EXPERIMENTAL STUDY ON THE BEHAVIOR OF RC BEAM-COLUMN JOINT RETROFITTED WITH FERROCEMENT JACKET UNDER CYCLIC LOADING

A Project report by

Mohammad Mahbubul Alam Noyan

Student No. 0409042373

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Dr. Khan Mahmud Amanat — Chairman (Supervisor)
Professor
Dept. of Civil Engineering, BUET

Dr. Sk. Sekender Ali — Member
Professor
Dept. of Civil Engineering, BUET

Dr. Tanvir Manzur — Member
Assistant Professor
Dept. of Civil Engineering, BUET

CANDIDATE'S DECLARATION

It is hereby declared that this project or any part of it has not been submitted elsewhere for the award of any degree or diploma.

Signature of the candidate

(Mohammad Mahbubul Alam Noyan)

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ABSTRACT

Beam-column joints, being the lateral and vertical load transferring connections in reinforced concrete structures are particularly vulnerable to failures during earthquakes and hence satisfactory performance of these joints is often the key to control the performance of connecting structural members during any seismic event.

The present study describes the result of performance of reinforced concrete beam-column joint specimens cast and tested to failure during the experimental work. Two specimens were coated with ferrocement and two were without ferrocement jacket. Reinforcement detailing was as per BNBC 2006. Axial load was applied along the column axis and cyclic loads with gradual increments were applied at tip of cantilever beam till the ultimate capacity was attained accompanied by formation and propagation of crack and failed after formation of hinges. Before formation of first crack ferrocement jacketed specimens experienced 33.26% more lateral load than normal specimens. Maximum deflection before failure was found to be less for ferrocement jacketed specimens. Numbers of cracks were fewer for ferrocement jacketed specimens relative to normal specimens. Hysteresis loops showed higher ductility for ferrocement jacketed specimens than that of normal specimens. Ultimate moment carrying capacity of beam-column joints of specimens with ferrocement jacket found 20% greater than that of normal specimens.

Therefore, it may be concluded that ferrocement jacketing may be effectively used to increase the strength and ductility of beam-column joints.

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LIST OF ABBREVIATIONS

ACI	American Concrete Institute
ASCE	American Society of Civil Engineers
BS	British Standard
CFRP	Carbon fibre-reinforced polymer
CMU	Concrete masonry unit
FEMA	Federal Emergency Management Agency
FRP	Fibre-reinforced polymer
GFRP	Glass fibre-reinforced polymer
RC	Reinforced concrete
RS1	Reference Specimen 1
RS2	Reference Specimen 2
JS1	Ferrocement Jacketed Specimen 1
JS2	Ferrocement Jacketed Specimen 2

Chapter 1
INTRODUCTION

1.1 GENERAL

Bangladesh has already been known to be an earthquake prone area. Here, almost all residential buildings are reinforced concrete (RC) structures. In RC buildings, portions of columns that are common to beams at their intersections are called beam-column joints. Since, their constituent materials have limited strengths; the joints have limited force carrying capacity. When forces larger than these are applied during earthquakes, joints are severely damaged.

Fig.1.1: Exterior joint failure during the 1999 Kocaeli Turkey earthquake (Ghobarah and Said, 2002)

Recent earthquakes in different regions of the world have exposed the vulnerability of existing reinforced concrete (RC) beam-column joints to seismic loading. Till early 1990s, concrete jacketing and steel jacketing were the two common methods adopted for strengthening the deficient RC beam-column joints. Concrete jacketing results in substantial increase in the cross sectional area and self-weight of the structure. Steel jackets are poor in resisting weather attacks. Both methods are, however, labour-intensive and sometimes difficult to implement at the site. A new technique has emerged recently which uses fibre-reinforced polymer (FRP) sheets to strengthen the beam-column joints. FRP materials have a number of favourable characteristics such as ease to install immunity to corrosion, high strength; availability in sheets etc., the simplest way to strengthen such joints is to attach FRP sheets in the joint region in two orthogonal directions.

Effectiveness of FRP and CFRP (carbon fibre-reinforced polymer) as a repair and retrofitting material is now well established. However, FRP/CFRP is expensive material and therefore may not be economically attractive in developing countries like Bangladesh. Low cost alternative should be sought after. Ferrocement laminates is a proven material for general purpose repair of RC structures. Over the past three decades, the use of ferrocement has gained tremendous popularity in different areas of civil engineering (e.g. masonry structures, water tanks, fluid retaining structures etc.). Therefore, the present study has been aimed at performing some experimental investigations using ferrocement as a replacement for FRP/CFRP in retrofitting/strengthening RC beam-column joints.

1.2 FERROCEMENT-AN OVERVIEW

Ferrocement is a highly versatile form of reinforced concrete, constructed of hydraulic cement mortar reinforced with closely spaced layers of continuous and relatively small diameter wire mesh. The mesh may be made of metallic or other suitable material. Ferrocement primarily differs from conventional reinforced or pre-stressed concrete by the manner in which the reinforcing elements are dispersed and arranged. In this regard, the American Concrete Institute (ACI) Committee 549 (1993) put forward the definition of ferrocement as follows:

"Ferrocement is a type of thin wall reinforced concrete construction where usually hydraulic cement is reinforced with layer of continuous and relatively small diameter mesh. Mesh may be made of metallic or other material."

Ferrocement has gained widespread popularity in the developing nations. It has certain inherent advantages and has been accepted as a suitable technology for developing countries for the following reasons:

(a) Its basic raw materials are readily available in most countries.
(b) It can be fabricated into any desired shape.
(c) The skills of ferrocement construction can be acquired easily.
(d) Heavy plants and machinery are not involved with ferrocement construction.
(e) In case of damage, it can be repaired easily.
(f) Being labor intensive, it is relatively expensive in developed countries.

1.3 OBJECTIVE OF THE PRESENT STUDY

The objectives of the present study are as follows:

i. To study the performance of RC beam-column joints retrofitted with ferrocement under cyclic loading.
ii. Comparative study of the bare RC beam-column joints with ferrocement jacketed joints.

1.4 METHODOLOGY AND SCOPE

The present research topic has been undertaken to study the behavior of reinforced concrete beam-column joint retrofitted with ferrocement under cyclic loading through experimental study. To study these effects four T-shaped RC frames representing beam-column joint were constructed in the laboratory. Two of these frames were jacketed with ferrocement and remaining two specimens were kept as cast without jacketing and denoted as reference specimen. Cyclic load was applied with same arrangement to all four specimens. Crack formation and propagation with deflection behavior throughout the loading period till complete failure were studied and comparison between reference specimens and jacketed specimens were studied. Findings were quantified for the purpose of presenting in graphs and tabular form.

This is not a true type sampling with respect to building frame, because in a building frame beam-columns are monolithic with slab. Here, only beam and column are demonstrated. T-shaped joint forms only in exterior zone, but in interior joint members are extended along all four directions. So, these are the limitations of this study.

1.5 ORGANIZATION OF THE THESIS

Chapter two presents literature review on beam-column joint strengthening methods. Constituents, properties, types are also discussed with previous numerical and experimental studies.

The details of comprehensive experimental program are presented in the Chapter 3, which includes the designation and description of the specimens, the constituent materials, the proposed ferrocement, experimental set-up, instruments as well as the loading routine.

Experimental results and discussions are presented in chapter 4.

Finally, conclusions are drawn and recommendations for future research are presented in Chapter 5.

Chapter 2
LITERATURE REVIEW

2.1 INTRODUCTION

Reinforced concrete beam-column joints have an important function in the structural concept of many structures. Often these joints are vulnerable to loads due to impact, explosion or seismic loads, improper reinforcement details in beam-column joints, increase in the applied loads, human errors in initial constructions, change in use or configuration, or of strength in structural members due to deterioration over time. Confinement in beam-column joints is an effective and efficient method for strengthening and increasing ductility of members.

2.2 PHILOSOPHY OF SEISMIC UPGRADE

Different strategies can be pursued to upgrade seismically deficient RC (reinforced concrete) structures. The objectives of the seismic upgrade are to increase the strength or ductility of overall structure. Seismic guidelines such as those by FEMA 356(2000), ASCE 31-03, ASCE 41-06 underline that the objective can be accomplished by:

(a) Local modification of components,
(b) Removal and lessening of existing irregularities and discontinuities,
(c) Global structural stiffening,
(d) Global structural strengthening,
(e) Mass reduction,
(f) Seismic isolation, and
(g) Supplemental energy dissipation.

Among these possible approaches, local modification of components has been adopted in seismic strengthening of RC beam-column joints.

2.3 REPAIR AND STRENGTHENING TECHNIQUES OF BEAM-COLUMN JOINTS

A variety of techniques have been developed to strengthen the beam-column joints. These techniques can be classified into six categories, including epoxy repair, removal and replacement, RC jacketing, concrete masonry unit jacketing, steel jacketing, and application of fiber-reinforcement polymer (FRP) composite. Each strengthening technique will be discussed below.

2.3.1 Epoxy Repair

Concrete structures have long been repaired using epoxy pressure injection. Filiatrault and Lebrun (1996) tested two full-scale exterior beam-column joint specimens, one with non-seismic and other one with closely spaced transverse reinforcement in the beam, column and joint. The specimens were repaired by epoxy pressure injection. Test results indicated that epoxy pressure injection was effective in increasing the strength, stiffness and energy-dissipation capacity of the non-seismically detailed specimen. French *et al.* (1990) studied the effectiveness of epoxy techniques (Fig. 2.1) to repair two; one-way interior joints that were moderately damaged due to inadequate anchorage of continuous beam bars.

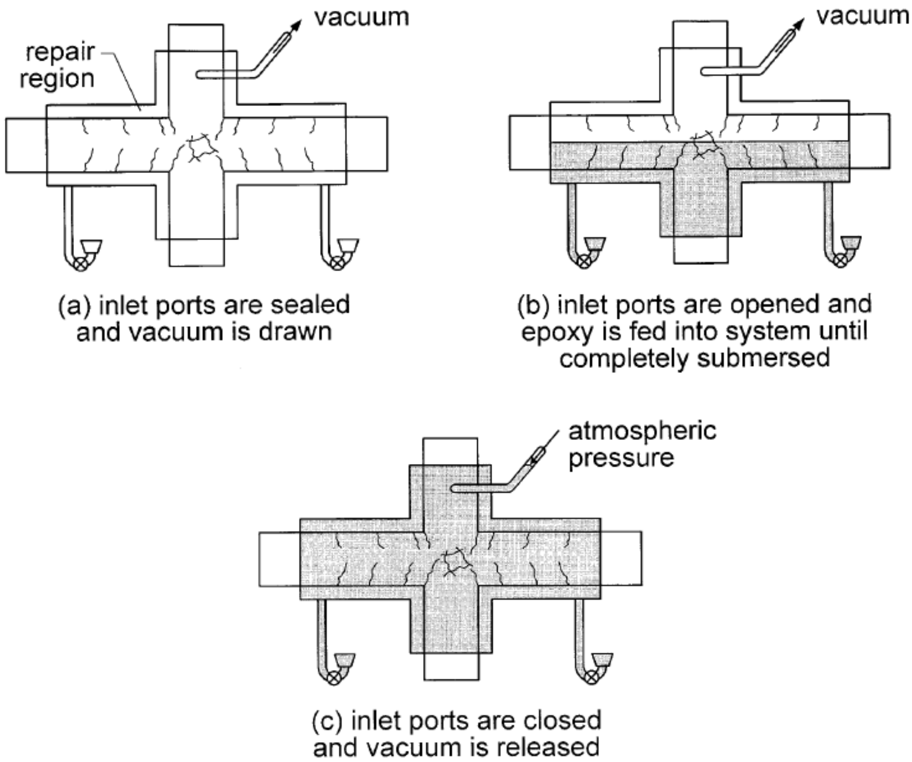


Fig.2.1: Vacuum impression procedure applied by French *et.al (*1990)

For vacuum impregnation (new method proposed by them) epoxy inlet ports were located at the bottom of each beam and at the base of the column repair region. The inlet ports were then sealed and vacuum was applied. Epoxy was fed into system until the system was completely submersed. The inlet ports were closed and vacuum was released. The repair technique was successful in restoring over 85% of the stiffness, strength and energy dissipation characteristics of the original specimen. The authors concluded that vacuum impregnation was effective means of repairing large region of damage at once and that it could be modified for joints with fewer accessible sides.

It has been noted that the results of the epoxy repair techniques often show partial recovery of stiffness of the original specimens. The bond around the reinforcement cannot be completely restored once destroyed. The effectiveness of the technique greatly depends on workmanship of the workers.

2.3.2 Removal and Replacement

Repair of heavily damaged joints with crushed concrete often involves partial or total involve and replacement of concrete. The damaged structure is usually supported temporarily to ensure stability. Prior to the replacement of the concrete, the damaged concrete and any loose particles will be removed. Generally high strength and non-shrink concrete will be used for replacement.

Karayannis *et al.* (1998) conducted an experimental study on six interior beam-column joints repaired with high strength non-shrink cement paste. Although significant increase in strength, stiffness and energy dissipation was recorded, the specimens were found to fail in joint shear. The enhancement could be attributed to the use of high strength concrete in the study.

Many research results indicate that this technique is feasible for the strengthening of beam-column joints. However, similar to the previous strengthening technique, the effectiveness of this technique is limited by the accessibility to the joint. Moreover, beam-column joints with buckled or ruptured reinforcement cannot be strengthened by this technique without replacement of the longitudinal reinforcement.

2.3.3 Concrete Jackets

RC jacketing involves encasing the column and joint region, sometimes as well as a portion of the beam, in new concrete with the additional longitudinal reinforcement and transverse reinforcements. The addition of longitudinal reinforcement requires opening of the slab at the column corners. Beams need to be cored as well to provide transverse reinforcement in the joint region.

Due to the difficulties in drilling the beams and placing joint confinement reinforcement, the additional joint reinforcement was usually substituted by some steel components. In the tests carried out by Alcocer and Jirsa (1993), the joint reinforcement was replaced with steel cage welded around the joint (Fig. 2.2). The cage consisted of steel angles designed to resist the lateral expansion of the joint and flat bars were used to connect the angles together. The variables under investigation were jacketing the column only or both beams and columns, jacketing after or prior to first damage, and using bundles or distributed column reinforcement. The specimens were found to fail in joint when the beams were not jacketed. It was reported that the steel cage and the corner ties confined the joint satisfactorily up to 4% drift.

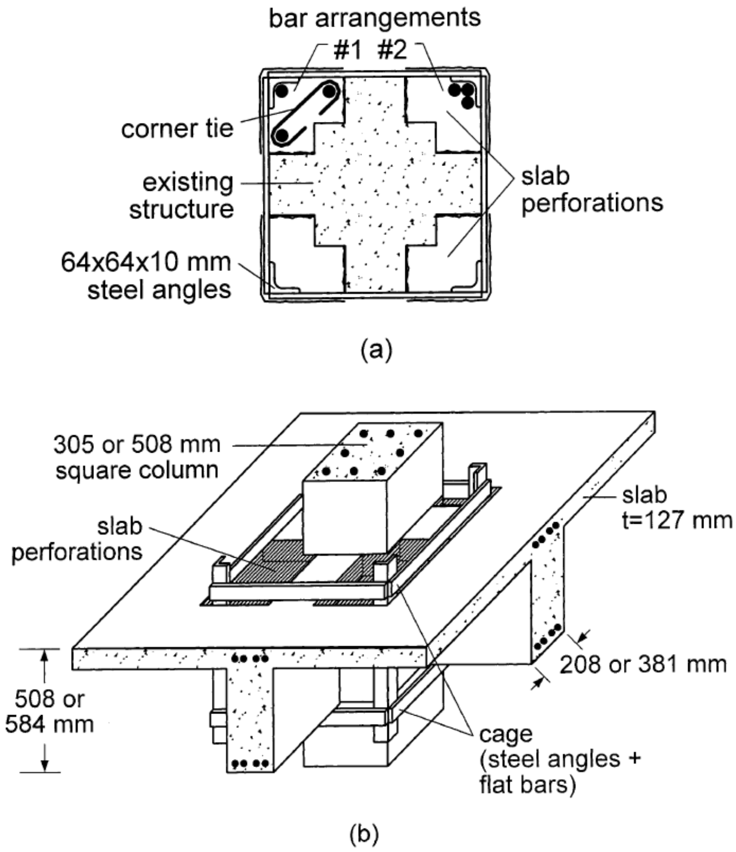


Fig. 2.2: Corner jacketing technique studied by Alcocer and Jisra: (a) plan, and (b) perspective

Corner jacket reinforced with steel fibers has been used in strengthening beam-column sub-assemblages as well. Shannag and Alhassan (2005) tested ten 1/3-scale interior beam-column sub-assemblages strengthened using a 25mm-thick jacket of high performance fiber-reinforcement concrete all around the joint column region. The test results indicated that the seismic behavior of upgraded specimens was improved substantially. Higher load levels and larger displacements were attained, slower stiffness degradation and higher energy dissipation were achieved, and a significant increase in the joint strength was recorded. The experimental results also revealed that as the value of the column axial load increased, the connections attained more lateral load capacity. Similar results were obtained by the study carried out by Parra-Montesinos *et al*. (2005).

Even though increase in joint strength overall lateral strength and energy dissipation were recorded in general with the concrete jacketing technique, the procedure is very labor-intensive. It usually involves perforating the floor slab, drilling through beams and adding joint transverse reinforcement. Concrete jacketing will definitely increase the member sizes and dead loads of the structure, and results in poor appearance.

2.3.4 Concrete and Masonry Unit Jacketing

Strengthening of beam-column joints using reinforced concrete masonry units (CMUs) has been analytically studied by Bracci *et al.* (1995). Basically this technique requires the existing columns to be jacketed by CMU's with additional longitudinal reinforcement within the corner cores extending continuously through the slabs and later post-tensioned. The space between the units and the column will then be grouted. Wire mesh is provided in the mortar bed joints to enhance the shear capacity. The non-linear analyses showed that strong-column weak-beam behaviour was enforced and adequate control of inter- story drift was achieved. However no experimental data are available to validate their performance and the same limitations of concrete jacketing applies to CMU jacketing.

2.3.5 Steel Jacketing

Steel jackets of various forms and shapes have been used to enhance the joint shear strength. They consist of flat or corrugated steel plates welded in place. The space between the jacket and joint-column region is grouted with non-shrink cement grout. The steel parts are mechanically anchored to the concrete to improve the confinement to the joint. Adhesive or bolts are normally used to attach the steel plates to the concrete surfaces.

Beres *et al.* (1992) strengthened the interior joints with discontinuous bottom beam reinforcement using steel components (Fig. 2.3). Steel channels were bolted to the underside of the beams and connected by steel tie-bars running alongside the column. A20% increase in peak strength and 10 to 20% increase in stiffness were recorded. Strengthening of exterior joints was conducted as well. Steel plates were placed along the opposite faces of the upper and bottom columns and they were connected with threaded rods. A 33% increase in peak strength and 12% increase in initial stiffness were recorded, with noticeable increase in energy dissipation.

Ghobarah *et al.* (1996) proposed corrugated steel jacket (Fig. 2.4) to improve the joint confinement. The gap between the concrete and the jacket was filled with grout. The shear strength of the rehabilitated joints was increased and the failure mode became flexural hinging in the beam.

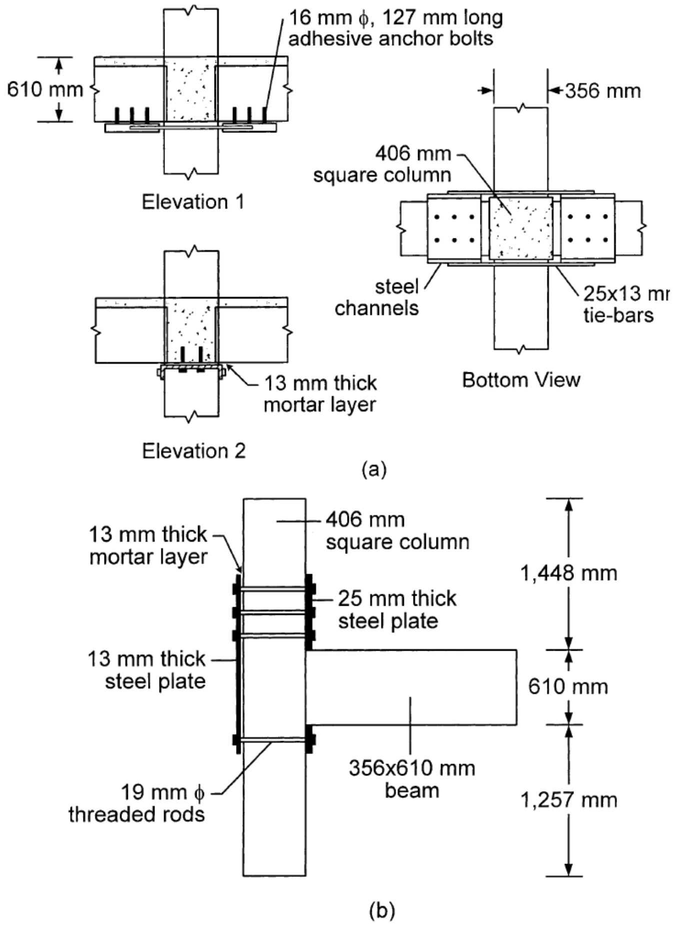


Fig.2.3: External steel configurations studied by Beres *et al.*

Compared with concrete jacketing, steel jacketing can significantly reduce the construction time due to prefabrication. However, problems arise from the corrosion of steel and difficulty in handing heavy steel plates. The steel jackets proposed cannot be practically applied in case where floor members are present.

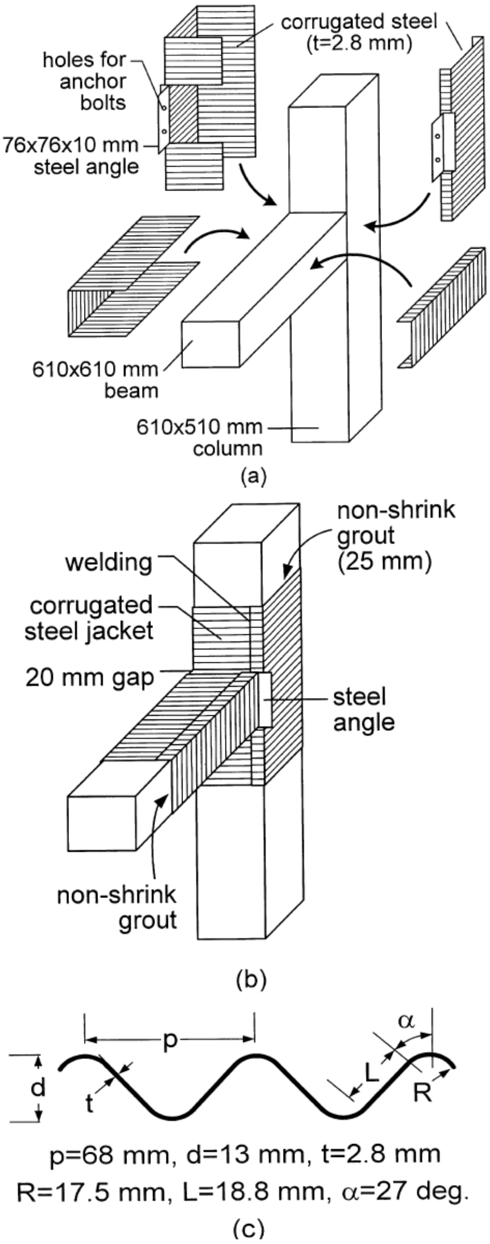


Fig.2.4: Corrugated steel jacketing (Ghobarah *et al*. 1997)

2.3.6 Fibre-reinforced Polymer Composites

More than a decade ago, a new technique for strengthening structural elements emerged. This involves the use of fibre-reinforced polymer composites (FRP) (Fig.2.5) as externally bonded reinforcement in critical regions of RC elements. FRP materials have a number of advantages over steel and concrete that make them an ideal material in strengthening the deficient beam-column joints. They are non-corrosive, highly durable, electro-magnetic neutral (except for carbon fibre), high strength to weight ratio, easy to apply, high mouldability and availability in many forms.

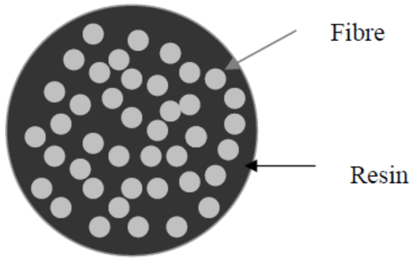


Fig.2.5: Schematic representation of cross section of FRP

Antonopoulos and Triantafillou (2003) conducted experimental investigation of fiber reinforced polymer -strengthened RC beam-column joints. Their experiments provided a fundamental understanding of the behavior of shear-critical exterior reinforced concrete RC joints strengthened with fiber reinforced polymers (FRP) under simulated seismic load. The role of various parameters on the effectiveness of FRP was examined through 2/3-scale testing of 18 exterior RC joints. The specimens were designed to fail in joint shear before and after the application of FRP so as to evaluate the contribution of FRP to the shear capacity of joints. The failures were due to partial or complete debonding of composites. An increment of 70%-80% was recorded in the cumulative energy dissipation and stiffness characteristics of poorly detailed RC joints with enhanced by bonding FRP reinforcement externally.

The feasibility of achieving large displacement capacity and damage tolerance in frame structures designed with simple reinforcement detailing in beams and connections by using High-Performance Fiber-Reinforced Cement Composites (HPFRCC) materials was evaluated by Parra-Montesinos, *et al.*(2005). The reductions in transverse reinforcement requirements and associated labor, and more importantly, the achievement of highly damage tolerant structures that would most likely require few or no post-earthquake repairs, would make the use of HPFRCCs in selected regions of frame structures attractive from both structural and economical viewpoints.

Perrone *et al.* (2003) applied CFRP-Based Strengthening Technique to increase the flexural and energy dissipation capacities of RC beams. The Laminates were applied according to the near surface mounted technique to increase the flexural resistance of the column, while the strips CFRP (carbon fiber-reinforced polymer) sheets were installed according to the externally bonded reinforced technique to enhance the concrete confinement. In groups of RC column with 29 MPa concrete compressive strength, this technique provided an increase of about 39% and 109% in terms of column’s load carrying capacity, undamaged and damaged columns, respectively.

Vijayalakshmi, N., *et al.* (2010) conducted experimental investigation on RC beam-column joint strengthening by FPP wrapping. A two bay five story RC frame was designed for seismic loads according to IS 1893 and IS 13920. The frame was subjected to forward and reverse cyclic loading and strengthening of beam column joint after post yielding using FRP Wrapping and behavior of joint is studied up to failure loading condition.

2.4 LITERATURE ON FERROCEMENT

To study the effect of ferrocement overlay on reinforced concrete elements, first it is necessary to understand the behaviour of ferrocement under different conditions. In this regard, it is necessary to identify the parameters affecting the properties of ferrocement and review relevant literature in this field. This chapter presents a brief literature review on the properties of ferrocement and review relevant literature in this field. Experimental investigations carried out by several researchers on the behaviour of a reinforced concrete beam with ferrocement overlay are also included.

Research and development work on ferrocement has progressed at a tremendous pace during recent years and a variety of structures using innovative design and construction techniques have been built worldwide. As a result, a large volume of technical information is now available on various aspects of ferrocement design, construction, maintenance and repair. Increasing popularity and growing public acceptance have made it necessary to formulate design and working guidelines by collecting the available information. Efforts have also been made in recent years to improve the performance of reinforced concrete elements by applying ferrocement overlay. The concept has been intuitively applied for repair and strengthening of distressed elements.

Anwar *et al.* (1991) investigated the rehabilitation technique for reinforced concrete structure beam elements using ferrocement. The technique involved strengthening of reinforced concrete beams by application of hexagonal chicken wire mesh and skeletal steel combine with the ordinary plastering. The test result is in good compliance with the original design capacity of the beams. From the test result obtained a design chart has been developed to determine the parameters of rehabilitation of the beam elements.

Lub and Wanroji (1998) reported that strengthening of existing beams in reinforced concrete building structures by means of shotcrete ferrocement. It was found that the mesh is fully effective and monolithic condition shotcrete layer and original concrete beam attained. The wire mesh was found to act as excellent shear reinforcement.

Rosenthal and Bljuger (1985) studied the flexural behavior of ferrocement reinforced concrete composite beam in the serviceability and ultimate limit states. The flexural behavior of four rectangular composite beams made of low strength ferrocement, was compared with four reference beams in the serviceability and ultimate limit states. In doing so, special deformation and crack formation properties of the encasing elements (reinforced with wire meshes) were exploited, resulting in hair cracks which appear in the beam under service load, rather than regular width cracks. Cracking moments of the composite beams were 11% and 13% higher than those of reference beams due to additional flexural tensile strength contributed by the elements. Crack in the composite beams have only reached, at failure, a width of 0.4 mm to 0.5 mm, as compared to twice as much in the reference beams. Composite actions between the skin and core components were fully obtained until crack appearance. Beyond that stage and up to failure, a partial separation might have appeared, according to somewhat different crack patterns of the reference and composite beams.

Kaushik and Dubey (1994) studied the performance of RC ferrocement composite beams through experimental investigation on RC beam cast on ferrocement and distressed beams rehabilitated by ferrocement jacketing. They reported that the increase in ultimate strength compared to RC beams was 44% for composite beams and 39% for rehabilitated beams. This showed that composite beams and rehabilitated beams are capable of performing equally well. Moreover, the ultimate strength and stiffness of RC beam can be significantly increased by strengthening with precast ferrocement plates in the shear failure zone. Therefore, ferrocement can satisfactorily be used as the precast part of the composite in which RC beam is cast.

An experimental investigation was carried by Kadir *et al.* (1997) to study the ultimate load, flexure behavior and mode of failure at collapse of reinforced concrete beams using ferrocement concrete beams using ferrocement permanent formwork (composite beams). The linkage between the two materials was achieved by placing shear connectors along the strength of the beam. Test result showed that the reinforced concrete beam with ferrocement permanent formwork failed by flexure. The composite beam with shear connectors carried about 12% higher load and 10% reserved flexural strength and show lower deflection when subjected to reinforced concrete beam without shear connectors. The ferrocement formwork with and without shear connectors contributed about 21%~75% and 16%~50% to the flexural strength respectively.

Afsaruding and Hoque (1998) performed an experimental research work on reinforced concrete beams with ferrocement overlay in the concrete laboratory, BUET. They investigated the possibility of using ferrocement as a permanent formwork for reinforced concrete beams. A total of twelve beams were constructed and tested in the investigation. Eight ferrocement formworks were made having different sizes. All of them were filled with reinforced concrete. Four reinforced concrete beams and eight reinforced concrete beams coated with ferrocement formworks containing single layer wire mesh were cast to compare the behavior of ferrocement formwork reinforced concrete beam with the normal concrete beam. The study demonstrates that the use of ferrocement as a permanent formwork increase the cracking load and ultimate load of the composite system compared to normal RC beams. The number of cracks and width of cracks have been found to have reduced considerably due to provision of ferrocement layer used as formwork. From the study it appears that permanent precast ferrocement formwork could become a reliable alternative to wooden formwork in the construction of reinforced concrete beams.

The ability of ferrocement to fit snugly into curved surface makes it an ideal material for the rehabilitation of domes and shells. An example of such rehabilitation is the restoration of domes in the Windmill theatre in UK (1998) (Rahman, 2002).

Sharma *et al.* (1984) rehabilitated an overhead circular tank of 21000 liter capacity using ferrocement. The superior crack resistance properties made it suitable for water retraining satisfactorily. The tank was put out of service due to heavy leakage soon after its construction. The inspection of tank revealed the presence of large cracked and honeycombed area in the center of tank wall which was along the wall periphery. At some point only coarse aggregate was deposited with no fine aggregate taking making it the major source of water leakage through the voids in such area. After repairing by using ferrocement no leakage was observed and the tank seemed to be performing with full efficiency.

Trikha *et al.* (1988) reported the process of repairing of damaged steel water tank using ferrocement. Steel water storage tank are widely used in every part of the world. One predominant problem associated with steel tank is that of corrosion. Due to the corrosion of steel tank have to be replaced after an extra period of time. A common remedial measure is to patch up to the corroded portion with new plates welded in place. But this option is not economical. The rehabilitation using ferrocement is quite economical and simple. The process consist of using the existing steel tank as a formwork while a new ferrocement inner lining is provided at hold water. Roorkee University has been successfully rehabilitated using ferrocement inner lining, (Rahman, 2002).

The process of sanitary sewer relining using ferrocement to rehabilitate the sewer has already gained wide acceptance in UK. It now is more commonly used in other countries as well. In Sewer application it is important to pay attention to the type of and porosity of the mortar matrix used. This should pose no problem since there are a variety of formulations that provide adequate resistance to chemical attack. The cement used for mortar should be sulphate resistant.

Ioms (1987) studied the performance of ferrocement in construction and repair of boat trawlers. He showed that in fact, the most successful and convincing application of ferrocement has been in construction and repair of boats. Ioms suggests and open mold of system to be used for better repair of boat. Instead of using no form of wire mesh layer is used directly.

Reinhorn and Prawel (1988) successfully used thin ferrocement coating on the sides of the unreinforced masonry wall that need enhanced in plane and out of plane strength and ductility. Ferrocement coating was mounted on the two sides of the wall with tension ties provided through the masonry. The result of the test showed the suitability of ferrocement as a retrofit (strengthening) material with a doubling of the wall.

Singh *et al.* (1988) suggest a simple procedure for the strengthening of brick masonry column using ferrocement. Brick masonry column on old structure and are usually used for low-rise structures. Although the performance of masonry columns under axial loads may be satisfactory, they pose a limited moment carrying capacity. Improving a moment carrying capacity becomes vital if structure is subjected to modifications resulting in eccentric loads to be transferred to the columns. Ferrocement encasement of masonry column can considerably increase its capacity to resist axial loads and moments. Applying the ferrocement encasement, (Singh *et al.*1988) report the failure loads to be double that of uncased columns. Failure is due to failure in casing under combined bending and tension under lateral loads.

Alam (2003) studied lateral strength of masonry unfilled reinforced concrete frame retrofitted with ferrocement and found greater lateral load capacity.

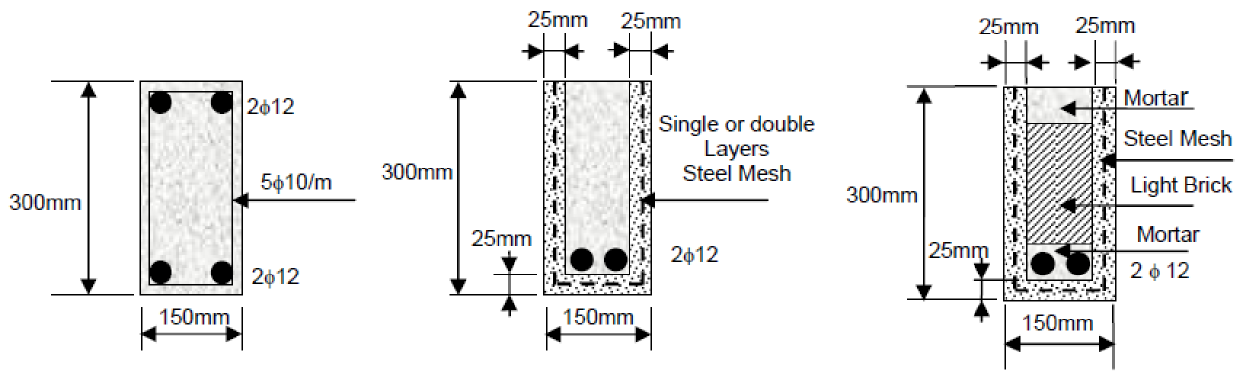
Kaish *et al.* (2013) conducted experiment on six small-scale columns jacketed with wire mesh and two non-jacketed columns. They showed increment in ultimate load carrying capacity from 28.86% to 44.68% than that of non-jacketed columns.

Kumar, P.R *et al*. (2007) conducted experiment in three scale model bridge pier specimnes (Fig.2.6) designed as shear deficient specimnes, tested under different axial loads, before and after retrofitting with ferrocement jackets. The three RC column swere strengthed with six layers of ferrocement jackets. The extrenal confinement using ferrocement jacket resulted in enhanced stiffness, ductility strength and energy dissipation capacity.

Fig.2.6: Ferrocement jacketing (Kumar, P.R *et al*. 2007)

Abdel Tawab *et al.* (2012) has presented the results of experimntal investigation to examine the feasibility and effectiveness of using pre cast U-shaped ferrocement laminates as permanent forms for construction of reinforced concrete beams. The precast permanent ferrocement formes were proposed as a viable alternative to the commonly used wooden and/or steel temporary forms. The authors used woven wire mesh, X8 expanded wire mesh, and EX156 expanded wire mesh for reinforceing the precast ferrocement forms. The precast ferrocement forms were filled with conventional concrete reinforced with two steel bars. Neither bonding agent not mechanical shear connection was used in that research to provide shear connection between the forems and the core. The reported results showed that high serviciability and ultimate loads, crack resistance control. And good energy absorption properties could be achieved by the proposed ferrocement forms.

Fahmy *et al. (*2005) conducted experiment on twelve beams (Fig.2.7 and Fig.2.8) having dimensions of 300×150×2000mm consisting of concrete core cast in 25mm-U shaped permanent ferrocement laminates forms, and six beams of total dimensions of 300×150×2000mm consisting of light weight brick core built in 25mm U-shaped permanent laminates form. Comparison studied with same size control beams. They reported high first crack, serviceability and ultimate loads, crack resistance control, and good energy absorption properties with ferrocement form work specimens. They concluded savings in total steel weight ranging from 27.4% to 37.7% could be achieved by employing permanent ferrocement forms depending on type of mesh and number of mesh layers.



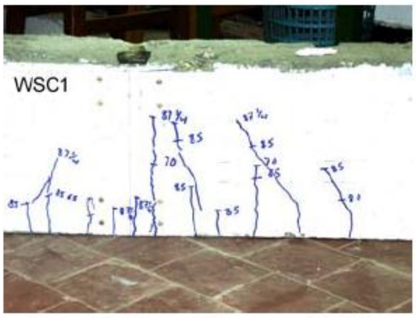
a) Control beams

b) Beams incorporating U-shaped forms and concrete core

c) Beams incorporating U-shaped forms and brick core

Fig.2.7 Cross sections of test specimens by Fahmy *et al.* 2005

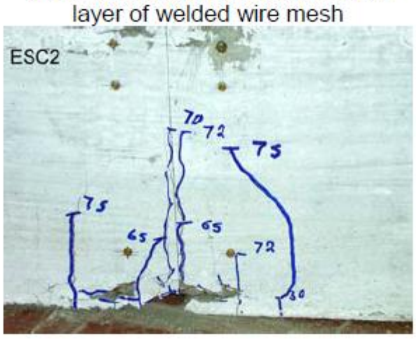
Group 1: Control Beams



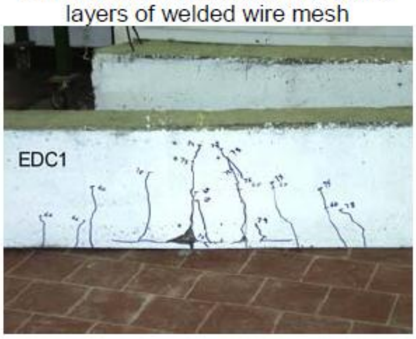
Group 2: Concrete core and single layer of welded wire mesh

Group 3: Concrete core and double layers of welded wire mesh

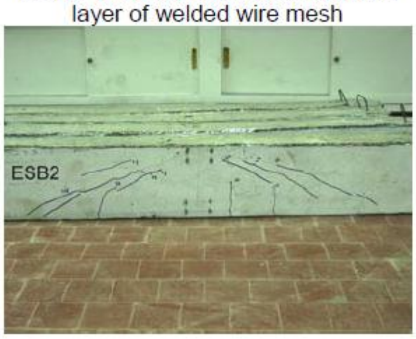
Group 4: Light brick core and single layer of welded wire mesh



Group 5: Concrete core and single layer of expanded wire mesh



Group 6: Concrete core and double layer of expanded wire mesh



Group 7: Light brick core and single layer of expanded wire mesh

Fig.2.8: Cracking pattern of test specimens, Fahmy *et al.* 2005

Fahmy *et al.* (2013) conducted experimental study on 30 beams laminated with U-shaped permanent precast ferrocement form (Fig.2.9) work sand compared with three control beams of same size. Beam size was 300mm×150mm×2000mm. Their study was continution of study of Abdel Tawab *et al.* (2007) with some modifications such as use of different types of concrete with recycled concrete, mechanical shear connections, different types of wire meshes,different number of layers. They achieved higher first cracking load, serviciability load, ultimate load and energy absorption for every specimen group compared to control specimens.


(a) Forming the steel mesh **(b)** Casting the bottom of the U-shaped forms


(c) Casting the sides of the U-shaped forms **(d)** The U-shaped ferrocement forms

Fig.2.9: Preparation and casting of U-shaped ferrocement forms (Fahmy *et al. 2013)*

Kannan *et al.* (2013) conducted experimental study on six scale-down models (Fig. 2.10) of the beam-column joint of a non-seismically designed structure. Two types of ferrocement jacketing schemes were used, first one is the conventional square jacketing and second one is the advanced jacketing system in which the beam and column corners were rounded prior to the application of jackets. All the specimens were subjected to quasi static reverse cyclic loading.

The experimental results showed that there is a 33.33% improvement in ultimate load carrying capacity and 27.2% increment in ultimate load deflection capacity for jacketed specimens compared to that of control specimens. The advanced ferrocement jacketing technique was found to have slightly better performance compared to conventional jacketing.

Ravichacdran and Jeyasehar (2012) conducted experimental study on six full scale RC exterior beam- column joint jacketed with ferrocement and compared results with two control specimen. They also made compare between experimental results and analytical results. They reported enhanced capacity of ultimate energy dissipation capacities for ferrocement jacketed specimens compare to control specimens.

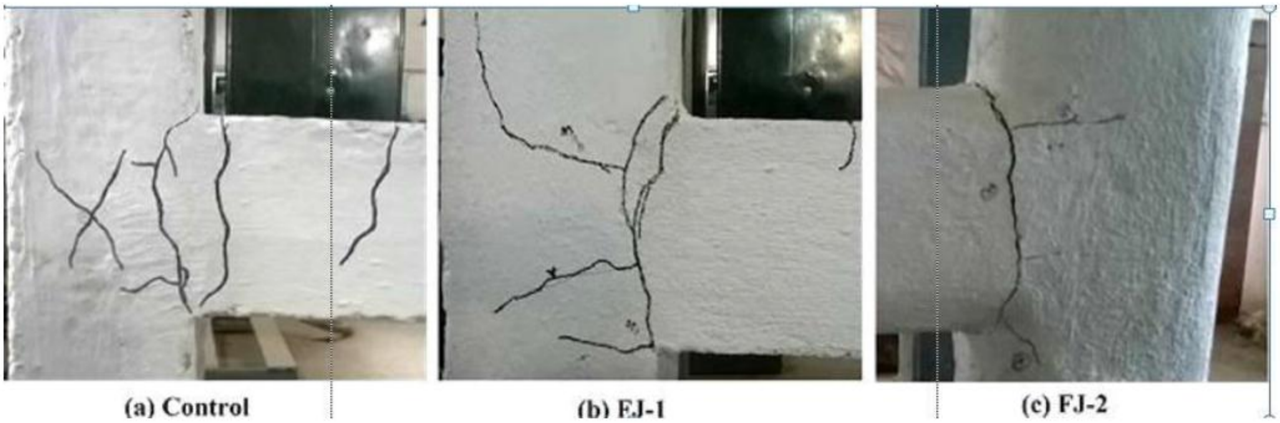
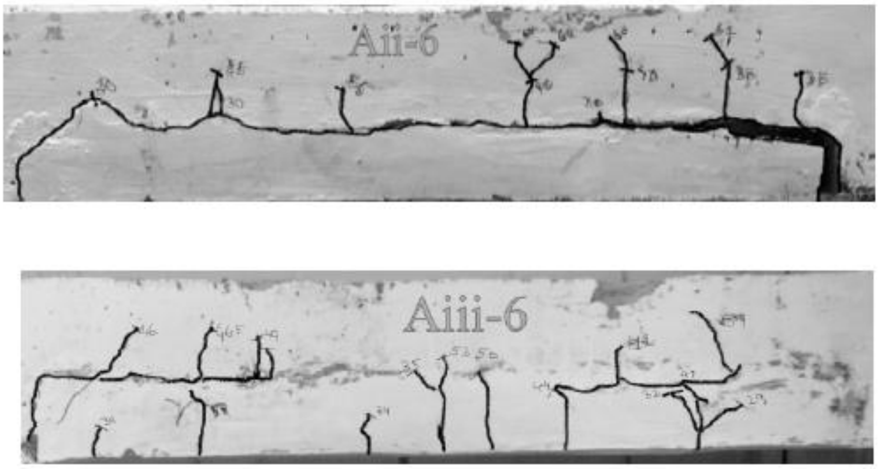
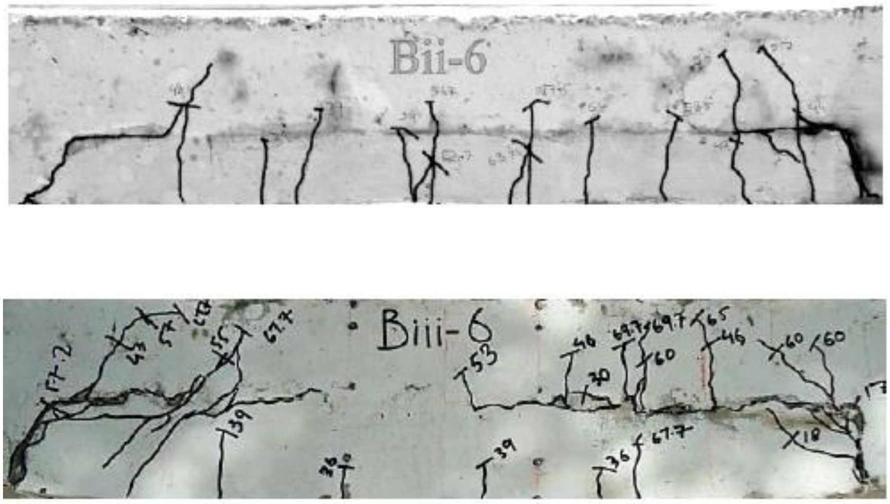


Fig.2.10: Cross section of test beams (Kannan *et al*. 2013)

Khan *et al.* (2013) conducted experimental study on four beams with cast in situ ferrocement lamination and five pre cast ferrocement lamination and compared flexural capacity with one control beam (Fig.2.11). All beam are of same size 150mm×200mm×1800mm. They reported increased capacity of ferrocement laminated beams.



Cast in situ ferrocement beams



Pre cast ferrocement beams

Control beam

Fig.2.11: Crack pattern of test beams (Khan *et al*. 2013)

For many repair and renovation programs of civil engineering structures**,** Chowdhury and Robles-Austriaco (1986) cite the suitability of ferrocement because of

1. Better cracking behavior
2. Capacity of improving some of the mechanical properties of the treated structures.
3. Further modification or repair of ferrocement treatment is not difficult.
4. Imposition of little additional dead load requiring no adjustment of the supporting structures.
5. Ability to withstand thermal changes very efficiently.
6. Ability of achieving water proofing property without providing any surface treatment.
7. Readily available constituent materials.
8. No need for special equipment.
9. Ability to be used in repair program with no distortion or down grading of architectural concept of the structures.
10. Flexibility of further modification.

2.5 CONSTITUENTS OF FERROCEMENT

The constituent materials of ferrocement are cement, sand, water and reinforcing mesh.

2.5.1 Cement

The cement is to be ordinary Portland cement of type1 or Portland composite cement and shall be conforming to ASTM standard.

2.5.2 Sand

Sand should be obtained from a reliable source and should be comply with ASTM C33 for aggregates. It should be clean, hard, strong and free of organic impurities and deleterious substances. The fineness of sand should be such that 100% of it passes through the sieve no.4 (2.36mm).

2.5.3 Water

Water that is used in the mixing should be free from any organic and harmful solution, which leads to the deterioration of the properties of mortar. In any case saline water should not be used. Any water with a pH (degree of acidity) of 6.0 to 8.0 that does not taste saline is suitable for use.

2.5.4 Reinforcing Mesh

The most essential component of ferrocement is steel wire mesh. Different types of wire meshes are available in Bangladesh. Wire mesh generally consists of thin wires either woven or welded in to mesh.

2.6 PROPERTIES OF FERROCEMENT

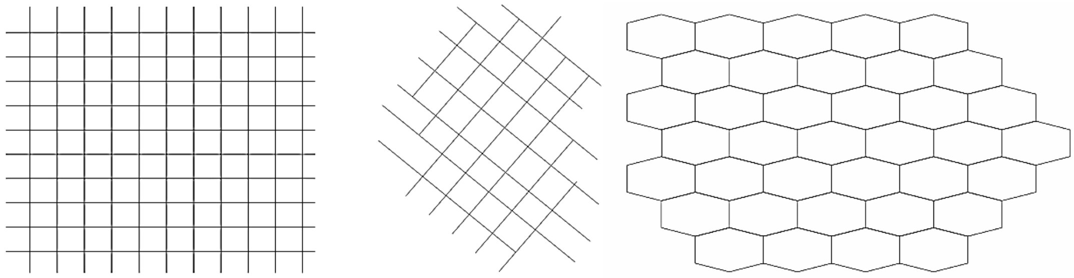
Ferrocement, considered to be extension of reinforced concrete technology, has relatively better mechanical properties and durability than ordinary reinforced concrete. Within certain loading limits, it behaves like a homogenous elastic material and these limits are wider for normal concrete. The uniform distribution and better crack arrest mechanism arrests propagation of cracks and results in high tensile strength of materials.

Many of the properties unique to ferrocement derive from the relatively large amount of two-way reinforcement made up of relatively small elements with much higher surface area than conventional reinforcement. In the words of Nevri who first used the term ferrocement, is most notable characteristic is "greater elasticity and resistance to the cement mortar by the extreme subdivisions and distribution of reinforcement is fundamental and in understanding many of the properties of ferrocement"

In 1993 ACI Committee 549 reported that one of the essential components of ferrocement is wire mesh. Different types of wire meshes (Fig.2.12) are available everywhere. Common wire meshes have hexagonal or square openings. Meshes with hexagonal opening are sometimes referred as to chicken wire mesh or aviary mesh. They are not structurally as efficient as meshes with square opening because the wires are not always oriented in the directions of the principal (maximum) stresses. However, they are very flexible and can be used in doubly curved elements.

Meshes with square openings are available in welded or woven form. Welded wire mesh is made of straight wires in both the longitudinal and transverse direction. Thus welded mesh thickness is equal to two wire diameters. Woven mesh is made of longitudinal wires woven around straight transverse wires.

Depending on the tightness of the wire, woven mesh thickness may be up to the three wire diameters. Welded wire meshes have a higher modulus and hence higher stiffness than woven meshes; they led to smaller crack width in the initial portion of the load deformation curve.



Square Mesh **Expanded Mesh** **Hexagonal Mesh**

Fig.2.12: Different types of wire mesh used in ferrocement, (ACI Committee 549, 1993).

Woven wire meshes are more flexible and easier to work with then welded meshes. Again, welding anneals the wire and reduces its tensile strength, (ACI Committee 549, 1993) Three parameters commonly used in characterizing the reinforcement in ferrocement applications are as follows:

- The volume fraction
- The specific surface of reinforcements
- The effective modulus of the reinforcement

2.7 STRENGTH PROPERTIES

The strength of ferrocement, as in ordinary concrete, is commonly considered as the most valuable property, although in many practical cases other characteristics, such as durability and permeability may in fact be more important. Nevertheless, strength always gives an overall picture of the quality of ferrocement, as strength is directly related to the properties of the hardness of cement paste and reinforcement.

2.7.1 Tensile Strength

The basis of the structural design is the knowledge of material properties. The tensile characteristics of ferrocement have not yet been defined and standardized. In tension the load is essentially independent of specimen thickness because the matrix cracks well before failure and does not contribute directly to composite strength. Naaman and Shah (1971) have studied the influence of types, sizes and volume of wire meshes on elastic cracking and ultimate behaviour of ferrocement in uniaxial tension. They observed that the ultimate tensile strength of ferrocement is the same as that of mesh alone while its modulus of elasticity can be predicted from those of mortar and mesh, Naaman and Shah(1971), Johnston and Mattar (1976**)** and Pama *et al.* (1974) .The specific surface of the reinforcement strongly influenced the cracking behaviour of ferrocement. In general, the optimal choice of reinforcement for ferrocement strength in tension depends on whether the loading is essentially uniaxial or significantly biaxial. Expanded metal in its normal orientation is more suitable than other reinforcing meshes for uniaxial loading because a higher portion of the total steel is effective in the direction of applied stresses, (Johnston and Mattar, 1976). For biaxial loading, square mesh is more effective because the steel is equally distributed in the two perpendicular directions, although the weakness in the 45-degree direction may govern in this case.

2.7.2 Compressive Strength

In this mode unlike tension, the matrix contributes directly to ferrocement strength in proportion to its cross sectional area. Compressive strength of ferrocement (regardless of the amount of mesh reinforcement) seems to be much the same as that of mortar alone. The experimental results (Pama *et al.* 1974) showed that under compression the ultimate strength is lower than that of equivalent pure mortar. The compressive strength at ultimate condition is assumed to be 0.85f'c where f'c is the ultimate compressive strength of the mortar. An investigation into the behaviour of ferrocement specimen in the direct compression has been discussed by Rao (1969).Conclusion was drawn with respect to the effect of the percentage of reinforcement and the size of reinforcement on the behaviour of ferrocement. Provision of reinforcement in excess of about 20 to 2.5% is uneconomical in ferrocement as the proportional increase in strength is not achieved (ACI Committee 549). Smaller diameter wire mesh would be preferable to use as this gives higher elasticity and higher ultimate compressive strength for the same percentage of reinforcement, all other factors remaining essentially the same. When mesh reinforcement is arranged parallel to the applied in one plane only (as opposite to close peripheral arrangement), nom improvement in strength is observed, (Pama*et al.* 1974). The only forms of reinforcement likely to result in significant strength gains in compression are square mesh reinforcement (ACI Committee Report, 549) fabricated in closed box or cylindrical arrangements which results in the matrix, thus forcing it to adopt the triaxial stress condition associate with higher strength.

Chapter 3
EXPERIMENTAL INVESTIGATION

3.1 SELECTION OF SPECIMEN SHAPE

Four reinforced concrete T-shaped frames were designed and constructed according to BNBC (Bangladesh National Building Code) 2006. Two of them were jacketed with ferrocement overlay. Rest two were reference beams. In this experimental study simulating cyclic loading was applied at top of cantilever beam in forward direction and reverse direction by hydraulic jack. Load was applied along column axis. The experiments were performed to investigate the performance of beam-column joint jacketed with ferrocement overlay under cyclic loading. For each test initial cracking load, crack patterns and deflection were observed.

3.2 DETAILS OF SPECIMEN

Fig. 3.1 shows a two bay RC (reinforced concrete) frame with exterior joint representing 'T' shaped beam-column joint. The specimen was constructed like a column that was separated from original column by slicing at a plane above and at plane below of beam-column joint and column lengthens 1500 mm; and a beam was cut at a plane 900 mm distant from face of the joint. The column size was 375mm×250mm×1500mm and beam size was 300mm×250mm ×900mm. There were four specimens. Specimen 1 and specimen 4 are reference specimens i.e. they were not jacketed with ferrocement overlay. Specimen 1 has been denoted RS1 and specimen 4 has been denoted RS2 respectively. Specimen 2 and specimen 3 were jacketed by ferrocement overlay and have been denoted JS1 and JS2 respectively.

3.2.1 Reinforcement Details

Reinforcement details of all four specimens are identical as shown in Fig.3.3 and Fig.3.4.

3.2.2 Cement

Portland composite cement Cem-II was used for casting of specimens and ferrocement plaster. Compressive strength of cement sand (1:3) at the day of experiment (636 days) found as 58N/mm^2.

3.2.3 Coarse Aggregate

Machine crushed locally available well graded 12.5 mm and downgraded stone chips were used for preparation of concrete.

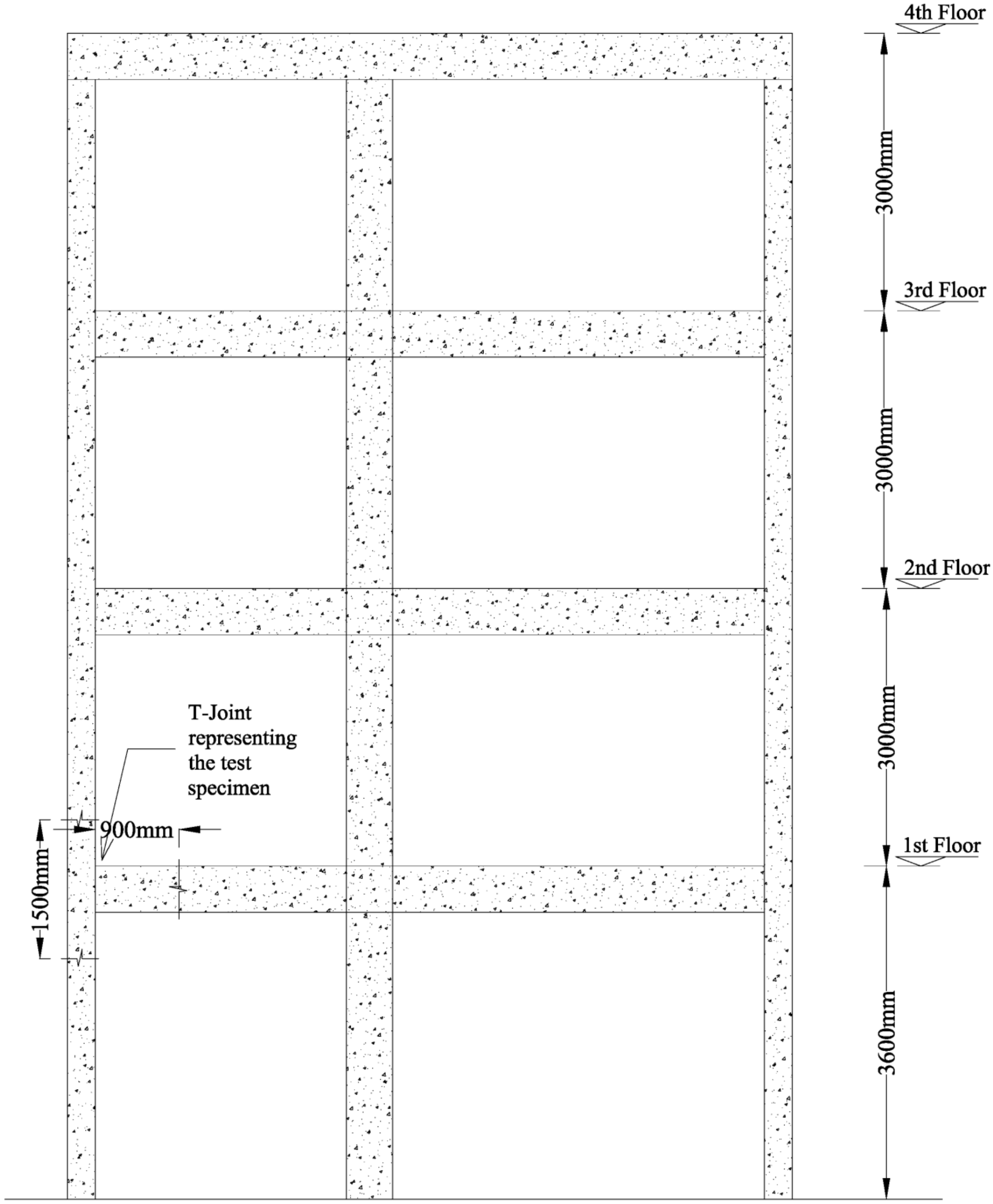


Fig.3.1: Exterior Beam-Column Joint of a typical beam-column frame representing the test specimen

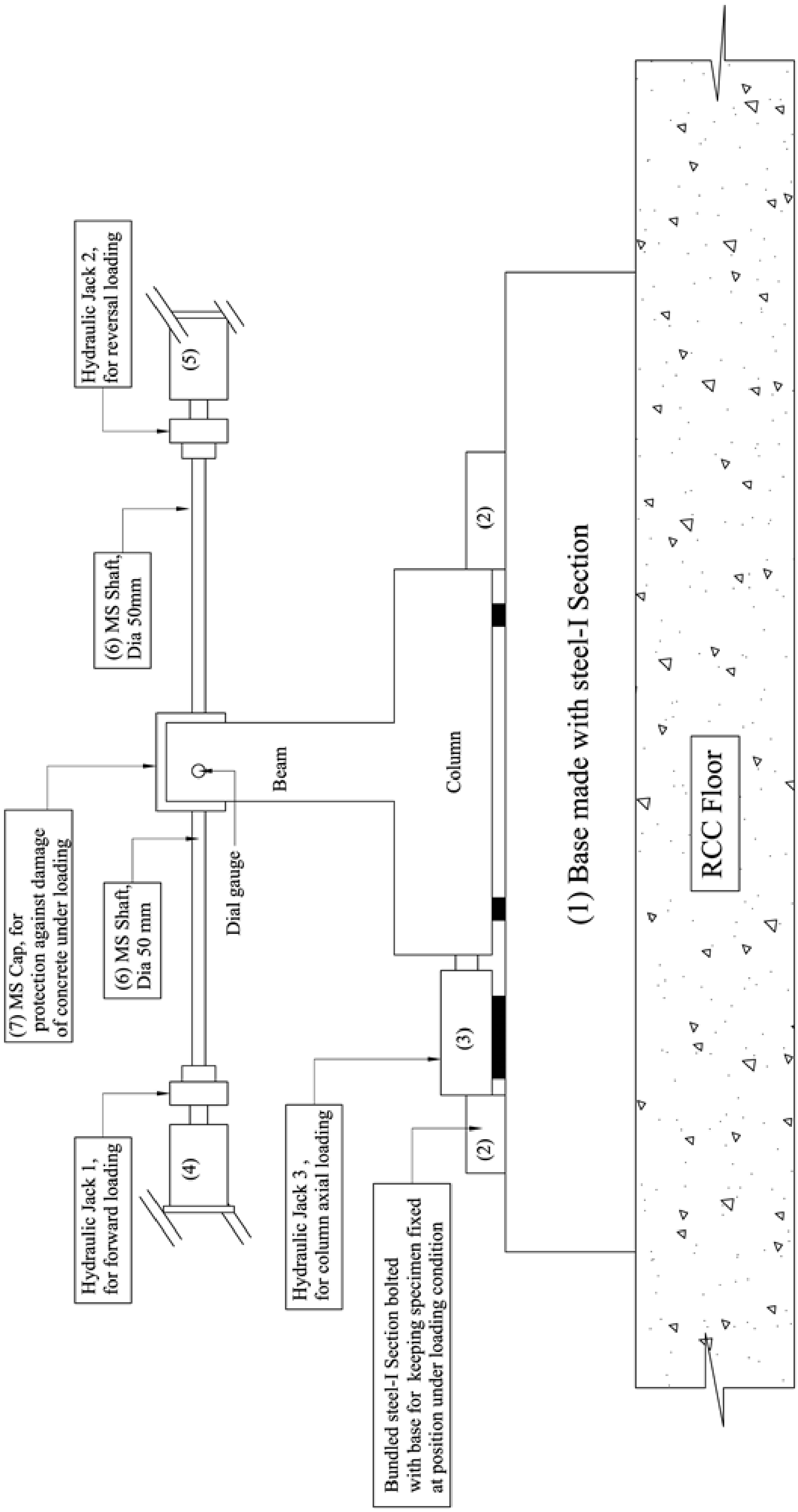


Fig.3.2: Experimental Arrangement

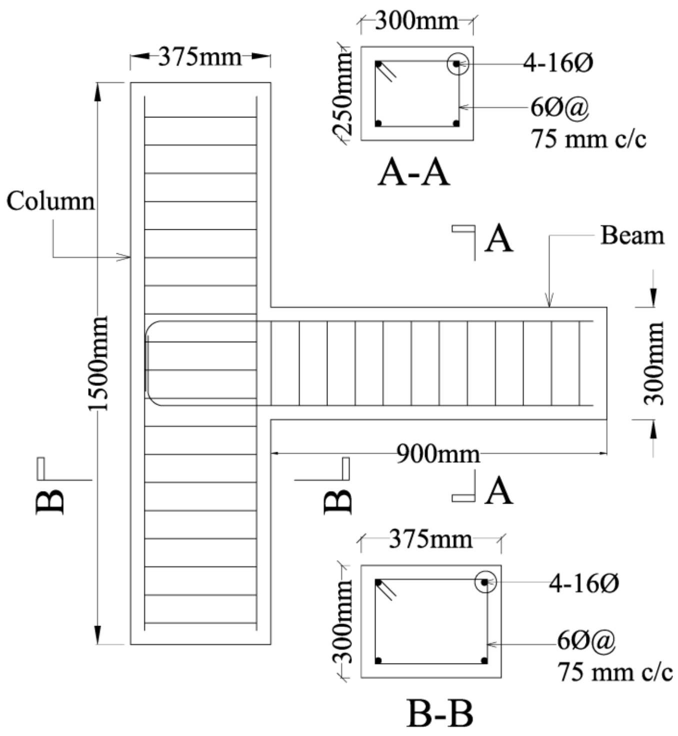


Fig.3.3: Reinforcement details

Fig.3.4: Fabricated reinforcement for test specimen

3.2.4 Fine Aggregate

Locally available coarse sand (locally known as Sylhet sand) was used as fine aggregate for concrete mixing.

3.2.5 Reinforcing Steel

16mm Deformed bar was used as main bar and 6mm plain bar was used as tie bar.

3.2.6 Water

The water from supply main was used for the preparation of concrete and its subsequent curing.

3.2.7 Formwork

Mango wood was used to prepare formwork (Fig.3.5 and Fig.3.6) for column and beam. The formwork was thoroughly cleaned and all the corners and junctions were properly sealed to avoid leakage of concrete through small openings. Shuttering oil was then applied to the inner face of the formwork. The reinforced cage is placed in position inside the formwork carefully keeping in view a clear cover of 38 mm for reinforcing bars.

3.2.8 Concrete

Concrete prepared by mixing raw ingredients with electric mixture machine and vibration was ensured by vibrator machine (Fig.3.7, Fig.3.8, Fig.3.9, and Fig.3.10). Concrete mix proportion was 1:1.25:2.5 (Volumetric). Concrete slump was 225 mm. Cylinder of 100 mm diameter and 200 mm height taken during casting and tested for compressive strength at the same date of experiment. Curing of specimen with water was done for 21 days.

3.2.9 Mortar

Properties of mortar used in ferrocement overlay for retrofitting beam-column joint are given below:

Mixing proportion of mortar:	1.0:2.0
Water Cement Ratio:	0.40
FM of sand:	2.54

Fig.3.5: Preparation of formwork before concreting

Fig.3.6 Reinforcement placed in the formwork

Fig.3.7: Concrete after mixing

Fig.3.8: Concrete pouring

Fig.3.9: Specimen immediate after concreting

Fig.3.10: Concrete cylinders for compressive strength test

3.2.10 Wire Mesh

Reinforcing wire mesh of BWG (British wire gauge) 18 with 12 square openings of woven type having mesh thickness 1.41mm were used is in this study (Fig.3.11 and Fig.3.12). No external bonding agent was used at the interface of wire mesh and RC frame.

3.3 EXPERIMENTAL SETUP

Some modifications were made with existing reaction frame and loading arrangements for performing experiments. The following subsections describe the details of the test arrangements.

3.3.1 Reaction Frame

The sketch of reaction frame diagram is shown in Fig. 3.1 a base (1) made with steel I-Joist fixed with RCC floor of lab with sufficient numbers of 25 mm diameter bolts.

3.3.2 Hydraulic jack for column axial load

To apply column axial load, hydraulic jack was set along axis of column and fixed with base with 25mm Ø bolt in such manner that no slip or dislocation happens during the loading condition. It was confirmed by trial and error method. The jack was calibrated (calibration chart attached in Appendix)

Fig.3.11: Wrapping with wire mesh

Fig.3.12: Jacketing with ferrocement overlay

Fig.3.13: Column axial load arrangement by hydraulic jack

3.3.3 Hydraulic Jack for Cyclic Loading

Two hydraulic jacks (Fig.3.14) were already available and fixed with MS frame in such a way that it could be adjustable in different vertical and horizontal positions for applying lateral loads. Capacities of the two jacks are 220 kN each and were duly calibrated before experiment.

Fig.3.14: Beam lateral loading arrangement

3.3.4 Location of the Dial Gauge

One dial gauge was set at the level of line of action of applied lateral load. Dial gauge was set such a manner that it could measure deflection in both directions. One small division in dial gauge is 0.001 inch. So, the conversion of dial gauge is .001 × 25.4=.0254 mm.

3.3.5 Mounting of the Dial Gauge

Frame made with MS angles (38mmx38mm) was fixed at base with RCC floor to hold dial gauge. It was ensured that the reading of dial gauge must be taken with respect to a stationary reference. Dial gauge was set (Fig.3.15 and Fig.3.16) with an extended small MS plate from specimen fixed with 12mm rowel bolts.

Fig.3.15: Dial gauge set up

Fig.3.16: Experimental setup before applying test load

3.4 TESTING OF SPECIMENS

Before testing, the specimens were checked dimensionally and detail visual inspection made with all information carefully recorded. After setting, the load was increased up to the failure of beam-column joint and deflection was recorded at each stage.

Four number of T-shaped RC beam-column specimens were tested by applying cyclic load by hydraulic jack machine of capacity 220 kN. Two of the specimens were as cast and two of the specimens were retrofitted with ferrocement jacket. Specimen marked as 1 and 4 were not jacketed, whereas specimen 2 and 3were jacketed by ferrocement. Reference specimens 1 and specimen 4 denoted RS1 and RS2 respectively. Ferrocement jacketed specimen 2 denoted JS1 and specimen 3 denoted JS2. Load of 229 kN was applied along column axis by hydraulic jack and remained constant throughout the experiment. Then cyclic load was applied at top of cantilever beam with a measured moment arm of 750 mm (±12mm).

For specimen, RS1and RS2 machine load was increased and released at uniform rate of 1 Ton (1000 kg) or 10 kN along both forward and reverse directions. Loading was applied gradually such as 1, 2, 3, 4, 5, and 6, 7 Ton respectively for forward direction and 1, 2, 3, 4, 5, 6, 7 respectively for reverse direction (Fig. 3.17). Calibrated load converted to SI unit (kN) was used for calculation.

For specimen, JS1 and specimen JS2 machine load was increased and released at uniform rate of 2 Ton (2000 kg) or 20 kN along both forward and reverse directions. Loading was applied gradually such as 2, 4, 6, 8 Ton respectively for forward direction and 2, 4, 6, 8 Ton respectively for reverse direction (Fig. 3.18). Calibrated load converted to SI unit (kN) was used for calculation.

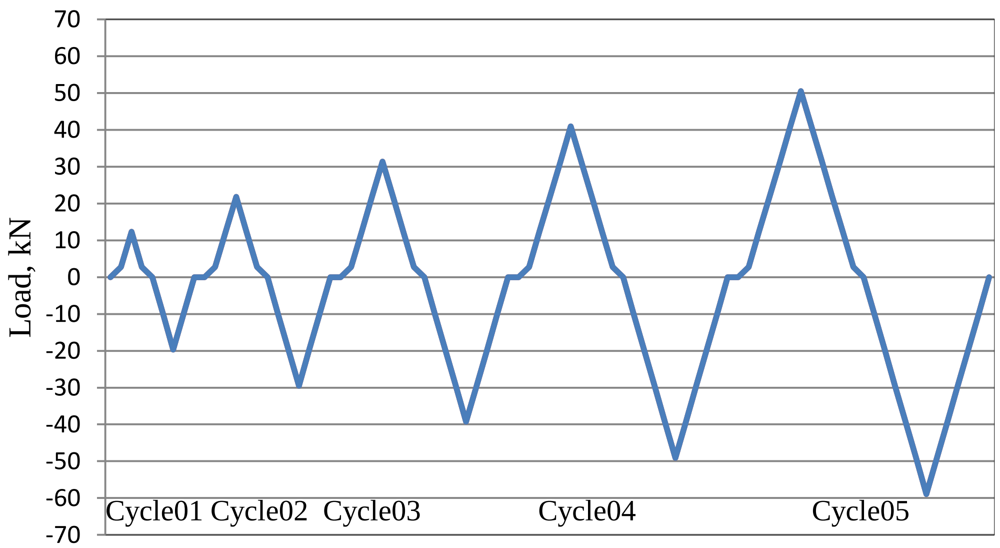


Fig.3.17: Loading Cycle for Reference Specimens RS1 and RS2

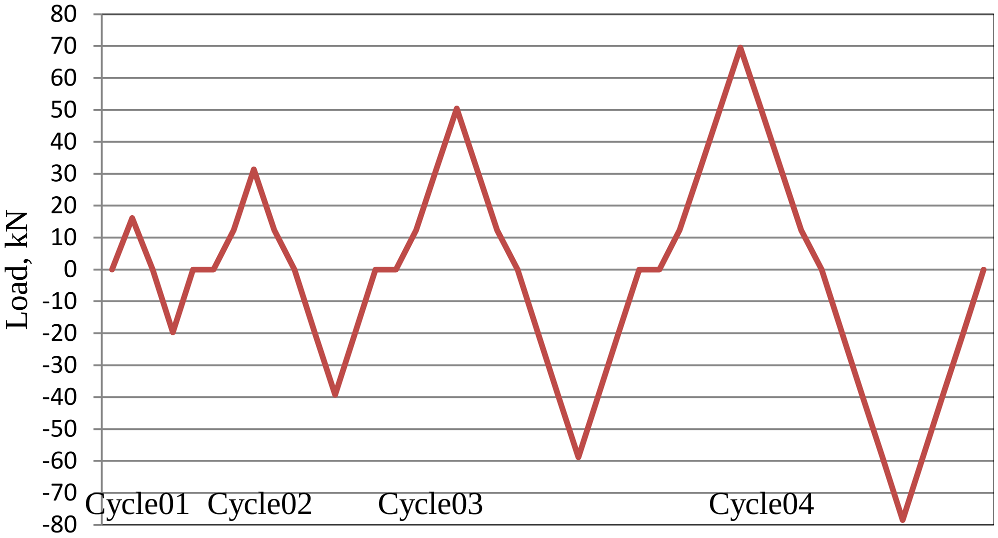


Fig.3.18: Loading Cycle for Ferocement Jacketed Specimens JS1 and JS2

Chapter 4
EXPERIMENTAL RESULTS AND DISCISSIONS

4.1 INTRODUCTION

In this chapter, the behavior of the reference specimens and ferrocement jacketed specimens are presented and discussed. During the testing deflections were measured by dial gauge and cracks were observed by naked eyes. Failure loads, cracks and deflections were observed for all frames.

4.2 VISUAL OBSERVATIONS

4.2.1 Reference Specimen RS1

Formation of the first crack at beam-column joint of the specimen RS1 occurred at a load of 22 kN during 2nd cycle of reverse direction. The cracks at the interface started widening at a load of 31 kN. Propagation of cracks developed with further increment of load and new cracks started to appear at zones adjacent to beam-column interface. Before failure by developing hinge at beam-column joint maximum deflection at beam tip was observed by 4.85 mm that was resulted from 40.92 kN load along forward direction. Complete failure happened by developing hinge at 50.47 kN in forward direction load. The application of load was stopped after gradual decrement from 50.47 kN to zero. Cracks are shown in Fig.4.1, Fig. 4.2 and 4.3.

Fig.4.1: Cracking type of failure in reference specimen RS1

Fig.4.2: Cracking type of failure in reference specimen RS1

Fig.4.3: After complete failure reference specimen RS1

4.2.2 Reference Specimen RS2

Formation of the first crack at beam-column joint of the specimen RS2occurred at a load of 22 kN during 2nd cycle reverse direction. Cracks in the interface started widening at a load of 31 kN. Propagation of cracks advanced with further increment of load and new cracks started to form at zones adjacent to beam-column interface. Before failure by developing hinge at beam-column joint maximum deflection was observed by 3.80 mm that was resulted from 49.80 kN load along reverse direction. Complete failure happened by developing hinge at 50.47 kN load in forward direction. The application of load was stopped after gradual decrement from 50.47 kN to zero. Cracks are shown in Fig.4.4 and Fig. 4.5.

Fig.4.4: Cracking type of failure in reference specimen RS2

Fig.4.5: After complete failure reference specimen RS2

4.2.3 Ferrocement Jacketed Specimen JS1

Formation of the first crack at beam-column joint of the specimen JS1occurred at a load of 39.30 kN during 3rd cycle reverse direction. The cracks in the interface started widening at a load of 40.92kN. Propagation of cracks developed with sequential increment of load and new cracks started to form at zones adjacent to beam-column interface. Before complete failure maximum deflection was observed at 50.47 kN load in reverse direction and corresponding value was recorded by 2.72 mm. Complete failure of the specimen occurred at 78.51 kN load along reverse direction. The application of load was stopped after gradual decrement from this load to zero. Cracks are shown in Fig.4.6 and Fig.4.7.

Fig.4.6: Cracking type of failure in ferrocement jacketed specimen JS1

Fig.4.7: After complete failure jacketed specimen JS1

4.2.4 Ferrocement Jacketed Specimen JS2

The first crack occurred at beam-column joint of JS2 at a load of 39.30 kN of 3rd cycle reverse direction. The cracks in the interface started widening at a load of 40.92 kN. Propagation of cracks developed with further increment of load and new cracks started to form at area adjacent to beam-column interface. Before complete failure maximum deflection was observed 2.98 mm resulted from 50.47 kN load in reverse direction. Complete failure of the specimen occurred at 78.51 kN load in reverse direction. The application of load was stopped after gradual decrement from this load to zero. Cracks are shown in Fig.4.8 and Fig. 4.9.

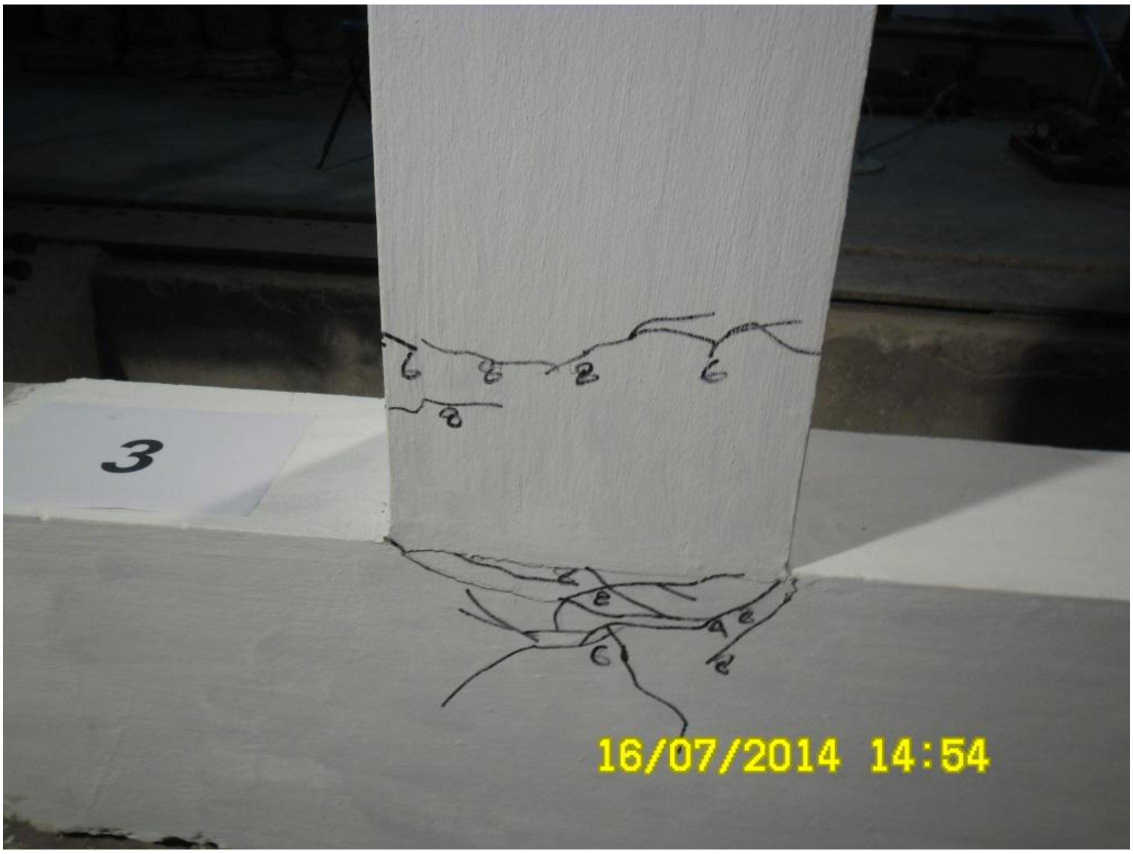


Fig.4.8: Cracking type of failure in ferrocement jacketed specimen JS2

Fig.4.9: After complete failure ferrocement jacketed specimen JS2

4.3 COMPARISON OF TEST RESULTS

4.3.1 Ultimate Load Carrying Capacity

Ultimate load carrying capacities of reference and ferrocement jacketed specimens are shown in Fig.4.10. It is found that ferrocement jacketed specimens JS1 and JS2 are having nearly 20% greater lateral load carrying capacity than that of reference specimens RS1 and RS2.

4.3.2 Maximum Deflection before Failure

Fig. 4.11 shows comparison of maximum deflections of retrofitted specimens with that of reference specimens. Deflections of beam measured at the same plane of lateral load applied were less for specimens that were jacketed by ferrocement overlay compared to that of reference specimens.

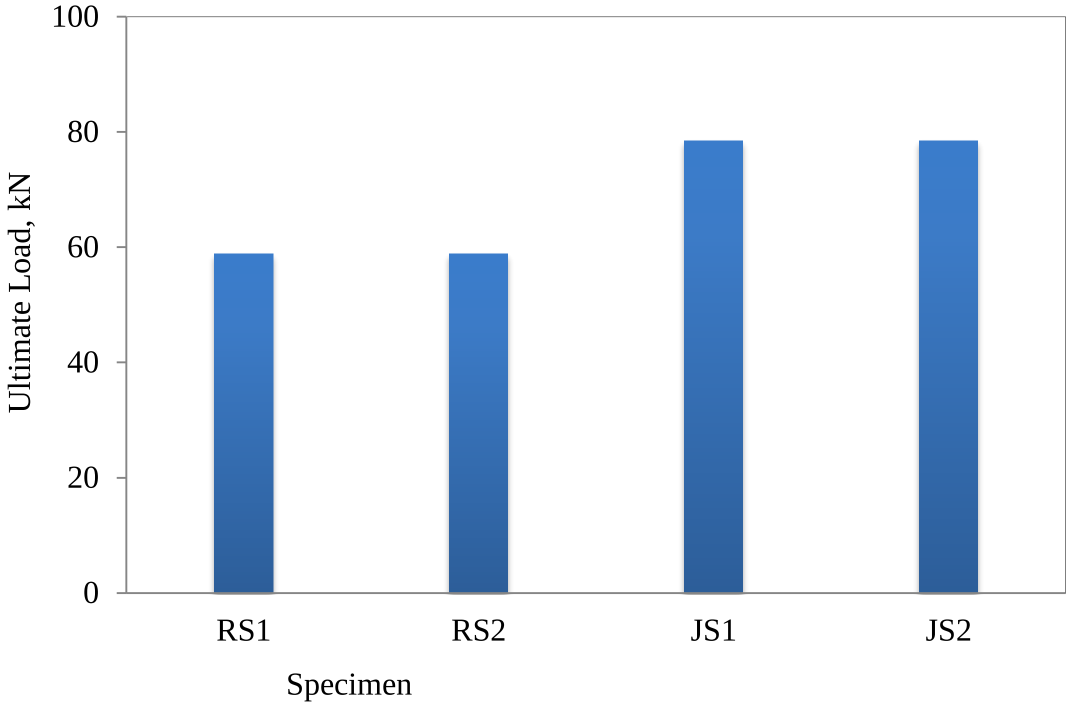


Fig.4.10: Comparison of ultimate lateral load carrying capacities of specimens

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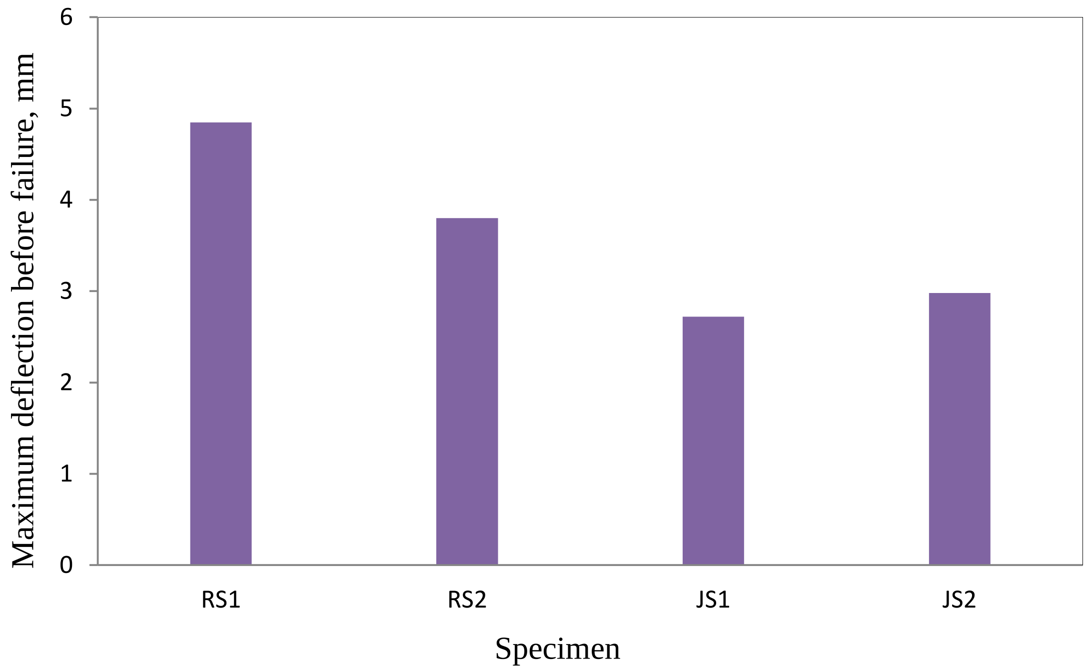

Fig.4: 11: Comparison of maximum deflection before failure

4.3.3 Hysteresis Loop

Hysteresis loop is used to measure the behavior of structure starting from elastic range until non-elastic range. It can be obtained by plotting the graph of load vs. displacement from loading and unloading branch to complete one cycle of movement. From hysteresis loops, parameters such as maximum lateral displacement, stiffness, ductility and equivalent viscous can be determined. Hysteresis loop of RS1 is shown in Fig. 4.12, RS2 is shown in Fig. 4.13, JS1 is shown in Fig.4.14 and JS2 is shown in Fig4.15. The stiffness of retrofitted beam-column joint increases and can withstand higher loading with smaller displacement as compared to non-retrofitted beam-column joint

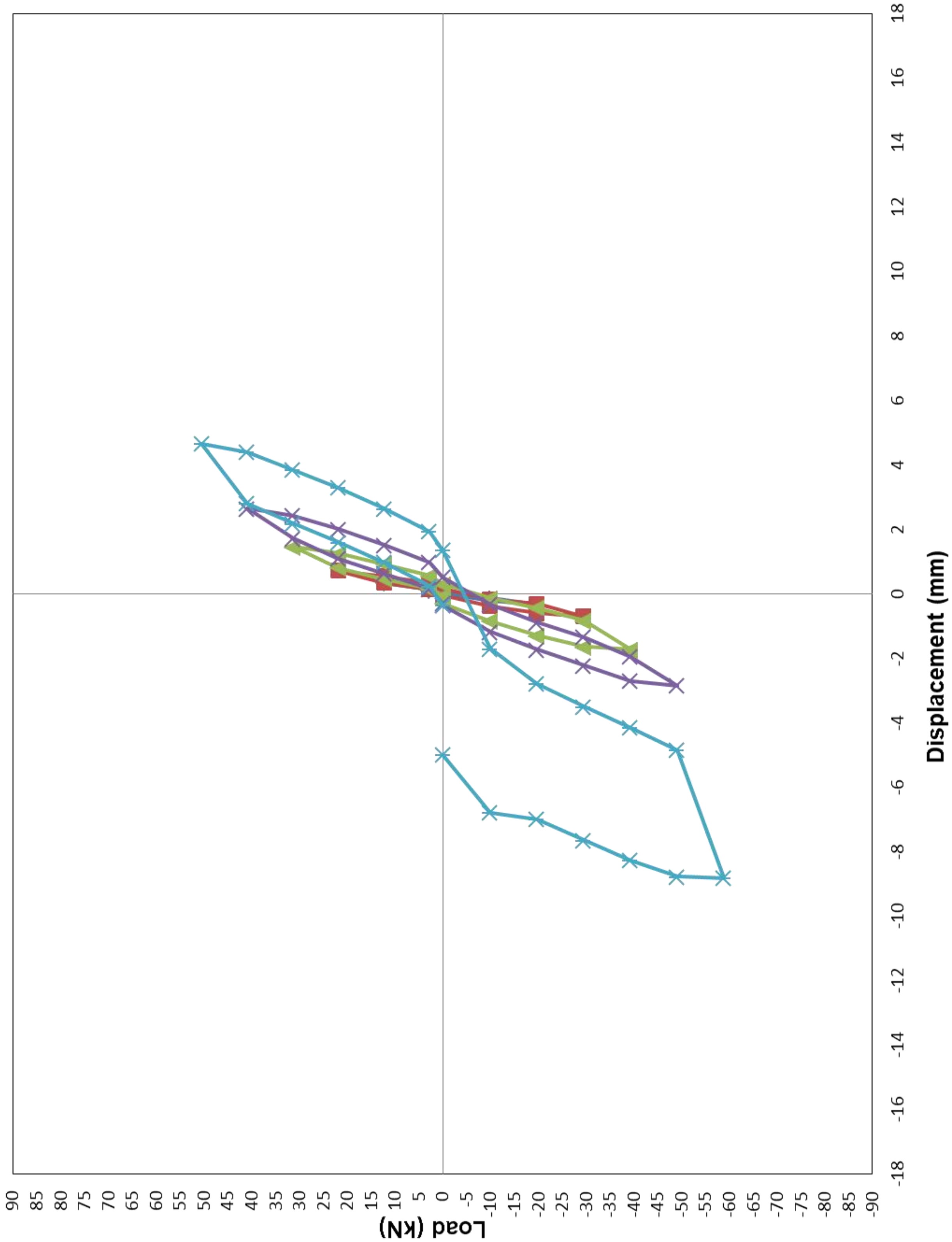

Fig.4.12: Load Vs. Deflection graph for reference specimen RS1

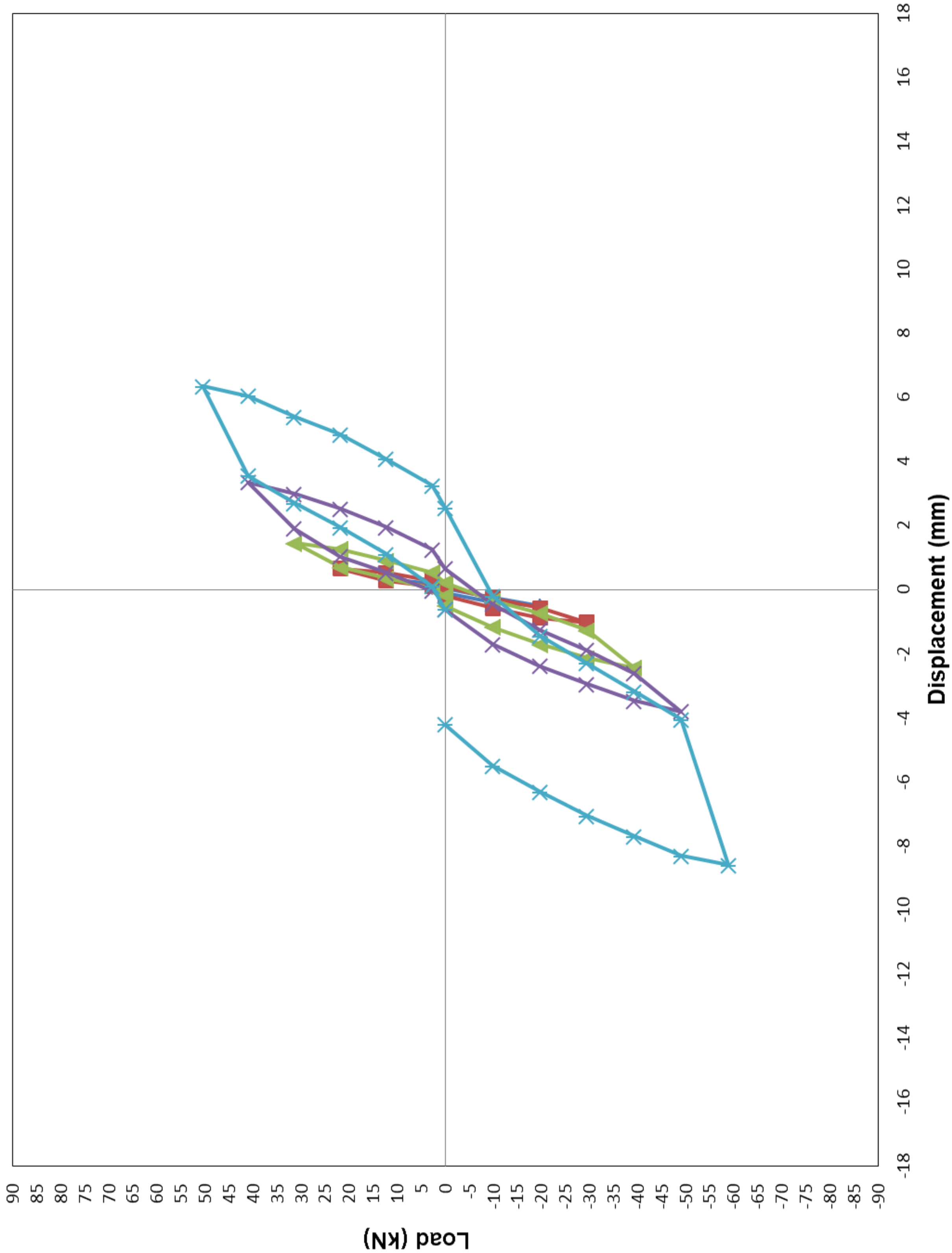

Fig.4.13: Load Vs. Deflection graph for reference specimen RS2

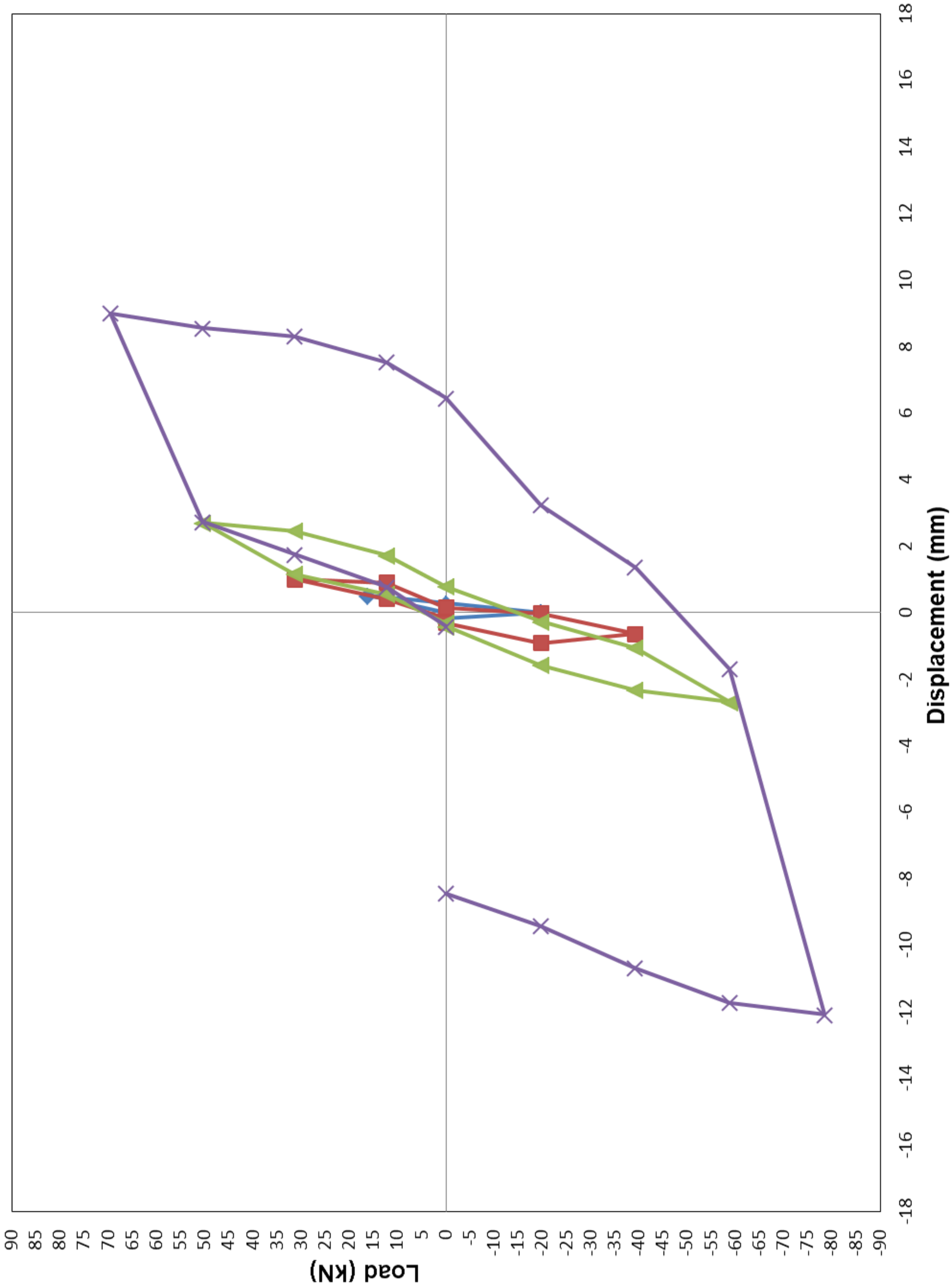

Fig.4.13: Load Vs. Deflection graph for ferrocement jacketed specimen JS1

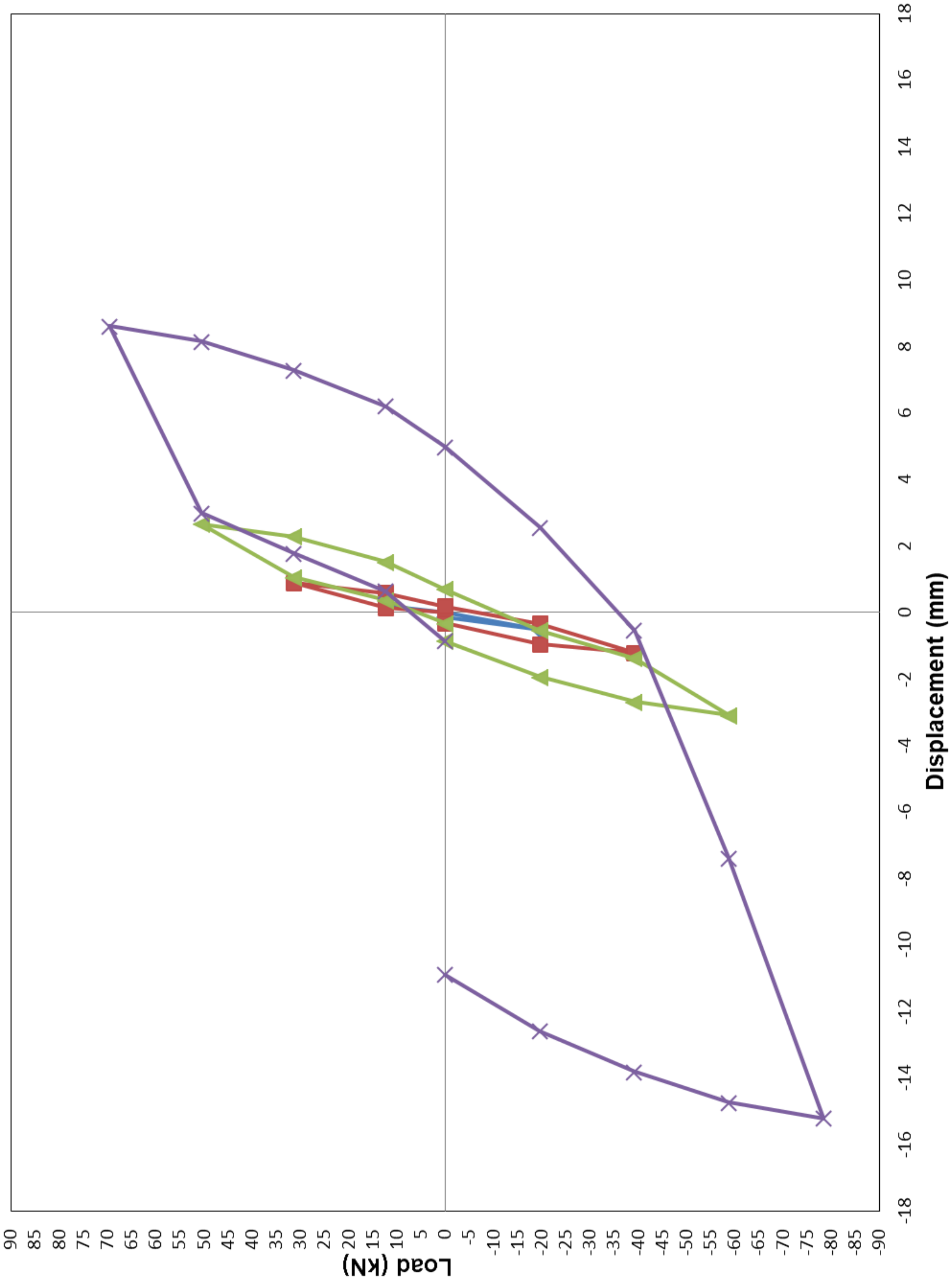

Fig.4.15: Load Vs. Deflection graph for ferrocement jacketed specimen JS2

4.3.4 Lateral Load Producing First Crack

Lateral load was applied at beam tip and beam-column joint was observed for cracks formed due to applied load. Specimens that were jacketed by ferrocement underwent higher lateral load than reference specimens before formation of first cracks. Fig. 4.16 shows comparison of loads producing first crack on specimen.

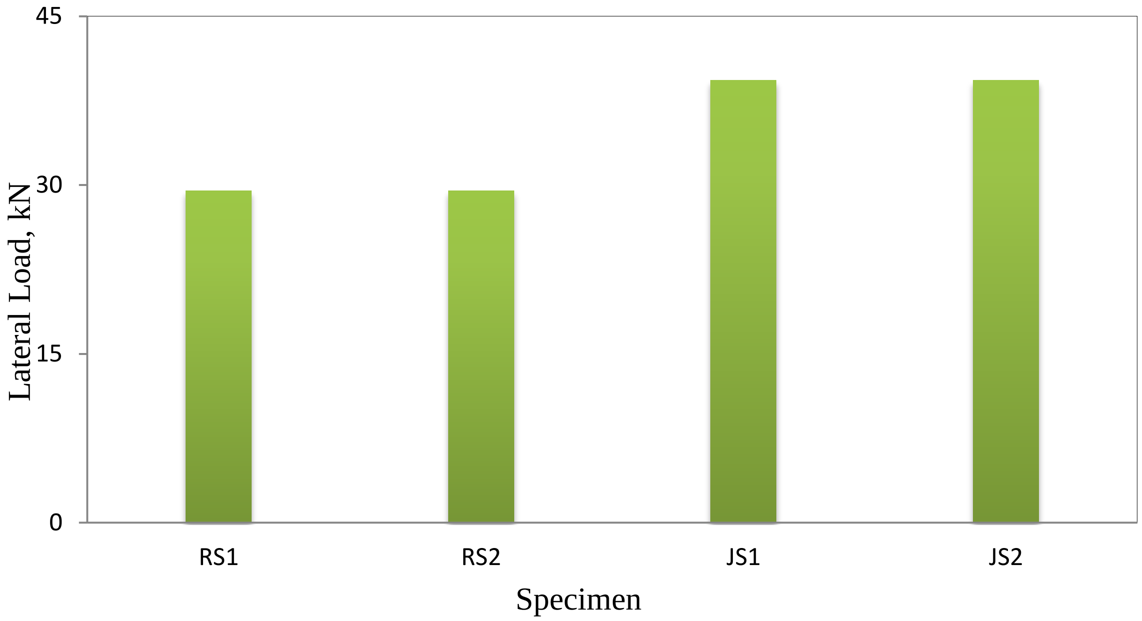

Fig.4.16: Comparison of loads producing first crack on specimens

4.3.5 Moment at Failure

Table 4.1: Comparison between theoretical moment carrying capacity (Under static load) and failure moment (under cyclic loading)

Specimen Label	Type of specimen	Calculated ultimate moment capacity, M_u, kN-m	Failure moment, kN-m (Under cyclic loading)	Difference in %
RS1	Without jacketing	32.30	44.17	36.75
RS2	Without jacketing	32.15	44.17	39.14
JS1	ferrocement Jacketed	31.67	52.16	62.23
JS2	ferrocement Jacketed	31.75	58.88	85.94

Note: For calculating moment carrying capacity contribution of ferrocement was not take into account. Though specimen size and shape were same, values of M_u are different due to different f'_c values.

Chapter 5
CONCLUSIONS AND RECOMMENDATIONS

5.1 CONCLUSIONS

The behavior of RC beam-column joint strengthened with ferrocement was investigated. Two RC beam-column joint specimens were jacketed with ferrocement and their performance was compared to that of reference specimens. The specimens were tested under cyclic loading . Findings, based on the experimental observations, are stated below:

- Ultimate moment carrying capacity of ferrocement jacketed beam-column joints was found to be about 20% greater than that of reference specimens.
- Before formation of first crack, the specimens jacketed with ferrocement overlay experienced about 33% more lateral load than the corresponding reference specimens.
- Maximum deflections before failure was found to be less for ferrocement jacketed specimens.
- Numbers of cracks were significantly fewer for specimens jacketed with ferrocement overlay than that of reference specimens.
- Hysteresis loops showed higher ductility for jacketed specimens than that of reference specimens.

5.2 RECOMMENDATIONS FOR FURTHER STUDY

1. More frames should be tested to obtain statistically significant results.
2. Effect of mesh layer in strengthening scheme could be studied for beam-column joint.
3. Different types of wire meshes other than GI wire mesh can be used for testing in future studies.
4. Finite element based comprehensive parametric study based on current experimental data can be performed to study more complex joints.

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APPENDIX-A

A.1: Sieve analysis of coarse aggregate

Sieve opening	Retained (gm)	% Mass retained	%Cumulative retained	% Passing
12.55 mm	5650	58.670	58.670	41.33
9.5 mm	3120	32.398	91.068	8.93
Pan	860	8.930	99.99	0
Total	496.1	99.99		

A.2: Sieve analysis of fine aggregate

Sieve opening	Retained (gm)	% Mass retained	%Cumulative retained	% Passing
2.36 mm	19.9	4.011	4.011	95.98
1.18 mm	114.2	23.02	27.031	72.97
600 µm	191.4	18.42	45.61	34.39
300 µm	131	46.56	92.011	7.989
150 µm	30.8	6.208	98.219	1.781
Pan	8.8	1.612	0	0
Total	496.1	99.83	322.811	

APPENDIX-B

Tensile properties of steel reinforcements (used in beam and column)

Sl No.	Nominal Bar Size (mm)	Unit weight (kg/m)	Yield strength (MPa)	Ultimate Strength (MPa)	Elongation
1	16 mm	1.583	373	552	24%
2	6 mm	0.311	155	166	19%

APPENDIX-C

C.1: Concrete cylinder test result for RS1, at the date of experiment

Cell Load (kN)	Age	Calibrated Load (kN)	Cylinder Dia.(m)	X-Sectional Area(sq.m)	Compressive strength, f'c(MPa)	Average f'c (Mpa)
331	645 Days	330.222	0.10125	0.0081	41.01	37.22
292		291.261	0.10125	0.0081	36.17	
278.4		277.6746	0.10125	0.0081	34.49	

C.2: Concrete cylinder test result for RS2, at the date of experiment

Cell Load (kN)	Age	Calibrated Load (kN)	Cylinder Dia(m)	X-Sectional Area(sq.m)	Compressive strength, f'c (MPa)	Average f'c (Mpa)
220	644 days	219.333	0.10125	0.0081	27.24	25.88
197		196.356	0.10125	0.0081	24.39	
210		209.343	0.10125	0.0081	26.00	

C.3: Concrete cylinder test result for JS1, at the date of experiment

Cell Load (kN)	Age	Calibrated Load (kN)	Cylinder Dia(m)	X-Sectional Area(sq.m)	Compressive strength, f'c (MPa)	Average f'c (Mpa)
292	635 days	291.944	0.10125	0.0081	36.26	33.28
252		252.024	0.10125	0.0081	31.30	
260		260.008	0.10125	0.0081	32.29	

C.4: Concrete cylinder test result for JS2, at the date of experiment

Cell Load (kN)	Age	Calibrated Load (kN)	Cylinder Dia.(m)	X-Sectional Area(sq.m)	Compressive strength, f'c (MPa)	Average f'c (Mpa)
192	641 days	192.144	0.10125	0.0081	23.86	24.77
216		216.096	0.10125	0.0081	26.84	
190		190.148	0.10125	0.0081	23.62	

APPENDIX-D

Concrete mortar test result for ferrocement plaster

Cell Load (kN)	Age	Calibrated Load (kN)	Cube size (mm^3)	X-Sectional Area(sq.m)	Compressive strength, f'c (MPa)	Average f'c (Mpa)
162.73	641 days	156.19173	50x50x50	2.50E-03	62.48	59.24
135.6		129.0346		2.50E-03	51.61	
165.63		159.09463		2.50E-03	63.64	

APPENDIX-E

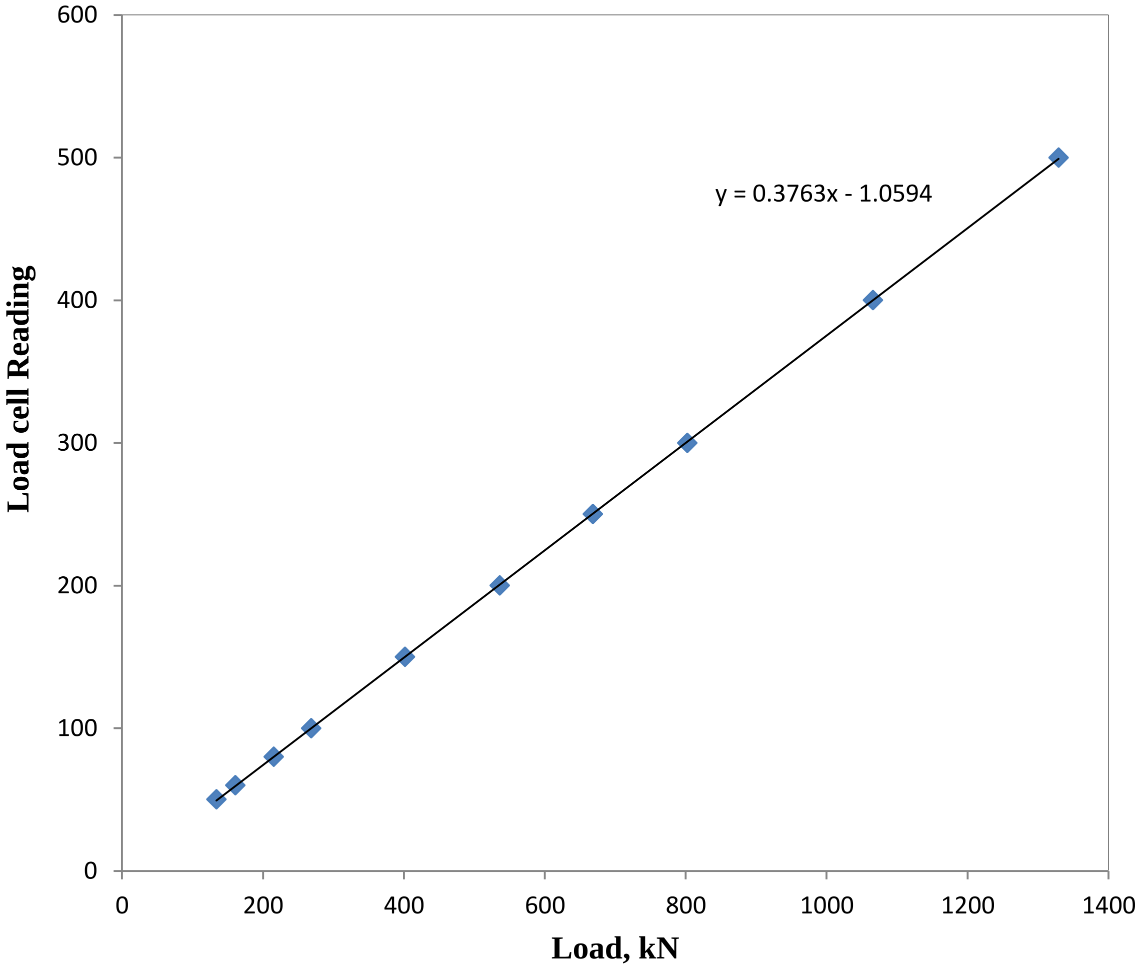

Fig. E.1: Load column graph used for Hydraulic Jacks calibration

APPENDIX-E

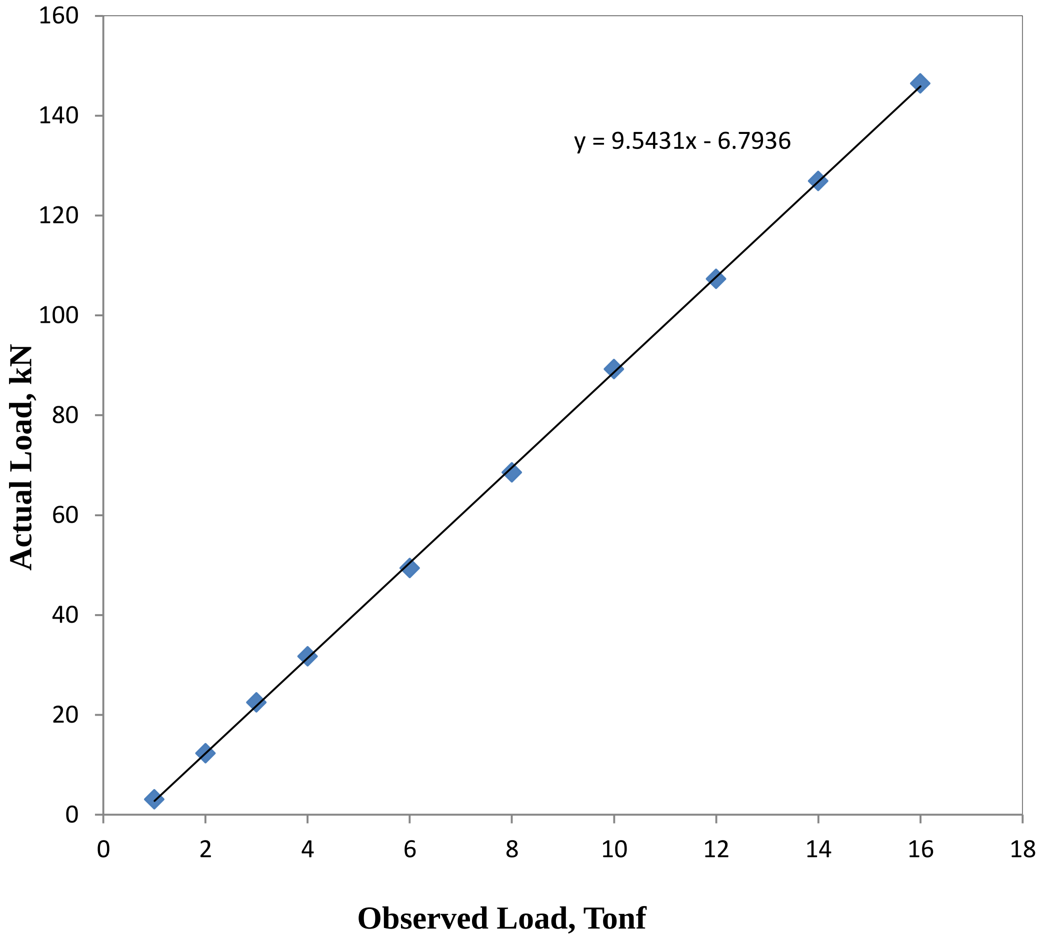

Fig. E.2: Calibration graph of Hydraulic Jack3, used for applying forward Loading

APPENDIX-E

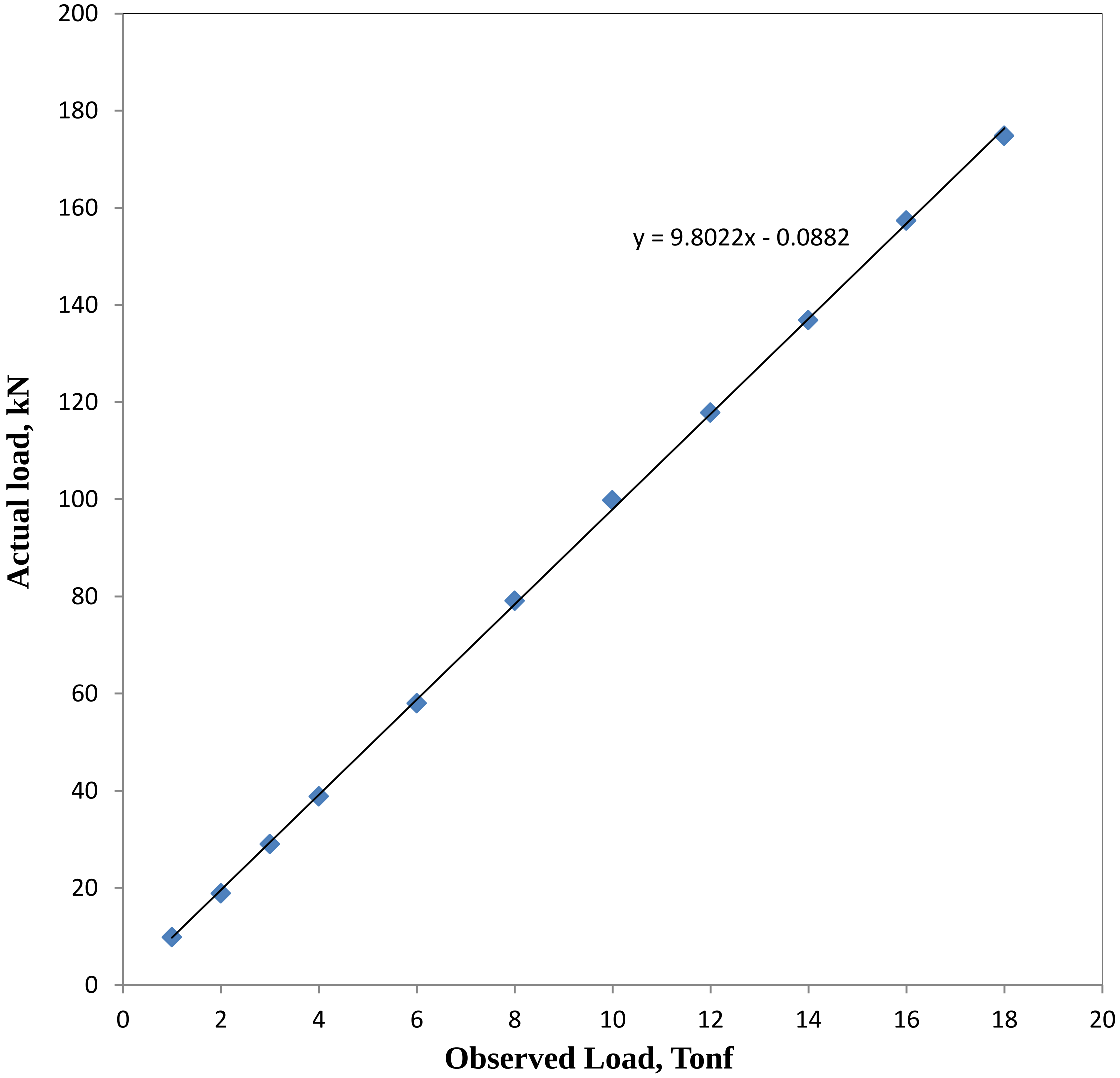

Fig. E.3: Calibration graph of Hydraulic Jack4, used for applying reversal loading

APPENDIX-E

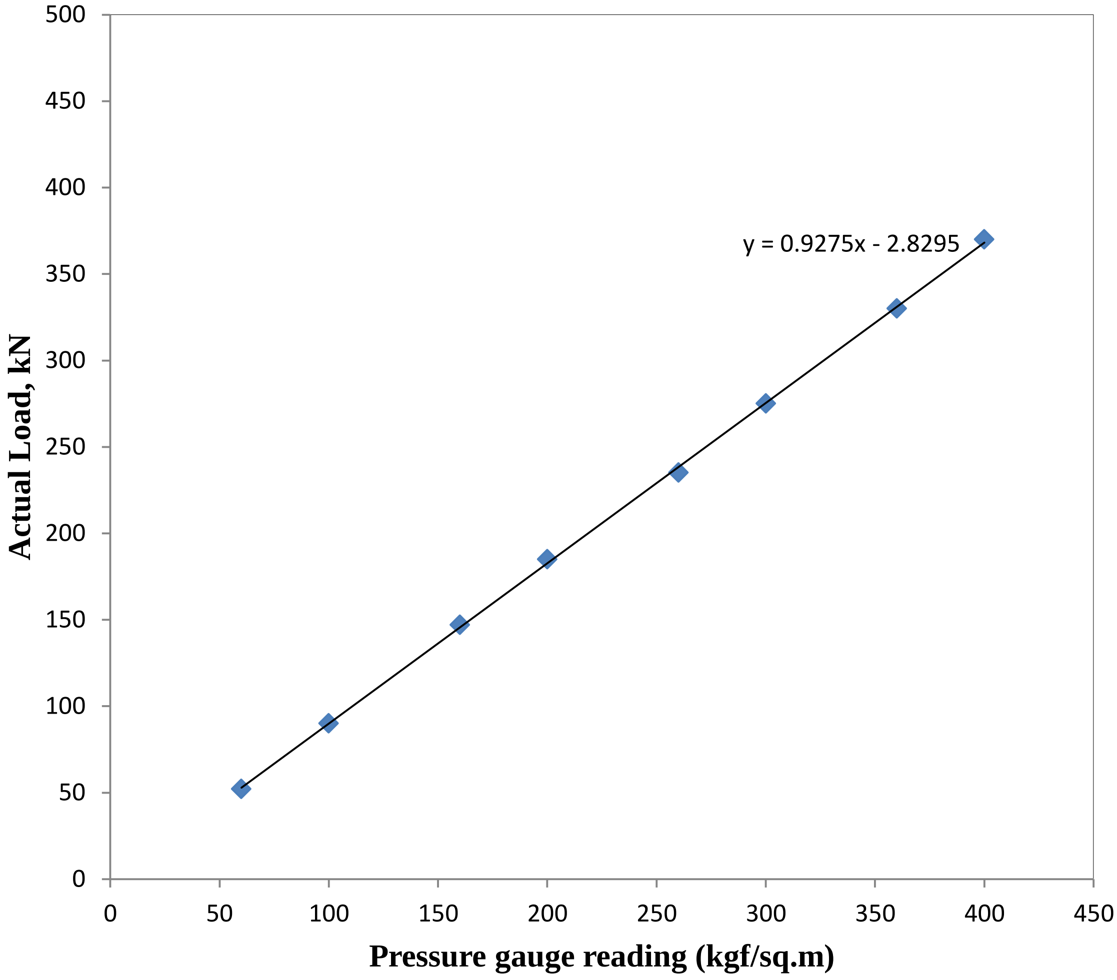

Fig. E.4: Calibration graph of Hydraulic Jack, used for column axial load

APPENDIX-F

Calculated moment carrying capacities of beams, kN-m (under static loading)

Specimen	Beam width, mm	Beam height, mm	effective depth, mm	Concrete comp. strength, f'_c (MPa)	Steel Area, sq.mm	Steel, f_y, (MPa)	Stress block, a, mm	Ultimate moment carrying capacity, M_u, kN-m
RS1	250	300	262	37.22	400	373	18.864	32.30
JS1	250	300	262	33.28	400	373	21.0973	32.15
JS2	250	300	262	24.77	400	373	28.3455	31.67
RS2	250	300	262	25.88	400	373	27.1297	31.75

APPENDIX-G

Table G.1: Data sheet for RS1

Load Cycle	Load Direction	Applied lateral load (kN)	Dial gauge reading	Deflection, mm
Cycle01	FL	0.00	0	0
	FL	2.75	12	0.12
	FL	12.29	34	0.34
	FU	2.75	22	0.22
	FU	0.00	5	0.05
	RL	-9.89	-14	-0.14
	RL	-19.69	-34	-0.34
	RU	-9.89	-22	-0.22
	RU	0.00	0	0
Cycle02	FL	0.00	0	0
	FL	2.75	14	0.14
	FL	12.29	34	0.34
	FL	21.84	71	0.71
	FU	12.29	54	0.54
	FU	2.75	34	0.34
	FU	0.00	15	0.15
	RL	-9.89	-18	-0.18
	RL	-19.69	-32	-0.32
	RL	-29.49	-70	-0.7
	RU	-19.69	-61	-0.61
	RU	-9.89	-39	-0.39
	RU	0.00	-5	-0.05
Cycle03	FL	0.00	-5	-0.05
	FL	2.75	15	0.15
	FL	12.29	45	0.45
	FL	21.84	81	0.81
	FL	31.38	145	1.45
	FU	21.84	125	1.25
	FU	12.29	92	0.92
	FU	2.75	56	0.56
	FU	0.00	30	0.3
	RL	-9.89	-14	-0.14
	RL	-19.69	-45	-0.45
	RL	-29.49	-84	-0.84
	RL	-39.30	-174	-1.74
	RU	-29.49	-165	-1.65
	RU	-19.69	-130	-1.3
	RU	-9.89	-85	-0.85
	RU	0.00	-30	-0.3

Note: FL: Forward Loading, FU: Forward unloading,
RL: Reverse loading, RU: Reverse unloading

Table G.1: Data sheet for RS1

Load Cycle	Load Direction	Applied lateral load (kN)	Dial gauge reading	Deflection, mm
Cycle04	FL	0.00	-30	-0.3
	FL	2.75	15	0.15
	FL	12.29	63	0.63
	FL	21.84	110	1.1
	FL	31.38	172	1.72
	FL	40.92	265	2.65
	FU	31.38	242	2.42
	FU	21.84	200	2
	FU	12.29	152	1.52
	FU	2.75	97	0.97
	FU	0.00	51	0.51
	RL	-9.89	-33	-0.33
	RL	-19.69	-90	-0.9
	RL	-29.49	-135	-1.35
	RL	-39.30	-195	-1.95
	RL	-49.10	-285	-2.85
	RU	-39.30	-270	-2.7
	RU	-29.49	-223	-2.23
	RU	-19.69	-174	-1.74
	RU	-9.89	-118	-1.18
	RU	0.00	-36	-0.36
Cycle05	FL	0.00	-36	-0.36
	FL	2.75	24	0.24
	FL	12.29	98	0.98
	FL	21.84	160	1.6
	FL	31.38	218	2.18
	FL	40.92	281	2.81
	FL	50.47	466	4.66
	FU	40.92	440	4.4
	FU	31.38	385	3.85
	FU	21.84	330	3.3
	FU	12.29	265	2.65
	FU	2.75	193	1.93
	FU	0.00	135	1.35
	RL	-9.89	-170	-1.7
	RL	-19.69	-280	-2.8
	RL	-29.49	-350	-3.5
	RL	-39.30	-415	-4.15
	RL	-49.10	-485	-4.85
	RL	-58.90	-883	-8.83
	RU	-49.10	-878	-8.78
	RU	-39.30	-828	-8.28
	RU	-29.49	-765	-7.65
	RU	-19.69	-700	-7
	RU	-9.89	-680	-6.8
	RU	0.00	-500	-5

APPENDIX-G

Table G.1: Data sheet for RS1

Load Cycle	Load Direction	Applied lateral Load (kN)	Dial gauge reading	Deflection, mm
Cycle06	FL	0.00	-500	-5
	FL	2.75	-385	-3.85
	FL	12.29	-122	-1.22
	FL	21.84	40	0.4
	FL	31.38	175	1.75
	FL	40.92	384	3.84
	FL	50.47	1628	16.28
	FU	40.92	1510	15.1
	FU	31.38	1480	14.8
	FU	21.84	1310	13.1
	FU	12.29	1215	12.15
	FU	2.75	1120	11.2
	FU	0.00	1030	10.3

Note: FL: Forward Loading, FU: Forward unloading,
RL: Reverse loading, RU: Reverse unloading

APPENDIX-G

Table G.2: Data sheet for specimen JS1

Load Cycle	Load Direction	Applied lateral load (kN)	Dial gauge reading	Deflection, mm
Cycle01	FL	0.00	0	0
	FL	16.11	50	0.5
	FU	0.00	27	0.27
	RL	-19.69	0	0
	RU	0.00	-18	-0.18
Cycle02	FL	0.00	-18	-0.18
	FL	12.29	40	0.4
	FL	31.38	99	0.99
	FU	12.29	89	0.89
	FU	0.00	15	0.15
	RL	-19.69	-3	-0.03
	RL	-39.30	-65	-0.65
	RU	-19.69	-92	-0.92
	RU	0.00	-32	-0.32
Cycle03	FL	0.00	-32	-0.32
	FL	12.29	55	0.55
	FL	31.38	115	1.15
	FL	50.47	270	2.7
	FU	31.38	245	2.45
	FU	12.29	172	1.72
	FU	0.00	78	0.78
	RL	-19.69	-27	-0.27
	RL	-39.30	-106	-1.06
	RL	-58.90	-270	-2.7
	RU	-39.30	-233	-2.33
	RU	-19.69	-159	-1.59
	RU	0.00	-42	-0.42

Note: FL: Forward Loading, FU: Forward unloading,
RL: Reverse loading, RU: Reverse unloading

APPENDIX-G

Table G.2: Data sheet for JS1

Load Cycle	Load Direction	Applied lateral load (kN)	Dial gauge reading	Deflection, mm
Cycle04	FL	0.00	-42	-0.42
	FL	12.29	76	0.76
	FL	31.38	174	1.74
	FL	50.47	272	2.72
	FL	69.55	900	9
	FU	50.47	855	8.55
	FU	31.38	830	8.3
	FU	12.29	752	7.52
	FU	0.00	645	6.45
	RL	-19.69	323	3.23
	RL	-39.30	136	1.36
	RL	-58.90	-170	-1.7
	RL	-78.51	-1209	-12.09
	RU	-58.90	-1173	-11.73
Cycle05	FL	0.00	-845	-8.45
	FL	12.29	162	1.62
	FL	31.38	600	6
	FL	50.47	780	7.8
	FL	69.55	1980	19.8
	FL	88.64	3000	30
	FU	69.55	2990	29.9
	FU	50.47	2791	27.91
	FU	31.38	2575	25.75
	FU	12.29	2330	23.3
	FU	0.00	2145	21.45
	RL	-19.69	1500	15
	RL	-39.30	470	4.7
	RL	-58.90	-1080	-10.8
	RL	-78.51	-5000	-50
	RL	-98.11	-5000	-50
	RU	-78.51	-5000	-50
	RU	-58.90	-1385	-13.85
	RU	-39.30	-1122	-11.22
	RU	-19.69	-435	-4.35
	RU	0.00	-400	-4

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Table G.3: Data sheet for JS2

Load Cycle	Load Direction	Applied lateral load (kN)	Dial gauge reading	Deflection, mm
Cycle01	FL	0.00	0	0
	FL	12.29	20	0.2
	FU	0.00	0	0
	RL	-19.69	-52	-0.52
	RU	0.00	-14	-0.14
Cycle02	FL	0.00	-14	0
	FL	12.29	13	0.13
	FL	31.38	88	0.88
	FU	12.29	58	0.58
	FU	0.00	18	0.18
	RL	-19.69	-35	-0.35
	RL	-39.30	-123	-1.23
	RU	-19.69	-96	-0.96
	RU	0.00	-33	-0.33
Cycle03	FL	0.00	-33	-0.33
	FL	12.29	36	0.36
	FL	31.38	105	1.05
	FL	50.47	265	2.65
	FU	31.38	228	2.28
	FU	12.29	151	1.51
	FU	0.00	69	0.69
	RL	-19.69	-55	-0.55
	RL	-39.30	-139	-1.39
	RL	-58.90	-310	-3.1
	RU	-39.30	-269	-2.69
	RU	-19.69	-195	-1.95
	RU	0.00	-86	-0.86

Note: FL: Forward Loading, FU: Forward unloading,
RL: Reverse loading, RU: Reverse unloading

APPENDIX-G

Table G.3: Data sheet for JS2

Load Cycle	Load Direction	Applied lateral load (kN)	Dial gauge reading	Deflection, mm
Cycle04	FL	0.00	-86	-0.86
	FL	12.29	64	0.64
	FL	31.38	178	1.78
	FL	50.47	298	2.98
	FL	69.55	860	8.6
	FU	50.47	815	8.15
	FU	31.38	728	7.28
	FU	12.29	620	6.2
	FU	0.00	497	4.97
	RL	-19.69	255	2.55
	RL	-39.30	-55	-0.55
	RL	-58.90	-740	-7.4
	RL	-78.51	-1522	-15.22
	RU	-58.90	-1475	-14.75
	RU	-39.30	-1382	-13.82
	RU	-19.69	-1260	-12.6
	RU	0.00	-1090	-10.9

Note: FL: Forward Loading, FU: Forward unloading,
RL: Reverse loading, RU: Reverse unloading

APPENDIX-G

Table G.4: Data sheet for RS2

Load Cycle	Load Direction	Applied lateral load (kN)	Dial gauge reading	Deflection, mm
Cycle01	FL	0.00	0	0
	FL	2.75	12	0.12
	FL	12.29	34	0.34
	FU	2.75	20	0.2
	FU	0.00	5	0.05
	RL	-9.89	-25	-0.25
	RL	-19.69	-52	-0.52
	RU	-9.89	-40	-0.4
	RU	0.00	-10	-0.1
Cycle02	FL	0.00	-10	0
	FL	2.75	10	0.1
	FL	12.29	28	0.28
	FL	21.84	66	0.66
	FU	12.29	52	0.52
	FU	2.75	28	0.28
	FU	0.00	9	0.09
	RL	-9.89	-27	-0.27
	RL	-19.69	-57	-0.57
	RL	-29.49	-103	-1.03
	RU	-19.69	-88	-0.88
	RU	-9.89	-58	-0.58
	RU	0.00	-18	-0.18
Cycle03	FL	0.00	-18	-0.18
	FL	2.75	4	0.04
	FL	12.29	36	0.36
	FL	21.84	70	0.7
	FL	31.38	145	1.45
	FU	21.84	126	1.26
	FU	12.29	90	0.9
	FU	2.75	54	0.54
	FU	0.00	22	0.22
	RL	-9.89	-35	-0.35
	RL	-19.69	-74	-0.74
	RL	-29.49	-127	-1.27
	RL	-39.30	-245	-2.45
	RU	-29.49	-214	-2.14
	RU	-19.69	-170	-1.7
	RU	-9.89	-117	-1.17
	RU	0.00	-50	-0.5

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TableG.4: Data sheet for RS2

Load Cycle	Load Direction	Calibrated Load (kN)	Dial gauge reading	Deflection, mm
Cycle04	FL	0.00	-50	-0.5
	FL	2.75	-4	-0.04
	FL	12.29	53	0.53
	FL	21.84	103	1.03
	FL	31.38	191	1.91
	FL	40.92	335	3.35
	FU	31.38	299	2.99
	FU	21.84	252	2.52
	FU	12.29	194	1.94
	FU	2.75	125	1.25
	FU	0.00	66	0.66
	RL	-9.89	-44	-0.44
	RL	-19.69	-125	-1.25
	RL	-29.49	-190	-1.9
	RL	-39.30	-262	-2.62
	RL	-49.10	-380	-3.8
	RU	-39.30	-348	-3.48
	RU	-29.49	-295	-2.95
	RU	-19.69	-238	-2.38
	RU	-9.89	-171	-1.71
	RU	0.00	-62	-0.62

Note: FL: Forward Loading, FU: Forward unloading,
RL: Reverse loading, RU: Reverse unloading

APPENDIX-G

TableG.4: Data sheet for RS2

Load Cycle	Load Direction	Applied lateral load (kN)	Dial gauge reading	Deflection, mm
Cycle05	FL	0.00	-62	-0.62
	FL	2.75	10	0.1
	FL	12.29	111	1.11
	FL	21.84	194	1.94
	FL	31.38	270	2.7
	FL	40.92	355	3.55
	FL	50.47	635	6.35
	FU	40.92	605	6.05
	FU	31.38	540	5.4
	FU	21.84	485	4.85
	FU	12.29	408	4.08
	FU	2.75	325	3.25
	FU	0.00	255	2.55
	RL	-9.89	-20	-0.2
	RL	-19.69	-145	-1.45
	RL	-29.49	-230	-2.3
	RL	-39.30	-318	-3.18
	RL	-49.10	-405	-4.05
	RL	-58.90	-860	-8.6
	RU	-49.10	-832	-8.32
	RU	-39.30	-770	-7.7
	RU	-29.49	-707	-7.07
	RU	-19.69	-632	-6.32
	RU	-9.89	-550	-5.5
	RU	0.00	-421	-4.21

Note: FL: Forward Loading, FU: Forward unloading,
RL: Reverse loading, RU: Reverse unloading