# Experimental Study on Seismic Performance of Precast Beam Column Connection Using Steel Fiber

**By**

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Certified that the contents and form of thesis titled **“Experimental Study on seismic performance of precast beam column Connection using Steel fiber”** submitted by Ahsin Mehmood have been found satisfactory for the requirement of the degree.

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## DEDICATION

*Dedicated to my beloved mother, father, brothers and the rest of the family and friends who have stood behind me till the accomplishment of this research.*

## ACKNOWLEDGEMENT

I am thankful to ALMIGHTY ALLAH, who gave the opportunity and enabled me to complete my master's degree. Firstly, I would like to express my profound gratitude to my supervisor, Associate Professor, Col. Dr Adeel Zafar for their guidance, unceasing support and motivation during all phases of conducting this research work. It has been an absolute privilege for me to work under their kind supervision. They were always open to discussions, encouraged innovative ideas and accomplishments and steered me in the right direction whenever they deemed necessary.

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## ABSTRACT

This study proposes a unique type of precast beam-column joint with a dry and wet connection that comprises 5D-Dramix. Special connectors, such as double grouted sleeves, are used to join the column and joint elements. The dry connection is quicker to put together than the wet kind, which can substantially limit the extent of manpower required on the construction site. In addition, it is simple to disassemble, making it possible to upgrade components and replace any that have been damaged by earthquakes during the course of the building's lifetime. It promotes industry and increases the building's resistance to earthquakes. It is ideal for areas that experience significant seismic activity and for building. A precast joint model and a cast-in-situ joint model were built, and pseudo-static tests were carried in order to assess the seismic performance of the dry connection and wet connection. According to the test results, the precast joint's carrying capacity is nearly similar to that of the cast-in-situ joint. Strong column and weak beam is the basis for the precast joint's failure mode, and beam damage is seen in the plastic hinge zone.

**Keywords**: Pre-cast Reinforced concrete structures; Dry and wet connection; Steel Fiber reinforced concrete; Grouted Sleeves; Ductile Connection; Pseudo-static.

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## LIST OF SYMBOLS AND ABBREVIATIONS

| Symbol | Description |
|---|---|
| SFC | Steel Fiber Reinforced Concrete |
| $f'_c$ | concrete compressive cylinder strength |
| OPC | Ordinary Portland Cement |
| C% | Cement Percentage by Weight |
| W/B | Water to Binder ratio |
| $A_g$ | gross area of column section |
| $F_{ib}$% | Fibre Amount percentage by weight |
| d | distance from extreme compression fiber |
| p | ratio of area of the top beam longitudinal |
| $f_h$ | the average axial stress in horizontal direction |
| $\varepsilon_y$ | steel yield strain |
| $d_b$ | diameter of longitudinal steel |
| s | spacing of transverse reinforcement |
| $\varphi$ | the strength reduction factor, being unity here |
| $h_b$ | beam depth |
| $\varphi_y$ | yield curvature |
| $V_{jh}$ | the imposed horizontal joint shear force |
| A | effective joint area |
| $\Delta$ | estimated storey displacement |
| $\delta_n$ | measurement of the left side of the joint panel |
| $l_f$ | the length of steel fibres |
| vjc | the joint shear stress carried by concrete |
| $h_b$ | beam depth |
| δs | measurement of the right side of the joint panel |
| δt | measurement of the top joint panel |
| δb | measurement of the bottom joint panel |
| $V_F$ | the volume content of steel fibres |
| vjc | the joint shear stress carried by concrete |
| *vjs* | the joint shear stress carried by stirrups |

# *CHAPTER 1*

# INTRODUCTION

Researchers and project managers are striving to decrease the time required for building associated with concrete technology across the world. The project's cost, which is directly determined by how quickly it is built, has been reduced by the allocation of several resources and efforts. Precast concrete structural elements, such as beams, columns, wall panels, and slab units, that are manufactured at a plant, brought to construction sites, raised, and put in place, have come under increased attention during the past two to three decades. Precast concrete construction is growing in popularity in several nations because to its obvious benefits, including time and money savings and quality guarantee. However, because of connectivity problems, its deployment in seismically active areas has been questioned [1]. Numerous studies on precast framed structures that are earthquake resistant have been performed, indicating the need to evaluate precast buildings' seismic performance [2].

Finding a cost-effective and practical way to join the precast parts together so that the seismic performance is as excellent as for monolithic structures is the main problem in the design of building structures utilizing precast concrete elements for earthquake resistance. The behavior of the connecting region differs significantly from that of monolithic cast-in-place buildings if the connection between the prefabricated structure and the critical zone, such as a potential plastic hinge zone, is positioned there. The concept of a strong connection and weak segment is utilised in earthquake engineering due to the fact that the strength of the beam-column connection is of the utmost importance in frame construction. Beam-to-column connections such monolithic bolting, emulative bolting, dry pinning, and dry nailing are commonly utilized in practice. Variety of tests have been carried out to assess the joint's seismic performance at various scales, but they have revealed a lack of ductility. Due to inadequate transverse reinforcement in the joints and poor anchorage of the primary reinforcing bars, the majority of these joints display brittle failures.

The study investigated the competitiveness of precast concrete structures in comparison to identical monolithic reinforced concrete structures by concentrating on the creation of structural connections. Better-performing concrete structures are provided by precast beam-column connections that use jointed systems technology. The performance of jointed precast connectors under cyclic and dynamic loads is the focus of this investigation. Steel fibre reinforced concrete (SFRC) is being used in the project to strengthen the pre-cast beam-column joints.

### 1.1. The Joint Vulnerability Problem

Joint inadequacies are mostly brought on by insufficient anchoring capacity and inadequate transverse reinforcement. The damages shown in the most recent, extremely destructive earthquakes that occurred in many nations has bringing these issues into focus. Recent earthquakes, like the one that struck Emilia, Northern Italy, in 2012, have provided evidence that the capacity of a beam was lost as a result of the concrete's inadequate grip on the steel bars, as illustrated in Figure 1.1.

Figure 1.1. 2012 Emilia Earthquake in Northern Italy

### 1.2. Performance Criteria of Pre-Cast Beam Column Joint

Precast concrete structures must meet performance standards that are largely the same as those for cast-in-place construction. Finding cost-effective and useful ways to join precast concrete pieces together to guarantee proper stiffness, strength, ductility, and stability is a challenge in the design and construction of moment-resisting frames and structural walls. Figure,2 The structure's performance throughout the service load region and at levels of performance close to the ultimate limit state must be satisfactorily achieved by the design [3].

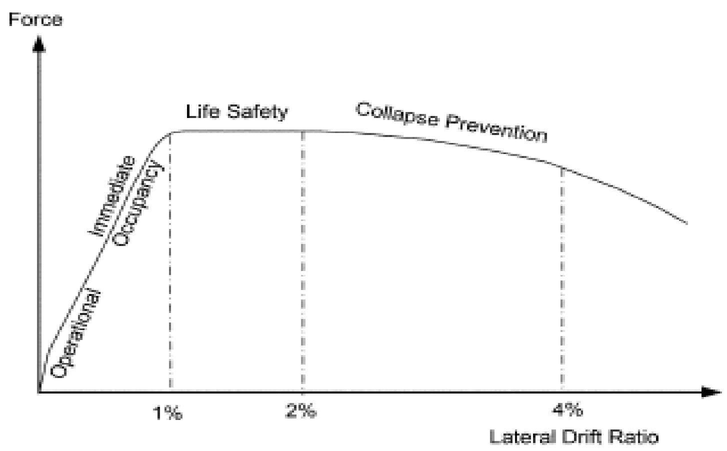



Figure 1.2. Shows structural Performance Level

### 1.3. Concrete Reinforced with Steel Fibres

Concrete that includes discrete steel fibers randomly and uniformly distributed throughout the mixture is known as fiber reinforced concrete. Previous studies have shown that steel fiber concrete offers superior qualities to conventional concrete [4-6]. Tensile strength, shear resilience, hardness, and earthquake resistance have all been enhanced. Additionally, it has been established that using SFRC and steel reinforcement together while building structural components causes a complicated interaction between the two forms of reinforcement and the structural performance of the final product. This interaction is caused by the combination of steel reinforcement and the utilization of SFRC in the construction of structural elements. The utilization of SFRC as a viable alternative structural material that can be used in flooring, paving, and precast products has just emerged as a new development. This is due to the various benefits that SFRC offers. It has been demonstrated that the use of SFRC in seismic beam column joints improves joint integrity as well as the structural ductility and energy dissipation capacity of the structure. This was demonstrated by carrying out a number of tests and studies. It is possible to lessen the amount of congestion that takes place during the process of joint formation and shear reinforcement.

### 1.4. Potential Application of Steel Fiber in Joint Areas

The spacing between the joint hoops is reduced in modern construction regulations because substantial reinforcing is needed to maintain adequate ductility in the junction regions. However, pouring and solidifying concrete in the joint has become incredibly challenging due to the densely packed hoops in the joint zones. If the joint hoops are not spaced far enough apart throughout the construction process, there is a potential of unbonded areas between the steel and the concrete, as well as spaces in the joint core. This is due to the fact that there is a probability of voids in the joint core. This is due to the fact that the weight of the reinforcement could potentially impede the fluidity of the concrete.

Steel fiber reinforced concrete, also known as SFRC, has a high capacity for dissipating seismic energy and a construction method that is relatively straightforward. As a result, SFRC could be used for beam-column joints in structural frames, which would make these joints less expensive and more flexible. To provide the requisite ductility without compromising the joint's shear strength, it is thus theorized that utilizing SFRC in beam-column joint sections might permit wider stirrup spacing in the joint zone. That would be case if SFRC had been used to connect the beams and columns. In this work, the effectiveness and failure mechanism of SFRC joints are studied using a strength degradation curve for joints based on the major tensile stress.[7].

## 1.5. Objective of Research

Precast construction, with a focus on beam-column joints, is being proposed and promoted as the main goal of this study in Pakistan due to its inherent benefits in terms of time and cost savings for the project while ensuring improved material and construction quality. The primary goals of this investigation are:

**(A)** To conduct an experimental examination of prefabricated beam-column joints under cyclic loads to assess the link between load deformation and energy absorption capacity.

**(B)** To examine how two distinct precast beam-column arrangements behave and determine the best kind of beam-column connections to use in Pakistan in light of the country's seismic requirements.

**(C)** To use steel fibre reinforced concrete (SFRC) as a bridging agent within pre-cast beam-column joints in order to improve the ability of joints to resist shear and manage cracks due to SFRC's improved compressive strength.

## 1.6. Outline of Thesis

This thesis is divided into the following six chapters.

**(D)** The significance and background of this study are discussed in Chapter 1. The study's objectives and methodology are also outlined.

**(E)** In Chapter 2, The current literature on steel-fiber reinforced structural components will be reviewed, and a brief overview of the basic characteristics of high-performance fibre concretes will be provided. Past research on the shear resistance capacity of SFRC beam-column connections is fully analyzed.

**(F)** An introduction and discussion of the evaluation of the contribution of the fiber to the joint shear of external beam-column joints can be found in Chapter 3. In this chapter, an analytical method that is based on principle stresses, shear strength degradation models, and how to apply them is introduced and discussed.

**(G)** Chapter 3 introduces us to the idea of evaluating the fiber addition to the joint shear of external beam-column joints. In this chapter, shear strength degradation models are presented and discussed. Additionally, a method of analysis based on primary stresses is presented in this chapter.

**(H)** In Chapter 4, we compare and contrast the analytical study's findings with the experimental ones, and we present a practical analytical approach and a streamlined equation for implementing steel fibre. Seismic joint design for precast reinforced concrete.

**(I)** Chapter 5 draws conclusions and makes recommendations for future research based on this project's findings.

# CHAPTER 2

# LITERATURE REVIEW

Pre-cast concrete connections and pre-fabricated parts are also covered in this analysis. Typical connection zones and connection methods for precast or hybrid concrete frame buildings are discussed in this chapter.

## 2.1. Pre-Cast Concrete Historical Development

Prefabricated concrete has been around since the Roman era and is today utilised extensively in the building and civil engineering industries. A variety of building systems, both complete and in part, are being constructed using precast technology. Precast concrete buildings, therefore, have been around since the turn of the 20th century and gained popularity in the 1960s. Because of its versatility and compatibility, pre-cast concrete is often the optimum material for a variety of building uses [8].

Precast concrete is composed of cement- and water-based concrete that has been cast into a predetermined shape somewhere other than where it will be used. Prior to being removed from the formwork or mould, which is commonly composed of steel or wood, the concrete is deposited inside and allowed to cure. The different elements of the building or structure are eventually connected once these components are carried to the building site and assembled there.

In cases where the prefabricated factory is too far away, where components are too large to transport, or where tilt-up prefabricated concrete components will be employed, the pre-cast concrete sections can also be carried out on site. Tilt-up components, in addition, are moulded, cast, and tilted right there on the construction site before being set.

Precast concrete methods often make it possible to quickly and efficiently complete a wide variety of buildings and concrete structures [9]. Some people have the wrong idea that precast technology is just the process of transforming cast-in-situ into a collection of pre-cast components that are assembled on-site to mimic the original cast-in-situ idea (FIB-Féd. Int. du Béton 2002). Possible causes of this confusion include inadequate familiarity with prefabricated concrete standards for design and construction. Menegotto (2006) argues that successful design and construction is the consequence of creating the appropriate connections to account for all service, environmental, and maximum load requirements. In addition to securing the components to one another, connections between precast concrete elements must do so in a way that ensures the structure's continuity and integrity.

The first requirement in the initial design of a concrete building is to determine whether the project, or certain parts of the project, can be erected using precast concrete elements in the most appropriate areas. On the other hand, precast concrete has many advantages when used in construction. especially according to [4] These are illustrated advantages may include the following areas:

**(A)** Longevity and a neat finish

**(B)** Quick and secure erections,

**(C)** High quality control thanks to a controlled manufacturing environment,

**(D)** Installation of precast parts requires less labour on the construction site.

**(E)** Prestressing is simple to do, allowing structural elements to be smaller and fewer in number.

### 2.2. Construction Of Concrete Prefabricated Parts

Precast concrete components are often manufactured in factory, moved to the construction site, transported, and then assembled or built there. Depending upon the structure that will be constructed, these precast parts may have standard or non-standard designs. The precast factory frequently keeps in stock common precast parts with conveniently accessible standard measurements, such as slabs, beams, columns, etc. The use of non-standard size elements will normally be specified by the architect or engineer and may include a certain dimension or shape that blends in with the desired structure or building. Precast concrete components with regular or non-standard dimensions can be broken up into multiple precast building units. Precast concrete components with standard or non-standard dimensions can be broken up into multiple precast building units. The following are some of the components that are generally present in a precast concrete building frame: Columns, beams, floor slabs, and foundations are only a few examples.

### 2.3. Joints and Connections

In the precast construction sector, it is critical to differentiate between joints and connections, particularly when considering the connections between the precast pieces. A connection can be defined as an assembly of a few components that is intended to resist the forces acting, as opposed to a joint, which is often a medium between two pieces in a construction where forces can be transferred. Forces [9].

The design concept for precast connections, in contrast to cast in-situ concrete work, takes into account both of the structural requirements and the preferred construction method. The connection design is frequently significantly influenced by manufacturing operating procedures [5]. The design and details of the structural link are impacted by the detail and layout of the neighboring structural parts since they interact closely with one another. The precast concrete components are joined together in close contact with the structure and specification of the connections are inspired by the detail and design of the nearby sections of the building. The connections must be created in such a way that the forces acting on it flow logically and may also be transferred to the structure's overall load-bearing parts [10]. Figure 2.1 illustrates typical joints that can be found in a precast concrete connection zone.

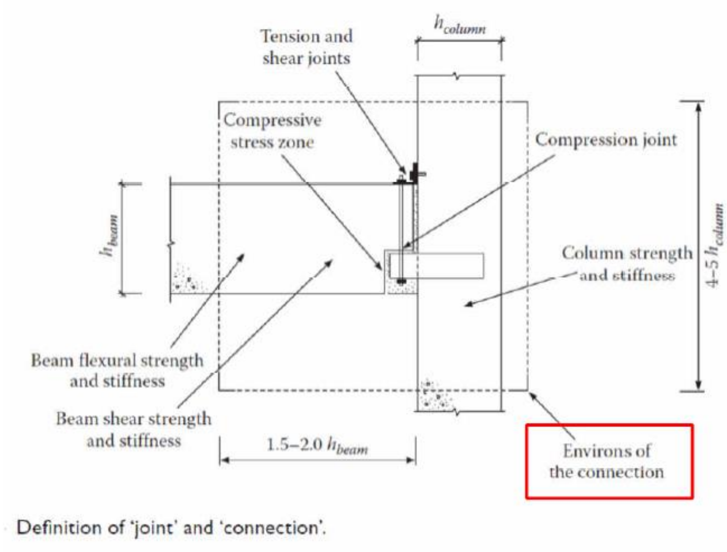



Figure 2.1. Typical Pre-cast connection

For the duration of the concrete structure's operational life, the connectors' materials must be sturdy. Steel loses strength when exposed to heat, thus the connection must also be insulated from any chemical or physical impacts and be fire resistant. Sometimes load bearing pads are used to support connection kinds that are simply supported. Therefore, it is crucial that contact pads that support the pieces are the proper size for the connection's design.

### 2.4. Precast Buildings Typical Connections

Precast concrete technology is sometimes misunderstood to be nothing more than the straightforward transformation of a cast-in-place structure into a number of precast concrete buildings pieces that must be joined on site in order to achieve the original cast-in-situ concept (Elliot 2002). This misunderstanding might be attributed to an ignorance of the design philosophy design principles, and unique characteristics and regulations related to prefabricated concrete construction and construction. The precast concrete industry has learned that there are many connecting zones in a precast structure or building where various types of connections might occur.

### 2.4.1. Precast Connection Design and Construction Requirements

The following crucial factors were determined for precast concrete design and construction.:

**(A)** **Standardization:** The benefits of a uniform connectivity infrastructure are consistent. The magnitude of the forces that structural connections are designed to carry varies from one connection to the next. As a result, it is preferable to strive to standardize a light, medium, and severely loaded type component of the same fundamental solution, each with a distinct capacity for force transfer. This makes it simple for the designer to select a standard answer, which saves time and minimizes the potential of calculating errors, while also encouraging the laborer's recurrence (possibility of less mistakes).

**(B)** **Simplicity:** For a connection feature that is affordable and unlikely to be used wrongly, simplicity is necessary to achieve. Therefore, every connection arrangement should strive to have as few individual parts that need to be put together as possible.

**(C)** **Tensile capacity:** Friction can never be a part of the created force transfer mechanism if a connection must have a tensile force capacity. As a result, all embedded units must have adequate anchoring. The connection between the anchored parts must also be capable of withstanding tensile force.

**(D)** **Durability:** Oxidation of exposing steel components or concrete Corrosion of the reinforcement bars causes cracking and spalling, which are classic symptoms of poor durability. Corrosion-prone connections need to be made of corrosion-resistant materials, such as galvanised steel, or have their steel components appropriately covered in concrete.

**(E)** **Fire resistance**: Numerous pre-fabricated concrete connecting details are fire resistant and don't need to be treated differently. To provide the same resilience as the structural frame, connections where fire damage could weaken them should be covered.

**(F)** **Aesthetics**: The significance of aesthetics in a construction cannot be overstated. Any structural connections that cannot be concealed can either be highlighted or incorporated into the structure's architecture.

### 2.4.2. Existing Connection Types

There are several different connection types in standard precast concrete frame constructions. The following connecting zones have already been identified for use throughout this entire study:

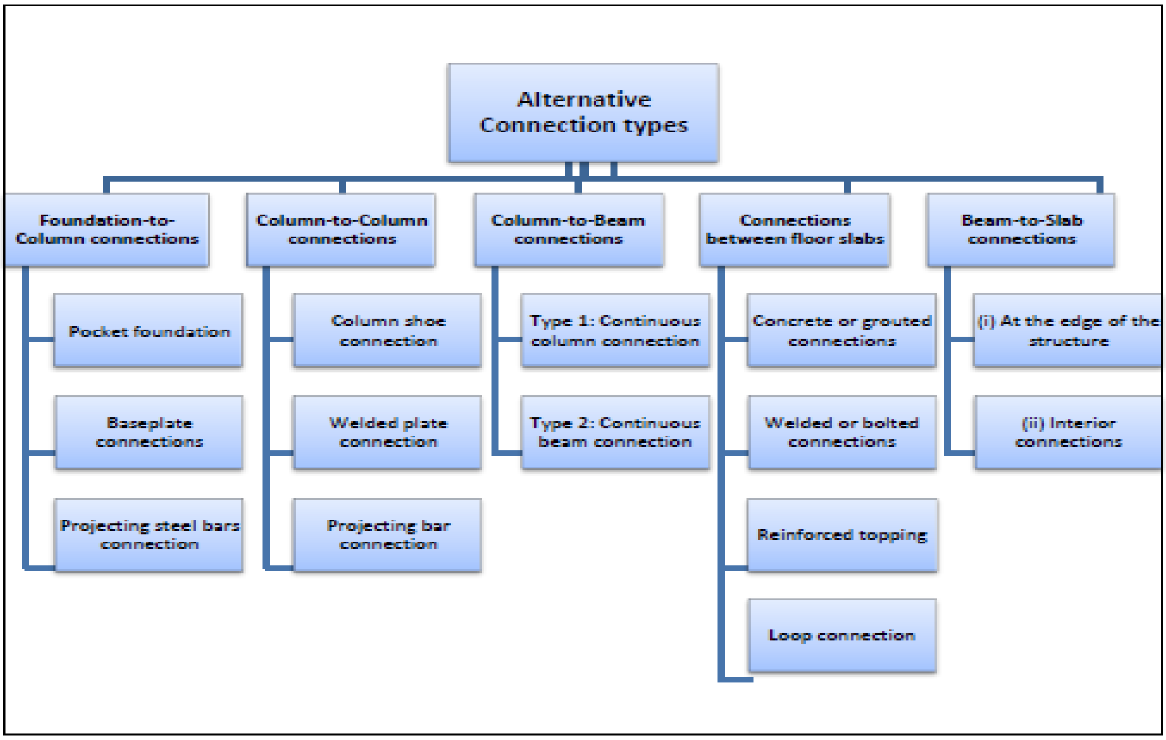



Figure 2.2. Types of pre-connection

### 2.4.3. Joint System

Precast concrete connections can be classified into two categories based on how they are constructed: dry connections and wet connections. For a dry connection, connecting pieces are welded or bolted together while being embedded with steel plates or other components. Concrete that has been cast in place or grout are used to assemble joining components for wet connections. Only the wet connection was used in this study's experimental inquiry.

In addition to cast-in-place beam-column junction regions, precast beam-column joint cores, and precast pretensioned joints, wet concrete is connected to a wide range of study areas and building techniques. There are countless potential links between the two topics, and these are only a handful of them. Plain columns and beams may be constructed with a regular structure when using the cast-in-place beam-column connection core, making them portable and simple to erect. In addition, their size is minimized, so they are not cumbersome. Additionally, the beams and columns are produced in a factory environment. On the other hand, the use of cast-in-place beam-column junction regions makes it simpler to construct the joint regions. Because of the intricate reinforcing connection and concrete pouring, the region presents difficulties in ensuring that the construction quality is maintained. There are different precast members available when using the prefabricated beam-column joint core, including cross type, T type, and straight type. [2,3] It makes transportation substantially more challenging as a result. On the other hand, in contrast to the use of cast-in-place construction.

The connecting core's proper construction would guarantee a minimal margin of error where the prefabricated beams and columns converge. Precast prestressed joints may still utilize the inner joint force to their fullest potential despite the difficult manufacturing process. In order to study it, the cast-in-place beam-column connecting unit was chosen for this project and constructed.

### 2.4.4. Dry and Wet Connection

Numerous investigations on the prefabricated concrete frame connection have been done in various nations. PRESSS (Pre-cast Seismic Structural System Research Program) [11] Japanese and American experts direct an important seismic research project on prefabricated concrete structures. The purpose of the Prefabricated Seismic Safety Standards (PRESSS) project is to produce design guidelines that may be incorporated into current building codes for prefabricated concrete structures located in regions of high and moderate seismicity. Daisuke [12] compared the performance of a cast-in-place connection with that of three different precast concrete beam-column connectors through the use of an experiment. The horizontal bar connection at the joint differentiates the three forms of precast connections, all of which are wet connections. This is the primary distinction between the three types of precast connections (bending lap of lower bar, sleeve splice of lower bar and sleeve splice of both upper and lower bar)

According to reports, the precast connections had reduced core zone shear strength and initial stiffness. Uramoto [13] It was discovered that raising the horizontal projection of the horizontal bar connection increased the seismic performance of the joint. Both Daisuke and Uramoto's study was included in the PRESSS program. study of several precast connections, Onur Ertas [14] examined and evaluated five distinct types of connections for concrete frames, including one that was cast in place, two precast wet connections (one at the end of the beam and the other at the end of the column), and two precast dry connections (welded and bolted with bracket). According to the findings of the experiment, bolted connections have excellent characteristics in terms of their strength, ductility, and ability to dissipate energy. In comparison to the other prefabricated wet connection, which is cast at the end of the beam, this one features a larger plastic hinge and more effective energy dissipation. Additionally, it is prefabricated in a different location. Jose et al. [15] conducted study on the two pre-cast connections described in "Guidelines for the Use of Standardized Structure Precast Concrete in Buildings," namely "Precast Beam Units between Column" and "Precast Beam Units between Columns" [16]. The analysis shows that both precast connections can withstand earthquakes with little to no damage, although the first connection is weak where cast-in-place and prefabricated parts merge. The latter, however, has a precast junction that does away with the challenging in situ construction procedure and ensures the structural endurance. Vidjeapriya et al. There was a comparison made between pre-cast connections with stiffeners and cast-on-site connections. He discovered that the cast-in-place specimen performed worse in terms of energy dissipation and elasticity than the precast specimen did, but the precast specimen had a lower maximum load-carrying potential than any of the other precast samples. Parastesh et al. [18] evaluated an unique moment-resistant ductile beam column connector. The connection had sufficient flexural strength, strength degradation, and drift capability, in addition to much greater ductility and energy dissipation than analogous cast-in-place specimen. Yuksel et al. [19] reported the findings of their scientific investigations of the various connections that are utilized in commercial and home settings respectively Both connections demonstrated steady load-displacement cycles and robust energy dissipation up to a structural drift of 2%; however, once the drift increased to 3%, considerable pinching and degradation of the crucial section began to take place. Wu et al. [20] during the test, both the cast-in-place connection and the prefabricated concrete connector were compared to one another. Although the hysteretic characteristics and tics of the two connections are comparable, the prefabricated connection's bearing capacity degrades more quickly than that of the cast-in-place connection. In addition to this, the cumulative damage to the prefabricated connection is far worse after yielding. Amadio et al. [21] investigated A complex finite-element computer model was utilized in order to evaluate the composite welded connections that were created between the beam and the column in order to determine how structurally sound they were (ABAQUS) . Through the use of this computational approach, full-scale experimental investigations—which are costly and time-consuming—could be replaced. This would allow for a detailed parametric analysis of composite joints and possible design improvements.

Since they function similarly to cast-in-place connections seismically, wet connections are the focus of a lot of research. Precast concrete buildings have varying seismic performance in recent earthquakes. The earthquake that occurred in Van, Turkey, in October 2011 did not appear to have any discernible effect on the structural frame components of a precast residential construction that had moment-resisting post-tensioned connections. However, due to the absence of elastomeric bearings in the beam-to-column connections at the interface for prefabricated concrete manufacturing units, the prefabricated frames that did not have walls or roof covers sustained damage of varied degrees during the same earthquake. At addition, there was crushing and cracking around the borders of the structural components where they joined together in the joint regions. [22]. The hollow members tended to shatter close to the bending cracks in the beam during the Canterbury earthquake sequence since the hollow rod support had no backing strip added [23]. These field investigations demonstrate the importance of connection performance and compatibility in achieving prefabricated pieces' earthquake resistance. Different wet connection types exist. The research of Jose et al. [24] indicates that the prefabricated beam units that are situated between the columns have a poorer dynamic response than those that are situated across the columns (both are wet connections). By connecting Prefabrication Beam Units with Columns, one can skip the challenging in-situ construction procedure, resulting in construction that is of higher quality and that is more practically realizable. The beam-column connections that are depicted in Figure 1 are comparable to the type of Precast Beam Units via Columns that was previously investigated in this study. This type of connection has been utilized in China for the construction of residential and commercial structures. The precast concrete upper column is assembled by inserting reinforcement from the lower column into the grout sleeves that are embedded in the higher column. After that, the grout sleeves that are present in the column joints are stuffed with grout. In order to successfully assemble this type of connection, the following steps must be done in sequence: A high-performance sealant is injected into the grouting groove, reinforcing bars are fitted through the allotted holes on the precast beam, and the precast top column is created in order to bind the precast beam and column together.

However, there is a lack of experimental data to support the resilience of the connections to seismic loads. In order to explore prefabricated connections under earthquake loading, an experimental programmed is created. Grout sleeve connects the rebar (refer to Fig. 2). The grout sleeve comes in two varieties. Full grout sleeves are used to link the rebar in the beam, whereas half sleeves are used to connect the rebar in the column. Beijing Jinmao Construction Equipment Co., Ltd. makes the grout sleeve. The coupling between a sleeve and the reinforcing at each end is done by grouting. The horizontal reinforcement is frequently connected together utilising full grouting sleeves. Sleeves with half-grouting are shorter than those with full-grouting and are used to connect vertical reinforcements. A screw thread on one side and grouting on the other are used to connect the sleeve to the reinforcement.

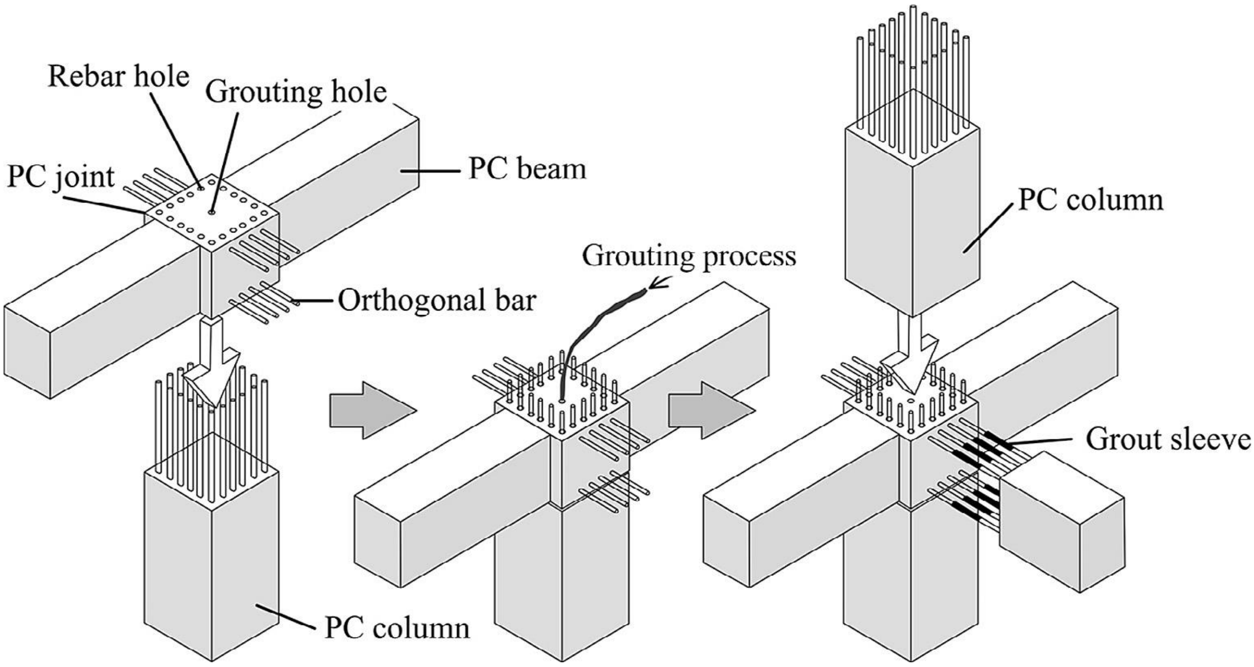
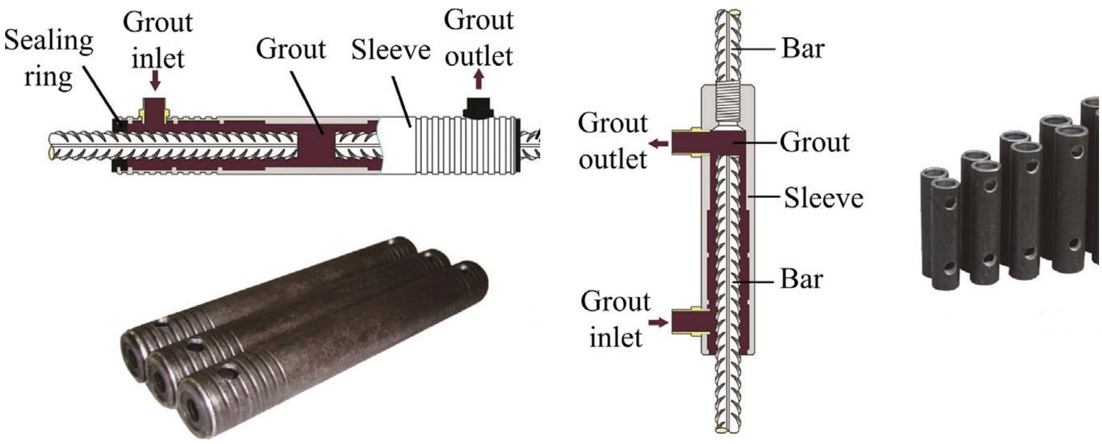



Figure 2.3. Connection consolidation.




(a) Full grout sleeve (b) Half grout sleeves

Figure 2.4. Grout sleeve

### 2.4.5. Welded Connections

To be used in precast concrete constructions, several welded connectors were designed by Bhatt and Kirk [25] and Seckin and Fu [26]. Although the behaviour of the connections seems satisfactory, the details call for welding the reinforcement in the beam and column, which could create some issues on the job site. French et al. [27, 28]

tested a variety of beam-column connections, some of which produced plastic hinges outside the connection area. It was discovered that the threaded reinforcing bar connections with tapered, threaded splices proven to be the most effective, economical, and practical approach. Ersoy and Tankut [29] tested dry-jointed precast concrete beams under reversing cyclic loading. The original beam was made up of two steel plates, one at the top and another at the bottom, which were welded to the anchoring steel plates in the beam and the column bracket. Later, side plates were added to the design to update it. The member having side plates has similar strength, stiffness, and energy dissipation as the monolithic member. However, putting such a detail into practise on location is rather challenging and necessitates a stringent quality control system. Priestley and Tao [30] suggested using unbonded prestressing-incorporated lateral load resisting devices for usage in earthquake-prone locations. A prefabricated concrete ductile frame has been developed that makes use of the precast concrete's natural discreteness by including flexible links in the connections [31]. These ductile connectors, which also do away with the requirement for corbels, have a rod inside of them that yields at a specific strength, effectively reducing the amount of weight that can be transmitted to the frame's less ductile components. PRESSS project [9, 32] is the most prominent attempt at an experimental investigation of precast constructions' seismic reaction. Prefabricated concrete frames with unbounded tendons, ductile connections for prefabricated concrete panel systems, high performance fiber-reinforced concrete energy-absorbing junctions for precast concrete frames, and ductile connections for prefabricated concrete frame systems are some of the primary issues investigated. Korkmaz and Tankut [14] six beam-to-beam connector subassemblies were put to the test using cyclic reversed loading. The behaviour of the prefabricated members and the monolithic specimen were compared. In the context of the publication, numerous suggestions for future research and practises are also presented in order to enhance the structural performance of the connection. Ertas et al. [33] tested one monolithic concrete connection and four different types of ductile moment-resisting prefabricated concrete frame connectors. In addition to being simple and quick to fabricate, the redesigned bolted connection demonstrated the good performance in terms of strength, flexibility, and energy dissipation. Three of the samples were capable of withstanding 3.5% tale drift. Kaya and Arslan [33] ANSYS software was used to carry out a number of analytical tasks and test post-tensioned prefabricated beam-to-column connections at various stress levels. The test results are contrasted with the findings of the analytical works.

### 2.4.6. Connection Information and Pre-Cast Members' Locations

One of the many experimental or analytical inquiry challenges for researchers is the connection detail and position between precast pieces [14]. Tested under reversed cyclic loads in the inelastic range were four different types of cast-in-place prefabricated (CIP) beam-column connections. 3.5% story drift angle was applied to specimens during ultimate loading. The hysteresis behaviour of the monolithic Specimen and the cast-in-place specimens of the column, beam, and modified bolted were identical. Due to the inclusion of steel fibre in the concrete and the U-shaped design of the reinforcing bars, the pinching effect and significant bond degradation were not seen in the CIP connections. The research of Restrepo [15] composed of precast concrete columns and beams with a CIP concrete joint core built at the intersection of the beam and column. The test findings demonstrated that it is possible to successfully design and build the link to mimic cast-in-place construction**.** Alcocer et al. [34] two complete precast beam to column connections were put to the test with either uni- or bi-directional reversed cyclic loads. Instead of welding or special bolts, conventional mild steel reinforcing bars or prestressed concrete strands were utilised to achieve beam continuity. In both specimens, the connection strength was 80% greater than that of monolithic reinforced concrete buildings. Up to a 3.5% drift, the connection capacity remained essentially constant. They reported that the column face experienced the predicted development of plastic hinges.

Khoo et al. [35] developed a modified assembly design where the connections are built on the beam span and kept away from the column faces for precast concrete frames. By using this assembly, the joint region and plastic hinge length during earthquake excitation were avoided. They used hooked bars at 90 and 180 degrees that were the same length as the effective beam depth d from the column faces. According to the test results, when 90-degree hook were utilised, there was significant bond weakening in the connection zones as a result of the short anchorage length. Building connections distant from column faces is a shortcoming of these systems that makes it challenging to transfer elements.

Some studies have been done related to prestressed and hybrid connections [36]. To construct the precast components, post tensioned steel bars were created. According to their findings, prestressing was crucial for maintaining continuity and preventing shear. Pre-tensioned strands maintained continuity and provided the necessary shear strength to the load applied in the absence of corbels and shear keys, whereas post-yielding of steel bars dissipated the energy [37]. To construct the precast components, post tensioned steel bars were created. According to their findings, prestressing was crucial for maintaining continuity and preventing shear. Pre-tensioned strands-maintained continuity and provided the necessary shear strength to the load applied in the absence of corbels and shear keys, whereas post-yielding of steel bars dissipated the energy. Li and Leong [38] examined two monolithic and two hybrid precast connection types.

They observed that the specimens' decreased moment capacity was caused by a discontinuity in the bottom reinforcement of precast beams. Additionally, pinching happened as a result of the tolerances being widened in bolted connections.

### 2.4.7. Modified Bolted Connection

The seismic performance of four different types of ductile beam-column connectors [14]. The test results showed that the enhanced connections had the advantages of being easy to manufacture and functioning effectively during earthquakes. For the dry beam column connection proposed by Vidjeapriya and Jaya [17]. The prefabricated reinforced concrete (RC) columns and beams with various stiffener levels were joined using the cleat angle [39]. Experimental testing was done to determine the efficacy of a semi-rigid beam-column connection in which the PC beams were supported by continuity rebars on the corbel of the PC columns. Park and Bull [40] large-scale exterior beam-column connections made of PC columns and composite beams with U-shaped shells were tested for performance. Kim et al. [41] a study that includes tests on the developed cruciform PC beam-column connection was reported. In this beam-column arrangement, a PC beam shell was also used, and straight reinforcing bars were used for straightforward construction. It has been determined that the precast connectors operate adequately during earthquakes.

Parastesh et al. [18] reported the results of studies on suitable beam-column connections for computer frames located in seismically active areas, both on the inside and exterior. Before the concrete was cast, the longitudinal reinforcing bars were joined at the bottom of the beam, and the prefabricated building beams had hollow cross sections with a U shape. The flat surface of the U-shaped segment prevented any slippage between the PC components and the cast-in-place concrete. The flexural strength, ductility, and energy dissipation characteristics of the PC connections were demonstrated to be on par with those of monolithic samples.

Hyeong et al. [42] based on the earlier research, a cyclic loading test on simulative large-scale beam-column couplings. Due to bond-slip in the reinforcing bars where the PC beam shells weren't fully integrated with the cast-in-place concrete, the yield stiffness of the PC connection in their study fell by 10% and energy dissipation by 36% when compared to the RC connection. To enhance the effective cross-sectional area of the beam core and the depths of the joint, it was suggested that the PC beam shell's thickness and seating length be reduced.

Considering the weakness of the PC beam-column connections, in order to boost the earthquake resistance of the beam-column connections, a plastic hinge relocation solution employing two strengthening techniques and one weakening technique has been proposed [43]. Certain researchers proposed several techniques for joining the reinforcement of opposing beams, and these technologies proved to be effective.

In some PC emulative beam-column connections, the PC beams were cast without the U-shell, and various alternative techniques were used to connect and attach the longitudinal reinforcing bars of the beam. Xue and Zhang [44] designed a hybrid beam-column connection using cast-in-place columns and composite concrete beams. It was shown that the interior and outside connections act in the same way to the monolithic connections.

### 2.5. Fundamental Characteristics of Fiber-Reinforced Concrete

Traditional portland cement-based concrete is weak in tension but strong in compression. Concrete uses reinforcing bars to compensate for this tension weakness. The deficiency in tension can now be somewhat compensated with the development of fibre technology by adding more fibres to the system. A sufficient amount of fibers in the concrete can improve the post-cracking behavior of composite fibre matrices, hence increasing their toughness. (Fig. 2-1) Fibers are used in certain applications for structural purposes as well as for dry crack reduction, chemical stability, resistance to abrasion, and high durability. Table 2-1 lists all of the various uses for fibers.

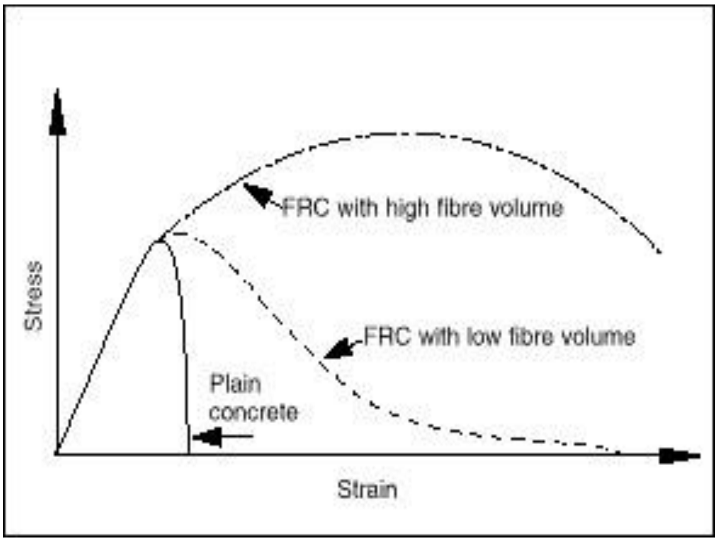



Figure 2.5. Typical fiber-reinforced concrete load-deflection curves from [45]

Table 2.1. Uses for various fibre kinds of concrete

| Fibre Type | Application |
|---|---|
| **Steel** | Structures that are resistant to earthquakes include things like bridge decks, cellular concrete roofing modules, pavement overlays, concrete piping, airport runways, pressure vessels, tunnel linings, ship hull construction, and concrete piping. Other examples include airport runways and pressure vessels. |
| **Glass** | Panels that were precast, smaller containers, sewer pipe, roofs made of thin concrete shells, and wall plaster for concrete block were some examples. Agriculture, cladding for buildings, and architectural components. |
| **Carbon** | Membrane structures with a single or double curvature, scaffolding boards and ship hulls. |

| | |
|---|---|
| **Polypropylene, nylon** | piles for foundations, prestressed piles, face panels, floating walkways and docks in marinas, patching material for roads, and heavy-duty coatings for underground pipelines |
| **Asbestos** | Sheets, pipes, boards, sewer pipes, corrugated and flat roofing sheets, and wall lining are some examples of building materials. |
| **Mica Flakes** | Replace asbestos in cement boards, concrete pipes, and repair supplies in part. |
| **Natural fibres** | Roof tiles, corrugated sheets, pipes, silos and tanks. |

## 2.6. Steel Fibre Reinforced Concrete

Steel fibers have been used in the building of shotcrete linings and pavement during the course of the previous three decades. However, due to a lack of authorized design equations and appropriate rules, the utilization of steel fibres concrete (SFRC) in the actual design of seismically structures is constrained in various aspects. This is the case because SFRC is more difficult to work with. Steel fiber reinforced concretes (SFRC) are now commercially available in a range of forms, and the use of these materials in projects located in seismically active locations has been steadily rising. Steel fiber reinforced concretes (SFRC) are now available in a number of commercial forms.

The term "steel fibre reinforced concrete" refers to a type of concrete that has been fortified by the addition of discrete, discontinuous fibers that have been distributed in an even and haphazard manner. Both the quality and quantity of steel fibers in concrete have an effect on the material's mechanical properties. It is general knowledge that incorporating steel fibers into a composite material result in a significant improvement in the material's compressive strength, tensile toughness, and ductility (Figure 2-2 and 2-3). After concrete has fractured, the tensile stress is transferred to the steel fibers, which reveals the advantages of using steel fibers in the repair process (Figure 2-4). Because of the way that structural design works, fiber dosage rates of less than one percent are not helpful for withstanding forces after significant cracking.

### 2.6.1. Workability

One of the problems with SFRC is that it is not very practical. The workability of plain concrete, the amount of fiber, and the aspect ratio of the fiber all affect the workability of SFRC (lf / d f ).

By including these admixtures, regulating size of aggregate, and decreasing the water to cement (w/c) ratio, the workability of SFRC is often increased. Numerous scholars highly advise that the maximum aspect ratio (l f / d f) and volume fraction of SFRC (V f) be set at 100 and 2.0%, respectively.

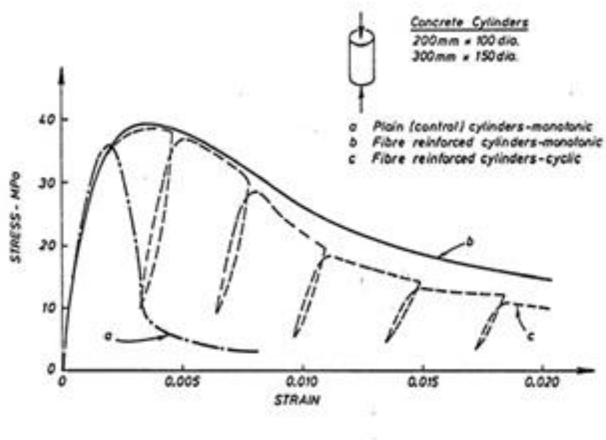



Figure 2.6. Relation Between Compressive Stress and Strain [46].

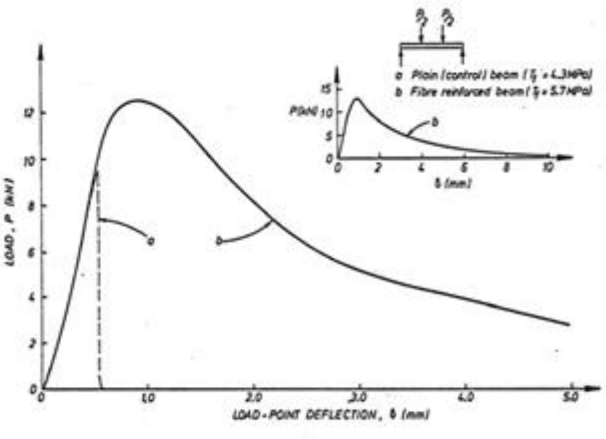



Figure 2.7. Flexural load-deflection relationship [46]

### 2.6.2. Shapes of Fibers

Steel fibers are available in a variety of diameters and shapes around the world. Steel fibers used in structural areas are normally limited in length to 1.5 to 75mm and aspect ratio to 30 to 100. Steel fibers' cross sections can be hooked, paddling, or crimped (in Figure2-5)

### 2.6.3. The Definition of Toughness

Toughness is defined as the capacity of a material to withstand considerable post-elastic strains and deformations before failing, in addition to its ability to impede the spread of cracks. Because of this distinguishing feature, fiber-reinforced concrete cannot be confused with conventional concrete. In conventional concrete, this property is not present. When flexure is being addressed, this region is occasionally referred to as the region that lies beneath a load-deflection or stress-strain curve. Figure 2-6 [14] shows how ordinary concrete samples and SFRC samples compare for common load-deflection curves. This illustration shows how concrete's durability can be significantly improved by the addition of fibres. The theory behind this is that even when the fibre matrix splits, the load-deformation energy is emitted as the fiber pull away from the matrix, increasing toughness, allowing SFRC to bear severe loads.

**Hooked (Dramix)**

**Paddled (Novotex)**

**Crimped(Xorex)**

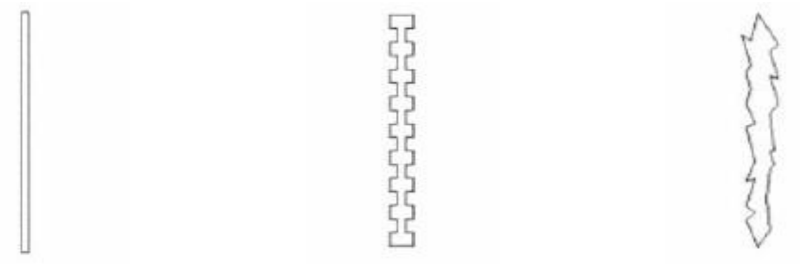



Figure 2.8. Steel fibre shapes

## CHAPTER 3

## SEISMIC BEHAVIOUR OF BEAM-COLUMN JOINT SUB ASSEMBLIES

It is common knowledge that beam-column connections, when compared to other regions of reinforced concrete members meant to withstand seismic assault, can be significant areas. The seismic moment applied by the beam directly above it, as well as the seismic moments applied to the columns directly above and below it, imposes significant shear stresses in both the horizontal and vertical planes on the joint. (Figure 3-1). Frame buildings that are subjected to seismic stress run the risk of collapsing if the joints in the structure are improperly made, as is commonly the case in buildings that were constructed before the 1970s (Figure 3-2). This chapter's study focuses on evaluating the effectiveness of high-performance steel fiber-pre-cast reinforced concrete beam-column joints.

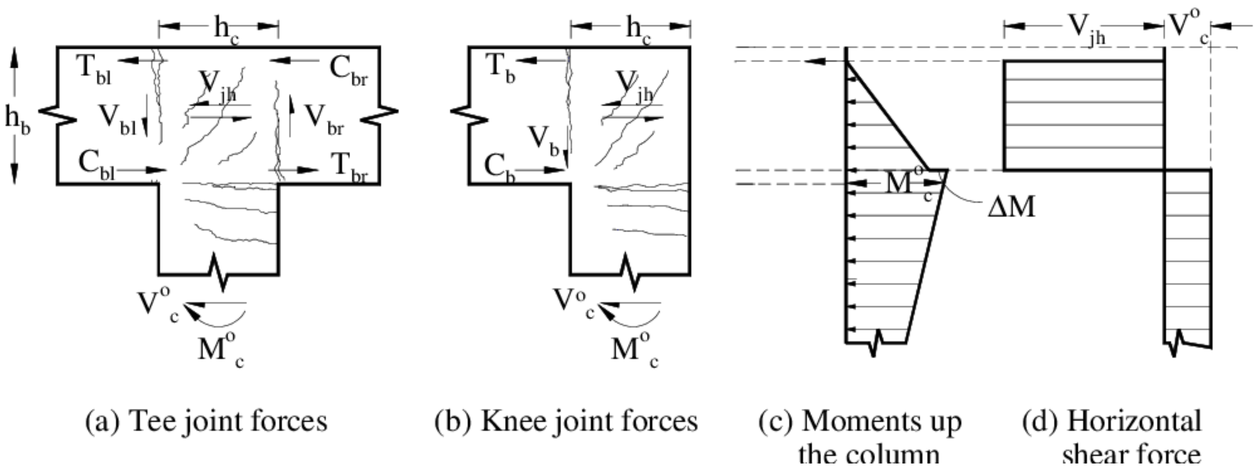



Figure 3.1. Tee-joint shear force.

### 3.1. Exterior frame joints' actions

Figure 3-3 depicts the geometry of a framed structure after it has undergone lateral seismic loading. The midway of the beam and column members is believed to be the site of every incident of contraflexure, according to the results of the inelastic investigation. In order to create an outside connection, only one of the beam frames needs to be coupled into a column, which is represented in Figure 3-3 by the circular area. Figures 3-4 and 3-5 illustrate the moments and shears that would be present at a typical external beam-column joint core site when the site was subjected to seismic loading. These moments and shears can be seen as a result of seismic loading. It is possible to illustrate this by taking into account the moment balance close to the joint. assuming that Vcol is the same as Vcol', Equation 3.1.

$$M_b + 0.5 h_c V_b = 0.5 l_c V_{col} + 0.5 l_c' V_{col}' \tag{3.1}$$

An outside beam-column joint core is subject to a horizontal shear stress that can be represented as follows:: in the Equation 3.2.

$$V_{jh} = T - V_{col} = A_s \cdot f_s - V_{col} \tag{3.2}$$

According on the yield stress of the beam bars, fs represents the stress in the top reinforcement of the beam. The overstrength factor of should be considered while constructing horizontal joint shear force. Thus the determination of fs can be expressed in Equation 3 [47].

$$f_s = \alpha f_y \qquad where \quad \alpha \geq 1.0 \tag{3.3}$$

Equation 3.1 makes it possible to accurately evaluate the major horizontal shear force that acts across the column. This level of accuracy is required for design purposes. as;

$$V_{col} = \frac{M_b + 0.5 h_c V_b}{0.5(l_c + l_c')} \tag{3.4}$$

The vertical joint shear force can therefore be determined using Equations 3.2 and 3.4.

$$V_{jv} = T'' + C_c' + C_s' = T' + C_c'' + C_s'' - V_b \tag{3.5}$$

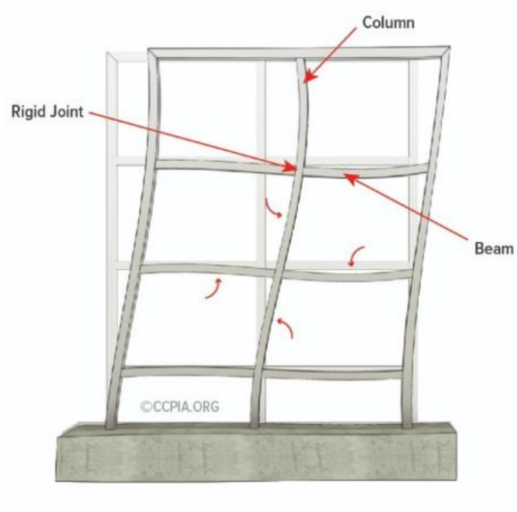



Figure 3.2. A framed structure's reaction to a subsequent load from

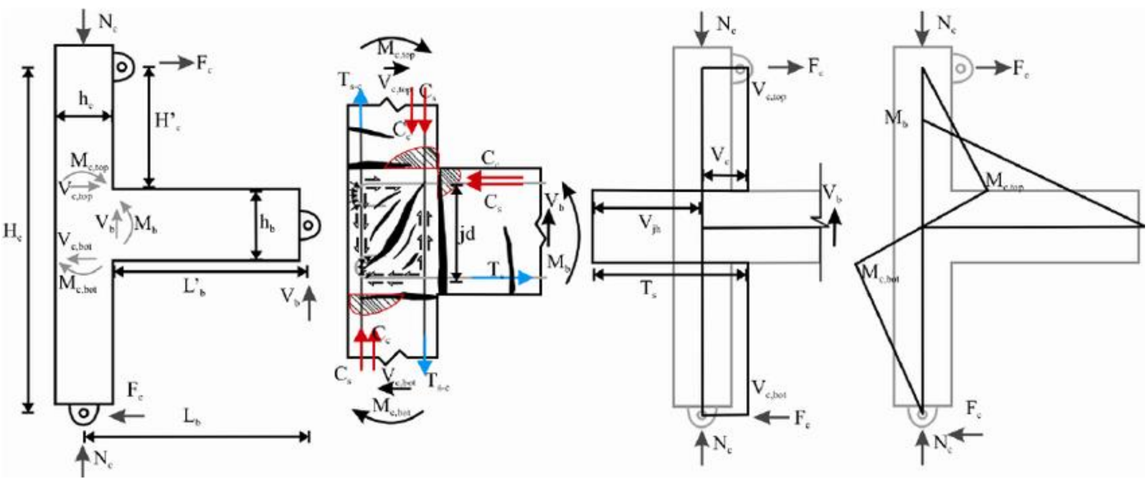


Figure 3.3. Action on a joint between an exterior beam and column

### 3.2. Mechanisms for Outside Beam-Column Joints to Prevent Shear

#### 3.2.1. The Joint Core Is Subjected to Shear Forces

In order for the horizontal shear stress applied to the bottom of the joint to balance out the horizontal shear force delivered to the top of the outer beam-column junction; it must also be applied to the whole joint. Both the column's exterior face and the face next to the beam must maintain their balance in the presence of a vertical joint shear stress.

# CHAPTER 4

# TEST PROGRAM

## 4.1. Introduction

This chapter provides an introduction to the units that will be assessed as part of the ongoing research study. In this project's experimental effort, the seismic behaviour of outside pre-cast beam-column joint sub - assemblies was examined under simulated cyclic loading patterns. Two innovative precast beam column connections will be compared to traditional pre-cast concrete connectors.

During these tests, a number of different aspects, including the joint's crucial primary strength, ductility, behavior, and energy dissipation, were put to the test. The first thing that must be done is to cast a precast conventional sample, which is then followed by the casting of a monolithically cast sample, a precast beam unit through column, a precast beam unit between column, and finally the insertion of 5D dramix in the beam column junction.

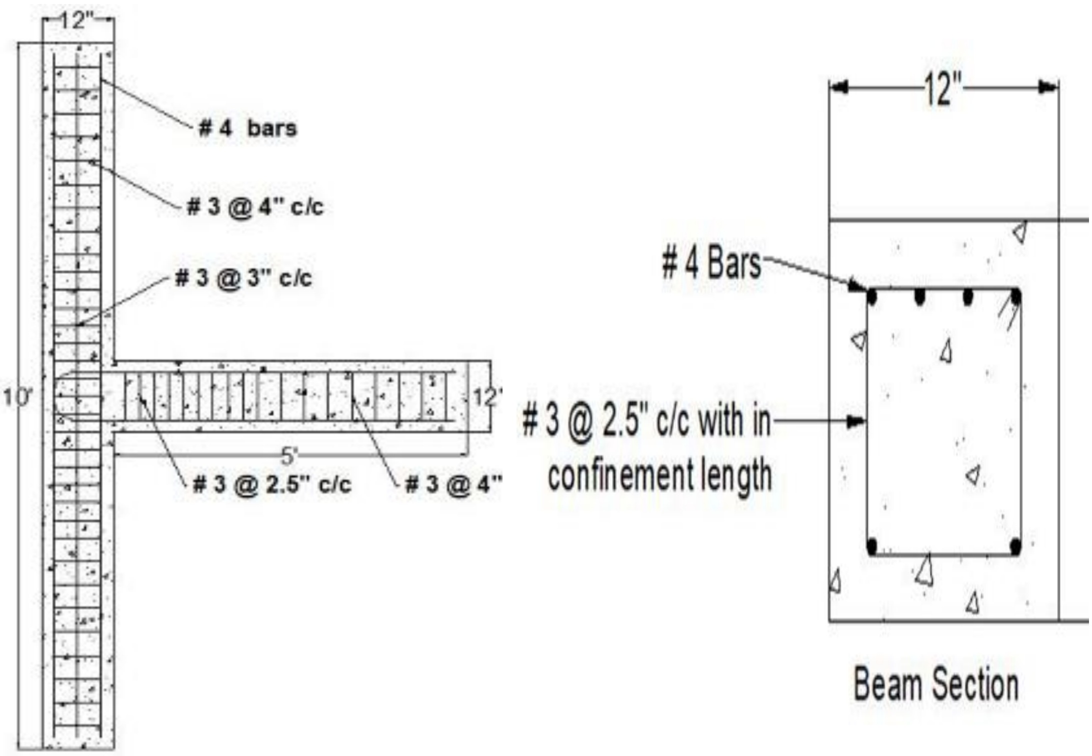
## 4.2. Test Specimens Details

### 4.2.1. Overall Dimensions and Loading

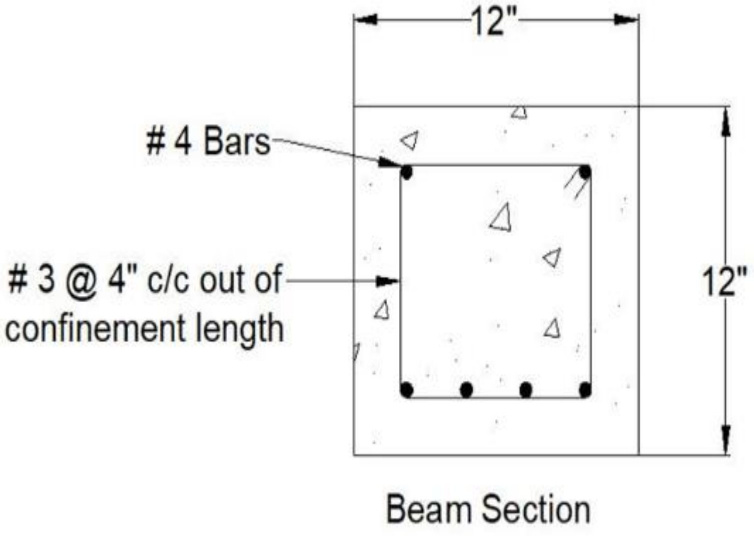
Four external beam-column junction components used in the current study. Each unit is a piece of a multi-story frame of planes. To simulate the weight of the building, the test components were hydraulically loaded to the top of the column, but the "P-effect" was not taken into consideration. The impacts of earthquake loading were reproduced by applying lateral loads to the column in the opposite direction, and the lateral loads caused responsive shear at the ends of the beam. The sample is separated into the subsequent groups.

### 4.2.2. Control Monolithically Casted Samples

The following figures provide a breakdown of the control monolithically units that make up Group I, including the diameters of the units as well as the reinforcement details (Figure 4-1 to 4.2). The cross section of the column was square and measured 304 millimeters by 304 millimeters, whereas the cross section of the beam was rectangular and measured 304 millimeters by 304 millimeters. The design of the shear reinforcement in the beam is shown below in the figure, and the longitudinal reinforcement of the beam is made up of eight main bars of grade 60 steel that are each 12.7 millimeters in diameter (figure 4.1). In addition to the four corner bars that make up the longitudinal reinforcement for the column, each face of each column also contains one intermediate bar. This reinforces the column along its length.

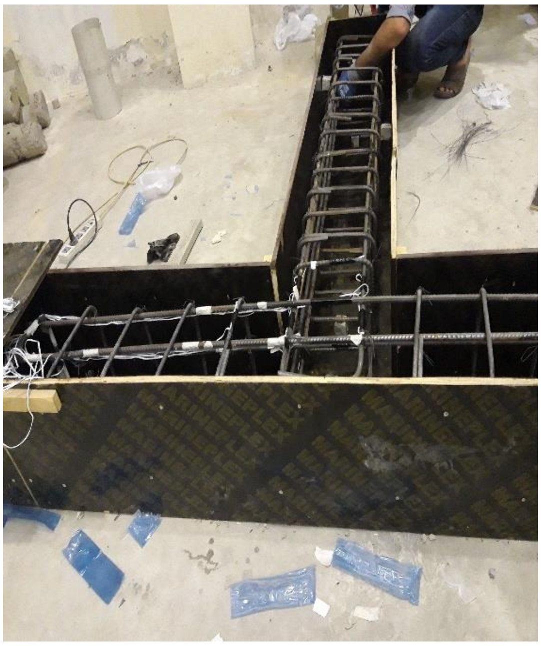



(a) (b)




**(c)**

Figure 4.1. Detail designing of samples (a) Detailing of beam colmn joint, (b) Beam crossection at joint, (c) beam cross-section .



**(a)** **(b)**

**(c)**

Figure 4.2. Casting of samples (a,b) Casting of Monothically sample, (c) Formwork

### 4.2.3. Pre-Cast Sample Of Beam Unit Between Column

A single piece of monolithic concrete is used in the construction of the connection, in addition to PC columns and PC beams that have U-shaped shells (Figure 4.3 -4.5). Utilizing longitudinal bar, which is then joined at the column bases by grouted steel sleeves, is a method that may be used to bring together PC columns of any height, whether they are one story or multiple stories tall.


**(a)** **(b)**


**(c)** **(d)**

Figure 4.3. Casting of pre-cast column and formwork (a) Pre-cast lower column, (b) pre-cast upper column, (c, d) Pre-cast U shaped beam

Pre-cast beam components with U-shaped cross-sections at the ends are situated on the concrete cover of the PC columns below and in the space between the columns. However, beam bottom longitudinal bars, which are small-diameter Grade 60 steel bars, are stacked in one or two layers and attached together using hooked anchorages. Beam top longitudinal bars are linked continuously throughout the joint. Throughout the entirety of the joint, the beam top longitudinal bars are linked together constantly. Open stirrups that are fitted with stringent 135 hooks are utilized so that the task of correctly positioning the steel bars in the U-shaped beam shell zone may be accomplished with greater ease (Figure 1). On the other hand, we make use of rectangular closed stirrups everywhere else. The standard method of confinement used for monolithic concrete projects can be applied in this prefabricated system to estimate the distance between stirrups in the same way that it is done in those constructions. Both the inside surfaces of the U-shaped shell and the top surfaces of the PC beam have had a roughening treatment applied to them. This was done in order to reinforce the bond that existed between the PC and the concrete that was put in situ.

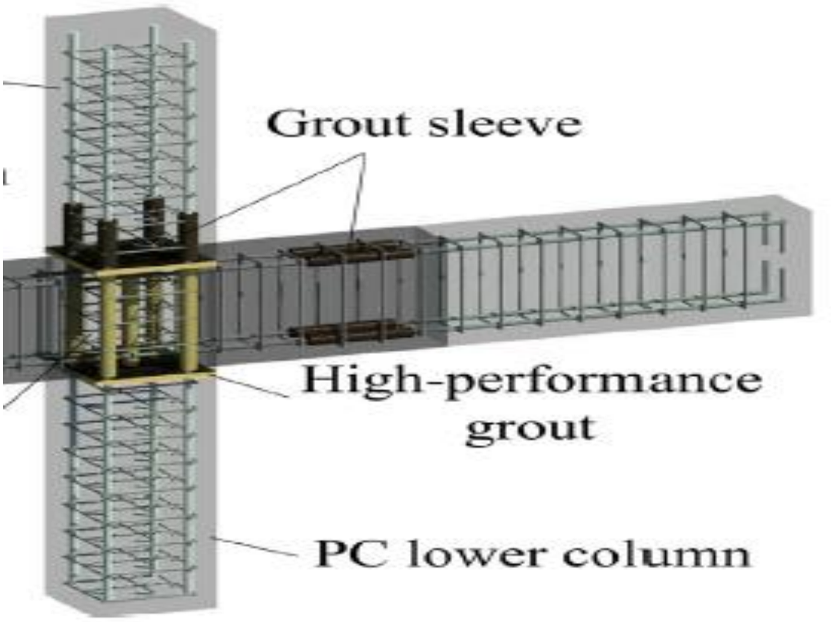
The technique for the fabrication is comprised of the following steps: (1) the location of precast columns; (2) the arrangement of precast beams and their location on the concrete column cover; (3) the connection of open stirrups, small-diameter hoops, and beam top longitudinal reinforcement; and (4) the casting of concrete in the beam-column joint core, U-shaped beam shells, and upper portion of the PC beam units in a single operation. (1) the placement of precast columns; (2) the assembly of precast beams and their placement on the concrete column cover;.

**(a)** **(b)**

Figure 4.4. Shows the Assembling of pre-cast beam on joint (a) Placement of U-beam on column, (b) Joint region of beam-column joint under consideration.

**4.2.4. Pre-Cast Beam Unit Through Column**

The precast specimen consists of a PC upper column, a PC lower column, and a PC beam. All three columns are cast in the same material. The whole can be broken down into these three parts. The geometrical measurements and information on the reinforcing elements of the precast specimen P2 are applicable to the cast-in-place specimens as well. There is a great deal of variety in the types of grout sleeves available. It was necessary to conduct separate experimental research on the grouting sleeve joints in order to validate the high quality of the grout sleeves that were utilised in this examination. The test results reveal that the damaged area is outside of the grouting sleeve, indicating that the grouting sleeve's joint's tensile strength exceeds that of the connected steel bar, as illustrated in figures 4.6 to 4.7.




**(a)**

**(b)**

**(c)**

**(d)**

Figure 4.5. Precast beam unit through column (a) Diagram of Pre-cast specimen, (b) Jointing mechanism, (c) Pre-cast column to column connection, (d) pre-cast column head.

## 4.3. Properties of Material

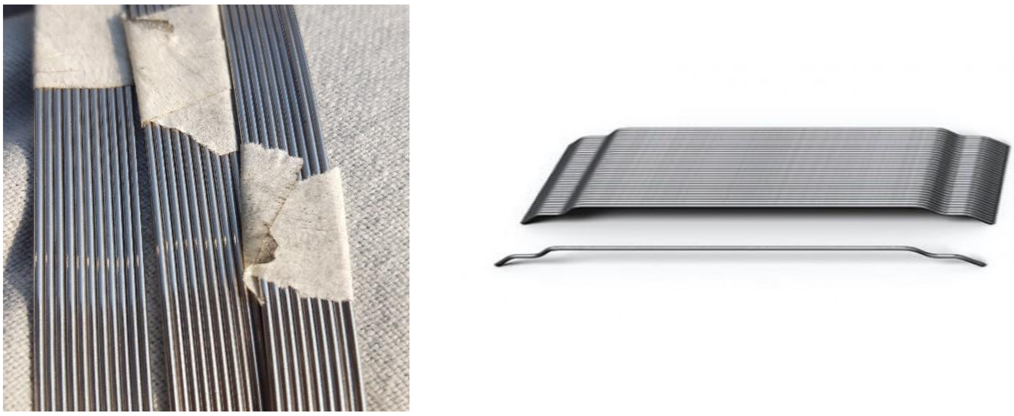
### 4.3.1. Concrete Plain Mix

The slump values for the prepared plain concrete were 150 mm and 170 mm, respectively. The intended breaking strengths at 28 days were determined by a cylinder compression test to be 36 MPa. A graded aggregates with a size of 17 mm was required per the specification. The material was examined using the CTM apparatus, as seen in figure 4.8.4


Figure 4.6. Show the Assembly of CTM

**4.3.2. Steel Fiber Reinforced Mix**

Figure 4-9 illustrates how the 5-Dramix hooked end kind of steel fiber was utilized in this experiment. This figure also demonstrates how the steel fiber was produced. The fibre had basic dimensions of 65 x 0.9 mm and 45o hooked ends, which are generally expected to gently deform during pull-out from concrete, so assuring a controlled ductile failure. The controlled ductile failure was ensured by the fibre having basic dimensions of 65 x 0.9 mm. The fundamental parameters of the fiber insured that the controlled ductile failure would take place. However, it should be stressed that the incorporation of a substantial number of relatively long and inflexible steel fibers into concrete may cause difficulties in the material's workability. This is something that should be kept in mind at all times.



**(a)** **(b)**

Figure. 4.7. Production of steel fibers (a) ships of fiber, (b) 5-D Dramix.

The fibre content of 79 kg/m3 and 158 kg/m3 that was used equated to 1% and 2%, respectively, of the volume of the concrete matrix. The aggregates, cement, and water for the concrete track were mixed together in the first step of the process. After that, the fibres and the concrete were combined in a mixer of the pan type (see Figure 4-10). For the purpose of determining the compression strength at 28 days and on the test day, respectively, three cylinders measuring 100 mm by 200 mm and containing the same volume of SFRC were cast in conventional steel mould and used for Groups I and II. The results of these tests, on an average basis, are presented in Table 4-2. Unit SF-2 required 0.064 m3, Unit SF-3 required 0.064 m3, Unit SF-4 required 0.053 m3, and Unit SF-5 required 0.083 m3 of SFRC, respectively. In Figure 4-11, as seen in the photographs, a few pieces of piece of plywood were used to distinguish between SFRC and regular concrete. The details of the steel fiber utilized, Dramix steel fiber, are provided below.

Table 4.1. Detail of used steel fibre

| **Name** | **Aspect ratio** | **Tensile strength (MPa)** | **Geometry** | **From manufacturer's website** |
|---|---|---|---|---|
| Dramix (65/35) | 65 | 1150 | Hooked end |  |

Table 4.2. Concrete cylinder results

| | | | | |
|---|---|---|---|---|
| **Category I** | | | | |
| **cylinder Type** | **Testing day** | **NO.** | **value (MPa)** | **Average (MPa)** |
| Normal concrete | 28 | 1 | 17.7224 | 17.935 |
| | | 2 | 17.35 | |
| | | 3 | 18.3325 | |
| 1% SFRC | 28 | 1 | 21.635 | 19.9325 |
| | | 2 | 19.135 | |
| | | 3 | 18.9935 | |
| 1.5% SFRC | 28 | 1 | 28.9325 | 28.3425 |
| | | 2 | 29.3975 | |
| | | 3 | 28.3875 | |
| **Unit RC-1** | | | | |
| Normal concrete | 35 | 1 | 20.155 | 19.33 |
| | | 2 | 18.35 | |
| | | 3 | 19.355 | |
| **Unit SF-2** | | | | |
| TEST NORMAL | 50 | 1 | 18.155 | 18.655 |
| | | 2 | 18.7455 | |
| | | 3 | 18.9575 | |
| 1% SFRC | 50 | 1 | 21.655 | 23.255 |
| | | 2 | 24.8525 | |
| | | 3 | 23.0575 | |
| **Unit SF-3** | | | | |
| Normal concrete | 59 | 1 | 16.85 | 18.1575 |
| | | 2 | 18.9575 | |
| | | 3 | 18.655 | |
| 2% SFRC | 59 | 1 | 24.755 | 28.55 |
| | | 2 | 29.7575 | |
| | | 3 | 29.7575 | |
| **Category II** | | | | |
| cylinder Type | Testing day | NO. | values (MPa) | Average (MPa) |
| Normal concrete | 28 | 1 | 23.3327 | 24.905 |
| | | 2 | 25.8827 | |
| | | 3 | 25.5 | |
| 1% SFRC | 28 | 1 | 25.81877 | 25.18125 |
| | | 2 | 24.227 | |

| | | | | |
|---|---|---|---|---|
| | | 3 | 25.7 | |
| **Unit SF-4** | | | | |
| TEST NORMAL | 41 | 1 | 22.97 | 25.245 |
| | | 2 | 24.6077 | |
| | | 3 | 28.1777 | |
| 1% SFRC | 41 | 1 | 28.6877 | 27.4945 |
| | | 2 | 26.6477 | |
| | | 3 | 27.1577 | |
| **Unit SF-5** | | | | |
| Normal concrete | 48 | 1 | 24.97 | 24.94 |
| | | 2 | 23.3725 | |
| | | 3 | 26.6775 | |
| 1% SFRC | 48 | 1 | 28.9725 | 28.04 |
| | | 2 | 27.2875 | |
| | | 3 | 27.9275 | |
| **Unit RC-6** | | | | |
| Normal concrete | 67 | 1 | 29.57 | 25.84 |
| | | 2 | 22.3725 | |
| | | 3 | 25.7575 | |

### 4.3.3. Measurement of Reinforcement Strains

A total of twenty-four electrical strain gauges with a TML 120-ohm resistance were used in order to measure the strain fluctuations along the longitudinal beam bars of the units that were included in Group I. This was done in order to ensure that accurate results were obtained. (Type FLA-3-11-3L). Figure 4-21 illustrates the layout of the strain gauges in its diagram (a).

The strain gauges that have been placed in the Group II units are depicted in a schematic form in figure 4-21. (b). The strain gauges that were used to measure the strains of the longitudinal beam bars and the strains of the transverse reinforcement in joints were the same ones that were used to measure the strains of the units that were in Group II. This was done so that an estimate could be obtained of the shear stress that hoops are subjected to when they are located in a joint.

Electrical strain gauges were used in the creation of each Group II unit throughout the entirety of the manufacturing process. In total, twenty-four of these gauges were used (see Table 4-4). The strain gauges were mounted on two opposing faces at the same spot, and the mean results were then utilized in order to quantify the actual steel strains for the purpose of further study.

All of the electrical transmission wires of the strain gauges that were cast into the units that were being tested were protected by being encased inside of plastic tubes before the units were put to the test. (see Figure 4-22).

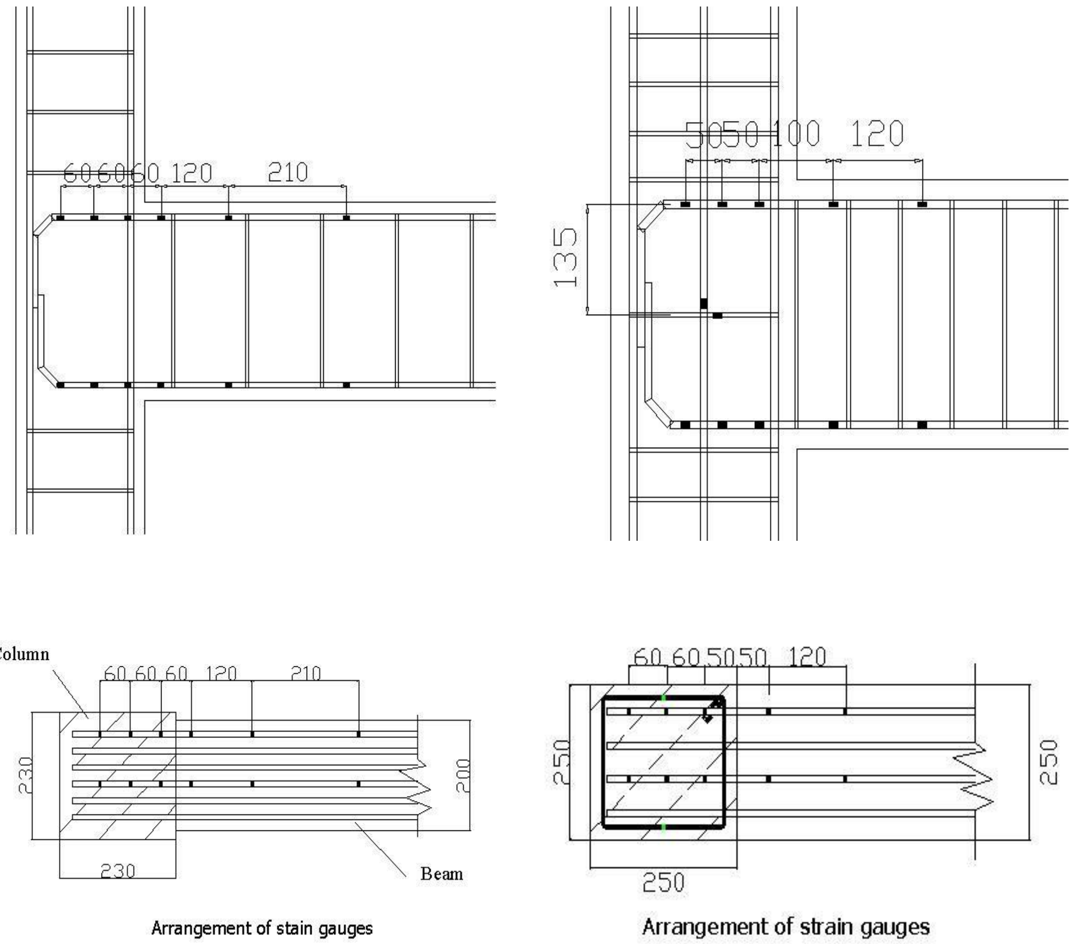



Figure 4.8. Strain gauge placement for tested units

Table 4.3. Distribution of electrical resistance strain gauges

| Test specimens | Components | No. of strain gauges |
|---|---|---|
| Beam-column joint units RC-1, SF-4 and SF-2 | Beam longitudinal bars(R-10) | 22 |
| Beam-column joint units SF-4, F-2, and RC-5 | Beam longitudinal bars(R-12) | 21 |
| | Column intermediate bars (R12) | 3 |
| | Joint hoops (R-6) | 3 |

## 4.4. Fabrication of Test Group, Test Apparatus, And Process

### 4.4.1. Seismic Loading Protocol

Same loading history was applied to all tests on the external pre-cast beam-column joint. Laterally from the top of the column, a steadily rising reversed cyclic displacement was applied; each step's displacement increment was 0.5 mm. Figure 4-9's preset load history illustrates the amount of movement and drift that occurs during each cycle. Following the application of two cycles for each drift at the top of the column, one low-level cycle (corresponding to 0.1% drift) was applied to each drift. The top of the column's displacements were measured and recorded on a computer throughout each loading cycle run. This continued until the desired level of displacement was achieved.

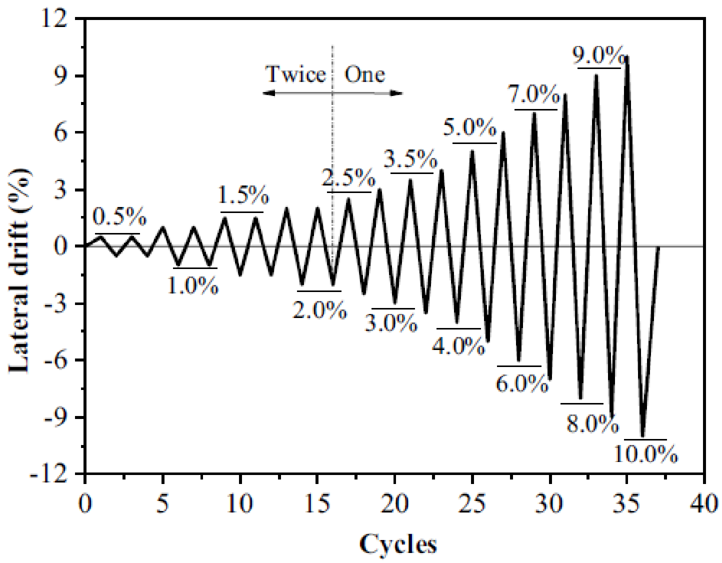


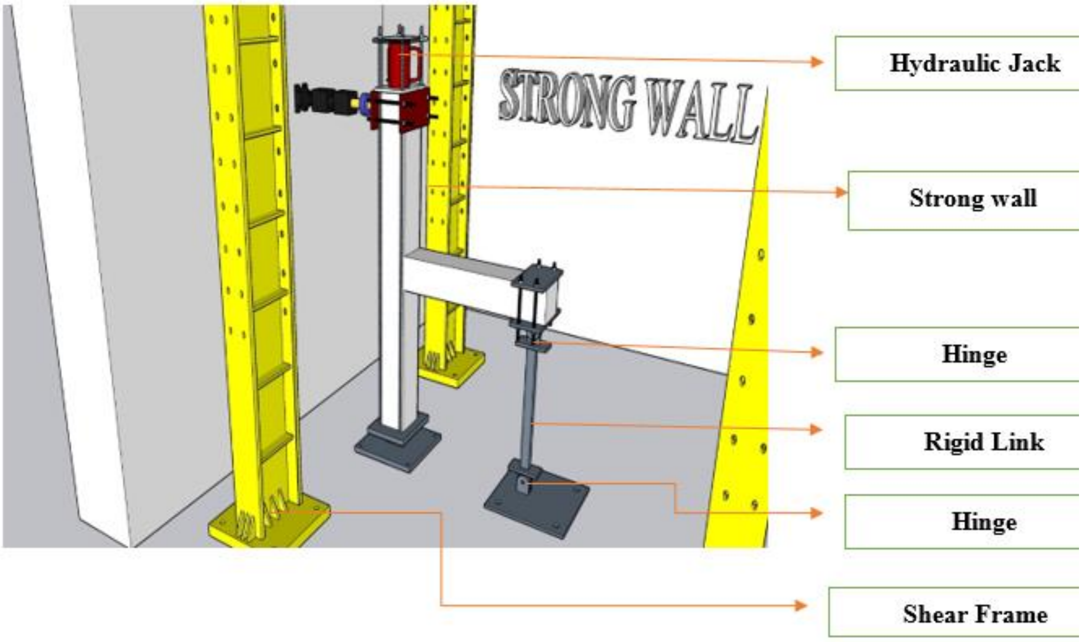

Figure 4.9. Lateral drift ratio against number of cycles for testing protocol.

### 4.4.2. Measurement to Determine the Hysteresis Loops

The force-displacement hysteresis reaction is a crucial statistic that needs to be produced in order to assess the building seismic performance. The force-displacement hysteresis response, which takes into account the area covered by the hysteresis loops, reveals the structure's capability to dissipate energy. For the purpose of designing the hysteresis loops in this study, the lateral stress on the column as well as its displacement were measured.

### 4.4.3. Force and Displacement Sensors and Measurements

In the Structures Laboratory at the National University of Science and Technology, tests were carried out (NUST). In order to simulate the effects of seismic loading, the samples were subjected to testing under a constant axial force of 17 tons, and the lateral load was delivered using a hydraulic actuator with a capacity of 50 tons.




**(a)**

**(b)**

Figure 4.10. Experimental test setup for (a) all the designed components for the assembly of the specimens are presented (b) On-site setup for testing protocol

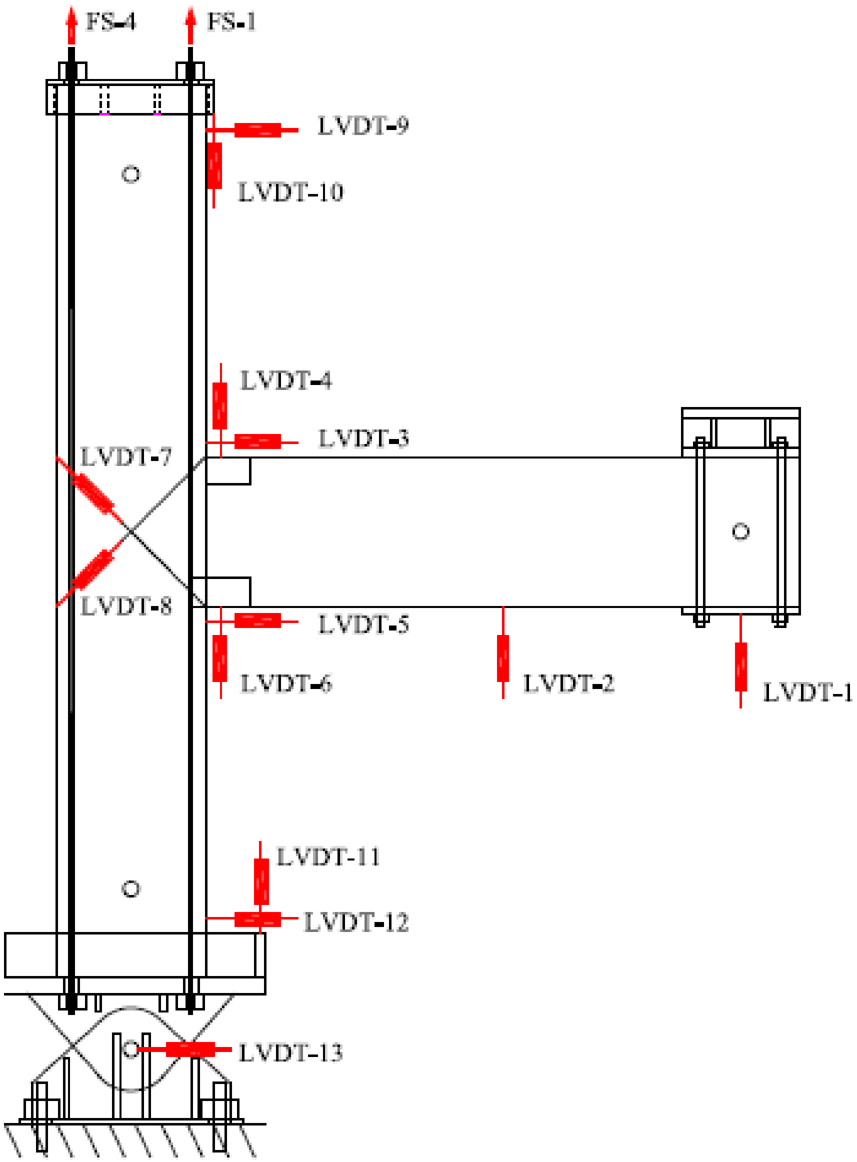



Figure 4.11. Setup of LVDTS

**4.4.5. Energy Dissipation**

The widths of the cracks were measured during the first five loading cycles, which correspond to 0.1% to 1.5% drift, using a "Eveready Microscope" hand crack detector in order to further compare the cracking patterns of a traditional concrete joint and an SFRC joint. This was done in order to determine whether or not there was a significant difference between the two types of joints (a high magnification hand microscope). On the surface that had been painted white were indicated all of the cracks that were there.

### 4.5. Testing Procedure

A more comprehensive set of data was acquired utilizing an electrical resistance strain gauge, a line potentiometer, and a beam end load cell before the testing of each individual item. Each of the test units that were part of Group I was subjected to an axial force of 17 tons. After the axial load had been applied to the column, a displacement-controlled cyclic loading was applied to the top of the column with the use of a hydraulic jack. In order to plot the force-displacement curve in the associated data gathering computer, a series of moderate force increments were applied during each cycle until the desired displacement was obtained. This was done until the desired force-displacement curve could be plotted. The following is a rundown of the procedures involved in each loading cycle:

**(G)** Verify each of the load cell and instrumentation connections, and II. Check sure they are linked to the computer that collects the data.

**(H)** Ensure that the appropriate channels are selected for each piece of instrumentation and load cell. Each instrumentation or load cell should correspond to a single channel.

**(I)** Before beginning testing, make sure that all channels are set to zero.

**(J)** Perform on the computer the displacement loading cycle that has been set.

**(K)** During each iteration of the cycle, check for and make a note of any cracking that appears on the white painted surface.

**(L)** Take photographs.

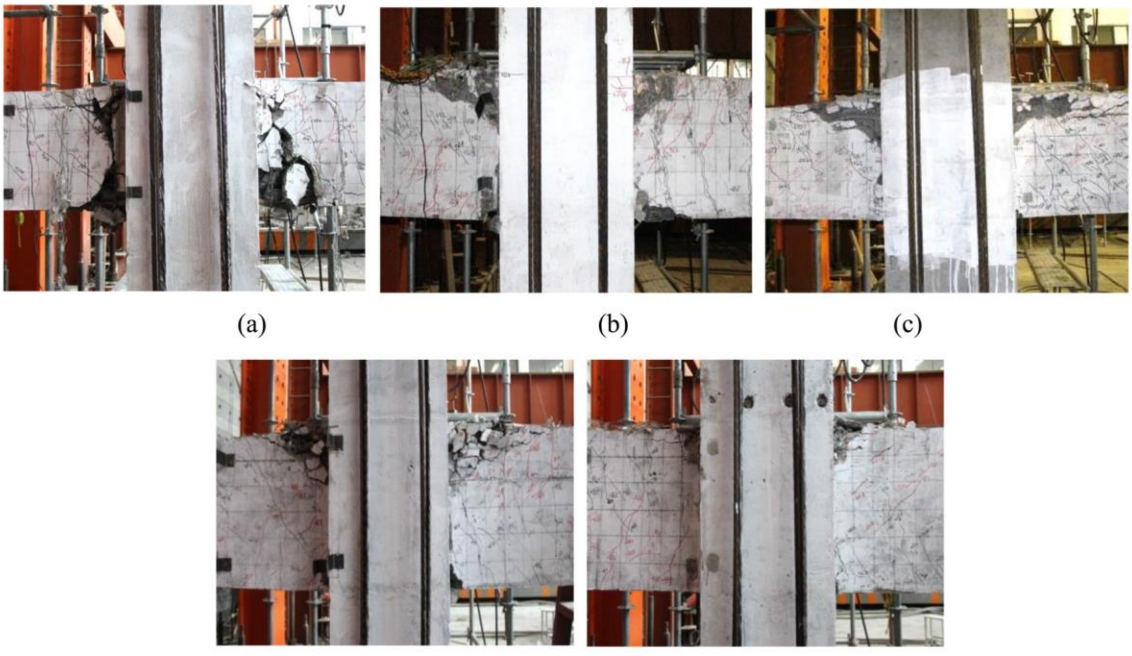
# CHAPTER 5

# TEST RESULTS AND THEORY VALIDATION

## 5.1. Test results of beam unit between columns

As can be seen in Figure 5.1, the flexural mode at a drift angle of 3.5% caused all five of the samples to fail and disintegrate. The strong column-weak beam paradigm was validated by the fact that only modest damage was sustained by the columns and junction panel locations At the beam portion of Specimen CP, a significant crack could be seen approximately 15 centimeters away from the column face. This crack was found within the specimen. In addition, there was significant concrete crushing found in the beam bottom region, as well as longitudinal bar buckling. The majority of the inelastic deformation and damage that occurred in the precast specimens was centered in the interface that was produced between the column and the beam. This interface was formed when the column was placed on top of the beam. When compared to Specimen CP, the lower section of the beam showed significantly less severe crushing; nevertheless, the higher section of the beam had significantly more severe crushing all around.

At drift angles larger than 1.5% in Specimen SP-4, a major vertical fracture that continued downhill developed at the junction of the column and the beam top cast-in-place concrete. This fracture was important since it continued downhill. The fact that this crack was widening and contracting while the beam-column junction was being examined shows that the beam top bar moved relative to the column during the examination. It's possible that this slippage happened because of the structure's inherent weakness, rather than due to the strength of the concrete that was formed in the location of the beam-column joint. After the tests were completed, the U-shaped shell was chiselled in order to examine the damage on the inside. The crushing and cracking of the inner concrete were significantly more severe than those of the outer concrete, particularly in the higher portion of the concrete, as shown in figure 5.2. This result, together with the fact that the surfaces of the inner concrete were clean, showed that a breakdown in the bond between the PC U-shaped shell and the inner concrete led to some relative sliding. Plastic hinges attached to the beam stretched all the way into the center of the cast-in-place concrete in the same direction as the beam. The external cracking and crushing that took place as a result of applying the air bubble film method to roughen the interfaces of Specimen SP-4 produced results that were comparable to those seen in Specimens SP-1 to SP-3. This was the case because the method was used to roughen the interfaces of Specimen SP-4. In contrast, the concrete on the inside showed signs of disintegration, but they were not quite as bad as those on the exterior. This finding demonstrated that the performance of the new solution in terms of force transfer was superior to that of the one that had been used previously. As a result, the connection was able to keep its structural integrity because of this discovery.



(a) (b) (c)

(d) (e)

Figure 5.1. Failure modes of specimens: (a) CP (b) SP-1 (c) SP-2 (d) SP-3 and (e) SP-4.

(a) (b)

Figure 5.2. Failure modes of specimens inside the U-shaped shells: (a) SP-1 and (b) SP-4.

### 5.1.1. Hysteretic Response

he hysteresis load-displacement correlations obtained from the various test specimens are presented graphically in Figure 5.3. In general, Specimens CP, SP-1, and SP-2 all possessed hysteretic loops that were equivalent to one another. In the beginning of the loading process, the hysteretic loops almost always presented themselves as straight lines, depending on the circumstances. The size of the hysteretic loops continued to grow, and they started to take on the appearance of fusiform shapes. The increase in drift angle that was the greatest was 1.0%. The hysteretic curves revealed a prominent yielding plateau at an angle of drift of 1.5%. Following that, the curves started to slowly pinch into one another, exhibiting a bent hysteretic loop. When the drift angle reached 2.75%, the hysteretic curves of the first cycle continued to show a bowing pattern. After that, the hysteretic curves began to acquire the form of an inverted S in the second cycle of the experiment. The pinching of the curve became more evident as a result of a drift angle of 3.5%. After two cycles of loading, the load was less than 80% of what it had been at its maximum. The only variation between the hysteretic loops of Specimen SP-3 and those of Specimen SP-1 was that the loops of Specimen SP-3 were a little bit thinner than those of Specimen SP-1. The hysteretic loops of Specimen SP-3 were equivalent to those of Specimen SP-1. At drift angles larger than 1.5% in Specimen SP-4, the beam top bar slippage caused the hysteretic loops to seem to be significantly more constrained than normal. Additionally, each organism went through three rounds until failing at the last drift stage. The envelope curves of the PC and CP specimens are presented in Figure 7(h) (h). The general morphologies of the curves were identical, and each one displayed a distinct yielding plateau.

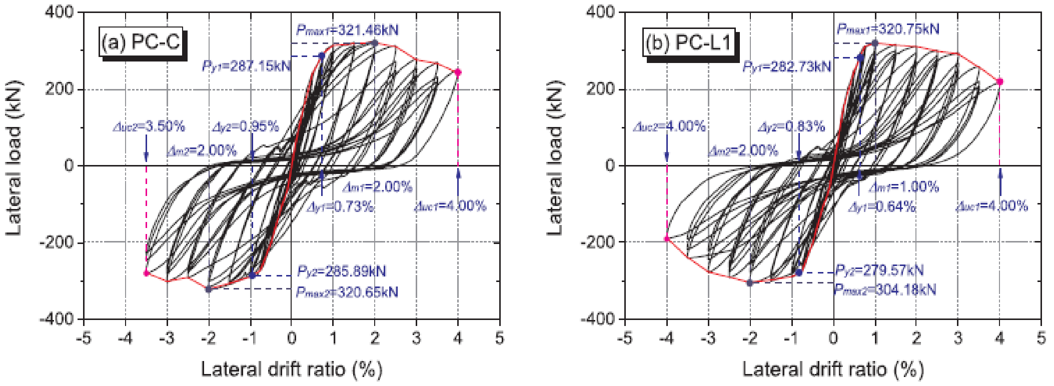



Figure 5.3. Lateral load-drift ratio relationships of the test specimens.

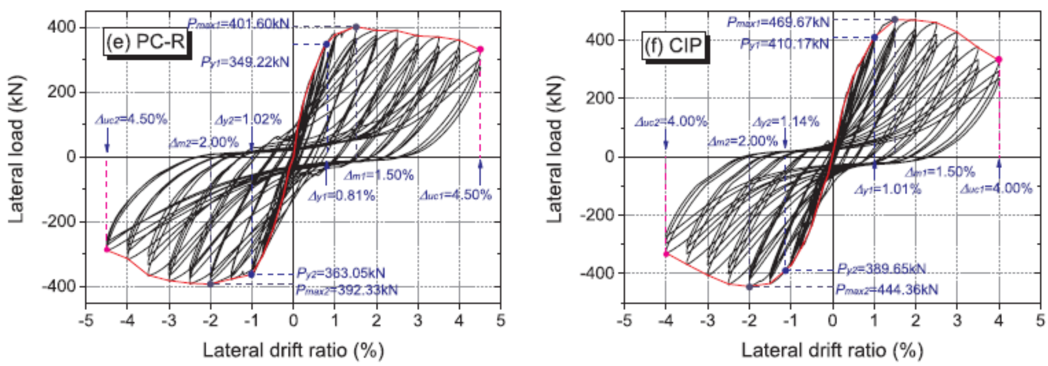
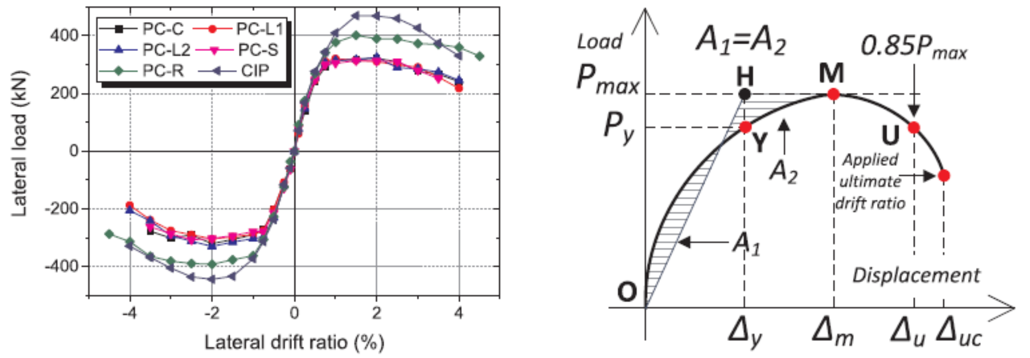


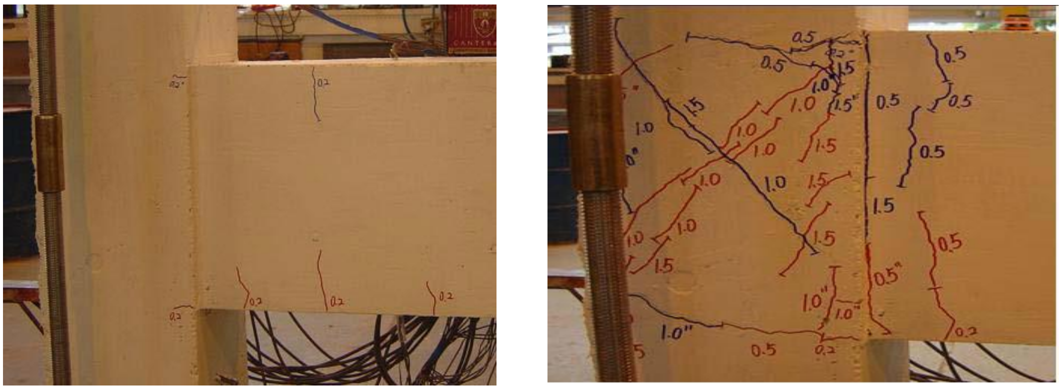

Figure 5.4. Lateral load-drift ratio relationships of the test specimens.



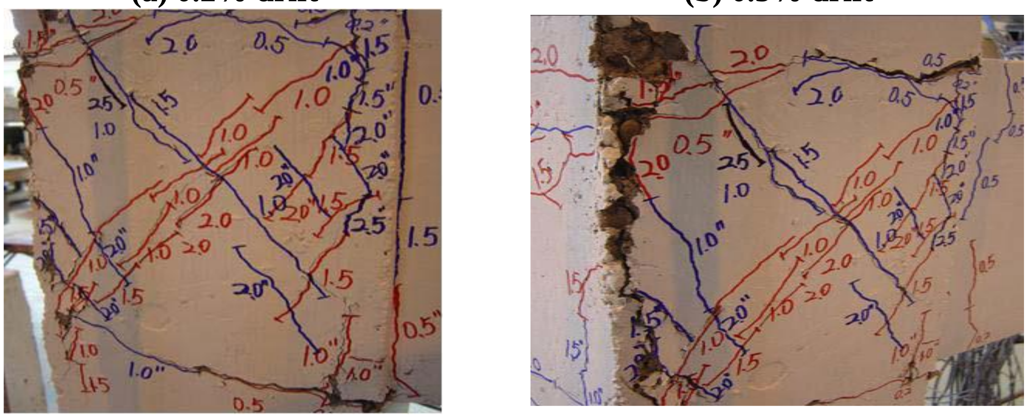

(g) Envelops curves

(h) Equivalent elasto-plastic energy principle

Figure 5.5. Lateral load-drift ratio relationships of the test specimens.



**(a) 0.2% drift** **(b) 0.5% drift**



**(c) 3% drift** **(d) 3.5% drift**

Figure 5.6. Observed damage of Unit SF-2

Table 5.1. Sequence of events of Unit PRC-1

| | Events | Force (kN) | $v_j$ (MPa) | $P_t$ (MPa) | $K=P_t/\sqrt{f'_c}$ | $\gamma$ (rad) | Drift |
|---|---|---|---|---|---|---|---|
| Unit PRC-1 | cracking | 20.657 | 2.325 | 1.3961 | 0.42466 | 0.0007694 | 0.64 |
| | Extensive cracking | 24.34 | 2.34098 | 1.93474 | 0.46915 | 0.001343 | 0.72 |
| | End test | 12.29 | 1.369106 | 0.903496 | 0.214259 | 0.0042245 | 2.91 |

### 5.1.2. Energy Dissipation

Figures 11(a) and (b) show, respectively, the amount of energy that was lost within each load circle as well as the overall amount of energy that was lost by the specimens during the experiment. In order to determine the amount of energy that is wasted throughout each cycle, one must first compute the region that is covered by a load-deflection loop at a particular displacement. The term "cumulative dissipated energy" refers to the total amount of energy that has been wasted across the entirety of all of the load cycles. The cast-in-place Specimen CP displayed the highest level of energy loss, which was to be expected given the nature of the experiment. The total quantity of energy that was lost by the specimens SP-1, SP-2, SP-3, and SP-4, in that sequence, was 84%, 90%, 79%, and 66% of what was lost by the Controll specimen. The Controll specimen served as the baseline for comparison. The overall quantity of energy lost by Specimen SP-2 was 6% higher than that lost by Specimen SP-1, and the energy lost by Specimen SP-2 each circle obviously surpassed Specimen CP levels in the final few circles of the experiment. Specimen SP-1 lost 6% less energy than Specimen SP-2 did overall. Because there was a difference of 5% in the total energy dissipation between Specimen SP-3 and Specimen SP-1, it was deduced that the quantity of high-strength steel bars at the top of the beam had been reduced, which resulted in an even lower level of energy dissipation. This was determined because there was a difference in the total energy dissipation between Specimen SP-3 and Specimen SP-1. The higher longitudinal bars of the beam slipping caused the SP-4 specimen to have the lowest level of energy dissipation when compared to the SP-1 specimen. This result was due to the SP-4 specimen having the lowest amount of energy dissipation. This disparity was about 20% in total.

Before loading cycle 21, the damping ratios were around 5%, and Figure 11 shows that the drift angle was 1% during that time. (c). All of the damping ratios went through a large increase during loading cycle 22, when the specimens first started to yield (with a drift angle of 1.5%). This was when the loading cycle reached its maximum. After yielding, the prefabricated samples had damping ratios that were SP-2, SP-1, SP-3, and SP-4 in that order. This sequence nearly perfectly matched the manner in which the energy was dissipated during each load cycle. After yielding had occurred, the damping ratio of CP achieved its highest value possible. When subjected to the greatest number of loading cycles, the damping ratio of Specimen SP-2 was identical to that of Specimen CP. This was the sole distinction that could be made out between the curves of Specimen SP-2 and Specimen SP-1. Specimen SP-2 had a steeper slope as compared to the SP1.

Compared with the top beam bars, the strains measured in the joint approached the yield strain in the loading cycle of 2.0% drift at both positive and negative loading direction. After that stage, the strain in this area became larger and larger indicating that the shear stress in the joint was mainly resisted by the beam bars adjacent to the column's inner face.

After a 1% drift cycle run, the diagonal cracks were accumulating and concentrating in the joint region with the increasing intensity of the cycle load. During the last cycle loading, the peculiar degrading mechanism (named "concrete wedge") occurred in the joint panel. After spalling of the concrete cover from, the joint completely lost its capacity.

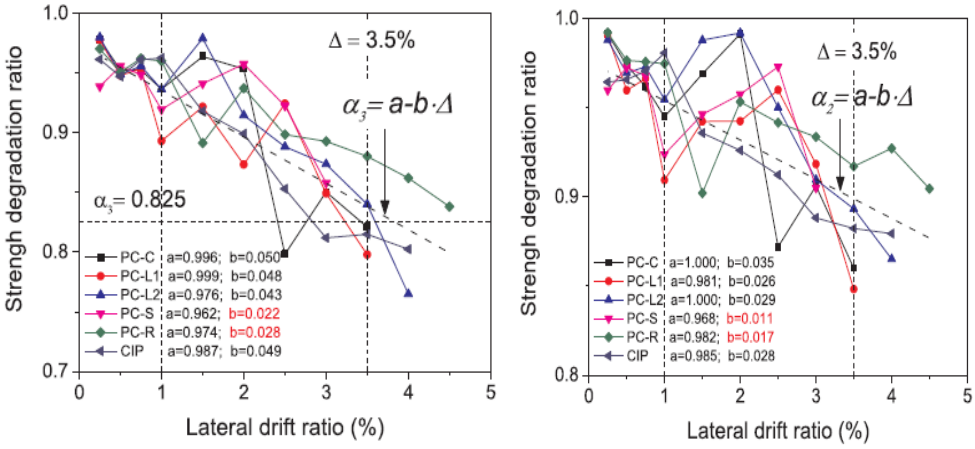



Figure 5.7. Strength degradation ratios of the test specimens.

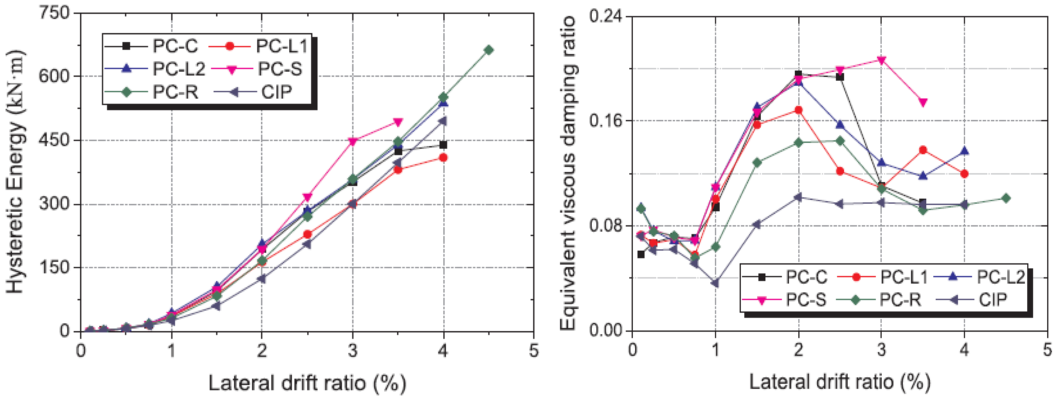


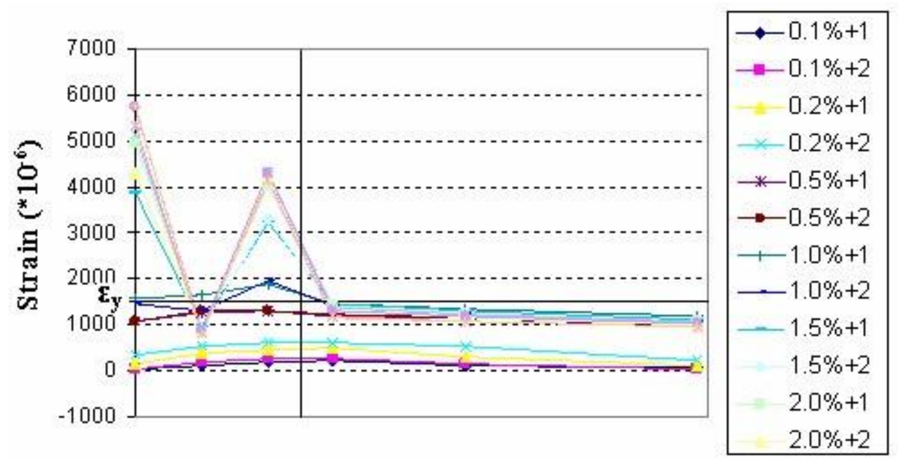

Figure 5.8. Shows the comulative eenergy and viscous damping ratio

Table 5.2. Displacement ductility.

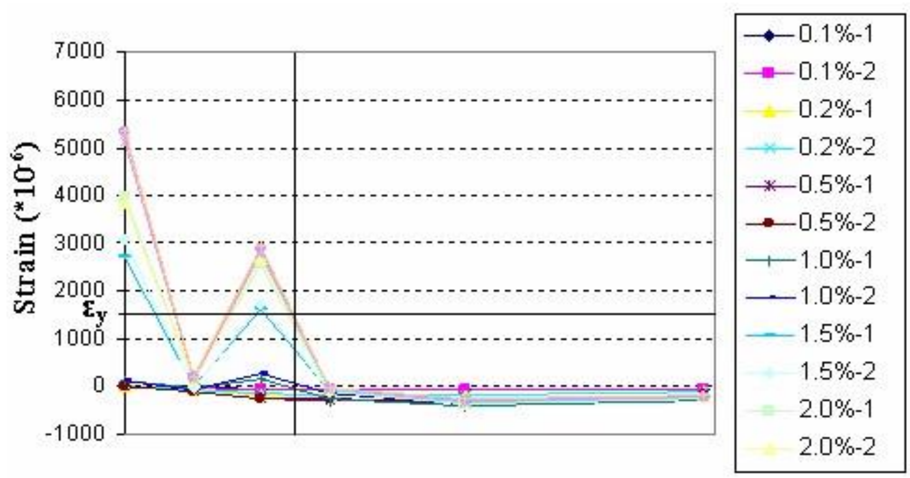
| Specimen | Direction | Yielding displacement $D_y$/mm ($d_y$, %) | Maximum displacement $D_u$/mm ($d_u$, %) | Ductility Du=Dy | Average |
|---|---|---|---|---|---|
| CP | Positive | 37.26 (1.19) | 109.2 (3.5) | 2.93 | 2.87 |
| | Negative | 38.85 (1.25) | 109.2 (3.5) | 2.81 | |
| SP-1 | Positive | 36.80 (1.18) | 109.2 (3.5) | 2.97 | 2.80 |
| | Negative | 41.34 (1.33) | 109.2 (3.5) | 2.64 | |
| SP-2 | Positive | 35.51 (1.14) | 109.2 (3.5) | 3.08 | 3.04 |
| | Negative | 36.36 (1.17) | 109.2 (3.5) | 3.00 | |
| SP-3 | Positive | 35.76 (1.15) | 109.2 (3.5) | 3.05 | 2.86 |
| | Negative | 41.07 (1.32) | 109.2 (3.5) | 2.66 | |
| SP-4 | Positive | 35.36 (1.13) | 109.2 (3.5) | 3.09 | 2.79 |
| | Negative | 43.67 (1.40) | 109.2 (3.5) | 2.50 | |

### 5.1.3. Longitudinal Beam Bar Strains

It was predicted that the junction would function as a shear hinge in Unit RC-1. Figures 5-4 and 5-5 provide a visual representation of the estimated strain profiles that run down the length of the longitudinal beam bars. The strain gauges were used to get the information for these strain profiles. Before the initial drift loading cycle of 1.0%, the tensile strain along the beam bars progressively grew until it reached its maximum value. This occurred before the cycle began. Following the loading cycle that included 1.0% drift, the top beam bars already had achieved their yield strain in the joint region close to the inner face of the column before the next loading cycle began. During the early discovery of joint diagonal stress cracks, which indicated yield penetration into the joint core, this was found out. Because of the joint shear hinge, which is also known as the "concrete wage" mechanism, the second loading cycle resulted in a very significant increase in the amount of tensile strain that was present along the region of the beam bar hooks. It was clear from this that the majority of the bond forces were supplied by the beam bars that went completely around the inner column face and were hooked at a 90-degree angle.




(a) beam top bar (positive cycle)




(b) Beam top bar (negative cycle)

Figure 5.9. Strain profiles of Monolithically sample

## 5.2. Test Results of Beam Unit Between Columns

### 5.2.1. Failure Mode

Figures 13–18 show the many patterns of cracking that can occur in concrete. The specimens went through four stages while being subjected to moderate cyclic loading. These stages are as follows: initial crack emergence and development, specimen yield state, ultimate state of the sample, and final failure. An intricate stress state was present at the connection point between the beam and the column, and it was caused by the interaction of axial pressure, shear force, and bending moment. Cracks caused by flexure were the first to appear on the beams' surfaces. Flexural fissures developed as a result of the stress being distributed away from the column, and eventually, a plastic hinge came into being.

The first diagonal cracks appeared near the centre of the connecting core and spread outward toward the core edgeIn the connection core, the reinforcement's longitudinal bond weakened as the load grew, lowering the connection's bearing capacity. At the same time, the shear forces at the connection core stopped growing, and the connection core could see longitudinal bar slip. The connection's stiffness and strength decreased as a result of a widening gap between the end of the beam and the concrete core, the development of flexure-shear fractures on the end of the beam, and the subsequent increase in the space between the two. In addition, the connection's strength decreased as a result of the development of flexure-shear fractures on the end of the beam. In the end, the link was severed as a result of the crushing of the concrete. Shearing was observed in the connecting core of specimen P1, whereas beam bending failure (the crushing of concrete in the beam's closeness to the core region) was observed in specimens P2 through P5 and R1 (the concrete of core region crush). Flexural cracks are evenly distributed over the beam of the cast-in-place specimen R1 (k = 1.07), indicating that these cracks were caused by flexure. When a connection fails, shear cracks can be seen developing in the joint. The cracks in R1 are closer together than the cracks between R2 and R1, which contributes to the overall low number of cracks. In addition, the density of the shear cracks is larger here in comparison to the specimen R1, which was cast in place. The fact that the part of the precast connection that was cast in place contains stronger concrete is the primary explanation for this phenomenon. The rigidity and strength of this component are substantially improved by the addition of grout sleeves.

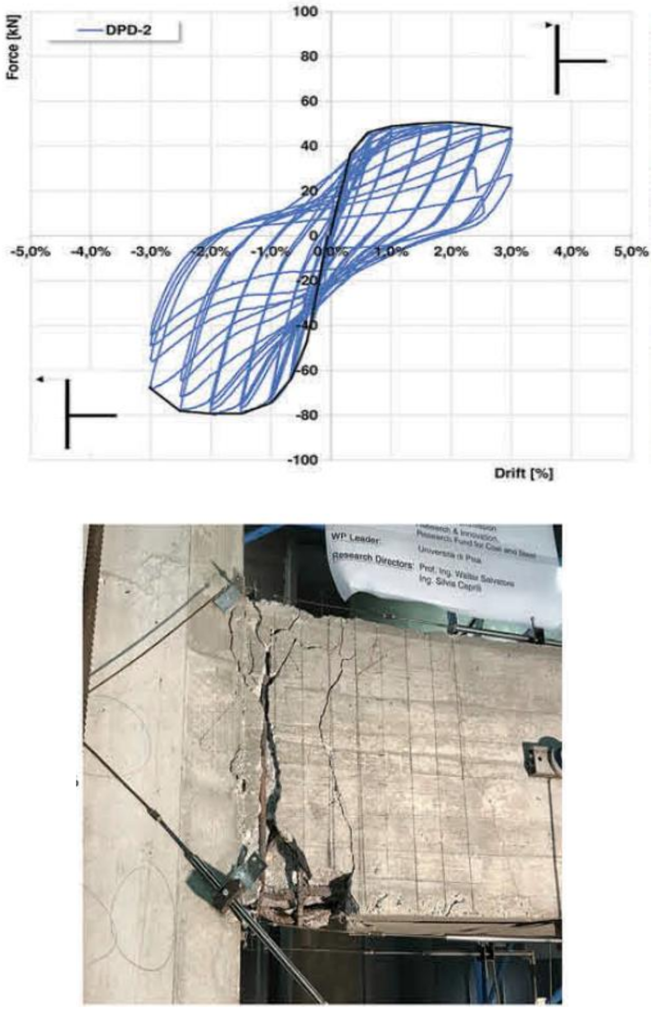



Figure 5.10. Shows crack analysis and  Force verses drift Ratio

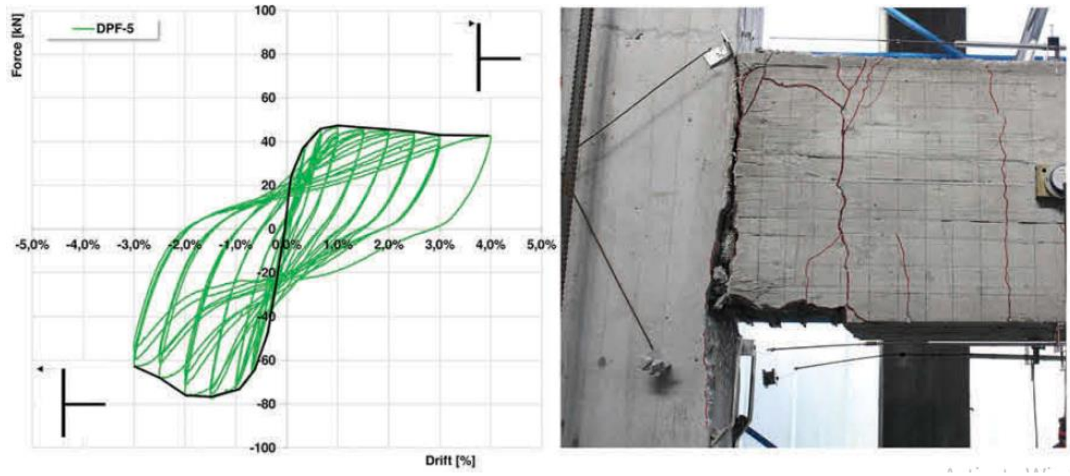
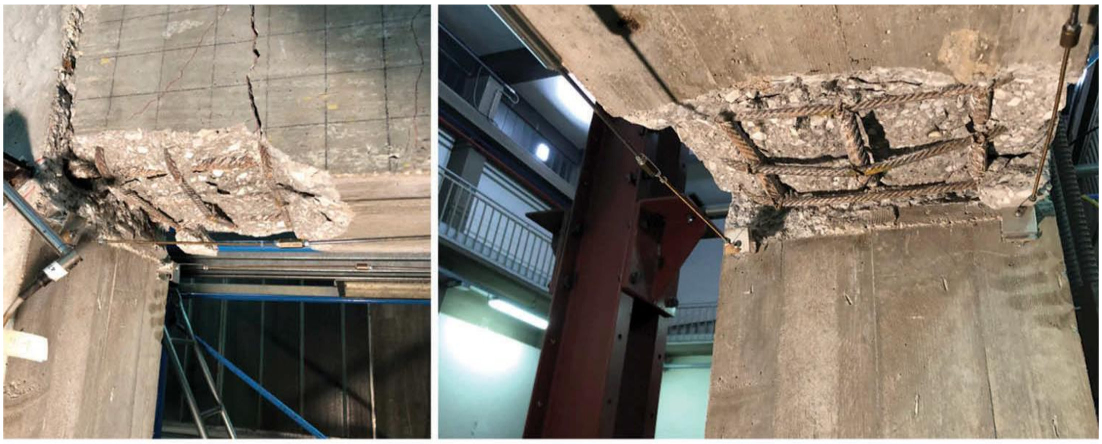



Figure 5.11. Shows the Forces verses Drift Ratio of exterior beam column joiunt



(a)

(b)

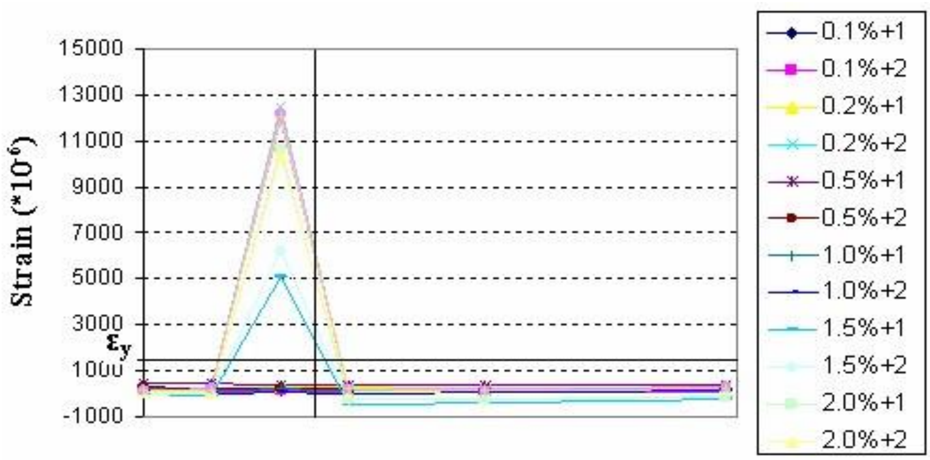
Figure 5.12. Shows the Forces verses Drift Ratio of exterior beam column joiunt

### 5.2.2. Longitudinal Beam Bar Strains

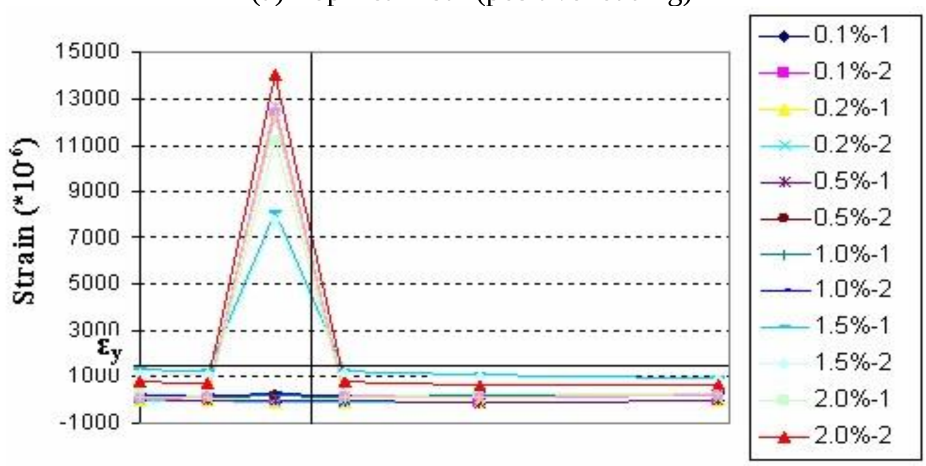
The strains in the longitudinal beam bars that are depicted in Figures 5-10 and 5-11. Because of the bridging action of the steel fiber, the beam bars remained in the elastic strain stage despite being forced to an inelastic loading cycle with a 1.0% drift. (See Fig. 5-9). The pullout resistance (dowel action) and bridging action of the fiber considerably boosted the post-cracking tensile strength of the concrete when it fractured in the joint area. Additionally, the concrete was better able to withstand the joint shear stress as a result of these two factors. After being subjected to these factors, the Unit PSF-2 longitudinal beam bars arrived at the stage of yield stresses after having completed only 1.5% of the loading cycle. In addition, the beam bar stresses of Unit

SF-2 were significantly higher than those of the nonfibre specimen of Unit RC-1. This indicates that the SFRC joint is capable of stronger binding and anchoring capabilities than the nonfibre specimen.

Loading cycle with 2% the though the yield stage was postponed to a, the bottom longitudinal beam bars behaved in a manner that was comparable to the behavior of the top beam bars. According to the placements of the beam bars' yield lines, which were found to be rather near to the inner face of the column for both the bottom and top bars, the joint continued to be the most important part of Unit SF-2.




(a) Top Beam bar (positive loading)




(b) Top Beam bar (negative loading)

Figure 5.13. Beam strain profiles through a column

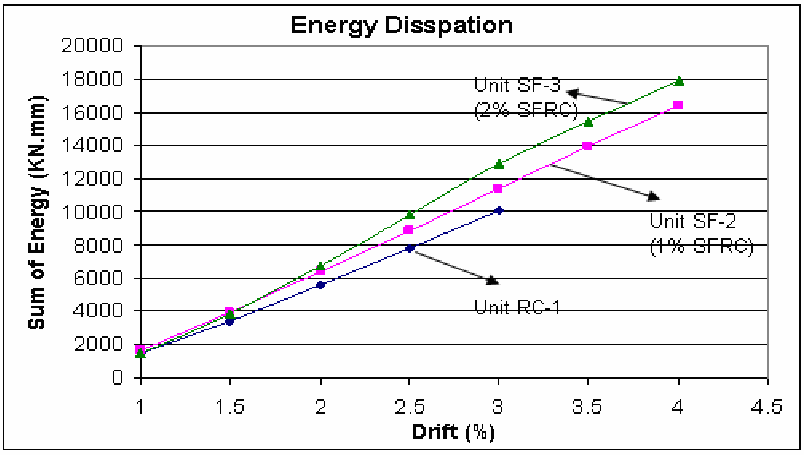
Table 5.3. Load-carrying capabilities of specimens

| Specimen | direction | yielding load (KN) | | | | Maximun Load (KN) | | |
|---|---|---|---|---|---|---|---|---|
| | | Experimental Py | Average absolute Py | Calculated Pn | Average Py/Pn | Experimental Pmax | Average absolute Pmax | Average Pmax/Py |
| Monolithically casted | positive | 258 | 253 | 225 | 1.13 | 282 | 275 | 1.09 |
| | negative | | | | | -269 | | |
| Pre-cast Controlled sample | positive | 216 | 217 | 204 | 1.06 | 236 | 238 | 1.1 |
| | negative | | | | | -240 | | |
| pre-cast with steel fibre | positive | 239 | 235 | 204 | 1.15 | 258 | 250 | 1.07 |
| | negative | | | | | -242 | | |
| Precast beam unit between column | positive | 207 | 204 | 204 | 1 | 232 | 227 | 1.12 |
| | negative | | | | | -222 | | |
| pre-cast beam through column | positive | 199 | 204 | 204 | 1 | 221 | 230 | 1.09 |
| | negative | | | | | -239 | | |

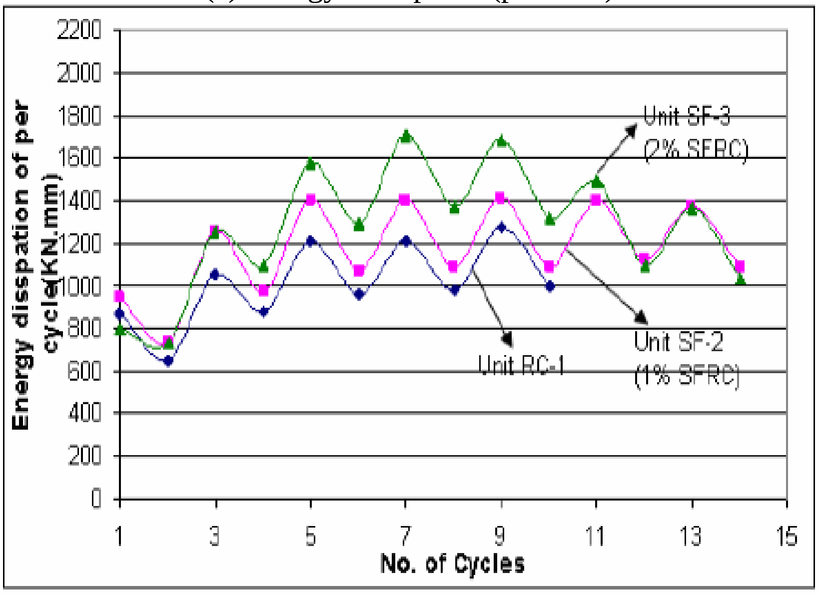
### 5.2.3. Sample Units' Equivalent Viscous Damping

Three sample in Group I's total energy dissipation capacity were compared. is shown in Figure 5-35 (a). When compared to the RC joint unit, the dissipation energy for SFRC units increased significantly more during the huge inelastic-deformation stage (Unit RC-1). The reason is that the toughness and dispersed energy can be raised appropriately as steel fibres are gradually drawn out of the matrix over time. As a result, there might be less cracking and more inelastic deformation at the SFRC joint. Additionally, it is obvious from Figure 5-35 (a) that the Unit SF-3 had the best energy dissipation, showing that the energy dissipated increased as the fibre volume percentage increased.

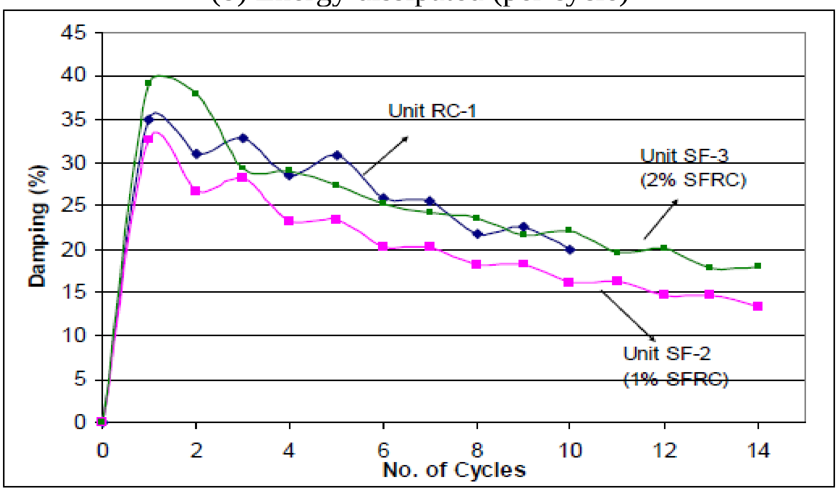
While taking into account the elastic deformation area, it is possible to determine the equivalent viscous damping of the tested components. In order to determine the energy dissipation capability, the damping of the  sample in units I Group was evaluated and compared. Cumulative damping for each cycle was added together at the results of the testing, as shown in Figure 5-35, to determine the damping coefficient (c).




(a) Energy Dissipated (per drift)




(b) Energy dissipated (per cycle)




(c) Viscous Equivalent damping ξ (per cycle)

Figure 5.14. Correlation of the Group I tested units' energy dissipation capacities

Table 5.4Unit RC-6's events in order

| **Unit** | **Sequence of events** | **Force(KN)** | **$v_j$ (MPa)** | **$P_t$(MPa)** | **$K=P_t/\sqrt{f'_c}$** | **γ (rad)** | **Drift** |
|---|---|---|---|---|---|---|---|
| **RC-6** | Beam hinging | 24.103 | 2.310 | 2.123 | 0.303 | 0.0001371 | 0.7 |
| | Extensive damage | 23.22 | 2.334 | 2.212 | 0.320 | 0.0031372 | 2.5 |
| | End test | 12.559 | 1.244 | 0.324 | 0.284 | 0.00513368 | 5.0 |

Table 5.5. Experimental results

| Units | Sequence of events | Force (KN) | $v_j$ (MPa) | $P_t$ (MPa) | $K=P_t/\sqrt{f'_c}$ | γ (rad) | Drift |
|---|---|---|---|---|---|---|---|
| **Unit RC-1** | First cracking | 20.26 | 2.4225 | 1.796 2 | 0.4246 | 0.0007694 | 0.65 |
| | Extensive cracking | 22.25 | 2.6402 | 1.984 2 | 0.4691 | 0.00136 | 0.71 |
| | End test | 13.229 | 1.569 22 | 0.906 2 | 0.214259 | 0.0042256 | 2.96 |
| **Unit SF-2** | First cracking | 23.026 | 2.725 | 2.078 2 | 0.491231 | 0.000162 | 0.71 |
| | Extensive cracking | 25.424 | 2.9726 | 2.306 2 | 0.545111 | 0.0004231 | 0.90 |
| | End test | 16.121 | 1.9225 | 1.316 2 | 0.311098 | 0.0099162 | 3.95 |
| **Unit SF-3** | First cracking | 25.121 | 2.9222 | 2.272 2 | 0.537192 | 0.0002343 | 0.96 |
| | End test | 19.724 | 2.2926 | 1.688 2 | 0.399006 | 0.0118285 | 3.90 |
| **Unit SF-4** | First cracking | 26.8229 | 3.1522 | 2.450 2 | 0.48766 | 0.0002 | 0.70 |
| | Extensive cracking | 26.6 2 | 3.2952 | 2.599 2 | 0.517328 | 0.0015 | 1.91 |
| | Beam hinging | 27.75 2 | 2.525 2 | 3.233 2 | 0.502704 | 0.0004 | 0.90 |
| | End test | 15.65 2 | 1.839 2 | 1.198 2 | 0.238454 | 0.0037 | 4.90 |
| **Unit SF-5** | First cracking | 25.3 2 | 2.938 2 | 2.228 2 | 0.443623 | 0.0007366 | 0.69 |
| | Extensive cracking | 24.28 2 | 2.826 2 | 2.102 2 | 0.418482 | 0.0031054 | 3.0 |
| | Beam hinging | 26.65 2 | 3.110 2 | 2.377 2 | 0.473253 | 0.0024974 | 1.0 |
| | End test | 13.35 2 | 1.565 2 | 0.957 2 | 0.190516 | 0.0122577 | 5.0 |
| **Unit RC-6** | Beam hinging | 23.10 2 | 2.710 2 | 2.022 2 | 0.402586 | 0.0001671 | 0.7 |
| | Extensive | 24.22 | 2.833 2 | 2.1112 | 0.420251 | 0.0031872 | 2.5 |
| | End test | 13.55 | 1.543 2 | 0.9224 | 0.183908 | 0.0051368 | 5.0 |

## CHAPTER 6

## CONCLUSIONS AND RECOMMENDATIONS

### 6.1. Conclusion

**(A)** Shear failure will occur in precast specimens when the ratio of the column strengths to the beam strengths is less than 1. During the design phase of the structure, it is essential to maintain proper control of the column-to-beam strength ratio..

**(B)** For precast specimens, the performance of the plastic hinge in the beam is most heavily influenced by two factors: the cast-in-place section and the grout sleeves. Because they can improve the stiffness and strength of the beam section near to the junction, the precast specimen has a larger yield and ultimate displacement than the cast-in-place specimen. This is due to the fact that precasting allows for greater flexibility. As a bonus the yield range of the beam reinforcement comes to the joint faster in the precast specimen, and cracks are less densely distributed throughout (and do not exist in) the grout sleeve.

**(C)** Nonlinear deformation concentration at the important connection interfaces in the proposed precast connections indicates the usefulness of the strong column-weak beam design notion. Precast specimens often had the same or worse performance as the cast-in-place specimen. As a result, this connection is suitable for usage in seismic regions provided that it is planned and built appropriately.

**(D)** The air bubble film technology used in the novel solution enabled a significant improvement in force transfer performance compared to the status quo. However, our team is currently conducting additional study to completely understand the bond characteristics.

**(E)** The seismic performance is highly sensitive to the bond conditions of the beam top continuous steel bars and the interfaces. Therefore, it is crucial to strictly control the interface roughening quality and concrete pouring in the joint zone during field construction..

### 6.2. Recommendation for future study

The following are areas where we need more study on the use of fibre reinforcement in earthquake design.:

**(A)** Stirrups and steel fibre transverse reinforcement are recommended for use in joint critical areas.

**(B)** The development of models of strength degradation through the use of principal tensile stress enables quick design of SFRC joints. To determine the precise shear failure threshold of SFRC joints, further research with designs that are identical to those used previously is still recommended.

**(C)** Additional experimental testing is required in order to develop a better version of the recommended shear strength degradation model for SFRC joints. In addition, the moment capacity of flexure elements of pre-cast beam-column junctions, such as the plastic hinge area, needs to be better analyzed by proper evaluation in order to anticipate the overall failure mode of an SFRC joint. This can be done by comparing the moment capacity of the plastic hinge area to the moment capacity of the plastic hinge area of a beam-column junction (addressed in Chapter 5). This enabled for the creation and adoption of a trustworthy hierarchy of strength diagram that comprised joints, beams, and columns within a beam-column subassembly for use in practical design applications.

**(D)** In addition, steel fibres can reduce the demand for transverse reinforcement in plastically hinged portions of pre-cast beam and column members. However, the amount of allowed decrease must be established by appropriate analytical methods and verified experimentally.

**(E)** It is generally recognised that the aspect ratio, volume, type, and dispersion of the fibres in the concrete mix all have a significant impact on the properties and behaviour of SFRC. By using numerical and experimental analysis, it should be independently explored how these parameters affect the joint behaviour.

# CHAPTER 7

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