fully Cast Beam and Beams Cast in Two Phases after One Day, Three Days and Seven Days Without Using Bonding Agent

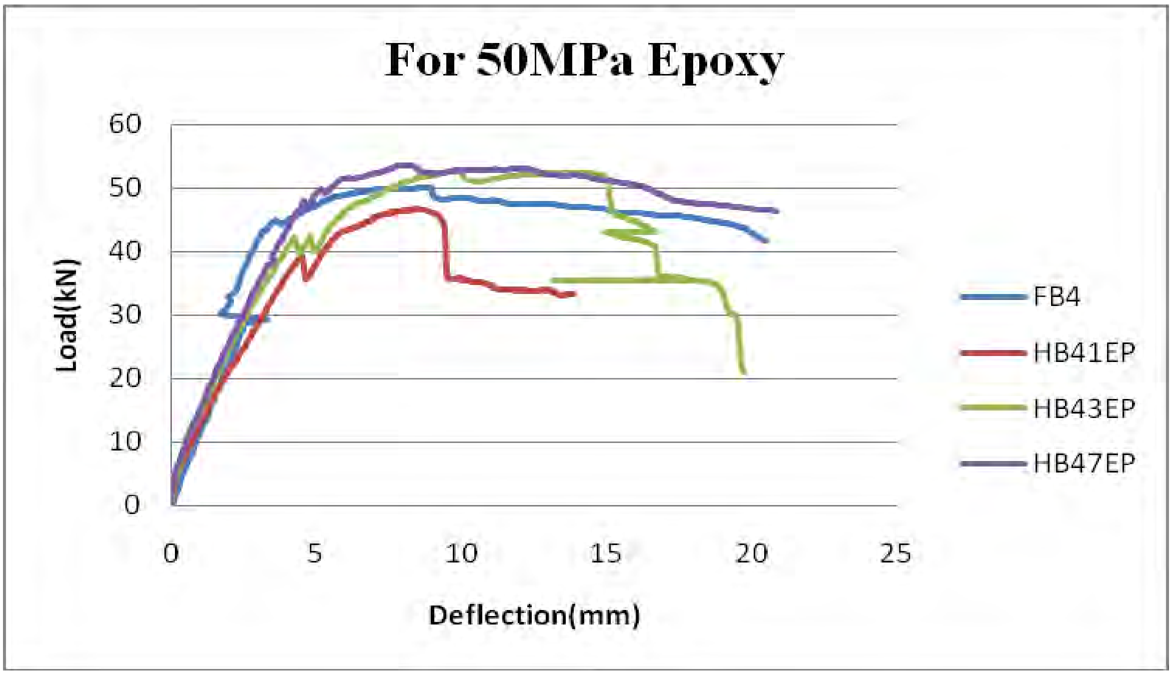

Figure 4.41 Comparison between Fully Cast Beam and Beams Cast in Two Phases after One Day, Three Days and Seven Days With Using Bonding Agent 50MPa Epoxy

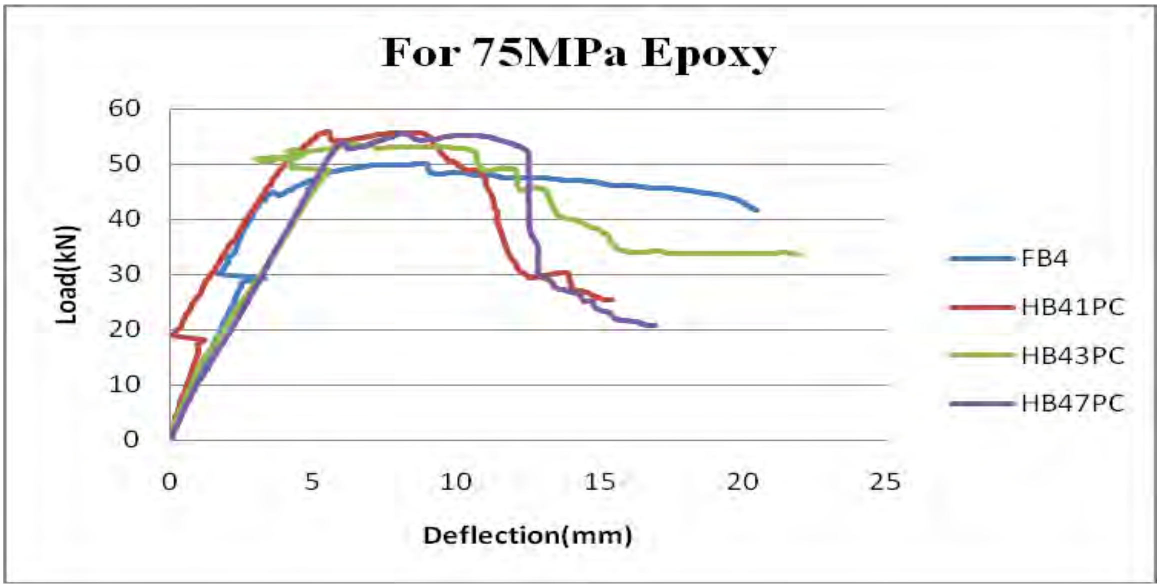

Figure 4.42 Comparison between Fully Cast Beam and Beams Cast in Two Phases after One Day, Three Days and Seven Days With Using Bonding Agent 75MPaEpoxy

By analyzing the above figures it is found that load deflection behavior of beams for all the specimens is similar within the elastics range and exhibits a linear relationship.

When load is further increased, the specimens loses elasticity and becomes plastic (i.e when the load is removed the deformation caused by deflection will be more or less permanent). At the point of maximum load, ultimate load or ultimate strength, specimens began to yield and fractured unless load was substantially reduced.

The load defection curve for beams cast in two phases after one day, three days and seven days by using bonding agent 50MPa Epoxy and 75MPa Epoxy are always higher than the control specimen (fully cast at a time, FB4) as shown in figures 4.37 to figure 4.39.And the curves for the beams cast in two phases by using 50MPa Epoxy and 75MPa Epoxy when cast after one day, three days and seven days are almost similar patterns as shown in figures 4.41 and figure 4.42. So, effect of applying epoxy with the variation of days are insignificant once the concrete is set .On the other hand, the load defection curve for beams cast in two phases without using any bonding agent are lower than the control specimen and other two beams cast by using bonding agent as shown in figures 4.37 to figure 4.39. The load deflection curves for these beams (cast in two phases without any bonding agent) are moving higher when they cast after one day, three days and seven days as shown in figure 4.40. By analyzing the load-deflection curve it is also found that 75MPa Epoxy is better as a bonding agent than 50MPa Epoxy.

For Group C

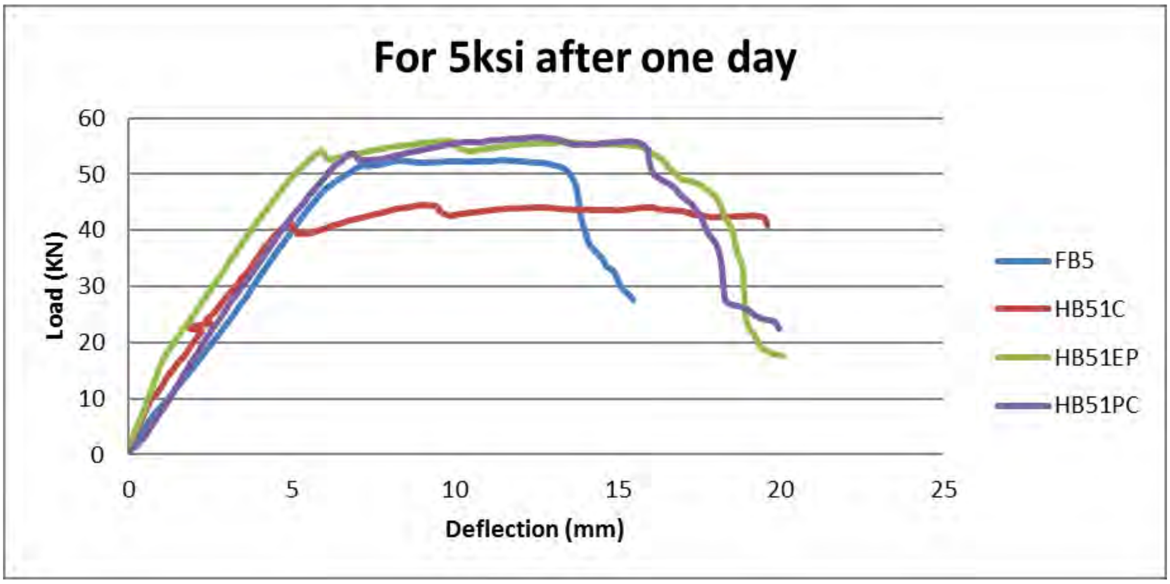

Figure 4.43 Comparison between Fully Cast Beam and Beams Cast in Two Phases after One Day

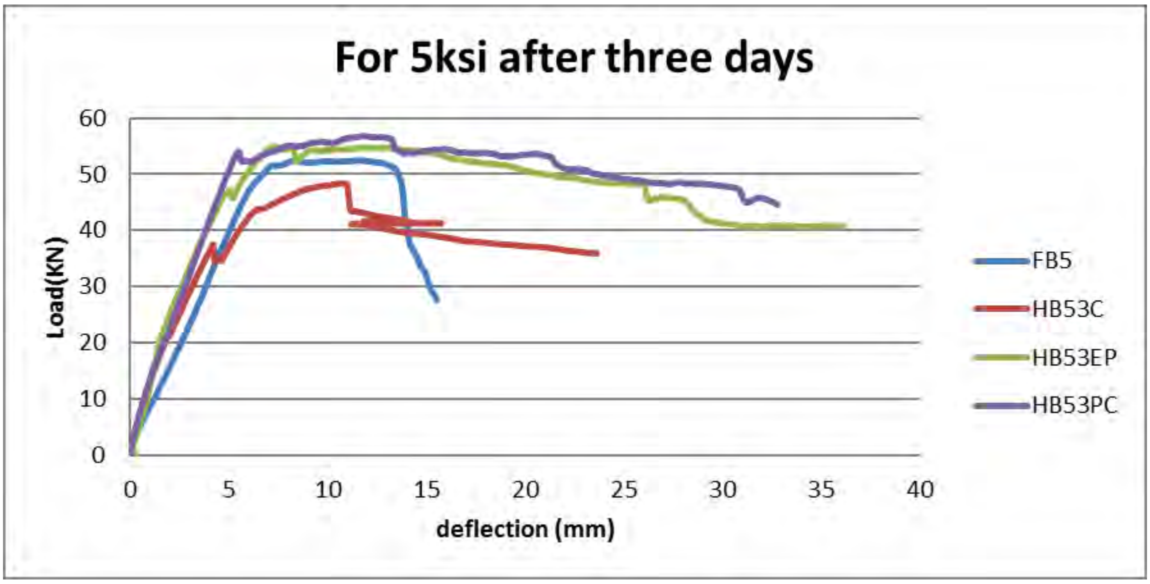

Figure 4.44 Comparison between Fully Cast Beam and Beams Cast in Two Phases after Three Days

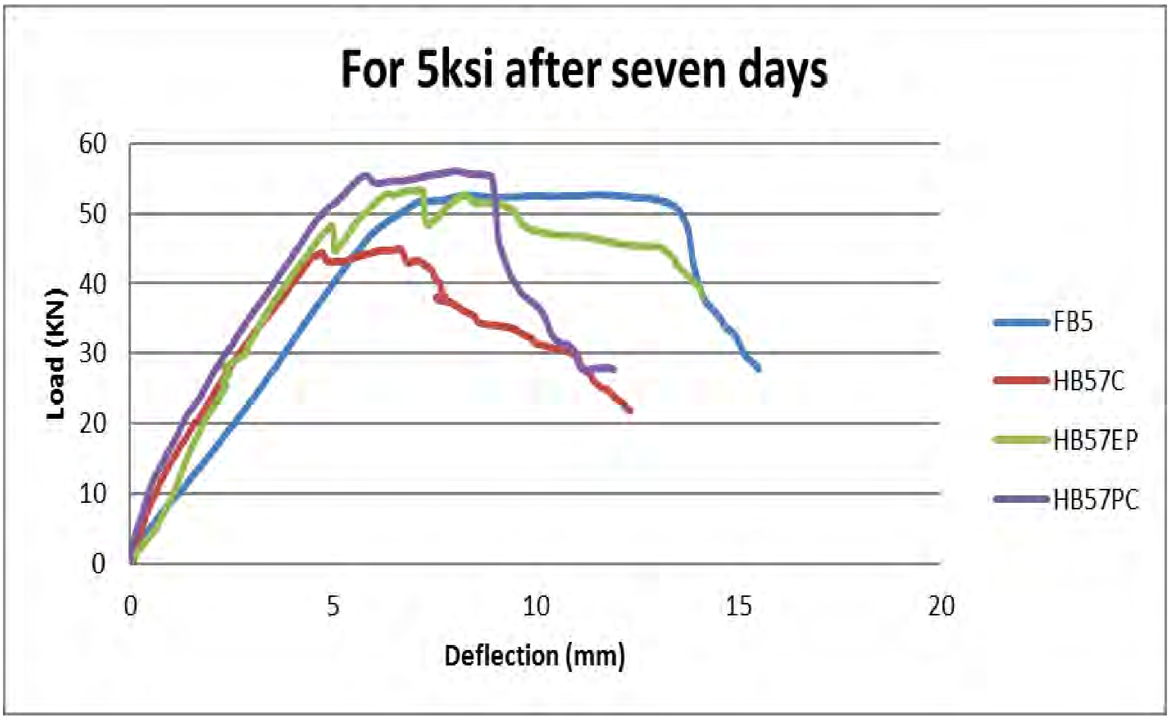

Figure 4.45 Comparison between Fully Cast Beam and Beams Cast in Two Phases after Seven Days

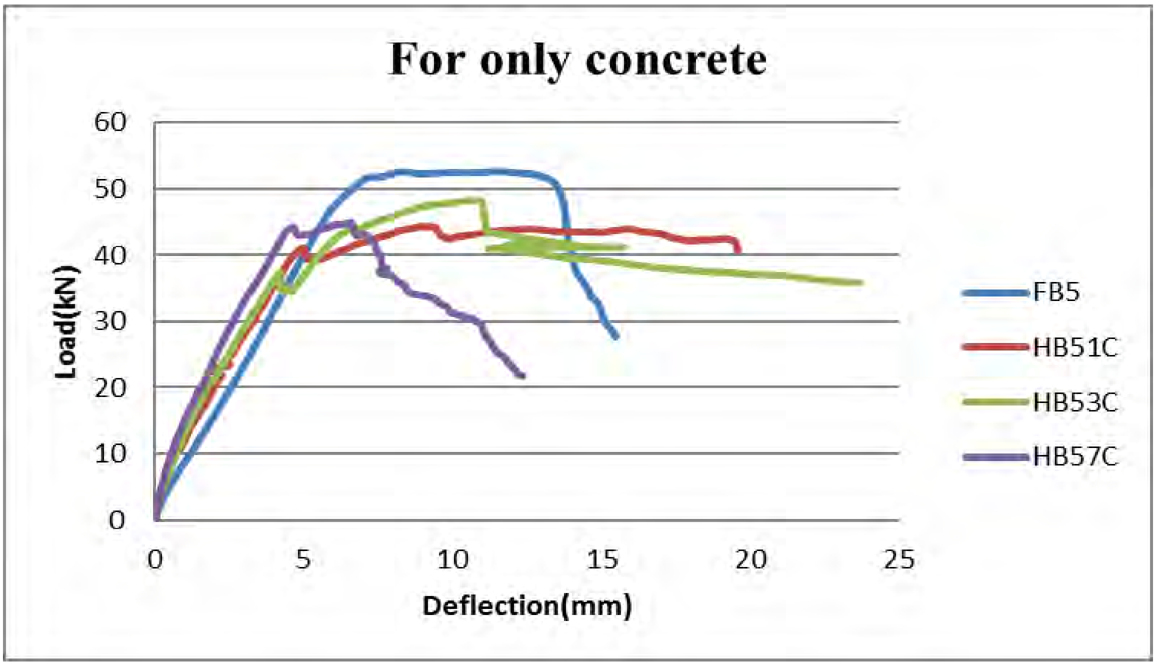

Figure 4.46 Comparison between Fully Cast Beam And Beams Cast in Two Phases After One Day, Three Days and Seven Days Without Using Bonding Agent

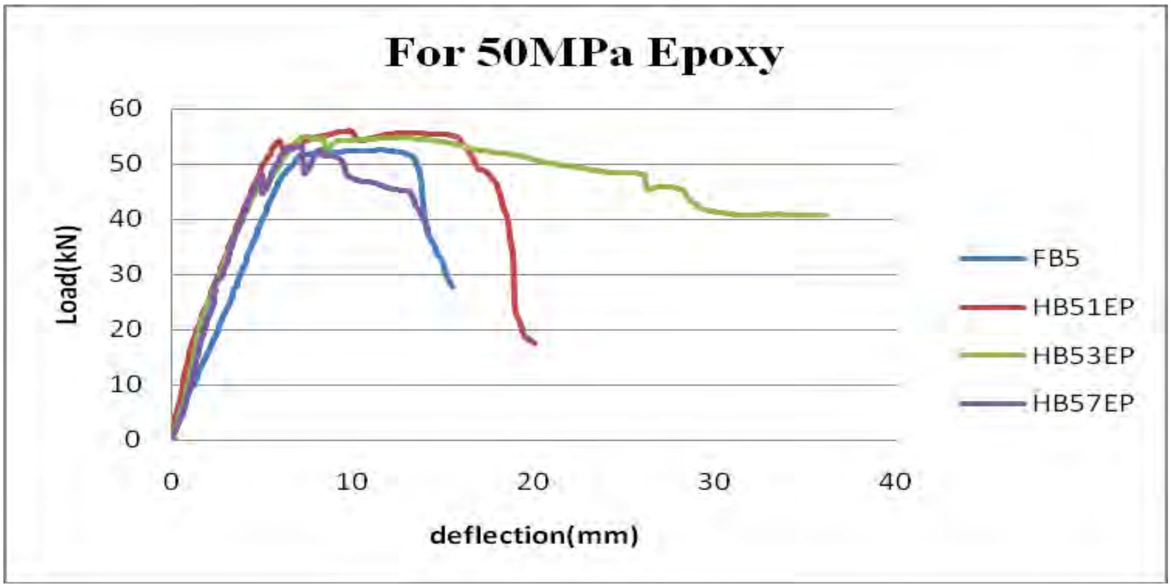

Figure 4.47 Comparison between Fully Cast Beam And Beams Cast in Two Phases After One Day, Three Days and Seven Days With Using Bonding Agent 50MPa Epoxy

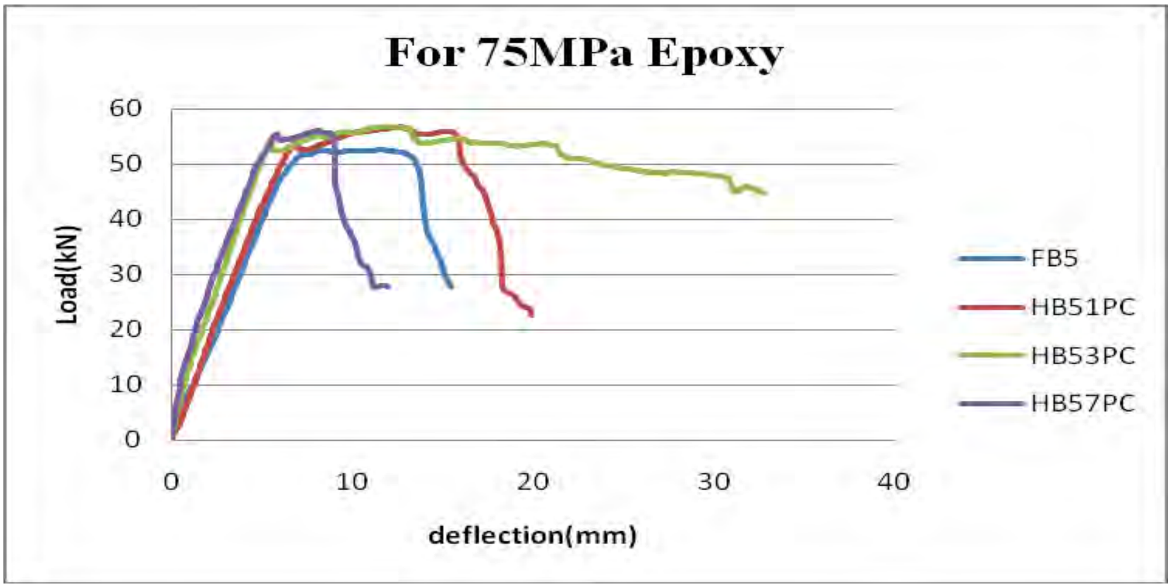

Figure 4.48 Comparison between Fully Cast Beam And Beams Cast in Two Phases After One Day, Three Days and Seven Days With Using Bonding Agent 75MPa Epoxy

By analyzing the above figures it is found that load deflection behavior of beams for all the specimens is similar within the elastics range and exhibits a linear relationship.

When load is further increased, the specimens loses elasticity and becomes plastic (i.e when the load is removed the deformation caused by deflection will be more or less permanent). At the point of maximum load, ultimate load or ultimate strength, specimens began to yield and fractured unless load was substantially reduced.

The load defection curve for beams cast in two phases after one day, three days and seven days by using bonding agent 50MPa Epoxy and 75MPa Epoxy are always higher than the control specimen(fully cast at a time, FB5) as shown in figures 4.43 to figure 4.45. And the curves for these beams when cast after one day, three days and seven days are almost similar as shown in figures 4.47 and figure 4.48. So, effect of applying epoxy with the variation of days are insignificant once the concrete is set On the other hand, the load defection curve for beams cast in two phases without using any bonding agent are lower than the control specimen and other two beams cast by using bonding agent as shown in figures 4.43 to figure 4.45. The load deflection curves for these beams (cast in two phases without any bonding agent) are moving higher when they cast after one day, three days and seven days except in case of curve HB57C as shown in figure 4.46. By analyzing the load-deflection curve it is also found that 75MPa Epoxy is better as a bonding agent than 50MPa Epoxy.

4.3.2 Strength Variation

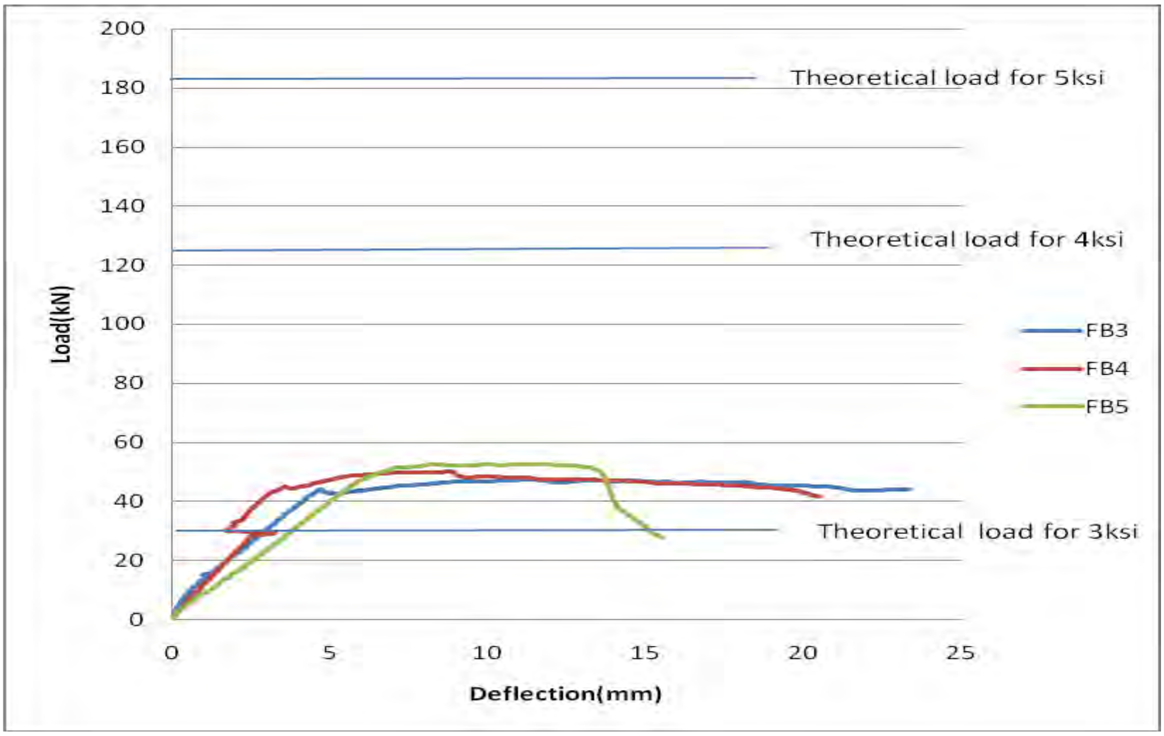

Figure 4.49 Strength Variation of Control Specimen (Fully Cast At A Time)

From the above figure it is observed that experimental loads are less than theoretical loads except in case of 3ksi. Practically it was quiet difficult to maintain the concrete strength of 3 ksi using stone chips. So instead of getting 3ksi for fully cast beam at a time, almost 3.8 ksi was obtained, which is close to 4ksi.Where as for 4ksi, it was 4.4 ksi and for 5 ksi it was 5.3 ksi.

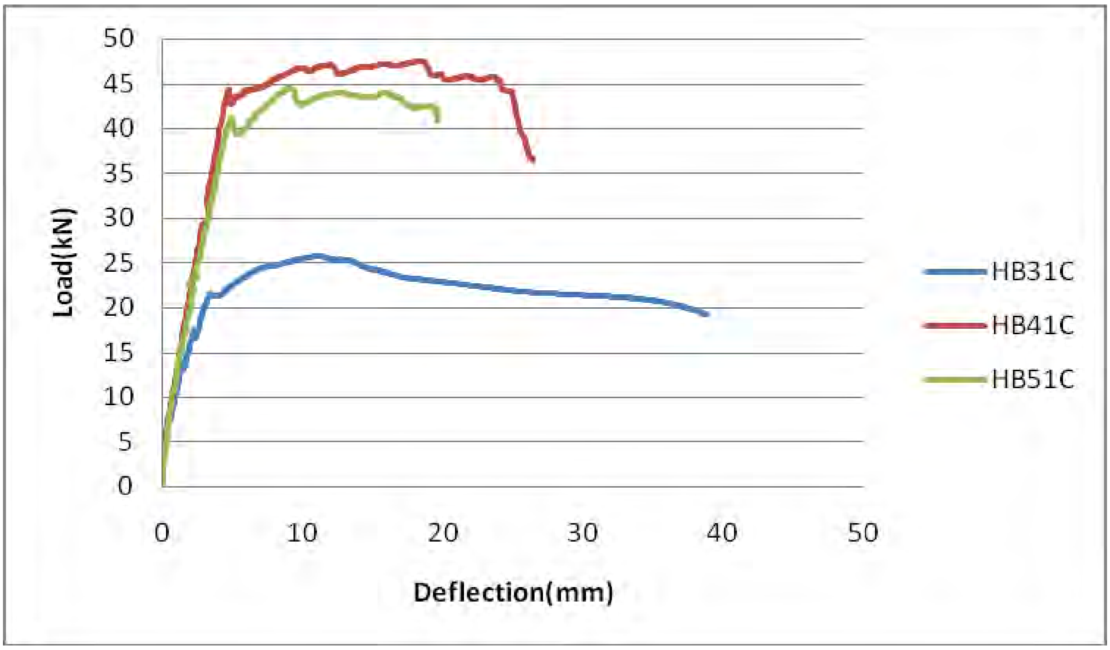

Figure 4.50 Comparison of Strength among the Beams When Beams Cast in Two Phases after Three Days Without Using Any Bonding Agent

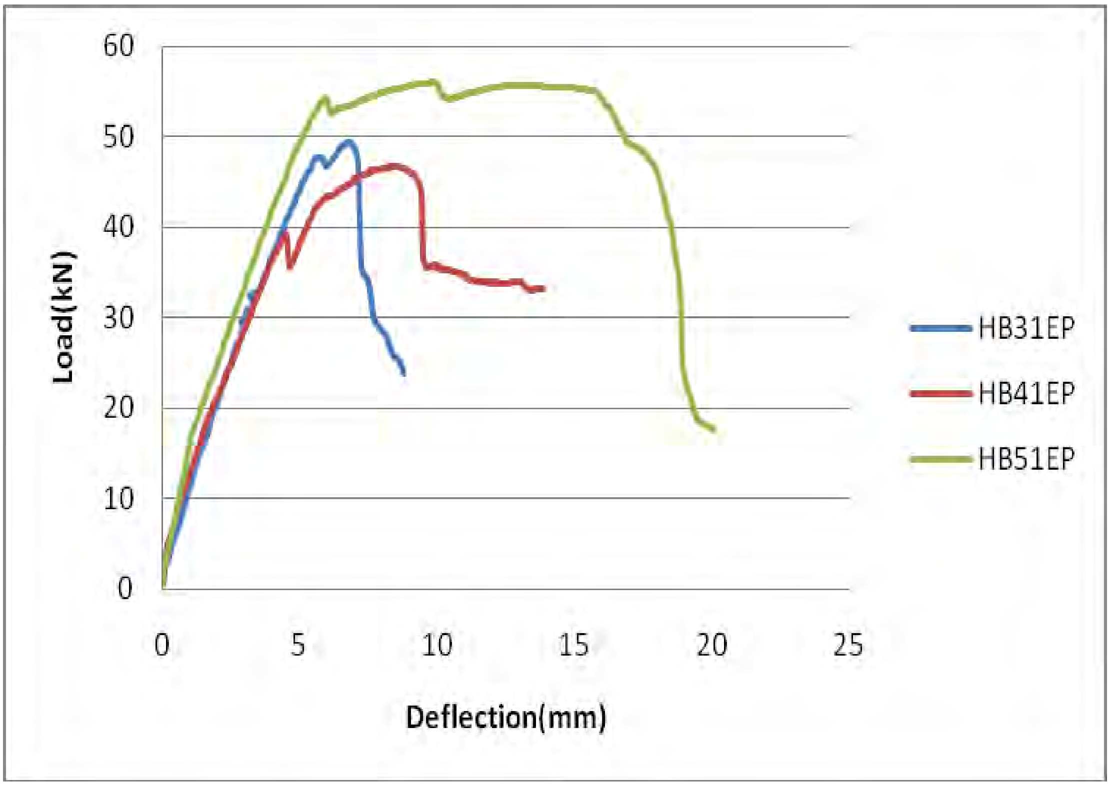

Figure 4.51 Comparison of Strength among the Beams When Beams Cast in Two Phases after One Day by Using Bonding Agent 50PMa Epoxy

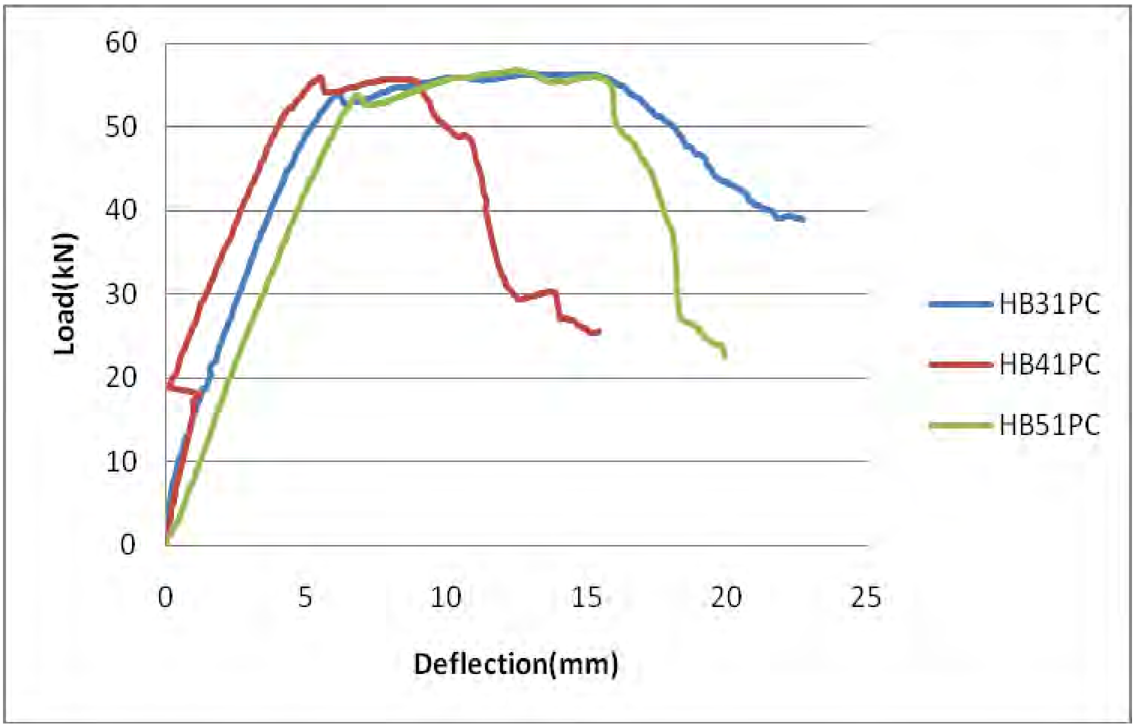

Figure 4.52 Comparison of Strength among the Beams When Beams Cast in Two Phases after One Day by Using Bonding Agent 75MPa Epoxy

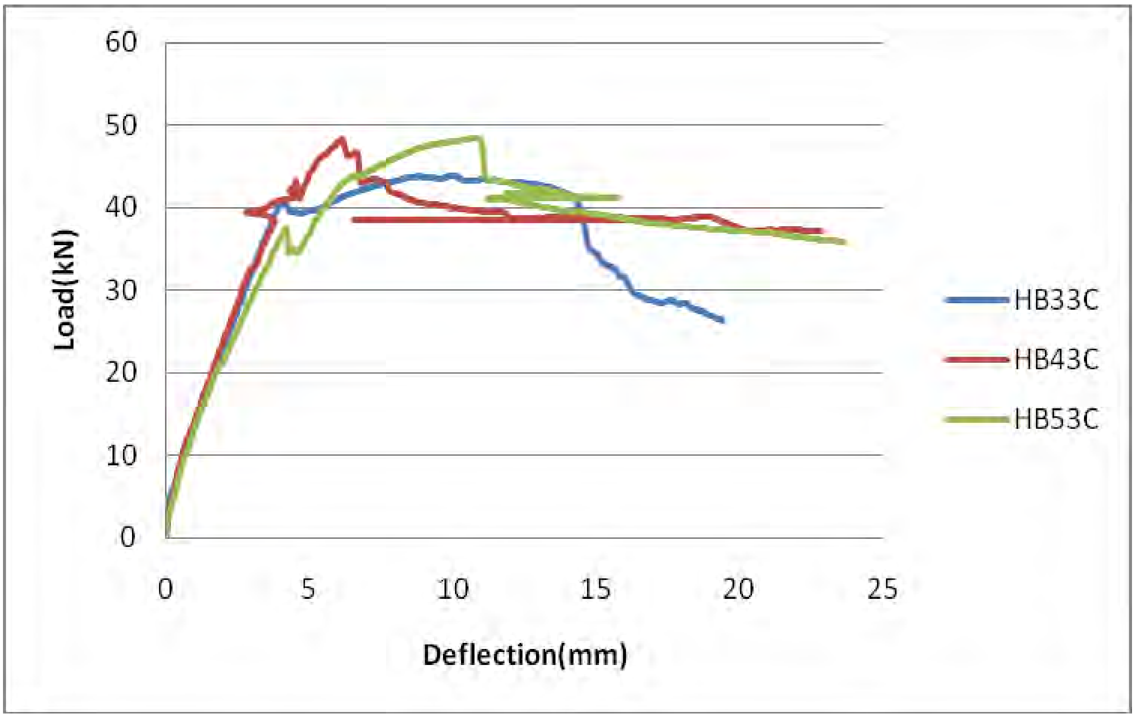

Figure 4.53 Comparison of Strength among the Beams When Beams Cast in Two Phases after Three Days Without Using Any Bonding Agent

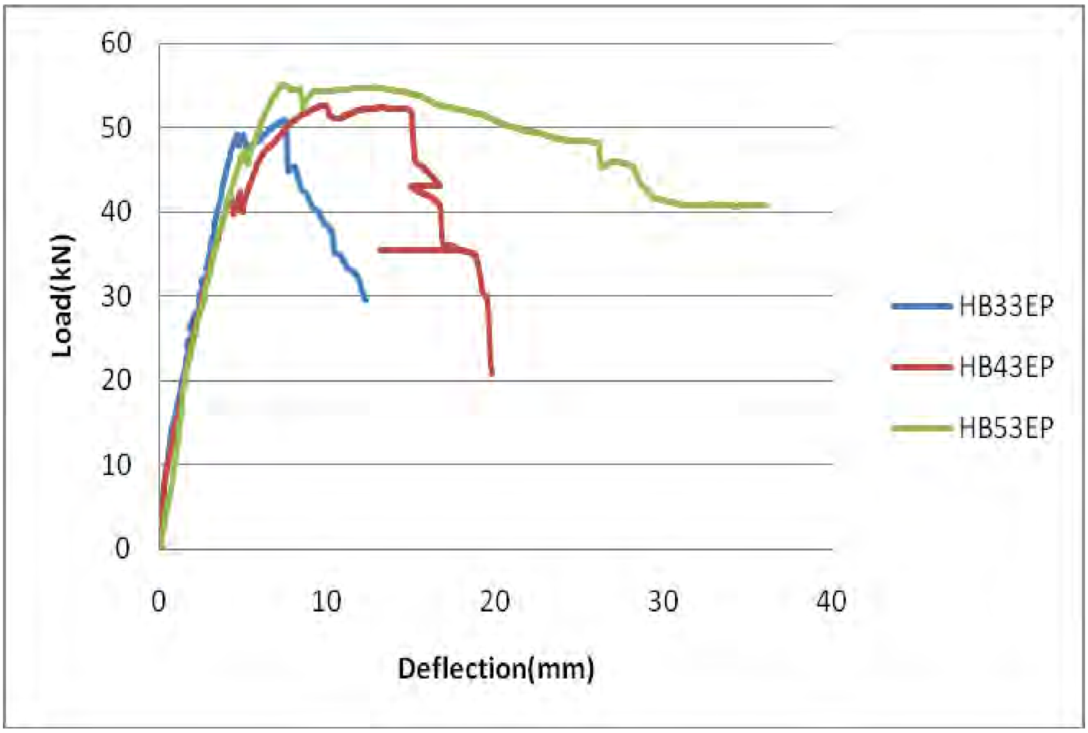

Figure 4.54 Comparison of Strength among the Beams When Beams Cast in Two Phases after Three Days by Using Bonding Agent 50MPa Epoxy

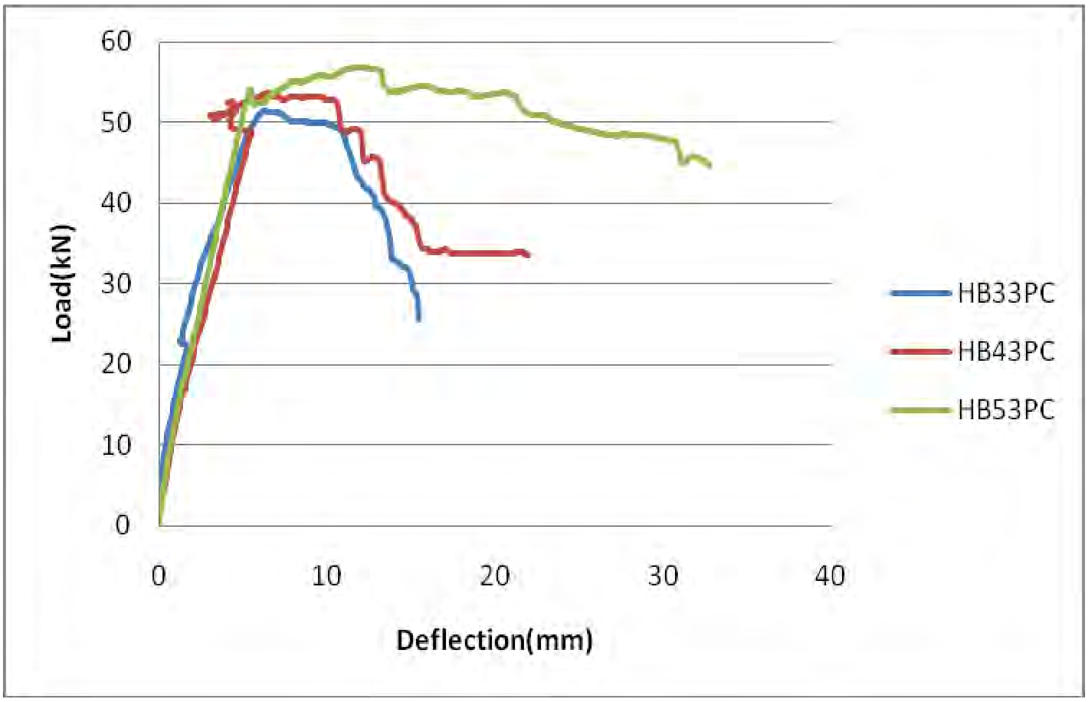

Figure 4.55 Comparison of Strength among the Beams When Beams Cast in Two Phases after Three Days by Using Bonding Agent 75MPa Epoxy

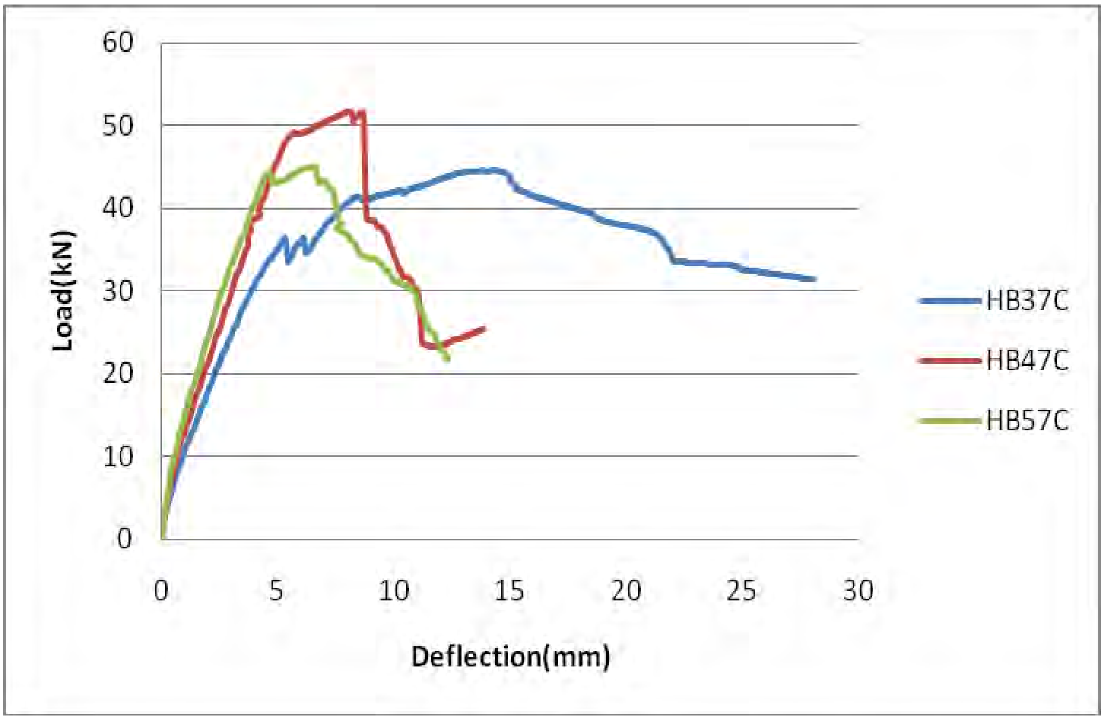

Figure 4.56 Comparison of Strength among the Beams When Beams Cast in Two Phases after Three Days Without Using Any Bonding Agent

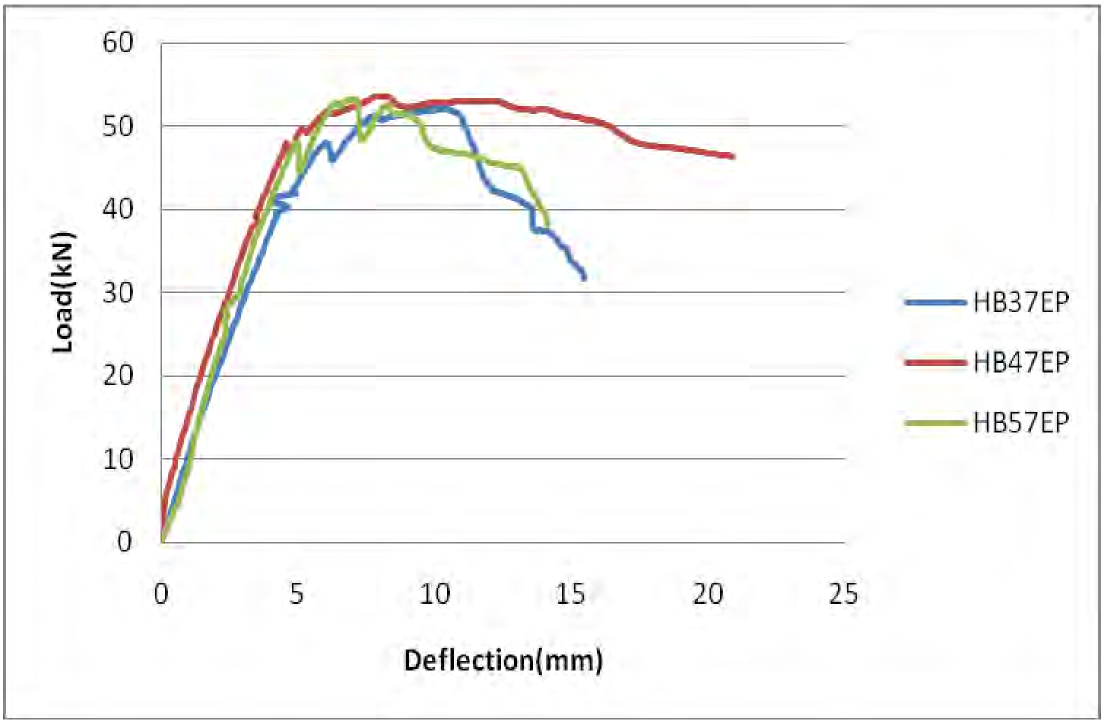

Figure 4.57 Comparison of Strength among the Beams When Beams Cast in Two Phases after Seven Days by Using Bonding Agent 50MPa Epoxy

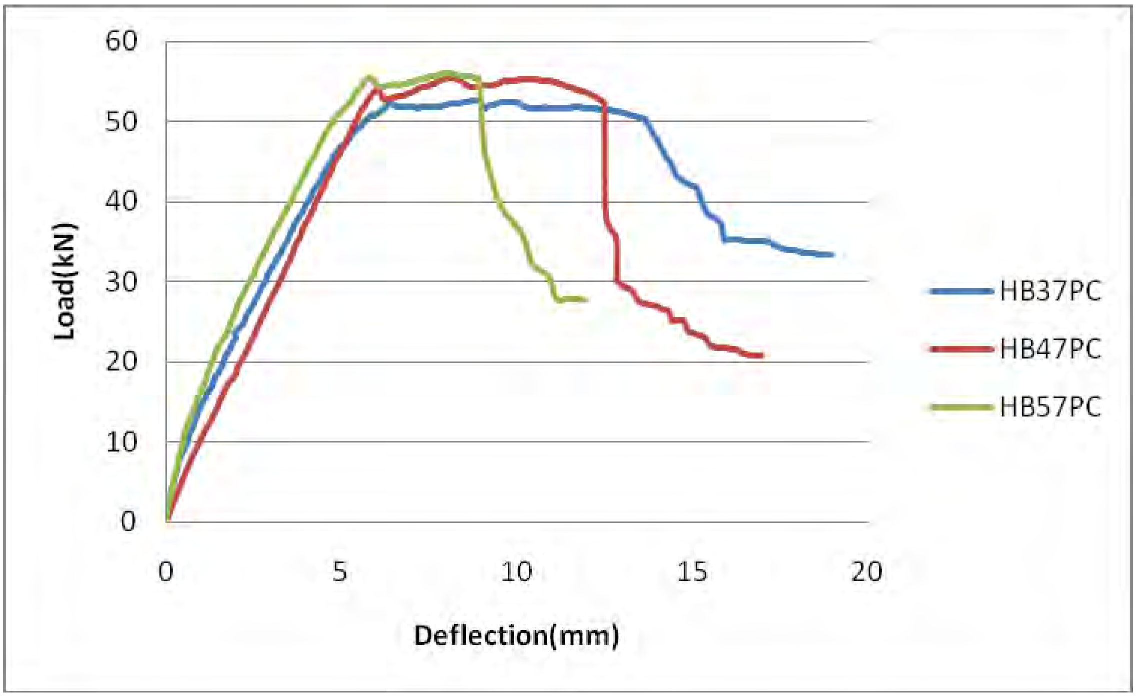

Figure 4.58 Comparison of Strength among the Beams When Beams Cast in Two Phases after Seven Days by Using Bonding Agent 75mpa Epoxy

4.4 EFFECT OF BONDING AGENT ON ULTIMATE LOAD

4.4.1 Effect of Bonding Agent on Ultimate Load with Variation of Strength

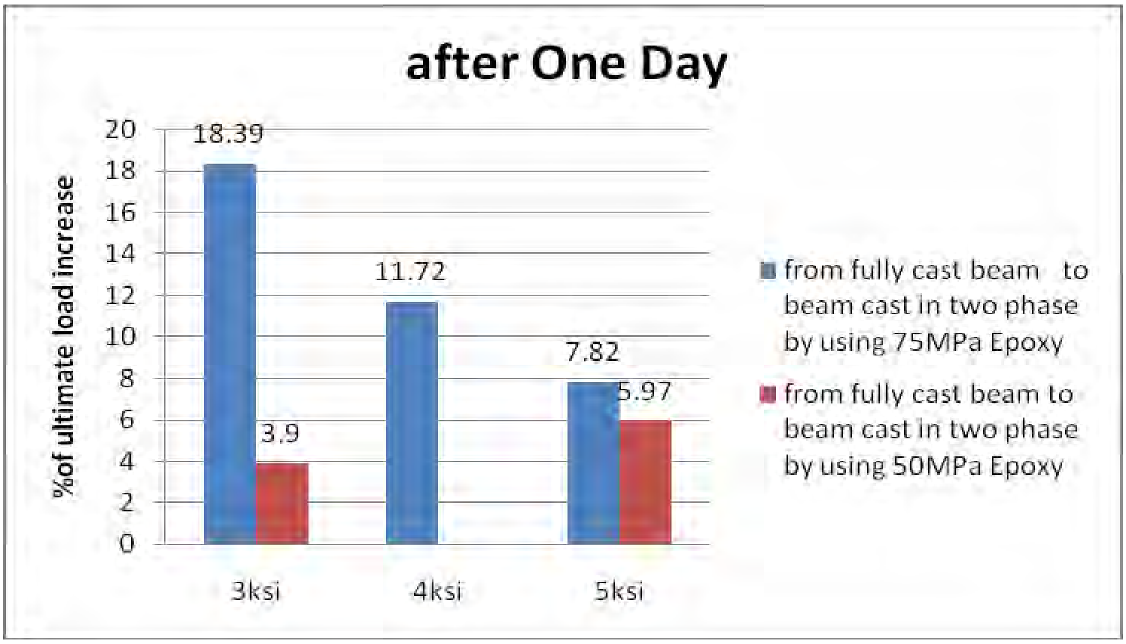

Figure 4.59 Effect of Bonding Agent on Ultimate Load When Beams Cast in Two Phases after One Day

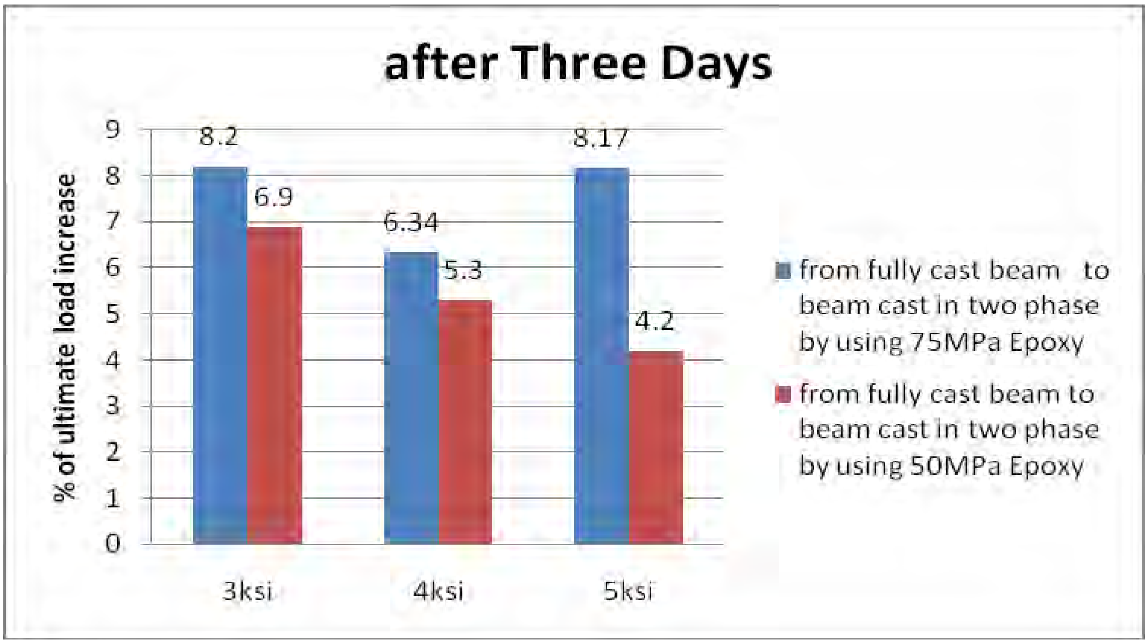

Figure 4.60 Effect of Bonding Agent on Ultimate Load When Beams Cast in Two Phases after Three Days

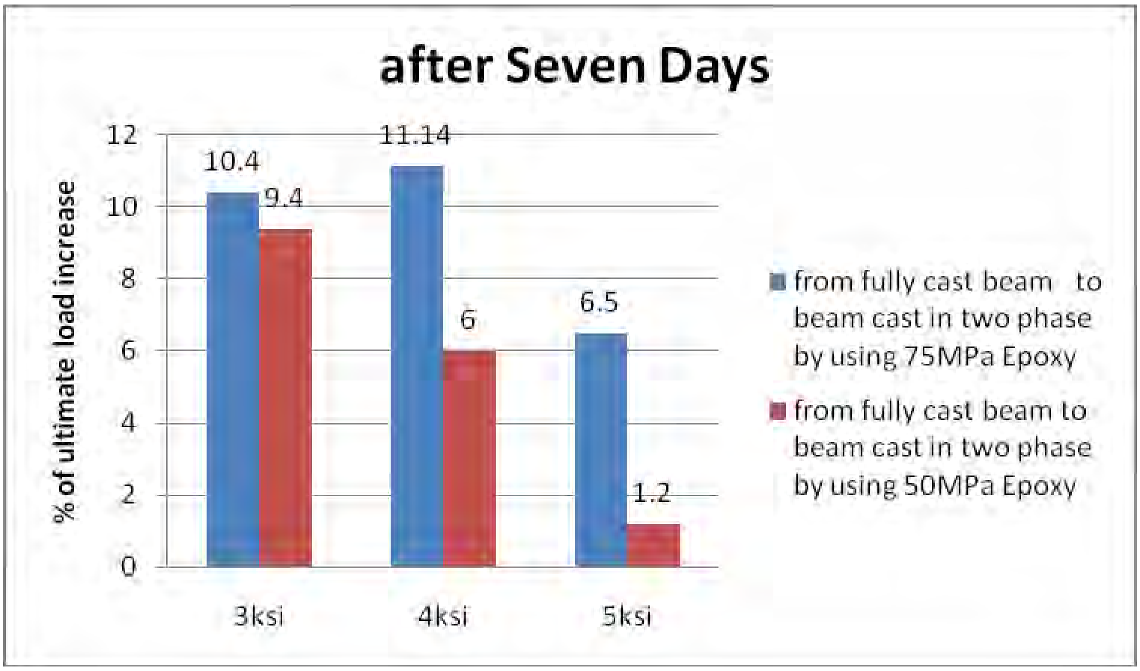

Figure 4.61 Effect of Bonding Agent on Ultimate Load When Beams Cast in Two Phases after Seven Days

4.4.2 Effect of Bonding Agent on Ultimate Load with Various Age of Concrete

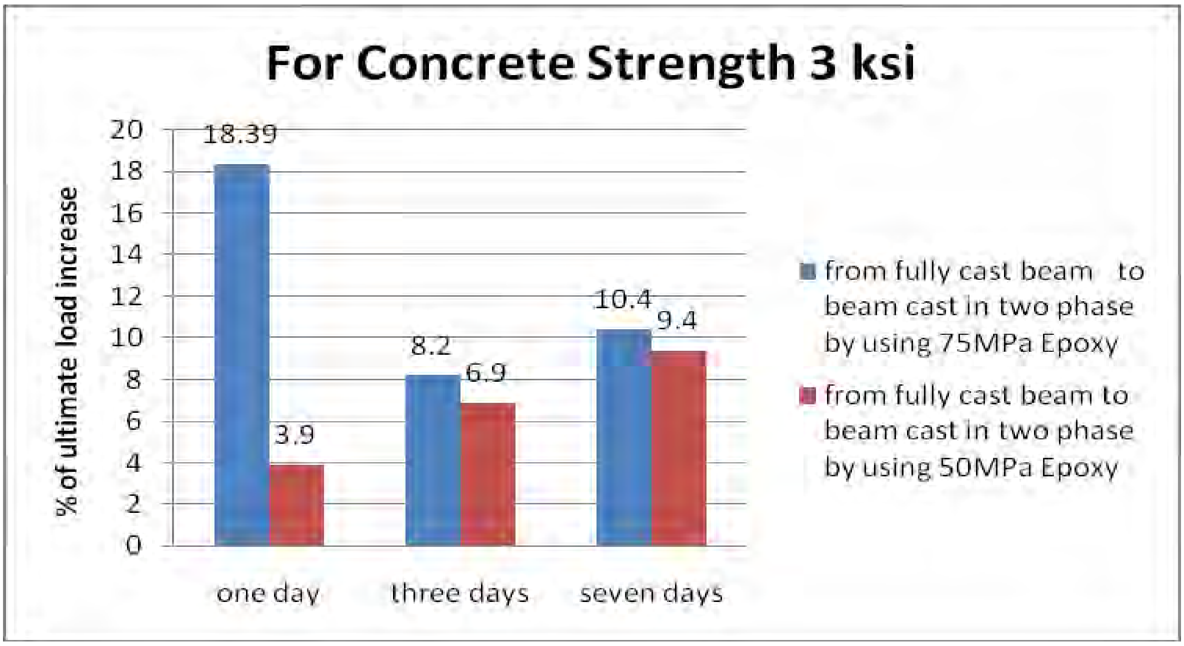

Figure 4.62 Effect of Various Age of Concrete on Ultimate Load for Concrete Strength 3 ksi

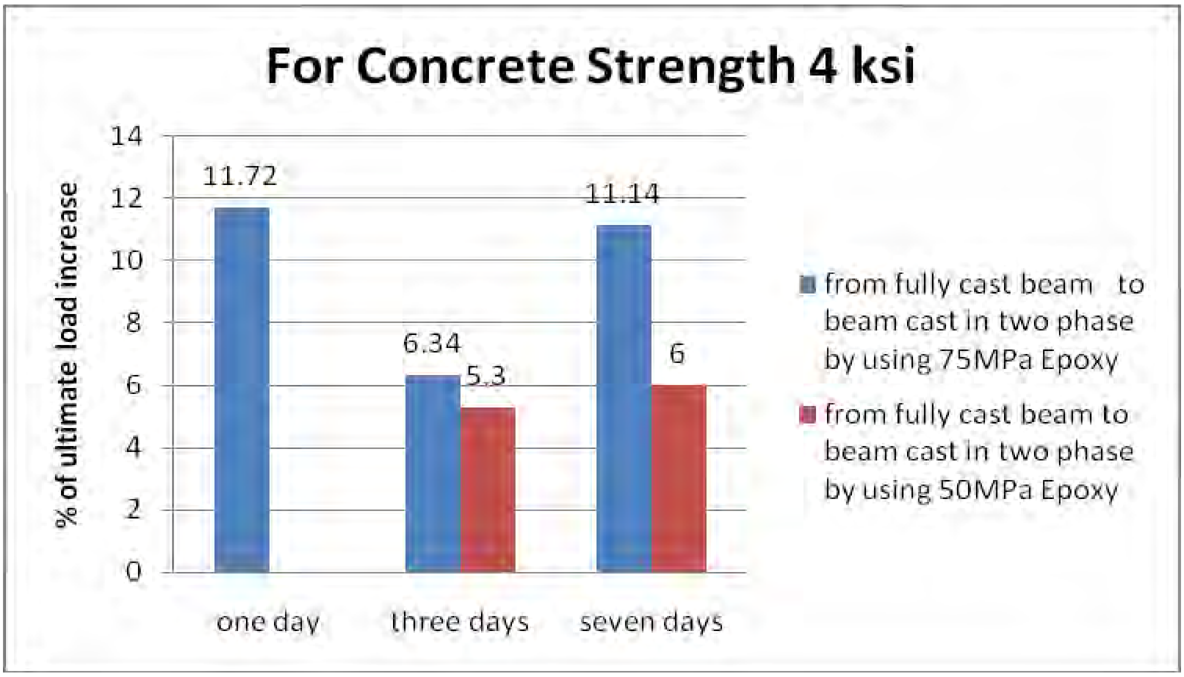

Figure 4.63 Effect of Various Age of Concrete on Ultimate Load for Concrete Strength 4 ksi

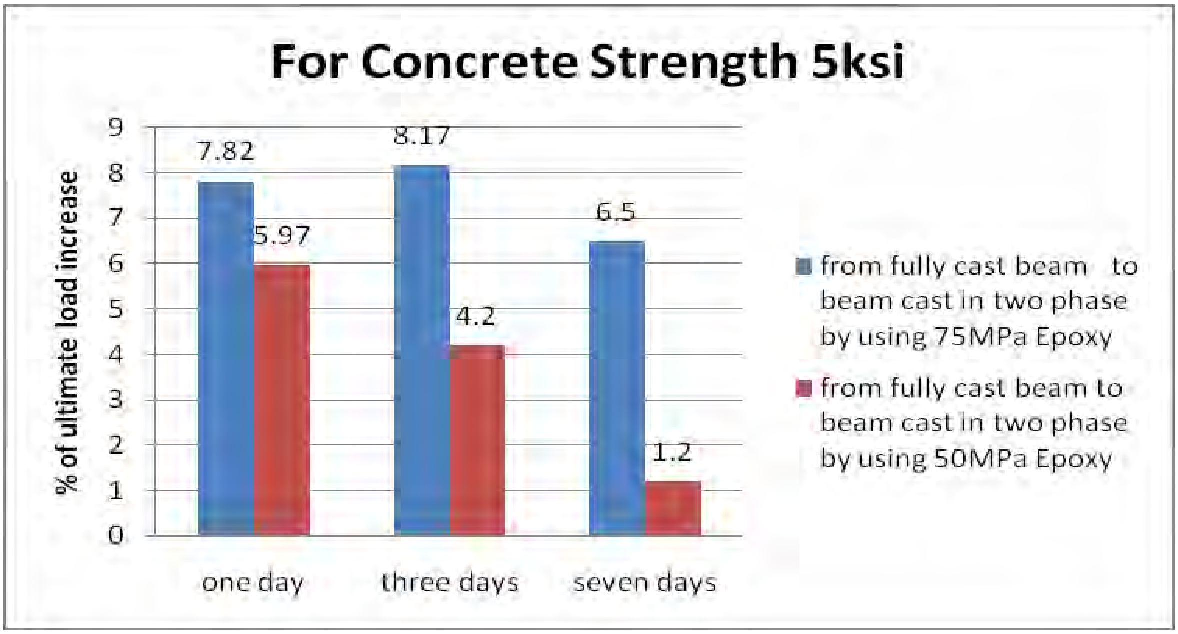

Figure 4.64 Effect of Various Age of Concrete on Ultimate Load for Concrete Strength 5 ksi

4.4 COMPARASION AMONG THE BEAMS CAST IN TWO PHASES BY USING BONDING AGENT 50MPa EPOXY AND 75 MPa EPOXY

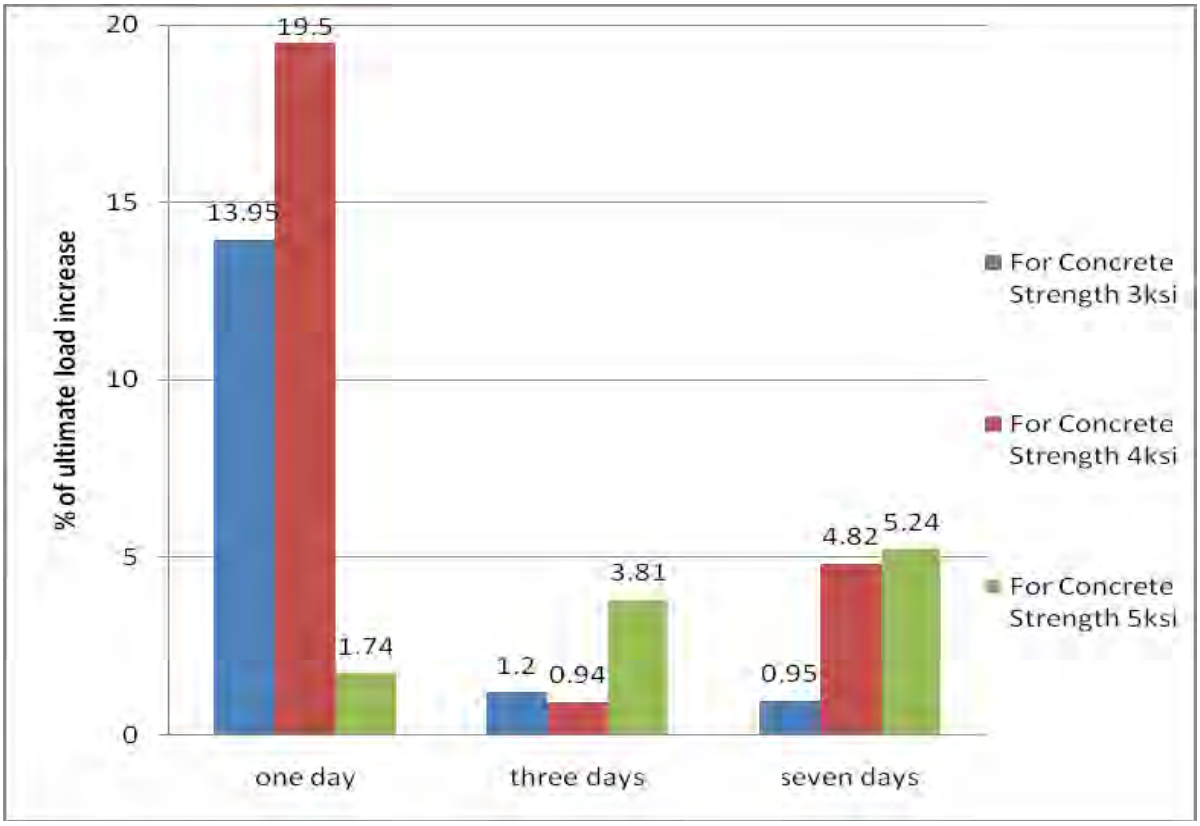

Figure 4.65 Percentage of Increase of Ultimate Load from Beams Cast in Two Phases by Using 50MPa Epoxy to Beams Cast in Two Phases by 75MPa Epoxy

CHAPTER FIVE
CONCLUSION

5.1 GENERAL

The overall aim of this research is to investigate on RC Beams flexural shear capacity which were cast in two phases and to investigate the suitability of using epoxy as bonding agent and without using any bonding agent in strengthening of RC beams under one point loading . To achieve the objectives, total thirty half scale specimens were constructed and tested. Ten of them, Group A, were constructed to get concrete strength 3ksi with using and without using bonding agent and next ten specimens, Group B, were constructed to get concrete strength 4ksi with using and without using bonding agent. And the rest ten specimens, Group C, were constructed to get concrete strength 5ksi with using and without using bonding agent.

All the specimens were constructed at two stages except the control specimen of each group. At first stage for three groups total three beams were fully cast and twenty seven beams were half cast. At second stage after one day nine half cast beams (from each group three beams) were cast fully with and without using bonding agent. Same procedure are followed after three days and seven days .So, total twenty seven half cast beams were fully cast at stage two.

Specimens were tested under incremental one point loading .The load was applied by Tinius Olsen Universal Testing Machine at midspan of the beam. A constant strain-controlled loading was applied with the movement rate of the platform being 5mm/min. Time vs load was continuously monitored and data was saved automatically .Deflection of the beams were also monitored with a video extensometer and time versus deflection data was stored continuously .These data are then combined to prepare load versus deflection curves which are the basis of analysis for flexural strength

Load-deflection curves of beams were plotted to study their behavior under one point loading and to compare with control specimen. Then ultimate strength of all the specimens were evaluated, plotted and compared. Finally some conclusions were drawn basing on the qualitative analysis and experimental results.

5.2 CONCLUSION

Based on the performed test results, behavior of the RC beams casts in two phases with using 50MPa Epoxy and 75MPa Epoxy under one point loading were investigated. From these results, the following conclusions can be drawn:

i. When beams are cast in two phases, a weak surface is created .In case of beams which has no bonding agent in the joining surface, the beams have lower ultimate load than the fully cast beam as the bonding between two surfaces is not so effective.
ii. When bonding agent (75MPa Epoxy and 50MPa Epoxy) is used in the joining surface for the casting of beams in two phases, the beams have higher values of ultimate load than the fully cast beams.
iii. So, by means of constructing beams in two phases with using bonding agent 75MPa Epoxy and 50MPa Epoxy, the flexural shear capacity of the beams can be increased.
iv. The beams cast in two phases with bonding agent 75MPa Epoxy have higher values of ultimate load than using 50MPa Epoxy.

v. So, if there is a need to increase the flexural shear capacity or there is a situation to cast beam in two phases, 75MPa Epoxy can be used as it is more effective as a bonding agent.

vi. The effect of age of concrete after which the second phase of construction is started on ultimate load with using bonding agent 50MPa Epoxy and 75MPa Epoxy is very less. So, the effects of applying epoxy as a bonding agent with the variation of days are insignificant once the concrete is set.

vii. The very first crack for the fully cast beams and the all other beams were appeared at the mid span at the bottom of the beams.

viii. Flexural crack was appeared as a first crack in all the beams.

ix. All the fully cast beams were failed by flexural crack. Diagonal tension crack helps to fail the beams in flexure.

x. Almost all the beams cast in two phases without using any bonding agent and with using bonding agent were failed by diagonal tension crack and the failure was initiated by bond crack.

xi. For concrete strength of 3ksi, the increase in ultimate load for beams cast in two phases after one day respectively by using 75MPa Epoxy and using 50MPa Epoxy are almost 18.4% and 3.9% than compared to that of fully cast beam. Similarly for concrete strength of 3ksi, ultimate load increases almost 8.2% and 6.9% respectively for beams cast in two phases after three days by using 75MPa Epoxy and using 50MPa Epoxy than compared to that of fully cast beam. Again for concrete strength of 3ksi, ultimate load increases almost 10.4% and 9.4% respectively for beams cast in two phases after seven days by using 75MPa Epoxy and using 50MPa Epoxy than compared to that of fully cast beam.

xii. For concrete strength of 4ksi, ultimate load increases 11.72% for beam cast in two phases after one day by using 75MPa Epoxy than fully cast beam but for the beam cast in two phases after one day by using 50MPa Epoxy, ultimate load did not increase than the fully cast beam. Similarly for concrete strength of 4ksi, increase in ultimate load for beam s cast in two phases after three days respectively by using 75MPa Epoxy and using 50MPa Epoxy are almost 6.34% and 5.3% compared to that of fully cast beam. Again for concrete strength of 4ksi, ultimate load increases almost 11.14% and 6.0% respectively for beams cast in two phases after seven days by using 75MPa Epoxy and using 50MPa Epoxy than compared to that of fully cast beam.

xiii. For concrete strength of 5ksi, the ultimate load increases almost 7.82% and 5.97% respectively for beams cast in two phases after one day by using 75MPa Epoxy and using 50MPa Epoxy than compared to that of fully cast beam. Similarly for concrete strength of 5ksi, ultimate load increases almost 8.2% and 4.2% respectively for beams cast in two phases after three days by using 75MPa Epoxy and using 50MPa Epoxy than compared to that of fully cast beam. Again for concrete strength of 5ksi, ultimate load increases almost 6.5% and 1.2% respectively for beams cast in two phases after seven days by using 75MPa Epoxy and using 50MPa Epoxy than compared to that of fully cast beam.

xiv. The increase in ultimate load for the beams cast in two phases by using 75MPa Epoxy than compared to the beams cast in two phases by using 50MPa epoxy after one day, three days and seven days for concrete strength 3ksi are respectively 13.95%, 1.2% and 0.95%.

xv. Similarly for concrete strength of 4ksi, the increase in ultimate load for the beam cast in two phases by using 75MPa Epoxy than compared to the beam cast in two phases by using 50MPa epoxy after one day, three days and seven days are respectively 19.5%, 0.94% and 4.82%.

xvi. Similarly for concrete strength of 5ksi, the increase in ultimate load for the beam cast in two phases by using 75MPa Epoxy than compared to the beam cast in two phases by using 50MPa epoxy after one day, three days and seven days are respectively 1.74%, 3.81% and 5.24%.

5.3 RECOMMENDATION FOR FURTHER STUDY

For further investigation this research suggests some recommendations as follows:

- The experimental results may be verified by finite element analysis.
- Full scale specimens may be investigated to get more accurate result.
- More parameters may be considered to achieve more specified result.
- Against each specimen, three specimens may be made to get the more accurate results.

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APPENDIX A

Table A.1: Sieve Analysis of Sylhet Sand

Sieve No	Sieve Size (mm)	Weight retained (gm)	% Retained	Cumulative Retained	% passing
4	4.75	0.0	0.0	0.0	100
8	2.36	26.7	5.36	5.36	94.64
16	1.19	104.6	20.99	26.35	73.65
30	0.59	177	35.51	61.86	38.14
50	0.3	127.3	25.54	87.4	12.6
100	0.15	41.7	8.37	95.77	4.23
Pan	0	21.1	4.23	100	0
Total		498.4		276.74	
			FM	2.77	

Table A.2: Specific Gravity and Absorption Capacity of Coarse Aggregates

SL No	Wt.(g) of Oven Dry Sample (a)	Wt.(gm) of the SSD Sample (b)	Wt.(gm) of Sample in Water (c)	Bulk Specific Gravity (oven dry) a/(b-c)	Bulk Specific Gravity (SSD) b/(b-c)	Apparent Specific Gravity a/(a-c)	Absorption Capacity (b-a)/a (%)
1	1980.5	2000	1183	2.42	2.45	2.48	0.98

Table A.3: Specific Gravity and Absorption Capacity of Fine Aggregates

Sample	Wt.(gm) of oven dry sample (a)	Wt.(gm) of pycnometer + water (b)	Wt.(gm) of SSD sample (s)	Wt.(gm) of pycnometer + water + sample (c)	Bulk Specific Gravity (oven dry) a/(b+s-c)	Bulk Specific Gravity (SSD) s/(b+s-c)	Apparent Specific Gravity a/(b+a-c)	Absorption Capacity (s-a)/a (%)
Sylhet sand	483	1302.7	500	1606.8	2.47	2.55	2.70	3.52

Table A.4: Unit weight of Aggregates

Aggregate	Wt.(kg) of oven dry sample	Volumee of Measure(m^3)	Ovendry rodded unit weight(kg/m^3)	SSD rodded Unit weight (kg/m^3)
Stone chips	4.07	2.832×10^{-4}	1437.3	1452
Sylhet sand	4.42	2.832×10^{-4}	1560.91	1616

Table A.5: Ultimate Strength of Steel Reinforcement

Dia, mm	Frog Mark	Wt	Length (cm)	Yield/ Proof load (kN)	Ultimate load (kN)	Elongation (%)	X Sec of Bar (mm^2)	Yield Capacity (MPa)	Average Yield Capacity (MPa)	Ultimate Capacity(MPa)	Average Ultimate Capacity (MPa)
10	BSRM XTREME 500W	366	61.0	45.9	52.2	14	76.4	600.79	587.69	683.25	672.75
		366	61.0	43.65	50.85	15	76.1	576.59		668.20	
		360	60.8	45.45	51.75	13	77.6	585.70		666.88	
8	BSRM XTREME 500W	239	61.2	26.55	31.05	12	49.4	537.45	559.12	628.54	650.77
		237	61.1	28.35	32.85	15	49.0	578.57		670.41	
		239	61.5	27.45	31.95	14	48.9	561.35		653.37	

Table A.6: 28 days compressive strength of concrete

13.07.2014							
	Cylinder Specimen	Observed load(kN)	Actual load (kN)	Area,m^2	Compressive strength (MPa)	Compressive strength (psi)	Average Compressive strength (psi)
For Group C	1	396	293.56	0.008107	36.21	5250.48	5383.08
	2	399	296.59	0.008107	36.58	5304.72	
	3	415	312.77	0.008107	38.58	5594.05	
For Group B	4	361	258.17	0.008107	31.85	4617.59	4430.74
	5	346	243.01	0.008107	29.97	4346.35	
	6	345	242.00	0.008107	29.85	4328.27	
For Group A	7	316	212.68	0.008107	26.23	3803.88	3773.74
	8	326	222.78	0.008107	27.48	3984.7	
	9	301	197.51	0.008107	24.36	3532.64	
After one day :14.07.2014							
For Group C	C1A	414	311.75	0.008107	38.45	5575.96	5431.30
	C2A	400	297.60	0.008107	36.71	5322.81	
	C3A	404	301.64	0.008107	37.21	5395.14	
For Group B	C5A	379	276.37	0.008107	34.09	4943.07	4943.08
For Group A	C7A	306	202.57	0.008107	24.99	3623.05	3550.72
	C8A	298	194.48	0.008107	23.99	3478.39	

After three days :16.07.2014							
For Group C	C1B	406	303.67	0.008107	37.46	5431.30	5075.68
	C2B	382	279.40	0.008107	34.46	4997.32	
	C3B	371	268.28	0.008107	33.09	4798.41	
For Group B	C4B	352	249.07	0.008107	30.72	4454.85	4424.71
	C5B	351	248.06	0.008107	30.60	4436.76	
	C6B	348	245.03	0.008107	30.22	4382.52	
For Group A	C7B	288	184.37	0.008107	22.74	3297.57	3363.87
	C8B	296	192.46	0.008107	23.74	3442.23	
	C9B	291	187.40	0.008107	23.12	3351.81	
After seven days :22.07.2014							
For Group C	C1C	447	345.12	0.008107	42.57	6172.69	5509.66
	C2C	383	280.41	0.008107	34.59	5015.40	
	C3C	401	298.61	0.008107	36.83	5340.89	
For Group B	C4C	362	259.18	0.008107	31.97	4635.67	4599.51
	C5C	352	249.07	0.008107	30.72	4454.85	
	C6C	366	263.23	0.008107	32.47	4708.00	
For Group A	C7C	307	203.58	0.008107	25.11	3641.13	3442.23
	C8C	288	184.37	0.008107	22.74	3297.57	
	C9C	293	189.42	0.008107	23.37	3387.98	

APPENDIX B