TRIBHUVAN UNIVERSITY
INSTITUTE OF ENGINEERING
PULCHOWK CAMPUS
DEPARTMENT OF CIVIL ENGINEERING
M .Sc. Program in Structural Engineering

Thesis no:SS00113

BEHAVIOUR OF CFRP (CARBON FIBER REINFORCED POLYMER) STRENGTHENED RC PIER UNDER LATERAL LOAD

ANUP KUMAR SUBEDI

(Final Thesis Draft)

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A thesis submitted by

ANUP KUMAR SUBEDI

In partial fulfillment of the requirement for the degree of

MASTER OF SCIENCE

IN STRUCTURAL ENGINEERING

FEBRUARY- 2010

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CERTIFICATE

This is to certify that the work contained in this thesis entitled **“BEHAVIOUR OF CARBON FIBER REINFORCED POLYMER (CFRP) STRENGTHENED RC PIER UNDER LATERAL LOAD”**, in partial fulfillment of the requirement for the degree of Master of Science in Structural Engineering, as a record of research work, has been carried out by **Mr. ANUP KUMAR SUBEDI** (062 / MSS / F / 102) under my supervision and guidance in the Institute of Engineering, Pulchowk Campus, Lalitpur, Nepal. The work embodied in this thesis has not been submitted elsewhere for a degree.

………………………………

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Anup subedi

ABSTRACT

Since FRP has gained popularity in retrofitting the RC structures, this paper presents the behaviour of RC Pier before and after retrofitting subjected to monotonic lateral load. This paper is based on the results obtained from experiment performed on column-footing assembly in laboratory.

Each specimen was subjected to monotonic lateral load and load, displacements as well as the patterns of cracks and damage behavior were monitored carefully. On the basis of degree of damage observed the retrofitting of damaged specimen with carbon fiber reinforced polymer (CFRP) was varied. The retrofitted specimens were also tested under monotonic lateral load and its crack patterns, damage behavior, load and displacements were monitored carefully.

An experimental result of this study indicates that CFRP used in this work enhanced the strength and stiffness of damage column-footing specimens.

As compared to the initial capacity of the specimens the capacity of retrofitted specimens was reduced by less than 20% only.

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CHAPTER - 1

1. INTRODUCTION

1.1. General.

Earthquake resistant structures need to be designed to dissipate seismic energy through post-elastic energy dissipation in the members. This is achieved by designing certain structural members to possess large ductility and usually these selected members in the frames are beams and column. Past earthquake have shown many damages in RC column which in turn led to the damage of the entire structure.

The behavior of columns in earthquakes is very important since column failures lead to additional structural failures and can result in total structure collapse. Since column take the load of entire structure as well as external load. Column when interact with lateral load the flexural deformations, reinforcement bond slip occurs at the column footing joint level. Firstly minor cracks are seen and these cracks propagate to become major cracks which in turn lead to the failure of steel as well. If cracks are seen in column footing interface, rather than ignoring, it should be well repaired to save the structures.

Strengthening of RC joints is a challenging task that poses major practical difficulties. A variety of techniques applicable to concrete elements have also been applied to joints with the most common ones being the construction of RC or steel jackets. Reinforced concrete jackets and some forms of steel jackets, namely steel ‘‘cages,’’ require intensive labor and artful detailing. Moreover, concrete jackets increase the dimensions and weight of structural elements. Plain or corrugated steel plates have also been tried. In addition to corrosion protection, these elements require special attachment through the use of either epoxy adhesives combined with bolts or special grouting. Few years ago, a new technique for strengthening structural elements emerged. The technique involves the use of fiber reinforced polymers as externally bonded reinforcement.

FRP materials used to strengthen and repair RC columns are popular applications of FRP composites in structural engineering. Collectively, these applications are known as retrofitting applications. Retrofitting applications can be classified broadly into two types. One type is strengthening, where the original column strength or ductility (typically, its displacement capacity) is increased from the loads (or displacements) for which it was originally designed. This increase may be necessitated by the desire to make the structure compatible with existing building codes (particularly in the case of seismic retrofitting) or may be desired due to changes in use of the structure

The other type of FRP retrofitting can be classified as repair. In this case, the FRP composite is used to retrofit an existing and deteriorated structure to bring its load-carrying capacity or ductility back to the loads or displacements for which it was designed (and hence is, in fact, a type of strengthening). Repair is necessitated when the original structure has deteriorated due to environmental effects, such as corrosion of steel reinforcing in concrete structures or when the original structure has been damaged in service or was not constructed according to the original design. For example, reinforcing bars may be omitted in a beam at the time of construction due to a design or construction error. Although these two types of applications are similar, there are important differences that are related primarily to evaluation of the existing structural capacity and the nature of the repair to be undertaken before FRP can be used. In many cases, a repair design will include strengthening to add a level of safety to the repaired structure and to account for uncertainty in the retrofit design.

The work presented in this research paper deals with the experimental study of 8 reinforced square column-footing assemblies. The column specimen were tested under flexural load(lateral load) and the same specimens were strengthened with CFRP and again tested under flexural load and comparative study on its strength, behavior and deflections was undertaken.

1.2. Problem Statement:

Recent earthquakes worldwide have illustrated the vulnerability of existing reinforced concrete column-Footing joints to seismic loading. Poorly detailed joints, especially exterior ones, have been identified as critical structural elements, which appear to fail prematurely, thus performing as ‘‘weak links’’ in RC frames. A typical failure mode in poorly designed joints lacking adequate transverse reinforcement is concrete shear in the form of diagonal tension.

In the case of RC bridge pier which were constructed before the development of proper seismic codes or constructed with lack of detailing of lateral reinforcement are vulnerable towards failure because lateral loads and moments induced by seismic loads results in large shear forces in bridge columns which are resisted mainly by lateral ties or spirals around the main reinforcement. Therefore in the columns with inadequate lateral reinforcement, it is essential to provide external confinement to save it against lateral loads and moments and the proper external confinement can be achieved by high strength fiber reinforced polymer.

As far as Nepal is concerned it is situated in Earthquake prone zone. Lots of old structures can be seen in Nepal, mostly in Kathmandu city area. Some of these structures are likely to damage but still it is occupied. So before disaster takes place if it can be repaired or strengthen properly than life and property could be saved to some extent. Demolishing all old structures and constructing new one is highly expensive than retrofitting. It is hard to imagine what would be in future if improper methodology and improper materials are used to repair the old structure or to strengthen it. So to overcome these problems the new retrofitting techniques using FRP could be fruitful.

1.3. Aims and Objective.

Since Nepal is occupied with lots of old structures some are damaged and some are leading towards damage. Nobody knows when the structure will damage until the natural disaster takes place. Recently FRP has been introduced in Nepal in basis of experiments and uses in foreign countries. This experimental study could be helpful for all Nepalese citizens who are using or going to use the FRP as retrofitting materials for their structure, after this experiment I hope in this type of structure (RC column) we don’t need to rely on foreign experiments. This present study is aimed to determine the strength of rehabilated (CFRP jacketed) column under monotonic lateral load and to demonstrate the behavior of CFRP strengthened RC column. Some objectives are listed below.

Comparative study on strength and deflections of as built and strengthened column.

Comparative study on failure modes of as built and retrofitted column.

1.4. Scope of Study.

This research work is an experimental investigation of reinforced concrete column-footing assembly. The scope of this research work is limited i.e. the column along with footing was tested under flexural load only by applying the load at one end only. The damaged specimen after testing was strengthened using FRP techniques. The strengthened specimen was again tested under flexural load. The point of application of load was same in both conditions.

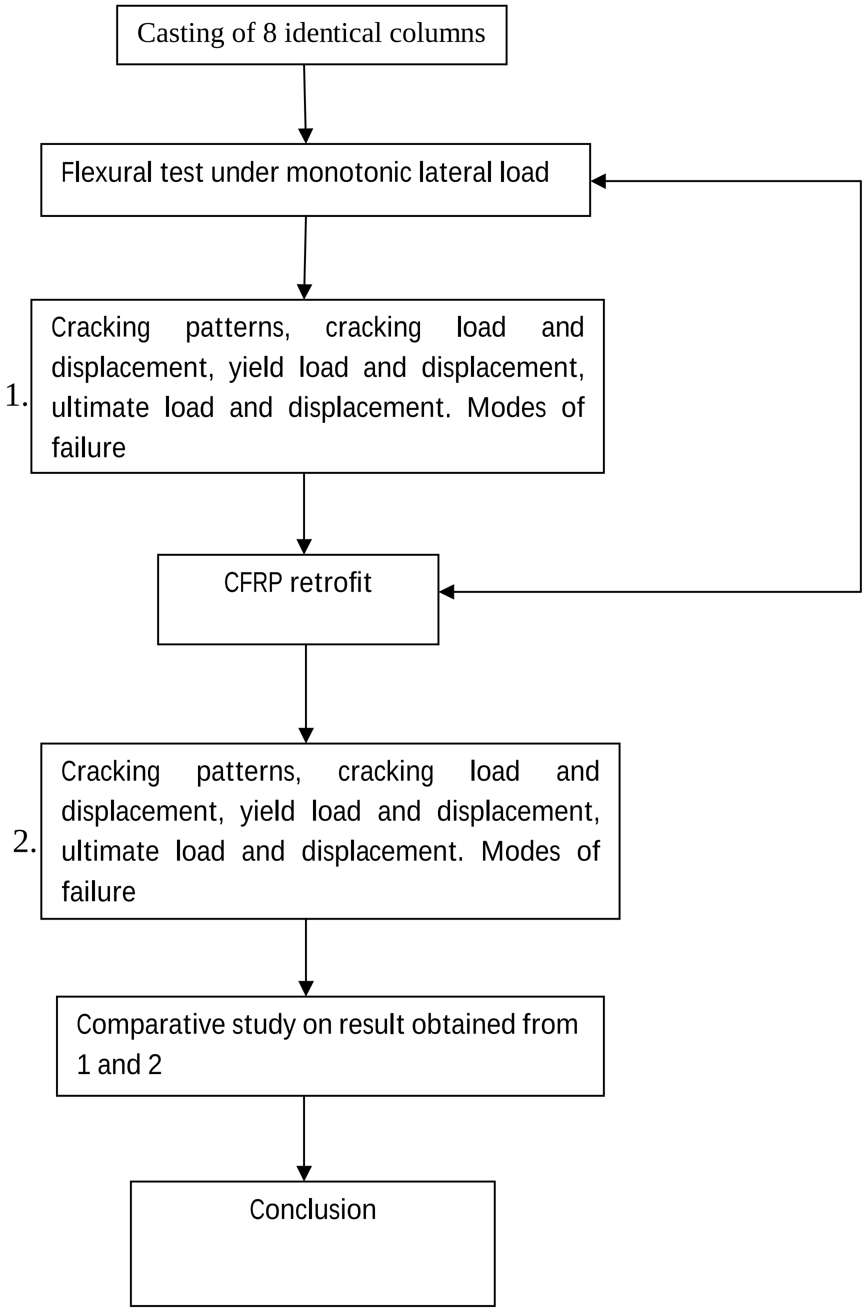
1.5. Contribution of the Study.

The researcher hopes this study will contribute to the retrofitting or strengthening techniques. Since this study focuses on strength of FRP It makes clear to the retrofitters and owner of the structures that how FRP could be helpful in retrofitting the RC column or saving the entire structures. It also decreases the vulnerability towards damage. It also contributes to the Nation in saving many bridges which are prone to failure. Moreover this technique increases the life of structure.

1.6. Methodology of research work.

This research work deals with an experimental investigation of 8 reinforced concrete column-footing assemblages. All the specimens are tested under monotonic lateral load.

According to damages and failure modes, these tested specimens are strengthened with CFRP and again tested under monotonic lateral load. The entire methodology is shown in flowchart below.



CHAPTER- 2

2. LITERATURE REVIEW.

2.1. Fiber reinforced polymer system.

Fiber-reinforced polymer is a composite material that consists of a polymer matrix with fiber reinforcement. Glass and Carbon are common fibers while the polymer is typically an epoxy resin. The polymer is placed on the concrete surface, and then the FRP is wrapped around the column. In wet-application, fibers are soaked in wet resin or polymer before FRP application. The polymer helps to connect the fibers of the wrap together while also making a strong connection with the surface of the concrete.

An FRP system wrapped around a column provides passive reinforcement to the column. As the concrete member is loaded axially, the FRP reinforcement system provides little or no effect on strength increase to the confined concrete initially. However, once the concrete dilates and begins to crack and weaken, the FRP reinforcement provides confinement for the concrete. The main advantage of the FRP system is the amount of confinement that it provides. The enveloping wrap or tube provides more confinement than a longitudinal or spirally wrapped steel rebar.

2.2. Carbon fibers.

Carbon fibers are used in structural engineering applications today in FRP strengthening sheets and fabrics, in FRP strengthening strips, and in FRP prestressing tendons. Carbon fiber is a solid semi crystalline organic material consisting on the atomic level of planar two dimensional arrays of carbon atoms. The two-dimensional sheet like array is usually known as the graphitic form; hence, the fibers are also known as graphite fibers (the three-dimensional array is well known as the diamond form). Carbon fiber is produced in grades known as standard modulus, intermediate modulus, high strength, and ultrahigh modulus.

Carbon fibers have diameters from about 5 to 10 micro m (0.00197 to 0.00394 in.). Carbon fiber has a characteristic charcoal-black color. Due to their two-dimensional atomic structure, carbon fibers are considered to be transversely isotropic, having different properties in the longitudinal direction of the atomic array than in the transverse direction. The longitudinal axis of the fiber is parallel to the graphitic planes and gives the fiber its high longitudinal Modulus and strength. Carbon fiber is produced at high temperatures 1200 to 2400deg. centigrade from three possible precursor materials: a natural cellulosic rayon textile fiber, a synthetic polyacrilonitrile (PAN) textile fiber, or pitch (coal tar). Pitch-based fibers, produced as a by-product of petroleum processing, are generally lower cost than PAN- and rayon-based fibers. As the temperature of the heat treatment increases during production of the carbon fiber, the atomic structure develops more of the sheet like planar graphitic array, giving the fiber higher and higher longitudinal modulus. For this reason, early carbon fibers were also known as graphite fibers. The term carbon fiber is used to describe all carbon fibers used in structural engineering applications. The term graphite fiber is still used in the aerospace industry; however, this term is slowly dying out. Similar to glass fibers, carbon fibers need to be sized to be compatible with a resin system. Historically, carbon fibers have been used primarily with epoxy resins, and suitable sizings for epoxy resin systems are readily available. Nowadays, carbon fibers are being used with vinylester and blended vinylester–polyester resins for FRP profiles and FRP strengthening strips. Sizing for carbon fibers for polyester and vinylester resins are not as common. Care must be taken when specifying a carbon fiber for use with a non epoxy resin system to ensure that the fiber is properly sized for the resin system used.

Carbon fibers are very durable and perform very well in hot and moist environments and when subjected to fatigue loads. They do not absorb moisture. They have a negative or very low coefficient of thermal expansion in their longitudinal direction, giving them excellent dimensional stability. They are, however, thermally and electrically conductive. Care must be taken when they are used in contact with metallic materials, as a galvanic cell can develop due to the electropotential mismatch between the carbon fiber and most metallic materials. Some research has suggested that this can lead to degradation of the polymer resin in the FRP composite, especially in the presence of chlorides and to corrosion of the metallic material.

(lawrence C bank- composite for construction, structural design with FRP materials)

2.3. Epoxy Resins

Epoxy resins are used in many FRP products for structural engineering applications. Most carbon fiber–reinforced precured FRP strips for structural strengthening are made with epoxy resins. In addition, epoxy resin adhesives are used to bond precured FRP strips to concrete (and other materials) in the FRP strengthening process. Epoxy resins are also used extensively in FRP strengthening applications, where the epoxy resin is applied to the dry fiber sheet or fabric in the field and then cured in situ, acting as both the matrix for the FRP composite and as the adhesive to attach the FRP composite to the substrate. When applied to dry fiber sheets or fabrics, the epoxy resins are often referred to saturants. Epoxy resins have also been used to manufacture FRP tendons for prestressing concrete and FRP stay cables for bridges. They are not used extensively to produce larger FRP profiles, due to their higher costs and the difficulty entailed in processing large pultruded FRP parts.

An epoxy resin contains one or more epoxide (or oxirane) groups that react with hydroxyl groups. Most common are the reaction products of bisphenol A and epichlorohydrin, called bis A epoxies, or those made from phenol or alkylated phenol and formaldehyde and called novolacs. The resins are cured (or hardened) with amines, acid anhydrides, (Lewis acids) by condensation polymerization and not, like polyesters, by free-radical chain polymerization.

The epoxy resin and the curing agent (or hardener) are supplied in two parts and are mixed in specific proportions (usually about 2 to 3 parts to 1 part by weight) just prior to use to cause the curing reaction. The first epoxy resin was produced by Schlack in 1939.

Epoxy resins are particularly versatile and can be formulated in a range of properties to serve as matrix materials for FRP composites or to serve as adhesives. The epoxies used as the resins in FRP parts for structural engineering belong to the same family as the more familiar epoxies currently used in a variety of structural engineering applications, such as for concrete crack injection, as anchors for concrete, and for bonding precast concrete elements. Epoxy resins are known to have excellent corrosion resistance and to undergo significantly less shrinkage than polyester or vinylester resins when cured. Consequently, they are less prone to cracking under thermal loads. Epoxy resins have been developed for high-temperature applications of 180 degC and higher and have been the thermosetting resins of choice in the aerospace industry for the last 50 years. The density of epoxy resin is about 1.05 g/cm3 (0.038 lb/ in3). Epoxy resins can be cured at room temperature or at high temperature. In many aerospace applications, epoxy resin composites are postcured at elevated temperatures

to raise their glass transition temperatures and to improve their physical and mechanical properties. The glass transition temperature of an epoxy is therefore highly formulation and cure (temperature dependent and can range from 40 degC up to 300 degC). Epoxy resins usually are clear to yellowish or amber in color.

(lawrence C bank- composite for construction, structural design with FRP materials)

2.4. Innovative techniques for seismic upgrade of RC square column

A total of 8 column specimens were constructed and tested under monotonic lateralload. All had the same square cross-section and were internally reinforced using smooth steel bars. The heights of columns were 2.0 m above the footing 0.60 m deep. 8-mm diameter ties spaced at 100 mm on center were placed along the height; first tie above the footing placed at 50 mm from the column-footing interface.

A total of 8 columns were tested 4 as built and 4 strengthened. Firstly the columns were put on to fixed axial loads (270kn and 540kn) and the lateral load was applied. Tests were performed under displacement control mode.

In this research upgrading of column was done with the combination of steel spikes and GFRP (Glass fiber reinforced polymer). Steel spikes were used as flexural reinforcement and GFRP for the external confinement. The strengthened column in this research increased its load withstanding capacity ranging between

33% and 54% with also increase in drift ratio corresponding to maximum shear force.

2.5. Seismic strengthening of circular bridge pier models with fiber composites.

This was an experimental research done to improve the seismic capacity of substandard concrete columns in an effective and economical way.

Five concrete column-footing assemblages were constructed, test specimen were designed to approximately model design of existing highway bridge columns in a zone of high seismic risk. Each specimen consists of a single column with strong footing details. The composite strap was wrapped only in the plastic hinge region prior to the beginning of the test.

The specimens were tested with combined axial and lateral loadings. The cracking and failure mechanism were concentrated mostly in columns especially in the plastic hinge region, due to heavy detailing of footings.

The failure modes of columns tested before strengthening were brittle because of rapid strength detoration following the debonding of longitudinal reinforcement.

On the other hand concrete column wrapped with FRP composites in plastic hinge region showed significant improvement in both strength and displacement ductility.

The FRP retrofitting schemes provide additional confinement to existing core concrete and were highly effective in preventing the columns from bond failure or longitudinal bar buckling and hence greatly increased the earthquake resistance of the column.

2.6. Analysis and behaviour of FRP-confined short concrete columns subjected to eccentric loading.

This paper has been concerned with the analysis and behaviour of FRP-confined RC circular and rectangular short columns subjected to eccentric loading which produces a combined action of axial load and bending. A parametric study based on a simple stress-strain model for FRP-confined concrete leads to the following conclusions:

The axial strength, moment capacity and curvature ductility of a RC column can be considerably enhanced by using the FRP confinement, and a higher amount of FRP produces a higher degree of the enhancement. In the case of pure bending and FRP Jackets with fibers oriented only in the hoop direction, a significant increase in the column ductility with little increase in the moment capacity of the columns results. In this case, the use of longitudinal FRP has to be considered in order to increase the bending moment capacity.

The ultimate axial strain of confined concrete can be higher in a column wrapped with AFRP than in a column wrapped with CFRP, if both FRP wraps provide the same maximum confining pressure. This is because the AFRP has a higher strain capacity than the CFRP. While this increased ultimate strain has no effect on the axial strength and moment capacity of the confined column, it yields a considerable improvement in the curvature ductility of the column.

An increase in the unconfined concrete strength has different effects on the moment capacity of the confined column at axial load levels above and below the axial strength of the unconfined column. An increased unconfined concrete strength reduces the curvature ductility of the column because the ultimate axial strain of confined concrete is reduced without increasing the amount of FRP.

The FRP confinement is much less effective for rectangular columns but an increase in the corner radius is beneficial to both strength and ductility. An increase in the aspect ratio has a negative effect on the axial strength of the FRP-confined column, but may have small beneficial effects on the moment capacity and ductility.

2.7. Retrofit of RC Frames Using FRP Jacketing or Steel Bracing

Due to non-ductile failure modes of existing structures in recent earthquake the researcher tried to overcome these failure using retrofitting techniques. Retrofit of these structures before the earthquake provides a feasible cost-effective approach to reduce the hazard to occupants' safety and owners' investment. The response of two reinforced concrete frames was examined under seismic excitation. The 9-storey and 18-storey frames are part of the lateral load resisting system in two office buildings that were designed according to the 1960s code provisions. The frames were analyzed assuming flexible joint response by considering the joint shear deformation or assuming traditional rigid joints. Two rehabilitation techniques were proposed to improve the dynamic response of these frames. Fiber reinforced polymer (FRP) jackets were used as a local rehabilitation technique to enhance the joint shear strength and ductility. As another option, X-steel braces were installed in the middle bay of the frame along its height as an alternate lateral load resisting system. For each frame, failure sequence and interstorey drift were examined. It was found that FRP wrapping eliminated the brittle failure modes without significant change in the structural response. However, steel bracing significantly contributed to the structural stiffness and reduced the maximum interstorey drift of the frames. The following conclusions were derived after this research work.

1. non-ductile failure pattern such as joint shear failure may occur at some locations.
2. An integrated scheme using FRP composites and steel braces might be more efficient for reducing the frame lateral deflection and eliminating undesirable non-ductile failure mechanisms.
3. Assuming rigid joints in the analysis give different damage pattern and interstorey drift than what are obtained when joint deformation is accounted for. Accurate

assessment of damage patterns in existing structures is needed for determining the optimum locations for joint strengthening.

4. The rehabilitation using FRP composites does not significantly alter the dynamic response of the frame. However, FRP-strengthening significantly changes the damage location and pattern in the frame. This is because the FRP composite materials do not significantly affect the initial stiffness of the concrete members but improves the strength.

   The rehabilitation technique using steel braces is very effective in increasing the frame stiffness by providing alternate stiff lateral load resisting system. However

2.8. Seismic retrofitting of bridge column with composite straps.

In this research work behavior of typical rectangular bridge columns with substandard design details for seismic forces were investigated. Five rectangular columns with different reinforcement detail were constructed and tested under reversed cyclic loading. Three columns were retrofitted with FRP and two columns were not retrofitted and were used as control specimens so that their response could be compared with those for retrofitted columns.

High strength FRP straps were wrapped around the column in the potential plastic hinge region to increase the confinement and to improve the behavior under seismic forces. The lateral load was applied with hydraulic actuator.

In this study the columns externally wrapped with FRP in the potential plastic hinge region showed the significant improvement in both strength and displacement ductility. The retrofitted columns developed very stable load-displacement hysteresis loop without significant structural deterioration associated with the bond failure of lapped starter bars or longitudinal reinforcement buckling.

CHAPTER- 3

3. EXPERIMENTAL PROGRAM

The experimental program comprises of 8 identical square column-footing assembly of size 100mm X 100mm cross-section and 1000mm ht. for column and 400mm X 400mm and 200mm depth for footing designed according to IS456-2000.

The experimental program aimed to study the flexural strength of the column-footing assembly. All the columns were tested under monotonic lateral load and later they were retrofitted using carbon FRP. All the retrofitted column specimens were tested again under the monotonic lateral load in order to compare with the initially tested results.

3.1. Reinforcement detail-

The column consists of 4nos. of 8mm HYSD longitudinal bar tied with 4.75mm HYSD bar spaced at 100mm C/C throughout the height of the column. Ref. fig.1

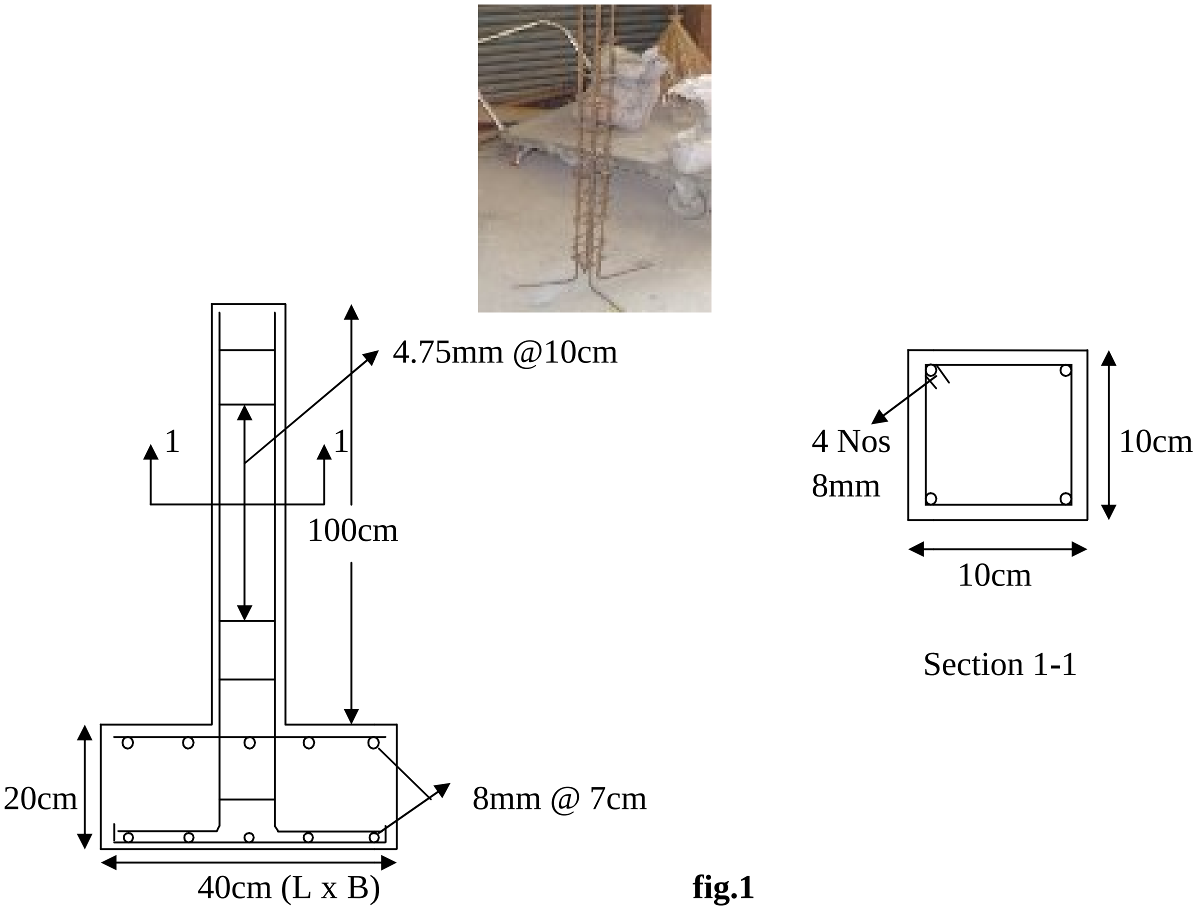


fig.1

3.2. Preparation of specimens-

For the preparation of column-footing assembly, strong form of 12mm thick plywood was prepared in the lab ref fig.2. Reinforcement cage was fabricated according to detailing of reinforcement and then it was placed inside the form. The clear covers for each specimen were 10mm. After placing the rebar cage, form was clamped with bracket in order to restrict the bulging. Ref. fig.3

Locally available sand and aggregate (20mm down) were used for the construction of specimen and were washed and dried properly prior making concrete. The cement used was OPC with 53 grade. The concrete mix was prepared in lab and the 2 specimen was casted at a time. All the specimens were casted horizontally. Ref. fig.4. The formwork was removed after 48 hours and it was wrapped with burlap and water curing was done continuously for 14 days. Ref. fig.5

Fig.2, Plywood Form.

Fig.3, Rebar cage inside Form and clamping of form

Fig.4, Casting of specimems

Fig.5, curing of specimens.

3.3. Material properties-

i. Steel properties-

The steel used was high yield strength deformed bar of 8mm for longitudinal reinforcement and 4.75mm for lateral ties. The steel used for all specimens was taken from same bundle so randomly selected 3 pieces was taken for testing The tensile strength given by manufacturer was 415Mpa. But the calculated tensile strength in lab is tabulated below. Ref. table 1.1.

Table 1.1

S.N	Dia.of bar(mm)	Yield strength (Mpa)	Ultimate tensile strength(Mpa)	Average yield tensile strength(MPa)	Elongation (%)
1	8	501.45	555.46	487.22	20.872
2	8	478.31	545.17		23.482
3	8	481.91	549.27		23.570

ii. Concrete properties-

The concrete used for the construction of each specimens were mix in the ratio of 1:1.5:3 by weight with water cement ratio(W/C) of 0.5. the column and footing assembly were casted monolithically. 3 nos of 150mm X 150mm cubes were casted in each batching and compressive strength were measured after 28days. The obtained compressive strength is tabulated below. Ref. table 1.2.

Table 1.2.

S.No	Specimen No.	Target strength(Mpa)	Achieved strength(Mpa)	Average strength (Mpa)
1	S1,S2	20	22.66	26.87
2			29.02	
3			28.97	
4	S3,S4		33.6	30.97
5			32.66	
6			26.66	
7	S5,S6		22.66	30.39
8			34.66	
9			33.86	
10	S7,S8		30	29.04
11			26.66	
12			31.55	

3.4. Test set up-

The test set up is displayed in fig.6. Footing was properly fixed and lateral load was applied with the help of hydraulic actuator. The columns were tested in a horizontal position. Load was applied at the free end of column i.e. 95cm from column-footing interface. The entire specimens were tested after 28 days.

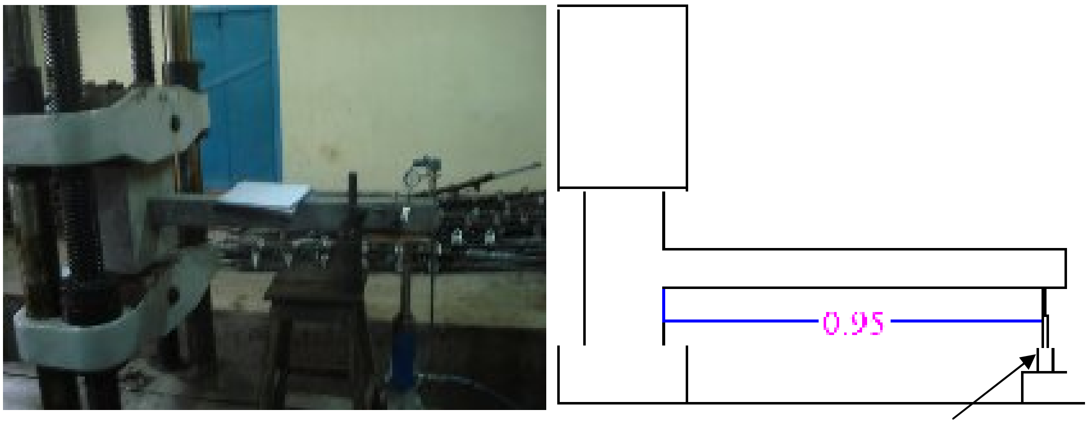


Fig.6. Test set up.

Load applied by actuator

3.5. Testing of specimens(first stage)-

All the specimens were tested for flexure. Point of application of lateral load was same for all the specimens. All the specimens were tested at the central material testing lab of I.O.E The date of casting and test date. are tabulated below. Ref. table 1.3.

Table 1.3

Specimens designation	Date of casting	Date of testing
S1	30/08/2009	27/09/2009
S2	30/08/2009	27/09/2009
S3	11/09/2009	24/10/2009
S4	11/09/2009	24/10/2009
S5	22/09/2009	26/10/2009
S6	22/09/2009	26/10/2009
S7	15/10/2009	13/11/2009
S8	15/10/2009	13/11/2009

The footings of the specimens were fixed properly. It was restricted against movement in all directions. The fixing force was approximately same for all the specimens. The lateral load was applied on the column exactly at 95cm from footing top.

Load was applied vertically with the hydraulic actuator of capacity 200 KN. Load was applied in the average incremental of 1.58KN which was measured by load dial gauge and corresponding deflections were measured with the help of deflection dial gauge, however load and deflections were also measured when cracks were observed.

3.6. Retrofitting of damaged specimens-

All the damaged specimens were retrofitted with acme’s carbon fiber reinforced polymer (CFRP). Properties of acme’s carbon wrap provided by “create acme associates, Bakhundole, G.P.O. box# 1344, Kathmandu, Nepal” are tabulated below. Ref. table, 1.4.

Table, 1.4. Mechanical properties of fibres.

Density(g/cm2)	Tensile strength(Gpa)	Thickness(mm)	Deformation ratio at failure(%)
1.8	3.9	0.45	2.1

3.7. Procedure of retrofitting –

Step by step procedure of FRP installation is described below.

1. Loose concrete were removed and damaged area were cleaned thoroughly with water and completely dried. Lap splices were fixed where the rebars were fractured and buckling were serious. Ref. Fig.7.1.


lap splices **Grouting with epoxy putty**

Fig. 7.1

2. Grouting was done by injecting epoxy resins wherever necessitated and possible.Ref. Fig.7.2. it was kept for 48 hours for hardening purpose.

Fig. 7.2. Epoxy grouting in cracks

3. After completion of grouting, voids were filled properly with high strength filler materials having good bonding strength ref fig.7.3. (Filler materials or epoxy putty was prepared in lab by mixing epoxy resin with dry and clean silica sand. The mixing proportion was 1: 18 by weight i.e. 1 part of epoxy resin with 18 parts of silica sand. The average compressive strength of epoxy putty was 30Mpa in 48hrs and 50Mpa in 7 days as measured in lab by testing the cubes of size 5cm x 5cm.) It was kept for 10 days to ensure that full strength was obtained.

Fig 7.3 **Gaps filled with Filler**

4. The surface was smoothened and cleaned and corners of columns were rounded in order to prevent stress concentrations which may cause premature failure of CFRP. Ref. fig.7.4.

Fig. 7.4 **surface smoothening with grinder**

5. The epoxy resin was applied on the surface of concrete in a thin uniform layer. A fiber sheet was cut to desired length and width and epoxy resin was applied on it and it was pressed on the concrete surface of column specimens where it was desired to apply. Ref. fig.7.5 and 7.6.

Epoxy resin on surface of Column.

Resin on FRP

Installation of FRP

Fig. 7.5

Application of resin on the surface Of laminate

Column after completion of FRP wrapping.

Fig. 7.6

Height and layers of CFRP used for all column specimens are shown in table.1.5. Ref table 1.5.

Table, 1.5.

Specimen Designation.	No. of layers for flexure.	Height in flexure.(cm)	No. of layers for shear.	Height in shear. (cm)
S1	2	35	2	20
S2	2	35	1	14
S3	1	34	2	25
S4	1	26	1	15
S5	1	35	2	20
S6	1	27	0	0
S7	1	35	1	18
S8	1	35	2	11

The numbers of layers of FRP wrap is kept different so as to achieve the best resulting wrap. The height of FRP wrap is according to the damage surface and cracks.

6. All the strengthened specimens were kept for curing for at least 10 days to ensure the full strength was obtained prior testing.

3.8. Testing of strengthened column-footing specimens-

The set-up and instrumentation was exactly same as of first stage testing. The lateral load was also applied exactly at the same point of application of first stage testing. The results obtained after the testing of strengthened specimens are illustrated in chapter ‘results and discussions’.

CHAPTER- 4

4. RESULTS AND DISCUSSIONS

The calculated allowable loads for the specimens are 6.704KN and deflection was 13.065mm.

Since the specimens are tested twice before retrofitting (first stage) and after retrofitting (second stage) the results are presented in two clauses.

4.1. First stage results-

The cracking load, yield load, ultimate load and corresponding deflections are tabulated in **table 1.6.**

Variation in load Vs deflection curves are due to varying properties of steel. Ref table.1.1.

Table 1.6

Specimen N0.	Cracking Load (KN)	Cracking Displacement (mm)	Yield Load (KN)	Yield Displacement (mm)	Ultimate Load (KN)	Ultimate Displacement (mm)	remarks
S1	3.11	11.9	8.78	62.65	9.04	353	Rupture failure
S2	1.58	14.1	6.03	48	6.5	210.3	Do
S3	3.11	22	7.43	55	8	203	Do
S4	3.11	18	7.43	50	8	150	Buckling failure
S5	4.01	28.04	7.43	66.7	-	-	Crushing failure
S6	2.2	18.4	7.15	63.5	-	-	do
S7	3.11	15.6	7.7	67	8	170	Buckling failure
S8	2.2	21.2	7.7	71.6	8	253.5	do

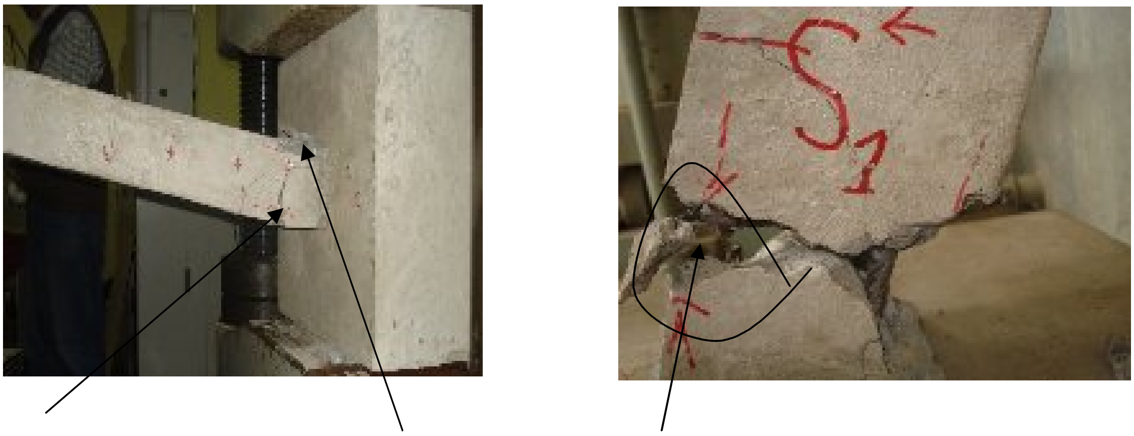
4.1.1. Observed damage and behavior-

Three different failure modes were seen on testing 8 specimens which are as-

a. Rupture of longitudinal reinforcement- S1, S2, S3.
b. Buckling of longitudinal reinforcement- S4, S7, S8.
c. Crushing failure- S5, S6.

The patterns of crack of each specimen are shown in fig. 9. Ref fig,9

In specimen S1 early flexural cracks were seen at 14cm, 7cm and 20cm from footing top at a load of 3.11KN and 1.19% of lateral drift ratio(lateral deflection/column height). After further increment of load these early cracks (specifically cracks at 7cm and 14cm) were increased significantly. Some flexural cracks were also seen nearly up to the mid span but these cracks width and depth were not increased significantly. The crushing of concrete on compression side was also seen near column-footing interface and concrete spalling took place at this zone at yield load 8.78KN at lateral drift ratio of 6.26%. After degradation of load carrying capacity of column, the steels were ruptured at 35% drift ratio. The steels were ruptured at 10cm from footing top. Ref fig, 8.1.



Distinct flexural crack and crushing
Of concrete on compression side.

Rupture of reinforcement

Fig. 8.1, specimen, S1

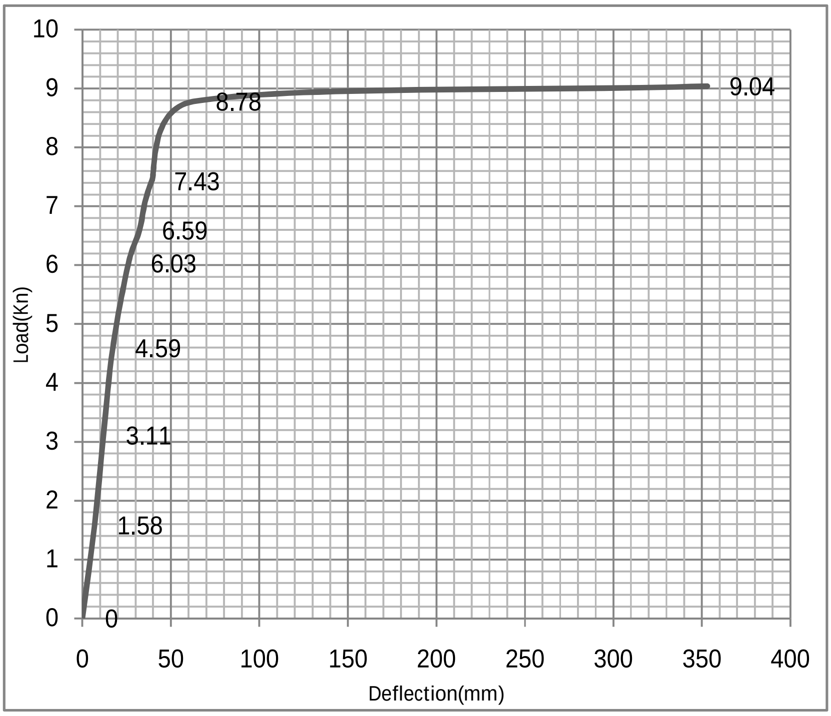


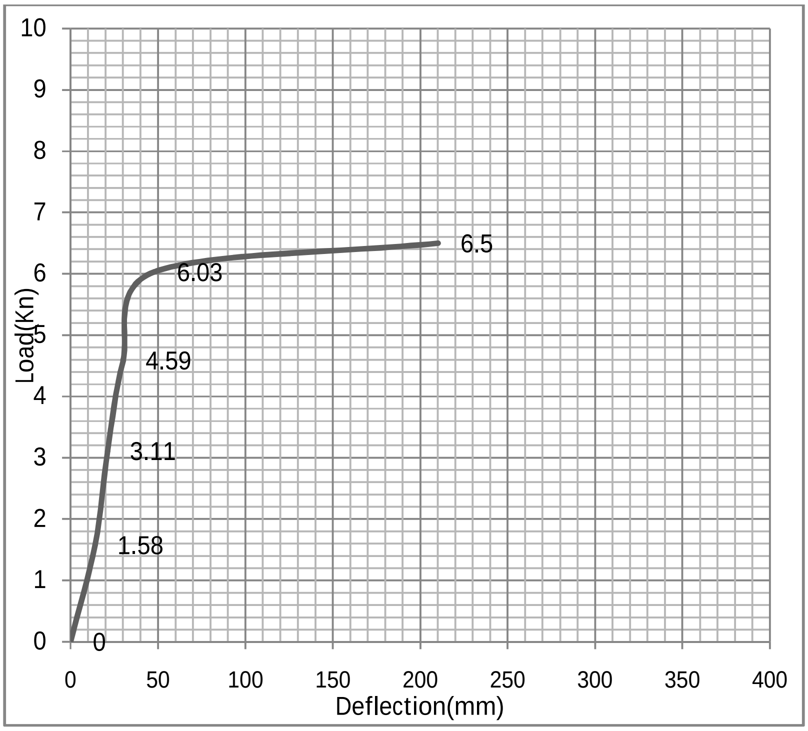
Fig. 8.1a, **Specimen, S1**

Similarly, in other specimens also the pattern and propagation of cracks were similar to that of S1 which is explained above but the damages seen were different. In specimens S2 and S3 only one longitudinal reinforcement was ruptured and damage was less serious than S1, the location of rupture was 7cm and 10cm from column-footing joint and corresponding drift ratio was 21% and 20.3% respectively. Ref. fig. 8.2.

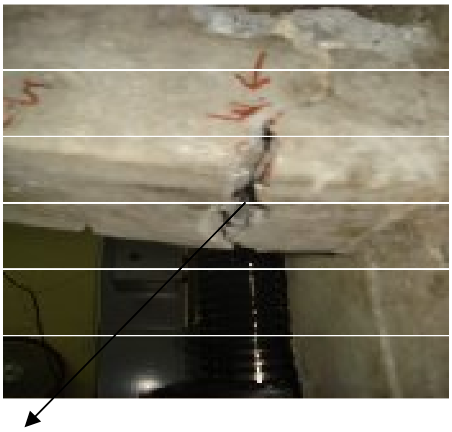
Progression of Flexural crack. Crack at joint. **Specimen, S2**

Damage seen after rupture of Reinforcement. **Specimen. S2**

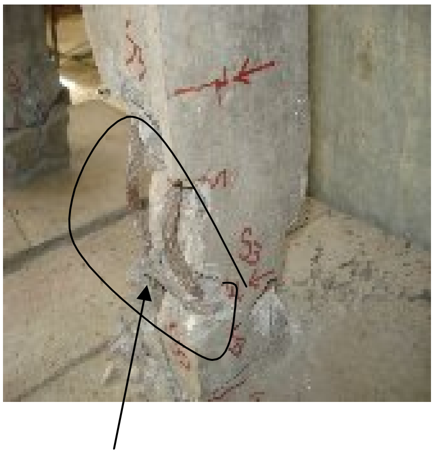
Fig 8.2a



Fig, 8.2b, Specimen, S2

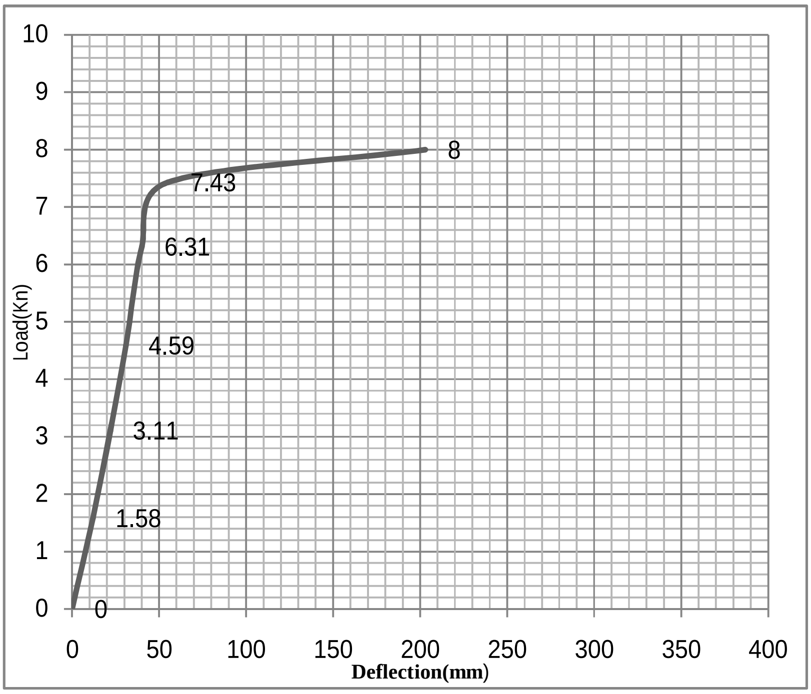


Flexural crack near interface and
Concrete spalling at top.
Specimen, S3



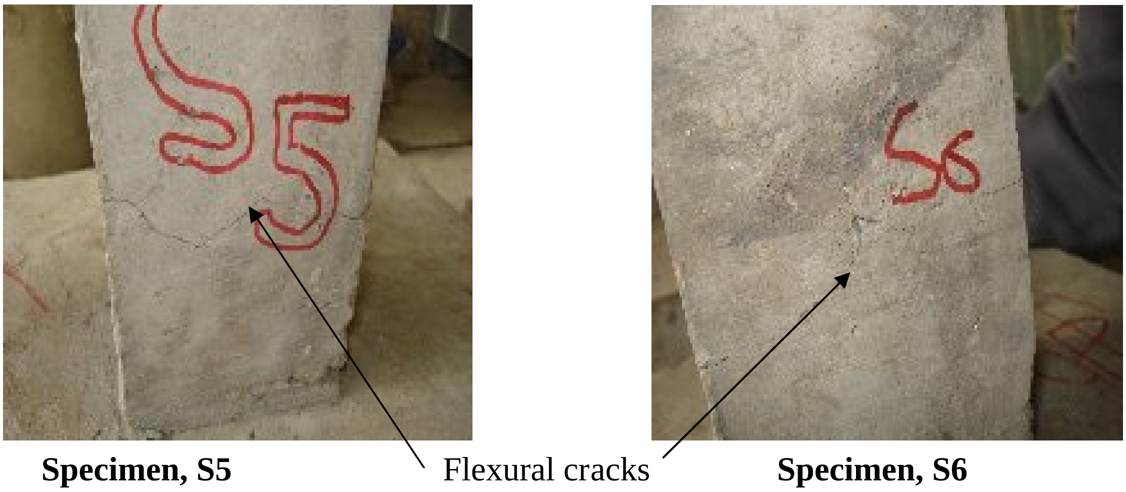
rupture of reinforcement and
Spalling of concrete cover.
Specimen, S3

Fig, 8.2c



Fig, 8.2d, Specimen, S3

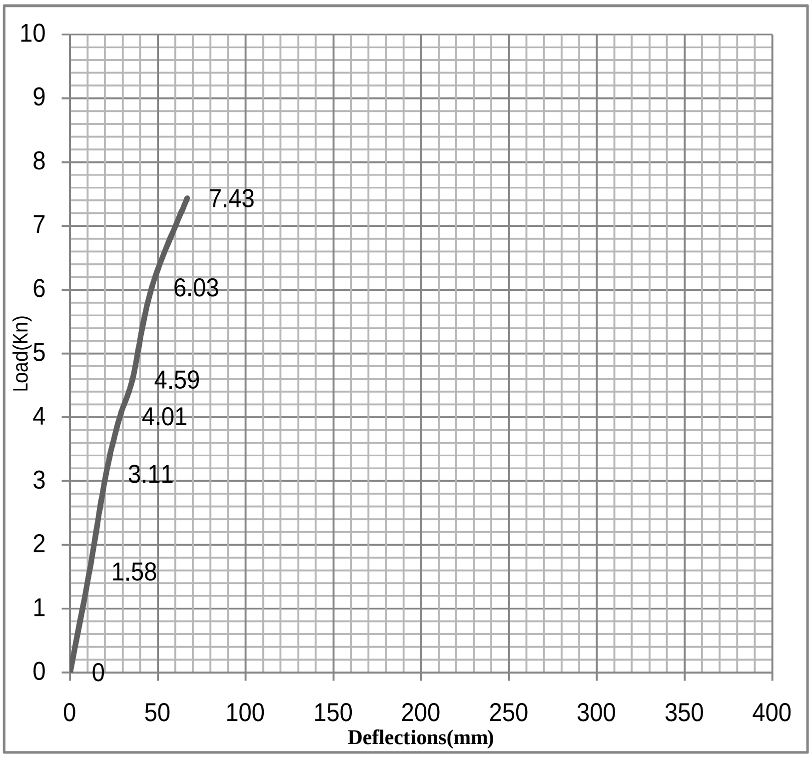
In specimens S5 and S6 after yielding the loading was removed and at this stage only one major flexural cracks were seen at 8.5cm and 10.6cm from column-footing joint however some other hairline cracks were also there nearly up to mid-span. Ref fig, 8.3.



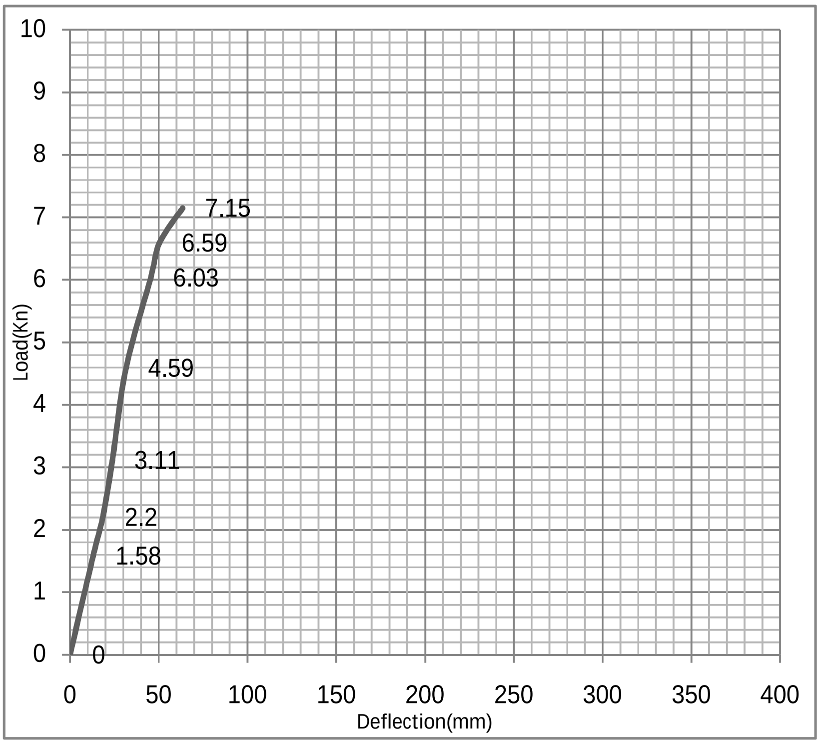
Specimen, S5

Specimen, S6

Fig, 8.3a



Fig, 8.3b, Specimen, S5



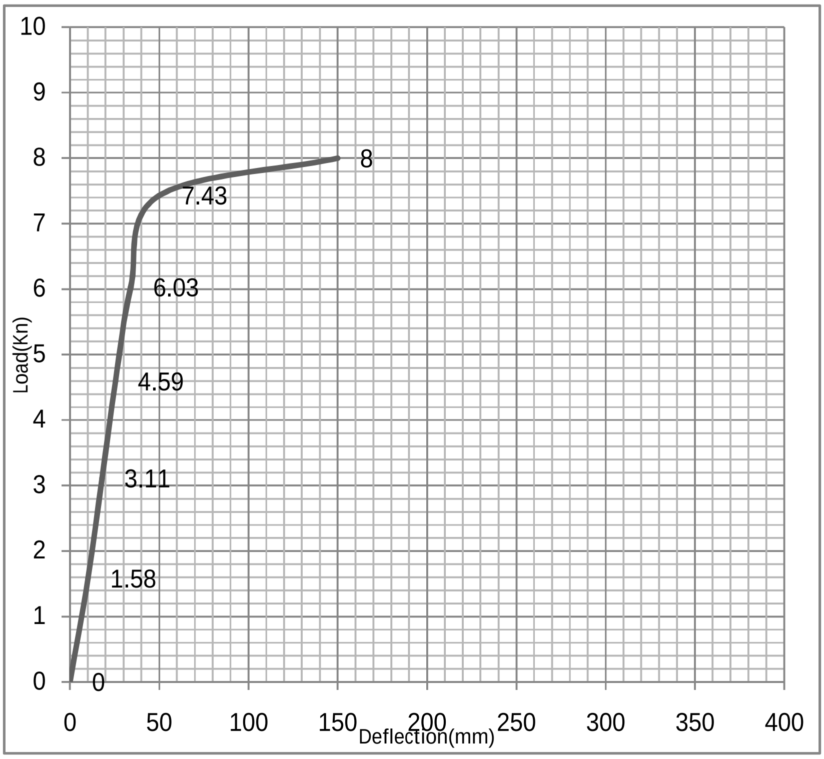
Fig, 8.3c, Specimen S6

In specimens S4, S7 and S8 longitudinal steels were buckled at a drift ratio of 15%, 17% and 25% amongst these specimens' damages seen on S8 was more serious than S4 and S7. Crushing of concrete on compression side, spalling of concrete and rebar exposure was noticed in all these 3 specimens. Ref fig 8.4

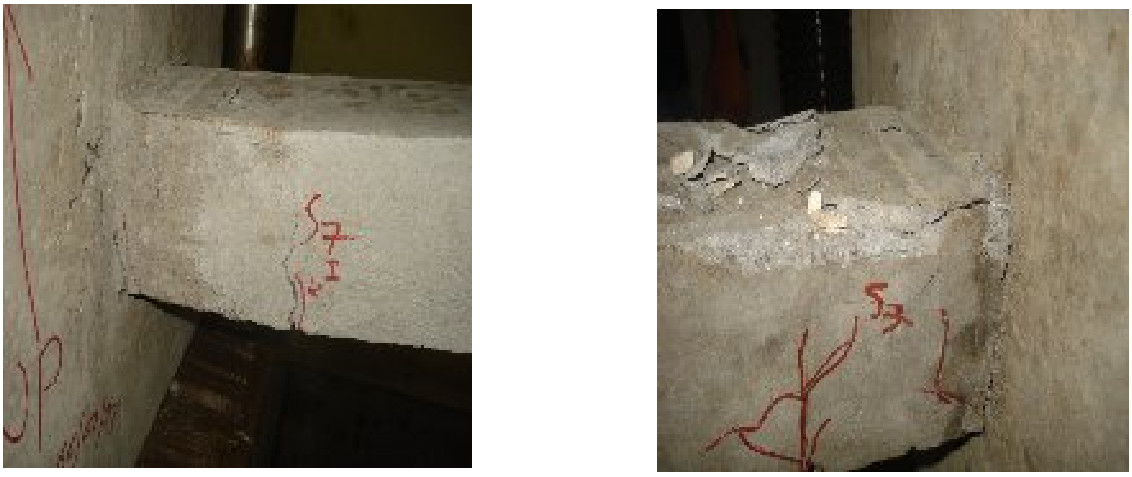
Concrete crushing at corner
Specimen, S4

Concrete spalling at corner
Buckling of reinforcement
Specimen, S4

Fig, 8.4a



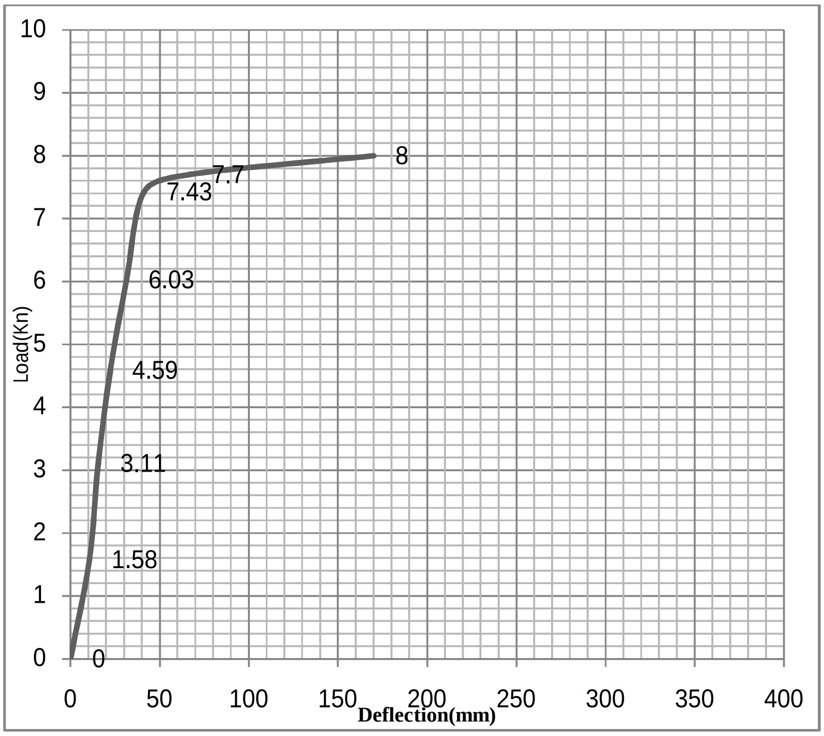
Fig, 8.4b, Specimen S4



Progression of flexural crack
Specimen, S7

Crushing and spalling of concrete at corner and joint crack.
Specimen, S7

Fig, 8.4c

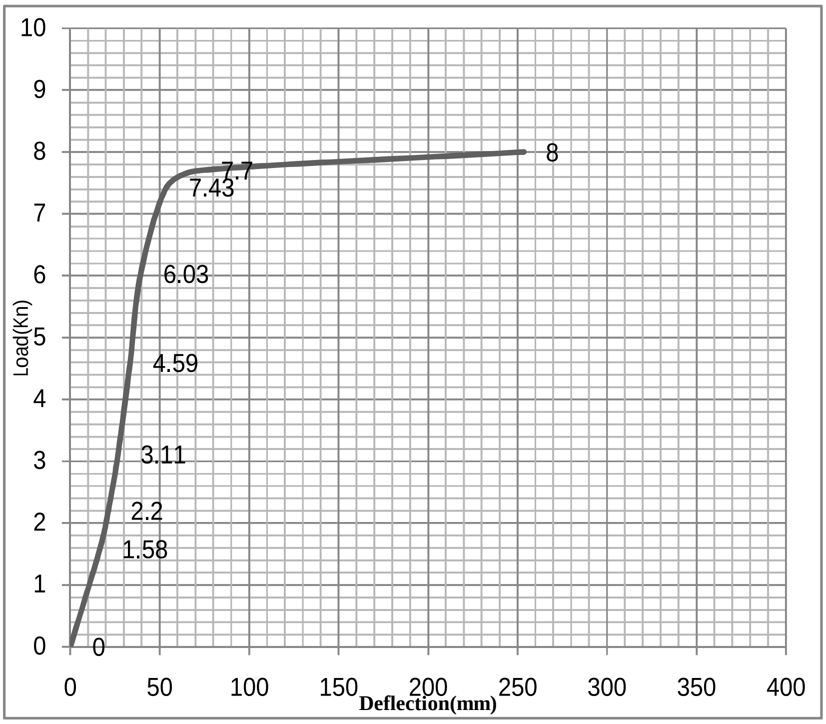


Fig, 8.4d, Specimen, S7

Widening of flexural crack
Specimen, S8

Development of plastic hinge
Crushing and spalling of
Concrete at corner.

Fig.8.4e **Specimen, S8**



Fig, 8.4f, Specimen, S8

4.1.2.CRACK PATTERNS

Refer Drawing file No- 4.1.2

4.1.2.CRACK PATTERNS

Refer drawing file No- 4.1.2

4.1.2.CRACK PATTERNS

Refer drawing file No 4.1.2

4.1.2. CRACK PATTERNS

Refer drawing file No. 4.1.2

4.2. Results(2nd Stage)

The results obtained during testing of retrofitted specimens are tabulated in table 1.7. Ref table, 1.7.

Table.1.7

Specimen N0.	Cracking Load(KN)	Cracking Displacement (mm)	Yield Load (KN)	Yield Displacement (mm)	Ultimate Load (KN)	Ultimate Displacement (mm)
S1	3.11	11.79	7.43	44.67	8	345
S2	2.81	18.41	5.75	55.73	6.15	210
S3	2.2	8.52	7.15	53.11	7.7	149
S4	2.81	13.73	6.31	77.82	6.5	181
S5	1.58	7.3	6.87	34.94	7.15	185.1
S6	1.58	15.34	5.46	60.4	6	173
S7	1.89	9.89	6.59	59.59	7	211
S8	1.89	10.7	6.59	54.12	7	200

4.2.1. Observed damage and behavior of retrofitted specimens-

Damage observed during testing of each specimen is explained below and patterns of crack are shown in fig. 11.

1. Specimen, S1-

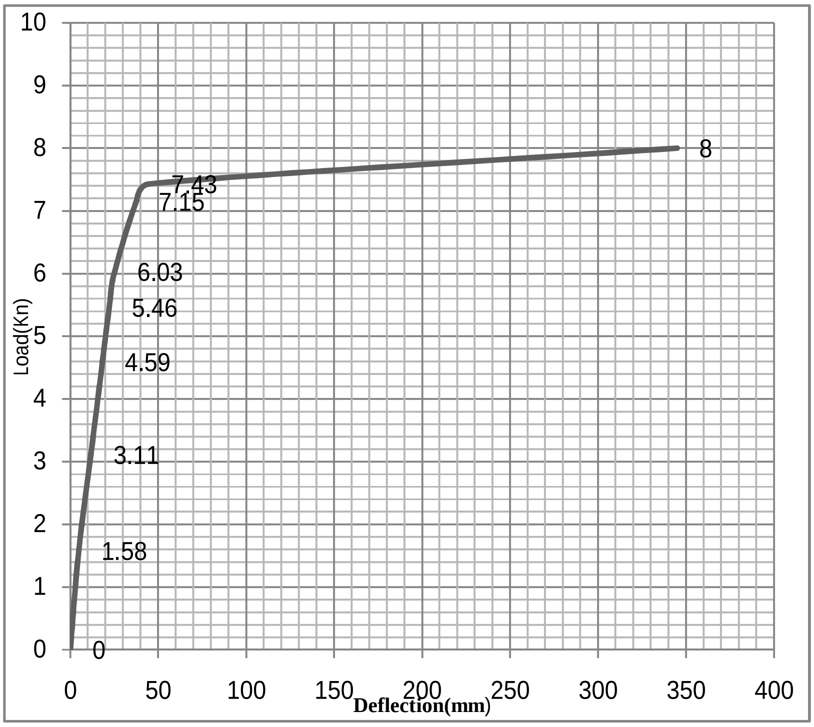
Initial flexural cracks were seen just at the end of FRP wrap and at mid-span at a load of 3.11KN and lateral drift ratio of 1.17%. After further increment of load the crack at the end of FRP wrap begun to propagate rapidly towards footing side, however others cracks also appeared but was not significant. Minor Delamination of FRP on the compression side started at a load of 7.15KN at a drift ratio of 3.755%. Crushing on compression side was observed at a yield load of 7.43KN and drift ratio of 4.4%. No cracks were visible near the column-footing joint. The longitudinal steels were ruptured at 27cm from footing top at a drift ratio of 34.5%. No tearing of FRP was observed. Ref fig. 10


Flexural crack above CFRP wrap

Delaminating of CFRP and Concrete spalling.


Rupture of Longitudinal Reinforcement.

Fig. 10a



Fig, 10b, Specimen, S1

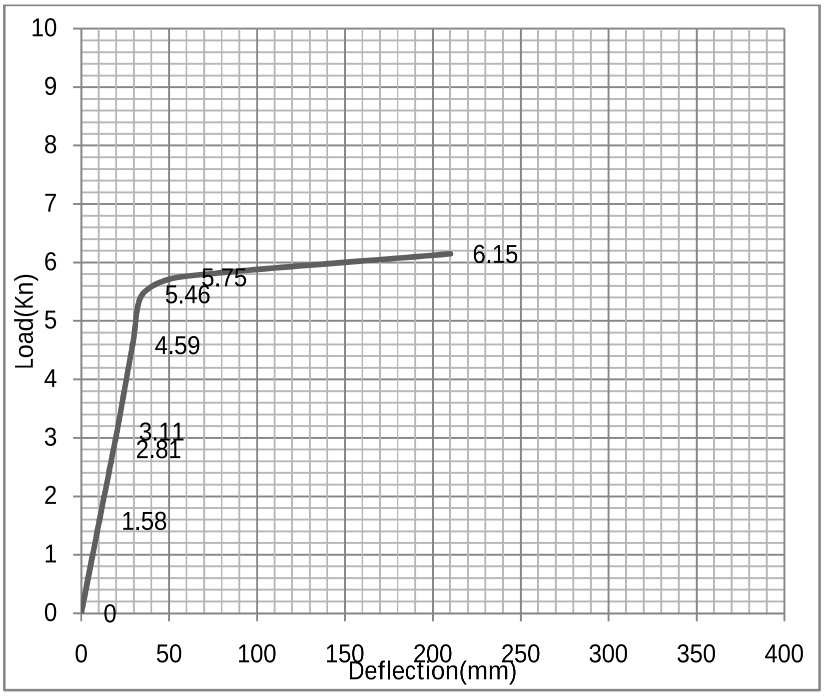
2. **Specimen, S2-**

Early flexural cracks appeared at 24cm and 34cm from column-footing interface at a drift ratio of 2.03% and 3.11KN load. Crack at column-footing joint were seen at a drift ratio of 3.43%. Tearing of FRP jacket at 4cm from column-footing interface started at a drift ratio of 3.43% at a load of 5.46KN. Further increment of load enhanced the tearing and degraded load carrying capacity of column. At a drift ratio of 21% complete delamination of FRP from footing surface took place. In this specimen flexural cracks along the column span did not propagate rapidly but the crack at joint increased comprehensively spalling the concrete and delaminating the FRP jacket of footing top. Ref fig.10.1.

Tearing of CFRP

delaminating of CFRP

Fig, 10.1a



Fig, 10.1b, Specimen S2

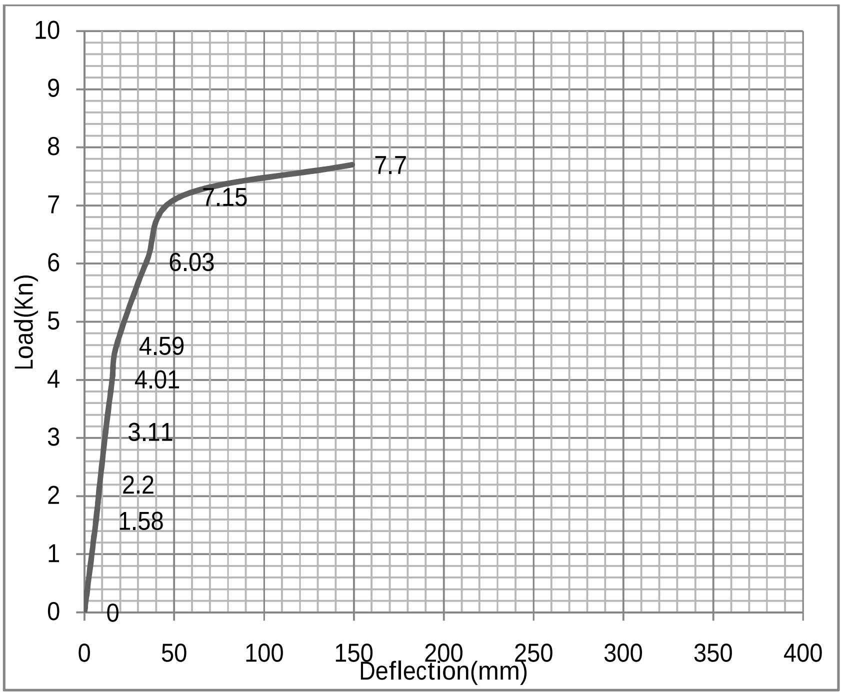
3. Specimen, S3-

In this specimen early flexural cracks were observed at 33cm, 40cm and 47cm from column-footing joint. After further increment of load several hairline cracks were seen but did not propagate significantly. The earlier crack seen at 33cm propagated rapidly and degraded the load carrying capacity of column at a drift ratio of 5.311%. No cracks were observed at the column- footing joint. Ref fig. 10.2.


Flexural crack and tearing
Of CFRP

Delaminating of CFRP

Fig, 10.2a



Fig, 10.2b, Specimen, S3

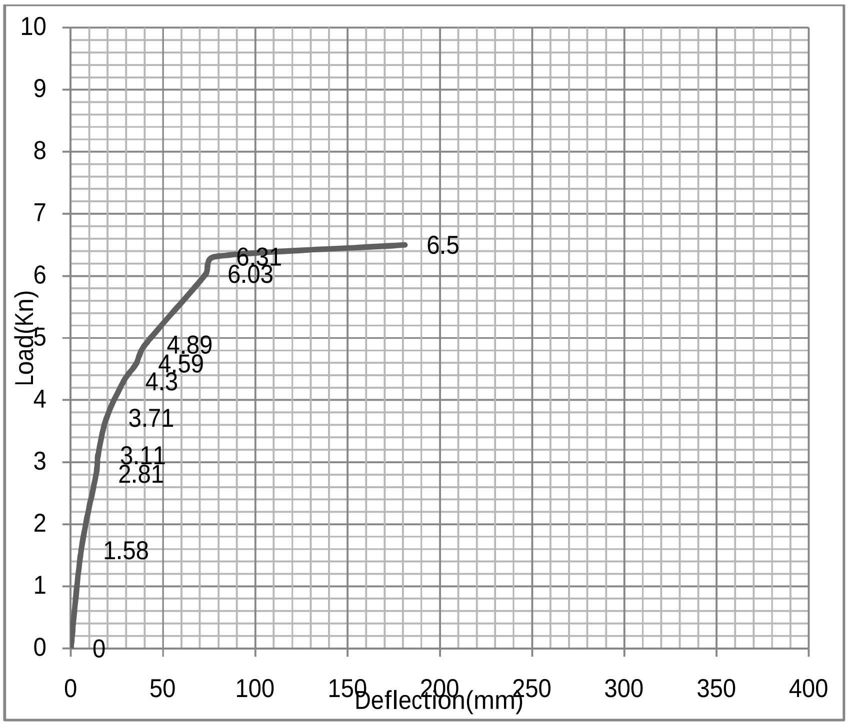
4. Specimen, S4-

In this specimen the behavior seen was similar to S2. The flexural cracks were seen at 35cm, 44cm and 51cm from column-footing joint, but none of these cracks were propagated rapidly but the crack seen at column-footing joint at lateral drift ratio 2.86% propagated rapidly. On further increment of load and lateral drift, crack width at joint level increased and delamination of FRP from footing surface started and complete delamination was seen at drift ratio 18.1%. Ref fig 10.3.

Crack at joint.

Delaminating of CFRP.

Fig, 10.3a

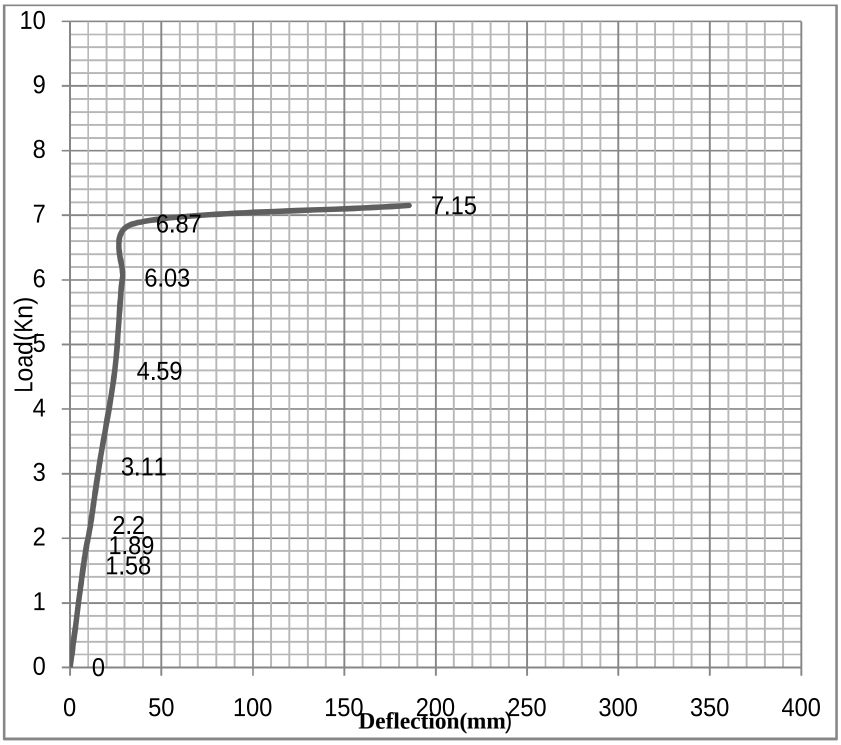


Fig, 10.3b, Specimen, S4

5. **Specimen, S5-**

In this specimen several vertical cracks were seen along the span but the initial crack at 38cm from column-footing joint propagated and was extended towards footing side. At the drift ratio of 18.55% crushing of concrete and spalling of concrete was noticed at this location. Cover of concrete was completely spalled and longitudinal reinforcement was visible. Ref. 10.4.

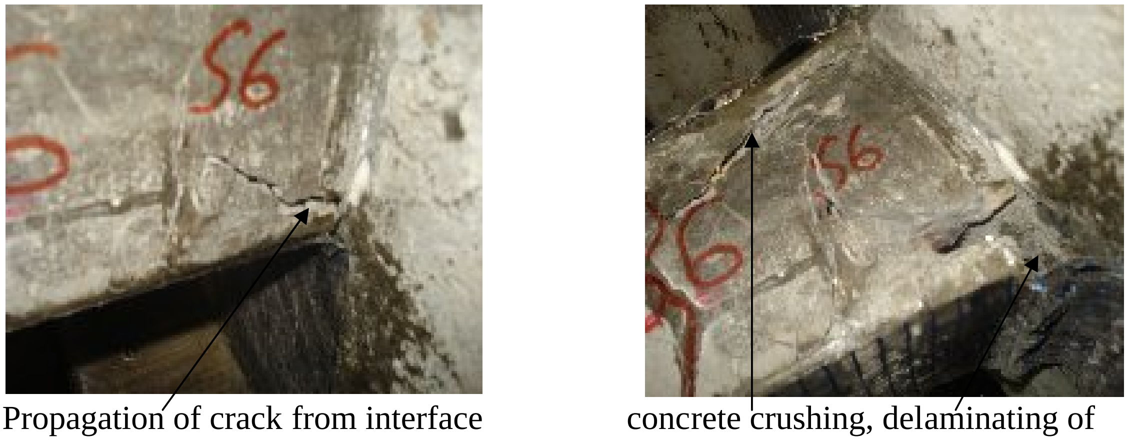
Fig, 10.4a



Fig, 10.4b, Specimen, S5

6. **Specimen, S6-**

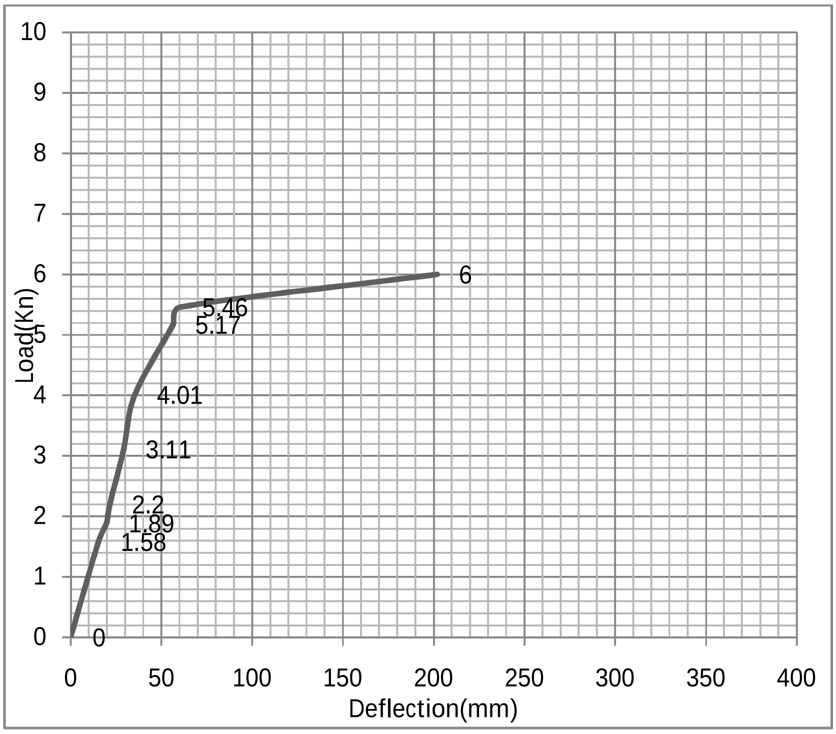
In this specimen the trend of cracks was different than other specimens. First crack was seen at 7cm from column-footing joint at drift ratio of 1.53% and 1.58KN load, but soon cracks were seen at column footing interface and along the span of column. On further increment of load the crack at interface propagated rapidly and it was extended along the footing heading towards compression zone. At the load of 3.11KN and drift ratio 2.918% crushing of concrete on compression zone was observed and it was propagated up to a length of 10cm on the increment of deflection. The load carrying capacity of column was decreased after 5.46KN. The major crack was interface crack which width increased rapidly and reinforcement was visible through it at drift ratio of 17.3%, delamination of FRP was also observed near column footing interface. Ref. fig. 10.5



Propagation of crack from interface

concrete crushing, delaminating of CFRP.

Fig, 10.5a



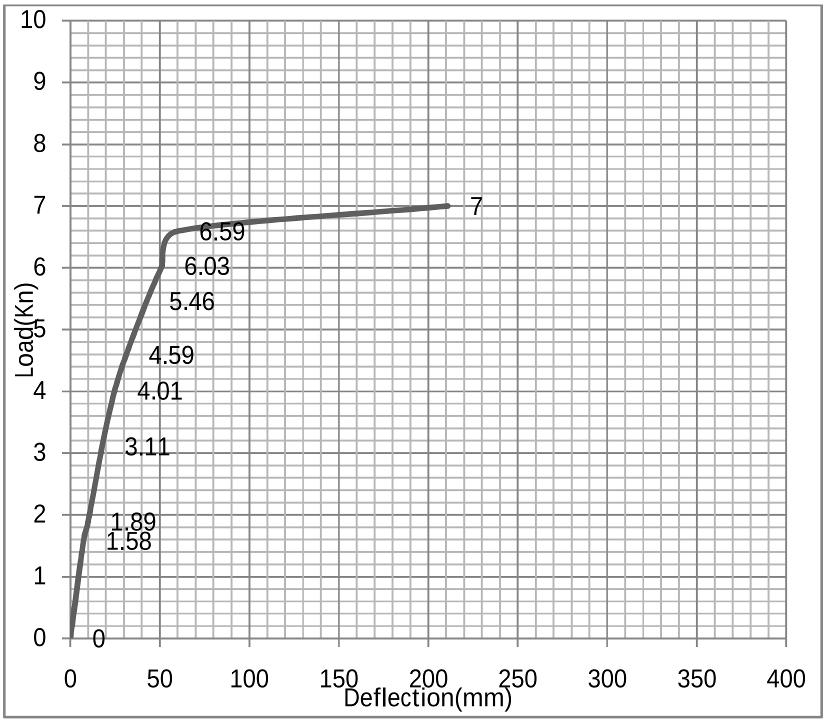
Fig, 10.5b, Specimen, S6

7. Specimen, S7-

The behavior of this specimen was similar to that of S2, only difference was no tearing of FRP laminates was observed up to the drift ratio of 21.1%. like in specimen S2 complete delamination of FRP from footing surface was seen at drift ratio of 21.1%. Ref. fig. 10.6.

Crack at joint complete delaminating of CFRP

Fig, 10.6a



Fig, 10.6b, Specimen, S7

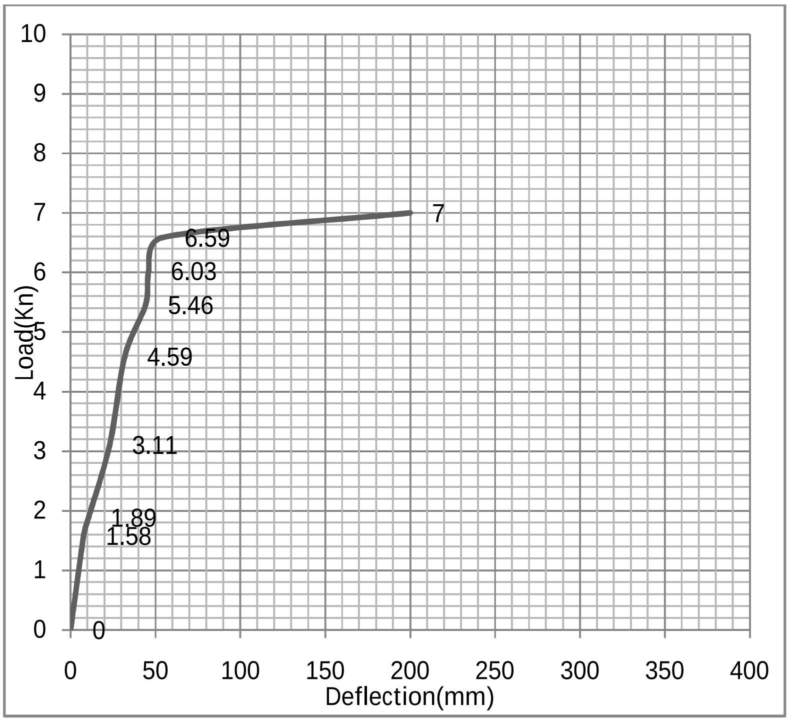
8. **Specimen, S8**

The pattern of failure in this specimen was similar to that of S5, but the lateral drift ratio was more at yielding i.e 5.41% whereas in S5 drift ratio at yielding was 3.49%. Ref fig, 10.7.

Delaminating of CFRP

Ceack extending towards footing
Side and concrete spallimg.

Fig, 10.7a



Fig, 10.7b, Specimen, S8

4.2.2CRACK PATTERNS

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4.2.2.CRACK PATTERNS

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4.2.2.CRACK PATTERNS

Refer Drawing file No-4.2.2

4.2.2.CRACK PATTERNS

Refer drawing file No-4.2.2

CHAPTER- 5

5. CONCLUSIONS AND RECOMMENDATIONS

5.1. Conclusions-

1. All the 8 specimens were tested under monotonic lateral load and three different failure modes were observed in initial testing. All the failures were dominant of flexure. Cracks in the entire specimen started nearby column-footing interface and cracks were seen nearly up to the mid span of column only. Damages observed were rupture of reinforcement, buckling of steel and crushing failure.
2. The localization of cracks was seen near the column-footing interface resulting failure, in initial test of specimens.
3. All the damaged specimens were retrofitted with Carbon fiber reinforced polymer (CFRP) using epoxy resin as bonding agent and all these specimens were tested exactly as before.

   CFRP changed the damage location and patterns of cracks in the column. In first stage testing cracks appeared near column-footing interface whereas in strengthened specimens [with double layer of CFRP in flexure and shear. S1 (with closed loops), with single layer of CFRP in flexure and double layer in shear (with closed loops). S3, S5, S8] the cracks appeared at the end point of CFRP laminate and initially the crack was vertical (flexural) and after reaching nearly the middle of width of column it was extended towards the footing side resembling the flexural-shear crack.

   In the specimen S6 the CFRP was laminated for flexure only (single layer) failure occurred at joint delaminating the CFRP and the load carrying capacity of this specimen was least as compared to other specimens. In specimen S2, S4 and S7 CFRP lamination was single layered for both flexure and shear, therefore all these 3 specimens failed after complete delamination of CFRP at footing surface.
4. With compared to rate of initial damage CFRP laminated with single layer in flexure and double layer in shear (closed loops) gave the best result increasing the joint stiffness.
5. No failure was observed in initial failure location in strengthened specimens due to the confinement by CFRP.

6. For the better confinement of concrete wrapping (with closed loops) of CFRP in column-footing interface is desirable along with flexure laminate along the height of column.
7. The plastic hinge location was shifted in CFRP retrofitted specimens.
8. Specimens wrapped with CFRP showed the significant improvement in ductility.
9. The lateral load carrying capacity of CFRP strengthened column-footing specimens corresponding to initial capacity are 84.62% for S1, 95% for S2, 96.23% for S3, 84.92% for S4, 92.46% for S5, 76.36% for S6, 85.58% for S7, 85.58% for S8.

5.2. Recommendations-

This present study on column-footing assemblages under monotonic lateral load has answered several questions on CFRP retrofitting regarding its strength, failure modes and cracking patterns. However there are other issues also which need to be addressed. Some of those are listed below.

1. The experimental investigation on CFRP retrofitted column-footing assemblages under axial load.
2. Experimental study under combined axial and monotonic lateral load of CFRP retrofitted column-footing assembly.
3. Analysis and behavior of FRP retrofitted concrete column subjected to eccentric loading.
4. Comparative study could be done on carbon fiber reinforced retrofitted column specimen and glass fiber reinforced retrofitted specimens.
5. Test of FRP retrofitted column-footing assembly under cyclic lateral load and constant axial load could be done.
6. FRP retrofitting on columns with lap splices.

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