FLEXURAL BEHAVIOR OF DOUBLE SKIN STEEL TUBULAR BEAMS SUBJECTED TO MONOTONIC AND CYCLIC LOADING.

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ABSTRACT

Double skin tubular members consist of two tubes placed concentrically or eccentrically having space between them which is filled with concrete. This thesis is concerned with the flexural properties of Concrete Filled Double Skin Tubular (CFDST) beams in which both inner and outer tubes are made up of Square Hollow Section (SHS) of steel. A steel strip is used to connect the two tubes. An experimental program consisting of performing four points loading tests on twelve specimens, six for monotonic and six for cyclic loading is carried out. In the study we will focus on effect of design parameter on flexural properties of CFDST beams. Parameters to be studied are the effect of presence of concrete in the inner tube, eccentricity of inner steel tube, provision of minimum steel reinforcement in addition to steel tubes and slip control with steel strip attachment at the end of tube. The CFDST beams fully filled with concrete have slightly higher strength and energy absorption capacity and have shown least deflection but are considered as uneconomical because of the presence of the concrete infill in the inner tube. Also, these members are very heavy due to additional concrete in the inner tube. Hollow CFDST beams with eccentric inner steel tube has higher strength and energy absorption capacity at a lower deflection as compared to the beams with concentric steel tubes. The CFDST beams having longitudinal reinforcement in addition to the steel tubes have shown increased ductility. For beams subjected to cyclic loading, the beams with longitudinal reinforcement have shown greater resistance to cyclic loading as compared to the beams having no longitudinal reinforcement. The failure loads at each point in cyclic loading beams is less or equal to the monotonic loading beams. Stiffness degradation ratio for cyclic loading beams is less than monotonic loading beams because the stiffness is degraded by the preceding cycles. The ultimate deflection value for cyclic loading is greater than the ultimate deflection value for monotonic loading.

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CHAPTER 1 INTRODUCTION

1.1 General

In modern construction steel-concrete composites are widely used as construction materials. In composite construction, steel and concrete are ideally combined to get inherent advantages such as increased strength, ductility, stiffness and economy of the section. One such example of composite construction is that of Concrete Filled Double Skin Tubular (CFDST) members. These members consist of two tubes that sandwiches concrete between them. They can be of circular and square cross sections. In concrete core, a part of concrete is replaced by hollow a steel tube of higher strength than concrete. Such an arrangement of concrete and steel imparts high strength, and light weight characteristics to the structure. These members require a smaller section for a specific value of axial and bending moment, as compared to the reinforced concrete section, which reduces the self-weight of a structure to a great degree. The hollow inner tube can be used to carry electric cables, telecommunication lines and drains. Concrete sandwiched between the two steel tubes is responsible for the delayed local buckling of the steel tubes. Steel tubes acts as a longitudinal reinforcement for these members and it also provide confinement to concrete, thereby limiting the stresses transferred to the sandwiched concrete. CFDST members are modified form of Concrete Filled Tubular (CFT) members. CFT members are used as an important structural element in buildings particularly to provide resistance to earthquake [1]. CFT members are used in civil engineering structures due to its good plasticity, increased capacity, good earthquake resistance and properties of ease in construction [2]. The CFT dissipates more energy as compared to RC elements and conventional steel elements [3]. In these members steel tube confines the inner concrete and the concrete delays the local buckling of the steel tube. CFDST provide increased ductility, greater energy absorption capacity and higher resistance to fire [4].

CFDST members can be found in Circular Hollow Section (CHS) or Square Hollow Section (SHS) [5]. CFDST members possesses all advantages that are found in CFT [6, 7]. The CFDST members have some additional advantages as compared to CFT. Presence of inner tube in CFDST reduces the self-weight of the structure [8-11]. The hollow space at the center of these materials can be used for accommodating wires and sewer pipes of buildings [12]. One of the most important advantage of using these members is its excellent fire resistance, the inner tube is protected by outer concrete covering due to which it retains its strength during fire [13]. Better fire resistance is expected because the inner tube is protected by concrete [7]. The excellent fire resistance of CFDST members as compared to CFT members is due to the protection of inner steel tube by concrete [14].

The CFDST members are considered as lighter in weight because these members have hollow inner tube. These members eliminate use of formwork in construction and that's why they can be used in offshore construction [8]. Concrete sandwiched between steel sheets was used for construction of submerged tunnels by the team of Consultants at Cardiff UK three decades ago [15]. Nowadays, world is aiming at developing high strength composites that are made from combination of concrete with FRP or steel. Various sections of CFDST members are shown in Figure 1. The double skin tubular members are mostly used for in the sections subjected to compression. The higher ductility of Hybrid-CFDST members is due to the FRP confinement provided to concrete and a very ductile steel tube that acts as a longitudinal reinforcement [16].

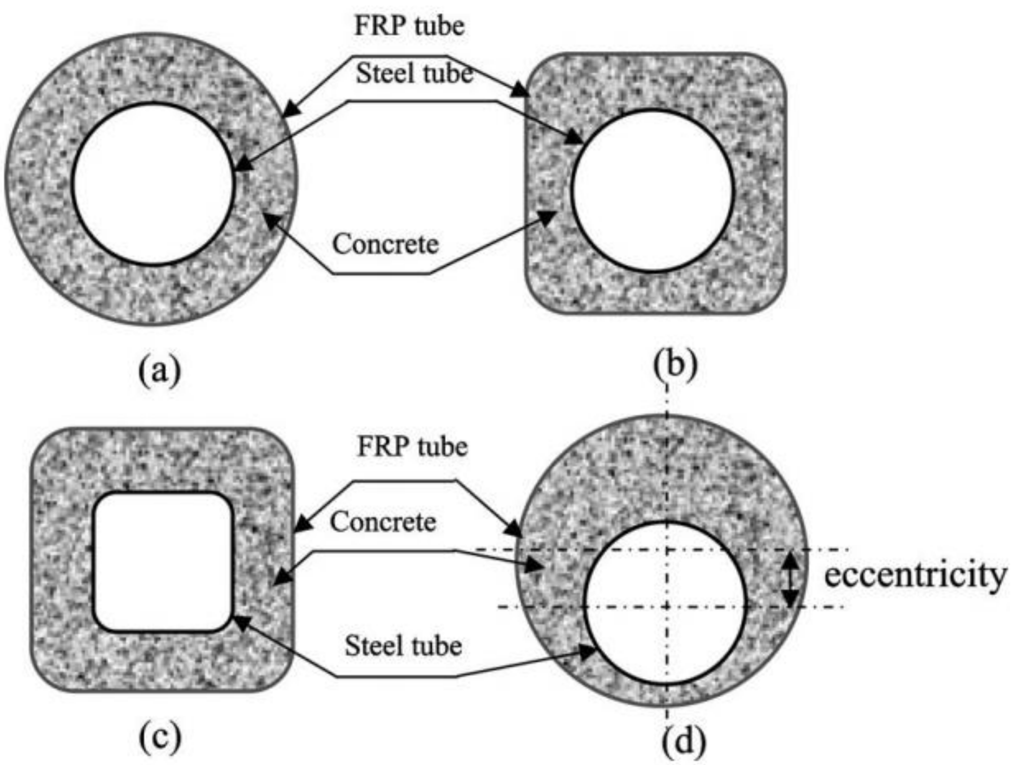


Figure 1 :Sections for Hybrid Double Skin Tubular Members [17]

In recent times the researchers came to know that using double skin tubular members have some additional advantages. Some of the advantages are increased section modulus, enhanced stability and higher strength, reduced weight, increased ductility and improved damping of the structure. Concrete Filled Double Skin Tubular (CFDST) sections possess good fire resistance because the inner tube is covered with concrete from fire. Another advantage in using these sections is that no formwork is needed during construction thereby limiting time of construction. These members can mostly be used for precast construction.

The double skin steel tubular members are mainly studied as columns that are subjected to compression. Little attention has been given to the study of CFDST members as flexural member. As steel is an isotropic material and have same lateral and longitudinal strength, so it can be used in enhancing the flexural strength of beams. On the other side, Fiber Reinforced Polymer (FRP) has greater hoop strength but it is relatively weak in longitudinal direction. The thesis is concerned with a member having both inner and outer square steel tubes made up of steel. In this paper, a brief overview on monotonic and cyclic loading of these members is given. All the specimens were tested using four points loading procedure.

1.2 Applications of double skin tubular members

These members have variety of applications. Nowadays, world is aiming at the development of materials that can be used for high speed construction. CFDST members can be used for precast construction which can save time and economy simultaneously. Concept of CFDST members was originally obtained from submerged tube tunnels. These structures are believed to be used for containing liquid, gasses and nuclear structures [15]. This type of construction can be used for constructing seabed vessels in deep water at the legs of platforms, for the columns having larger diameter and, in the structures that are exposed to ice loading [18]. Recently, it was used in high rise piers in japan which has reduced weight accompanied by increased energy absorption capacity [19]. The fiber reinforced polymer has found many applications in civil engineering. Main advantage for using FRP is its high strength to self-weight ratios. Researchers are trying to use them in new construction despite their applications in retrofitting [17].

CFDST has been used throughout the world as beams and as beam-columns, in both braced and unbraced elements of structural systems. They have wide range of applications such as compression member in low rise structures, construction of open floor, steel circular tubes filled with concrete or pre cast concrete etc. Concrete filled square box and circular columns have been used in some of the world tallest buildings [20].

1.3 Research Problem

Nowadays, Fiber Reinforced Polymers FRP are used for external strengthening of structural elements such as beams and columns. It is good to be used for columns because it has high hoop strength. For beam we need a material that can resist forces in transverse and longitudinal direction. As we know steel is an isotropic material i.e. it has same properties in both directions. When we externally reinforce beam with steel, it provides both high hoop and longitudinal strength. Also, there is limited study available on CFDST beams having both inner and outer square steel tubes. As this type of member has two tubes so, there is no need of formwork for their construction.

1.4 Relevance to the national needs

Trend of using double skin tubular sections is rising in many developed countries. However, they are of little use in Pakistan. It can aid to our national needs, by building safe and stable structures. CFDST members provide excellent monotonic and seismic resistance to the structure. These members also behave good in situation where they are subjected to axial load and biaxial bending. They should be used because they have high strength to weight ratios, provides continuous confinement which delays local buckling, improved damping behavior, high toughness and ductility of the structure. Smaller sections are given by these members for large loads. Ductility of these members is very high as compared to reinforced concrete structures i.e. can be used in high seismic zones. Using minimum longitudinal reinforcement in these members, cyclic loading strength and ductility can be improved. These members can be used as pre-fabricated elements, so it can save time of construction. In addition, the member can be reused after dismantling of a structure.

1.5 Research Methodology

Research methodology consists of following steps.

a. Mix design for high strength concrete having compressive cylinder strength of 50 MPa was prepared performing many trials.
b. Fabrication process consist of making two L-shaped sections from steel sheets of thickness 2 mm. The 2 L-shaped sections were welded together from two ends to make square tubes.

c. Then the two tubes were placed at specified eccentricity and welded with metallic strips of 12.5 mm width and thickness of 3.125 mm.
d. The members having longitudinal steel reinforcing bars are tied with binding wire to maintain spacing for bonding.
e. The assembly formed by two tubes is filled with high strength concrete.
f. The samples were kept for 28 days to gain required compressive strength.
g. The samples were cured at their top and bottom for 28 days.
h. After 28 days, samples were shifted to reaction floor. Samples were marked with numbers for recognition.
i. Samples were lifted with crane. First 6 samples for monotonic loadings were tested and rest of 6 samples were tested for cyclic loadings.

1.6. Research Objectives

Main objective of research is to study the effect of some parameters on the flexural behavior of Concrete Filled Double Skin Steel Tubular (CFDSST) beams. The parameters studied under monotonic and cyclic loadings are

a. The effect of hollowness on the flexural behavior of CFDST Beams. This parameter was studied because by comparing behavior of this sample with hollow beams we can judge the effect of concrete infill in the inner tube.
b. When the inner tube is shifted towards the tension side, the flexural response is affected. The greater quantity of concrete in compression zone can increase the flexural capacity of these members.
c. Steel is a high strength material and can resist both tension and compression. That's why flexural response can be improved by using more steel in addition to the steel tubes. The effect of provision of longitudinal steel reinforcement on the flexural behavior of CFDST beams is studied.
d. Main problem in using double skin tubular members is the slip between concrete and steel interface. Slip can be controlled by providing some obstruction in or outside the steel tube. Slip control due to the provision of small steel strips is studied.

1.7 Scope of the research

Scope of research include study of the flexural behavior of Concrete Filled Double Skin Steel Tubular (CFDSST) beams subjected to cyclic and monotonic loadings. From literature, one can infer that there is limited knowledge on the flexural behavior of CFDST members having both inner and outer steel tubes made up of steel. This study reports the minor changes in design of CFDSST beams which can improve strength, ductility, ultimate deflection and stiffness.

CHAPTER 2: LITERATURE REVIEW

2.1 History of Double Skin Tubular Members

Double skin tubular members were first studied in late 1980s, which consists of two circular tubes placed concentrically [21]. Concrete sandwiched between steel sheets was used for construction of submerged tunnels by the team of Consultants at Cardiff UK three decades ago [15]. CFDST members have lighter weight because they are hollow. Use of formwork in construction of these members is eliminated that's why they can be used in offshore construction [8]. This member was considered good for constructing seabed vessels, in deep water at the legs of platforms, in the columns having larger diameter and to the structures exposed to ice loadings [18]. Recently it was used in high rise piers in japan which has reduced weight with increased energy absorption capacity [19].

Concrete filled steel tubes (CFST) have been used throughout the world as beams and as beam-columns in both braced and unbraced elements structural systems. They have been used in variety of constructions such as members of compression in low rise, construction of open floor, steel circular tubes filled with concrete or pre cast concrete etc. Concrete filled square box and circular columns have been used in few of the world tallest buildings [20]. These members are thought to be a sustainable alternative to bridge piers and heavy building columns. They are also used as transmission towers. These members provide saving in material and construction cost. Their use has been very common since 2000. They have been used in new construction as well as retrofitting of the existing structural elements.

2.2 Types of Double Skin tubular members

These members can be classified based on materials and geometry.

2.2.1 Classification based on materials

Based on the outer material used they can be classified as under

1. Members having inner steel tube with outer FRP Covering, shown in Figure 2.
2. Members having inner and outer tubes made up of steel, shown in Figure 3.

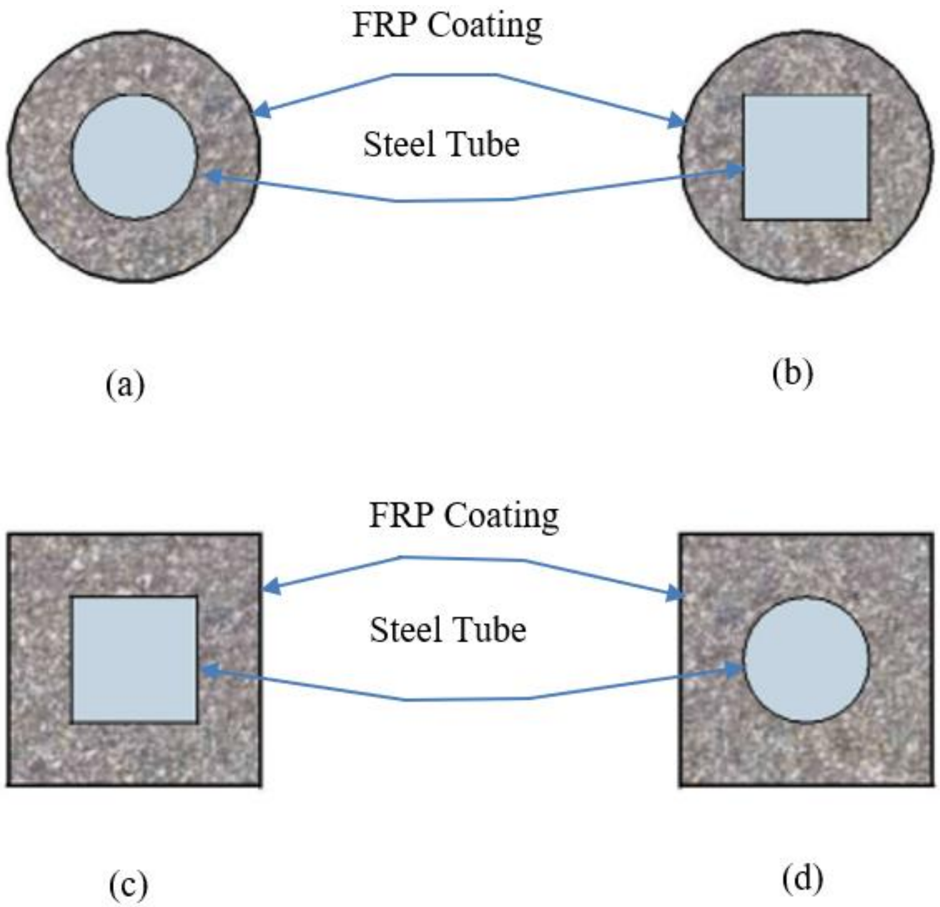


Figure 2: Classification of CFDST beams based on material

2.2.2 Classification based on Geometry

Based on Geometry the members can be classified as

a) Both inner and outer tubes are of circular shape.
b) Having outer circular and inner square tube.
c) Both inner and outer tubes are of square shape.
d) Having outer square and inner circular tube.

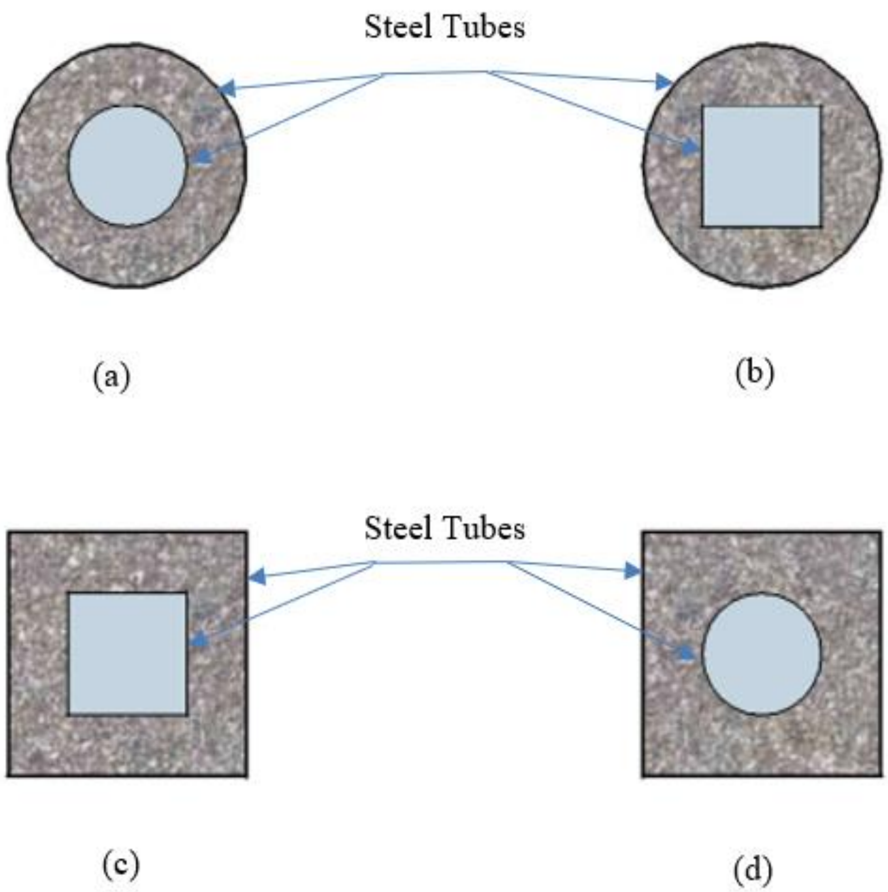


Figure 3: Classification of CFDST beams based on geometry

2.3 Previous studies conducted

The literature is classified in three categories. First the concrete filled steel tubes are examined, second the Hybrid-CFDST are explained and third the CFDST members are discussed briefly.

2.3.1 Concrete Filled Steel Tubes

Large no of concrete filled square tubular beam-columns are studied under the application of the shear force in cycles. Little experimental work is available on these members under uniform cyclic bending. Main objective of the research is to make some improvement in the behavior of square CFT beam-columns without consideration of the shear behavior. Experiments are carried out on eleven specimens of size 200*200*600 mm subjected to cyclic and monotonic loading. The main parameters considered are history of deformation, ratio of axial load, width to thickness ratio of the steel tubes. The tensile strength of steel and concrete cylinder strength is taken to be 50 MPa and 400 MPa. For confined concrete and locally buckling steel, the load at failure and deformation time histories were compared with those elastic plastic responses based on proposed stress-strain behavior. There was good agreement between experimental and analytical behavior [22].

Experimental investigations are performed on concrete filled square steel tubular (CFT) beam columns with varied strength of materials. The strength of steel used are 400 MPa ,500 MPa and 780 MPa and strength of concrete used are 40 MPa and 90 MPa. In addition to the variation in material, the width to thickness ratio of steel tube is also considered. Information is obtained about the hysteric behavior of CFT beam-columns with varying material strengths. 33 beams were used in the study. Higher the steel thickness and strength lead to better overall behavior of the specimen. While, the greater concrete strength has negative effect on the overall behavior of the specimen. An excellent agreement between experimental and analytical results except few exceptions is observed in this study [23].

The behavior of columns subjected to the constant axial load, accompanied by lateral cyclic load is studied. Six columns are tested in this study. Parameters studied are level of axial load, ratio of diameter to thickness and compressive strength of concrete tube. Increased ductility and enhanced strength are shown, by the specimens due to the confinement provided by the steel tube. The model foe circular CFT beam-columns is developed, that simulates interaction between concrete and steel. Good agreement between analytical and experimental result was found [24].

2.3.2 Members having inner steel tube with outer FRP Covering

Hybrid concrete filled tubes of fiber polymers are studied under the application of axial loading in compression. Fibers have large rupture strains, made up of recycled materials and are considered as environment friendly. Finite element study is also performed on these members using the software LS-DYNA. The main parameters studied are column's aspect ratio, size of column, unconfined concrete strength and confinement ratio of concrete. Comparison between these members and the ordinary FRP tubes is also carried out. A hybrid material that combines the traditional material with new material in a specific manner have shown a remarkable improvement behavior under axial loading. Some equations for new hybrid system are also suggested [3].

Finite element analysis of CFDST column having outer tube of Fiber Reinforced Polymer (FRP) and inner tube of steel is conducted. Pushover analysis is used to simulate the seismic performance of three dimensional CFDST by using FE software. The results from analytical model are compared with experimental study, conducted on seven columns. The effect of load level, number of FRP layers, diameter to thickness ratio of the steel tube and thickness of the concrete used are also studied. A complex behavior was shown by CFDST columns; effected by combination of stiffnesses of FRP layer, steel tube and concrete [25].

The FRP-concrete-steel Double-Skin Tubular columns, in which high strength (HSC) concrete is sandwiched between outer FRP tube and inner steel tube is studied. The sample of column is fixed in the foundation for testing, as shown in Figure 4. For the first time, this type of member subjected to both cyclic lateral loading and axial compressive loading is studied. 8 columns are tested with the outer diameter of 300mm. Result shows that Hybrid DSTCs have high ductility and resistance to seismic loading even with high strength of concrete of 120 MPa. The plastic hinge development is observed at the bottom of the column. It is suggested that plastic hinge can be prevented by filling the internal gap in the inner tube. An important data is provided researcher, for the verification of hysteric behavior of these members with the theoretical results [26].

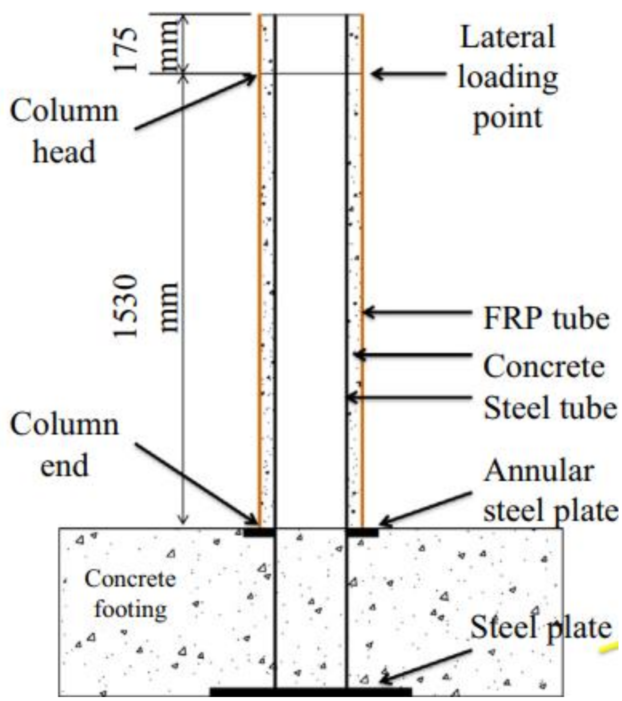


Figure 4: Foundation supporting column [26]

Work on the member which inner tube is made up of steel and the outer tube is made up of high strength fiber reinforced polymer is carried out. Comprehensive tests are conducted on 18 specimens; to obtain values of rotation, bending and strain at the top and bottom fibers. Detailed information about ductility, stiffness and strength of externally reinforced members subjected to flexural loading is given. Different layouts and quantities of sheets are used in Circular Hollow Section (CHS) of beams. Effect of loading capacity on orientation of fibers in transverse and longitudinal direction is studied, as shown in Figure 5. The fibers amount and its orientation have a pronounced effect on strength of the member. An analytical method was devised for these members using the concept of modular ratio and the section's slenderness limits [27].

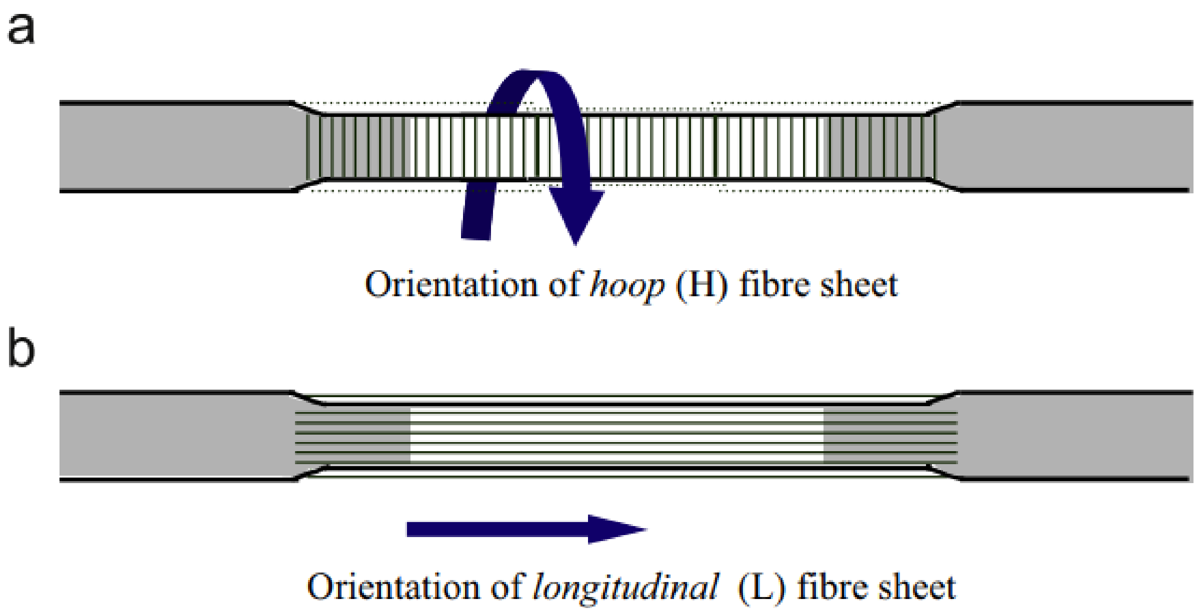


Figure 5: Orientation of fibers on beam [27]

Hybrid-CFDST members filled with high strength concrete is studied and discussed in detail. Beams were tested in four-point loading setup, as shown in Figure 6. The effect of inner tube's diameter and influence of mechanical connecters on the flexural behavior of the DSTBs is studied. All the specimens have 150 mm cross section and length is 1500 mm. One specimen has inner steel tube's diameter of 114.4 mm and yield stress of 436 MPa. Other two specimens have inner tube's diameter of 76.1 mm and yield strength of 398 MPa. All the specimens are filled with high strength concrete. The outer layer of all the specimens is made up of two layers of aramid FRP. The third specimen is provided with steel rings of 8 mm diameter that served as mechanical connecters. These members can resist high deformations in inelastic stage. The slip between the concrete and steel can be controlled by the use of mechanical connectors [28].

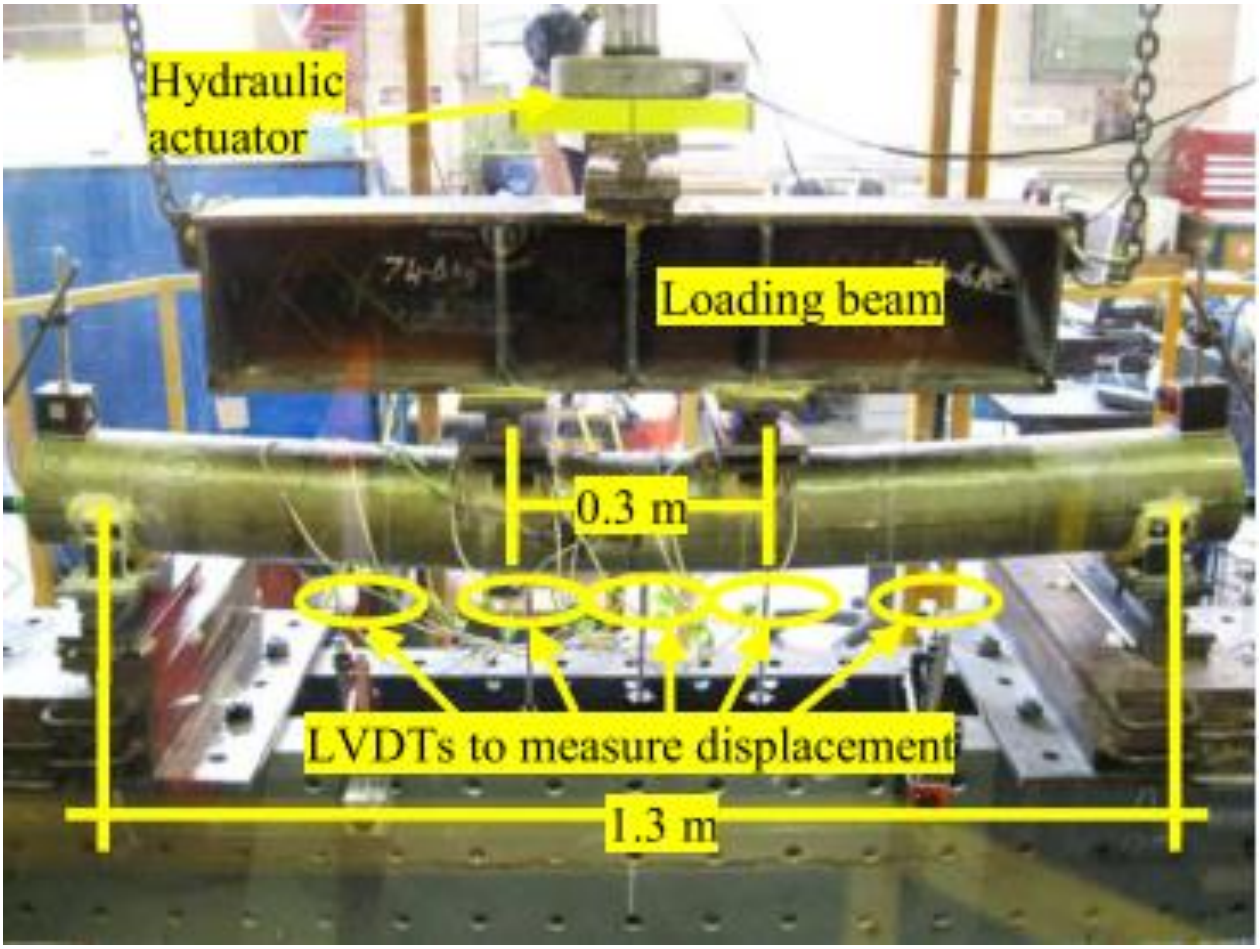


Figure 6: Four Point Loading of Hybrid-CFDST beams [28]

Four beams with Area of 200*150 mm and length of 2300 mm are tested. One beam is studied as a reference which is not covered with FRP sheet. Three beams are reinforced with different number of layers of FRP. Second series beams consist of testing the beams of cross section 200*150 mm and length of 2000 mm. These beams are wrapped with different numbers and widths of CFRP layers. These members have shown increase in stiffness and flexural response. Cracks of less width and the decreased number of cracks are observed. Moreover, a satisfactory behavior of these members is found from force-displacement diagram in cyclic loading. A good agreement between analytical and theoretical results is observed [29].

Flexural behavior of circular Hybrid FRP-concrete-steel member is studied, along with theoretical model based on fiber element approach and plan section assumption. The parameters addressed are configuration of a section, strength of concrete, and thicknesses of both steel and FRP tube. Two series of experiments were conducted on Hybrid-CFDST beams (Figure 7). All the specimens have inner diameter of 69 mm and outer diameter of 152.5 mm with overall length of 1500 mm. In first series 8 tests are performed and in second series 6 tests are performed on these members. All the specimens are tested under four-point loading tests. An improvement in flexural behavior, when the inner steel tube in beam is shifted towards the tension zone is observed. There is an agreement between theoretical and experimental model, the difference is attributed to the neglect of factors that should be otherwise considered in the design [16].

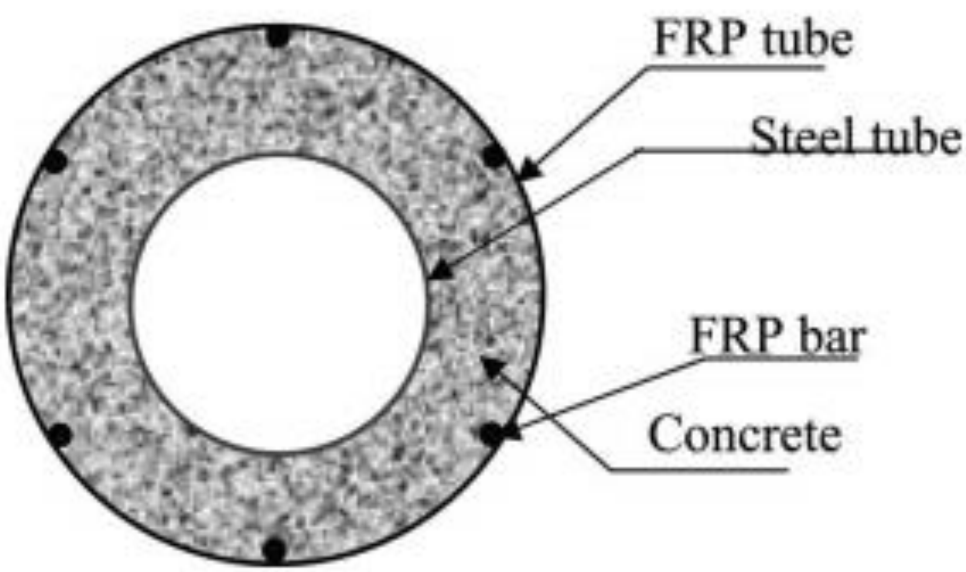


Figure 7: Cross section of Hybrid-CFDST beam [16]

2.3.3 Members having inner and outer tubes made up of steel.

Double skin tubular (DST) stub-columns are studied experimentally. The inner rectangular tube is made up of Stainless steel (grade 1.4003) with the range of depth to thickness ratios from 26.1 to 42.1. Whereas, the outer tube is made up of carbon steel of grade S235 with range of width to thickness from 7.5 to 13.5. To evaluate the axial compressive behavior total of 14 beams are tested. Concrete filled between the two tubes has cylinder strengths of 40, 80 and 120 MPa. The column strengths, axial load-strain relationships and failure modes are studied in this paper. The calculated strengths are compared with different design codes. The designed models that were obtained from design codes give good anticipation for CFDST columns. The test results match with AS5100 and

EC. Results given by American specifications ANSI/AISC 360-10 and ACI are also conservative [30].

The behavior of CFDST stub columns with outer square hollow section (SHS) and inner square or circular hollow section (SHS or CHS) is studied. The models are developed for different possibilities of CFDST stub columns using FEM software i.e. verification of FE model using different results from research papers, stress and strains in different components of the section and parameters that effect the sectional capacities are three main objectives of the study. There is good agreement between experimental and analytical results. The effect of hollow section ratio is more pronounced on sections having inner circular tubes, as compared to the sections having inner rectangular tubes. Furthermore, the increase in stiffness of a member at elastic plastic stage of axial load verses Strain is observed with the increase in hollow section ratio [31].

Series of experiments on 14 CFDST stub columns are conducted, that contains 4 beams and 12 beam-columns. Outer skin tubes comprise of square hollow section, while the inner section of all the members is circular in shape. Parameter studied are variation of hollow section ratio, outer tube width to thickness ratio varying from 40-100, eccentricity of load from 15-80mm and slenderness ratio of columns from 29-58 are considered. In addition, a mechanical model is developed to illustrate the above three members. The enhanced durability is found due to composite action of these members. Secondly, load verses axial strain relationships for stub columns and load verses deflection relationship for beams and beams-columns is also developed. The models developed for the above three members have shown good agreement with the experimental results [32].

The behavior of Hollow Core Fiber reinforced polymer Concrete Steel HC-FCS column subjected to seismic (cyclic) loading is studied. The HC-FCS section consist of inner steel tube and outer fiber-reinforced polymer that sandwiches concrete between them. The test specimen consists of fitting of column into the foundation by socket connection and the length of embedment of steel tubes is kept as 25 inches. The study consists of casting 0.4-scale HC-FCS. In this member the outer and inner diameter of column is 610 and 406mm respectively, while the height to diameter ratio is kept as 4. Research findings are compared with the column tested by ‘’Abdelkarim et al. 2015’’ under the identical loading conditions. It was found that column failed at drift of 8.4% due to wall crushing and buckling of steel, which is then followed by ductile tearing at column-footing [33].

Tests are conducted on columns in two Series. In first Series, axial force is applied at both ends both eccentrically and concentrically. The length of buckling and section depth ratio (L_k/D) is selected as 4,8,12,18,24,30. In Second Series, the cantilever beams are both subjected to axial and lateral loads. The L_k/D is kept as 6,9,12,18 and 24. The parameters addressed are section depth ratio and buckling length of the column. Evaluation of the behavior and maximum load capacity by experiments is the main objective of the study. In addition, the design formulas for concrete filled slender steel tubular columns is related to those of short columns. The outer tube is a square hollow section and the inner tube is a circular hollow section, both made up of mild steel. The $L_k/D=4$ gives very conservative strength and the L_k/D ratios from 8 to 30 gives compatible results with the design method. Also, the design formula in cases like described above needs refinement [34].

The Concrete-filled double skin tubular (CFDST) stub columns having outer tube of stainless ferritic steel(grade1.4003) and the inner tube(S235) is made up of carbon steel are tested. 14 specimens have been tested, to investigate the axial behavior in compression and the strength acquired by CFDST specimens. Depth to thickness ratios ranges for outer and inner tubes are 26.1 to 42.1 and 7.5 to 13.5 respectively. Concrete of 40,80 and 120 MPa is used in the study. Length is changed, in order to keep the length to depth ratio constant. A uniform axial compressive force on the specimens is applied. Parameters evaluated in this research are axial strength of columns, load verses axial strain relationships and modes of failure. Test results and design codes are compared. Guidelines from European, Australian and American specifications are obtained. The existing design codes have given conservative predictions about the CFDST stub columns with outer ferritic stainless-steel tubes [30].

Experiments are carried out on twelve beam-columns and twelve stub columns having, both outer and inner tube made up of circular steel hollow sections. Parameters taken into consideration are hollow section ratio, diameter to thickness ratio for stub columns. Parameters for beam-columns were slenderness ratio and eccentricity of load. A theoretical model is also developed for these members. To describe confinement action between steel and concrete tube, the concept of confinement ratio is introduced. Good agreement is observed between analytical and experimental results [7].

Fire tests are conducted on full sized self-consolidated Concrete Filled Double Skin Tubular (CFDST) columns. Failure mode of each component is studied in detail while keeping deformation, temperature and fire resistance into account. Six full size beams are casted with length of 3810 mm and having different geometrical shapes of outer and inner tubes. Performance in fire is tested by outer tube's limiting temperature, concrete and steel's composite action and the effect of fire endurance on different parameters. Limiting temperature in the CFST columns is lower than that of CFDST, and the higher fire resistance of CFDST is due to the composite action of concrete and steel tube. Also, other parameters such as cavity ratio are also dependent on fire exposure [13].

Research is conducted on Concrete Filled Double Skin Tubular CFDST members used as deep beams. The objective of study is to make a deep insight into the mechanical behavior of a CFDST members used as deep beams, subjected to both shear and bending due to static three points loading. The parameters considered are outer tube to thickness ratio and inner tube to outer tube ratio. 12 specimens are casted. Length of the tube is 420 mm, outer tube's diameter is 160 mm. Inner diameter to thickness ratio varied from 69.6-160 and the inner diameter to outer diameter ratio Di/Do varied from 0 to 0.70. Ratio of inner tube diameter to the outer tube diameter governs the failure mode. The strength recorded for the value of Di/Do less than 0.47 matches the calculated values [11].

Experiments are carried out on twelve beam-columns and twelve stub columns having both outer and inner tube made up of circular steel hollow sections. The parameters taken into consideration are hollow section ratio, diameter to thickness ratio for stub columns. The parameters for beam-columns were slenderness ratio and eccentricity of load. They also developed a theoretical model for two types of members. To describe the action of confinement between steel and concrete tube, the author introduces concept of confinement ratio. The author found a good agreement between analytical and experimental results [7].

A mechanical model of Concrete Filled Double Skin Tubes (CFDST) beam-columns under the application of increasing flexural cyclic load and constant axial load was developed. In this paper, sections studied have outer square tube and the inner tube is circular (Figure 8). The cyclic response obtained for the CFDST columns are in good agreement to the theoretical results. Based on theoretical model; the parametric analysis is performed based on moment verses curvature response, lateral load verses corresponding deflection relationships and ductility coefficient are also discussed. It is inferred that the theoretical and analytical results are in good agreement and the formulae generated can be used in building codes to anticipate the ductility of these members [10].

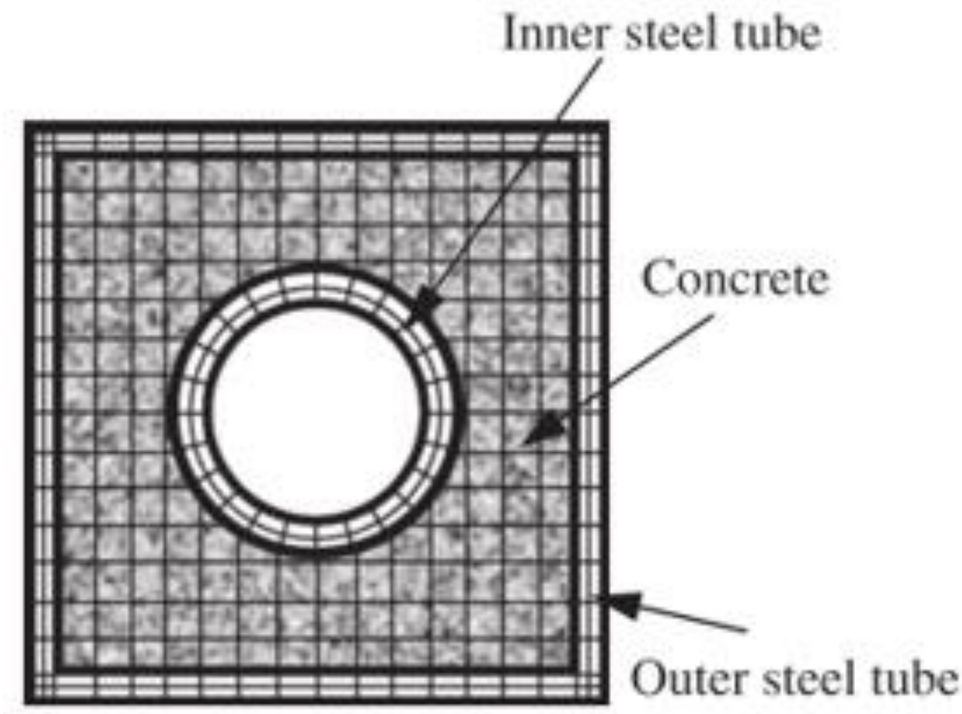


Figure 8: Sections of CFDST members [10]

Tests are conducted on number of Concrete Filled Double Skin Tubes CFDST exposed to bending and compression. The inner and outer tubes both are made up of Square Hollow Sections SHS of cold formed steel(C450), as shown in Figure 9. The width to thickness ratio of outer tubes 16.7 to 25 are chosen for four different section sizes. Whereas, the section size for inner tube is kept as 20, for all sections. AS4600 is used to determine the geometry of the specimen to be tested. Length of the specimen is taken as 375 mm. An increase in ductility of these members and three time increase in rotational capacity of CFDST member, as compared to empty outer tubes with outer tube to thickness ratio of 17 to 25, is observed. There is good agreement between theoretical and experimental models [9].

Figure 9: CFDST stub column [9]

Hybrid-CFDST sections used as beams with circular cross sections have some limitations. FRP has greater tensile strength only in one direction. FRP is relatively weak in bending and have linear stress strain behavior. The lower value of ductility makes performance of these sections weak in earthquake loads [35]. In addition, the square section has moment of inertia greater than circular section. Stress distribution in circular section is also difficult to understand as compared to the section with square cross section.

This thesis discusses in detail the flexural properties of Concrete Filled Double Skin (SHS outer and SHS inner) Tubular (CFDST) beams having both inner and outer square tubes made up of steel and the space between them is filled with concrete. It can provide better bending resistance to the applied loads, since the square cross section has greater moment of inertia. Steel is used for reinforcing concrete from outside because steel has same lateral and longitudinal strengths. High initial cost of steel is compensated by elimination of the cost of formwork required for casting regular concrete members. There is a little study on flexural performance of CFDST members, with square cross section. In this paper, improvement in flexural properties of these members under monotonic loading and cyclic loading is studied. Experimental program consists of testing twelve beams in which six were tested under monotonic loading and rest were tested under cyclic loading.

CHAPTER 3 EXPERIMENTAL PROGRAM

3.1 Mechanical Properties

Material selected for fabrication of Steel tubes was made up of steel sheets having thickness of 2 mm. Material properties such as modulus of elasticity E, yield strength f_y and ultimate strength f_u of steel sheet was found using ASTME8/E8M-13a. Tensile properties of steel bars were obtained using testing standard ACI 440.3R (2004). Steel sheets were bent into two L shaped sections which were then welded to make square cross sections of 152.5 mm and 76.25 mm. Mechanical properties of steel tubes are shown in the Table 1**.**

Table 1: Mechanical Properties of steel tube

Steel Tube Properties	
Thickness of steel tubes(t)	2 mm
Inner diameter of steel tubes(di)	76.25 mm
Outer diameter of steel tubes(do)	152.5 mm
Elastic modulus(E) of steel tubes	200 MPa
Yield strength of steel tubes (fy)	305 MPa
Ultimate strength of steel tube (fu)	368 MPa
Length of steel bars	1450 mm
Elastic modulus of steel bars (E)	276 MPa
Diameter of steel bars	6 mm

Mix design of concrete to achieve the target strength was found by making some trials. The cylinders of size 100*200 mm were tested, in compliance with ASTM C31, for each trial. Finally, mix design for the compressive strength of 50 MPa was obtained. Mix Design for 50 MPa is shown in the Table 2.

Table 2: Mix Design for concrete

Concrete mix design for compressive strength of 50 MPa	
Concrete Mix Design	1:1.5:1.5
Coarse Aggregate	Maximum particle size is 12mm
Fine Aggregate	Nizampur sand
W/C ratio used	0.35
Superplasticizer Viscocrete 3110(1st generation)	1% by weight of Cement
Silica Fume	10% by weight of cement
Chemicals Brought from	Sika Chemicals Rawalpindi

Six beams were provided with minimum longitudinal steel of 15.7 mm^2, according to the ACI requirements. Five 6 mm bars of 276 MPa were provided to each sample with three in tension zone and two in compression zone of concrete. The tensile properties of steel bars were obtained using testing standards ACI 440.3R(2004).

3.2 Materials

The two main components of beams used in research work are concrete and steel. The materials for concrete are coarse aggregate, fine aggregate, Cement, superplasticizer and silica fume. The coarse aggregate having maximum particle size of 12.5 mm was brought from Margalla Crush Plant. Fine aggregate was coarse sand brought from Nizampur. In fine aggregate and in coarse aggregate, the maximum sizes are 2 mm and 12 mm respectively. Cement used was manufactured by Bestway Cement Pvt Limited. Cement was ordinary Portland cement of grade 53 conforming to the Pakistan Standards PS-232-2008. To gain high strength the superplasticizer of first generation (Viscocrete 3110) and Silica Fume was used. These products were bought from Sika Chemicals Rawalpindi.

The steel sheet used for fabrication of steel tubes was bought from Lal Steel City, Saddar Road Rawalpindi. The two sections were welded together at Lal Kurthi Rawalpindi. The framework for experiment was made from a girder bought from Rawalpindi. The girder is welded with a collar above it to hold the load transferring shaft of Jack.

3.3. Description of Specimens

Testing program consists of testing twelve specimens, six for monotonic loading and six cyclic as shown in Table 3. Nomenclature adopted is, Ref is symbol for fully filled CFDST beam which is reference beam for other CFDST beams. First alphabet B stands for beam, second numerical value categorizes beam and third value gives the eccentricity of inner steel tube. In second set of three Beams, the alphabet R shows that the sample is reinforced with minimum longitudinal steel. Width of all inner and outer steel square tubes is 152.5 mm and 76.25 mm respectively.

Table 3: Description of specimens

Specimens	Outer tube Diameter, di(mm)	Inner tube diameter, d_o(mm)	Concrete infill in the inner tube	No of Beams	Eccentricity (mm)	Reinforcement
Ref	152.5	76.25	Yes	2	0	No
B1-0	152.5	76.25	No	2	0	No
B2-12.5	152.5	76.25	No	2	12.5	No
Ref-R	152.5	76.25	Yes	2	0	Yes
B1-0R	152.5	76.25	No	2	0	Yes
B2-12.5R	152.5	76.25	No	2	12.5	Yes

3.4. Test Setup and Procedures

Test setup was same for all twelve specimens. Four-point loading test was used for all the samples, in compliance to ASTM D790-17 [36]. Beams are tested as simply supported with one end pin and the other end as roller support, whereas Load P is applied at two loading points (Figure 10) Instruments used were three LVDTs and two deflection dial gauges. Three LVDTs were fixed at sections 2,3 and 4, as shown in Figure 10. To record deflections at section 1 and 5 deflection dial gauges were attached (Figure 10). To record data, NI 9363 Module was used. To measure a slip value at end Vernier caliper was used. The test setup was same for all twelve specimens. Four-point loading test was used for all the samples compliance to ASTM D790-17 [36]. The LVDTs can measure the displacement up to 100 mm. The deflection dial gauge can measure the deflection up to 50 mm.

The hydraulic system used in the reaction floor has capacity of 400 KN. The jack can apply both cyclic and monotonic loadings. The jack was supported by a reaction floor made from two C sections welded together back to back. To record data, NI digital data acquisition system was used. The system has three slots for LVDTs, that’s why 3 LVDTs can be connected at same time. The jack is operated by Laptop, having Lab view software. The software controls the movement of fluids with pressure and thereby movement of jack is controlled. To adjust the jack in order to save time the test setup is changed to manual in Lab view software which lead to the quick movement of jack. To adjust the optimum height of the jack supports are lifted upward and a cylinder is provided below the jack. The cylinder has diameter of 100 mm and length of 380 mm. Supports were lifted upward by using steel plates of 380 mm diameter and height of 50 mm. Three plates are provided over each bench of height 760 mm. In order to provide the simply supported behavior, one support is arranged as roller and the other is arranged as pin (Figure 10). Hinge support is provided by placing a concrete filled steel block over the steel plates. Pin support is formed by placing a cylinder horizontally over the frame. To measure a slip value at end Vernier caliper was used.

The specimens were very heavy, in order to lift the specimens, a manually controlled crane was used. First a steel reinforced plastic rope was tied over beam, then the rope was hanged in the hook of a crane. The crane was also supported by reaction floor.

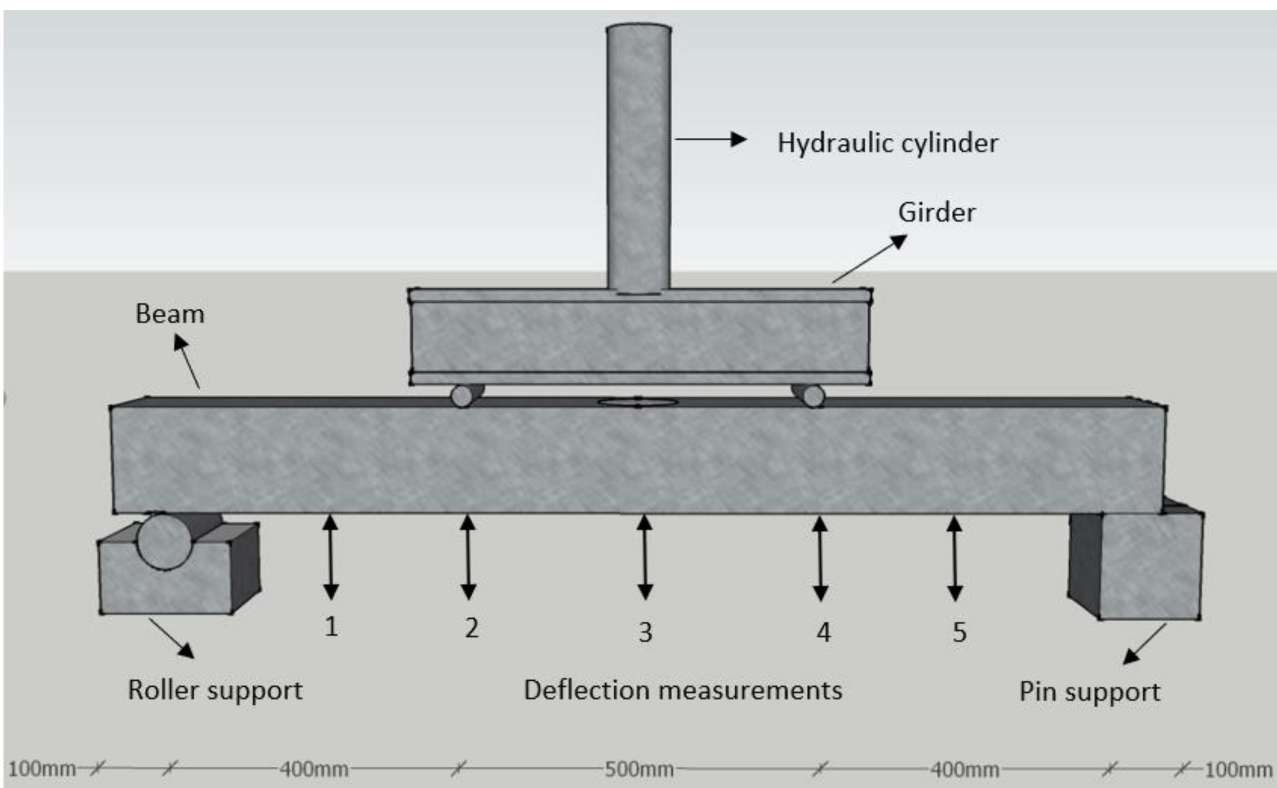


Figure 10: Test setup

3.5 Preparation of Specimens

Steel samples were fabricated in two steps. First, a sheet of 2 mm thickness was bent to make two L-shaped sections. Then, the two L-shaped sections were welded together to make a square section. This process was repeated both for inner and outer tubes. The inner tube is welded in larger tube at required eccentricity through a steel strip as shown in Figure 11. Width and thickness of steel strip was 12.5 mm and 3.125 mm.

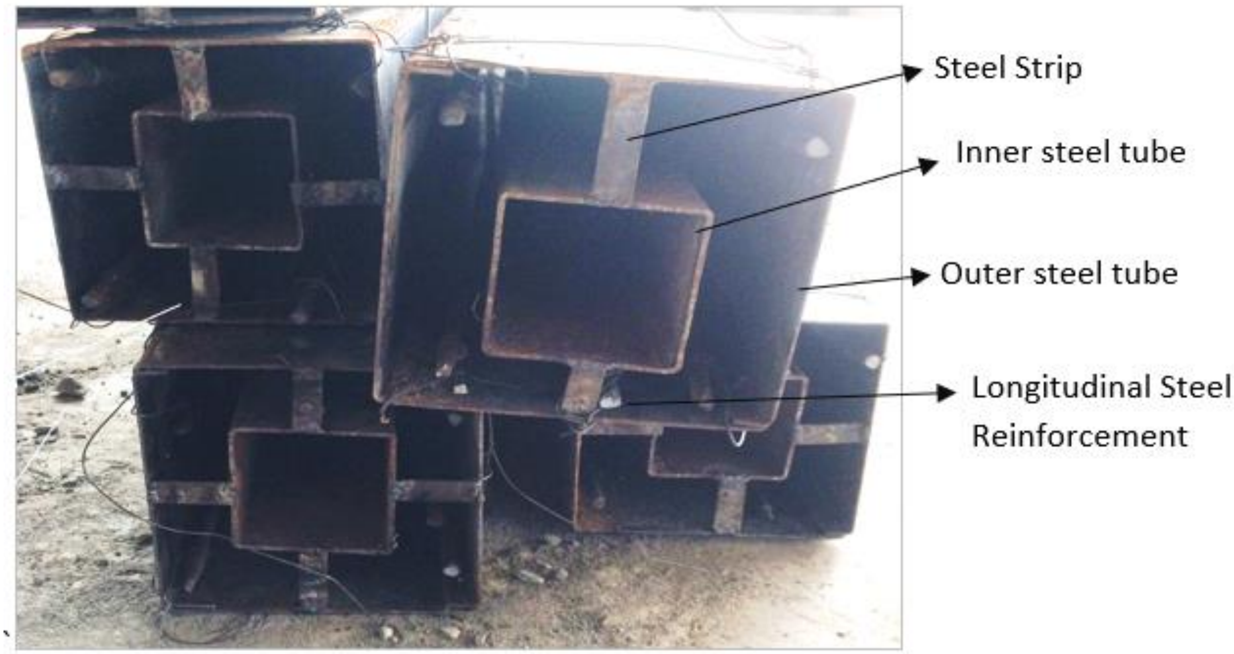


Figure 11: Fabrication of specimen

After fabrication, there is casting stage, in which tubes are filled with concrete by standing them vertically. Compaction is done with the help of long steel rod. Binding of reinforcement in the reinforced beam is done, with the help of binding wire to maintain proper gape for bonding. A beam prepared is shown in the Figure 12.

Figure 12: Casting of beams

3.6 Four Point Loading Tests

The major difference between the three and four-point bending tests is the distribution of the bending moments. Four-point loading method is used for high modulus composites. The four-point loading give uniform stress distribution between the loading points, as shown in the Figure 13. The three-point bending test gives stress distributions below the central point where the load is applied. In our case, a steel girder was used to distribute the load at two points from girder to the supports. To record the deflection at points of loading, at mid span and the midspan between the point of load applied and support LVDTs and Deflection Dial Gauges were used.

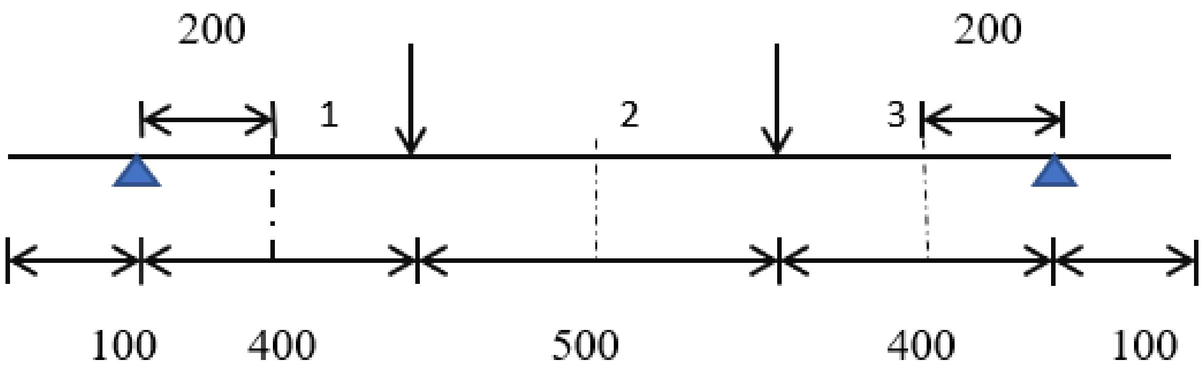


Figure 13:Test Setup

3.7. Loading Protocols for Cyclic and Monotonic Tests

Two types of loading can be applied in testing members, either monotonic or cyclic. Both tests are conducted to evaluate the mechanical properties of metals, many other materials such as composites or materials made from combination of these materials. The examples of monotonic tests are tension and compression tests. In this test load is applied at a constant increase to the specimen to obtain its properties such as yield strength and ultimate strength etc. Six samples were casted for monotonic loading, three with reinforcement and three without reinforcement. Test was conducted at a slow rate of loading, the rate of loading used was 29.42 KN/min.

Cyclic load testing involves the testing procedures in which constantly changing load is applied, the load may be in tension or compression or combination of two. The second series of testing consist of six specimens, three with longitudinal reinforcement and three without longitudinal reinforcement. The details of loadings are provided in the table. The cyclic loads increments were decided according to ASTM E-529-04 for six specimens of 152.5*152.5 mm cross section and length of 1500 mm. The arrangement of beams was same as that for monotonic loading. The loading increments were 9.81 KN. Each increment is applied in two cycles i.e. first cycle is applied as 0,9.81KN,0,9.81KN,0. Graph for first four cycles is given in the Figure 14. The first cycle is called as primary cycle for each pair of cycle. Backbone Curve is the Curve drawn between load and displacement of the primary cycle of each increment [37].

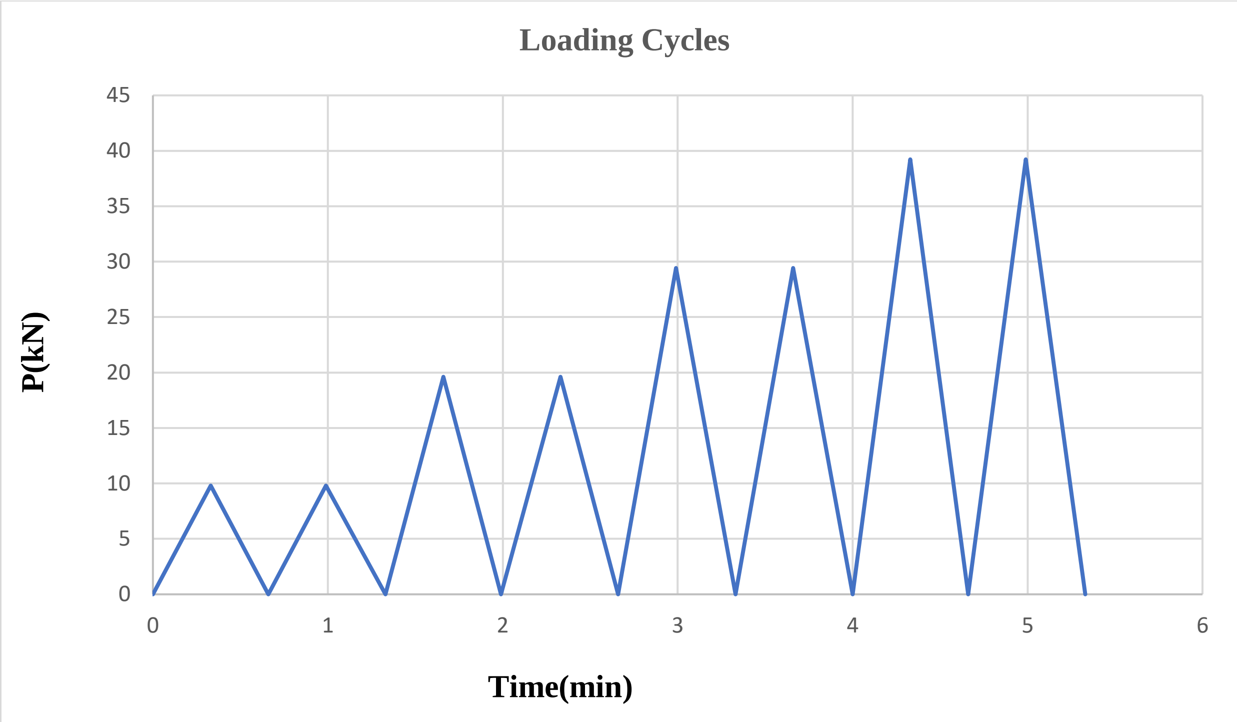


Figure 14:Graph for Loading of first four cycles

3.8 Parameters Studied in Experiments

The main parameters studied in experiment are the effect of hollowness, eccentricity of inner steel tube and effect of the longitudinal reinforcement on the flexural behavior of double skin steel tubular members. The slip is controlled at the edges of the steel tubes by welding a strip of steel. When the tube is fully filled with concrete, it means that it has more concrete, which may therefore provide increase in ultimate capacity and Energy dissipation capacity. It also decreases ultimate deformation. When inner tube is moved towards tension side, more concrete is available in the compression zone of concrete which increases the strength [16]. The presence of longitudinal reinforcement in double skin tubular members also changes the response [16]. The above parameters will be studied in terms of the following factors.

3.8.1 Load-Deflection relationship

When we apply load on beam, it will deflect, increasing load will increase deformation. To study the response, three points are taken into consideration i.e. yield point, Peak point and ultimate load. The yield displacement of an elastoplastic system is calculated by taking reduced stiffness which can be found by taking secant stiffness, taken at 75 percent of the ultimate load. The method used to calculate yield point is shown in Figure 15.

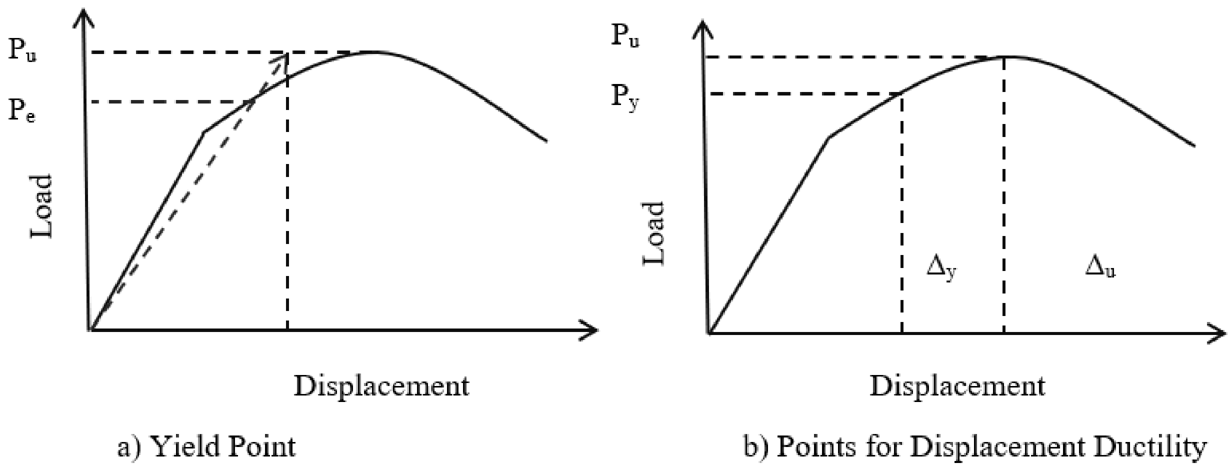


Figure 15: Determination of yield point and ductility

3.8.2 Displacement Ductility and Response factor

The ability of a structure to deflect beyond yield point, without significant loss in strength is called ductility. The method used for calculation of ductility is from displacements, therefore it is called displacement ductility. Displacement ductility is ultimate deformation divided by yield deformation, given by equation 1.

$$\mu = \Delta_{u/} \Delta \tag{1}$$

Response factor is an important factor of seismic design. Here we will determine the response factor using Paulayand Priestley (1992) which is given by equation (2).

$$Rf = \sqrt{\ } (2\mu - 1) \tag{2}$$

Where μ is the displacement ductility of the specimen.

3.8.3 Stiffness degradation

To define the degradation in stiffness three points considered are yield, peak and ultimate points. The force per unit deformation at that points is called stiffness at that points.

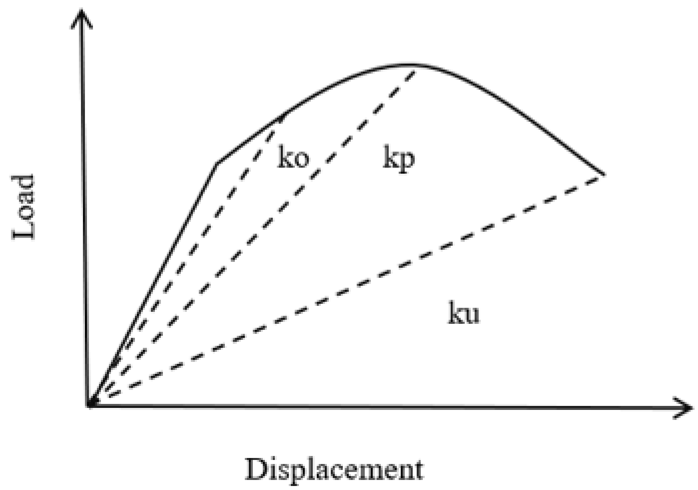


Figure 16: Determination of stiffness at different points

Stiffness degradation ratio Ck is the ratio of secant modulus K at specified displacement to the secant modulus Ko at yield which can be seen in equation 3.

$$Ck = K / Ko \quad (3).$$

Here we will use peak of load displacement curve stage. Greater the value of stiffness degradation ratio means that degradation in stiffness is less. Highest value of Ck is 100 percent.

CHAPTER 4: RESULTS AND DISCUSSION

4.1 Load-Deflection Relationship

Out of first three beams the Ref has highest value of Peak Load $P_{o,}$ yield point P_y and ultimate load P_u showing Least value of deflection. This is due to availabity of more concrete at center of inner tube. Sample B2-12.5 shows better response as compared to B1-0 because more concrete is available in the compression zone. In next three beams specimen Ref-B has highest value of load at Peak, Yield and Ultimate points, while showing Least value of deflection as shown in Figure 17. Specimens without reinforcement have shown bilinear response. They showed a large deflection at mid-span exceeding 1/20 span at less than 3 percent reduction of Peak load P_o. The specimens were able to take more load after a sudden drop, but the test was stopped at 20 percent reduction in Peak load Po due to limitation of slippage at supports. The specimens showed a small decrease in flexural strength with large deformation.

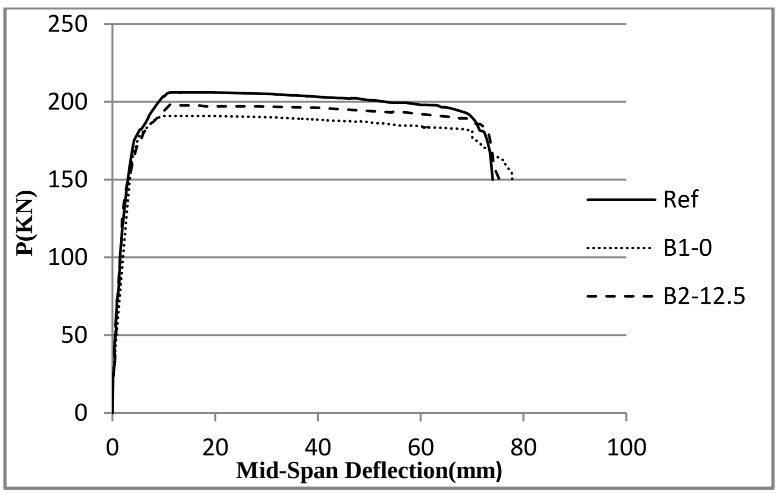


Figure 17:Mid-span deflection of beams without reinforcement

The sample B1-12.5R has better response as compared to B1-0R. Providing minimum reinforcement in CFDST beams has imparted a new behavior to these members. Reinforced samples have greater value of laod at Peak, Ultimate and Yield but have shown greater deflection at these points as compared to the unreinforced samples. The reinforced samples has shown increased ultimate deflection due to rupture of steel bars followed by rupture of steel sheet at or just after ultimate load. Specimens with reinforcement shows a linear response at start and then strength is decreased after ultimate load at a greater rate (Figure 18). Although, these members have large ultimate load carrying capacity but decrease in the second part of force-deformation curve is due to rupture of longitudinal reinforcement followed by rupture of steel tube in the tension zone as shown in Figure 7. Flexural strength of CFDST beams was found to be greater than hybrid-CFDST beams. When strengths are compared, strength of CFDST is found to be 4-5 times the strength of hybrid-CFDST beams. The higher strength is due to greater bending resistance provided by the square section and isotropic properties of steel sheet. The ultimate deflection is also reduce to half in case of CFDST beams [16].

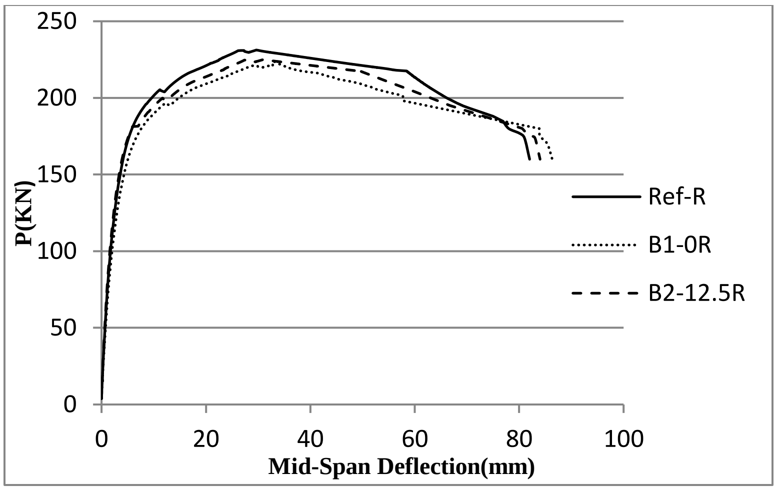


Figure 18: Mid-span deflection of beams with reinforcement

Table 4: Details of monotonic and cyclic loading beams

Monotonic loading									
Specimen	**Deflection at yield point Δ_y (mm)**	**Deflection at peak point Δ_p (mm)**	**Ductility μ**	**Energy (KJ)**	**Stiffness values**			**C_k**	**R_f**
					K_o	**K_p**	**K_u**		
Ref	3.200	9.451	2.953	**14.230**	48.30	21.89	2.24	**45.30**	2.21
B1-0	3.240	10.600	**3.437**	13.509	44.15	17.99	1.96	40.76	**2.42**
B2-12.5	3.210	10.256	3.195	13.807	46.33	19.36	2.13	41.8	2.32
Ref-R	5.100	29.647	5.813	**16.214**	33.86	7.83	2.39	**23.12**	3.26
B1-0R	5.210	32.419	**6.222**	15.267	31.86	6.83	2.09	21.40	**3.38**
B2-12.5R	5.151	31.004	6.020	15.903	32.77	7.26	2.23	22.20	3.33
Cyclic loading									
Specimens	**Deflection at yield point Δ_y (mm)**	**Deflection at peak point Δ_p (mm)**	**Ductility μ**	**Energy (KJ)**	**Stiffness values**			**C_k**	**R_f**
					K_o	**K_p**	**K_u**		
Ref	2.90	10.306	3.554	**13.064**	51.69	19.40	2.20	**42.00**	2.47
B1-0	3.01	11.000	**3.654**	12.512	46.19	16.85	2.03	36.48	**2.51**
B2-12.5	2.95	10.509	3.562	12.900	49.39	18.58	2.14	37.62	2.47
Ref-R	4.79	29.110	6.077	**15.336**	34.85	7.65	2.29	**22,00**	3.34
B1-0R	5.20	32.610	**6.271**	14.964	29.70	6.31	2.01	21.20	**3.40**
B2-12.5R	5.01	30.560	6.099	15.100	32.30	7.06	2.13	21.90	3.35

Deflection at peak point Δ_p (mm**)** is deflection calculated at peak point of loading. Displacement ductility **(**μ) is calculated by taking the ratio of deformation at Peak (Δ_p) to deformation at yield (Δ_y). Energy dissipation is computed by calculating the area under force deformation curve in monotonic loading. Total energy dissipated in cyclic loading is computed by adding area of each cycle. **K_o, K_p, K_u** are stiffnesses at yield, peak and ultimate points of load-deformation curve. Stiffness degradation ratio (C_k) is the ratio of stiffness at peak point to stiffness at yield point. It is expressed in percentage.

Backbone Curve (BBC) is a useful tool for comparing cyclic loading and monotonic loading responses. BBC is drawn between load and displacement of the primary cycle of each increment of loadings. Primary cycle is the first cycle of each cyclic loading increment. Firstly, peak values of each primary cycle of force-deformation graph of beams under cyclic loading are plotted. The Backbone curve is then plotted by joining the 0 and Peaks of all primary cycles from force-deformation relationship, in cyclic loading of beams. Comparing monotonic curve of Figure 17 and Figure 18 with cyclic loading back bone curve in Figure 19 shows that cyclic response is less or equal to monotonic response. The responses seem to be equal up to the yield point. After yield point the monotonic loading response is greater because stiffness of cyclic loading samples is degraded by the preceding cycles.

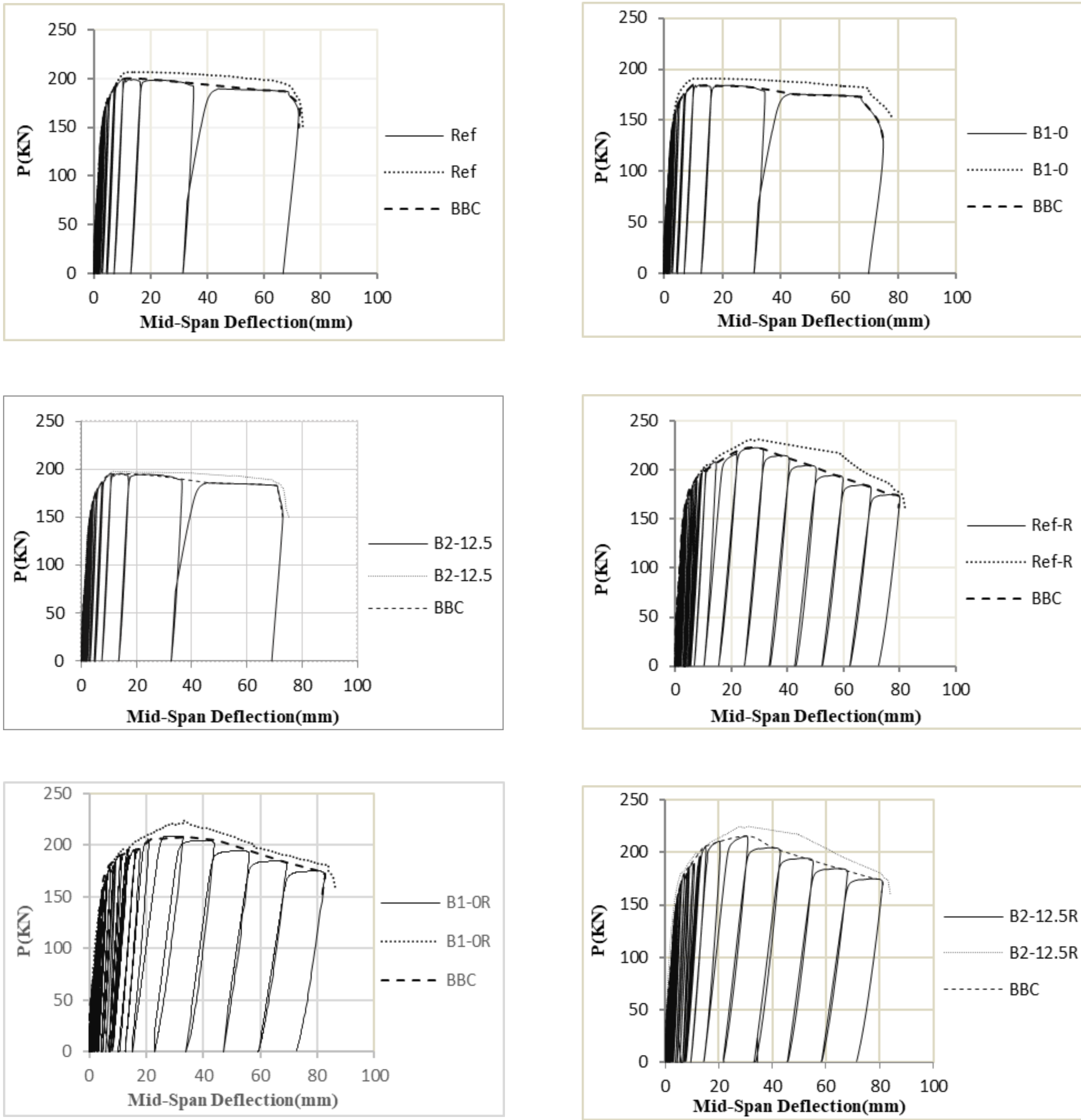


Figure 19:Monotonic Load Curve, Cyclic load Curves and Corresponding Back Bone Curve

4.2 Failure of Specimens

Figure 20 shows some specimens at the end of testing. Loading was stopped after decreasing of load to 20 percent of the Peak load. In monotonic loading, first three samples without longitudinal steel reinforcement failed due to buckling of steel tube in compression zone and based on excessive deflection of the beam we can assume that concrete in the steel tube is also cracked Figure 20. Buckling of steel tubes at point of loading can be seen in the monotonic loading samples. Next three beams having longitudinal reinforcement, failure occurs due cracking of concrete, buckling at the top and rupture of steel tube at bottom as shown in Figure 20. Rupture and buckle of outer steel tube is at point of loadings in all cases. The specimens having reinforcement showed delayed rupture.

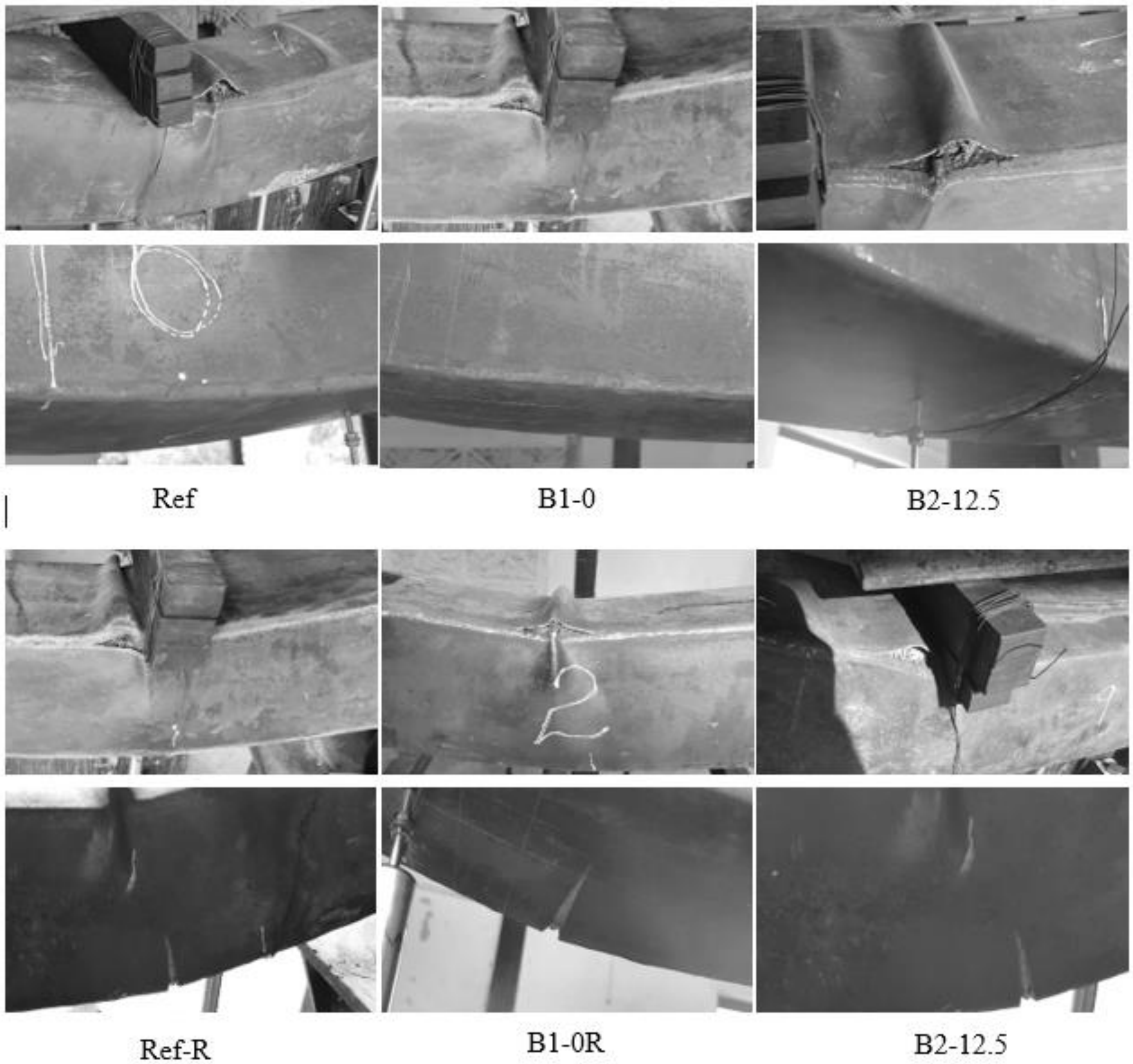


Figure 20: Monotonic Loading Specimens after testing

If we analyze samples subjected to cyclic loading after testing, it can be seen that all the specimens are failed due to buckling of steel tube and cracking of the concrete between the two tubes. None of the sample has failed due to the rupture of steel tube Figure 21.

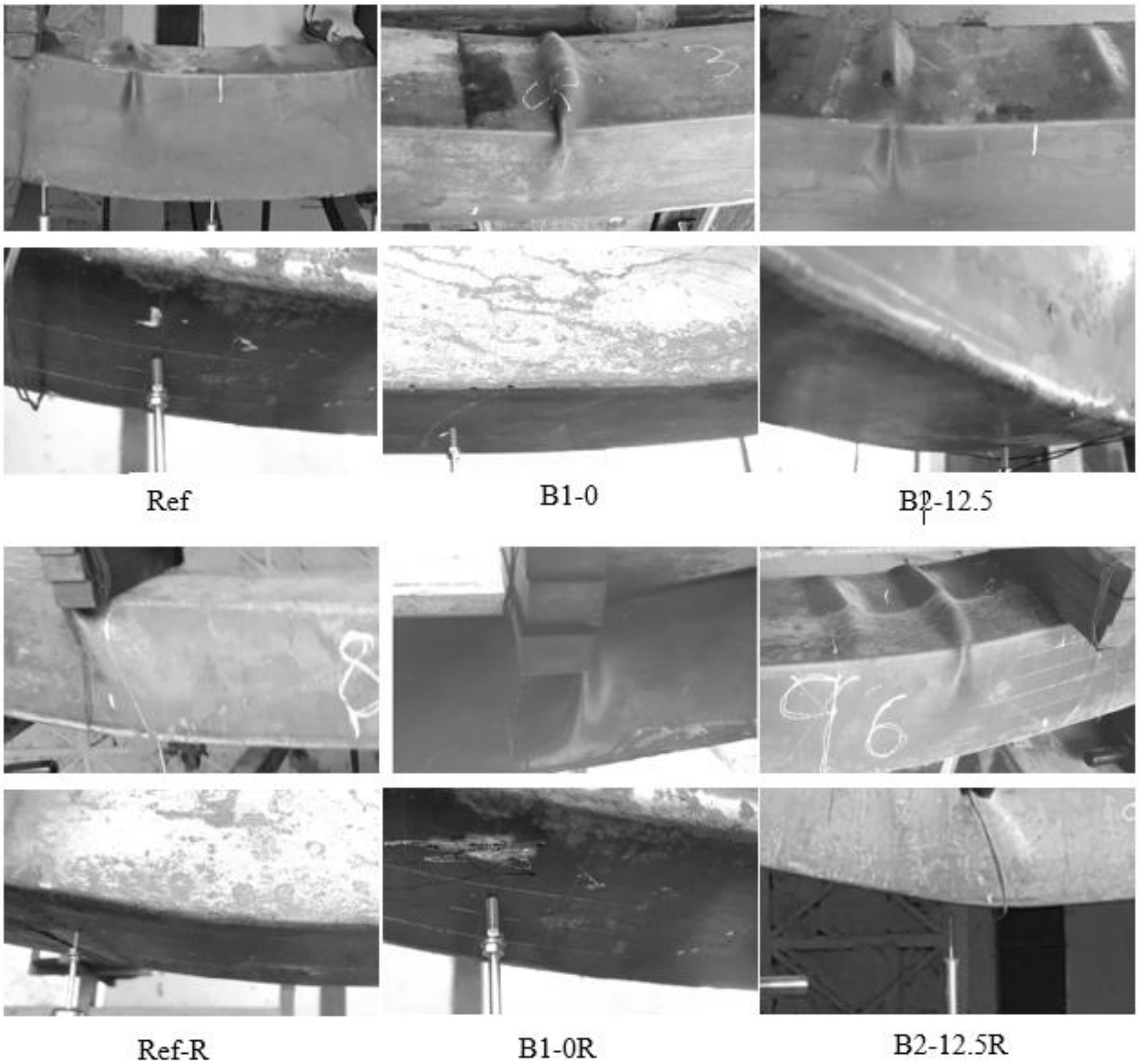


Figure 21: Cyclic loading specimens after testing

4.3 Hysteresis Behavior

Figure 19 shows that the monotonic loading curve and hysteresis loops with corresponding backbone curve for cyclic loading. Loops are well fanned out for unreinforced beams. At elastic branch of force deformation curve, cycles are small showing deflection of few millimeters. The cycles become wider showing large permanent deformations in inelastic branch. Permanent deformations are due to cracking of concrete and weakening of the bond between concrete and steel sheet. Test was stopped after two cycles when the load is reduced to the 20 percent of the Peak value and followed by reducing it to zero value. In reinforced beams loops are well fanned and have shown relatively low permanent deformation at yield point. In these samples number of loops after yield point are in greater number as compared to unreinforced beams i.e. it can resist more loading cycles. Longitudinal reinforcement helped to maintain the bond between concrete segments. The load reduction in these samples is slow, stepwise degradation of strength takes place in case of cyclic loading.

4.4 Energy Dissipation

Energy dissipated by monotonic loading beams is the area under load deflection curve. From Figure 22, it can be seen that energy absorbed by Ref beam is greater than B1-0 and B2-12.5. The energy dissipation is greater due to the availability of more concrete. More concrete in compression zone increase the energy dissipation of B2-12.5. On the other side, the overall energy of reinforced beams is greater than unreinforced beams due to presence of longitudinal steel reinforcement. In reinforced samples energy is dissipated in cracking concrete, breaking of bond between two steel and concrete, damaging of steel sheet and steel bars. In these members Ref-R has greatest energy dissipation capacity due to the presence of more concrete to absorb energy (Figure 22). B2-12.5R has more concrete in compression zone, thereby it has absorbed more energy as compared to B1-0R.

Out of unreinforced samples, Ref has greatest energy absorbing capacity because there is more concrete available to absorb energy. However, sample B2-12.5 has value of energy dissipation capacity closer to Ref. Overall energy absorption capacity of reinforced beams is greater than unreinforced beams. The sample having eccentric steel tube have energy dissipation capacity in between the other two samples (Figure 22). Comparing energy dissipation capacity of monotonic loading sample is greater than cyclic loading samples because monotonic loading samples have greater force deformation response. Stiffness of cyclic loading samples is degraded by the preceding cycles which reduces the force deformation response of specimens.

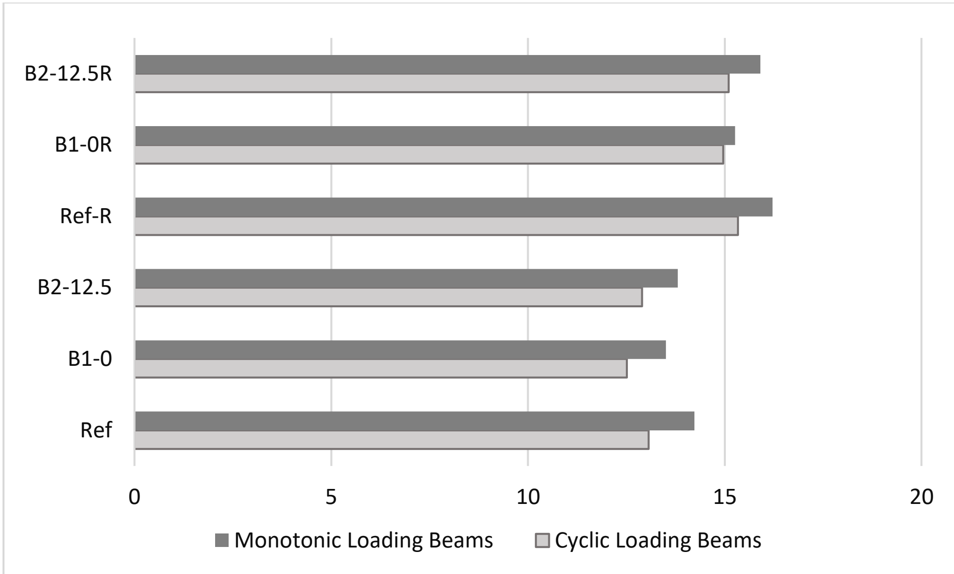


Figure 22:Energy Dissipation in Monotonic and cyclic loading (KJ)

4.5 Ductility

Displacement ductility μ studied is the ratio of displacement at peak Δ_o to the displacement at yield $\Delta_{y\}}$ [38]. In Figure 23, in unreinforced beams, the deflection at the peak of B1-0 is greatest which resulted in the higher value of ductility. Beams having longitudinal reinforcement gave an almost double value of displacement ductility. B1-0 shows highest ductility.

The ductility of Ref beam in cyclic loading is less than other two because it deforms less at yield point. Same trend is followed in backbone curve of next three beams. Ductility of reinforced sample is greater than unreinforced samples. Which means that this type of members shows more deflection at ultimate load.

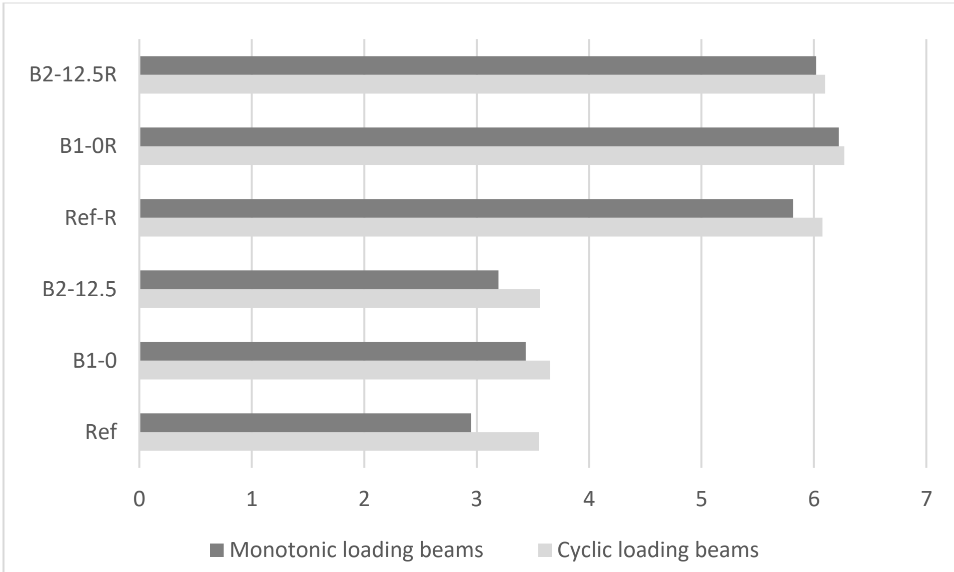


Figure 23:Displacement Ductility for monotonic and cyclic loading(μ)

4.6 Response Factor Rf

Response factor is an important factor of seismic design. Here we will determine the response factor Rf for Displacement ductility μ using Paulayand Priestley (1992) which is given by equation 2.

Greater value of ductility leads to greater response factor. In unreinforced samples the deflection at peak of B1-0 is greatest which resulted in the higher values of response factor. B1-0R shows highest ductility and response factor. Figure 24 gives the response factor of beams subjected to monotonic loading. Response factor of Ref beam is less than other two because it deforms less at yield point as compared to the other two samples. Same trend is followed in the next three samples. Response factor of reinforced sample is greater than the unreinforced samples. Which means that this type of members provides greater displacement at ultimate load.

Figure 24 also gives the response factor for the BBC for beams subjected to cyclic loading. The ductility and response factor of Ref beam is less than the other two because it deforms less at yield point as compared to the other two beams. Same trend is followed in backbone curve of next three samples. For reinforced beam, the response factor is greater than unreinforced samples. These members show more deflection at ultimate load.

If we compare the response factor for both monotonic and cyclic loading beams, the response factor for monotonic loading is less than cyclic loading. Reduction in response factor is due to the degradation in stiffness of beams due to cyclic loading.

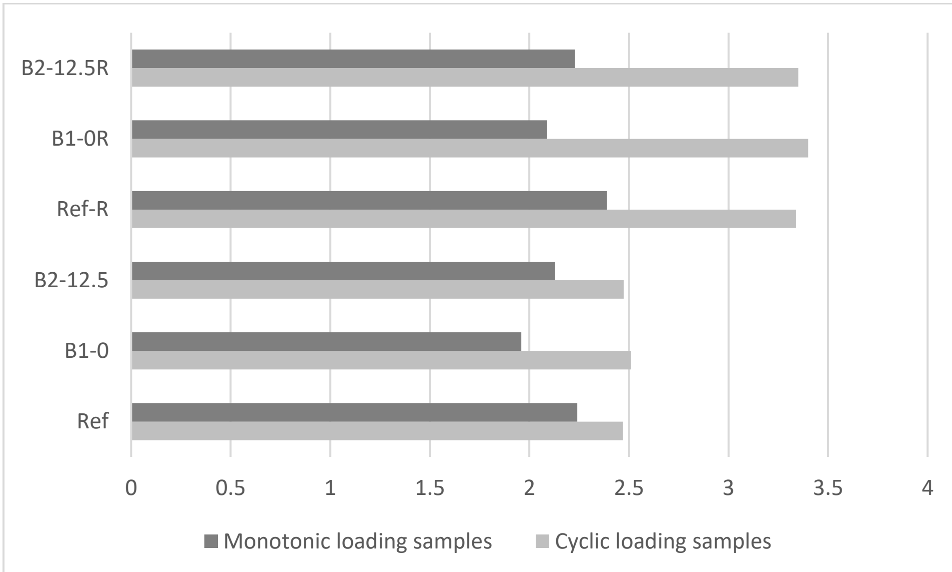


Figure 24:Response factor R_f for monotonic and cyclic loading(μ)

4.7 Stiffness degradation

Stiffness degradation ratio C_k is the ratio of secant modulus K at specified displacement to the secant modulus K_o at yield. Here we will use peak displacement of load-displacement curve as our specified displacement as shown in equation 3.

In Figure 25, it can be seen that there is little difference between stiffnesses of first three beams, but the stiffness degradation is less in first beam because the value of stiffness at peak is greatest in the beams with reinforcement. The average stiffness degradation in first three samples is about 22 percent and that for next three samples is 14 percent.

Out of first three samples Ref beam has greatest stiffness degradation ratio. Which means that the rate of strength degradation from yield to peak is slow in these types of members. If we see the reinforced samples the Ref-R beam has the highest value of C_k. If we compare two sets of samples, it can be seen that rate of stiffness degradation ratio is decreased by adding longitudinal reinforcement i.e. stiffness is greatly degraded in these members.

When we analyze force deformation backbone curve for the cyclic loading samples, the stiffness degradation rate is relatively faster (Figure 25). Stiffness degradation is due to damage of concrete, yielding of steel sheets and longitudinal steel bars up to the ultimate point. Ref beam has the largest value of stiffness degradation ratio C_k. The degradation in stiffness is due to cracking of concrete in the steel tube. Sample B1-0 has less degradation as compared to the B2-12.5 due to the unsymmetrical arrangement of concrete in compression zone. If we compare the above beams with reinforced beams, we came to know that the reinforced samples have smaller values of stiffness degradation. Which means that the stiffness at yield point is less as compared to the unreinforced beams. In cyclic loading the stiffness degradation is more as compared to monotonic loading. The stiffness degraded by incrementally increasing cyclic loads.

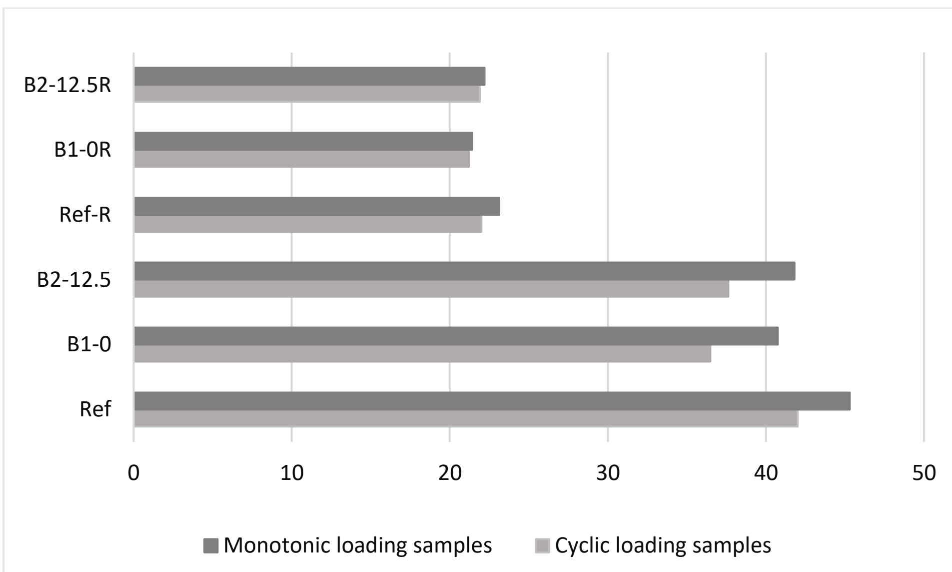


Figure 25: Stiffness degradation ratio (C_k)

4.8 Profile of Beams

In order to show the deflection pattern of beams, profile of all the beams is plotted with increment of 10 KN. In monotonic loading of beams, the beams without reinforcement has shown a drastic decrease in load carrying capacity of beams, after peak loading point, as shown in Figure 26. The sudden decrease in capacity is due to the shear failure of beam. While the reinforced beams have shown uniform increase in deformation with the increase of load after Peak point load, in monotonic loading. This type of failure is desirable in the design of structural members.

In cyclic loading of beams, the profile shows that the reinforced beams have shown stepwise degradation of strength, as compared to the beams having no longitudinal reinforcement which can be seen in Figure 27. Moreover, the reinforced beams can resist more loading cycles as compared to the unreinforced beams in the monotonic loading of beams.

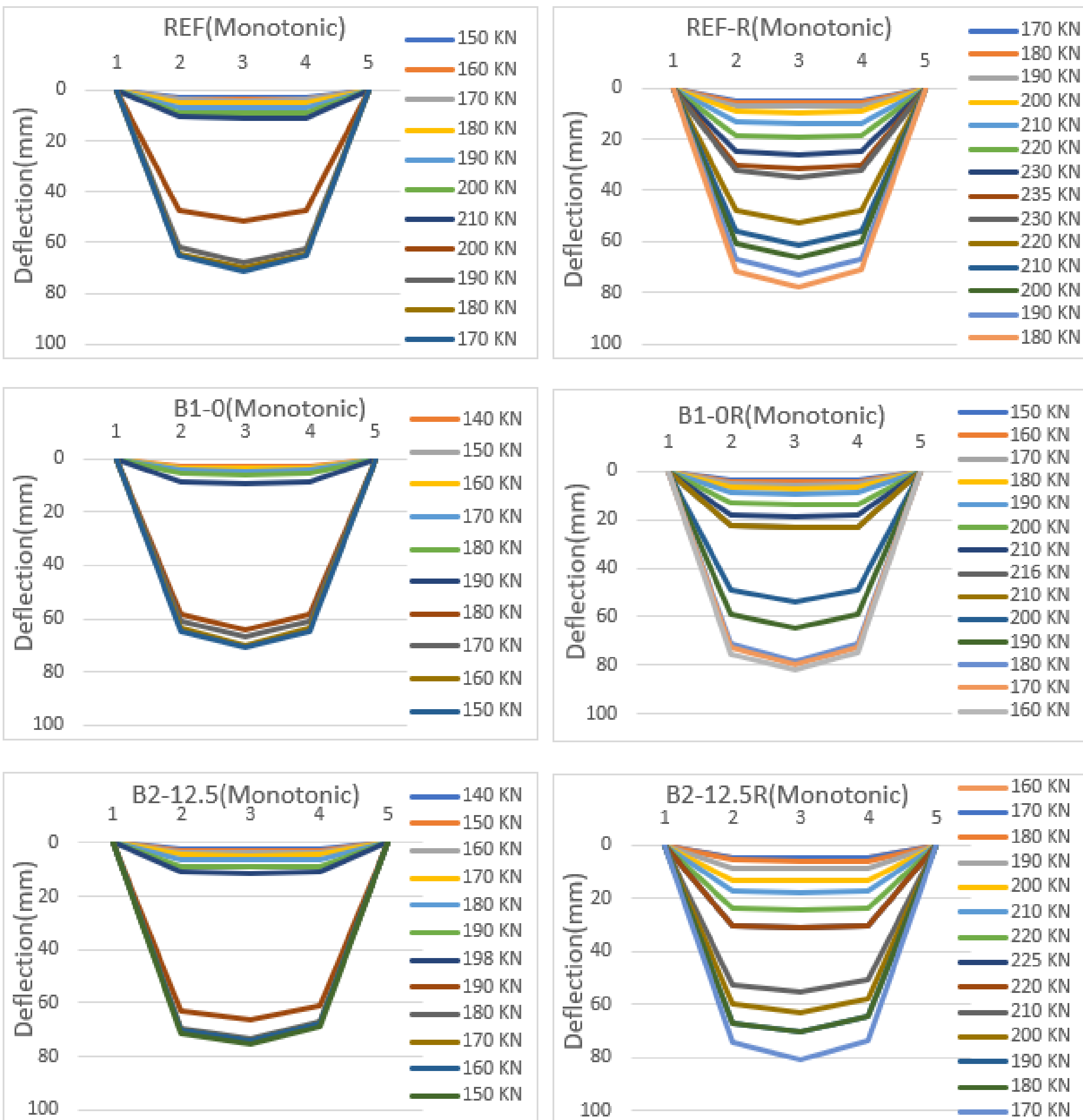


Figure 26 : Profile of beams subjected to monotonic loading

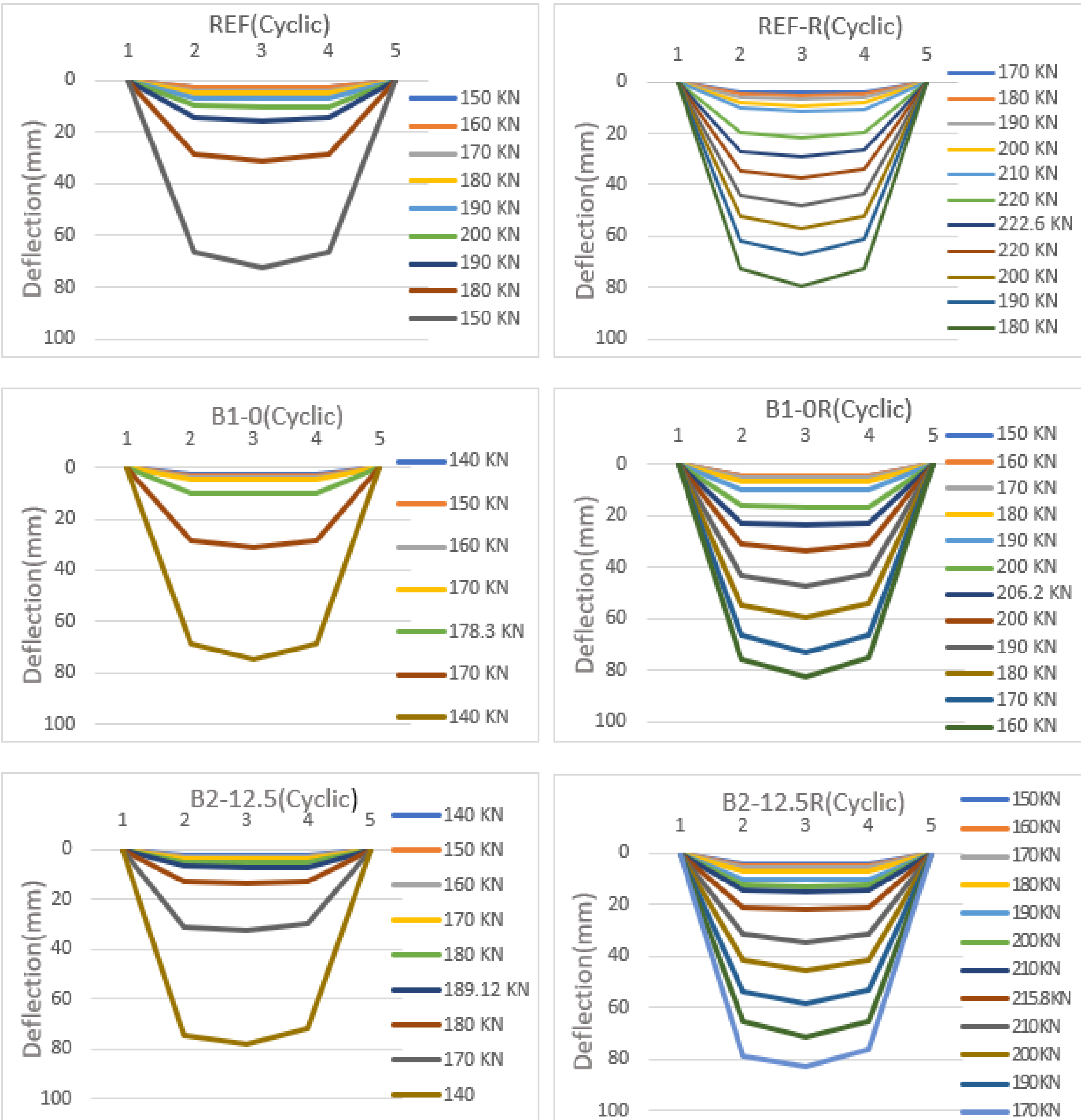
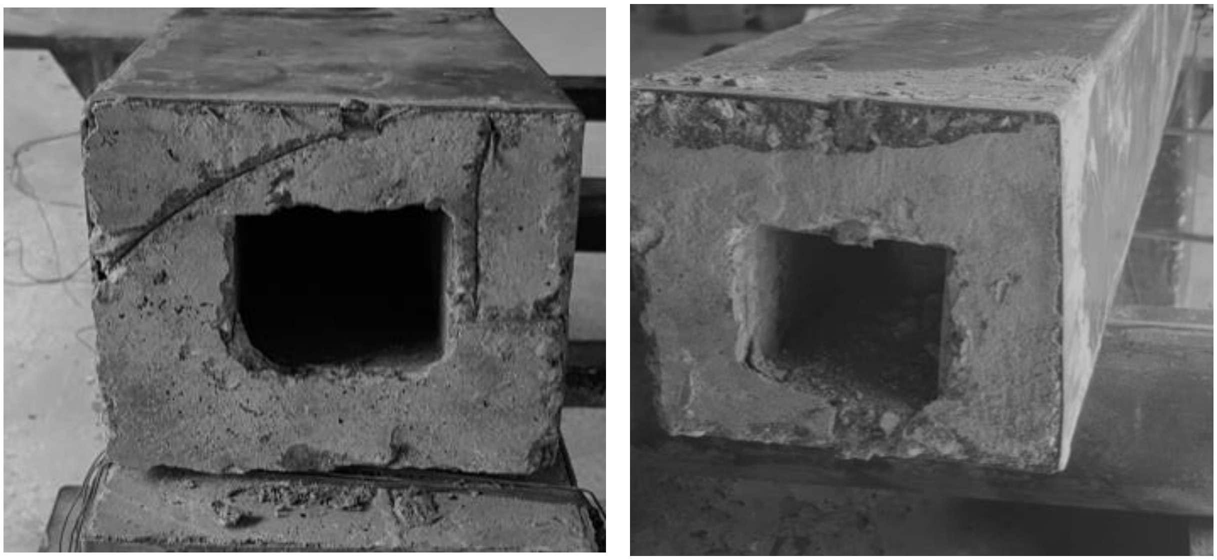


Figure 27 : Profile of beams subjected to cyclic loading

4.9 Behavior at the interface of steel and concrete

Main problem in using double skin tubular members is the slip between concrete and other material such as FRP or Steel Tube. In all the twelve beams slip was partially reduced by the friction between steel and concrete, remaining slip is prevented by small rectangular steel strip shown in the Figure 11, at both ends of the tube. The strips also helped to maintain the inner tube at the desired position. After observing all the samples, we came to know that although concrete is cracked but still it did not show any slip as shown in the Figure 28.



a) B1-0

b) B2-12.5

Figure 28:Beams showing cracking of concrete with no slip

4.10 Comparison of monotonic and cyclic loading

The application of cycles to the samples degrades their stiffness which led to the increase in deformation and decrease in strength. The cracking of concrete at the ends was observed at all the twelve specimens but there was no slip between steel and concrete.

If we see the failure mechanism for six specimens for monotonic loading, first three fails by buckling of outer steel tube and second set of reinforced sample fails both by rupture and buckling of steel tube. If we analyze 6 samples for cyclic loading, neither of them failed by rupture of steel tube. They are failed by cracking of concrete and buckling of steel tube.

Figure 29 shows the graphs for each set of samples. The dotted line at the top shows the sample subjected to monotonic loading. The line touching the cycles at the top is back bone curve for the cyclic loading. The backbone curve is draw by joining the peaks of the primary cycles. The monotonic response for all the samples is equal or greater than the cyclic load response. For fix value of load the displacement or deflection for the cyclic load is greater. The reason for the increase in deflection is the degradation in stiffness by the preceding cycles.

The energy absorbed and the strength during monotonic loading is more than that of cyclic loading from Table. The difference is different for all the samples. The stiffness degradation ratio in first three samples is greater for monotonic as compared to the first three samples for cyclic loading. Which means that stiffness has been degraded to a greater degree in first three samples. The second group of beams both for monotonic and cyclic loading has approximately same stiffness degradation ratio. The stiffness degradation response is greatly improved for the reinforced samples in both monotonic and cyclic loading.

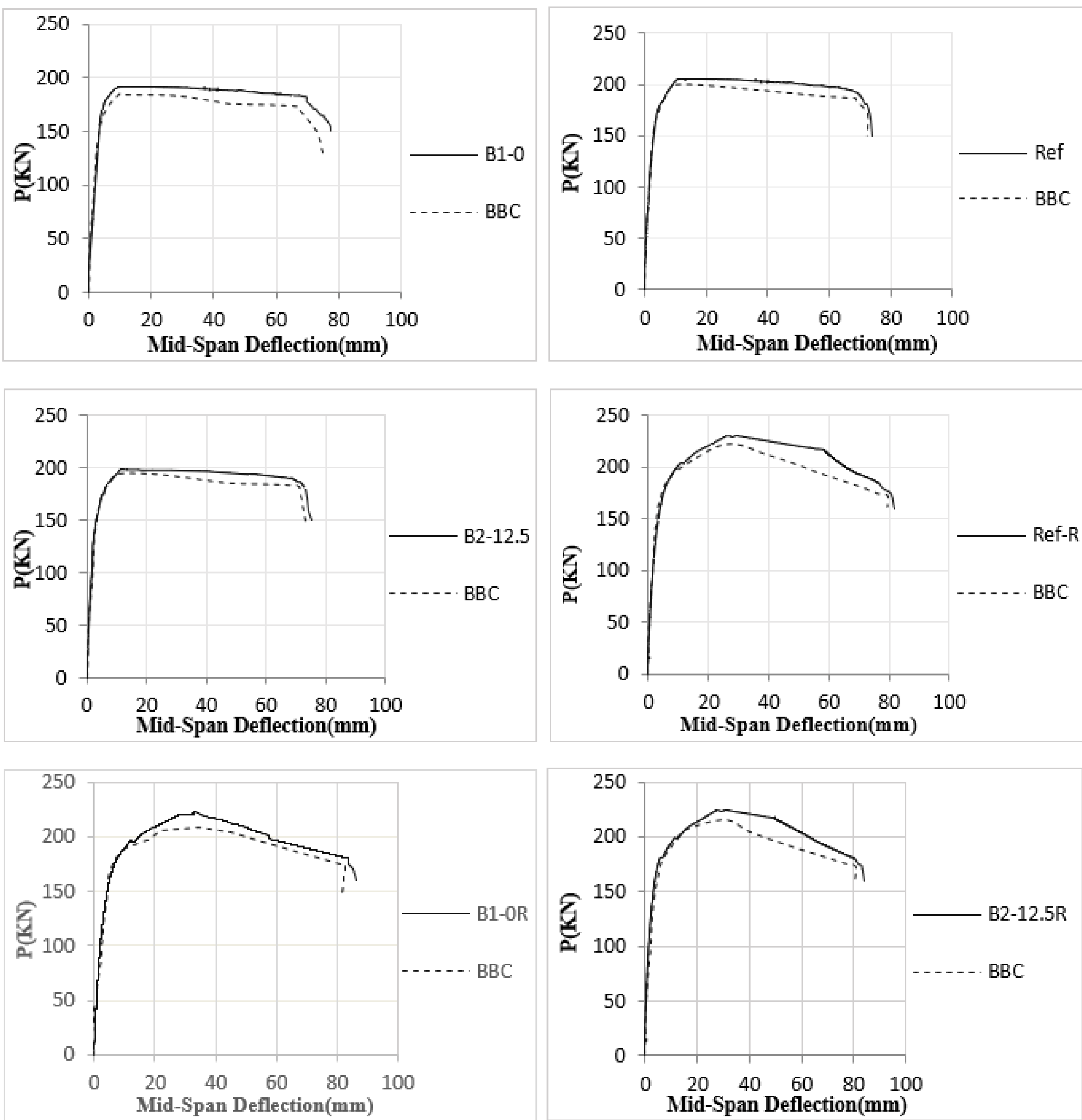


Figure 29 : Comparison of monotonic and cyclic loadings

The response factor is greater for cyclic loading samples. The application of cycles degraded the stiffness, increasing the deformation at yield stage. That’s why, the ductility of cyclic loading samples is greater.

CHAPTER 5 CONCLUSIONS AND RECOMMENDATIONS

5.1. Conclusions

The focus of the study was placed on the enhancing flexural properties of Concrete Filled Double Skin Tubular (CFDST) beams. Improvements in the section of CFDST beams were proposed by replacing outer FRP layer by steel and making the geometry of the section as square. The strength of proposed beam was studied under monotonic loading and strength of CFDST beams is about 4 to 5 times greater than the beams reported in the literature having circular cross section and outer FRP covering. Reason for increase in strength is the greater bending resistance provided by square section and isotropic properties of steel. It is concluded that CFDST beams with square section present best alternative to hybrid-CFDST beams with circular cross section. By placing inner steel tube in CFDST beams eccentric flexural response is further improved. The hollow CFDST beams save significant cost by reducing the concrete and elimination of formwork requirement during casting. It also imparts light weight characteristics to the section by keeping the inner tube hollow. Energy dissipation capacity and ductility is increased by providing minimum longitudinal reinforcement. Cyclic response for these members is less or equal to the monotonic load response. Beams, having minimum reinforcement in addition to steel tubes can resist more cycles after peak loading, therefore they are considered better for cyclic loading.

5.2 Recommendations

Concrete filled double skin tubular members are gaining importance nowadays due to many advantages. Therefore, it is very important to study their behavior in every aspect. Further work on these type of members in flexure need to be done. CFDST beams having outer FRP covering and inner square hollow section can be studied and compared to the beam studied. Also, the experimental data can be verified by analytical model using finite element software i.e. Abaqus or ANSYS.

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